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2	1	Design Authority	<i>JW Bailey</i>	JW Bailey	7/20/08	S2-48		
		Design Agent	<i>M C Davenport</i>	M C Davenport	7/19/08	S2-48		
2	1	Cog.Eng.	<i>CE Graves</i>	CE Graves	7/19/08	S2-48		
2	1	Cog. Mgr.	JW Bailey	JW Bailey	7/20/08	S2-48		
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		Safety						
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20. <i>JW Bailey</i> J.W. Bailey Design Authority/ Cognizant Manager						21. DOE APPROVAL (if required) Ctrl. No. ☐ Approved ☐ Approved w/comments ☐ Disapproved w/comments		

Project W-320, 241-C-106 Sluicing Civil/Structural Calculations, Vol. 8

John W. Bailey

Numatec Hanford Co., Richland, WA 99352

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Abstract: This supporting document has been prepared to make the FDNW calculations for Project W-320, readily retrievable.

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Project W-320, 241-C-106 Sluicing Civil/Structural Calculations, Vol. 8

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W320-24-025

Cover Block Lifting Bail Calculation

**KAISER ENGINEERS
HANFORD****CALCULATION IDENTIFICATION AND
INDEX**

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Date

7/6/95

This sheet shows the status and description of the attached Design Analysis sheets.

Discipline Structural

WO/Job No. ER4319/W-320

Calculation No. W320-24-025

Project No. & Name W-320, Tank 241-C-106 Sluicing

Calculation Item Cover Block Lifting Bail Calculation

These calculations apply to:

Dwg. No. H-2-818448 SH.2, 3, & 5

Rev. No. 0

Dwg. No. H-2-818450 and H-2-818454, SH. 2 & 3

Rev. No. 0

Other (Study, CDR) NA

Rev. No.

The status of these calculations is:

☐

Preliminary Calculations

☒

Final Calculations

☐

Check Calculations (On Calculation Dated)

☐

Void Calculation (Reason Voided)

Incorporated in Final Drawings?

☒ Yes☐ No

This calculation verified by independent "check" calculations?

☐ Yes☒ No

Original and Revised Calculation Approvals:

	Rev. 0 Signature/Date	Rev. 1 Signature/Date	Rev. 2 Signature/Date
Originator	James R. Booth <i>[Signature]</i> 7/6/95 Michael B. Lassota <i>[Signature]</i> 7/21/95	H.B. Lassota <i>[Signature]</i> 7/24/97	
Checked by	<i>[Signature]</i> 8/1/95	<i>[Signature]</i> 5/8/97	
Approved by	<i>[Signature]</i> 8/1/95	<i>[Signature]</i> 5/8/97	
Checked Against Approved Vendor Data		N/A	

INDEXDesign Analysis
Page No.

Description

1	OBJECTIVE, DATA, ASSUMPTIONS, REFERENCES, AND CONCLUSION
2-6	Concrete Cover Block Lifting Bail Calc
7-8	Pump Penetration Lifting Bail Calc
9	Cover Block Port Plug Evaluation
10	Math Model of Cover Block Lifting Bail
11	Math Model of Pump Penetration Plug
Attach. 1	IMAGES Analysis of Cover Block Lifting Bail
Attach. 2	IMAGES Analysis of Pump Penetration Plug Lifting Bail

REV 1: REVISED SH.6; REPLACED SH 7, 8 & 9 & Attach 2; ADDED SH 8a,b,c

DESIGN VERIFICATION SCREENING CRITERIA

ii of iii

Project/Document No. W-320/W320-24-025

When the design or design change affects hardware, formal design verification must be performed if one or more of the following questions must be answered affirmatively (YES).

YES

NO

<u> </u>	<u> ✓ </u>	1. Does the design or design change involved meet the established criteria to be considered Safety Class 1 or 2?
<u> </u>	<u> ✓ </u>	2. Does this design or design change cause or permit changes to Safety Class 1 or 2 instrument or alarm setpoints outside of previously approved operational limits?
<u> </u>	<u> ✓ </u>	3. Does this design or design change significantly affect the nuclear safety consequences of a malfunction or failure of the structure, system, or component?
<u> </u>	<u> ✓ </u>	4. Does this design or design change involve or change design that has previously undergone formal design verification?

Red Davidson
Assigned Lead Engineer

8/1/95
Date

Red Davidson
Responsible Discipline Manager

8/1/95
Date

Original Design Package Distribution:

Project Engineer

Chief Design Engineer

Engineering Document Control

Design Change Distribution:

Attach to Engineering Change Notice

CALCULATION CROSS INDEX (Typical)

Subject Calculation No. W320-24-025

Page 111 of 111

[illegible]

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HANFORD

DESIGN ANALYSIS

Calc. No. W320-24-025Revision 0Page No. 1 of 11Client: Westinghouse Hanford CompanyWO/Job No. ER4319/W-320Subject: Cover Block Lifting Bail CalculationsDate 7/6/95By JR BoothChecked 7/20/95By H.B. LASOTALocation: C-Farm & AY-Farm

Revised

By

OBJECTIVE: To perform a stress analysis of the lifting bail shown on page 2, used for lifting of all new cover blocks to be installed on project W-320. The analysis will apply a factor of safety of at least 5 and will assume a impact force to be added to the load. The lifting bails will be rotated 20 degrees from vertical and loaded in the direction shown on page 2. Additionally, a table will be added listing the weights and type of lifting bail of the various port plugs on the cover blocks for inspection and the lifting bail for the steel plug of the Pump Penetration will be designed.

CRITERIA:

1. Functional Design Criteria, WHC-SD-W320-FDC-001, Rev. 2.
2. Hanford Site Hoisting and Rigging Manual, DOE-RL-92-36, January 1993.

GIVEN DATA:

1. Greatest cover block weight appears at the west side Pump Pit (C-06A) cover block at 28,700 lbs with an additional 2,500 lbs for the Pump Penetration Plug for a total weight of 31,200 lbs. (See H-2-818448, sh. 2 & 5)
2. Minimum design factor of 5, based on yield strength, for load bearing structural components. (Ref. 4, section 8.2.1.1). (Nominal strength/rated load)
3. $f_c = 4000$ psi Construction specifications W-320-C5 & C6 of Project W-320.
 $f_{yt} = 36$ ksi Minimum Yield Strength for A36 Steel. (Ref. 8)
 $f_{ys} = 21$ ksi Shear Yield Strength for A36 Steel. (Ref. 6, pg. 584)
 $f_{us} = 58000$ psi Ultimate Tensile Strength for A36 Steel. (Ref. 8)
 $f_{uw} = 72$ ksi Ultimate Tensile Strength for E70XX (AWS A5.1) electrode. (Ref. 2, Table 4.1)

4. Safety class of the cover blocks is SC-3. (Ref. 7)

ASSUMPTIONS: See body of calculation.

REFERENCES:

1. ACI Manual of Concrete Practice (ACI 349-94). American Concrete Institute.
2. Structural Welding Code, Steel. ANSI/AWS D1.1-94. American Welding Society.
3. GC-ANCB-01. Kaiser Engineers Hanford Architectural/Civil Standard.
4. Hanford Site Hoisting and Rigging Manual (DOE-RL-92-36). Hanford Hoisting and Rigging Safety Committee. January 1993.
5. Load and Resistance Factor Design (LRFD). Manual of Steel Construction, First Edition. American Institute of Steel Construction.
6. Mechanics of Materials. Beer and Johnston. McGraw-Hill Book Company. 1981.
7. White Paper on Seismic Design Approach for Safety Related Structures, Systems, Components, and Equipment Provided or Modified by the Project. (W-320/ER4319) Transmittal Number TR-W-320-253.
8. American Society for Testing and Materials (ASTM A-36). 1995

CONCLUSION:

Four lifting bails per cover block at pits C-06A, C-06C, & AY-02E will be sufficient. It is required that the covers blocks are lifted with a rigging system that applies a lifting force at a 20 degree orientation from vertical (See figure on page 2). The lifting bail system analyzed will be the same for all new cover blocks constructed for Tank 241-C-106 and Tank 241-AY-102 pits. By inspection, the lifting bails listed on page 9 are adequately designed for lifting the port plugs. The allowable lifting force per cover block lifting bail is 9271 LBS directed 20 degrees from vertical.

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DESIGN ANALYSIS

Calc. No.: W320-24-025

Revision: 0

Page No.: 2 of 11

Client: WHC

Subject: COVER BLOCK LIFTING BAIL CALCULATION

Location: C-FARM & AY-FARM

WO/Job No.: ER4319/W-320

Date: 7/6/95

Checked: 7/25/95

Revised:

By: JR BOOTH *JRB*By: M. B. LASOTA *MBL*

By:

COVER BLOCK LIFTING BAIL

NTS

KAISER ENGINEERS HANFORD

DESIGN ANALYSIS

HNF-2468, Rev.0

Calc. No. W320-24-025

Revision 0

Page No. 3 of 11

Client: Westinghouse Hanford Company

WO/Job No. ER4319/W-320

Subject: Cover Block Lifting Bail Calculations

Date 7/6/95

By JR Booth

Checked 7/28/95

By M.B. Lasota

Location: C-Farm & AY-Farm

Revised

By

1. Determine Load on Individual Lifting Bail:

- The heaviest cover block was found to be the west block of the Pump Pit at approximately 31,200 lbs. (See H-2-818448, sh. 2)
- There will be four lifting bails per cover block at each pit.
- An impact load will be determined from an assumed lift speed.
- The Dead Weight of the Cover Block plus the impact load will be increased by a factor of five to account for the safety factor. The requirement from Ref. 4, sec. 11.3.1 is a Safety Factor of 3 but a S.F. of 5 is more commonly used as a factor of safety for such devices and is used here for added conservatism.

$$C_b := 31.2\text{-kips}$$

Cover Block Weight

Determine impact force on lifting bail from lifting. Ref. 9, page 2.8-9.

$$V_2 := 500 \frac{\text{ft}}{\text{min}}$$

Speed of Lift

$$V_1 := 0 \frac{\text{ft}}{\text{min}}$$

At Rest Speed

$$g := 32.2 \frac{\text{ft}}{\text{sec}^2}$$

Acceleration do to Gravity

$$t := 5\text{-sec}$$

Time to accelerate to V2.

$$a := \frac{V_2 - V_1}{t}$$

$$a = 1.667 \frac{\text{ft}}{\text{sec}^2}$$

Acceleration

$$F_a := \frac{C_b \cdot a}{g}$$

Force From Acceleration

$$F_a = 1.615 \text{-kips}$$

$$SF := 5$$

Factor of Safety

$$\theta := 20\text{-deg}$$

$$W := \frac{SF \cdot (C_b + F_a)}{4 \cos(\theta)}$$

Factored Weight of Cover Block per Lifting Bail acting in the direction shown on page 2.

$$W = 43.651 \text{-kips}$$

$$W_n := W \cdot \cos(\theta)$$

$$W_n = 41.019 \text{-kips}$$

Force normal to the surface of the cover block used as input into the IMAGES run.

$$W_p := W \cdot \sin(\theta)$$

$$W_p = 14.93 \text{-kips}$$

Force parallel to the surface of the cover block used as input into the IMAGES run.

KAISER ENGINEERS

HANFORD

DESIGN ANALYSIS

HNF-2468, Rev. U
Calc. No. W320-24-025
Revision 0
Page No. 4 of 11

Client: Westinghouse Hanford Company

WO/Job No. ER4319/W-320

Subject: Cover Block Lifting Bail Calculations

Date 7/6/95

By JR Booth

Checked 7/28/95

By M.B. Labeta

Location: C-Farm & AY-Farm

Revised

By

3. Determine Lift Bail Strength: Lifting bails are designed to be loaded in the strong direction only. NO BENDING ABOUT WEAK AXIS. Based on the lifting bail design shown on page 2, tension, shear, and moment stresses were determined using IMAGES and are shown in attachment 1. page 7.

$$\sigma_t := 33.9 \text{ ksi}$$

Maximum combined tensile stress in outer fiber of lifting bail which is less than 36 ksi; therefore, it is OK.

$$SF_t := 5 \cdot \frac{f_{yt}}{\sigma_t} \quad SF_t = 5.31$$

$$\sigma_s := 16.9 \text{ ksi}$$

Combined shear stress which is less than 21 ksi (Shear Yield Strength); therefore, it is OK.

$$SF_s := 5 \cdot \frac{f_{ys}}{\sigma_s} \quad SF_s = 6.213$$

4. Determine the maximum allowable lifting force per bail.

$$W_a := \frac{\left(\frac{f_{yt}}{\sigma_t} \right) \cdot W}{SF}$$

$$W_a = 9.271 \cdot \text{kips}$$

5. Check Weld Strength of Bail to 2" x 3" x 1'-8" Bar:

$$t_e := \left(\frac{5}{16} \cdot \text{in} \right) \cdot 0.707$$

$$t_e = 0.221 \cdot \text{in}$$

Effective Throat Thickness (Ref. 5, sec J.2.2.a)

$$\phi = 0.75 \quad \text{Resistance Factor (Ref. 5, Table J2.3)}$$

$$t = 2 \cdot \text{in} \quad \text{Bail thickness}$$

$$\phi R_n := \phi \cdot t_e \cdot (0.6 \cdot f_{uw})$$

$$\phi R_n = 7.158 \cdot \frac{\text{kips}}{\text{in}}$$

Design Strength of Fillet Weld

$$\phi = 0.9$$

Resistance Factor for metal in tension

$$\phi R_n := \phi \cdot t_e \cdot \left(\frac{f_{yt}}{\sqrt{3}} \right)$$

$$\phi R_n = 4.133 \cdot \frac{\text{kips}}{\text{in}}$$

Design Strength of Base Metal

CONTROLS

$$T_n := 1.4 \cdot 21.83 \cdot \text{kips}$$

$$T_n = 30.56 \cdot \text{kips}$$

1.4 x Reaction at node 8 page 7.

$$T_u := \phi R_n \cdot 4 \cdot t$$

$$T_u = 33.063 \cdot \text{kips} >$$

$$T_n$$

Tensile Strength of Weld for each leg of Bail.

OK!!

6. Concrete Pullout/Shear Strength at Lifting Bail Location.

$$D := 2 \cdot \text{in}$$

Nominal Dimension of Bent Bar

$$f_{us} := 58000$$

$$m_t := D \cdot \sqrt{\frac{f_{us}}{73 \cdot \sqrt{f_c}}}$$

Edge Distance for Tension Load (Ref. 1, sec. B.5.1.1 of 349R)

$$m_t = 7.089 \cdot \text{in}$$

Edge distance for shear will not be a consideration for three reasons:

- The Lifting Bails will be positioned so the stress cones will be developed within the reinforcing cage.
- The lifting force applied will be oriented no greater than 20 degrees from vertical toward the center of the cover block. (See H-2-818448, sh. 2 & 3).

Use 12 inch spacing from edge of cover block to centerline of lifting bail.

Page A-7

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DESIGN ANALYSIS

Calc. No.: W320-24-025

Revision: 0

Page No.: 5 of 11

Client: WHC

Subject: Cover Block Lifting Bail Calculations

Location: C Tank Farm and AY Tank Farm

WO/Job No.: ER4319/W320

Date: 7/6/95

Checked: 7/28/95

Revised:

By: J. R. Booth *JRB*By: M. B. Lasota *MBL*

By:

FIGURE 3

Effective projected area of the shear cone

Client: WHC
Subject: Cover Block Lifting Bail Calculations.

WO/Job No.: ER4319/W 320
Date: 7/6/95 By: J. R. Booth
Checked: 8/1/95 By: M. B. Lasota
Revised: 03/05/96 By: M. B. Lasota *MB*

Location: C-Farm & AY-Farm

$$\phi := 0.65$$

Strength reduction Factor for concrete anchorage. (Ref. 1 sec. B.4.2)

$$P_u := 4 \cdot \phi \cdot \sqrt{f_c} \cdot A_e \cdot \sqrt{\text{psi}}$$

Design strength of concrete anchorage. The Strength Reduction Factor is for additional conservatism above the load factor that will be used to reduce the overall ultimate strength of the lifting bail. (Ref 3, pg.4)

$$P_u = 160.015 \cdot \text{kips}$$

$$SF := \frac{P_u}{1.4 \cdot \frac{W}{5}} \quad SF = 13.092 \quad SF \geq 5.0 \quad \text{OK!!!}$$

Note: The concrete pullout strength (P_u) has a factor of safety greater than 2 times the factor of safety that was used to obtain the allowable load for the lifting bail and more than offsets the difference between designing the lifting bail based on yield stress and concrete being analyzed with ultimate stress method.

7. Check Concrete Bearing:

$$\phi := 0.70$$

Strength Reduction Factor, ACI 318, Sec. 9.3.2.4

$$A_1 := (2 \cdot \text{in})^2 \cdot 2$$

$$A_1 = 8 \cdot \text{in}^2$$

Multiplied by two since supporting surface is much wider than the loaded area,
ACI 318, Sec. 10.15.1.

$$\phi P_p := \phi \cdot 0.85 \cdot (f_c) \cdot A_1$$

$$\phi P_p = 19.04 \cdot \text{kips} \quad \text{lbs}$$

Ultimate Bearing Strength of concrete.

$$\phi P_p > \frac{12.94 \cdot \text{kips}}{2}$$

OK, 12.94 kips is from attachment 1, page 7 reaction at node 8. Actual bearing is significantly smaller since 1 sq. in. crossbar will tend to balance horizontal reaction from both supports.

8. Check the lifting bail of the Pump Penetration Port Plug.

$$S := 490 \cdot \text{pcf}$$

Weight density of steel

Weight of plug...

$$V_{\text{plug}} := \left[(6.5^2 - 5^2) \cdot \text{in}^2 \cdot \frac{\pi}{2} + 2 \cdot 2 \cdot \text{in} \cdot 6 \cdot \text{in} \right] \cdot 2 \cdot \text{in} + 3 \cdot \text{in} \cdot \frac{\pi}{4} \cdot (48 \cdot \text{in})^2 + (6 \cdot \text{in} + 5 \cdot \text{in}) \cdot \frac{\pi}{4} \cdot (45.25 \cdot \text{in})^2 \dots$$

$$+ 0.5 \cdot \text{in} \cdot \frac{\pi}{4} \cdot (52.25^2 - 45.25^2) \cdot \text{in}^2 + \frac{\pi}{3} \cdot 12 \cdot \text{in} \cdot \frac{45.25^2 + 43.125^2 + 45.25 \cdot 43.125}{4} \cdot \text{in}^2 - 18 \cdot \text{in} \cdot \frac{\pi}{4} \cdot (35.25 \cdot \text{in})^2$$

$$W_{\text{plug}} := V_{\text{plug}} \cdot S \quad W_{\text{plug}} = 6899 \cdot \text{lbf}$$

In further calculations use weight of the plug:

$$W_{\text{plug}} := 7000 \cdot \text{lbf}$$

ICF KAISER HANFORD COMPANY

DESIGN ANALYSIS

Client: WHC
Subject: Cover Block Lifting Bail Calculations.

Location: C-Farm & AY-Farm

HNF-2468, Rev. 0
Calc. No. W320-24-025
Revision 1
Page No. 7 of 11

WO/Job No.: ER4319/W 320

Date: 03/05/96 By: M. B. Lasota

Checked: *5/30/96* By: R. W. Davidson

Revised: By:

$$\begin{aligned} V_2 &:= 500 \frac{\text{ft}}{\text{min}} & \text{Speed of Lift} & & V_1 &:= 0 \frac{\text{ft}}{\text{min}} & \text{At Rest Speed} \\ g &:= 32.17 \frac{\text{ft}}{\text{sec}^2} & \text{Acceleration do to Gravity} & & t &:= 5 \text{ sec} & \text{Time to accelerate to } V_2. \\ a &:= \frac{V_2 - V_1}{t} & & & a &:= 1.667 \frac{\text{ft}}{\text{sec}^2} & \text{Acceleration} \\ W &:= W_{\text{plug}} \left(1 + \frac{a}{g} \right) & & & W &:= 7363 \cdot \text{lbf} & \text{Lifting Load acting on the Bail} \end{aligned}$$

From the attachment 2, the critical section of the lifting bail is at the load application ie node 11. The magnitude of the load used in analysis is 1000 lbf. Since the bail is loaded by single vertical load, and analysis is linear elastic, the stresses as well as the internal forces are proportional to the applied load. The following are the internal forces at node 11:

$$P_{1000} := 1000 \cdot \text{lbf}$$

Due to unit load:

$$\begin{aligned} P_{x11} &:= \frac{0.1791 \cdot 10^3 \cdot \text{lbf}}{P_{1000}} & v_{y11} &:= \frac{0.500 \cdot 10^3 \cdot \text{lbf}}{P_{1000}} & m_{z11} &:= \frac{1.393 \cdot 10^3 \cdot \text{lbf} \cdot \text{in}}{P_{1000}} \\ P_{x11} &:= 0.1791 & v_{y11} &:= 0.5000 & m_{z11} &:= 1.3930 \cdot \text{in} \end{aligned}$$

Internal forces in the Bail (node 11) resulting from application of the load W are:

$$\begin{aligned} P_{x11} &:= W \cdot p_{x11} & P_{x11} &:= 1318.64 \cdot \text{lbf} \\ V_{y11} &:= W \cdot v_{y11} & V_{y11} &:= 3681.31 \cdot \text{lbf} \\ M_{z11} &:= W \cdot m_{z11} & M_{z11} &:= 10256.12 \cdot \text{lbf} \cdot \text{in} \end{aligned}$$

From Ref. 4 Safety Factors required for the under the hook devices are:

FS=5 - against the ultimate strength, and

FS=3 - against the yield strength

Allowable stress with respect to ultimate strength of the Bail is:

$$\begin{aligned} f_{au} &:= \frac{f_y t}{5} & f_{au} &:= 7200 \cdot \text{psi} & \text{Shape factor for rectangular section is:} & & v &:= 1.5 \\ f_{bu} &:= \frac{f_y r v}{5} & f_{bu} &:= 10800 \cdot \text{psi} \\ f_{vu} &:= \frac{f_y t}{5 \cdot \sqrt{3}} & f_{vu} &:= 4157 \cdot \text{psi} \end{aligned}$$

ICF KAISER HANFORD COMPANY

DESIGN ANALYSIS

HNF-2468, Rev. C
Calc. No. W320-24-025
Revision 1
Page No. 8 of 11

Client: WHC
Subject: Cover Block Lifting Bail Calculations.

WO/Job No.: ER4319/W 320

Date: 03/06/96

By: M. B. Lasota

Checked: 5/30/96

By: R. W. Davidson

Location: C-Farm & AY-Farm

Revised:

By:

Assuming, the cross-section of the Bail is square with the leg "a", the section properties are as follow:

$$A_x = d^2 \quad S_z = \frac{d^3}{6}$$

Interaction formula for the axial and bending stress is:

$$IF = \frac{P_{x11}}{A_x \cdot f_{au}} + \frac{M_{z11}}{S_z \cdot f_{bu}} \leq 1.0$$

After substitution for variables the equation with respect to a is:

$$m_1 = \frac{P_{x11}}{f_{au}} \cdot \frac{1}{in^2} \quad m_1 = 0.1831 \quad m_2 = \frac{M_{z11} \cdot 6}{f_{bu}} \cdot \frac{1}{in^3} \quad m_2 = 5.6978$$

$$f(x) := x^3 - m_1 \cdot x - m_2 \quad x := 1$$

$$d := \text{root}(x^3 - m_1 \cdot x - m_2, x) \quad d \cdot in = 1.8202 \cdot in$$

Use 2" square bar. $d := 2 \cdot in$

Actual Factor of Safety using 2" square bar is:

With respect to bail's ultimate strength

$$SF_u := d^2 \cdot f_{yt} \left(P_{x11} + \frac{6 \cdot M_{z11}}{d \cdot v} \right)^{-1} \quad SF_u = 6.596 \quad SF_u \geq 5 \quad OK$$

With respect to bail's yield strength

$$SF_y := d^2 \cdot f_{yt} \left(P_{x11} + \frac{6 \cdot M_{z11}}{d} \right)^{-1} \quad SF_y = 4.488 \quad SF_y \geq 3 \quad OK$$

With respect to bail's shear strength

$$SF_v := \frac{d^2 \cdot f_{yt}}{V_{y11} \cdot \sqrt{3}} \quad SF_v = 22.584 \quad SF_v \geq 3 \quad OK$$

Check Weld Strength of Bail to steel plate...

Due to unit load at node 1 (in member local coordinate system):

$$P_{x01} := \frac{0.5 \cdot 10^3 \cdot lbf}{P_{1000}} \quad v_{z01} := \frac{0.1791 \cdot 10^3 \cdot lbf}{P_{1000}} \quad m_{y01} := \frac{0.6223 \cdot 10^3 \cdot lbf \cdot in}{P_{1000}}$$

$$P_{x01} = 0.5000 \quad v_{z01} = 0.1791 \quad m_{y01} = 0.6223 \cdot in$$

Internal forces in the Bail (node 1) resulting from application of the load W are:

$$\begin{aligned} P_{x01} &:= W \cdot P_{x01} & P_{x01} &= 3681.31 \cdot lbf \\ V_{z01} &:= W \cdot v_{z01} & V_{z01} &= 1318.64 \cdot lbf \\ M_{y01} &:= W \cdot m_{y01} & M_{y01} &= 4581.75 \cdot lbf \cdot in \end{aligned}$$

ICF KAISER HANFORD COMPANY

DESIGN ANALYSIS

HNF-2468, Rev.0
Calc. No. W320-24-025
Revision 1
Page No. 8a of 11

Client: WHC
Subject: Cover Block Lifting Bail Calculations.

WO/Job No.: ER4319/W 320
Date: 03/06/96 By: M. B. Lasota
Checked: 5/30/96 By: R. W. Davidson
Revised: By:

Location: C-Farm & AY-Farm

$$d = 2 \cdot \text{in}$$

$$A_w := 4 \cdot d \quad A_{ws} := 8 \cdot \text{in}$$

$$A_{ws} := 2 \cdot d \quad A_{ws} := 4 \cdot \text{in}$$

$$S_w := d^2 + \frac{d^2}{3} \quad S_w = 5.33 \cdot \text{in}^2$$

$$R := \left[\left(\frac{P_{x01}}{A_w} + \frac{M_{y01}}{S_w} \right)^2 + \left(\frac{V_{z01}}{A_{ws}} \right)^2 \right]^{\frac{1}{2}}$$

$$t_e := \left(\frac{3}{8} \cdot \text{in} \right) \cdot \frac{1}{\sqrt{2}} \quad t_e = 0.27 \cdot \text{in}$$

$$\phi = 0.75$$

$$\phi R_{wm} := \phi \cdot t_e \cdot (0.6 \cdot f_{uw}) \quad \phi R_{wm} = 8.35 \cdot \frac{\text{kips}}{\text{in}}$$

$$SF_{wm} := \frac{\phi R_{wm}}{R} \quad SF_{wm} = 6.14$$

$$\phi_v = 0.9$$

$$f_{yt} = 36 \cdot \text{ksi}$$

$$\phi R_{bm} := \phi_v \cdot t_e \cdot (0.6 \cdot f_{yt}) \cdot \sqrt{2} \quad \phi R_{bm} = 7.29 \cdot \frac{\text{kips}}{\text{in}}$$

$$SF_{bm} := \frac{\phi R_{bm}}{R} \quad SF_{bm} = 5.36$$

Weld area subject to tension.

Weld area subject to shear.

Weld section modulus.

$$R = 1.3598 \cdot \frac{\text{kips}}{\text{in}}$$

Effective Throat Thickness (Ref. 5, sec J.2.2.a)

Resistance Factor (Ref.5, Table J2.3)

Design Strength of Fillet Weld

$$SF_{wm} \geq 5 \quad \text{OK.}$$

Resistance Factor (Ref.5, F2.2)

Yield Strength of Base Metal

Design Strength of Base Metal **CONTROLS**

$$SF_{bm} \geq 5 \quad \text{OK.}$$

Use 3/8" fillet weld. It satisfies minimum weld requirements. (Ref. 5, Table J2.5)

The 2" square bar used as the lifting bail for the Pump Penetration Plug is sufficient to provide the Safety Factor of 5 during lifting.

Client: Numatec Hanford Company
Subject: Cover Block Lifting Bail Calculations.

Location: C-Farm and AY- Farm

WO/Job No.: ER4319/W320

Date: 4/25/97

Checked: 5/8/97

Revised

By: MB Lasota

By:

By

**REPLACEMENT PLUG ACCOMMODATING
LEAK DETECTOR WIRING REQUIREMENTS**

MATERIAL ASTM A36 STEEL

Client: Numatec Hanford Company
Subject: Cover Block Lifting Bail Calculations.

WO/Job No.: ER4319/W320

Date: 4/25/97

By: MB Casota

Checked: 5/14/97

By: [Signature]

Revised

By

Location: C-Farm and AY-Farm

The replacement Plug consist of 6 circular discs and the bail constructed from the round stock 1/2" diameter.
The following is a calculation of the weight of the Plug:

i := 0..6

d_i :=

t_i :=

7.625-in
9.75-in
9.75-in
9.75-in
14.0-in
3.5-in

2.5-in
2-in
2-in
2.5-in
1-in
1.5-in

The length of the round stock is:

$$t_g := 2.6\text{-in} + \pi \cdot 3.25\text{-in} \quad t_g = 22.21\text{-in} \quad d_g := 0.5\text{-in}$$

Volume of the steel used in construction of the Plug is:

$$V_s := \frac{\pi}{4} \left[\sum_i (d_i)^2 \cdot t_i - \left[t_1 + \frac{\sum_{i=1}^4 t_i}{\sin(60)} \right] \cdot (1\text{-in})^2 \right]$$

$$V_s = 789.95\text{-in}^3 \quad \gamma_s := 490\text{-pcf}$$

$$W_{pl} := V_s \cdot \gamma_s \quad W_{pl} = 224\text{-lbf} \quad \text{use} \quad W_{pl} := 225\text{-lbf}$$

Check shielding properties of the plug at the least thickness in terms of equivalent concrete thickness.

$$\gamma_c := 140\text{-pcf} \quad t_{min} := 7.13\text{-in}$$

$$t_{con} := t_{min} \cdot \frac{\gamma_s}{\gamma_c} \quad t_{con} = 24.95\text{-in} \quad t_{con} \geq 24\text{-in} \quad \text{OK}$$

DESIGN ANALYSIS

Calc. No.: W-320-24-025

Revision No.: 1

Page No.: 9 of 11

Client: Numatec Hanford Company
 Subject: Cover Block Lifting Bail Calculations

WO/Job No.: ER4319/W-320
 Date: 4/28/1997 By: M.B. Lasota
 Checked: 5/8/97 By: R.W. Davidson
 Revised:

Location: C-Farm and AY-Farm

Cover Block Port Plug Evaluation

	Weight (lbf)	Bail Type	By Inspection
<u>Pump Pit (241-C06A)</u>			
3" Dia Valve Port Plug	40	BAR 1/2" Dia.	OK
(3) 8" Dia Camera Port Plug	220	BAR 1/2" Dia.	OK
(1) 8" Dia Leak Detector Access Plug	225	BAR 1/2" Dia.	OK
Winch Port Shielding Plug	100	BAR 1/2" Dia.	OK
Slot Plug, Block A	590	BAR 3/4" Dia.	OK
Slot Plug, Block B	653	BAR 3/4" Dia.	OK
<u>Sluice Pit (241-C-06C)</u>			
8" Dia Camera Port Plug	220	BAR 1/2" Dia.	OK
(1) 8" Dia Leak Detector Access Plug	225	BAR 1/2" Dia.	OK
Sluicer Port Plug			Existing
3" Dia Valve Port Plug	40	BAR 1/2" Dia.	OK
Slot Plug, Block A	460	BAR 3/4" Dia.	OK
<u>Pump Pit (241-AY-02A)</u>			
12" Dia Valve Port	350	BAR 3/4" Dia.	OK
<u>Sluice Pit (241-AY-02E)</u>			
12" Dia Valve Port	350	BAR 3/4" Dia.	OK
(2) 8" Dia Camera Port Plug	220	BAR 1/2" Dia.	OK
Winch Port Shielding Plug	100	BAR 1/2" Dia.	OK
(3) Slot Plugs, Block A	365	BAR 3/4" Dia.	OK

ICF KAISER HANFORD**DESIGN ANALYSIS**

Calc. No.: W320-24-025

Revision: 0

Page No.: 10 of 11

Client: WHC

Subject: Cover Block Lifting Bail Calculations

Location: C Tank Farm and AY Tank Farm

WO/Job No.: ER4319/W320

Date: 7/21/95

Checked: 8/1/95

Revised:

By: M.B. Lasota

By: J.R. Booth

By:

MATH MODEL - SCALE 1:3
 NODES & MEMBER INCIDENCES

ATTACHMENT 1

TOTAL OF 10 PAGES

RUN ID#: AA77104

DATE: 7-21-95

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CHECK GEOMETRY

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07/21/95

09:57:13

MATERIAL PROPERTIES

Material No	Modulus of Elasticity	Weight Density	Coeff of Thermal Exp.	Poisson's Ratio	Shear Web Modulus
1	3.00000E+07	2.83000E-01	6.33000E-06	3.00E-01	0.00000E+00

NODE COORDINATES

Node	X-Coord.	Y-Coord.	Z-Coord.
1	8.00000E+00	1.26250E+01	0.00000E+00
2	7.21541E+00	1.25391E+01	0.00000E+00
3	6.46801E+00	1.22854E+01	0.00000E+00
4	5.79324E+00	1.18759E+01	0.00000E+00
5	5.22309E+00	1.13301E+01	0.00000E+00
6	4.92200E+00	1.09140E+01	0.00000E+00
7	0.00000E+00	3.00000E+00	0.00000E+00
8	0.00000E+00	0.00000E+00	0.00000E+00
9	8.78459E+00	1.25391E+01	0.00000E+00
10	9.53199E+00	1.22854E+01	0.00000E+00
11	1.02068E+01	1.18759E+01	0.00000E+00
12	1.07769E+01	1.13301E+01	0.00000E+00
13	1.10780E+01	1.09140E+01	0.00000E+00
14	1.60000E+01	3.00000E+00	0.00000E+00
15	1.60000E+01	0.00000E+00	0.00000E+00

BEAM PROPERTIES

Multiplier = 1 (For AISC database properties only)

Prop No	X-Section Area	Moment of Inertia Iy / Iz	Torsional Const. J
1	4.000E+00	1.333E+00 1.333E+00	2.667E+00

Prop No	Max. Fiber Cy	Dist / Cz	Shear SSFy / SSFz	Shape Fact	Shear Stress SSTRFy / SSTRFz
1	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.50E+00 1.50E+00

KAISER ENGINEERS HANFORD S/N:801854

PAGE 2

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CHECK GEOMETRY

Version 2.0 07/01/90

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BEAM CONNECTIVITY

Beam No	Nodes From/ To	Ref	Prop No	Mat No	pincodes I / J	Rigid End I / J	Offset	Length	Beam Type
1	8 7	0	1	1		0.00E+00 0.00E+00		0.30E+01	Beam
2	7 6	0	1	1		0.00E+00 0.00E+00		0.93E+01	Beam
3	6 5	0	1	1		0.00E+00 0.00E+00		0.51E+00	Beam
4	5 4	0	1	1		0.00E+00 0.00E+00		0.79E+00	Beam
5	4 3	0	1	1		0.00E+00 0.00E+00		0.79E+00	Beam
6	3 2	0	1	1		0.00E+00 0.00E+00		0.79E+00	Beam
7	2 1	0	1	1		0.00E+00 0.00E+00		0.79E+00	Beam
8	1 9	0	1	1		0.00E+00 0.00E+00		0.79E+00	Beam
9	9 10	0	1	1		0.00E+00 0.00E+00		0.79E+00	Beam
10	10 11	0	1	1		0.00E+00 0.00E+00		0.79E+00	Beam
11	11 12	0	1	1		0.00E+00 0.00E+00		0.79E+00	Beam
12	12 13	0	1	1		0.00E+00 0.00E+00		0.51E+00	Beam
13	13 14	0	1	1		0.00E+00 0.00E+00		0.93E+01	Beam
14	14 15	0	1	1		0.00E+00 0.00E+00		0.30E+01	Beam

RESTRAINTS

Node No	Global/Local	Restraint Directions
8	GLOBAL	X Y Z RX RY RZ
15	GLOBAL	X Y Z RX RY RZ

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LOAD CASE 1

VERTICAL LOAD

GRAVITY LOADING

Direction	Gravity Factor	Load Factor
X	.0000E+00	.1000E+01
Y	.0000E+00	.1000E+01
Z	.0000E+00	.1000E+01

INERTIA LOADING

Direction	Translational Accel.	Rotational Accel.	Rotational Velocity	Origin
X	.0000E+00	.0000E+00	.0000E+00	.0000E+00
Y	.0000E+00	.0000E+00	.0000E+00	.0000E+00
Z	.0000E+00	.0000E+00	.0000E+00	.0000E+00

Gravity Acceleration = .3864E+03

CONCENTRATED LOADS

Node	Fx	Fy	Fz	Mx	My	Mz
1	.0000E+00	.4365E+05	.0000E+00	.0000E+00	.0000E+00	.0000E+00

REFERENCE TEMPERATURE = .0000E+00

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LOAD CASE 2

INCLINED LOAD

GRAVITY LOADING

Direction	Gravity Factor	Load Factor
X	.0000E+00	.1000E+01
Y	.0000E+00	.1000E+01
Z	.0000E+00	.1000E+01

INERTIA LOADING

Direction	Translational Accel.	Rotational Accel.	Rotational Velocity	Origin
X	.0000E+00	.0000E+00	.0000E+00	.0000E+00
Y	.0000E+00	.0000E+00	.0000E+00	.0000E+00
Z	.0000E+00	.0000E+00	.0000E+00	.0000E+00

Gravity Acceleration = .3864E+03

CONCENTRATED LOADS

Node	Fx	Fy	Fz	Mx	My	Mz
10	.1493E+05	.4102E+05	.0000E+00	.0000E+00	.0000E+00	.0000E+00

REFERENCE TEMPERATURE = .0000E+00

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LOAD CASE 1

VERTICAL LOAD

APPLIED LOAD VECTOR

Node	Fx	Fy	Fz	Mx	My	Mz
1	.0000E+00	.4365E+05	.0000E+00	.0000E+00	.0000E+00	.0000E+00

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LOAD CASE 1

VERTICAL LOAD

DISPLACEMENTS

Node	Translations			/	Rotations		
	X	Y	Z		X	Y	Z
1	.2347E-09	.7316E-02	.0000E+00	/	.0000E+00	.0000E+00	.4595E-10
2	-.3236E-04	.6691E-02	.0000E+00	/	.0000E+00	.0000E+00	.6337E-03
3	-.1486E-03	.5763E-02	.0000E+00	/	.0000E+00	.0000E+00	.9807E-03
4	.5623E-03	.4798E-02	.0000E+00	/	.0000E+00	.0000E+00	.1106E-02
5	-.1130E-02	.3973E-02	.0000E+00	/	.0000E+00	.0000E+00	.1085E-02
6	-.1528E-02	.3551E-02	.0000E+00	/	.0000E+00	.0000E+00	.1030E-02
7	-.2630E-02	.5456E-03	.0000E+00	/	.0000E+00	.0000E+00	.7075E-03
8	.0000E+00	.0000E+00	.0000E+00	/	.0000E+00	.0000E+00	.0000E+00
9	-.3236E-04	.6691E-02	.0000E+00	/	.0000E+00	.0000E+00	.6337E-03
10	-.1486E-03	.5763E-02	.0000E+00	/	.0000E+00	.0000E+00	.9807E-03
11	-.5623E-03	.4798E-02	.0000E+00	/	.0000E+00	.0000E+00	.1106E-02
12	-.1130E-02	.3973E-02	.0000E+00	/	.0000E+00	.0000E+00	.1085E-02
13	-.1528E-02	.3551E-02	.0000E+00	/	.0000E+00	.0000E+00	.1030E-02
14	-.2630E-02	.5456E-03	.0000E+00	/	.0000E+00	.0000E+00	.7075E-03
15	.0000E+00	.0000E+00	.0000E+00	/	.0000E+00	.0000E+00	.0000E+00

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LOAD CASE 2

INCLINED LOAD

APPLIED LOAD VECTOR

Node	Fx	Fy	Fz	Mx	My	Mz
10	.1493E+05	.4102E+05	.0000E+00	.0000E+00	.0000E+00	.0000E+00

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LOAD CASE 2

INCLINED LOAD

DISPLACEMENTS

Node	Translations			/	Rotations		
	X	Y	Z		X	Y	Z
1	.1368E-01	.5365E-02	.0000E+00	/	.0000E+00	.0000E+00	.3003E-02
2	.1384E-01	.2602E-02	.0000E+00	/	.0000E+00	.0000E+00	.3011E-02
3	.1451E-01	.1237E-03	.0000E+00	/	.0000E+00	.0000E+00	.2724E-02
4	.1546E-01	-.1806E-02	.0000E+00	/	.0000E+00	.0000E+00	.2228E-02
5	.1642E-01	-.3093E-02	.0000E+00	/	.0000E+00	.0000E+00	.1620E-02
6	.1692E-01	-.3614E-02	.0000E+00	/	.0000E+00	.0000E+00	.1208E-02
7	.5711E-02	.5935E-03	.0000E+00	/	.0000E+00	.0000E+00	-.2272E-02
8	.0000E+00	.0000E+00	.0000E+00	/	.0000E+00	.0000E+00	.0000E+00
9	.1408E-01	.8025E-02	.0000E+00	/	.0000E+00	.0000E+00	.2627E-02
10	.1488E-01	.1016E-01	.0000E+00	/	.0000E+00	.0000E+00	.1829E-02
11	.1539E-01	.1084E-01	.0000E+00	/	.0000E+00	.0000E+00	.9067E-03
12	.1563E-01	.1095E-01	.0000E+00	/	.0000E+00	.0000E+00	.1585E-03
13	.1559E-01	.1083E-01	.0000E+00	/	.0000E+00	.0000E+00	-.2569E-03
14	.1335E-02	.4320E-03	.0000E+00	/	.0000E+00	.0000E+00	-.1218E-02
15	.0000E+00	.0000E+00	.0000E+00	/	.0000E+00	.0000E+00	.0000E+00

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SOLVE BEAM LOADS/STRESSES Version 2.0 07/01/90

Load Case 1: VERTICAL LOAD

BEAM LOADS AND/OR STRESSES

LLoads	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Maximum	Minimum	Cmb. Shear			
BEAM NO. 1							
Stress	8	.5456E+04	.0000E+00	-.4851E+04	.0000E+00	-.2163E+05	.0000E+00
Stress	8	.2709E+05	-.1618E+05	.1354E+05			
Stress	-7	.5456E+04	.0000E+00	-.4851E+04	.0000E+00	.7483E+04	.0000E+00
Stress	7	.1294E+05	-.2027E+04	.6470E+04			
BEAM NO. 2							
Stress	7	.6341E+04	-.2029E+03	.0000E+00	.0000E+00	.7483E+04	
Stress	7	.1382E+05	-.1142E+04	.6912E+04			
Stress	6	.6341E+04	-.2029E+03	.0000E+00	.0000E+00	.3700E+04	
Stress	6	.1004E+05	.2641E+04	.5021E+04			
BEAM NO. 3							
Stress	6	.6316E+04	-.8677E+03	.0000E+00	.0000E+00	.3700E+04	
Stress	6	.1002E+05	.2616E+04	.5008E+04			
Stress	5	.6316E+04	-.8677E+03	.0000E+00	.0000E+00	.2809E+04	
Stress	5	.9125E+04	.3508E+04	.4536E+04			
BEAM NO. 4							
Stress	5	.6109E+04	-.2557E+04	.0000E+00	.0000E+00	.2809E+04	
Stress	5	.8918E+04	.3301E+04	.4459E+04			
Stress	4	.6109E+04	-.2557E+04	.0000E+00	.0000E+00	-.1229E+04	
Stress	4	.7339E+04	.4880E+04	.3669E+04			
BEAM NO. 5							
Stress	4	.5596E+04	-.4480E+04	.0000E+00	.0000E+00	-.1229E+04	
Stress	4	.6825E+04	.4366E+04	.3412E+04			
Stress	3	.5596E+04	-.4480E+04	.0000E+00	.0000E+00	-.8304E+04	
Stress	3	.1390E+05	-.2708E+04	.6950E+04			
BEAM NO. 6							
Stress	3	.4816E+04	-.6191E+04	.0000E+00	.0000E+00	-.8304E+04	
Stress	3	.1312E+05	-.3487E+04	.6560E+04			
Stress	2	.4816E+04	-.6191E+04	.0000E+00	.0000E+00	-.1808E+05	
Stress	2	.2290E+05	-.1326E+05	.1145E+05			
BEAM NO. 7							
Stress	2	.3809E+04	-.7608E+04	.0000E+00	.0000E+00	-.1808E+05	
Stress	2	.2189E+05	-.1427E+05	.1094E+05			
Stress	1	.3809E+04	-.7608E+04	.0000E+00	.0000E+00	-.3009E+05	
Stress	1	.3390E+05	-.2628E+05	.1695E+05			
BEAM NO. 8							
Stress	1	.3809E+04	-.7608E+04	.0000E+00	.0000E+00	-.3009E+05	
Stress	1	.3390E+05	-.2628E+05	.1695E+05			

KAISER ENGINEERS HANFORD S/N:801854 07/21/95
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 ***** I N A G E S 3 D *****
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SOLVE BEAM LOADS/STRESSES Version 2.0 07/01/90

Load Case 1: VERTICAL LOAD

LLoads	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Maximum	Minimum	Cmb. Shear			
BEAM NO. 9							
Stress	9	.3809E+04	-.7608E+04	.0000E+00	.0000E+00	.0000E+00	-.1808E+05
Stress	9	.2189E+05	-.1427E+05	.1094E+05			
BEAM NO. 10							
Stress	9	.4816E+04	.6191E+04	.0000E+00	.0000E+00	.0000E+00	-.1808E+05
Stress	9	.2290E+05	-.1326E+05	.1145E+05			
Stress	10	.4816E+04	.6191E+04	.0000E+00	.0000E+00	.0000E+00	-.8304E+04
Stress	10	.1312E+05	-.3487E+04	.6560E+04			
BEAM NO. 11							
Stress	10	.5596E+04	.4480E+04	.0000E+00	.0000E+00	.0000E+00	-.8304E+04
Stress	10	.1390E+05	-.2708E+04	.6950E+04			
Stress	11	.5596E+04	.4480E+04	.0000E+00	.0000E+00	.0000E+00	-.1229E+04
Stress	11	.6825E+04	.4366E+04	.3412E+04			
BEAM NO. 12							
Stress	11	.6109E+04	.2557E+04	.0000E+00	.0000E+00	.0000E+00	-.1229E+04
Stress	11	.7339E+04	.4880E+04	.3669E+04			
Stress	12	.6109E+04	.2557E+04	.0000E+00	.0000E+00	.0000E+00	.2809E+04
Stress	12	.8918E+04	.3301E+04	.4459E+04			
BEAM NO. 13							
Stress	12	.6316E+04	.8677E+03	.0000E+00	.0000E+00	.0000E+00	.2809E+04
Stress	12	.9125E+04	.3508E+04	.4536E+04			
Stress	13	.6316E+04	.8677E+03	.0000E+00	.0000E+00	.0000E+00	.3700E+04
Stress	13	.1002E+05	.2616E+04	.5008E+04			
BEAM NO. 14							
Stress	13	.6341E+04	.2029E+03	.0000E+00	.0000E+00	.0000E+00	.3700E+04
Stress	13	.1004E+05	.2641E+04	.5021E+04			
Stress	14	.6341E+04	.2029E+03	.0000E+00	.0000E+00	.0000E+00	.7483E+04
Stress	14	.1382E+05	-.1142E+04	.6912E+04			
BEAM NO. 15							
Stress	14	.5456E+04	.0000E+00	.4851E+04	.0000E+00	.7483E+04	.0000E+00
Stress	14	.1294E+05	-.2027E+04	.6470E+04			
Stress	15	.5456E+04	.0000E+00	.4851E+04	.0000E+00	-.2163E+05	.0000E+00
Stress	15	.2709E+05	-.1618E+05	.1354E+05			

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SOLVE BEAM LOADS/STRESSES Version 2.0 07/01/90

Load Case 1:VERTICAL LOAD

MAXIMUM STRESS SUMMARY FOR BEAMS/TRUSSES
 WITHIN SPECIFIED RANGE 1- 14

Maximum (absolute) Stress = .3390E+05 at BEAM 8

Beam	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
8	.3809E+04	.7608E+04	.0000E+00	.0000E+00	.0000E+00	-.3009E+05
	Maximum	Minimum	Cmb. Shear			
	.3390E+05	-.2628E+05	.1695E+05			

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SOLVE REACTIONS Version 2.0 07/01/90

Load Case 1:VERTICAL LOAD

REACTIONS

Node	Fx	Fy	Fz	Mx	My	Mz
8	-.1294E+05	-.2183E+05	.0000E+00	.0000E+00	.0000E+00	.2884E+05
15	.1294E+05	-.2183E+05	.0000E+00	.0000E+00	.0000E+00	-.2884E+05

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SOLVE BEAM LOADS/STRESSES Version 2.0 07/01/90

Load Case 2: INCLINED LOAD

BEAM LOADS AND/OR STRESSES

LLoads	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Maximum	Minimum	Cmb. Shear	Torsion	Y-Bending	Z-Bending
BEAM NO. 1							
Stress	8	.5935E+04	.0000E+00	-.7122E+04	.0000E+00	-.4409E+05	.0000E+00
Stress	8	.5002E+05	-.3815E+05	.2501E+05			
Stress	7	.5935E+04	.0000E+00	-.7122E+04	.0000E+00	-.1348E+04	.0000E+00
Stress	7	.7282E+04	.4587E+04	.3641E+04			
BEAM NO. 2							
Stress	7	.7547E+04	.1346E+04	.0000E+00	.0000E+00	.0000E+00	-.1348E+04
Stress	7	.8895E+04	.6199E+04	.4447E+04			
Stress	6	.7547E+04	.1346E+04	.0000E+00	.0000E+00	.0000E+00	.2375E+05
Stress	6	.3130E+05	-.1621E+05	.1565E+05			
BEAM NO. 3							
Stress	6	.7591E+04	.5512E+03	.0000E+00	.0000E+00	.0000E+00	.2375E+05
Stress	6	.3134E+05	-.1616E+05	.1567E+05			
Stress	5	.7591E+04	.5512E+03	.0000E+00	.0000E+00	.0000E+00	.2432E+05
Stress	5	.3191E+05	-.1673E+05	.1595E+05			
BEAM NO. 4							
Stress	5	.7534E+04	-.1505E+04	.0000E+00	.0000E+00	.0000E+00	.2432E+05
Stress	5	.3185E+05	-.1679E+05	.1593E+05			
Stress	4	.7534E+04	-.1505E+04	.0000E+00	.0000E+00	.0000E+00	.2194E+05
Stress	4	.2948E+05	-.1441E+05	.1474E+05			
BEAM NO. 5							
Stress	4	.7138E+04	-.3916E+04	.0000E+00	.0000E+00	.0000E+00	.2194E+05
Stress	4	.2908E+05	-.1480E+05	.1454E+05			
Stress	3	.7138E+04	-.3916E+04	.0000E+00	.0000E+00	.0000E+00	.1576E+05
Stress	3	.2290E+05	-.8621E+04	.1145E+05			
BEAM NO. 6							
Stress	3	.6403E+04	-.6140E+04	.0000E+00	.0000E+00	.0000E+00	.1576E+05
Stress	3	.2216E+05	-.9355E+04	.1108E+05			
Stress	2	.6403E+04	-.6140E+04	.0000E+00	.0000E+00	.0000E+00	.6063E+04
Stress	2	.1247E+05	.3401E+03	.6233E+04			
BEAM NO. 7							
Stress	2	.5366E+04	-.8074E+04	.0000E+00	.0000E+00	.0000E+00	.6063E+04
Stress	2	.1143E+05	-.6977E+03	.5715E+04			
Stress	1	.5366E+04	-.8074E+04	.0000E+00	.0000E+00	.0000E+00	-.6685E+04
Stress	1	.1205E+05	-.1319E+04	.6025E+04			
BEAM NO. 8							
Stress	1	.4074E+04	-.9624E+04	.0000E+00	.0000E+00	.0000E+00	-.6685E+04
Stress	1	.1076E+05	-.2611E+04	.5379E+04			

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SOLVE BEAM LOADS/STRESSES Version 2.0 07/01/90

Load Case 2: INCLINED LOAD

LLoads	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Maximum	Minimum	Cmb. Shear	Torsion	Y-Bending	Z-Bending
BEAM NO. 9							
Stress	9	.4074E+04	-.9624E+04	.0000E+00	.0000E+00	.0000E+00	-.2188E+05
Stress	9	.2595E+05	-.1781E+05	.1298E+05			
BEAM NO. 10							
Stress	9	.2588E+04	-.1072E+05	.0000E+00	.0000E+00	.0000E+00	-.2188E+05
Stress	9	.2447E+05	-.1929E+05	.1223E+05			
Stress	10	.2588E+04	-.1072E+05	.0000E+00	.0000E+00	.0000E+00	-.3881E+05
Stress	10	.4139E+05	-.3622E+05	.2070E+05			
BEAM NO. 11							
Stress	10	.3109E+04	.4750E+04	.0000E+00	.0000E+00	.0000E+00	-.3881E+05
Stress	10	.4191E+05	-.3570E+05	.2096E+05			
Stress	11	.3109E+04	.4750E+04	.0000E+00	.0000E+00	.0000E+00	-.3131E+05
Stress	11	.3441E+05	-.2820E+05	.1721E+05			
BEAM NO. 12							
Stress	11	.3721E+04	.3628E+04	.0000E+00	.0000E+00	.0000E+00	-.3131E+05
Stress	11	.3503E+05	-.2758E+05	.1751E+05			
Stress	12	.3721E+04	.3628E+04	.0000E+00	.0000E+00	.0000E+00	-.2558E+05
Stress	12	.2930E+05	-.2186E+05	.1465E+05			
BEAM NO. 13							
Stress	12	.4095E+04	.2565E+04	.0000E+00	.0000E+00	.0000E+00	-.2558E+05
Stress	12	.2474E+05	-.2148E+05	.1488E+05			
Stress	13	.4095E+04	.2565E+04	.0000E+00	.0000E+00	.0000E+00	-.2294E+05
Stress	13	.2704E+05	-.1885E+05	.1352E+05			
BEAM NO. 14							
Stress	13	.4205E+04	.2129E+04	.0000E+00	.0000E+00	.0000E+00	-.2294E+05
Stress	13	.2715E+05	-.1874E+05	.1357E+05			
Stress	14	.4205E+04	.2129E+04	.0000E+00	.0000E+00	.0000E+00	.1675E+05
Stress	14	.2096E+05	-.1255E+05	.1048E+05			
BEAM NO. 15							
Stress	14	.4320E+04	.0000E+00	.1523E+04	.0000E+00	.1675E+05	.0000E+00
Stress	14	.2107E+05	-.1243E+05	.1054E+05			
Stress	15	.4320E+04	.0000E+00	.1523E+04	.0000E+00	.7612E+04	.0000E+00
Stress	15	.1193E+05	-.3292E+04	.5966E+04			

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SOLVE BEAM LOADS/STRESSES Version 2.0 07/01/90

Load Case 2: INCLINED LOAD

MAXIMUM STRESS SUMMARY FOR BEAMS/TRUSSES
 WITHIN SPECIFIED RANGE 1- 14

Maximum (absolute) Stress = .5002E+05 at BEAM 1

Beam	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
1	.5935E+04	.0000E+00	-.7122E+04	.0000E+00	-.4409E+05	.0000E+00
	Maximum	Minimum	Cmb. Shear			
	.5002E+05	-.3815E+05	.2501E+05			

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SOLVE REACTIONS Version 2.0 07/01/90

Load Case 2: INCLINED LOAD

REACTIONS

Node	Fx	Fy	Fz	Mx	My	Mz
8	-.1899E+05	-.2374E+05	.0000E+00	.0000E+00	.0000E+00	.5877E+05
15	.4061E+04	-.1728E+05	.0000E+00	.0000E+00	.0000E+00	.1015E+05

Attachment 2.

Analysis of the bail for Port Plug - Run ID = YD04897

Date: 03-05-1996; Time: 08:49:49

Rep
5/30/96
1

KAISER ENGINEERS HANFORD S/N:801854 03/05/96
PAGE 1 RUN ID=YD04897 08:49:49

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CHECK GEOMETRY Version 2.0 07/01/90

Analysis of the lifting bail for the Pump Penetration Port P

MATERIAL PROPERTIES

Material No	Modulus of Elasticity	Weight Density	Coeff of Thermal Exp.	Poisson's Ratio	Shear Web Modulus
1	3.00000E+07	2.83000E-01	6.33000E-06	3.00E-01	0.00000E+00

NODE COORDINATES

Node	X-Coord.	Y-Coord.	Z-Coord.
1	0.00000E+00	0.00000E+00	0.00000E+00
2	0.00000E+00	6.00000E+00	0.00000E+00
3	8.73555E-02	6.99848E+00	0.00000E+00
4	3.46768E-01	7.96662E+00	0.00000E+00
5	7.70355E-01	8.87500E+00	0.00000E+00
6	1.34525E+00	9.69603E+00	0.00000E+00
7	2.05397E+00	1.04048E+01	0.00000E+00
8	2.87500E+00	1.09796E+01	0.00000E+00
9	3.78339E+00	1.14032E+01	0.00000E+00
10	4.75152E+00	1.16626E+01	0.00000E+00
11	5.75000E+00	1.17500E+01	0.00000E+00
12	6.74848E+00	1.16626E+01	0.00000E+00
13	7.71662E+00	1.14032E+01	0.00000E+00
14	8.62500E+00	1.09796E+01	0.00000E+00
15	9.44603E+00	1.04047E+01	0.00000E+00
16	1.01547E+01	9.69603E+00	0.00000E+00
17	1.07296E+01	8.87500E+00	0.00000E+00
18	1.11532E+01	7.96661E+00	0.00000E+00
19	1.14126E+01	6.99848E+00	0.00000E+00
20	1.15000E+01	6.00000E+00	0.00000E+00
21	1.15000E+01	0.00000E+00	0.00000E+00

BEAM PROPERTIES

Multiplier = 1 (For AISC database properties only)

Prop No	X-Section Area	Moment of Inertia Iy / Iz	Torsional Const.- J
---------	----------------	---------------------------	---------------------

KAISER ENGINEERS HANFORD S/N:801854 03/05/96
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CHECK GEOMETRY Version 2.0 07/01/90

Analysis of the lifting bail for the Pump Penetration Port P

Prop No	X-Section Area	Moment of Inertia Iy / Iz	Torsional Const.- J
1	4.000E+00	1.330E+00 1.330E+00	2.670E+00

Prop No	Max. Fiber Cy / Cz	Dist	Shear Shape Factor SSFy / SSFz	Ctors	Shear Stress Factor SSTRFy / SSTRFz
1	1.00E+00 1.00E+00	1.33E+00 1.33E+00		1.42E+00	1.50E+00 1.50E+00

BEAM CONNECTIVITY

Beam No	Nodes From To / Ref	Prop Mat No	pincode I J	Rigid End Offset I / J	Length	Beam Type
1	1 2 0 1	1		0.00E+00 0.00E+00	0.60E+01	Beam
2	2 3 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
3	3 4 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
4	4 5 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
5	5 6 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
6	6 7 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
7	7 8 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
8	8 9 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
9	9 10 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
10	10 11 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
11	11 12 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
12	12 13 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
13	13 14 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
14	14 15 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
15	15 16 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
16	16 17 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
17	17 18 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
18	18 19 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
19	19 20 0 1	1		0.00E+00 0.00E+00	0.10E+01	Beam
20	20 21 0 1	1		0.00E+00 0.00E+00	0.60E+01	Beam

RESTRAINTS

Node No	Global/Local	Restraint Directions
1	GLOBAL	X Y Z RX RY RZ

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03/05/96

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RUN ID=YD04897

08:49:50

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CHECK GEOMETRY

Version 2.0 07/01/90

Analysis of the lifting bail for the Pump Penetration Port P

Node No	Global/Local	Restraint Directions
21	GLOBAL	X Y Z RX RY RZ

KAISER ENGINEERS HANFORD S/N:801854

03/05/96

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Run ID=YD04897

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SOLVE DISPLACEMENTS

Version 2.0 07/01/90

Analysis of the lifting bail for the Pump Penetration Port P

LOAD CASE 1

LIFT LOAD

GRAVITY LOADING

Direction	Gravity Factor	Load Factor
X	.0000E+00	.1000E+01
Y	.0000E+00	.1000E+01
Z	.0000E+00	.1000E+01

INERTIA LOADING

Direction	Translational Accel.	Rotational Accel.	Rotational Velocity	Origin
X	.0000E+00	.0000E+00	.0000E+00	.0000E+00
Y	.0000E+00	.0000E+00	.0000E+00	.0000E+00
Z	.0000E+00	.0000E+00	.0000E+00	.0000E+00

Gravity Acceleration = .3864E+03

CONCENTRATED LOADS

Node	Fx	Fy	Fz	Mx	My	Mz
11	.0000E+00	.1000E+04	.0000E+00	.0000E+00	.0000E+00	.0000E+00

REFERENCE TEMPERATURE = .0000E+00

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SOLVE DISPLACEMENTS Version 2.0 07/01/90

Analysis of the lifting bail for the Pump Penetration Port P

LOAD CASE 1

LIFT LOAD

APPLIED LOAD VECTOR

Node	Fx	Fy	Fz	Mx	My	Mz
11	.0000E+00	.1000E+04	.0000E+00	.0000E+00	.0000E+00	.0000E+00

KAISER ENGINEERS HANFORD S/N:801854 03/05/96
PAGE 6 Run ID=YD04897 08:52:08

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SOLVE DISPLACEMENTS Version 2.0 07/01/90

Analysis of the lifting bail for the Pump Penetration Port P

LOAD CASE 1

LIFT LOAD

DISPLACEMENTS

Node	Translations			/	Rotations		
	X	Y	Z		X	Y	Z
1	.0000E+00	.0000E+00	.0000E+00	/	.0000E+00	.0000E+00	.0000E+00
2	.1501E-03	.2500E-04	.0000E+00	/	.0000E+00	.0000E+00	-.1279E-04
3	.1609E-03	.2836E-04	.0000E+00	/	.0000E+00	.0000E+00	-.2609E-06
4	.1557E-03	.3434E-04	.0000E+00	/	.0000E+00	.0000E+00	-.1556E-04
5	.1351E-03	.4880E-04	.0000E+00	/	.0000E+00	.0000E+00	-.3079E-04
6	.1035E-03	.7619E-04	.0000E+00	/	.0000E+00	.0000E+00	-.4364E-04
7	.6750E-04	.1178E-03	.0000E+00	/	.0000E+00	.0000E+00	-.5187E-04
8	.3476E-04	.1709E-03	.0000E+00	/	.0000E+00	.0000E+00	-.5338E-04
9	.1154E-04	.2281E-03	.0000E+00	/	.0000E+00	.0000E+00	-.4628E-04
10	.7204E-06	.2782E-03	.0000E+00	/	.0000E+00	.0000E+00	-.2892E-04
11	.8525E-09	.3077E-03	.0000E+00	/	.0000E+00	.0000E+00	-.3614E-11
12	-.7187E-06	.2782E-03	.0000E+00	/	.0000E+00	.0000E+00	-.2892E-04
13	-.1153E-04	.2281E-03	.0000E+00	/	.0000E+00	.0000E+00	-.4628E-04
14	-.3476E-04	.1709E-03	.0000E+00	/	.0000E+00	.0000E+00	-.5338E-04
15	-.6750E-04	.1178E-03	.0000E+00	/	.0000E+00	.0000E+00	-.5187E-04
16	-.1035E-03	.7619E-04	.0000E+00	/	.0000E+00	.0000E+00	-.4364E-04
17	-.1351E-03	.4880E-04	.0000E+00	/	.0000E+00	.0000E+00	-.3079E-04
18	-.1557E-03	.3434E-04	.0000E+00	/	.0000E+00	.0000E+00	-.1556E-04
19	-.1609E-03	.2836E-04	.0000E+00	/	.0000E+00	.0000E+00	-.2611E-06
20	-.1501E-03	.2500E-04	.0000E+00	/	.0000E+00	.0000E+00	-.1279E-04
21	.0000E+00	.0000E+00	.0000E+00	/	.0000E+00	.0000E+00	.0000E+00

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SOLVE BEAM LOADS/STRESSES Version 2.0 07/01/90

Analysis of the lifting bail for the Pump Penetration Port P

Load Case 1:LIFT LOAD

BEAM LOADS AND/OR STRESSES

GLoads	Node	Fx	Fy	Fz	Mx	My	Mz
Node	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Maximum	Minimum	Min. Shear			
BEAM NO. 1							
GLoads	1	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	.6223E+03
GLoads	2	-1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	.4521E+03
LLoads	1	-5000E+03	.0000E+00	-1791E+03	.0000E+00	.0000E+00	.6223E+03
LLoads	2	.5000E+03	.0000E+00	-1791E+03	.0000E+00	.0000E+00	.4521E+03
Stress	1	.1250E+03	.0000E+00	-.6715E+02	.0000E+00	-.4679E+03	.0000E+00
Stress	2	.5929E+03	.3429E+03	.2964E+03			
Stress	2	.1250E+03	.0000E+00	-.6715E+02	.0000E+00	.3399E+03	.0000E+00
Stress	2	.4649E+03	-.2149E+03	.2325E+03			
BEAM NO. 2							
GLoads	2	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.4521E+03
GLoads	3	-1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	-.5872E+03
LLoads	2	.5137E+03	.1348E+03	.0000E+00	.0000E+00	.0000E+00	-.4521E+03
LLoads	3	.5137E+03	.1348E+03	.0000E+00	.0000E+00	.0000E+00	-.5872E+03
Stress	2	.1284E+03	.5055E+02	.0000E+00	.0000E+00	.0000E+00	.3399E+03
Stress	2	.4684E+03	-.2115E+03	.2342E+03			
Stress	3	.1284E+03	.5055E+02	.0000E+00	.0000E+00	.0000E+00	.4415E+03
Stress	3	.5700E+03	-.3131E+03	.2850E+03			
BEAM NO. 3							
GLoads	3	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.5872E+03
GLoads	4	-1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	-.6309E+03
LLoads	3	.5293E+03	.4356E+02	.0000E+00	.0000E+00	.0000E+00	-.5872E+03
LLoads	4	.5293E+03	.4356E+02	.0000E+00	.0000E+00	.0000E+00	-.6309E+03
Stress	3	.1323E+03	.1633E+02	.0000E+00	.0000E+00	.0000E+00	.4415E+03
Stress	3	.5739E+03	-.3092E+03	.2869E+03			
Stress	4	.1323E+03	.1633E+02	.0000E+00	.0000E+00	.0000E+00	.4744E+03
Stress	4	.6067E+03	-.3420E+03	.3033E+03			
BEAM NO. 4							
GLoads	4	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.6309E+03
GLoads	5	-1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	-.5818E+03
LLoads	4	.5288E+03	.4902E+02	.0000E+00	.0000E+00	.0000E+00	-.6309E+03
LLoads	5	.5288E+03	.4902E+02	.0000E+00	.0000E+00	.0000E+00	-.5818E+03
Stress	4	.1322E+03	.1838E+02	.0000E+00	.0000E+00	.0000E+00	.4744E+03
Stress	4	.6066E+03	-.3422E+03	.3033E+03			
Stress	5	.1322E+03	.1838E+02	.0000E+00	.0000E+00	.0000E+00	.4374E+03
Stress	5	.5696E+03	-.3052E+03	.2848E+03			

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Analysis of the lifting bail for the Pump Penetration Port P

Load Case 1:LIFT LOAD

GLoads	Node	Fx	Fy	Fz	Mx	My	Mz
Node	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Maximum	Minimum	Min. Shear			
BEAM NO. 5							
GLoads	5	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.5818E+03
GLoads	6	-1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	-.4413E+03
LLoads	5	.5123E+03	-.1401E+03	.0000E+00	.0000E+00	.0000E+00	-.5818E+03
LLoads	6	.5123E+03	-.1401E+03	.0000E+00	.0000E+00	.0000E+00	-.4413E+03
Stress	5	.1281E+03	-.5254E+02	.0000E+00	.0000E+00	.0000E+00	.4374E+03
Stress	5	.5655E+03	-.3094E+03	.2827E+03			
Stress	6	.1281E+03	-.5254E+02	.0000E+00	.0000E+00	.0000E+00	.3318E+03
Stress	6	.4599E+03	-.2038E+03	.2300E+03			
BEAM NO. 6							
GLoads	6	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.4413E+03
GLoads	7	-1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	-.2139E+03
LLoads	6	.4802E+03	-.2269E+03	.0000E+00	.0000E+00	.0000E+00	-.4413E+03
LLoads	7	.4802E+03	-.2269E+03	.0000E+00	.0000E+00	.0000E+00	-.2139E+03
Stress	6	.1200E+03	-.8510E+02	.0000E+00	.0000E+00	.0000E+00	.3318E+03
Stress	6	.4519E+03	-.2118E+03	.2259E+03			
Stress	7	.1200E+03	-.8510E+02	.0000E+00	.0000E+00	.0000E+00	.1608E+03
Stress	7	.2809E+03	-.4078E+02	.1404E+03			
BEAM NO. 7							
GLoads	7	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.2139E+03
GLoads	8	-1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	-.9367E+02
LLoads	7	.4335E+03	-.3069E+03	.0000E+00	.0000E+00	.0000E+00	-.2139E+03
LLoads	8	.4335E+03	-.3069E+03	.0000E+00	.0000E+00	.0000E+00	-.9367E+02
Stress	7	.1084E+03	-.1151E+03	.0000E+00	.0000E+00	.0000E+00	.1608E+03
Stress	7	.2692E+03	-.5246E+02	.1346E+03			
Stress	8	.1084E+03	-.1151E+03	.0000E+00	.0000E+00	.0000E+00	-.7043E+02
Stress	8	.1788E+03	-.3794E+02	.8940E+02			
BEAM NO. 8							
GLoads	8	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.9367E+02
GLoads	9	-1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	-.4720E+03
LLoads	8	.3736E+03	-.3775E+03	.0000E+00	.0000E+00	.0000E+00	-.9367E+02
LLoads	9	.3736E+03	-.3775E+03	.0000E+00	.0000E+00	.0000E+00	-.4720E+03
Stress	8	.9340E+02	-.1416E+03	.0000E+00	.0000E+00	.0000E+00	-.7043E+02
Stress	8	.1638E+03	-.2297E+02	.8192E+02			
Stress	9	.9340E+02	-.1416E+03	.0000E+00	.0000E+00	.0000E+00	-.3549E+03
Stress	9	.4483E+03	-.2615E+03	.2241E+03			
BEAM NO. 9							
GLoads	9	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	.4720E+03

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Analysis of the lifting bail for the Pump Penetration Port P

Load Case 1:LIFT LOAD

Gloads	Node	Fx	Fy	Fz	Mx	My	Mz
Lloads	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Maximum	Minimum	Cmb. Shear			
Gloads	10	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.9096E+03
Lloads	9	-3024E+03	-4366E+03	.0000E+00	.0000E+00	.0000E+00	-.4720E+03
Lloads	10	-3024E+03	-4366E+03	.0000E+00	.0000E+00	.0000E+00	-.9096E+03
Stress	9	-7559E+02	-1637E+03	.0000E+00	.0000E+00	.0000E+00	-.3549E+03
Stress	9	-4305E+03	-2793E+03	.2152E+03			
Stress	10	-7559E+02	-1637E+03	.0000E+00	.0000E+00	.0000E+00	-.6839E+03
Stress	10	-7595E+03	-.6083E+03	.3798E+03			
BEAM NO. 10							
Gloads	10	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.9096E+03
Gloads	11	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.1393E+04
Lloads	10	-2220E+03	-4825E+03	.0000E+00	.0000E+00	.0000E+00	-.9096E+03
Lloads	11	-2220E+03	-4825E+03	.0000E+00	.0000E+00	.0000E+00	-.1393E+04
Stress	10	-5549E+02	-1809E+03	.0000E+00	.0000E+00	.0000E+00	-.6839E+03
Stress	10	-7394E+03	-.6284E+03	.3697E+03			
Stress	11	-5549E+02	-1809E+03	.0000E+00	.0000E+00	.0000E+00	-.1048E+04
Stress	11	-1103E+04	-.9920E+03	.5515E+03			
BEAM NO. 11							
Gloads	11	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.1393E+04
Gloads	12	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.9096E+03
Lloads	11	-2220E+03	-4825E+03	.0000E+00	.0000E+00	.0000E+00	-.9096E+03
Lloads	12	-2220E+03	-4825E+03	.0000E+00	.0000E+00	.0000E+00	-.9096E+03
Stress	11	-5549E+02	-1809E+03	.0000E+00	.0000E+00	.0000E+00	-.1048E+04
Stress	11	-1103E+04	-.9920E+03	.5515E+03			
Stress	12	-5549E+02	-1809E+03	.0000E+00	.0000E+00	.0000E+00	-.6839E+03
Stress	12	-7394E+03	-.6284E+03	.3697E+03			
BEAM NO. 12							
Gloads	12	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.9096E+03
Gloads	13	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.4720E+03
Lloads	12	-3024E+03	-4366E+03	.0000E+00	.0000E+00	.0000E+00	-.9096E+03
Lloads	13	-3024E+03	-4366E+03	.0000E+00	.0000E+00	.0000E+00	-.4720E+03
Stress	12	-7559E+02	-1637E+03	.0000E+00	.0000E+00	.0000E+00	-.6839E+03
Stress	12	-7595E+03	-.6083E+03	.3798E+03			
Stress	13	-7559E+02	-1637E+03	.0000E+00	.0000E+00	.0000E+00	-.3549E+03
Stress	13	-4305E+03	-2793E+03	.2152E+03			
BEAM NO. 13							
Gloads	13	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.4720E+03
Gloads	14	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.9367E+02
Lloads	13	-3736E+03	-3775E+03	.0000E+00	.0000E+00	.0000E+00	-.4720E+03

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SOLVE BEAM LOADS/STRESSES Version 2.0 07/01/90

Analysis of the lifting bail for the Pump Penetration Port P

Load Case 1:LIFT LOAD

Gloads	Node	Fx	Fy	Fz	Mx	My	Mz
Lloads	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Maximum	Minimum	Cmb. Shear			
Lloads	14	-3736E+03	-3775E+03	.0000E+00	.0000E+00	.0000E+00	-.9367E+02
Stress	13	-9340E+02	-1416E+03	.0000E+00	.0000E+00	.0000E+00	-.3549E+03
Stress	13	-4483E+03	-2615E+03	.2241E+03			
Stress	14	-9340E+02	-1416E+03	.0000E+00	.0000E+00	.0000E+00	-.7043E+02
Stress	14	-1638E+03	-.2297E+02	.8192E+02			
BEAM NO. 14							
Gloads	14	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.9367E+02
Gloads	15	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.2139E+03
Lloads	14	-4335E+03	-3069E+03	.0000E+00	.0000E+00	.0000E+00	-.9367E+02
Lloads	15	-4335E+03	-3069E+03	.0000E+00	.0000E+00	.0000E+00	-.2139E+03
Stress	14	-1084E+03	-1151E+03	.0000E+00	.0000E+00	.0000E+00	-.7043E+02
Stress	14	-1788E+03	-3794E+02	.8940E+02			
Stress	15	-1084E+03	-1151E+03	.0000E+00	.0000E+00	.0000E+00	-.1608E+03
Stress	15	-2692E+03	-.5246E+02	.1346E+03			
BEAM NO. 15							
Gloads	15	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.2139E+03
Gloads	16	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.4413E+03
Lloads	15	-4802E+03	-2269E+03	.0000E+00	.0000E+00	.0000E+00	-.2139E+03
Lloads	16	-4802E+03	-2269E+03	.0000E+00	.0000E+00	.0000E+00	-.4413E+03
Stress	15	-1200E+03	-8510E+02	.0000E+00	.0000E+00	.0000E+00	-.1608E+03
Stress	15	-2809E+03	-.4078E+02	.1404E+03			
Stress	16	-1200E+03	-8510E+02	.0000E+00	.0000E+00	.0000E+00	-.3318E+03
Stress	16	-4519E+03	-.2118E+03	.2259E+03			
BEAM NO. 16							
Gloads	16	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.4413E+03
Gloads	17	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.5818E+03
Lloads	16	-5123E+03	-1401E+03	.0000E+00	.0000E+00	.0000E+00	-.4413E+03
Lloads	17	-5123E+03	-1401E+03	.0000E+00	.0000E+00	.0000E+00	-.5818E+03
Stress	16	-1281E+03	-5254E+02	.0000E+00	.0000E+00	.0000E+00	-.3318E+03
Stress	16	-4599E+03	-.2038E+03	.2300E+03			
Stress	17	-1281E+03	-5254E+02	.0000E+00	.0000E+00	.0000E+00	-.4374E+03
Stress	17	-5655E+03	-.3094E+03	.2827E+03			
BEAM NO. 17							
Gloads	17	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.5818E+03
Gloads	18	-1791E+03	-5000E+03	.0000E+00	.0000E+00	.0000E+00	-.6309E+03
Lloads	17	-5288E+03	-.4902E+02	.0000E+00	.0000E+00	.0000E+00	-.5818E+03
Lloads	18	-5288E+03	-.4902E+02	.0000E+00	.0000E+00	.0000E+00	-.6309E+03
Stress	17	-1322E+03	-1838E+02	.0000E+00	.0000E+00	.0000E+00	-.4374E+03
Stress	17	-5696E+03	-.3052E+03	.2848E+03			

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Analysis of the lifting bail for the Pump Penetration Port P

Load Case 1:LIFT LOAD

GLoads	Node	Fx	Fy	Fz	Hx	Hy	Mz
LLoads	Node	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
Stress	Node	Maximum	Minimum	Cmb. Shear			
Stress	18	.1322E+03	.1838E+02	.0000E+00	.0000E+00	.0000E+00	.4744E+03
Stress	18	.6066E+03	.3422E+03	.3033E+03			
BEAM NO. 18							
GLoads	18	-.1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	-.6309E+03
LLoads	18	-.1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	.5872E+03
LLoads	19	-.5293E+03	-.4356E+02	.0000E+00	.0000E+00	.0000E+00	-.6309E+03
LLoads	19	.5293E+03	.4356E+02	.0000E+00	.0000E+00	.0000E+00	.5872E+03
Stress	18	.1323E+03	.1633E+02	.0000E+00	.0000E+00	.0000E+00	.4744E+03
Stress	18	.6067E+03	.3420E+03	.3033E+03			
Stress	19	.1323E+03	.1633E+02	.0000E+00	.0000E+00	.0000E+00	.4415E+03
Stress	19	.5739E+03	-.3092E+03	.2869E+03			
BEAM NO. 19							
GLoads	19	-.1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	-.5872E+03
LLoads	20	-.1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	.4521E+03
LLoads	19	.5137E+03	-.1348E+03	.0000E+00	.0000E+00	.0000E+00	-.5872E+03
LLoads	20	.5137E+03	.1348E+03	.0000E+00	.0000E+00	.0000E+00	.4521E+03
Stress	19	.1284E+03	.5055E+02	.0000E+00	.0000E+00	.0000E+00	.4415E+03
Stress	19	.5700E+03	-.3131E+03	.2850E+03			
Stress	20	.1284E+03	.5055E+02	.0000E+00	.0000E+00	.0000E+00	.3399E+03
Stress	20	.4684E+03	-.2115E+03	.2342E+03			
BEAM NO. 20							
GLoads	20	-.1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	-.4521E+03
LLoads	21	-.1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	.6223E+03
LLoads	20	.5000E+03	.0000E+00	-.1791E+03	.0000E+00	.0000E+00	.0000E+00
LLoads	21	.5000E+03	.0000E+00	-.1791E+03	.0000E+00	.6223E+03	.0000E+00
Stress	20	.1250E+03	.0000E+00	.6715E+02	.0000E+00	.3399E+03	.0000E+00
Stress	20	.4649E+03	-.2149E+03	.2325E+03			
Stress	21	.1250E+03	.0000E+00	.6715E+02	.0000E+00	-.4679E+03	.0000E+00
Stress	21	.5929E+03	-.3429E+03	.2964E+03			

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Analysis of the lifting bail for the Pump Penetration Port P

Load Case 1:LIFT LOAD

MAXIMUM STRESS SUMMARY FOR BEAMS/TRUSSES
WITHIN SPECIFIED RANGE 1- 20

Maximum (absolute) Stress = .1103E+04 at BEAM 11

Beam	Axial	Y-Shear	Z-Shear	Torsion	Y-Bending	Z-Bending
11	.5549E+02	.1809E+03	.0000E+00	.0000E+00	.0000E+00	-.1048E+04
	Maximum	Minimum	Cmb. Shear			
	.1103E+04	-.9920E+03	.5515E+03			

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SOLVE REACTIONS

Version 2.0 07/01/90

Analysis of the lifting bail for the Pump Penetration Port P

Load Case 1:LIFT LOAD

REACTIONS

Node	Fx	Fy	Fz	Mx	My	Mz
1	-.1791E+03	-.5000E+03	.0000E+00	.0000E+00	.0000E+00	.6223E+03
21	.1791E+03	.5000E+03	.0000E+00	.0000E+00	.0000E+00	-.6223E+03

W320-24-026

Seismic Shutdown System
Pad/Wind Missile Evaluation

KAISER ENGINEERS
HANFORD**CALCULATION IDENTIFICATION AND
INDEX**

Page i of iii

Date

10/5/94

This sheet shows the status and description of the attached Design Analysis sheets.

Discipline **Structural**WO/Job No. **ER4319/W-320**Calculation No. **W320-24-026**Project No. & Name **Project W-320, Tank 241-C-106 Sluicing**Calculation Item **Seismic Shutdown System Pad/Wind Missile Evaluation**

These calculations apply to:

Dwg. No. **H-2-818456**Rev. No. **0**

Dwg. No.

Rev. No.

Other (Study, CDR)

Rev. No.

The status of these calculations is:

☐ Preliminary Calculations☒ Final Calculations☐ Check Calculations (On Calculation Dated)☐ Void Calculation (Reason Voided)

Incorporated in Final Drawings?

☒ Yes ☐ No

This calculation verified by independent "check" calculations?

☐ Yes ☐ No

Original and Revised Calculation Approvals:

	Rev. 0 Signature/Date	Rev. 1 Signature/Date	Rev. 2 Signature/Date
Originator	James R. Booth/10-5-94 <i>James R. Booth</i>		
Checked by	<i>Y. S. Rupert</i> 10/1/94 <i>J. S. Rupert</i> 10/27/94		
Approved by	<i>Y. S. Rupert</i> 10/3/94		
Checked Against Approved Vendor Data	<i>Ann Langwin</i> 7/7/98		

Design Analysis
Page No.INDEX

Description

1-2	Objective, Criteria, Data, Assumptions, Method, References, and Conclusion.
3-9	Structural Pad
10-12	Material thickness calculation for missile protection for SC3 enclosure
13	Weld Calculation
Attachment 1	(H-2-818456) Seismic Shutdown System Concrete Pad - Plan & Sections
Attachment 2	Missile Shield Enclosure
Attachment 3	Vendor Information

DESIGN VERIFICATION SCREENING CRITERIA

ii of iii

Project/Document No. W-320 / W320-24-015

When the design or design change affects hardware, formal design verification must be performed if one or more of the following questions must be answered affirmatively (YES).

YES NO

☒

☐

1. Does the design or design change involved meet the established criteria to be considered Safety Class 1? SC1 strength requirement & SC 3/1 environmental enclosure considerations.

☐

☒

2. Does this design or design change cause or permit changes to Safety Class 1 instrument or alarm setpoints outside of previously approved operational limits?

☐

☒

3. Does this design or design change significantly affect the nuclear or environmental safety consequences of a malfunction or failure of the structure, system, or component?

☐

☒

4. Does this design or design change involve or change design that has previously undergone formal design verification?

Assigned Lead Engineer

11/1/94

Date

Responsible Discipline Manager

11/3/94

Date

Original Design Package Distribution:

Project Manager

Design Verification Officer

Engineering Document Control

Design Change Distribution:

Attach to Engineering Change Notice

Subject Calculation No. W320-24-015

Page iii of iii[illegible]

KAISER ENGINEERS
HANFORD

Calc. No. W320-24-026

Revision 0

Page No. 1 of 13

DESIGN ANALYSIS

client WHC

WO/Job No. ER4319/W-320

Subject Seismic Shutdown System Pad/Wind
Missile Evaluation

Date 7-25-94

By J.R. Booth *JRB*

Checked

By *Ren*

Location 200E

Revised

By

OBJECTIVE: The objective of this calculation is to design a structural concrete pad with embedded plates to secure a SC3 Missile Shield Enclosure. A Safety Class 1 level seismic strength requirement will be imposed on the design of the pad for the support of Safety Class 1 components enclosed within the Safety Class 3 Missile Shield Enclosure. Also, as part of this calculation the material thickness requirements of the enclosure to resist wind missile impact has been determined.

- CRITERIA:**
1. Functional Design Criteria for Tank 241-C-106 Waste Retrieval, Project W-320; Document No. WHC-W320-FDC-001, Rev.2.
 2. SDC 4.1, Rev. 11 & Rev. 12 for clarity of figures.
 3. SDC 4.2, Rev. 0.
 4. Topical Report: DESIGN OF STRUCTURES FOR MISSILE IMPACT. SEPT. 1974. Rev. 2.
 5. UCRL 15910, June 1990.

- DATA:**
1. $f'_c = 4000$ psi
 2. $f_y = 60$ ksi for rebar
 3. A36 steel for plates.
 4. Type H4L Nelson Weld Studs

ASSUMPTIONS: See body of calc.

METHOD: MATHCAD 4.0

- REFERENCES:**
1. American Concrete Institute (ACI). ACI 318-89 (Revised 1992).
 2. American Concrete Institute (ACI). ACI SP-17(91).
 3. American Concrete Institute (ACI). ACI 349-85.
 4. American Society of Civil Engineers (ASCE). ASCE 7-88.
 5. Final Report of Geotechnical Engineering Studies, W-320 Waste Retrieval and Sluicing System. Prepared by Shannon and Wilson, INC. April 1994.
 6. Standard ARCH-Civil Design Criteria, SDC-4.1. Design Loads for Facilities. Rev. 12.

KAISER ENGINEERS
HANFORD

Calc. No. W320-24-026

Revision 0

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DESIGN ANALYSIS

Client WHC

Subject Seismic Shutdown System Pad/Wind
Missile Evaluation

Location 200E

WO/Job No. ER4319/W-320

Date 7-25-94

Checked

Revised

By J. R. Booth

By

By

7. Standard ARCH-Civil Design Criteria, SDC-4.2. Design and Installation of Expansion Anchors, Rev. 0.
8. Uniform Building Code (UBC-91).
9. Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards. UCRL 15910, June 1990.
10. Ryerson Stock List. 1991.
11. Embedment Properties of Headed Studs. TRW/Nelson.

CONCLUSION: The concrete slab will be 8" thick and be reinforced with #5 rebar spaced 12" O.C. each way. A 3 1/2" dia. piece of conduit will be embedded in the concrete and stubbed up into the contactor boxes as shown in attachment 1. The embedded plates will be 1/2" thick A36 steel anchored with two 1/2" x 4 1/8" Nelson Studs. From the missile impact analysis it was determined that 12 gage sheet steel would be sufficient to withstand a wind blown missile. By inspection it was determined that the size of the HVAC unit mounted over top of any openings would be sufficient to stop a wind missile.

**ICF Kaiser Hanford
Company**
DOCUMENT TRANSMITTAL

Transmittal No.

TR-W-320-531

Date August 15, 1995

Page 1 of 1

To
Engineering Document ControlFrom
R.W. Davidson *RW*

Project/Work Order Number W-320/ER4319

Project/Work Order Title Tank 241-C-106 Sluicing

Subject Civil/Structural Calculations for C-Farm - Package 4

No. of
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Attached Are	Purpose	Comments	Please
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☒ Other: Calcs	☐	☐ Other:	☐

Document Numbers, Titles, and/or Comments

Calculations:

1. W320-24-014, Rev 0, Process Bldg. & Equip. Skid Fdn
2. W320-24-015, Rev 0, 241-C-106 Dome Load Summary Report
3. W320-24-021, Rev 0, Equipment Fdn-Exhaust Air Cleanup Train
4. W320-24-023, Rev 0, Equipment Fdn- Chiller & Pumps
5. W320-24-024, Rev 0, Electrical Equip. Pad, 241-C-51
6. W320-24-025, Rev 0, Cover Block Lifting Bail Calculation
7. W320-24-026, Rev 0, Seismic Shutdown System Pad/Wind Missile Evaluation
8. W320-24-027, Rev 0, Structural Analysis for Safety Related Equip. Attachments
9. W320-24-028, Rev 0, Engineering Evaluation of Commercial Grade Items

☐ Check here if receipt acknowledgment is required.

Receipt Acknowledgment:

Please sign below acknowledging receipt of documents listed above and return receipt to at .

Signature _____

Date _____

ICF Kaiser Hanford Company

INTEROFFICE MEMORANDUM

TO C. B. Organ E6-01
Design Verification Officer

DATE March 28, 1995

FROM G. L. Hopkins G3-17

COPIES TO J. R. Booth S3-07
R. W. Davidson S3-07
M. A. Haq G3-17
M. S. Ruben E6-46
Eng Doc Control G3-11

JOB NO. ER4319

SUBJECT STRUCTURAL ANALYSIS OF SYSTEM PAD/WIND MISSILE EVALUATION

REFERENCES 1. Design Calculation Number W320-24-026
2. H-2-818456, ESP Spec - Structural Seismic Shutdown System Long Pads

This memo and attached form KEH 1980.00 (06/91) are in response to your IOM of February 9, 1995, to perform a design verification for the subject project. This design verification is in accordance with "Design Review Method" as defined by ICF Kaiser (ICF KH) procedure ES 7, Rev. 3, Section 6.2.

Responses to the questions of Section 6.2.1.4 of procedure ES 7 are as follows:

- 1) Were the design inputs correctly selected?

YES This conclusion is based on the review of the following document: FDC
Rev 2 Tank 241-C-106 Sluicing, Definitive Design.

- 2) Were all necessary conditions considered in the design?

YES

- 3) Are required assumptions adequately described? Are sources identified reasonable for the design?

YES

- 4) Was appropriate design method used?

YES

- 5) Where necessary, are the assumptions reverified when the design is complete?

YES

WORKING TOWARD VPP "STAR" STATUS THROUGH EMPLOYEE INVOLVEMENT

INTEROFFICE MEMORANDUM

C. B. Organ

E6-01

2

March 27, 1995

- 6) Were the design inputs correctly incorporated?
YES
- 7) Is the design output reasonable compared to the design input?
YES
- 8) Are necessary design input and verification requirements for interfacing organizations specified in design documents or in supporting procedures or instructions?
YES
- 9) Was the applicable Hanford Plant Standards (HPS) followed? If not, is documented approval included from DOE-RL?
YES SDC 4.1, Rev. 12 was followed.

DESIGN VERIFICATION RECORD

Project/Document Number STRUCTURAL ANALYSIS OF SKYEMPAD/WIND
MISSILE EVALUATION ~ CALC # W320-24-026

Prepared By C. L. HOPKINS ; _____ ; _____
 (Print) (Print) (Print)

Verifier Name C. L. HOPKINS Initial CLH Date 7-27-95
 (Print)

Verifier Name _____ Initial _____ Date _____
 (Print)

Verifier Name _____ Initial _____ Date _____
 (Print)

Formal Design Verification is determined to be necessary per the Design Verification Screening Criteria and was performed by the following method(s).

- Alternate Calculation Method YES NO (Circle One)
 Calculation Number _____ Revision _____
- Testing Method YES NO (Circle One)
 Test Number _____ Revision _____
 Test Title _____
- Identical Method YES NO (Circle One)
 Document Number _____ Revision _____
 Title _____ Date _____
- Design Review Method YES NO (Circle One)
 Design Review Chairman _____
 Finding or Attached Memo Report Number SEE ATTACHED COPY OF 2-8-95
FROM G. L. HOPKINS TO CB. ORGAN
 Date 7-27-95

Formal design verification is complete and acceptable:

Design Verification Officer CB Date 7/27/95

Page B-9

ATTACH ALL DOCUMENTATION RELATED TO THE DESIGN VERIFICATION CONDUCTED.

**KAISER ENGINEERS
HANFORD**Calc. No. W320-24-026Revision 0Page No. 3 of 13**DESIGN ANALYSIS**Client: Westinghouse Hanford CompanyWO/Job No. ER4319/W-320Subject Structural Concrete PadDate 6/24/94By JR BoothChecked 10/27/94By J.S. RuppertLocation: 200ERevisedBy**DESIGN CONCRETE PAD**

Determine Design Loads:

1. Snow (REF. ASCE 7-88, pg. 23-27)

$$C_e := 0.9$$

$$C_t := 1.1$$

$$I := 1.2$$

$$p_g := 15 \frac{\text{lb}}{\text{ft}^2}$$

Ref. SDC 4.1

$$p_f := 0.7 \cdot C_e \cdot C_t \cdot I \cdot p_g$$

$$p_f = 12.474 \frac{\text{lb}}{\text{ft}^2} \quad \text{Use } p_f := 20 \frac{\text{lb}}{\text{ft}^2}$$

(Minimum Snow Load per SDC 4.1)

2. Wind: (Ref. SDC 4.1 and ASCE 7-88, section 6)

Dimensions: 6'-6" X 4'-0" X 6' high (See Attachment 2)

Exposure Class=C (SDC 4.1, pg.9)

Wind Speed=90 mph (SDC 4.1, pg.9)

Importance Factor=1.0 (SDC 4.1, pg.9)

$$I := 1.0$$

$$V := 90$$

$$K_z := 0.80$$

(Table 6 of ASCE 7-88)

$$q_z := 0.00256 \cdot K_z \cdot (I \cdot V)^2 \frac{\text{lb}}{\text{ft}^2}$$

$$q_z = 16.589 \frac{\text{lb}}{\text{ft}^2}$$

(Velocity pressure, Eq.3 of ASCE 7-88)

$$G_h := 1.32$$

(Gust response factor, Table 8 of ASCE 7-88)

$$C_f := 1.2$$

(Force Coeff. for Solid Signs, Table 13)

$$A_f := 6.5 \text{ ft} \cdot 6 \text{ ft}$$

(Projected area normal to wind in sq. ft)

$$W_w := q_z \cdot G_h \cdot C_f \cdot A_f$$

(Design Wind Force from Table 4, ASCE 7-88)

$$W_w = 1.025 \cdot 10^3 \text{ lbs}$$

Determine wind force on least dimension side (4'-0" x 6'-0")

$$A_{f2} := 4 \text{ ft} \cdot 6 \text{ ft}$$

$$W_{w2} := q_z \cdot G_h \cdot C_f \cdot A_{f2}$$

$$W_{w2} = 630.64 \text{ lb}$$

Note: Wind loading controls horizontal loading.

3. Ashfall Roof Loading

$$A := 24 \frac{\text{lb}}{\text{ft}^2}$$

(Per SDC 4.1, pg. 12)

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DESIGN ANALYSIS

HNF-2468, Rev.0
Calc. No. W320-24-026
Revision 0
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Client: Westinghouse Hanford Company

WO/Job No. ER4319/W-320

Subject Structural Concrete Pad

Date 6/24/94

By JR Booth

Checked

By W.D. JRB

Location: 200E

Revised

By

4. Live Floor Load

$L := 0 \cdot \text{lb}$

Live assumed zero, panels to be front accessible and enclosures are not designed for occupancy or any other equipment.

5. Seismic: Ref. SDC 4.1, UCRL 15910, UBC-91 and NUREG-0800.

An Equivalent Static Analysis will be used to determine base shear. Base shear will be applied 100% in two horizontal directions and 2/3 vertically.

Critical Damping = 7% for welded steel per UCRL 15910 Table 4-6.

Peak Acceleration = 0.38 g, this value will be multiplied by 1.5 for the static equivalent analysis.

a. Determine weight of components and metal enclosure.

$$w_{s12} := 4.375 \cdot \frac{\text{lb}}{\text{ft}^2} \quad (\text{Weight of 12 gage steel, Ryerson pg. 453})$$

$$SF := 2 \cdot (4 \cdot \text{ft} \cdot 6 \cdot \text{ft}) + 2 \cdot (6.5 \cdot \text{ft} \cdot 6 \cdot \text{ft}) + (4 \cdot \text{ft} \cdot 6.5 \cdot \text{ft}) \quad (\text{Square footage of enclosure sides})$$

$$W_{se} := w_{s12} \cdot SF \quad (\text{Weight of enclosure})$$

$$W_{se} = 665 \cdot \text{lb}$$

$$W_e := 15 \cdot \text{lb} + 150 \cdot \text{lb} \quad (\text{Weight of electrical equipment, 15 lb for seismic switch and 150 lb for electrical panel enclosure, see Attach 3})$$

$$W_t := W_{se} + W_e \quad (\text{Total weight of enclosure and equipment})$$

$$W_t = 830 \cdot \text{lb}$$

$$W_t := 900 \cdot \text{lb}$$

Round up for miscellaneous loads.

Design Base Shear

$$a := 0.38$$

$$E_{ssh} := a \cdot W_t \cdot 1.5 \quad (1.5 \text{ is the multiplier for static equivalent analysis})$$

$$E_{ssh} = 513 \cdot \text{lb} \quad (\text{Base shear applied in both horizontal directions 100\%})$$

Verticle force from seismic

$$E_{ssv} := \frac{2}{3} \cdot E_{ssh}$$

$$E_{ssv} = 342 \cdot \text{lb}$$

6. Dead Loads: Slab dimensions 15'-6" X 5'-6" X 0'-8"

$$W_c := 150 \cdot \frac{\text{lb}}{\text{ft}^3} \cdot \left(15.5 \cdot \text{ft} \cdot 5.5 \cdot \text{ft} \cdot \frac{8}{12} \cdot \text{ft} \right) \quad (\text{Weight of concrete slab})$$

$$W_c = 8.525 \cdot 10^3 \cdot \text{lb} \quad W_t = 900 \cdot \text{lb}$$

$$D := W_c + 2 \cdot W_t$$

$$D = 1.033 \cdot 10^4 \cdot \text{lb}$$

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HANFORD

DESIGN ANALYSIS

Calc. No. W320-24-026

Revision 0

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Client: Westinghouse Hanford Company

WO/Job No. ER4319/W-320

Subject: Structural Concrete Pad

Date 6/24/94

By JR Booth

Checked

By

Location: 200E

Revised

By

7. Load Combinations

- a. Determine worst case load case. (Ref. ACI 349, sec 9.2).

$$D_1 := 0\text{-lb}$$

(Meaningless adder to make MathCAD work)

$$L_f := (p_f + A) \cdot 15.5\text{ ft} \cdot 5.5\text{ ft}$$

Snow Load and Ashfall

$$L_f = 3.751 \cdot 10^3 \cdot \text{lbs}$$

Conservatively including snow load as a live load.

$$L_1 := 0\text{-lb}$$

(Meaningless adder to make MathCAD work)

$$F := 0\text{-lb}$$

Fluid Load

$$H := 0\text{-lb}$$

Soil Loads

$$R_o := 0\text{-lb}$$

$$Y_r := 0\text{-lb}$$

High-energy pipe break

$$R_a := 0\text{-lb}$$

$$Y_j := 0\text{-lb}$$

$$P_a := 0\text{-lb}$$

$$Y_m := 0\text{-lb}$$

$$T_a := 0\text{-lb}$$

$$T_o := 0\text{-lb}$$

$$E_o := 0\text{-lb}$$

OBE

$$W_t := 0\text{-lb}$$

Extreme wind condition

$$1. W_v := 1.4 \cdot D + 1.4 \cdot F + 1.7 \cdot L_f + 1.7 \cdot H + 1.7 \cdot R_o$$

$$W_v = 2.083 \cdot 10^4 \cdot \text{lb}$$

Controls

$$2. U := 1.4 \cdot D + 1.4 \cdot F + 1.7 \cdot L_f + 1.7 \cdot H + 1.7 \cdot E_o + 1.7 \cdot R_o$$

$$U = 2.083 \cdot 10^4 \cdot \text{lb}$$

$$3. U := 1.4 \cdot D_1 + 1.7 \cdot L_1 + 1.7 \cdot H + 1.7 \cdot (2 \cdot W_w) + 1.7 \cdot R_o$$

$$U = 3.484 \cdot 10^3 \cdot \text{lb}$$

$$4. U := D_1 + F + L_1 + H + T_o + R_o + E_{ssh}$$

$$U = 513 \cdot \text{lb}$$

$$U := D + F + L_f + H + T_o + R_o + E_{ssv}$$

$$U = 1.442 \cdot 10^4 \cdot \text{lb}$$

$$5. U := D_1 + F + L_1 + H + T_o + R_o + W_t$$

$$U = 0 \cdot \text{lb}$$

$$6. U := D + F + L_f + H + T_a + R_a + 1.25 \cdot P_a$$

$$U = 1.408 \cdot 10^4 \cdot \text{lb}$$

$$7. U := D + F + L_f + H + T_a + R_a + 1.15 \cdot P_a + 1 \cdot (Y_r + Y_j + Y_m) + 1.15 \cdot E_o$$

$$U = 1.408 \cdot 10^4 \cdot \text{lb}$$

KAISER ENGINEERS

HANFORD

DESIGN ANALYSIS

Calc. No. W320-24-026

Revision 0

Page No. 6 of 13

Client: Westinghouse Hanford Company

WO/Job No. ER4319/W-320

Subject Structural Concrete Pad

Date 6/24/94

By JR Booth

Checked

By

Location: 200E

Revised

By

$$8. U := D + F + L_f + H + T_a + R_a + 1.0 \cdot P_a + 1.0 \cdot (Y_r + Y_j + Y_m) + 1.0 \cdot E_{ssv}$$

$$U = 1.442 \cdot 10^4 \cdot \text{lb}$$

$$U := D_1 + F + L_1 + H + T_a + R_a + 1.0 \cdot P_a + 1.0 \cdot (Y_r + Y_j + Y_m) + 1.0 \cdot E_{ssh}$$

$$U = 513 \cdot \text{lb}$$

$$9. U := 1.05 \cdot D + 1.05 \cdot F + 1.3 \cdot L_f + 1.3 \cdot H + 1.05 \cdot T_o + 1.3 \cdot R_o$$

$$U = 1.572 \cdot 10^4 \cdot \text{lb}$$

$$10. U := 1.05 \cdot D + 1.05 \cdot F + 1.3 \cdot L_f + 1.3 \cdot H + 1.3 \cdot E_o + 1.05 \cdot T_o + 1.3 \cdot R_o$$

$$U = 1.572 \cdot 10^4 \cdot \text{lb}$$

$$11. U := 1.05 \cdot D_1 + 1.05 \cdot F + 1.3 \cdot L_1 + 1.3 \cdot H + 1.3 \cdot (2 \cdot W_w) + 1.05 \cdot T_o + 1.3 \cdot R_o$$

$$U = 2.664 \cdot 10^3 \cdot \text{lb}$$

b. UCRL 15910, pg. 5-10 recommended load combinations.

$$W_h := D_1 + 1.3 \cdot (2 \cdot W_w)$$

$$W_h = 2.664 \cdot 10^3 \cdot \text{lb}$$

Controls

$$W_{h2} := D_1 + 1.3 \cdot W_{w2}$$

$$W_{h2} = 819.832 \cdot \text{lb}$$

$$U := 1.1 \cdot D_1 + 0.5 \cdot L_1 + 1.2 \cdot (2 \cdot W_w)$$

$$U = 2.459 \cdot 10^3 \cdot \text{lb}$$

c. Controlling loading conditions:

$$W_v = 2.083 \cdot 10^4 \cdot \text{lb}$$

Controls vertical loading

$$W_h = 2.664 \cdot 10^3 \cdot \text{lb}$$

Controls horizontal loading in two directions.

8. Check Soil Bearing Properties

a. Check Qmax against Qapplied.

$$Q_{\max} := 4000 \cdot \frac{\text{lb}}{\text{ft}^2}$$

(Shannon and Wilson Report pg. 10, para. 2)

$$Q_{\text{applied}} := \frac{W_v}{15.5 \cdot \text{ft} \cdot 5.5 \cdot \text{ft}}$$

$$Q_{\text{applied}} = 244.36 \cdot \frac{\text{lb}}{\text{ft}^2}$$

Foundation Bearing pressure OK.

KAISER ENGINEERS

HANFORD

HNF-2468, Rev.0
 Calc. No. W320-24-026
 Revision 0
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DESIGN ANALYSIS

Client: Westinghouse Hanford Company

WO/Job No. ER4319/W-320

Subject: Structural Concrete Pad

Date: 6/24/94

By JR/Booth JRB

Checked

By

Location: 200E

Revised

By

b. Check longitudinal sliding.

$$F_f = 0.35 \cdot D$$

(Lateral Sliding coefficient given in UBC, pg. 590)

$$F_f = 3.614 \cdot 10^3 \cdot lb$$

Resistance to sliding by friction is sufficient in both directions.

9. Concrete Slab Design. Using the design procedures of ACI SP-17(91).

$$kip := 1000 \cdot lb$$

$$f_c := 4000 \text{ psi}$$

$$f_y := 60000 \text{ psi}$$

$$w_u := \left(\frac{Q_{\text{applied}} \cdot 1 \cdot ft^2}{1000 \cdot lb} \right) \cdot 1.6$$

(Bearing pressure as determined above and service factor)

$$w_u = 0.391 \frac{kip}{ft}$$

$$l := 2.25$$

Length of slab in ft. Assumes cantilever length from edge of enclosure to end of slab is half the slab width (conservative).

$$M_u := \frac{w_u \cdot l^2}{2}$$

$$M_u = 0.99 \text{ ft} \cdot kip$$

$$h := 8$$

(Slab thickness in inches)

$$d := h - 3 - 0.625 \quad d = 4.375$$

(Effective depth of slab with 3" min. cover for rebar)

$$b := 12$$

$$\rho_{\max} := 0.0214$$

SP17, Flexure 2.2

$$F := \frac{d^2}{1000} \quad F = 0.019$$

$$K_n := \frac{M_u}{F}$$

$$K_n = 51.705$$

$$K_n := 175$$

$$a_n := 4.37$$

SP17, Flexure 2.2 For grade 60 steel

$$A_s := \frac{M_u}{d \cdot a_n}$$

Area of steel required per 12" strip of concrete, sq. in

$$A_s = 0.052 \frac{in^2}{ft}$$

$$\rho_{\min} := 0.0018$$

For slabs per ACI 318, sec 7.12.2.1.

$$A_{smin} := \rho_{\min} \cdot b \cdot h$$

Controls

$$A_{smin} = 0.173 \text{ in}^2$$

DESIGN ANALYSIS

Client: Westinghouse Hanford Company

WO/Job No. ER4319/W-320

Subject Structural Concrete Pad

Date 6/24/94

By JR Booth *JRB*

Checked

By

Location: 200E

Revised

By

9. Continued:

Check shear

$$V_u := \frac{w_u \cdot l}{2}$$

$$V_u = 0.44 \text{ kips}$$

$$b_w := 12 \text{ in} \quad \phi := 0.85 \quad d := 9 \text{ in}$$

$$V_c := \phi \left(2 \sqrt{f_c} \right) b_w d$$

$$V_c = 1.161 \cdot 10^4 \text{ lb}$$

Ok

Use #5 bars spaced 12" O.C. As=0.31 sq. in. See H-2-818456 for sketch of reinforcing.

10. Check overturning moment

Figure 1

$$M_{ot} := W_h \cdot 3 \cdot \text{ft}$$

$$M_{ot} = 7.993 \cdot 10^3 \cdot \text{lbs} \cdot \text{ft}$$

$$M_r := D \cdot 2.75 \cdot \text{ft}$$

$$M_r = 2.839 \cdot 10^4 \cdot \text{lbs} \cdot \text{ft}$$

$$\frac{M_{ot}}{M_r} = 0.282$$

Overturning moment applied at the estimated mid-height of the enclosure.

Resisting moment to overturning

OK, since less than 2/3.

Client: Westinghouse Hanford Company

WO/Job No. ER4319/W-320

Subject: Structural Concrete Pad

Date 6/24/94

By JR Booth

Checked

By JOB

Location: 200E

Revised

By

11. Design embedded plates

Assume each embedded plate will see half the horizontal force and half the tension force created by the overturning moment. Design guide: Nelson Construction - Design Data. Try (2) 1/2 X 4 1/8 studs mounted to a 12" X 6" X 0'-0 1/2" embedded steel plate.

$$S_1 := \frac{W_h}{2} \cdot 2$$

Shear force in large surface direction times S.F. of 2.

$$S_1 = 2.664 \cdot 10^3 \cdot \text{lb}$$

$$S_2 := \frac{W_{h2}}{2} \cdot 2$$

Shear force in small surface direction times S.F. of 2.

$$S_2 = 819.832 \cdot \text{lb}$$

$$S_b := \sqrt{S_1^2 + S_2^2}$$

$$S_b = 2.788 \cdot 10^3 \cdot \text{lb}$$

$$T_b := \left(\frac{M_{ot}}{4.33 \cdot \text{ft}} \right) \cdot \frac{1}{2} \cdot 2$$

Tension force acting on plate times S.F. of 2

$$T_b = 1.846 \cdot 10^3 \cdot \text{lb}$$

$$S_u := \frac{S_b}{2}$$

$$S_u = 1.394 \cdot \text{kip}$$

Shear force per stud

$$T_u := \frac{T_b}{2}$$

$$T_u = 0.923 \cdot \text{kip}$$

Tension force per stud

$$D_e := 3 \cdot \text{in}$$

Minimum required distance from free edge to center of stud w/o reduction.

$$P_{uc} := 10.6 \cdot \text{kip}$$

Table No. 10, no reduction

$$S_{uc} := 10.33 \cdot \text{kip}$$

Table No. 18, no reduction

$$\left(\frac{T_u}{P_{uc}} \right)^{\frac{5}{3}} + \left(\frac{S_u}{S_{uc}} \right)^{\frac{5}{3}} = 0.053 \quad 0.049 \leq 1.0 \quad \text{OK}$$

Figure 2

Client: Westinghouse Hanford Company

WO/Job No. ER4319/W-320

Subject: Wind Missile Evaluation

Date 6/24/94

By JR Booth *JB*

Location: 200E

Checked 10/27/94

By J. B. Ruppert

MATERIAL THICKNESS FOR PROTECTION FROM MISSILE IMPACT

1. Assumptions:

- The worst case horizontal missile is given in SDC 4.1 as a 2 X 4 timber plank traveling at 50 mi/h weighing 15 lbs. Vertical missile velocity is 2/3 of horizontal.
- The missile has normal (direct) impact with wall resulting in the most conservative estimate.
- Only direct missile force with no other loads taken simultaneously are considered for structural response.
- Elasto-Plastic target response with no concurrent loading.
- One dimensional approximation.
- Plastic Impact (Coefficient of Restitution $e=0$)
- See Attachment 2 for target.

2. Calculate the Perforation Thickness:

T=Steel plate thickness

M=Mass of Missile

W := 15-lb

(Weight of Missile)

$$g := 32.2 \frac{\text{ft}}{\text{sec}^2}$$

(Acceleration due gravity)

$$M_m := \frac{W}{g}$$

$$M_m = 0.466 \cdot \text{lbs} \cdot \text{ft}^{-1} \cdot \text{sec}^2$$

$$V_s := 73.5 \frac{\text{ft}}{\text{sec}} \cdot 12 \frac{\text{in}}{\text{ft}}$$

(Striking velocity of missile)

$$A := 5.25 \cdot \text{in}^2$$

$$D := \sqrt{\frac{A}{\left(\frac{\pi}{4}\right)}}$$

$$D = 2.585 \cdot \text{in}$$

(Equivalent diameter of missile)

$$T := \frac{\left(\frac{M_m}{1 \cdot \text{lb} \cdot \text{ft}} \cdot V_s^2 \right)^{\frac{2}{3}}}{672 \cdot D} \cdot 1 \cdot \text{in}^2$$

(BRL Formula for Steel Plates, Bechtel Report eq. 2-7)

$$T = 0.067 \cdot \text{in}$$

$$t_s := 1.25 \cdot T$$

$$t_s = 0.084 \cdot \text{in}$$

(Increase thickness by 25% to prevent perforation, Bechtel Report eq. 2-8)

3. Determine Structural Response by Energy Balance Method.

a. Determine Effective Mass of Target During Impact.

$$D_x := 0.125 \cdot \text{ft}$$

(Lateral missile contact dimension)

$$D_y := 0.292 \cdot \text{ft}$$

(Vertical to ground)

$$\gamma_s := 492 \frac{\text{lb}}{\text{ft}^3}$$

(Weight per unit volume of steel)

$$t := 0.0087 \cdot \text{ft}$$

(Cold rolled 12 gage steel thickness = 0.1046")

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HANFORD

DESIGN ANALYSIS

Calc. No. W320-24-026Revision 0Page No. 11 of 13Client: Westinghouse Hanford CompanyWO/Job No. ER4319/W-320Subject: Wind Missile EvaluationDate 6/24/94By JR BoothChecked 10/27/94By f.a. RuppertLocation: 200E

Revised

By

3. Continued:

$$M_e := D_x \cdot D_y \left(\gamma_s \frac{t}{g} \right) \quad (\text{Average effective mass of target during impact, eq. 3-18})$$

$$M_e = 0.005 \cdot \text{lbs} \cdot \text{ft}^{-1} \cdot \text{sec}^2$$

b. Determine required target strain energy.

$$E_s := \frac{M_m^2 \cdot V_s^2}{2 \cdot (M_m + M_e)} \cdot 12 \frac{\text{in}}{\text{ft}} \quad (\text{EQ.3-8}) \quad \text{Multiplied by 12 in/ft to get units to work out.}$$

$$E_s = 1.494 \cdot 10^4 \cdot \text{lbs} \cdot \text{in}$$

c. Determine the required ductility ratio.

$$b := 1 \text{ in}$$

Use 1 inch wide strip to determine Mu, per Table 4-3.

$$d := 0.1046 \text{ in}$$

$$S := \frac{\left(\frac{1}{12} \right) \cdot b \cdot d^3}{\left(\frac{d}{2} \right)} \quad (\text{Section Modulus})$$

$$S = 0.002 \cdot \text{in}^3$$

$$f_y := 50000 \cdot \frac{\text{lb}}{\text{in}^2} \quad (\text{Yield Strength, A607(50)})$$

$$M_u := S \cdot f_y \cdot 1.2 \quad (\text{Dynamic increase factor of 1.2 per table 4-1})$$

$$M_u = 109.412 \cdot \text{lbs} \cdot \text{in} \quad (\text{Ultimate moment})$$

$$R_m := 2 \cdot \pi \cdot M_u \quad (\text{Resistance, Table 4-3 of Bechtel Report})$$

$$R_m = 687.453 \cdot \text{lbs} \cdot \text{in}$$

$$\alpha := 0.1518 \quad (\text{Height to Width ratio, where } a=6 \text{ ft and } b=6.5 \text{ ft per attachment 2})$$

$$E := 30 \cdot 10^6 \cdot \frac{\text{lb}}{\text{in}^2} \quad (\text{Modulus of elasticity})$$

$$I := \frac{b \cdot d^3}{12}$$

$$I = 9.537 \cdot 10^{-5} \cdot \text{in}^4 \quad (\text{Moment of inertia})$$

$$\nu := 0.3 \quad (\text{Poisson's ratio})$$

$$x_e := \left(\frac{\alpha \cdot R_m^2}{12 \cdot E \cdot I \cdot 1 \cdot \text{lb}} \right) \cdot (1 - \nu^2) \cdot \text{in}$$

$$x_e = 1.901 \cdot \text{in} \quad (\text{Displacement, Table 4-3})$$

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DESIGN ANALYSIS

HNF-2468, Rev.0
Calc. No. W320-24-026
Revision 0
Page No. 12 of 13

Client: Westinghouse Hanford Company

WO/Job No. ER4319/W-320

Subject Wind Missile Evaluation

Date 6/24/94

By JR Booth *JRB*

Checked 10/27/94

By f.a. Ruppert

Location: 200E

Revised

By

$$\mu_r := \left(\frac{E_s}{x_e R_m} + 0.5 \cdot \frac{1}{\text{in}} \right) \cdot \text{in} \quad \text{(Ductility ratio for steel needs to be less than 20 for the plate to survive a missile impact, see Table 4-4)}$$

$$\mu_r = 11.932$$

OK

Note: 12 gage, grade 50 steel will survive a wind missile impact. The equipment inside the enclosure are at a minimum of 6 inches away from the enclosure wall and will not be affected by the wall deflection.

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DESIGN ANALYSIS

Calc. No. W320-24-026Revision 0Page No. 13 of 13Client: Westinghouse Hanford CompanyWO/Job No. ER4319/W-320Subject: Weld CalculationDate 6/24/94By JR Booth

Checked

By

Location: 200E

Revised

By

DESIGN EQUIPMENT ANCHORING WELD

Attach enclosure to embedded plates with 4 - 8" long L 3 x 3 x 3/8 angles at base of enclosure. See Attachments 1 & 2.

$$a := 0.133 \text{ in}$$

Try 3/16" SMAW weld

$$R_{\text{total}} := \sqrt{S_b^2 + T_b^2}$$

SSRS of load acting on weld

$$R_{\text{total}} = 3.344 \cdot \text{kip}$$

$$F_{\text{EXX}} := 70 \cdot \frac{\text{kip}}{\text{in}^2}$$

E70XX weld electrodes

$$\phi R_{\text{nw}} := 0.75 \cdot 0.6 \cdot F_{\text{EXX}} \cdot a$$

Weld strength /in

$$\phi R_{\text{nw}} = 4.189 \cdot \frac{\text{kip}}{\text{in}}$$

$$F_u := 58 \cdot \frac{\text{kip}}{\text{in}^2}$$

Tensile strength of material

$$t := \frac{3}{8} \text{ in}$$

$$\phi R_{\text{ns}} := 0.75 \cdot 0.6 \cdot F_u \cdot t$$

Shear rupture strength of angle

$$\phi R_{\text{ns}} = 9.787 \cdot \frac{\text{kip}}{\text{in}}$$

Welds will be as shown on attachment 1.

2 CONNECTION DETAIL
SCALE: 3"=1'-0"

HOLD TABLE		
HOLD NO.	DESCRIPTION	EXPECTED RISE DATE
1	C-FARM - PGC 4	TBD

- NOTES:**
1. FOR CONDUIT TYPE, SIZE AND BEND INFORMATION SEE H-2-618679, SH.2.
 2. VERIFY BLOCKOUT & EMBEDMENT LOCATIONS W/ VENDOR INFORMATION/OBSERVATIONS PRIOR TO CONCRETE PLACEMENT.
 3. FIELD TO SEAL OR CALLK AS REQUIRED TO PROVIDE A WEATHERTIGHT SEAL BETWEEN ENCLOSURE AND CONCRETE BASE.

[illegible]

① CONNECTION DETAIL

ELEVATION

8
—
ELEVATION
NTS

ELEVATION

[illegible]

DON'T SAY IT --- Write It!

DATE: 6/28/94

TO: Jim Booth

FROM: Moe Khorsandian

Telephone: 373-2331

cc: Tim Kasnick
Juanito Del Rosario

SUBJECT: Seismic Switch Assembly Structure

1. Size of enclosure box for contactor assembly is 36" x 36" x 12". The box is typically designed for wall mounting application, but it can be modified for floor standing (option). This box will house the contactor, Seismic Switch Power Supply, Aux Relays and Misc Control Relays (up to 4) & instruments (see attached copy for more info for the enclosure box).
2. Weight of contactor itself is approximately 45 lb. Add 5 lb for SS Power Supply, $\approx$ 50 lb for enclosure box and misc loads. Total weight not to exceed $\approx$ 150 lb.
3. Clearance required for cable and conduit entrances: Preliminary design is to bring conduit from bottom straight through (underground conduit). Panels should have a clearance of at least 3 ft from the floor. Connection between two panels is straight line, no bending/clearance is required for panels interconnections.

Question: Have you considered the possibility of having a heavy metal enclosure cabinet, walk-in type, possibly with HVAC unit, light, etc. to be anchored on the concrete pad to serve as a missile barrier for seismic switch assembly instead of concrete building or pit ?

GENERAL DESCRIPTION

The TS-3/TS-3A consists of orthogonal acceleration transducers which are individually adjustable and are mounted in a rugged, watertight housing. As a triaxial seismic trigger for centrally-located accelerographs (SMA-3, DSA-3 or CRA-1), external starter (for the SMA-1, DSA-1 or SMA-2) or for low-level alarms, the set point of the TS-3/TS-3A is adjustable from 0.005g to 0.05g. As a seismic switch for remote alarms (for nuclear plant annunciation), the TS-3/TS-3A set point is adjustable from 0.025g to 0.25g. The detectors for the unit are small moving coils of 4 Hz natural frequency, and the seismic trigger version includes a filter which limits the flat response to the 1 Hz-10 Hz band. Energy outside this band is attenuated to preclude trigger response to non-earthquake events such as rotating equipment, blasts, vehicles, and industrial vibrations.

When seismic energy exceeds the set point on any axis, a relay in the TS-3/TS-3A housing closes, remains closed from 6 to 20 seconds (adjustable) after the acceleration falls below the threshold, and then resets automatically. On the TS-3, an MS connector on the housing links a remote 12 Vdc power supply and the user's remote accelerograph or alarm on the TS-3. On the TS-3A, the power supply consists of two internal, rechargeable 6V batteries in series. The components of the TS-3 can withstand up to 1×10^4 rads radiation. The set points in either unit may be tested or adjusted in the field with the FC-1 Field Calibrator (see Accessories).

TECHNICAL SPECIFICATIONS

Natural Frequency

4 Hz

Actuating Acceleration

0.005 g to 0.05 g, field adjustable with FC-1; optional range 0.025 g to 0.25 g

Damping

1.5 critical, provides flat response between 1 Hz and 10 Hz (optional damping 3.5 critical used for 0.025 g to 0.25 g range to provide flat response between 0.5 Hz and 15 Hz)

Output

Switch closure SPST: contact rating 10 watts for dc, or 12 VA for ac, 0.5 amp maximum

Closure Timing

On actuation the relay contacts close and stay closed for 6 to 20 seconds (adjustable) after motion falls below preset displacement threshold

Current Drain

0.45 mA standby, 60 mA operating from 12 Vdc

PHYSICAL CHARACTERISTICS

Dimensions

20.3 cm x 20.3 cm x 20.3 cm (8" x 8" x 8")

Weight

6.3 kg (14 lbs)

Operating Temperature

-20°C to +55°C (0°F to 130°F)

RECOMMENDED ACCESSORIES

1. FC-1 Field Calibrator, designed to adjust electromagnetic triggers to the desired actuation acceleration; powered by internal batteries.
2. SP-1 Seismic Switch Power Supply for use with TS-3 (SP-1 data sheet available).

ORDERING INFORMATION

1. Specify set point of actuating acceleration.

FC-1 Field Calibrator

W320-24-027

Structural Analysis for Safety
Related Equipment Attachments

KAISER ENGINEERS
HANFORD**CALCULATION IDENTIFICATION AND
INDEX**Page X of iii

Date

2/2/95

This sheet shows the status and description of the attached Design Analysis sheets.

Discipline **STRUCTURAL**WO/Job No. **ER4319/W-320**Calculation No. **W320-24-027**Project No. & Name **PROJECT W-320, TANK 241-C-106 SLUICING**Calculation item **STRUCTURAL ANALYSIS FOR SAFETY RELATED EQUIPMENT ATTACHMENTS**

These calculations apply to:

Dwg. No. **H-2-818498**Rev. No. **0**Dwg. No. **H-2-818499, SH. 1 & 2**Rev. No. **0**Other (Study, CDR) **N/A**Rev. No. **N/A**

The status of these calculations is:

- ☐ Preliminary Calculations
- ☒ Final Calculations
- ☐ Check Calculations (On Calculation Dated)
- ☐ Void Calculation (Reason Voided)

Incorporated in Final Drawings?

☒ Yes ☐ No

This calculation verified by independent "check" calculations?

☐ Yes ☒ No

Original and Revised Calculation Approvals:

	Rev. 0 Signature/Date	Rev. 1 Signature/Date	Rev. 2 Signature/Date
Originator	James B. Booth <i>James B. Booth 2/2/95</i>	Michael R. Carter <i>Michael R. Carter 1/2/95</i>	Michael B. Carter <i>Michael B. Carter 2/1/97</i>
Checked by	<i>Red D. Smith 7/2/95</i>	<i>Red D. Smith 1/24/96</i>	<i>Red D. Smith 2/12/97</i>
Approved by	<i>Red D. Smith 7/2/95</i>	<i>Red D. Smith 2/13/96</i>	<i>Red D. Smith 2/12/97</i>
Checked Against Approved Vendor Data			<i>Red D. Smith 3/31/97</i>

REVISION 1 : ADDED PAGES 11 AND 12. ADDED ATTACH. 3, REVISED PAGE NUMBERING ON PAGES 1 THRU 10.Design Analysis
Page No.**ADDED PAGES 13, 14 AND ATTACH. 4, REVISED PAGE NUMBERING**

Description

1	OBJECTIVE, CRITERIA, DATA, ASSUMPTIONS, METHOD, REFERENCES, AND CONCLUSIONS
2-6	EVALUATION OF SUPPORT FRAME & ATTACHMENT REQUIREMENTS OF THE COOLING COIL TO BE MOUNTED ON THE WALL OF SLUICE PIT 241-AY-02E
6-7	TYPICAL ELECTRICAL PANEL OVERTURNING/EXPANSION ANCHOR CHECK
8	SEISMIC BOLT SHEAR CALC OF BOLTS ATTACHING THE SLUICE DRIVE TO SLUICE RING
8-9	EXPANSION ANCHOR BOLT CALC OF SEISMIC RESTRAINT FOR PUMP IN PIT AY-02E
9-14	EXPANSION ANCHOR BOLT CALC FOR SEISMIC SHUTDOWN CONTACTOR BOX ATTACHMENT
ATTACH. 1	PIT COOLING COIL FAN ASSEMBLY
ATTACH. 2	PIT COOLING COIL FAN SUPPORT FRAME
ATTACH. 3	EQUIPMENT VENDOR CALC. (NUTHERM INTERNATIONAL INC., CALC. NO. 7455-03, REV. 0), 9 PAGES
ATTACH. 4	VENDOR DWG. W25255BA (AIR MONITOR CORP, SPEC. W-320-P41) / ECN W-320-365, SHT. 3

15-20

**EVALUATION OF CONNECTION BETWEEN CAMERA ASSEMBLY AND
MODIFIED FLANGE (R-7, 06A PIT)**

K&H 030.00 (06/92) K&H 072

Page C-1

Revision 2: Added pages 15 thru 20. Revised page numbering of pages 1 thru 14.

CALCULATION CROSS INDEX (Typical)

Subject Calculation No. W320-24-077

Page ii of iii

Subject Calculation Revision No.	Superseded by Calculation No.	These interfacing calculation/documents provide input to the subject calculation, and if revised may require revision of the subject calculation.		Results and conclusions of the subject calculation are used in these interfacing calculations and/or documents.		Does the output interface calculation/documents require revision?		Has the output interface calculation/documents been revised?		Discipline manager's signature and date indicating evaluation complete.
		Calculation/Document No.	Revision No.	Calculation/Document No.	Revision No.	Yes	No	Yes	No	
Ø	N/A	N/A	—	H-2-B18409	Ø		✓		✓	7/28/95
Ø	N/A	N/A	—	H-2-B18490, sh. 1 & 2	Ø		✓		✓	
Ø	N/A	N/A	—	H-2-B18702, sh. 1 & 2	Ø		✓		✓	
Ø	N/A	N/A	—	H-2-B18704, sh. 1-4	Ø		✓		✓	
Ø	N/A	N/A	—	H-2-B18462	Ø		✓		✓	
Ø	N/A	N/A	—	H-2-B18543, sh. 2	Ø		✓		✓	
Ø	N/A	N/A	—	H-2-B18690, sh. 3	Ø		✓		✓	
Ø	N/A	N/A	—	H-2-B18679, sh. 2	Ø		✓		✓	1/6/96
3	N/A	N/A	—	H-2-B18519, sh. 1	Ø		✓		✓	

DESIGN VERIFICATION SCREENING CRITERIA

 LV LV
 Page 111 of 111
Project/Document No. W-320/W320-24-027

When the design or design change affects hardware, formal design verification must be performed if one or more of the following questions must be answered affirmatively (YES).

YES

NO

- _____ X 1. Does the design or design change involved meet the established criteria to be considered Safety Class 1, ~~2~~ (Design involves anchorage/attachment to SC-1 designated seismic strength qualified structures). *Rev'd 7/28/95*
- _____ X 2. Does this design or design change cause or permit changes to Safety Class 1 or 2 instrument or alarm setpoints outside of previously approved operational limits?
- _____ X 3. Does this design or design change significantly affect the nuclear safety consequences of a malfunction or failure of the structure, system, or component?
- _____ X 4. Does this design or design change involve or change design that has previously undergone formal design verification?

[Signature]
 Assigned Lead Engineer

7/28/95
 Date

[Signature]
 Responsible Discipline Manager

7/28/95
 Date

Original Design Package Distribution:

Project Engineer
 Chief Design Engineer
 Engineering Document Control

Design Change Distribution:

Attach to Engineering Change Notice

KAISER ENGINEERS HANFORD

DESIGN ANALYSIS

HNF-2468, Rev.0

Calc. No. W320-24-027

Revision 8 X 2

Page No. 2 of 18 14 20

Client: Westinghouse Hanford Company

Subject: Structural Analysis for Safety Related

Equipment Attachments

Location: AY-Farm & C-Farm

WO/Job No. ER4319/W-320

Date 2/2/95

Checked 7/27/95

Revised 1/2/96

By JR Booth

By W.D. Sullivan

By M.R. Custer MRC

1. Evaluate the support frame and attachment requirements of the Cooling Coil to be mounted on the wall of the Sluice Pit (241-AY-02E).

a. Determine dead load.

$$D_1 := \left(5.4 \frac{\text{lb}}{\text{ft}} \right) \cdot ((4 \cdot 1.67 \text{ ft}) + (2 \cdot 1.65 \text{ ft}) + (4 \text{ ft}) + (2 \cdot 4.67 \text{ ft}) + (0.33 \text{ ft}))$$

$$D_1 = 127.71 \cdot \text{lb}$$

(C4 X 5.4 Channels, see pg. A-2)

$$D_2 := 300 \cdot \text{lb}$$

(Approx. weight of Pit Cooling Fan Coil and misc. piping. Ref. 6, sh. 1.)

$$D := D_1 + D_2$$

$$D = 427.71 \cdot \text{lb}$$

b. Seismic loading. Use a Equivalent Static Analysis as mentioned in Ref. 3 and analyze at safety class 1 seismic accelerations due to attachment of SC-3 support frame to existing SC-1 designated seismic strength qualified structures.

$$a := 0.42$$

(Peak acceleration from figure 3 of Ref. 4 at 5% critical damping)

$$f := 1.5$$

(Equivalent Static Factor from Ref. 3)

$$E_h := 1.5 \cdot a \cdot D$$

$$E_h = 269.457 \cdot \text{lb}$$

(Horizontal earthquake load)

$$E_v := E_h \cdot \frac{2}{3}$$

$$E_v = 179.638 \cdot \text{lb}$$

(Vertical earthquake load)

c. Evaluate the following load conditions from Ref. 2, Sec. A4.1.

$$L := 0 \cdot \text{lb}$$

(No live load)

Vertical

$$L_{v1} := 1.4 \cdot D$$

$$L_{v1} = 598.794 \cdot \text{lb}$$

$$L_{v2} := 1.2 \cdot D + 1.5 \cdot E_v + 0.5 \cdot L$$

$$L_{v2} = 782.709 \cdot \text{lb}$$

Controls vertical loading

$$L_{v3} := 0.9 \cdot D - 1.5 \cdot E_v$$

$$L_{v3} = 115.482 \cdot \text{lb}$$

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DESIGN ANALYSIS

HNF-2468, Rev.0
Calc. No. W320-24-027
Revision 812
Page No. 3 of 10 14 20

Client: Westinghouse Hanford Company

WO/Job No. ER4319/W-320

Subject: Structural Analysis for Safety Related

Date 2/2/95

By JR Booth

Equipment Attachments

Checked 7/27/96

By J. R. DeLoach

Location: AY-Farm & C-Farm

Revised 1/2/96

By M.R. CUSTER MRL

$$D_h := 0 \cdot \text{lb}$$

(Horizontal dead load)

Horizontal

$$L_{h1} := 1.2 \cdot D_h + 1.5 \cdot E_h + 0.5 \cdot L$$

$$L_{h1} = 404.186 \cdot \text{lb}$$

Controls horizontal loading

$$L_{h2} := 0.9 \cdot D_h - 1.5 \cdot E_h$$

$$L_{h2} = -404.186 \cdot \text{lb}$$

- d. Determine the ultimate moment in member A-C and ultimate axial force in member B-D. See page A-2, Detail 2.

$$\text{kip} := 1000 \cdot \text{lb}$$

$$x := 2 \cdot \text{lb}$$

guess

$$R_a := \text{root} \left[\left(\frac{14}{12} \right) \cdot \text{ft} \cdot x - \left(\frac{4}{12} \right) \cdot \text{ft} \cdot L_{v2}, x \right]$$

(Sum of the moments about point B)

$$R_a = 0.224 \cdot \text{kip}$$

(Reaction at point A)

$$z := 2 \cdot \text{lb}$$

guess

$$R_b := \text{root} (R_a - L_{v2} + z \cdot \sin(45 \cdot \text{deg}), z)$$

$$R_b = 0.791 \cdot \text{kip}$$

Axial force acting in member B-D

$$x1 := \left(\frac{4}{12} \right) \cdot \text{ft}$$

Short leg

$$x2 := \left(\frac{10}{12} \right) \cdot \text{ft}$$

Long leg

$$l := 1.17 \cdot \text{ft}$$

Beam length

$$M_u := \frac{L_{v2} \cdot x1 \cdot x2}{1}$$

$$M_u = 0.186 \cdot \text{kip} \cdot \text{ft}$$

Ultimate Moment

KAISER ENGINEERS HANFORD

DESIGN ANALYSIS

HNF-2468, Rev.0

Calc. No. W320-24-027

Revision 012

Page No. 4 of 18

Client: Westinghouse Hanford Company

WO/Job No. ER4319/W-320

Subject: Structural Analysis for Safety Related

Date 2/2/95

By JR Booth

Equipment Attachments

Checked 7/27/95

By Dean S. Sisson

Location: AY-Farm & C-Farm

Revised 1/2/96

By M.R. CUSTER

MRC

e. Check C 4 x 5.4 channels for required bending and axial strength.

Bending, Ref. 2 Beam Design Section

$$ksi := \frac{1000 \cdot lb}{in^2}$$

$$\phi_b := 0.9$$

(Strength reduction factor)

$$F_y := 36 \cdot ksi$$

(Yield Stress)

$$Z_y := 0.569 \cdot in^3$$

$$\phi M_n := \frac{\phi_b \cdot Z_y \cdot F_y}{12 \cdot \frac{in}{ft}}$$

$$\phi M_n = 1.536 \cdot kip \cdot ft$$

OK, since nominal bending moment greater than ultimate.

Axial, Ref. 2 chapter E pg. 6-39

$$K := 1$$

(Effective length factor, assuming pinned ends)

$$l := \left(\frac{1.65 \cdot ft}{1 \cdot ft} \right) \cdot 12 \cdot in$$

$$l = 19.8 \cdot in$$

(length of member)

$$r_y := 0.449 \cdot in$$

$$\frac{K \cdot l}{r_y} = 44.098$$

$$\phi F_{cr} := 27.63 \cdot ksi$$

(Design stress, Ref. 2 Table 3-36, pg. 6-124)

$$A_g := 1.59 \cdot in^2$$

$$\phi P_n := \phi F_{cr} \cdot A_g$$

$$\phi P_n = 43.932 \cdot kip$$

OK, design exceeds ultimate.

Shear, Ref. 2 Eq. F2.1, pg. 6-45

$$l := 1.17 \cdot ft$$

$$V_u := \frac{L \cdot v_2 \cdot x_2}{1}$$

$$V_u = 0.557 \cdot kip$$

$$\phi_v := 0.9$$

KAISER ENGINEERS HANFORD

DESIGN ANALYSIS

HNF-2468, Rev.0
Calc. No. W320-24-027
Revision 8 X 2
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Client: Westinghouse Hanford Company
Subject: Structural Analysis for Safety Related
Equipment Attachments
Location: AY-Farm & C-Farm

WO/Job No. ER4319/W-320
Date 2/2/95
Checked 1/27/95
Revised 1/2/96

By JB Booth
By W.D. Anderson
By M.R. CUSTER MRC

$$A_w := (2.625 \cdot \text{in}) \cdot \left(\frac{3}{16} \cdot \text{in}\right)$$

$$\phi V_n := \phi_v \cdot 0.6 \cdot F_y \cdot A_w$$

$$\phi V_n = 9.568 \cdot \text{kip}$$

OK, design shear strength is greater than the ultimate

- f. Determine strength of 3/16" weld per inch. (Ref. 2, Sec. J2.2)

$$\phi = 0.75$$

(Strength reduction factor)

$$t_e := 0.707 \cdot \frac{3}{16} \cdot \text{in}$$

$$F_{exx} := 70 \cdot \text{ksi}$$

$$\phi R_n := \phi \cdot t_e \cdot (0.6 \cdot F_{exx})$$

$$\phi R_n = 4.176 \cdot \frac{\text{kip}}{\text{in}}$$

It can be seen that a 3/16" weld will easily carry any loads

- g. Expansion Anchor Design. Per Ref. 1. Five Hilti-KWIK II bolts will be used to attach the frame to the wall.

$$V_{\text{applied}} := \frac{\sqrt{L_{h1}^2 + L_{v2}^2}}{4}$$

Shear force per bolt. Divide shear force among four of the five bolts.

$$V_{\text{applied}} = 220.227 \cdot \text{lb}$$

$$T_{ot} := \frac{L_{v2} \cdot \left(\frac{10}{12} \cdot \text{ft}\right)}{2 \cdot 1.5 \cdot \text{ft}}$$

Tension caused from overturning. Tension divided into 2 bolts.

$$T_{ot} = 217.419 \cdot \text{lb}$$

$$T_p := \frac{L_{h1}}{4}$$

$$T_p = 101.046 \cdot \text{lb}$$

$$T_{\text{applied}} := T_{ot} + T_p$$

Try 3/8" DIA. KB II w/ 1 5/8" Min. Embed. (Ref. 1, Table 11). Assumed concrete strength (f'c) is 3000psi.

$$V := 780 \cdot \text{lb} \quad (\text{Allowable})$$

$$T := 610 \cdot \text{lb} \quad (\text{Allowable})$$

$$\frac{V_{\text{applied}}}{V} + \frac{T_{\text{applied}}}{T} \leq 1.0$$

$$\frac{V_{\text{applied}}}{V} + \frac{T_{\text{applied}}}{T} = 0.804$$

ok

KAISER ENGINEERS HANFORD

DESIGN ANALYSIS

HNF-2400, Rev.0
Calc. No. W320-24-027
Revision 012
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Client: Westinghouse Hanford Company
Subject: Structural Analysis for Safety Related
Equipment Attachments
Location: AY-Farm & C-Farm

WO/Job No. ER4319/W-320
Date 2/2/95 By JR Booth
Checked 7/27/95 By R. R. Custer
Revised 1/2/96 By M. R. Custer

CONCLUSION: Cooling coil support frame shall be made of C 4 x 5.4 channels and welded together with 3/16" fillet welds. Anchorage to the pit wall shall be made with 3/8" DIA. HiTi KWIK II bolts. See drawings H-2-818498, sh. 1 and H-2-818499, sh.1 for details.

TYPICAL ELECTRICAL PANEL OVERTURNING/EXPANSION ANCHOR CHECK, SAFETY CLASS 3

1. Determine controlling lateral force.

$$D_1 := 100 \cdot \text{lb}$$

Weight of worst case Hoffman box

$$D_2 := 50 \cdot \text{lb}$$

Additional weight of breakers and cable

$$W_p := D_1 + D_2$$

$$W_p = 150 \cdot \text{lb}$$

a. Seismic

$$Z := 0.2$$

Seismic Acceleration, Ref. 7.

$$I := 1.25$$

Importance Factor, Ref. 4.

$$C_p := 2 \cdot (0.75)$$

Horizontal force factor per Ref. 7, Table 23-P. Factor multiplied by 2 per UBC sec. 2336(b).

$$F_p := Z \cdot I \cdot C_p \cdot W_p$$

Lateral seismic force per UBC sec. 2336(b)

$$F_p = 56.25 \cdot \text{lb}$$

$$F_{p2} := 0.3 \cdot F_p$$

$$E := \sqrt{F_p^2 + F_{p2}^2}$$

SSRS of two orthogonal directions

$$E = 58.727 \cdot \text{lb}$$

b. Wind

$$I := 1.07$$

Importance factor, Ref. 4, pg. 8

$$V := 70$$

Wind speed in mph, Ref. 4, pg.8

$$K_z := 0.8$$

Table 6 of Ref. 8

$$q_z := 0.00256 \cdot K_z \cdot (I \cdot V)^2 \frac{\text{lb}}{\text{ft}^2}$$

Velocity pressure, EQ.3 of Ref.8

$$G_h := 1.32$$

Gust response factor, Table 8 of Ref. 8

$$C_f := 1.2$$

Force Coeff. for Solid signs, Table 13 of Ref. 8

KAISER ENGINEERS HANFORD

DESIGN ANALYSIS

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Client: Westinghouse Hanford Company
Subject: Structural Analysis for Safety Related
Equipment Attachments
Location: AY-Farm & C-Farm

WO/Job No. ER4319/W-320
Date 3/27/95 By JR Booth
Checked 7/27/95 By W.D. Gunkles
Revised 1/2/96 By M.R. CUSTER MRC

$$A_f := 9 \text{ ft}^2$$

Conservative, assuming 3 ft x 3 ft panel

$$W := q_z \cdot G_h \cdot C_f \cdot A_f$$

$$W = 163.791 \text{ lb}$$

WIND FORCE CONTROLS!!!!

2. Check overturning moment. See H-2-818704, SH2 section A for case.

$$M_{ot} := W \cdot (5 \text{ ft})$$

$$M_{ot} = 818.957 \text{ lb-ft}$$

$$W_s := (3 \text{ ft} \cdot 3 \text{ ft} \cdot 0.67 \text{ ft}) \cdot 150 \cdot \frac{\text{lb}}{\text{ft}^3}$$

Weight of 3'-0" x 3'-0" x 0'-8" pad

$$W_s = 904.5 \text{ lb}$$

$$M_r := W_s \cdot 1.5 \text{ ft}$$

$$M_r = 1.357 \cdot 10^3 \text{ lb-ft}$$

$$\frac{M_{ot}}{M_r} = 0.604$$

OK!!

3. Determine expansion anchor requirements

Try 5/8" DIA. expansion anchors with 2 3/4" min embed

$$T_a := 2000 \text{ lb}$$

Allowables given are for HKB II bolt installed with special inspection. Ref. 9

$$V_a := 2625 \text{ lb}$$

$$T_{\text{applied}} := \frac{M_{ot}}{2 \cdot 0.5 \text{ ft}} \quad T_{\text{applied}} = 818.957 \text{ lb}$$

$$V_{\text{applied}} := \frac{W}{2} \quad V_{\text{applied}} = 81.896 \text{ lb}$$

$$\frac{V_{\text{applied}}}{V_a} + \frac{T_{\text{applied}}}{T_a} \leq 1.0$$

$$\frac{V_{\text{applied}}}{V_a} + \frac{T_{\text{applied}}}{T_a} = 0.441$$

OK!!

Conclusion: A 3 ft square by 8" deep pad will be sufficient for proposed electrical panels around C106 and 5/8" DIA expansion anchors are sufficient for unistrut base plates. unless otherwise stated. See Calc W320-24-029 for detailed structural analysis of safety related electrical equipment supports & anchorage/fdn req'mnts.

**KAISER ENGINEERS
HANFORD**

DESIGN ANALYSIS

HNH-2468, Rev.0
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Client: Westinghouse Hanford Company
Subject: Structural Analysis for Safety Related Equipment Attachments
Location: AY-Farm & C-Farm

WO/Job No. ER4319/W-320
Date 3/27/95 By JR Booth
Checked 7/27/95 By J.R. Booth
Revised 1/2/96 By M.R. CUSTER MRC

SEISMIC BOLT SHEAR CALC OF BOLTS ATTACHING THE SLUICER DRIVE ASSEMBLY TO THE SLUICE RING.
SAFETY CLASS 3

$$W_p := 1600 \cdot \text{lb}$$

Ref. H-2-818550, sh-5

Note: Because of the relatively low profile of the Sluice Drive versus weight, seismic will control horizontal loading

1. Combined loadin from Sluice Drive and calc W320-27-034.

$$Z := 0.2$$

Seismic Acceleration, Ref. 7.

$$I := 1.25$$

Importance factor, Ref.4.

$$C_p := 2 \cdot (0.75)$$

Horizontal force factor per Ref.7, Table 23-P. Factor multiplied by 2 per UBC sec. 2336(b).

$$F_p := Z \cdot I \cdot C_p \cdot W_p$$

Lateral seismic force per UBC sec. 2336(b).

$$F_p = 600 \cdot \text{lb}$$

$$F_{p2} := 0.3 \cdot F_p$$

$$E := \sqrt{F_p^2 + F_{p2}^2}$$

SSRS of two orthogonal directions

$$E = 626.418 \cdot \text{lb}$$

$$V := 1.27 \cdot \text{kip}$$

Combined seismic and operating loads from W320-27-034.

$$S := 1.5 \cdot E + V$$

1.5 is for equivalent staic analysis

2. Check 3/4" Dia. bolts ASTM A307, Grade B. Ref. H-2-818462, sh-1.

$$U := 1.5 \cdot S$$

LRFD load factor design.

$$U = 3.314 \cdot 10^3 \cdot \text{lb}$$

Factored seismic load acted as shear force on the bolts

$$\phi R_n := 7.16 \cdot \text{kip}$$

Ultimate design load of 3/4" Dia. bolts with threads included in the shear plain. Ref. 2, page 5-5.

$$\phi F_p := 78.3 \cdot \text{kip}$$

Ultimate desgn bearing strength of 1" thick A36 steel plate. Ref. 2, pg. 5-7.

Since the ultimate design loads of the bolt and connected parts are greater than the factored load, the bolt pattern for connecting the Sluice Drive Assembly adequately designed to resist seismic forces.

EXPANSION ANCHOR BOLT CALC OF SEISMIC RESTRAINT FOR PUMP IN PIT AY-02E

$$D_p := 3180 \cdot \text{lb}$$

Boosterpump jumper weight. Ref. DWG H-2-818503, sh-1.

1. Determine seismic forces acting on expansion anchors. Use a Equilvant Static Analysis as mentioned in Ref. 3 and analyze at Safety Class 1 seismic accelerations.

$$a := 0.42$$

(Peak acceleration from figure 3 of Ref. 4 at 5% critical damping)

$$f := 1.5$$

(Equivalent Static Factor from Ref. 3)

$$E_h := 1.5 \cdot a \cdot D_p$$

$$E_h = 2.003 \cdot 10^3 \cdot \text{lb}$$

(Horizontal earthquake load)

KAISER ENGINEERS HANFORD

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DESIGN ANALYSIS

Client: Westinghouse Hanford Company

WO/Job No. ER4319/W-320

Subject: Structural Analysis for Safety Related

Date 3/27/95

By JR Booth

Equipment Attachments

Checked 7/27/95

By DRD

Location: AY-Farm & C-Farm

Revised 1/2/96

By M.R. CUSTER MRC

$$E_v := E_h \cdot \frac{2}{3}$$

$$E_v = 1.336 \cdot 10^3 \cdot \text{lb}$$

(Vertical earthquake load)

2. Expansion Anchor Check. (4) Hilti Kwik-Bolt II 1/2" Dia. w/ 2 1/4" min embed will be used to restrain the pump to the pit wall (H-2-818543, sh-2). Assumed concrete strength (f'_c) is 3000psi. The seismic restraint that attaches the pump to the wall will not transfer a moment on to the plate shown on H-2-818543, sh-2 since there is no rigid connection to the wall plate.

$$V_{\text{applied}} := \frac{\sqrt{E_h^2 + E_v^2}}{4}$$

Shear force per bolt. Divide shear force among two bolts.

$$V_{\text{applied}} = 601.947 \cdot \text{lb}$$

$$T_{\text{applied}} := \frac{E_h}{4}$$

$$T_{\text{applied}} = 500.85 \cdot \text{lb}$$

$$V := 1390 \cdot \text{lb} \text{ (Allowable)}$$

$$T := 1000 \cdot \text{lb} \text{ (Allowable)}$$

Ref. 1, Table 11

$$\frac{V_{\text{applied}}}{V} + \frac{T_{\text{applied}}}{T} = 0.934$$

$$\frac{V_{\text{applied}}}{V} + \frac{T_{\text{applied}}}{T} \leq 1.0$$

ok

EXPANSION ANCHOR BOLT CALC. FOR SEISMIC SHUTDOWN SWITCH CONTACTOR BOX ATTACHMENT

The contactor box will be inside a missile shield enclosure; therefore, seismic loads will be evaluated. The contactor box is safety class 1.

1. Determine an estimated weight.

$$D_1 := 200 \cdot \text{lb}$$

Weight of worst case Hoffman box

$$D_2 := 100 \cdot \text{lb}$$

Additional weight of breakers and cable

$$W_t := D_1 + D_2$$

$$W_t = 300 \cdot \text{lb}$$

Total weight

1. Determine seismic forces acting on expansion anchors. Use a Equivalent Static Analysis as mentioned in Ref. 3 and analyze at Safety Class 1 seismic accelerations due to elevated SC-1 seismic strength qualification of foundation pad.

$$a := 0.42$$

(Peak acceleration from figure 3 of Ref. 4 at 5% critical damping)

$$f := 1.5$$

(Equivalent Static Factor from Ref. 3)

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HANFORD**

DESIGN ANALYSIS

HNF-2468, Rev.0

Calc. No. W320-24-027

Revision 0X2

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Client: Westinghouse Hanford Company

WO/Job No. ER4319/W-320

Subject: Structural Analysis for Safety Related

Date 3/27/95

By JR Booth

Equipment Attachments

Checked 7/27/95

By DRW

Location: AY-Farm & C-Farm

Revised 1/2/96

By M.R. CUSTER MRC

$$E_h := 1.5 \cdot a \cdot W_t$$

$$E_h = 189 \cdot \text{lb}$$

(Horizontal earthquake load)

$$E_v := E_h \cdot \frac{2}{3}$$

$$E_v = 126 \cdot \text{lb}$$

(Vertical earthquake load)

2. Determine overturning moment.

$$M_{ot} := E_h \cdot (3 \cdot \text{ft})$$

$$M_{ot} = 567 \cdot \text{lb} \cdot \text{ft}$$

3. Expansion Anchor Check. (4) Hilti Kwik-Bolt II 3/8" Dia. w/ 1 5/8" min embed will be used to attach the contactor box to the Seismic Shutdown Pad (H-2-818456). Concrete strength (f'c) is 4000psi.

$$T_a := 710 \cdot \text{lb}$$

Allowables given are for HKB II bolt installed with special inspection. Ref. 9

$$V_a := 875 \cdot \text{lb}$$

$$T_1 := \frac{M_{ot}}{2 \cdot 1 \cdot \text{ft}}$$

$$T_1 = 283.5 \cdot \text{lb}$$

$$T_{\text{applied}} := T_1 + \frac{E_v}{4}$$

$$T_{\text{applied}} = 315 \cdot \text{lb}$$

$$V_{\text{applied}} := \frac{E_h}{2}$$

$$V_{\text{applied}} = 94.5 \cdot \text{lb}$$

$$\frac{V_{\text{applied}}}{V_a} + \frac{T_{\text{applied}}}{T_a} \leq 1.0$$

$$\frac{V_{\text{applied}}}{V_a} + \frac{T_{\text{applied}}}{T_a} = 0.552$$

OK!!

NOTE :

REFER TO PAGES 11 AND 12 FOR REVISED ANCHOR CHECK, BASED ON THE VENDOR'S SUBMITTAL OF EQUIPMENT FOUNDATION LOADS PER ATTACH. 3.

DESIGN ANALYSIS

Calc. No. W320-24-027
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Client WESTINGHOUSE HANFORD COMPANY WO/Job No. ER4319/W-320
Subject STRUCTURAL ANALYSIS FOR SAFETY Date 12/22/95 By M. R. CUSTER / MRC
RELATED EQUIPMENT ATTACHMENTS Checked 1/24/96 By [Signature]
Location AY-FARM & C-FARM Revised By

RE-EVALUATE THE EXPANSION ANCHORS ATTACHING THE SEISMIC SHUTDOWN SYSTEM CABINETS (AY-SSS-1A AY-SSS-1B, C-SSS-1A AND C-SSS-1B) TO THE SEISMIC SHUTDOWN PAD.

REFER TO VENDOR CALCULATION NO. 7455-03, REV. 0 (NUTHERM INTERNATIONAL, INC.) AND PAGE 9 OF THIS CALCULATION.

THE ENVELOPING FORCES AND MOMENTS PER THE VENDOR ARE AS FOLLOWS:

DEAD LOADS:

$$F_x = 0$$

$$F_y = 546.5 \text{ lbs}$$

$$F_z = 0$$

$$M_x = 699 \text{ IN-lbs}$$

$$M_y = 0$$

$$M_z = 3634 \text{ IN-lbs}$$

DYNAMIC LOADS

$$F_x = 475.4 \text{ lbs}$$

$$F_y = 316.4 \text{ lbs}$$

$$F_z = 475.4 \text{ lbs}$$

$$M_x = 15606 \text{ IN-lbs}$$

$$M_y = 3713 \text{ IN-lbs}$$

$$M_z = 17346 \text{ IN-lbs}$$

DETERMINE TENSION (T) AND SHEAR (V) LOADS:

$$T_1 = 316.4 / 6 = 52.7 \text{ lb}$$

$$T_2 = M_x / d(2) = (699 + 15606) / 12(2) = 679.4 \text{ lb}$$

$$T_3 = M_z / d(3) = (3634 + 17346) / 24.25(3) = 288.4 \text{ lb}$$

$$V_1 = 475.4 / 6 = 79.2 \text{ lb}$$

$$V_2 = 475.4 / 6 = 79.2 \text{ lb}$$

$$V_3 = 3713 / 24.25(3) + 3713 / 12(2) = 205.8 \text{ lb}$$

$$T_A = 52.7 + 679.4 + 288.4 = 1020.5 \text{ lb}$$

DESIGN ANALYSIS

12/22/95
12/24/95

Calc. No. W320-24-077
Revision 1 2
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Client	WESTINGHOUSE HANFORD COMPANY		WO/Job No.	ER4319/W-320	
Subject	STRUCTURAL ANALYSIS FOR SAFETY		Date	12/22/95	By M.R. CUSTER / MRL
	RELATED EQUIPMENT ATTACHMENTS		Checked	12/24/95	By W. [Signature]
Location	AY-FARM & C-FARM		Revised		By

$$V_A = (79.2^2 + 79.2^2 + 205.8^2)^{1/2} = 234.1bs$$

CHECK UNITY :

$$\frac{V_A (APPLIED)}{V_A (ALLOWABLE)} + \frac{T_A (APPLIED)}{T_A (ALLOWABLE)} \leq 1.0$$

$$V_A (ALLOWABLE) \quad T_A (ALLOWABLE)$$

REFERENCE : ICBO REPORT NO. 4627, FEB. 1992 AND
ICF KAISER PROCEDURE GC-ANCR-01, REV.0

CHECK 1/2" Ø ANCHOR :

- MINIMUM EMBEDMENT = 2 1/4"
- $F_u = 4000$ PSI
- SPECIAL INSPECTION PROVIDED
- ANCHOR (CARBON STEEL HILTI KWIK-BOLT II)

ALLOWABLES : GC-ANCR-01 ICBO 4627

V_A 1470 LBS 1840 LBS

T_A 1360 LBS 1365 LBS

$$\frac{234}{1470} + \frac{1020}{1360} = 0.91 < 1.0 \dots \text{O.K.}$$

USE 1/2" Ø EXPANSION ANCHOR W/ 2 1/4" MIN.
EMBEDMENT W/ SPECIAL INSPECTION.

Final: Use 1/2" dia. Hilti Kwik-bolt II's, w/2-1/4" min emb for all SC 1
electrical equipment attachments to Seismic Shutdown Sys pads. Use a
minimum of (4) exp. bolts to anchor contactor boxes and a minimum of
(6) exp. bolts to anchor cabinets.

Calc. No. W320-24-027

Revision 2

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DESIGN ANALYSISClient **WHC**WO/Job No. **ER-6159/W-320**Subject **Structural Analysis for Safety Related Equipment Attachments**Date **06/20/96**By **M.R. Custer**Checked **6/20/96**By **[Signature]**Location **AY- Farm & C-Farm**

Revised

By

EVALUATION FOR CHANGES PER ECN W-320-365:

The following provides design evaluation of modifications to the equipment anchorage for the Instrumentation and Control Cabinet (reference vendor drawing W25255BA by "Air Monitor Corporation") per specification W-230-P41. The cabinet is mounted on the Exhaust Skid provided per specification W-320-P1 and is Safety Class 3.

The modifications involve providing (6)- 3 x 3 x 1/4" angles, 4" long. (3) angles positioned each side, along the short dimension of the cabinet.

DESIGN LOADS:

Dead Load, weight of the cabinet = 1100 lb

(Ref. Attachment 4)

Seismic Loads:

$$E_n-s = .375 W_p = .375 (1100) = 413 \text{ lb, use } 420 \text{ lb}$$

$$E_e-w = 1/3 (.375) W_p = 138 \text{ lb, use } 140 \text{ lb}$$

$$E_v = 0$$

Wind Load: (Use 20 psf x maximum projected area of cabinet)

$$\text{Maximum projected area, } A_{max} = 82"/12 \times 35.3"/12 = 20.1 \text{ sq ft}$$

$$\text{Maximum wind load, } P_w = 20 \text{ lb/sq ft} \times 20.1 \text{ sq ft} = 402 \text{ lb}$$

Determine the governing load combination:

The governing load combination will be determined based on the magnitude of the reactions at the base of the cabinet, resulting from the overturning moment(s).

Maximum Reaction due to Wind loading:

$$\text{Max. uplift force at the support} = [(82"/2 \times 402 \text{ lb}) + (1100 \text{ lb} \times 13.03")]/22.03" - 1100 \text{ lb}(.85) = 463 \text{ lb}$$

$$\text{Reaction/ connection} = 463 \text{ lb}/2 = 232 \text{ lb/ connection}$$

Maximum Reaction due to Earthquake loading:

$$\text{Max. uplift force at the support, } E_n-s = [(420 \text{ lb} \times 8") + (1100 \text{ lb} \times 13.03")]/22.03" - 1100 \text{ lb}(.85) = -132 \text{ lb}$$

Reaction = no uplift

$$\text{Max. uplift force at the support, } E_e-w = [(140 \text{ lb} \times 8") + (1100 \text{ lb} \times 14")]/35.3" - 1100 \text{ lb}(.85) = -467 \text{ lb}$$

Reaction = no uplift

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Revision ¹2

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DESIGN ANALYSISClient **WHC**WO/Job No. **ER-6159/W- 320**Subject **Structural Analysis for Safety Related Equipment Attachments**Date **06/20/96**By **M.R Custer** *MRC*Checked *6/24/96*By *Paul Johnston*Location **AY- Farm & C-Farm**

Revised

By

Maximum reaction due to wind loading, R = 232 lb, Load Combination D + W governs uplift.**W shear = $402/6 = 67$ lb/ connection****W uplift = 232 lb/ connection****SUMMARY OF FORCES AND MOMENTS: (Per connection)**

$$F_x = 232 \text{ lb}, \quad F_y = 0, \quad F_z = 67 \text{ lb} \quad F_r = \left[\left(\frac{232^2}{2} + \frac{67^2}{2} \right)^{1/2} \right] = 241 \text{ lb}$$

$$M_x = 0, \quad M_y = 232 \text{ lb (1.5')} = 348 \text{ in-lb}, \quad M_z = 0$$

$$f_w = F_x/A_w + M_y/S_wy + F_z/A_w, \quad A_w = 6, \quad S_wy = d^3/6 = 3$$

$$f_w = 232/6 + 348/3 + 67/6 = 166 \text{ lb/in}, \quad w = f_w/15800 = 0.010", \text{ min. weld} = 3/16", \text{ OK}$$

Check 1/2" bolt : $A_b = .196 \text{ in}^2$ (Based on the nominal bolt dia.)

$$f_b = .241 \text{ k}/.196 \text{ in}^2 = 1.2 \text{ ksi} < 10 \text{ ksi} \dots \text{ OK}$$

**FLUOR DANIEL
NORTHWEST INC.**

DESIGN ANALYSIS

Calc. No.: W320-24-027

Revision: 2

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Client: NHC

Subject: Structural Analysis of Safety Related

Equipment Attachments

Location: AY-Farm & C-Farm

WO/Job No.: ER4319/W 320

Date: 2/06/97

Checked: *2/12/97*

Revised:

By: MB Lasota

By: *[Signature]*

By:

TANK C-106,
RISER 7 & VIDEO SYSTEM

DESIGN ANALYSIS

Page No.: 16 of 20

By:

MODIFIED FLANGE FOR VIDEO SYSTEM

Client: NHC
Subject: Structural Analysis of Safety Related
Equipment Attachments.
Location: AY- Farm & C- Farm

WO/Job No.: ER4319/W320
Date: 2/10/1997
Checked: J. [Signature]
Revised:

By: MB Lasota [Signature]
By: [Signature]
By:

Evaluation of the connection between the Camera Assembly and modified flange of 12" Riser.

For an evaluation of the bolted flange of the Riser, reactions at the fixed end of Camera Assembly is required.
The following is a determination of the magnitude of the reaction.

MATH MODEL OF THE MAST COLOR
VIDEO SYSTEM

From the vendor calculations, the following are parameters used for evaluation of the mast:

$$g_h := 0.63 \quad i := 1..4$$

$$w_1 := 33.4 \cdot \text{lbf}$$

$$w_2 := 46.0 \cdot \text{lbf}$$

$$w_3 := 128.3 \cdot \text{lbf}$$

$$w_4 := 20.6 \cdot \text{lbf}$$

$$\gamma_{st} := 490 \cdot \text{pcf} \quad \gamma_w := 62.4 \cdot \text{pcf}$$

Mast consist of the steel tubing 3.5" OD and 2.9" ID, the weight of the mast is:

$$d_o := 3.5 \cdot \text{in} \quad d_i := 2.9 \cdot \text{in}$$

$$w_{mst} := \frac{\pi}{4} (d_o^2 - d_i^2) \cdot \gamma_{st} \quad w_{mst} = 10.26 \cdot \text{plf}$$

Attached to the mast are 1/2" tube filled with water and cables which estimated weight is 20 lbf.

$$d_{ot} := 0.5 \cdot \text{in} \quad d_{it} := (0.5 - 2 \cdot 0.035) \cdot \text{in}$$

$$w_t := \frac{\pi}{4} (d_{ot}^2 - d_{it}^2) \cdot \gamma_{st} + \frac{\pi}{4} d_{it}^2 \cdot \gamma_w \quad w_t = 0.24 \cdot \text{plf}$$

$$w_c := 20 \cdot \text{lbf}$$

The combined weight of the shielding plate, upper junction box and flange is:

$$w_{upp} := 110 \cdot \text{lbf} + 90.9 \cdot \text{lbf} \quad w_{upp} = 200.9 \cdot \text{lbf}$$

The combined shear and moment at the fixed end of the mast are:

$$l_1 := 245.2 \cdot \text{in} \quad l_2 := 235.7 \cdot \text{in} \quad l_3 := 213.5 \cdot \text{in} \quad l_4 := 192.5 \cdot \text{in}$$

$$V_{upp} = w_{upp} \cdot g_h \quad V_l := \left[(w_{mst} + w_t + \frac{w_c}{l_1}) \cdot l_1 + \sum_i w_i \cdot g_h \right]$$

$$V = V_{upp} + V_l \quad V = 418.2 \cdot \text{lbf}$$

DESIGN ANALYSIS

Calc. No.: W320-24-027

Revision: 2

Page No. 18 of 20

Client: NHC

Subject: Structural Analysis of Safety Related
Equipment Attachments.

Location: AY- Farm & C- Farm

WO/Job No.: ER4319/W320

Date: 2/10/1997

Checked: 2/12/97

Revised:

By: MB Lasota

By: [Signature]

$$M := \left[\sum_i w_i l_i + \left(w_{mst} + w_t + \frac{w_c}{l_1} \right) \cdot \frac{(l_1)^2}{2} \right] \cdot g \cdot h \quad M = 49.86 \cdot \text{in} \cdot \text{kips}$$

$$M_4 := \left[\sum_i w_i (l_1 - l_4) + \left(w_{mst} + w_t + \frac{w_c}{l_1} \right) \cdot \frac{(l_1 - l_4)^2}{2} \right] \cdot g \cdot h \quad M_4 = 4.9 \cdot \text{in} \cdot \text{kips}$$

$$V_4 := \left[\sum_i w_i + \left(w_{mst} + w_t + \frac{w_c}{l_1} \right) \cdot (l_1 - l_4) \right] \cdot g \cdot h \quad V_4 = 175.6 \cdot \text{lb} \cdot \text{f}$$

Check deflection of the mast at the lower junction box.

Actual bending moment as a function of a variable in vertical direction (y) is:

$$M_{\alpha}(y) := M_4 + V_4 \cdot y + \left(w_{mst} + w_t + \frac{w_c}{l_1} \right) \cdot \frac{y^2}{2} \quad M_{\alpha}(0 \cdot \text{ft}) = 4.9 \cdot \text{in} \cdot \text{kips} \quad M_{\alpha}(l_4) = 56.42 \cdot \text{in} \cdot \text{kips}$$

and virtual moment as a function of y is:

$$m_{\alpha}(y) := y$$

Sectional properties of the mast are:

$$I := \frac{\pi}{64} (d_o^4 - d_i^4) \quad I = 3.89 \cdot \text{in}^4 \quad E_s := 29000 \cdot \text{ksi}$$

Deflection at the lower junction box is: $l_4 = 192.5 \cdot \text{in}$

$$\delta_{ljb} := \frac{g \cdot h}{E_s \cdot I} \int_{0 \cdot \text{in}}^{192.5 \cdot \text{in}} M_{\alpha}(y) \cdot m_{\alpha}(y) \, dy \quad \delta_{ljb} = 3.75 \cdot \text{in}$$

Above found deflection is in excess of the gap provided at the lower junction box. As a consequence the junction box will travel to close provided gap and will impose a reaction at the Riser. Let the reaction be x.

The gap provided is:

$$\Delta_{\min} := 0.64 \cdot \text{in} \quad \Delta_{\max} := 1.096 \cdot \text{in}$$

$$Q := \frac{g \cdot h}{E_s \cdot I} \int_{0 \cdot \text{in}}^{192.5 \cdot \text{in}} m_{\alpha}(y)^2 \, dy \quad Q = 0.0133 \cdot \frac{\text{in}}{\text{lb} \cdot \text{f}}$$

$$\delta_{ljb}(x) = Q \cdot x - (\delta_{ljb} + \Delta_{\min}) \quad \text{let} \quad \delta_{ljb}(x) = 0 \cdot \text{in} \quad \text{thus}$$

$$x_1 := \frac{\delta_{ljb} - \Delta_{\min}}{Q} \quad x_1 = 234.5 \cdot \text{lb} \cdot \text{f} \quad \text{Similarly, for max gap:}$$

$$x_2 := \frac{\delta_{ljb} - \Delta_{\max}}{Q} \quad x_2 = 200.2 \cdot \text{lb} \cdot \text{f}$$

Client: NHC
 Subject: Structural Analysis of Safety Related
 Equipment Attachments.
 Location: AY- Farm & C- Farm

WO/Job No.: ER4319/W320
 Date: 2/10/1997
 Checked: 2/12/97
 Revised:

By: M. Laspra
 By: R. D. Smith

Adjusted reaction at the flange when the minimum gap is closed is:

$$V_1 := (V_{upp} + V_l) - x_1 \quad V_1 = 183.6 \cdot \text{lb}f$$

$$M_1 := \left[\sum_i w_i \cdot l_i + \left(w_{mst} + w_t + \frac{w_c}{l_1} \right) \cdot \frac{(l_1)^2}{2} \right] \cdot g_h - x_1 \cdot l_4 \quad M_1 = 4.71 \cdot \text{in} \cdot \text{kips}$$

Adjusted reaction at the flange when the maximum gap is closed is:

$$V_2 := (V_{upp} + V_l) - x_2 \quad V_2 = 218 \cdot \text{lb}f$$

$$M_2 := \left[\sum_i w_i \cdot l_i + \left(w_{mst} + w_t + \frac{w_c}{l_1} \right) \cdot \frac{(l_1)^2}{2} \right] \cdot g_h - x_2 \cdot l_4 \quad M_2 = 11.33 \cdot \text{in} \cdot \text{kips}$$

Since the Web Protrusions, when subjected to the load other than radial will exhibit relatively low stiffness due to plate bending, the reactions at the upper flange will be taken as the envelope of all boundary conditions considered.

$$\begin{aligned} V &= 418 \cdot \text{lb}f & V_1 &= 184 \cdot \text{lb}f & V_2 &= 218 \cdot \text{lb}f \\ M &= 50 \cdot \text{in} \cdot \text{kips} & M_1 &= 5 \cdot \text{in} \cdot \text{kips} & M_2 &= 11 \cdot \text{in} \cdot \text{kips} \end{aligned}$$

Properties of the bolted flange connection.

Assume the critical axis to be rotated 45 degrees from E-W direction. Sectional properties of the bolt group is calculated as follows:

$$\begin{aligned} \kappa &:= 1, 2, \dots, 12 \\ \phi_0 &:= 30 \cdot \text{deg} \end{aligned}$$

$$\phi_\kappa := (\kappa - 1) \cdot \phi_0$$

$$A_b := 0.462 \cdot \text{in}^2$$

Determine location of the center of gravity of the bolt group:

$$r_b := 8.5 \cdot \text{in}$$

$$x_\kappa := r_b \cdot \cos(\phi_\kappa) \quad y_\kappa := r_b \cdot \sin(\phi_\kappa)$$

$$Q_x := \sum_\kappa A_b \cdot y_\kappa - A_b \cdot (y_8 + y_9 + y_{11} + y_{12}) \quad Q_x = 10.73 \cdot \text{in}^3$$

$$Q_y := \sum_\kappa A_b \cdot x_\kappa - A_b \cdot (x_8 + x_9 + x_{11} + x_{12}) \quad Q_y = 0 \cdot \text{in}^3$$

$$A_z := 8 \cdot A_b \quad A_z = 3.7 \cdot \text{in}^2$$

Client: NHC
Subject: Structural Analysis of Safety Related
Equipment Attachments.
Location: AY- Farm & C- Farm

WO/Job No.: ER4319/W320
Date: 2/11/1997
Checked: *2/12/97*
Revised:

By: MB Lasota
By: *ReSutter*

$$x_o := \frac{Q_y}{A_z} \quad x_o = 0 \cdot \text{in}$$

$$y_o := \frac{Q_x}{A_z} \quad y_o = 2.9 \cdot \text{in}$$

$$I_x := \sum_k A_b \cdot (y_k - y_o)^2 - A_b \cdot [(y_8 - y_o)^2 + (y_9 - y_o)^2 + (y_{11} - y_o)^2 + (y_{12} - y_o)^2] \quad I_x = 102.37 \cdot \text{in}^4$$

$$I_y := \sum_k A_b \cdot (x_k - x_o)^2 - A_b \cdot [(x_8 - x_o)^2 + (x_9 - x_o)^2 + (x_{11} - x_o)^2 + (x_{12} - x_o)^2] \quad I_y = 133.52 \cdot \text{in}^4$$

Conservatively, ignoring the gravity loads the forces in critical bolt (10) are:

$$T_b := \frac{M \cdot (|y_{10}| + y_o)}{I_x} \cdot A_b \quad T_b = 2566 \cdot \text{lbf}$$

$$V_b := \frac{V}{A_z} \cdot A_b \quad V_b = 52 \cdot \text{lbf}$$

Check tension in the anchor (1), when earthquake direction is in North-East direction:

$$T_b := \frac{M \cdot (|x_1 + x_o|)}{I_y} \cdot A_b \quad T_b = 1466 \cdot \text{lbf}$$

As shown, the assumption that the bolt 10 is critical holds true.

The magnitude of the maximum forces in 7/8 inch diameter bolt is a fraction of its capacity (ASTM A-307 Gr B bolt $T_{all}=12$ kips, and $V_{all}=6$ kips), it is therefore safe to conclude, that the connection will perform satisfactory during postulated earthquake.

Fluor Daniel Northwest

Calc. No. W-320-24-027

Revision 3

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DESIGN ANALYSIS

client Numatec

WO/Job No. W-320

Subject Structural Analysis for Safety Related
Equipment Attachments

Date 12/29/97

By A.S. Langevin *all*

Checked 1/7/98

By *Michael K. Curtis*

Location AY-Farm & C-Farm

Revised

By

C-105 Air Breather Filter

Analyze the C-105 Air breather filter support.

This equipment is added by ECN W-320-691

Equipment is Safety Class per HNF-1559 Rev 2 and HNF-SD-WM-SEL-040 Rev 0 (references 10 & 11).

A description of the SC class components is provided in the TWRS SEL (reference 11) which shows the supports to be GS. Design the supports for SC load criteria but otherwise as GS. (conservative)

Use load criteria from SDC 4.1 Rev 12 (reference 13) in accordance with the latest revision of the FDC, Rev 4 (reference 12).

DESIGN ANALYSIS

Client	<u>Numatec</u>	WO/Job No.	<u>W-320</u>
Subject	<u>Structural Analysis for</u>	Date	<u>1/5/98</u>
	<u>Safety Related Equip. Attach.</u>	By	<u>Ann Handelin</u>
Location	<u>AY-Farm & C-Farm</u>	Checked	<u>1/7/98</u>
		By	<u>Michael R. Carter</u>
		Revised	
		By	

Evaluate air-breather filter support

Dead load

Housing (includes filter and valve) = 150 lb
per D. Evans

Frame:

$$2'' \text{ pipe } [(2 \times 2') + (4 \times 5') + (4 \times 0.5')] \frac{3.65 \text{ lb/ft}}{4} = 95 \text{ lb}$$

$$\frac{3}{8}'' \text{ bar } [(2 \times 2.75') + (2 \times 2')] \frac{2.55 \text{ lb/ft}}{4} = 25 \text{ lb}$$

$$\text{TOTAL} = 270 \text{ lb}$$

$$\text{TOTAL FRAME} = 120 \text{ lb}$$

Seismic: Use equivalent static as per Ref 3

Use Seismic for Non-Reactor SC/11 per figure 3 of Ref 13.

$$\text{Peak accel at 5\% damping} = 0.42 H$$

$$\frac{2}{3}(0.42) = 0.28 V$$

$$\text{Housing } E_{H1} = 1.5(0.42)(150 \text{ lb}) = 95 \text{ lb}$$

$$\text{Frame } E_{H2} = 1.5(0.42)(120 \text{ lb}) = 76 \text{ lb}$$

$$\text{Housing \& Frame } E_V = 1.5(0.28)(270) = 113 \text{ lb}$$

DESIGN ANALYSIS

Client Numatec

WO/Job No. W-320

Subject Structural Analysis for

Date 1/5/98

By Ann Hangerman

Safety Related Equipment Attach.

Checked 1/7/98

By Michael R. Quisenberry

Location At-farm 80 C-farm

Revised

By

Wind: - use ref 8. ASCE-7 - use latest edition (1995) and Table in paragraph 3.2.1 of Ref 13.

$$q_z = 0.00256 K_z K_{zt} V^2 I \quad (\text{Eq. 6-1})$$

$$= 0.00256 (0.85) (1.0) (80)^2 (1.0)$$

$$= 14 \text{ psf} \quad \rightarrow \text{Table 6-3}$$

$$F = q_z G C_f A_f$$

$$= 14 (0.85) (1.2) (2' \times 2') = 57 \text{ lb}$$

$\rightarrow 6.6.1 \quad \rightarrow \text{Table 6-8}$
for solid signs

Housing Wind = 57 lb

$\therefore$ Seismic Controls.

Seismic loads are very small and the frame is very stiff and heavy.

Frame OK by inspection

Look at anchorage.

FLUOR DANIEL NORTHWEST, INC.

Calc. No. W320-24-02Revision 3Page No. 24 of —

DESIGN ANALYSIS

Client NumatecWO/Job No. W-320Subject Structural Analysis forDate 1/5/98By AmuthangurinSafety Related Equipment Attach.Checked 1/7/98By Michael R. CarterLocation AY-Farm & C-Farm

Revised

By

Anchorage

$$T_1 = \frac{(95 \text{ lb})(5.65') + (76 \text{ lb})(2.5')}{(2.0')(2 \text{ legs})}$$

$$= 182 \text{ lb/leg.}$$

 T_2 due to seismic in orthogonal direction

$$T_2 = 182 \left(\frac{2.0}{2.75} \right) = 132 \text{ lb/leg}$$

$$T_{\text{resultant}} = \sqrt{(182)^2 + (132)^2} = \frac{157 \text{ lb}}{4 \text{ legs}}$$

$$T_{\text{resultant}} = 186 \text{ lb/leg} = 46 \text{ lb/bolt}$$

$$V_{\text{resultant}} = \sqrt{2} \left(\frac{95 + 76}{4 \text{ legs}} \right) = 42 \text{ lb/leg} = 11 \text{ lb/bolt}$$

Per Ref. 1, Table 23, for Hilti KBII $\frac{3}{8}$ " w/ $1\frac{5}{8}$ " emb:

$$T_a = 690 \text{ lb} >> 46 \text{ lb}$$

$$V_a = 1240 \text{ lb} >> 11 \text{ lb}$$

} $\therefore$ Hilti's OK

Slab $(3.5')(4')(\frac{1}{2})(150 \text{ lb/ft}^3) = 1050 \text{ lb}$

Slab is much heavier than equipment itself and loads on equipment legs are small

Check overturning - see page 27
Punching OKUse reinforcement per the standard detail
WWF 6x6 - W2.9 x W2.9

Plus add (4) #4 bars x 3'-0" long around riser

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Calc. No. W320-24-027Revision 3Page No. 25 of —

DESIGN ANALYSIS

Client	<u>Numatec</u>	WO/Job No.	<u>W-320</u>
Subject	<u>Structural Analysis for</u>	Date	<u>1/5/98</u>
	<u>Safety Related Equipment Attach.</u>	By	<u>David Hangerman</u>
		Checked	<u>1/7/98</u>
Location	<u>AY-Farm & OC Farm</u>	By	<u>Michael R. Quate</u>
		Revised	

Existing Riser: frame will take most lateral load however, the exist riser may take some of the seismic load - Look at potential additional stresses by conservatively assuming all lateral loads are taken by riser.

Moment at base due to Housing & Frame

$$M = (95 \text{ lb})(5.65) + (76 \text{ lb})(2.5') = 727 \text{ ft-lb}$$

Say 800 ft-lb

Riser is 12" ϕ sch 40 pipe to 14" above grade. with 12" ϕ 14 ga sheet metal above that.

$$- 12" \phi \text{ Sch 40: } f_b = \frac{M}{S} = \frac{800 \text{ ft-lb}(12)}{43.8} = 219 \text{ psi}$$

$$\frac{f_b}{F_b} = \frac{0.219}{0.60(35 \text{ ksi})} = 0.010 \quad - \text{ approx } 1\% \text{ increase in stress.}$$

- At 1.2' above base (14 ga sheet metal)

$$M = (95 \text{ lb})(4.45) + (76 \text{ lb})(1.3') = 521 \text{ ft-lb.}$$

$$S_{14 \text{ ga.}} = \pi R^2 t = \pi (6")^2 (0.0785") = 8.88 \text{ in}^3$$

$$f_b = \frac{521(12)}{8.88} = 704 \text{ psi}$$

$$\frac{f_b}{F_b} = \frac{0.704}{0.60(38 \text{ ksi})} = 0.03 \quad - 3\% \text{ stress increase}$$

$\therefore$ negligible impact on riser.

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Calc. No. W-320-24-02Revision 3Page No. 26 of —

DESIGN ANALYSIS

Client NumatecWO/Job No. W-320Subject Structural Analysis forDate 1/5/98By Ann HangerSafety Related Equipment AttachmentChecked 1/7/98By Michael R. Curtis

Location

Revised

By

DOME LOAD

Look at additional weight added by air breather filter support and pad.

(A dome load report was not prepared by this project for tank 241-C-105)

Housing = 150 lbs

Frame = 120

Concrete = 1050

Concrete Void at riser

$$\pi (0.5')^2 \left(\frac{6}{12}\right) (150) = -59$$

TOTAL ADDED WEIGHT = 1261 lb.

Per OSD (Reference #14) the allowable live load limit is 200,000 lb — The weight of this equipment is negligible in comparison

DESIGN ANALYSIS

Client NumatecWO/Job No. W320Subject Structural Analysis forDate 1/6/98By Amr HaneenSafety Related Equipment AttachChecked 1/7/98By Michael E. CusterLocation AY-Farm & C-Farm

Revised

By

Slab - Check for overturning

$$M = 95 \text{ lb} (5.65' + 0.5') + 76 \text{ lb} (2.5' + 0.5')$$

$$M = 812 \text{ lb-ft}$$

$$P = 157 + 1050 = 1207 \text{ lb}$$

$$\text{Overturning about pt. A: } SF = \frac{(0.85P)(\frac{3.5'}{2})}{M} = \frac{(0.85)(1207)(\frac{3.5'}{2})}{812 \text{ lb-ft}}$$

$$SF = 2.2 > 1.5$$

OK

$$e = \frac{M}{P} = \frac{812}{1207} = 0.67' = 8" \quad (0.67)(6) = 4'$$

resultant is slightly outside of middle third of slab - OK

Check slab bending

$$p_{\max} = \frac{2(P)}{(39/12)(4')} = 186 \text{ psf}$$

Soil pressure small

Apply $p_{\max}$ to slab span between support legs

$$M = \frac{wl^2}{8} = \frac{186 \text{ lb/ft} (2')^2}{8} = 93 \text{ ft-lb}$$

very small moment
WDF reinf OK

VIEW A-A

SECTION B-B

241-AY-02E
SLUCE PIT
KEY PLAN
SCALE: NONE

CLOSURE PLATE
SCALE: 3"=1'-0"
12 GA SHEET, ASTM A569

MTG BRACKET
SCALE: 3"=1'-0"
1/4" THK PLATE, ASTM A36

ALL Channels shown are C 4 X 5.4

W320-24-027
REV. D
Pg. A-1

[illegible][illegible]

-C4x5.4 (TVP)

[illegible]

DETAIL 2

[illegible]

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Calculation Cover Sheet

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Page 1 of 8

Calculation No. 7455-03 Group Eng.

Subject: Determination of Foundations Loads of Seismic Shutdown Systems.

☐ Computer Output Listing(s) on file ☒ No Computer Output Listing

Revision No.	Description	Originator/Date	Checker/Date	Approved/Date
0	Initial Issue	<i>Lee</i> 12/4/95	N/A	<i>SP</i> 12/4/95

Abstract:

Objective: This calculation was performed to determine the foundation loads of the Seismic Shutdown Systems identified in reference no.1.

- References:
1. Westinghouse Hanford Company Specification No. W-320-P35, Rev. 0.
 2. IEEE Standard 344-1987.
 3. Nutherm Calculation Nos. 7455-01CG & 7455-02CG, "Center of Gravity of the Seismic Shutdown System Cabinets"

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Calculation Continuation Sheet

Page 2 of 8

Calculation No. 7455-03

Rev. No.	0																		
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Calculation Details

Procedure: In accordance with Reference 1 the foundation loads shall be calculated for the Seismic Shutdown Systems. The calculations are performed individually for shutdown systems AY-SSS-1A & AY-SSS-1B. System C-SSS-1A & C-SSS-1B are identical to the above analyzed systems. The configuration, dimensions, and symbols of the shutdown system are shown on page 4 of this calculation.

The dimensions and weights of the equipment are presented on page 5 of this calculation. The center of gravity locations are extracted from Reference 3.

The operability loads and nozzle loads are not applicable to the equipment analyzed.

The foundation loads are determined for two load conditions:

1. Dead weight loads (which would include normal operating conditions);
2. Dynamic loads, SSE conditions.

The foundation loads for the SSE condition was determined by implementation of static coefficient analysis, as described in paragraph 6.3 of reference 2.

The peak acceleration levels of the RRS' are presented on page 6 of this calculation. They were derived from a composite of the RRS provided in Appendix "A" of specification W-320-P35. The RRS' for the following damping value was used:

SSE condition - 5%

A static coefficient of 1.5 is used to take into account the effects of both multifrequency excitation and multi-mode response.

The orientation of the shutdown systems in the nuclear power plant is unknown. Therefore, the highest horizontal acceleration level is used in the calculation for both horizontal directions.

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Calculation Continuation Sheet

Page 3 of 8

Calculation No. 7455-03

Rev. No.

9

Calculation Details

The base of the shutdown system is bolted to the foundations with symmetrically spaced bolts. That is why the centroid of the foundation bolts is located in the center of the base. The foundation loads were calculated using the following equations:

Dead Weight Loads

$$F_x = 0; F_y = W; F_z = 0$$

Dead-Weight Moments

$$M_x = F_y \left| \frac{B}{2} - b \right|; M_y = 0; M_z = F_y \left| \frac{L}{2} - \ell \right|$$

$$= W \left| \frac{B}{2} - b \right|; M_y = 0; M_z = W \left| \frac{L}{2} - \ell \right|$$

Dynamic Loads

$$F_{vd} = 1.5 H_v \times W; F_{vd} = 1.5 V_d \times W; F_{zd} = 1.5 H_d \times W;$$

Dynamic Moments

$$\begin{aligned} M_x &= F_{xd} \times h + F_{yd} \left[\frac{B}{2} - b \right] \\ &= 1.5 \text{ W} \left[H_x \times h + V_x \left[\frac{B}{2} - b \right] \right] \\ M_y &= F_{xd} \left[\frac{B}{2} - b \right] + F_{yd} \left[\frac{L}{2} - \ell \right] \\ &= 1.5 \text{ W} H_x \left[\left[\frac{B}{2} - b \right] + \left[\frac{L}{2} - \ell \right] \right] \\ M_z &= F_{xd} \times h + F_{yd} \left[\frac{L}{2} - \ell \right] \\ &= 1.5 \text{ W} \left[H_x \times h + V_x \left[\frac{L}{2} - \ell \right] \right] \end{aligned}$$

where:

W = weight, see page 5;
 H_x = horizontal acceleration, see page 6;
 V = vertical acceleration, see page 6;
 L, B, H = seismic shutdown system dimensions, see pages 4 & 5;
 ℓ, b, h = center of gravity location, see pages 4 & 5;
 1.5 = static coefficient, ref. 2

The results of foundation load calculation are presented on pages 7 & 8.

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HNF-2463, Rev.0

Calculation Continuation Sheet

Page 4 of 8

Calculation No. 7455-03

Rev. No. 0

Calculation Details

SYMBOLS:

- - CENTER OF GRAVITY
- - CENTROID OF THE FOUNDATION BOLTS
- H1 & H2 - HORIZONTAL DYNAMIC ACCELERATIONS
- V - VERTICAL DYNAMIC ACCELERATION
- g - GRAVITY ACCELERATION

L:239

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HNF-2468, Rev.0

Calculation Continuation Sheet

Page 5 of 8

Calculation No. 7455-03

Rev. No. 0

Calculation Details

SEISMIC SHUTDOWN SYSTEM DIMENSIONS, CENTER OF GRAVITY LOCATIONS, AND WEIGHTS

Equipment Tag No.	Dimensions In.			Center of Gravity Location, In			Weight W, lbs.
	L	B	H	ℓ	b	h	
SSS1A	36	16	60	11.35	6.84	32.06	546.50
SSS1B	36	16	60	17.47	6.31	30.79	413.50

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HNF-2468, Rev.0

Calculation Continuation SheetPage 6 of 8Calculation No. 7455-03Rev. No. 0**Calculation Details****PEAK ACCELERATION LEVELS**

SSE	
Vertical, g's	Horizontal, g's
V	H _x
0.386	0.58

Note: H_x - acceleration used in calculation for both horizontal directions and includes 10% margin of composite RRS from Appendix A of specification W-320-P35.

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Calculation Continuation Sheet

Page 7 of 8

Calculation No. 7455-03

Rev. No.	0																		
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Calculation Details

FOUNDATION LOAD SUMMARY

Equipment Title: Seismic Shutdown SystemEquipment Number: AY-SSS-1A

FOUNDATION LOADS		FORCES (LBS)			MOMENTS (IN-LBS)		
TYPE OF LOADS		F _x	F _y	F _z	M _x	M _y	M _z
DEADWEIGHT LOAD		0	546.5	0	634	0	3634
DYNAMIC LOADS	SSE Conditions	475.4	316.4	475.4	15,606	3,713	17,346

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Calculation Continuation Sheet

Page 8 of 8

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Calculation No. 7455-03

Rev. No. 0

Calculation Details

FOUNDATION LOAD SUMMARY

Equipment Title: Seismic Shutdown SystemEquipment Number: AY-SSS-1B

FOUNDATION LOADS		FORCES (LBS)			MOMENTS (IN-LBS)		
TYPE OF LOADS		F_x	F_y	F_z	M_x	M_y	M_z
DEADWEIGHT LOAD		0	413.5	0	699	0	219
DYNAMIC LOADS	SSE Conditions	360	239	360	11,482	799	11,202

3 EYE,
ERN
ASHER

/8-11 UNC

10-32 UNF X .5"
SLOTTED ROUND
HEAD MACHINE
SCREW

3/8-16 UNC X 1
HEX HEAD MACHINE SCREW
(2) 3/8 SAE FLATWASHERS
3/8 LOCKWASHER
3/8-16 UNC HEX NUT

6-32 UNF X .5"
SLOTTED ROUND HEAD
MACHINE SCREW
30 REQ'D.

3 7

10-32 UNC X .5
SLOTTED ROUND HEAD
MACHINE SCREW
10 FLATWASHER
10 STARWASHER
AS REQ'D.

HNF-2468, Rev 0

W320-24-027, Rev 1
ATTACH. 3, PAGE 9099

REV	DESCRIPTION	DATE	DWN	APV
1	SEE DCR # 1214	11/9/95	SFM	
2	SEE DCR # 1233	4/1/96	SFM	

HNF-2468, Rev.0

NOTES:
ENCLOSURE IS HOFFMAN A-72H3724FS
TOTAL ENCLOSURE WEIGHT IS APPROXIMATELY 1100 POUNDS

DS-1 = STACK LOW FLOW ALARM
DS-2 = RECORD SAMPLE NON-ISO FLOW ALARM
DS-3 = BETA/GAMMA SAMPLE NON-ISO FLOW ALARM
DS-4 = PUMP #1 FAILURE ALARM
DS-5 = PUMP #2 FAILURE ALARM
DS-6 = CABINET POWER
DS-7 = PUMP #1 ON
DS-8 = PUMP #2 ON
DS-9 = RADIATION DETECTOR FAILURE ALARM

SW-5 = PUMP #1 LEAD/LAG

PB-1 = PUMP START/STOP
PB-2 = PUMP FAILURE ALARM RESET
PB-3 = FAILURE ALARM ACKNOWLEDGE
PB-4 = HIGH RADIATION ALARM ACKNOWLEDGE
PB-5 = RAD. MON. CALIBRATION IN PROGRESS

ATTACHMENT 4
CALC. W-320-027, REV. 1
PAGE 1 OF 2

PROJECT:
PROJECT W-320-P41
WESTINGHOUSE HANFORD CO.
RICHLAND, WA
AMC W.O. 25255
P.O. 80166

CONNECTION CODE:

A. STACK STATIC PRESSURE LINE
B. STACK TOTAL PRESSURE LINE
C. STACK RECORD SAMPLE LINE
D. STACK BETA/GAMMA MONITOR SAMPLE LINE
E. STACK RETURN EXHAUST LINE
F. STACK RTD & BETA/GAMMA MONITOR SIGNAL
G. RECORD SAMPLE HEAT TRACE
H. BETA/GAMMA SAMPLE HEAT TRACE
I. SAMPLE RETURN LINE HEAT TRACE
J. POWER TO RECORD SAMPLE CABINET
K. POWER TO BETA/GAMMA CABINET
L. INCOMING POWER FROM CUSTOMER

CONNECTION SIZE:

1/4" COMPRESSION
1/4" COMPRESSION
1/2" FPT
1/2" FPT
1" COMPRESSION
1-1/4" CONDUIT
3/4" CONDUIT
3/4" CONDUIT
3/4" CONDUIT
1/2" CONDUIT
1/2" CONDUIT
3/4" CONDUIT

UNLESS OTHERWISE SPECIFIED, DIMENSIONS ARE IN INCHES, TOLERANCES ARE:

PER Q.A. PROCEDURE # 26

DRAWN BY: S. MINAKER DATE: 8/21/95
DESIGN BY: S. Minaker DATE: 9/6/95
DESIGN VERIF: P. Dave DATE: 9/6/95
QA REVIEW BY: D. Slatte DATE: 9/6/95
APPROVED BY: D. DeBoun DATE: 9/7/95

AIR MONITOR CORPORATION P.O. BOX 6358 SANTA ROSA CA 95406 TELEPHONE (707) 544-2706		THIS DOCUMENT CONTAINS PROPRIETARY INFORMATION WHICH SHALL NOT BE REPRODUCED, COPIED, OR DISCLOSED TO OTHERS FOR ANY PURPOSE WITHOUT WRITTEN PERMISSION FROM AIR MONITOR CORP.
INSTRUMENT & CONTROL CABINET GENERAL ARRANGEMENT		
CAD PLOT 04/02/96 10:05:26	FORMAT/DWG NO. B W25255BA	REV 2
SCALE FACTOR: 17.00	SCALE: 1:17.00	DO NOT SCALE DRAWING

ICF KAISER HANFORD

ENGINEERING CHANGE NOTICE SKETCH

Ref Dwg	Sh	Rev	Prepared By	Checked By	ECN No	Page
H-2-818585	1	0	SM DUNHAM	<i>R. E. Dunham</i>	W-320-365	3

ZONE F-1/2

MATERIAL LIST			
QTY.	P/N	DESCRIPTION	MATERIAL/REF
AR	1	TUBING, 1/4" ODT X 0.035" WALL, SST	SPECIFICATION W-320-C5 SEC 13440
AR	2	TUBING, 1/2" ODT X 0.049" WALL, SST	SPECIFICATION W-320-C5 SEC 13440
AR	3	TUBING, 1" ODT X 0.065" WALL, SST	SPECIFICATION W-320-C5 SEC 13440
6	4	HEX HEAD CAP SCREW, 1/2"-13UNC-2A X 1-1/2" L	ASTM A307-92A
6	5	NUT, 1/2"-13 UNC-2B	ASTM A194 GR 2
6	6	WASHER, 1/2"	ASTM F-463 TYPE 3
6	7	ANGLE, 3" X 3" X 1/4" X 0'-4"	ASTM A36

REVISE

REVISE

REVISE

REVISE

ZONE B-7/8

(B) VIEW

ZONE B/C-6/7

(C) VIEW

ATTACHMENT 4
CALC. W-320-027, REV. 1
PAGE 2 OF 2

W320-24-028

Engineering Evaluation of
Commercial Grade Items

**KAISER ENGINEERS
HANFORD****CALCULATION IDENTIFICATION AND
INDEX**Page **i** of **i**

Date

8-1-95

This sheet shows the status and description of the attached Design Analysis sheets.

Discipline Civil/Structural WO/Job No. ER4319

Calculation No. W320-24-028

Project No. & Name W-320, Tank 241-C-106 Sluicing

Calculation Item Engineering Evaluation of Commercial Grade Items

These calculations apply to:

Dwg. No. NA

Rev. No. NA

Dwg. No.

Rev. No.

Other (Study, CDR)

Rev. No.

The status of these calculations is:

☐ Preliminary Calculations☒ Final Calculations☐ Check Calculations (On Calculation Dated)☐ Void Calculation (Reason Voided)

Incorporated in Final Drawings?

☒ Yes ☐ No

This calculation verified by independent "check" calculations?

☐ Yes ☒ No

Original and Revised Calculation Approvals:

	Rev. 0 Signature/Date	Rev. 1 Signature/Date	Rev. 2 Signature/Date
Originator	RW Davidson 8/1/95	<i>AKhangervin</i> 4/1/98	
Checked by	NA	NA	
Approved by	<i>Rob Dink</i> 8/8/95	<i>AKhangervin</i> 4/1/98	
Checked Against Approved Vendor Data		NA	

INDEXDesign Analysis
Page No.

Description

1-2	ACME Pre-Approved Mix Design No. 6452
3	Deformed Steel Bars for Concrete Reinforcement, ASTM A 615
4	Rolled Steel Shapes, Plates, Bars, ASTM A 36
5	Steel Pipe, ASTM A 53
6-7	Nelson Weld Stud Anchors, Type H4L
8-9	Nonshrink Epoxy Grout, ASTM C 827
Δ	<i>Revised p. 9</i>

KAISER ENGINEERS
HANFORD

Calc. No. W320-24-028

Revision 0

Page No. 1 of

DESIGN ANALYSIS

Client WHC

WO/Job No. ER4319/W320

Subject Engineering Evaluation of Commercial
Grade Items

Date 8-1-95

By RW Davidson

Location 200E, Tank 241-G-106 Sluicing

Checked

By NA

Revised

By

Commercial Grade Item: ACME Pre-Approved Mix Design No. 6452

1. What is the function that the concrete must perform to enable the safety class 1 system to perform its safety or critical function.
 - a. The concrete forming the cover blocks must provide an impervious barrier to mitigate the release of a spray leak to the outside environment during Sluicing activities only. Note: Enviromental/seismic qualification must satisfy only a safety class 3 over 1 function to insure structural integrity of concrete cover blocks and pits is maintained when Sluicing activities are not taking place.
2. What characteristics of the concrete are critical to the performance of its critical function.
 - a. Tension, compression and/or shear failure due to self weight and applied normal operating loads during Sluicing.
 - b. Propagation of shrinkage cracks due to low water/cement (W/C) ratio.
 - c. Void spaces or segregation within a concrete pour due to viscosity of concrete mix.
3. What characteristics does the item have which are critical to the predictability of its performance? (Critical characteristics are identifiable and/or measurable attributes of a part/item which provides assurance that the part/item will perform as predicted.)
 - a. Tension, compression and shear strengths are all a function of the compressive strength of the concrete at 28 days after batching.
 - b. Compressive strength, W/C ratio and the concrete mix viscosity are all a function of the reliability of the mix proportion data.
4. What type of acceptance testing, when required, (load tests, Brinell hardness tests, nondestructive examination (NDE) tests, destructive physical tests, etc.) is required to be performed by KEH to provide confidence that the part/item has the necessary critical characteristics?
 - a. Perform an evaluation of the concrete mix proportions in accordance with ACI 349, Sections 4.2 and 4.3 for the specified class of concrete.
 - b. Perform an evaluation of the concrete mix ingredients to insure compliance with specifications and applicable codes and standards. The following is a list of concrete mix ingredients to be evaluated:

Cement in accordance with ASTM C 150, Type II

KAISER ENGINEERS
HANFORD

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DESIGN ANALYSIS

Client WHC

WO/Job No. ER4319/W320

Subject Engineering Evaluation of Commercial
Grade Items

Date 8-1-95

By RW Davidson

Location 200E, Tank 241-C-106 Sluicing

Checked

By NA

Revised

By

Commercial Grade Item: ACME Pre-Approved Mix Design No. 6452.... Cont'd

- Fly Ash in accordance with ASTM C 618, Class F. Amount of fly ash shall be $25\% \pm 1\%$ by weight of cement.
 - Aggregates in accordance with ASTM C 33, maximum size of course aggregate $3/4"$.
 - Air Entraining Admixture in accordance with ASTM C 260, no chloride added during manufacture.
 - Water Reducing Admixture in accordance with ASTM C 494, Type D, no chloride added during manufacture.
- c. Perform an evaluation of the concrete mix properties to insure compliance with specifications and applicable codes and standards. The following is a list of concrete mix properties to be evaluated:
- Slump $4" \pm 1"$
 - Air Content $5\% \pm 1\%$
- d. Perform the following test procedures on the concrete mix at the time of placement in the field. Each test procedure shall be performed a minimum of one time for each pour or one time for each 100 yards delivered for a pour, whichever is greater.
- Slump test in accordance with ASTM C 143, $4" \pm 1"$.
 - Air Content test in accordance with ASTM C 173 $5\% \pm 1\%$.
 - Obtain concrete test cylinders in accordance with ASTM C 31 for determination of concrete compressive strength in accordance with ASTM C 39, $f'_c \geq 4000$ psi.
5. What manufacturing controls are required, if any, in addition to receipt inspection and/or acceptance testing (such as KEH witnessing test, KEH shop inspection, etc.)?
- a. The Concrete Testing Agency(s) performing tests during development and field placement of the concrete mix shall be evaluated in accordance with ASTM E 329.

KAISER ENGINEERS
HANFORD

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DESIGN ANALYSIS

Client WHC

WO/Job No. ER4319/W320

Subject Engineering Evaluation of Commercial
Grade Items

Date 8-1-95

By RW Davidson

Checked

By NA

Location 200E, Tank 241-C-106 Sluicing

Revised

By

COMMERCIAL GRADE ITEM (CGI): Deformed Steel Bars for Concrete Reinforcement (ASTM A 615, Grade 60)

1. What is the function that the rebar must perform to enable the safety class 1 system to perform its critical function.
 - a. Provide tensile and shear strength to concrete to resist live, dead loadings during Sluicing activities.
 - b. Minimize temperature and shrinkage cracking in concrete at right angles to principal reinforcement.
2. What characteristics does the item have which are critical to the predictability of its performance? (critical characteristics are identifiable and/or measurable attributes of an item which provide assurance that the item will perform as predicted.)
 - A. Tension and shear strength of the material is by yield strength of the material.
 - B. Physical shape, such as deformation size and spacing.
 - C. Weight of the material.
3. What type of acceptance testing, when required, (load tests, Brinnell hardness tests, NDE tests, destructive physical tests, etc.) is required to be performed by KEH to provide confidence that the part/item has the necessary critical characteristics?

Submit certified test reports (mill test reports) verifying yield strength and chemical composition of rebars in accordance with ASTM A 615, grade 60. The test reports must verify that the material meets the weight, spacing and height of deformations requirements in accordance with ASTM A 615, grade 60.
4. What manufacturing controls are required, if any, in addition to receipt inspection and/or acceptance testing (such as KEH witnessing test, KEH shop inspection, etc.)?

None
5. Is part/item identifiable to a manufacturer's published product description which meet the critical characteristics?
 - A. Product description for deformed steel bars for concrete reinforcing is ASTM A 615, Grade 60. Each bundle of reinforcing rebar with certified test report (at placement) shall be traceable with a permanently marked tag identifying manufacturer and heat numbers from which the product was made.

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HANFORD**

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DESIGN ANALYSIS

Client WHC

WO/Job No. ER4319/W320

Subject Engineering Evaluation of Commercial
Grade Items

Date 8-1-95

By RW Davidson

Checked

By NA

Location 200E, Tank 241-C-106 Sluicing

Revised

By

Commercial Grade Item: Rolled Steel Shapes, Plates, Bars (ASTM A 36)

1. What would cause the part/item to fail (what are the credible failure modes)?
 - a. Tension, compression, shear and/or buckling failure due to applied loads.
 - b. Tension or shear failure near a weld due to applied loads.
 - c. Insufficient material to resist the applied loads.
 - d. Incorrect chemical composition.
2. What characteristics does the item have which are critical to the predictability of its performance? (Critical characteristics are identifiable and/or measurable attributes of a part/item which provides assurance that the part/item will perform as predicted.)
 - a. Tension, compression, shear and buckling strength of the material is based on the yield strength of the material.
 - b. Chemical composition.
 - c. The physical dimensions of the shapes or plates.
3. What type of acceptance testing, when required, (load tests, Brinell hardness tests, nondestructive examination (NDE) tests, destructive physical tests, etc.) is required to be performed by KEH to provide confidence that the part/item has the necessary critical characteristics?
 - a. Perform a tensile test in accordance with ASTM A 370 to verify the minimum yield strength of the material is greater than 36,000 psi.
 - b. Perform a chemical analysis in accordance with ASTM A 751 to verify the material meets the chemical requirements of ASTM A 36 Table 2.
 - c. Perform a check of the dimensions to verify the plates or shapes are within permissible variations of ASTM A 6.
4. What manufacturing controls are required, if any, in addition to receipt inspection and/or acceptance testing (such as KEH witnessing test, KEH shop inspection, etc.)?

None
5. Is part/item identifiable to a manufacturer's published product description which meet the critical characteristics?

Product description for rolled steel shapes and plates is ASTM A 36.

KAISER ENGINEERS
HANFORD

Calc. No. W-320-24-028

Revision 0

Page No. 5 of

DESIGN ANALYSIS

Client WHC

WO/Job No. ER4319/W320

Subject Engineering Evaluation of Commercial
Grade Items

Date 8-1-95

By RW Davidson

Checked

By NA

Location 200E, Tank 241-C-106 Sluicing

Revised

By

Commercial Grade Item: Steel pipe (ASTM A 53)

1. What would cause the part/item to fail (what are the credible failure modes)?
 - a. Insufficient material integrity.
 - b. Incorrect chemical composition.
2. What characteristics does the item have which are critical to the predictability of its performance? (Critical characteristics are identifiable and/or measurable attributes of a part/item which provides assurance that the part/item will perform as predicted.)
 - a. Material integrity is a function of the yield strength of the material.
 - b. Chemical composition.
3. What type of acceptance testing, when required, (load tests, Brinell hardness tests, nondestructive examination (NDE) tests, destructive physical tests, etc.) is required to be performed by KEH to provide confidence that the part/item has the necessary critical characteristics?
 - a. Perform a tensile test in accordance with ASTM A 370 to verify the minimum yield strength of the material is greater than 35,000 psi.
 - b. Perform a chemical analysis in accordance with ASTM A 751 to verify the material meets the chemical requirements of ASTM A 53 Table 1 and 2 for Type S, Grade B.
4. What manufacturing controls are required, if any, in addition to receipt inspection and/or acceptance testing (such as KEH witnessing test, KEH shop inspection, etc.)?

None
5. Is part/item identifiable to a manufacturer's published product description which meet the critical characteristics?

Product description for steel pipes is ASTM A53, Type S, Grade B.

DESIGN ANALYSIS

Client WHC

WO/Job No. ER4319/W-320

Subject Engineering Evaluation of Commercial
Grade Items

Date 8-1-95

By RW Davidson

Checked

By NA

Location 200E, Tank 241-C-106 Sluicing

Revised

By

Commercial Grade Item: Nelson Stud Welded Concrete Anchor type H4L, manufactured by TRW Nelson Stud Welding Division

1. What would cause the part/item to fail, (what are the credible failure modes)?
 - a. Tension or shear
 - b. Compressive strength of concrete
 - c. Stud size, spacing and edge distance
 - d. Welding of studs
2. What characteristics does the part/item have which are critical to the predictability of its performance? (Critical characteristics are identifiable and/or measurable attributes of an item which provide assurance that the item will perform as predicted.)
 - a. Tensile or shear strength is based on studs installed in accordance with the manufacturer's instructions.
 - b. Ultimate compressive stress of the concrete.
 - c. Stud size, spacing and edge distance
 - d. Welding of studs
3. What type of acceptance testing, when required, (load tests, Brinell hardness tests, nondestructive examination (NDE) tests, destructive physical tests, etc.) is required to be performed by KEH to provide confidence that the part/item has the necessary critical characteristics?

Extensive testing on Nelson Weld Studs were performed by the Engineering Division of The Nelson Stud Welding Company and was witnessed by an independent agency, Herron Testing Laboratories of Cleveland, Ohio. The tensile and compressive results from these tests became the basis of the Concrete Cone Failure Concept, published in Prestressed Concrete Institute Design Handbook in 1971, (Ref. d). Another comprehensive test program at Lehigh University, Pennsylvania resulted in the following publication, "

"Headed Steel Anchors Under Combined Loading", AISC Engineering Journal, Second Quarter 1973, pages 43-52 (Ref. e). The failure mechanism of stud anchors is controlled by yielding of steel rather than shear cone failure. ACI 349-85 discusses this concept in pages 18-26. (Ref. b)

The shear and tensile ultimate values developed by testing and used by TRW Nelson Stud Welding Division were compared with analytical values based on the shear cone concept. The ultimate loads computed by analytical method were lower than loads achieved by actual testing.

Based on the above mentioned history, the track record of the studs for the last thirty years, and the fact that they are almost a sole source in industry use, their tensile and shear ultimate values as given by the manufacturer are acceptable provided the following conditions are met:

DESIGN ANALYSIS

Client WHC

WO/Job No. ER4319/W-320

Subject Engineering Evaluation of Commercial
Grade Items

Date 8-1-95

By RW Davidson

Checked

By NA

Location 200E, Tank 241-C-106 Sluicing

Revised

By

- a. Perform a tensile test to verify that the minimum tensile strength of the studs is 60,000 psi. Tensile test will be performed in accordance with ASTM A370 and samples selected in accordance with ASTM A29.
- b. Perform a bending, torquing or tensioning test in accordance with AWS D1.1, Article 7.6.6 to ensure that the weld attaching the weld stud to the base material will not fail.
- c. Perform a check of the stud dimensions to ensure permissible variations in dimensions are in accordance with ASTM A29.
- d. Perform visual inspection of stud spacings and edge distances to verify that no stud is placed with less than the minimum requirement, as specified by the manufacturer, and more than the maximum requirement, as specified by design.
- e. Certificate of conformance will be required from the manufacturer to ensure that the studs meet the requirements of ASTM A108.
- f. Concrete with required compressive strength shall be furnished by a qualified supplier who meets specified NQA-1 requirements or be furnished by a manufacturer where their product has been independently tested and qualified as a Commercial Grade Item.

4. What manufacturing controls are required, if any, in addition receipt inspection and/or acceptance testing (such as KEH witnessing test, KEH shop inspection, etc.)?

None

5. Is part/item identifiable to a manufacturer's published product description which meet the critical characteristics?

Product is the Nelson Stud Welded Concrete Anchor type H4L manufactured by TRW Nelson Stud Welding Division.

References:

- a. Design Data, Nelson Concrete Anchor, studs for securing steel to concrete, Published in 5/76 by TRW Nelson Division.
- b. ACI349-85, Commentary on Code Requirements for Nuclear Safety Related Concrete Structures.
- c. ANSI/AWS D1.1-88, Structural Welding Code.
- d. Prestressed concrete Institute Design Handbook.
- e. "Headed Steel Anchors Under combined Loading", AISC Engineering Journal Second Quarter, 1973.

KAISER ENGINEERS
HANFORD

Calc. No. W320-24-028

Revision 0

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DESIGN ANALYSIS

Client WHC

WO/Job No. ER4319/W320

Subject Engineering Evaluation of Commercial
Grade Items

Date 8/1/95

By RW Davidson

Checked

By NA

Location 200E, Tank 241-C-106 Sluicing

Revised

By

Commercial Grade Item (CGI): Nonshrink Epoxy Grout (ASTM C 827)

1. What would cause the part/item to fail (what are the credible failure modes)?
 - a. Volume change as a result of shrinkage or excessive expansion.
 - b. Compressive failure due to applied loads.
 - c. Thermal expansion as a result of excessive heat.
 - d. Creep as a result of excessive loading over time.
2. What characteristics does the item have which are critical to the predictability of its performance? (Critical characteristics are identifiable and/or measurable attributes of a part/item which provides assurance that the part/item will perform as predicted.)
 - a. Volume change.
 - b. Compressive strength.
 - c. Thermal expansion.
 - d. Creep Resistance.
3. What type of acceptance testing, when required, (load tests, Brinell hardness tests, nondestructive examination (NDE) tests, destructive physical tests, etc.) is required to be performed by ICF KH to provide confidence that the part/item has the necessary critical characteristics?
 - a. Grout shall show no shrinkage (0.0%) and a maximum 2.0% expansion from time of placement when tested according to ASTM C 827, modified to use a ball with a specific gravity between 0.9 and 1.1.
 - b. Grout shall show a minimum 7-day compressive strength of 16,000 psi when tested according to ASTM C 579.
 - c. Grout shall show a maximum coefficient of thermal expansion of 30×10^{-6} in/in/°F when tested according to ASTM C 531.

**KAISER ENGINEERS
HANFORD**

Calc. No. W320-24-028

Revision *81*

Page No. 9 of

DESIGN ANALYSIS

Client WHC

WO/Job No. ER4319/W320

Subject Engineering Evaluation of Commercial
Grade Items

Date 8/1/95

By RW Davidson

Checked

By NA

Location 200E, Tank 241-C-106 Sluicing

Revised *4/7/98*By *allanger*

- d. Grout shall show creep equal to or less than 5.0×10^{-3} in/in at 180°F for one year with a load of 400 psi when tested according to CPR Creep Test.

4. What manufacturing controls are required, if any, in addition to receipt inspection and/or acceptance testing (such as ICF KH witnessing test, ICF KH shop inspection, etc.)?

None

5. Is part/item identifiable to a manufacturer's published product description which meet the critical characteristics?

Product description is Nonshrink Epoxy Grout (ASTM C 827): U.S. Grout Corporation "Five Star Epoxy Grout" or approved substitute. Properties of Headed Studs, TRW Nelson Division, 1977.

NOTE

Safety Classification of Grout was changed to 3/1 (see ECN W-320-290) making CGI no longer a requirement

W320-24-029

Structural Analysis of Safety Related
Electrical Equipment Supports

**ICF KAISER HANFORD
COMPANY****CALCULATION IDENTIFICATION AND
INDEX**Page *i* of *iv*

Date

7/29/1995

This sheet shows the status and description of the attached Design Analysis sheets.

Discipline **Structural**WO/Job No. **ER 4319/W320**Calculation No. **W320-24-029**Project No. & Name **W320, Tank 241-C-106 Sluicing.**Calculation item **Structural analysis of Safety Related Electrical Equipment Supports.**

These calculations apply to:

Dwg. No.: **H-2-818678 shts 1 and 5, H-2-818690 sht 1, H-2-818702 shts 1 and 2**Rev. No.: **0/0/0/0/0**Dwg. No.: **H-2-818695 shts 1,2,4 and 5, H-2-818704 sh 1,3 and 4, H-2-820752 sh 3**Rev. No.: **0/0/0/0/0/0/0**Other (Study, CDR): **ECN W-320-251**Rev. No. **N/A**

The status of these calculations is:

- ☐ Preliminary Calculations
- ☒ Final Calculations
- ☐ Check Calculations (On Calculation Dated)
- ☐ Void Calculation (Reason Voided)

Incorporated in Final Drawings?

☒ Yes ☐ No

This calculation verified by independent "check" calculations?

☐ Yes ☒ No

Original and Revised Calculation Approvals:

	Rev. 0 Signature/Date	Rev. 1 Signature/Date	Rev. 2 Signature/Date
Originator	Michael B. Lasota 12/11/95	Michael B. Lasota 05/08/1997	Ann Hangerin 1/14/98
Checked by	Ronald W. Davidson 12/19/95	Ronald W. Davidson 5/08/1997	M.R. Custer 5/08/1997
Approved by	<i>[Signature]</i> 12/19/95	<i>[Signature]</i> 5/12/97	<i>[Signature]</i> 1/20/98
Checked Against Approved Vendor Data			<i>[Signature]</i> 7/7/98

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Page No.

Description

1	Objectives
1	Criteria and assumptions
1	Structural Analysis - description
1	References
1	Index
2 to 30	Calculations
31 through 49	Figures 1 through 17
50	Conclusions

ICF KAISER HANFORD COMPANY		CALCULATION IDENTIFICATION AND INDEX Continuation Sheet		Page <i>ii</i> of <i>iv</i>
				Date 7/29/1995
Discipline <i>Structural</i>		WO/Job No. <i>ER 4319/W320</i>		Calculation No. <i>W320-24-029</i>
	Rev. 0 Signature/Date	Rev. 1 Signature/Date	Rev. 2 Signature/Date	
Originator	<i>Michael B. Lasota 12/11/95</i>	<i>Michael B. Lasota 05/08/97</i> <i>ghe</i>	<i>H. B. Langerin 1/4/98</i>	
Checked by	<i>Ronald W. Davidson 12/19/95</i>	<i>Ronald W. Davidson 5/08/97</i> <i>DO</i>	<i>M.R. Custer 5/08/97</i> <i>MRC</i> <i>H. B. Langerin 1/2/98</i>	

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Design Analysis Page No.	Description

REVISION	
Revision No.	Description
0	Initial issue
1	Revised Page number for figures 1, 3 thru 9, 12, 13; replaced figures 2, 10; added figures 11 and 14 thru 17. Replaced sheet 1; Revised sheets 10, 11, 12, 13; added new sheets 13a, 13b, 14, 15, 16, 17a thru 17g, 18 thru 23, 24 thru 30 and 50.
2	Revised page 10 and added p. 27a per changes from ECN W320-709 (deleted anchor bolt) and ECN W-320-682 (added equipment pad). Correct page #s on pages <i>ii</i> and <i>iii</i>

iv or iv $\triangle 2$

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DESIGN VERIFICATION SCREENING CRITERIA

HNF-2468, Rev.0

Project/Document No. W-320/W320-24-029

When the design or design change affects hardware, formal design verification must be performed if one or more of the following questions must be answered affirmatively (YES).

YES

NO

 ✓

1. Does the design or design change involved meet the established criteria to be considered Safety Class 1?

 ✓

2. Does this design or design change cause or permit changes to Safety Class 1 instrument or alarm setpoints outside of previously approved operational limits?

 ✓

3. Does this design or design change significantly affect the nuclear or environmental safety consequences of a malfunction or failure of the structure, system, or component?

 ✓

4. Does this design or design change involve or change design that has previously undergone formal design verification?

Assigned Lead Engineer

12/19/95

Date

Responsible Discipline Manager

12/19/95

Date

Original Design Package Distribution:

Project Manager

Design Verification Officer

Engineering Document Control

Design Change Distribution:

Attach to Engineering Change Notice

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Revision: 1

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DESIGN ANALYSIS

Client: WHC

Subject: Structural analysis of Safety Related
Electrical Equipment Supports.

Location: 241-C-106 & 241-AY-102

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Revised N/A

By: M. B. Lasota

By: R. W. Davidson

By: N/A

1. OBJECTIVES

The purpose of this calculations is to demonstrate that the design of the Electrical Equipment Supports is adequate to survive postulated Earthquake and Wind loads when installed at either Tank Farm during sluicing process without adversely effecting sluicing operation.

2. CRITERIA AND ASSUMPTIONS

DESIGN CRITERIA

The criteria for design of the Safety Class 3 structures is outlined in the SDC 4.1 rev. 11. According to LOI 9360642 rev. 2, the functional classification of the Electrical Equipment is SC 3.

In addition to SDC 4.1 rev 11 the Criteria is supplemented by "Specification for the Design of Cold-Formed Steel Structural Members" whenever cold formed sheet metal members, such as Unistrut channel etc., are used. Also, capacities for connections in Unistrut were extracted from manufacturers tests and design reports.

The design of expansion anchors was based on ref 5.

3. STRUCTURAL ANALYSIS.

The Electrical Equipment Supports consist of small, simple structures, that is why the manual Structural Analysis was chosen.

4. REFERENCES

1. "Manual of Steel Construction - Allowable Stress Design"
Ninth Edition, AISC 1989.
2. "Cold-Formed Steel Design Manual" August 19, 1986, Edition, AISI.
3. SDC 4.1 Rev. 11.
4. "Code requirements for Nuclear Safety Related Structures"
ACI 349-85.
5. A/E Standard GC-ANCR-01 Rev. 0.
6. UBC 1991.

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241-CR-106 - Rack for In Tank Imaging System CCTV - Figure 8 p. 38, (ref dwg. H-2-818704 sh 1 & H-2-818690).

Wind pressure ref. ASCE 7-93 "Minimum Design Loads for Buildings and Other Structures".

From SDC 4.2 rev. 11 the following are parameters used in wind pressure calculation for Safety Class 3 structures:

Basic wind speed: $V = 70$ mph
Importance Factor: $I = 1.07$
Exposure Category: C

$$K_z = 0.8$$

$$q_z = 0.00256 \cdot K_z \cdot \left(\frac{I \cdot V}{\text{mph}} \right)^2 \cdot \text{psf} \quad q_z = 11.49 \cdot \text{psf}$$

$$\text{use } q_z = 11.5 \cdot \text{psf}$$

$$\text{From Table 4, for other structures: } G_h = 1.32$$

$$\text{and from Table 13. } C_f = 1.6 \quad p_w = q_z \cdot G_h \cdot C_f \quad p_w = 24.29 \cdot \text{psf} \quad F_w = p_w \cdot A$$

Seismic load per UBC (36-1)

$$F_p = Z \cdot I \cdot C_p \cdot W_p$$

$$\text{From SDC 4.1 rev 11: } Z = 0.12 \quad I = 1.25 \quad C_p = 2.12 \quad a_h = Z \cdot I \cdot C_p \quad a_h = 0.32$$

$$\text{Box 01: } h_1 = 36\text{-in} \quad w_1 = 24\text{-in} \quad A_1 = h_1 \cdot w_1 \quad y_1 = 48\text{-in} \quad A_1 = 6.00 \cdot \text{ft}^2 \quad Q_1 = A_1 \cdot y_1$$

$$\text{Box 02: } h_2 = 24\text{-in} \quad w_2 = 24\text{-in} \quad A_2 = h_2 \cdot w_2 \quad y_2 = 54\text{-in} \quad A_2 = 4.00 \cdot \text{ft}^2 \quad Q_2 = A_2 \cdot y_2$$

$$\text{Box 03: } h_3 = 6\text{-in} \quad w_3 = 6\text{-in} \quad A_3 = h_3 \cdot w_3 \quad y_3 = 33\text{-in} \quad A_3 = 0.25 \cdot \text{ft}^2 \quad Q_3 = A_3 \cdot y_3$$

$$\text{Uni 04: } h_4 = 1.625\text{-in} \quad w_4 = 57\text{-in} \quad A_4 = h_4 \cdot w_4 \quad y_4 = 52.13\text{-in} \quad A_4 = 0.64 \cdot \text{ft}^2 \quad Q_4 = A_4 \cdot y_4$$

$$\text{Uni 05: } h_5 = 1.625\text{-in} \quad w_5 = 57\text{-in} \quad A_5 = h_5 \cdot w_5 \quad y_5 = 56.25\text{-in} \quad A_5 = 0.64 \cdot \text{ft}^2 \quad Q_5 = A_5 \cdot y_5$$

$$\text{Uni 06: } h_6 = 1.625\text{-in} \quad w_6 = 57\text{-in} \quad A_6 = h_6 \cdot w_6 \quad y_6 = 22\text{-in} \quad A_6 = 0.64 \cdot \text{ft}^2 \quad Q_6 = A_6 \cdot y_6$$

$$\text{TS 07: } h_7 = 66\text{-in} \quad w_7 = 3.0\text{-in} \quad A_7 = h_7 \cdot w_7 \quad y_7 = 78\text{-in} \quad A_7 = 1.37 \cdot \text{ft}^2 \quad Q_7 = A_7 \cdot y_7$$

$$\text{TS 08: } h_8 = 66\text{-in} \quad w_8 = 3.0\text{-in} \quad A_8 = h_8 \cdot w_8 \quad y_8 = 23\text{-in} \quad A_8 = 1.37 \cdot \text{ft}^2 \quad Q_8 = A_8 \cdot y_8$$

$$i = 1 \dots 8 \quad A = \sum_i A_i \quad A = 14.93 \cdot \text{ft}^2 \quad Q = \sum_i Q_i \quad Q = 61.25 \cdot \text{ft}^3 \quad y_o = \frac{Q}{A} \quad y_o = 4.10 \cdot \text{ft}$$

$$\text{for wind load computation increase area by 15\% } A_f = 1.15 \cdot A \quad A_f = 17.17 \cdot \text{ft}^2$$

Wind load therefore is:

$$F_w = q_z \cdot G_h \cdot C_f \cdot A_f \quad F_w = 417 \cdot \text{lb}$$

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By:

Overturning moment caused by wind forces will be amplified by weight eccentricity.

Distance from the center of the column to the back of electrical boxes is a:

$$a := \frac{3.25 \cdot \text{in}}{2} + 1.625 \cdot \text{in} \quad a = 3.25 \cdot \text{in}$$

$$W_1 := 200 \cdot \text{lb} \quad d_1 := 8 \cdot \text{in} \quad m_1 := W_1 \cdot \left(a + \frac{d_1}{2} \right) \quad W_2 := 100 \cdot \text{lb} \quad d_2 := 16 \cdot \text{in} \quad m_2 := W_2 \cdot \left(a + \frac{d_2}{2} \right)$$

$$W_3 := 5 \cdot \text{lb} \quad d_3 := 4 \cdot \text{in} \quad m_3 := W_3 \cdot \left(a + \frac{d_3}{2} \right)$$

$$m_{1to3} := m_1 + m_2 + m_3 \quad m_{1to3} = 2601.25 \cdot \text{lb} \cdot \text{in}$$

Moment due to unistrut eccentricity:

$$a_u := \frac{3.22 \cdot \text{in}}{2} + 0.709 \cdot \text{in} \quad a_u = 2.32 \cdot \text{in}$$

Unit weight of the unistrut and its total length are as follows:

$$w_u := 1.9 \cdot \frac{\text{lb}}{\text{ft}} \quad l_u := w_4 + w_5 + w_6 \quad l_u = 14.25 \cdot \text{ft}$$

$$m_{4to6} := w_u \cdot l_u \cdot a_u \quad m_{4to6} = 62.79 \cdot \text{lb} \cdot \text{in}$$

$$M_e := m_{1to3} + m_{4to6} \quad M_e = 222.00 \cdot \text{lb} \cdot \text{ft}$$

Check the foundation, 6'-6"x4'-0"x0'-8" deep pad: $a := 6.5 \cdot \text{ft} \quad b := 4 \cdot \text{ft} \quad d := 8 \cdot \text{in} \quad w_t := 9.42 \cdot \frac{\text{lb}}{\text{ft}}$

Assume density of concrete to be: $\gamma := 150 \cdot \text{pcf}$

$$\text{Dead Weight: } W_e := W_1 + W_2 + W_3 + (w_4 + w_5 + w_6) \cdot w_u + (h_6 + h_7) \cdot w_t \quad W_e = 385.16 \cdot \text{lb}$$

$$W_c := a \cdot b \cdot d \cdot \gamma$$

$$W_c = 2600.00 \cdot \text{lb}$$

Bearing capacity of the soil from UBC Table 29B for sandy gravel is:

$$p_a := 2000 \cdot \text{psf}$$

The load on the footing is:

$$W := W_e + W_c \quad W = 2985.16 \cdot \text{lb}$$

$$M := F_w \gamma_o + M_e \quad M = 1932.76 \cdot \text{ft} \cdot \text{lb}$$

Check for negative soil pressure:

$$p := \frac{W}{a \cdot b} - \frac{6 \cdot M}{a \cdot b^2}$$

$$p = 3.3 \cdot \text{psf}$$

uplift at the end of foundation will not occur.

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Find maximum soil pressure due to eccentricity:

$$p_{max} := \frac{W}{a \cdot b} + \frac{6 \cdot M}{a \cdot b^2} \quad p_{max} = 226.3 \cdot \text{psf}$$

 $p_{max} \leq p_a$ soil pressure is acceptable.

Check footing for overturning:

$$M_s := \frac{W \cdot b}{2} \quad FS_o := \frac{M_s}{M} \quad FS_o = 3.09 \quad FS_o \geq 1.5 \quad \text{stability of the foundation is satisfactory.}$$

Check TS3X3X0.1875 section for stress:

Bending about major axis:

$$M_x := \frac{M}{2} \quad M_x = 11596.58 \cdot \text{lbf} \cdot \text{in}$$

Bending about minor axis:

$$P_{wy} := (5.5 \cdot \text{ft} \cdot 3.25 \cdot \text{in} + 24 \cdot \text{in} \cdot 16 \cdot \text{in} + 12 \cdot \text{in} \cdot 8 \cdot \text{in}) \cdot p_w \quad P_{wy} = 117.1 \cdot \text{lbf}$$

$$P_{sy} := W_e \cdot a_h \quad P_{sy} = 122.5 \cdot \text{lbf}$$

$$M_y := P_{sy} \cdot y_o \quad M_y = 6029.7 \cdot \text{lbf} \cdot \text{in}$$

Properties for the TS3X3X0.1875 are as follows:

$$A := 2.02 \cdot \text{in}^2 \quad S_x := 1.73 \cdot \text{in}^3 \quad S_y := 1.73 \cdot \text{in}^3 \quad r_x := 1.13 \cdot \text{in} \quad r_y := 1.13 \cdot \text{in}$$

$$\sigma_x := \frac{M_x}{S_x} \quad \sigma_x = 6.7 \cdot \text{ksi} \quad \sigma_y := \frac{M_y}{S_y} \quad \sigma_y = 3.49 \cdot \text{ksi}$$

The stresses found above occur at different location of the member, also they are a result of two non-concurrent events, the combination is therefore not required.

Check members for axial load.

$$C := \frac{W_e}{2} \quad C = 192.58 \cdot \text{lbf}$$

$$I_x := 5 \cdot \text{ft} \quad K_x := 2 \quad Sr_x := \frac{K_x \cdot I_x}{r_x} \quad Sr_x = 106.19 \quad \text{Maximum slenderness ratio.}$$

$$I_y := 5 \cdot \text{ft} \quad K_y := 2 \quad Sr_y := \frac{K_y \cdot I_y}{r_y} \quad Sr_y = 106.19$$

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Check members for axial load - continued.

$$E := 29000 \text{ ksi} \quad F_y := 36 \text{ ksi} \quad C_c := \sqrt{\frac{2 \cdot \pi^2 \cdot E}{F_y}} \quad C_c = 126.10 \quad S_{r_x} \leq C_c$$

$$R := \frac{S_{r_x}}{C_c} \quad R = 0.84 \quad \Omega_c := \frac{5}{3} + \frac{3}{8} \cdot R - \frac{1}{8} \cdot R^3 \quad \Omega_c = 1.91$$

$$F_a := \left(1 - \frac{R^2}{2}\right) \cdot \frac{F_y}{\Omega_c} \quad F_a = 12.18 \text{ ksi}$$

$$P_a := A \cdot F_a \quad P_a = 24600.29 \text{ lbf} \quad P_a = 24600.29 \text{ lbf} \quad P_a \geq C \quad \text{O.K.}$$

Check Base Plate for capacity.

Check expansion anchors:

$$l_1 := 2.125 \text{ in} \quad l_{01} := 1.5 \text{ in} + 2 \cdot 0.5 \text{ in} + 5 \text{ in} - 1.25 \text{ in} - 0.875 \text{ in} \quad l_{01} = 5.38 \text{ in}$$

$$l_2 := 2.125 \text{ in} \quad l_{02} := 1.5 \text{ in} + 2 \cdot 0.5 \text{ in} + 5 \text{ in} - 1.25 \text{ in} - 0.875 \text{ in} \quad l_{02} = 5.38 \text{ in}$$

$$M := M_x - \frac{W_e \cdot d_1}{2} \quad M = 10.06 \text{ kips-in}$$

$$M = T_{\max} \left[l_{01} + \left(\frac{l_1}{l_2} \right)^2 \cdot l_{02} \right]$$

$$T_{\max} := \frac{M}{l_{01} + \left(\frac{l_1}{l_2} \right)^2 \cdot l_{02}} \quad T_{\max} = 935.44 \text{ lbf} \quad T_{\min} := T_{\max} \left(\frac{l_1}{l_2} \right)^2 \quad T_{\min} = 935.44 \text{ lbf}$$

$$T_b := T_{\max} \quad T_b = 935.44 \text{ lbf} \quad V_b := \frac{F_w}{8} \quad V_b = 52.13 \text{ lbf}$$

For 5/8" dia HKB II with 2 3/4" min. embedment, installed in $f'_c = 4000 \text{ psi}$ concrete, allowable loads are:

$$T_a := 2000 \text{ lbf} \quad V_a := 2530 \text{ lbf} \quad \text{Ref. A/E Standard GC-ANCR-01 rev 0 table 21.}$$

Interaction factor is:

$$IF := \frac{T_b}{T_a} + \frac{V_b}{V_a} \quad IF = 0.49 \quad IF \leq 1.0 \quad \text{O.K.}$$

Check stress in the base plate:

$$t_{pl} := 0.5 \text{ in} \quad f_b := \frac{6 \cdot T_b}{t_{pl}^2} \quad f_b = 22.45 \text{ ksi} \quad F_b := \frac{4}{3} \cdot (0.75 \cdot F_y) \quad F_b = 36.00 \text{ ksi} \quad f_b \leq F_b \quad \text{O.K.}$$

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Process Building Electrical Rack #1 ER-1361 - Figure 6, page 36, (ref dwg H-2-818678 sheets 1 and 5).

The analysis of the rack and its foundation will include an influence of the environmental loads i.e.. Wind and Seismic. The loads are calculated on the basis outlined in UBC.

Wind Load.

From observation, the critical wind exposure appears on the frontal area of the rack (maximum Area of exposure). The following is the calculation of the exposure area. It will include overlaps of the unistrut that will compensate for the area of exposed conduit.

Box 01:	$h_1 := 21.5\text{-in}$	$w_1 := 9\text{-in}$	$A_1 := h_1 \cdot w_1$	$y_1 := 54.5\text{-in}$	$A_1 = 1.34 \cdot \text{ft}^2$	$Q_1 := A_1 \cdot y_1$
Box 02:	$h_2 := 15.5\text{-in}$	$w_2 := 8\text{-in}$	$A_2 := h_2 \cdot w_2$	$y_2 := 57.5\text{-in}$	$A_2 = 0.86 \cdot \text{ft}^2$	$Q_2 := A_2 \cdot y_2$
Box 03:	$h_3 := 15.5\text{-in}$	$w_3 := 8\text{-in}$	$A_3 := h_3 \cdot w_3$	$y_3 := 57.5\text{-in}$	$A_3 = 0.86 \cdot \text{ft}^2$	$Q_3 := A_3 \cdot y_3$
Box 04:	$h_4 := 26.25\text{-in}$	$w_4 := 10\text{-in}$	$A_4 := h_4 \cdot w_4$	$y_4 := 52.13\text{-in}$	$A_4 = 1.82 \cdot \text{ft}^2$	$Q_4 := A_4 \cdot y_4$
Box 05:	$h_5 := 18\text{-in}$	$w_5 := 9\text{-in}$	$A_5 := h_5 \cdot w_5$	$y_5 := 56.25\text{-in}$	$A_5 = 1.12 \cdot \text{ft}^2$	$Q_5 := A_5 \cdot y_5$
Box 06:	$h_6 := 14\text{-in}$	$w_6 := 12\text{-in}$	$A_6 := h_6 \cdot w_6$	$y_6 := 22\text{-in}$	$A_6 = 1.17 \cdot \text{ft}^2$	$Q_6 := A_6 \cdot y_6$
Box 07:	$h_7 := 12\text{-in}$	$w_7 := 12\text{-in}$	$A_7 := h_7 \cdot w_7$	$y_7 := 78\text{-in}$	$A_7 = 1.00 \cdot \text{ft}^2$	$Q_7 := A_7 \cdot y_7$
Box 08:	$h_8 := 12\text{-in}$	$w_8 := 12\text{-in}$	$A_8 := h_8 \cdot w_8$	$y_8 := 23\text{-in}$	$A_8 = 1.00 \cdot \text{ft}^2$	$Q_8 := A_8 \cdot y_8$
Box 09:	$h_9 := 7\text{-in}$	$w_9 := 8\text{-in}$	$A_9 := h_9 \cdot w_9$	$y_9 := 38.5\text{-in}$	$A_9 = 0.39 \cdot \text{ft}^2$	$Q_9 := A_9 \cdot y_9$
Box 10:	$h_{10} := 7\text{-in}$	$w_{10} := 8\text{-in}$	$A_{10} := h_{10} \cdot w_{10}$	$y_{10} := 38.5\text{-in}$	$A_{10} = 0.39 \cdot \text{ft}^2$	$Q_{10} := A_{10} \cdot y_{10}$
Box 11:	$h_{11} := 6\text{-in}$	$w_{11} := 48\text{-in}$	$A_{11} := h_{11} \cdot w_{11}$	$y_{11} := 23\text{-in}$	$A_{11} = 2.00 \cdot \text{ft}^2$	$Q_{11} := A_{11} \cdot y_{11}$
Box 12:	$h_{12} := 19.5\text{-in}$	$w_{12} := 9.12\text{-in}$	$A_{12} := h_{12} \cdot w_{12}$	$y_{12} := 55.5\text{-in}$	$A_{12} = 1.23 \cdot \text{ft}^2$	$Q_{12} := A_{12} \cdot y_{12}$
Uni 13:	$h_{13} := 1.625\text{-in}$	$w_{13} := 85\text{-in}$	$A_{13} := h_{13} \cdot w_{13}$	$y_{13} := 83\text{-in}$	$A_{13} = 0.96 \cdot \text{ft}^2$	$Q_{13} := A_{13} \cdot y_{13}$
Uni 14:	$h_{14} := 1.625\text{-in}$	$w_{14} := 46\text{-in}$	$A_{14} := h_{14} \cdot w_{14}$	$y_{14} := 73\text{-in}$	$A_{14} = 0.52 \cdot \text{ft}^2$	$Q_{14} := A_{14} \cdot y_{14}$
Uni 15:	$h_{15} := 1.625\text{-in}$	$w_{15} := 85\text{-in}$	$A_{15} := h_{15} \cdot w_{15}$	$y_{15} := 65.25\text{-in}$	$A_{15} = 0.96 \cdot \text{ft}^2$	$Q_{15} := A_{15} \cdot y_{15}$
Uni 16:	$h_{16} := 1.625\text{-in}$	$w_{16} := 85\text{-in}$	$A_{16} := h_{16} \cdot w_{16}$	$y_{16} := 51.75\text{-in}$	$A_{16} = 0.96 \cdot \text{ft}^2$	$Q_{16} := A_{16} \cdot y_{16}$
Uni 17:	$h_{17} := 1.625\text{-in}$	$w_{17} := 46\text{-in}$	$A_{17} := h_{17} \cdot w_{17}$	$y_{17} := 47.75\text{-in}$	$A_{17} = 0.52 \cdot \text{ft}^2$	$Q_{17} := A_{17} \cdot y_{17}$
Uni 18:	$h_{18} := 1.625\text{-in}$	$w_{18} := 46\text{-in}$	$A_{18} := h_{18} \cdot w_{18}$	$y_{18} := 41\text{-in}$	$A_{18} = 0.52 \cdot \text{ft}^2$	$Q_{18} := A_{18} \cdot y_{18}$
Uni 19:	$h_{19} := 1.625\text{-in}$	$w_{19} := 46\text{-in}$	$A_{19} := h_{19} \cdot w_{19}$	$y_{19} := 36\text{-in}$	$A_{19} = 0.52 \cdot \text{ft}^2$	$Q_{19} := A_{19} \cdot y_{19}$
Uni 20:	$h_{20} := 1.625\text{-in}$	$w_{20} := 85\text{-in}$	$A_{20} := h_{20} \cdot w_{20}$	$y_{20} := 28\text{-in}$	$A_{20} = 0.96 \cdot \text{ft}^2$	$Q_{20} := A_{20} \cdot y_{20}$
Uni 21:	$h_{21} := 1.625\text{-in}$	$w_{21} := 85\text{-in}$	$A_{21} := h_{21} \cdot w_{21}$	$y_{21} := 24\text{-in}$	$A_{21} = 0.96 \cdot \text{ft}^2$	$Q_{21} := A_{21} \cdot y_{21}$

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Uni 22:	$h_{22} := 1.625 \cdot \text{in}$	$w_{22} := 85 \cdot \text{in}$	$A_{22} := h_{22} \cdot w_{22}$	$y_{22} := 18 \cdot \text{in}$	$A_{22} = 0.96 \cdot \text{ft}^2$	$Q_{22} := A_{22} \cdot y_{22}$
Col 23:	$h_{23} := 90 \cdot \text{in}$	$w_{23} := 3 \cdot \text{in}$	$A_{23} := h_{23} \cdot w_{23}$	$y_{23} := 45 \cdot \text{in}$	$A_{23} = 1.87 \cdot \text{ft}^2$	$Q_{23} := A_{23} \cdot y_{23}$
Col 24:	$h_{24} := 90 \cdot \text{in}$	$w_{24} := 3 \cdot \text{in}$	$A_{24} := h_{24} \cdot w_{24}$	$y_{24} := 45 \cdot \text{in}$	$A_{24} = 1.87 \cdot \text{ft}^2$	$Q_{24} := A_{24} \cdot y_{24}$
Col 25:	$h_{25} := 90 \cdot \text{in}$	$w_{25} := 3 \cdot \text{in}$	$A_{25} := h_{25} \cdot w_{25}$	$y_{25} := 45 \cdot \text{in}$	$A_{25} = 1.87 \cdot \text{ft}^2$	$Q_{25} := A_{25} \cdot y_{25}$

$j := 1 \dots 25$

$$A := \sum_j A_j \quad A = 26.65 \cdot \text{ft}^2 \quad Q := \sum_j Q_j \quad Q = 101.37 \cdot \text{ft}^3$$

$$y_o := \frac{Q}{A} \quad y_o = 45.65 \cdot \text{in} \quad \text{for wind load computation increase area by 25\%} \quad A_f := 1.25 \cdot A \quad A_f = 33.31 \cdot \text{ft}^2$$

Wind pressure ref. ASCE 7-93 "Minimum Design Loads for Buildings and Other Structures".

Basic wind speed: $V := 70 \cdot \text{mph}$

Importance Factor: $I := 1.07$

Exposure Category: C

$$K_z := 0.8$$

$$q_z := 0.00256 \cdot K_z \cdot \left(\frac{V}{\text{mph}} \right)^2 \cdot \text{psf} \quad q_z = 11.49 \cdot \text{psf}$$

$$\text{use} \quad q_z := 11.5 \cdot \text{psf}$$

From Table 4. for other structures: $G_h := 1.32$

and from table 13: $C_f := 1.6$

Wind load therefore is: $F_w := q_z \cdot G_h \cdot C_f \cdot A_f \quad F_w = 809.1 \cdot \text{lbf}$

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By: M.B. Lasota

By

By

Overtuning moment caused by wind forces will be amplified by weight eccentricity.

Distance from the center of the column to the back of electrical boxes is:

$$a := \frac{3\text{-in}}{2} + 1.625\text{-in} \quad a = 3.12\text{-in}$$

$$W_1 := 20\text{-lbf} \quad d_1 := 7\text{-in} \quad m_1 := W_1 \cdot \left(a + \frac{d_1}{2}\right) \quad W_2 := 20\text{-lbf} \quad d_2 := 5\text{-in} \quad m_2 := W_2 \cdot \left(a + \frac{d_2}{2}\right)$$

$$W_3 := 20\text{-lbf} \quad d_3 := 5\text{-in} \quad m_3 := W_3 \cdot \left(a + \frac{d_3}{2}\right) \quad W_4 := 50\text{-lbf} \quad d_4 := 8\text{-in} \quad m_4 := W_4 \cdot \left(a + \frac{d_4}{2}\right)$$

$$W_5 := 20\text{-lbf} \quad d_5 := 6\text{-in} \quad m_5 := W_5 \cdot \left(a + \frac{d_5}{2}\right) \quad W_6 := 10\text{-lbf} \quad d_6 := 6\text{-in} \quad m_6 := W_6 \cdot \left(a + \frac{d_6}{2}\right)$$

$$W_7 := 20\text{-lbf} \quad d_7 := 6\text{-in} \quad m_7 := W_7 \cdot \left(a + \frac{d_7}{2}\right) \quad W_8 := 20\text{-lbf} \quad d_8 := 6\text{-in} \quad m_8 := W_8 \cdot \left(a + \frac{d_8}{2}\right)$$

$$W_9 := 10\text{-lbf} \quad d_9 := 7\text{-in} \quad m_9 := W_9 \cdot \left(a + \frac{d_9}{2}\right) \quad W_{10} := 10\text{-lbf} \quad d_{10} := 7\text{-in} \quad m_{10} := W_{10} \cdot \left(a + \frac{d_{10}}{2}\right)$$

$$W_{11} := 25\text{-lbf} \quad d_{11} := 6\text{-in} \quad m_{11} := W_{11} \cdot \left(a + \frac{d_{11}}{2}\right) \quad W_{12} := 20\text{-lbf} \quad d_{12} := 5\text{-in} \quad m_{12} := W_{12} \cdot \left(a + \frac{d_{12}}{2}\right)$$

$$m_{1to12} := m_1 + m_2 + m_3 + m_4 + m_5 + m_6 + m_7 + m_8 + m_9 + m_{10} + m_{11} + m_{12} \quad m_{1to12} = 1540.63\text{-lbf-in}$$

Moment due to unistrut eccentricity:

$$a_u := \frac{3\text{-in}}{2} + 0.709\text{-in} \quad a_u = 2.21\text{-in}$$

Unit weight of the unistrut and its total length are as follows:

$$w_u := 1.9 \frac{\text{lbf}}{\text{ft}} \quad l_u := \sum_{j=13}^{22} w_j \quad l_u = 57.83\text{-ft}$$

$$m_{13to22} := w_u \cdot l_u \cdot a_u \quad m_{13to22} = 242.73\text{-lbf-in}$$

$$M_e := m_{1to12} + m_{13to22} \quad M_e = 1783.36\text{-lbf-in}$$

Adjusted wind load by amount of weight eccentricity

$$F := F_w + \frac{M_e}{y_o} \quad F = 848.16\text{-lbf}$$

$$\text{Use} \quad F := 850\text{-lbf}$$

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DESIGN ANALYSIS

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Location: 241-C-106 & 241-AY-102

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By: K.S.

By

From Shannon & Wilson inc. "Final Report of Geotechnical Studies W 320 Waste Retrieval and Sluicing System
200 East Area, Hanford Site", April 1994, soil properties are:

$$\gamma := 115 \cdot \text{pcf} \quad \phi := 37 \cdot \text{deg}$$

Rankine passive lateral earth pressure is:

$$p := \frac{1 + \sin(\phi)}{1 - \sin(\phi)} \cdot \gamma \quad p = 462.62 \cdot \frac{\text{psf}}{\text{ft}}$$

From UBC 2907 6 (g) 2.

$$S_1 := p$$

$$b := 15 \cdot \text{in} \quad h := y_o \quad h = 45.65 \cdot \text{in}$$

$$P := \frac{F}{3}$$

$$\alpha := 3 \cdot 2.34 \cdot \frac{P}{b \cdot S_1} \quad \alpha = 3.43 \cdot \text{ft}^2$$

Assume: $d := 34.68638 \cdot \text{in}$

$$A_o := \frac{\alpha}{d} \quad A_o = 1.19 \cdot \text{ft}$$

$$u := \frac{A_o}{2} \cdot \left(1 + \sqrt{1 + \frac{4.36 \cdot h}{A_o}} \right) \quad u = 34.68638 \cdot \text{in}$$

Use embedment depth: 3'-0"

Check stress in TS3X3X0.1875:

$$M := \frac{F \cdot y_o}{3} \quad M = 12.91 \cdot \text{kips} \cdot \text{in} \quad S_t := 1.73 \cdot \text{in}^3 \quad f_b := \frac{M}{S_t} \quad f_b = 7.46 \cdot \text{ksi}$$

Total weight of the rack and equipment is:

$$C := W_1 + W_2 + W_3 + W_4 + W_5 + W_6 + W_7 + W_8 + W_9 + W_{10} + W_{11} + W_{12} + w_u \cdot l_u + 3 \cdot 90 \cdot \text{in} \cdot 6.87 \cdot \frac{\text{lb}}{\text{ft}}$$

$$C = 509.46 \cdot \text{lb}$$

Compression in the column is:

$$P_a := \frac{C - 3 \cdot 90 \cdot \text{in} \cdot 6.87 \cdot \frac{\text{lb}}{\text{ft}}}{2} + 90 \cdot \text{in} \cdot 6.87 \cdot \frac{\text{lb}}{\text{ft}} \quad P_a = 228.97 \cdot \text{lb}$$

$$f_a := \frac{P_a}{2.02 \cdot \text{in}^2} \quad f_a = 0.11 \cdot \text{ksi}$$

From individual stress levels it is apparent that the interaction equation is satisfied.

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Date: 10/17/95 By: M.B. Lasota
Checked: 12/19/95 By: R.W. Davidson
Revised: 5/05/97 By: M.B. Lasota
Revised: 1/16/98 By: A. S. Langevin *ask*

241-AY-102 - Racks for electrical enclosures: CTB-0621- Figure 3, (ref dwg. H-2-818695 sh 2 & 4), ITB-0621- Figure 4, (ref dwg. H-2-818695 sh 2 & 4), PTBX-0621- Figure 5, (ref dwg. H-2-818695 sh 2 & 5), NEMA 3R 600A SAFETY SWITCH- Figure 1, (ref dwg. H-2-818695 sh 1 & 4), ***NEMA 3R 60A SAFETY SWITCH (PIT COOLING) - Figure 2, (ref dwg. H-2-818695 sh 2 & 4), LKD-102-32 - Figure 15, (ref dwg. H-2-818695 sh 4) and LD-1363/1366 - Figure 16 (ref dwg. H-2-818704 sh 1). *Δ1

241-CR-106 - Racks for electrical enclosures: TB-1- Figure 9, (ref dwg. H-2-818704 sh 2), TB-2 & TB-3- Figure 7, (ref dwg. H-2-818704 sh 3) and TB-4 and LD-1364/1368- Figure 10, (ref dwg. H-2-818702 sh 1 & 2). *Δ1

241-C - Rack for Standby Generator Disc Pad (ref dwg. H-2-818674 and ECN W-320-682) *Δ2

Wind pressure ref. ASCE 7-93 "Minimum Design Loads for Buildings and Other Structures".
From SDC 4.2 rev. 11 the following are parameters used in wind pressure calculation for Safety Class 3 structures:

Basic wind speed: $V := 70 \text{ mph}$
Importance Factor: $I := 1.07$
Exposure Category: C
 $K_z := 0.8$

$$q_z := 0.00256 \cdot K_z \left(\frac{I \cdot V}{\text{mph}} \right)^2 \cdot \text{psf} \quad q_z = 11.49 \cdot \text{psf}$$

use $q_z := 11.5 \cdot \text{psf}$

From Table 4. for other structures: $G_h = 1.32$

and from table 13. $C_f = 1.6 \quad p_w := q_z \cdot G_h \cdot C_f \quad p_w = 24.29 \cdot \text{psf} \quad F_w = p_w \cdot A$

Seismic load per UBC (36-1) $F_p = Z \cdot I \cdot C_p \cdot W_p$

From SDC 4.1 rev 11: $Z := 0.12 \quad I = 1.25 \quad C_p := 2.12 \quad a_h := Z \cdot I \cdot C_p \quad a_h = 0.32 \quad i := 1, 2, 10$

CTB-0621 $h_1 = 20 \cdot \text{in} \quad w_1 := 16 \cdot \text{in} \quad A_1 := h_1 \cdot w_1 \quad y_1 := 54.5 \cdot \text{in} \quad A_1 = 2.22 \cdot \text{ft}^2 \quad W_1 := 40 \cdot \text{lb}$

$$F_{w_1} := p_w \cdot A_1 \quad F_{p_1} := a_h \cdot W_1 \quad F_{w_1} = 54 \cdot \text{lb} \quad F_{p_1} = 13 \cdot \text{lb}$$

ITB-0621 $h_2 := 24 \cdot \text{in} \quad w_2 := 30 \cdot \text{in} \quad A_2 := h_2 \cdot w_2 \quad y_2 := 57.5 \cdot \text{in} \quad A_2 = 5.00 \cdot \text{ft}^2 \quad W_2 := 75 \cdot \text{lb}$

$$F_{w_2} := p_w \cdot A_2 \quad F_{p_2} := a_h \cdot W_2 \quad F_{w_2} = 121 \cdot \text{lb} \quad F_{p_2} = 24 \cdot \text{lb}$$

NEMA 3R 600A S.S. $h_3 := 60.75 \cdot \text{in} \quad w_3 := 25 \cdot \text{in} \quad A_3 := h_3 \cdot w_3 \quad y_3 := 47.63 \cdot \text{in} \quad A_3 = 10.55 \cdot \text{ft}^2 \quad W_3 := 109 \cdot \text{lb}$

$$F_{w_3} := p_w \cdot A_3 \quad F_{p_3} := a_h \cdot W_3 \quad F_{w_3} = 256 \cdot \text{lb} \quad F_{p_3} = 35 \cdot \text{lb}$$

Standby Generator Disc - dimensions are similar to NEMA 3R 600A S. S. - Use the same analysis *Δ2

NEMA 3R 60A S.S. $h_4 := 20 \cdot \text{in} \quad w_4 := 8.5 \cdot \text{in} \quad A_4 := h_4 \cdot w_4 \quad y_4 := 52.13 \cdot \text{in} \quad A_4 = 1.18 \cdot \text{ft}^2 \quad W_4 := 20 \cdot \text{lb}$

$$F_{w_4} := p_w \cdot A_4 \quad F_{p_4} := a_h \cdot W_4 \quad F_{w_4} = 29 \cdot \text{lb} \quad F_{p_4} = 6 \cdot \text{lb}$$

***The above NEMA 3R 60A S.S. rack had been modified to include a Pneumatic Valve Actuator Control Station.
Analysis of modified rack can be found on page 28 of these calculations. *Δ1

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By: R.W. Davidson

By: M.B. Lasota

PTBX-0621	$h_5 := 30\text{-in}$	$w_5 := 24\text{-in}$	$A_5 := h_5 \cdot w_5$	$y_5 := 56.25\text{-in}$	$A_5 = 5.00\text{-ft}^2$	$W_5 := 75\text{-lbf}$	$F_{w_5} := p_w \cdot A_5$	$F_{p_5} := a_h \cdot W_5$	$F_{w_5} = 121\text{-lbf}$	$F_{p_5} = 24\text{-lbf}$
TB-1	$h_6 := 24\text{-in}$	$w_6 := 24\text{-in}$	$A_6 := h_6 \cdot w_6$	$y_6 := 30\text{-in}$	$A_6 = 4.00\text{-ft}^2$	$W_6 := 50\text{-lbf}$	$F_{w_6} := p_w \cdot A_6$	$F_{p_6} := a_h \cdot W_6$	$F_{w_6} = 97\text{-lbf}$	$F_{p_6} = 16\text{-lbf}$
TB-2&TB-3	$h_7 := 30\text{-in}$	$w_7 := 30\text{-in}$	$A_7 := h_7 \cdot w_7$	$y_7 := 23\text{-in}$	$A_7 = 6.25\text{-ft}^2$	$W_7 := 50\text{-lbf}$	$F_{w_7} := p_w \cdot A_7$	$F_{p_7} := 2 \cdot a_h \cdot W_7$	$F_{w_7} = 152\text{-lbf}$	$F_{p_7} = 32\text{-lbf}$
TB-4& LD-1364/1368	$h_8 := 24\text{-in}$	$w_8 := 30\text{-in}$	$A_8 := h_8 \cdot w_8$	$y_8 := 26\text{-in}$	$A_8 = 5.00\text{-ft}^2$	$W_8 := 100\text{-lbf}$	$F_{w_8} := p_w \cdot A_8$	$F_{p_8} := a_h \cdot W_8$	$F_{w_8} = 121\text{-lbf}$	$F_{p_8} = 32\text{-lbf}$
LKD-102-32	$h_9 := 16\text{-in}$	$w_9 := 16\text{-in}$	$A_9 := h_9 \cdot w_9$	$y_9 := 27\text{-in}$	$A_9 = 1.78\text{-ft}^2$	$W_9 := 30\text{-lbf}$	$F_{w_9} := p_w \cdot A_9$	$F_{p_9} := a_h \cdot W_9$	$F_{w_9} = 43\text{-lbf}$	$F_{p_9} = 10\text{-lbf}$
LD-1364/1368	$h_{10} := 24\text{-in}$	$w_{10} := 30\text{-in}$	$A_{10} := h_{10} \cdot w_{10}$	$y_{10} := 32\text{-in}$	$A_{10} = 5.00\text{-ft}^2$	$W_{10} := 30\text{-lbf}$	$F_{w_{10}} := p_w \cdot A_{10}$	$F_{p_{10}} := a_h \cdot W_{10}$	$F_{w_{10}} = 121\text{-lbf}$	$F_{p_{10}} = 10\text{-lbf}$

$\Delta 1$

$\Delta 1$

$\Delta 1$

$\Delta 1$

$\Delta 1$

The maximum load occurs at 600A Safety Switch.

$$P_w := F_{w_3} \quad P_w = 256.16\text{-lbf} \quad \text{For simplicity and added conservatism use: } P_w := 275\text{-lbf}$$

Seismic load is also controlling at 600A Safety Switch.

$$P_e := F_{p_3} \quad P_e = 35\text{-lbf} \quad \text{Use: } P_e := 40\text{-lbf}$$

Check the foundation, 3'-0"x4'-0"x0'-8" deep pad: $a := 4\text{-ft}$ $b := 3\text{-ft}$ $d := 8\text{-in}$

Average density of reinforced concrete is: $\gamma := 150\text{-pcf}$

$$\text{Dead Weight} \quad W_e := W_3 + 2 \cdot 1.9 \cdot \frac{\text{lbf}}{\text{ft}} \cdot (2 \cdot 73.5\text{-in} + 26\text{-in}) + 2 \cdot 4.08\text{-lbf} \quad W_e = 82.94\text{-lbf}$$

$$W_c := a \cdot b \cdot d \cdot \gamma \quad W_c = 1200.00\text{-lbf}$$

Bearing capacity of the soil from UBC Table 29B for sandy gravel is: $p_a := 2000\text{-psf}$

The load on the footing is:

$$W := W_e + W_c \quad W = 1282.94\text{-lbf} \quad \text{say} \quad W := 1400\text{-lbf}$$

$$M := P_w \cdot 5\text{-ft} \quad M = 1375.00\text{-ft}\cdot\text{lbf}$$

$\Delta 1$

$\Delta 1$

$\Delta 1$

$\Delta 1$

$\Delta 1$

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Location: 241-C-106 & 241-AY-102

WO/Job No.: ER 4319/W 320
Date: 10/17/95 By: M.B. Lasota
Checked: 12/19/95 By: R.W. Davidson
Revised: 5/05/97 By: M.B. Lasota *Mue*

Check for negative soil pressure:

$$p := \frac{W}{a \cdot b} - \frac{6 \cdot M}{a \cdot b^2} \quad p = -112.5 \cdot \text{psf} \quad \text{uplift at the end of foundation will occur.}$$

Find actual pressure distribution due to eccentricity:

$$\Sigma M = 0$$

$$b_1 := \left(\frac{b}{2} - \frac{M}{W} \right) \cdot 3 \quad b_1 = 1.55 \cdot \text{ft}$$

$$\Sigma Y = 0$$

$$p_{\max} := \frac{2 \cdot W}{a \cdot b_1} \quad p_{\max} = 450.6 \cdot \text{psf} \quad p_{\max} \leq p_a \quad \text{soil pressure is acceptable.}$$

Check footing for overturning:

$$M_s := \frac{W \cdot b}{2} \quad FS_o := \frac{M_s}{M}$$

$$FS_o = 1.53 \quad FS_o \geq 1.5 \quad \text{stability of the foundation is satisfactory.}$$

Check unistrut section for stress:

Bending about major axis:

$$M_x := \frac{M}{2} \quad M_x = 8250.00 \cdot \text{lbf} \cdot \text{in}$$

Bending about minor axis:

$$P_{wy} := (7 \cdot \text{ft} \cdot 3.25 \cdot \text{in} + 12.5 \cdot \text{in} \cdot 60.75 \cdot \text{in}) \cdot p_w$$

$$P_{wy} = 174.1 \cdot \text{lbf}$$

$$P_{sy} := W_e \cdot a_h$$

$$P_{sy} = 26.4 \cdot \text{lbf}$$

$$M_y := P_{wy} \left(7 \cdot \text{ft} - \frac{60.75 \cdot \text{in}}{2} \right) \cdot 0.5 \quad M_y = 4668.8 \cdot \text{lbf} \cdot \text{in}$$

Properties for the channel P1001 are as follows:

$$A = 1.11 \cdot \text{in}^2 \quad S_x = 0.572 \cdot \text{in}^3 \quad S_y = 0.588 \cdot \text{in}^3 \quad r_x = 0.915 \cdot \text{in} \quad r_y = 0.656 \cdot \text{in}$$

$$\sigma_x := \frac{M_x}{S_x} \quad \sigma_x = 14.42 \cdot \text{ksi} \quad \sigma_y := \frac{M_y}{S_y} \quad \sigma_y = 7.94 \cdot \text{ksi}$$

The stresses found above occur at different location of the member, also they are a result of two non-concurrent events, the combination is therefore not required.

Check members for axial load.

$$C := \frac{W_e}{2} \quad C = 41.5 \cdot \text{lbf}$$

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By: R.W. Davidson

By: M.B. Lasota

$$I_x := 5\text{-ft} \quad K_x := 2 \quad S_{r_x} := \frac{K_x \cdot I_x}{r_x} \quad S_{r_x} = 131.15 \quad \text{Maximum slenderness ratio.}$$

$$I_y := 5\text{-ft} - \frac{41\text{-in}}{2} \quad K_y := 2 \quad S_{r_y} := \frac{K_y \cdot I_y}{r_y} \quad S_{r_y} = 120.43$$

$$E := 29000\text{-ksi} \quad F_y := 33\text{-ksi} \quad \frac{F_y}{2} = 16.50\text{-ksi}$$

$$F_{ex} := \frac{\pi^2 \cdot E}{S_{r_x}^2} \quad F_{ex} = 16.64\text{-ksi} \quad F_{ex} \geq \frac{F_y}{2} \quad F_n := \left(1 - \frac{F_y}{4 \cdot F_{ex}}\right) \cdot F_y \quad F_n = 16.64\text{-ksi}$$

$$R := \sqrt{\frac{F_y}{2 \cdot F_{ex}}} \quad R = 0.995757 \quad \Omega_c := \frac{5}{3} + \frac{3}{8} \cdot R - \frac{1}{8} \cdot R^3 \quad \Omega_c = 1.92$$

$$P_n := A \cdot F_n \quad P_n = 18470.10\text{-lbf} \quad P_a := \frac{P_n}{\Omega_c} \quad P_a = 9636.61\text{-lbf} \quad P_a \geq C \quad \text{O.K.}$$

Check standard Base for capacity.

Unistrut nut slip capacity is (ref Unistrut catalog No. 11 page 9):

$$V_1 := 1500\text{-lbf}$$

$$V_a := 2 \cdot V_1 \quad V_a = 3000.00\text{-lbf}$$

Distance between centers of the bolts for part P 2073 A is: $D := 3.25\text{-in}$

Shear in the bolt due to moment is:

$$V := \frac{M_x}{D} \quad V = 2538.46\text{-lbf} \quad V \leq V_a \quad \text{Part P 2073 A is adequate.}$$

Check expansion anchors:

$$d := 3\text{-in} + 0.8 \cdot 1.625\text{-in} + 2 \cdot 0.25\text{-in} \quad d = 4.80\text{-in}$$

$$T_b := \frac{M_x}{d} \quad T_b = 1718.75\text{-lbf} \quad V_b := \frac{P_w}{8} \quad V_b = 34.38\text{-lbf}$$

For 5/8" dia HKB II with 2 3/4" min embedment, installed in $f'_c = 4000\text{psi}$ allowable loads are:

$$T_a := 2000\text{-lbf} \quad V_a := 2530\text{-lbf} \quad \text{Ref. A/E Standard GC-ANCR-01 rev. 0, table 21.}$$

Interaction factor is:

$$IF := \frac{T_b}{T_a} + \frac{V_b}{V_a} \quad IF = 0.87 \quad IF \leq 1.0 \quad \text{O.K.}$$

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DESIGN ANALYSIS

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WO/Job No.: ER 4319W 320

Date: 04/18/1996 By: M.B. Lasota

Checked: A. J. Davidson By: R.W. Davidson

Revised By

Location: 241-C-106 & 241-AY-102

During installation the minimum edge distance at ITB-0621 could not be satisfied. The following is an evaluation of that particular case.

Recall: $F_{w2} = 121.44 \cdot \text{lbf}$

$$M := F_{w2} \cdot 2.5 \cdot \text{ft} \quad M = 3643.20 \cdot \text{lbf} \cdot \text{in} \quad M_x := \frac{M}{2} \quad M_x = 1821.60 \cdot \text{lbf} \cdot \text{in}$$

Check expansion anchors:

$$d := 3 \cdot \text{in} + 0.8 \cdot 1.625 \cdot \text{in} + 2 \cdot 0.25 \cdot \text{in} \quad d = 4.80 \cdot \text{in}$$

$$T_b := \frac{M_x}{d} \quad T_b = 379.50 \cdot \text{lbf} \quad V_b := \frac{F_{w2}}{8} \quad V_b = 15.18 \cdot \text{lbf}$$

For 5/8" dia HKB II with 2 3/4" min embedment, installed in $f'_c = 4000 \text{ psi}$ allowable loads are:

$$T_a := 2000 \cdot \text{lbf} \quad V_a := 2530 \cdot \text{lbf} \quad \text{Ref. A/E Standard GC-ANCR-01 rev. 0, table 21.}$$

Due to violation of the edge distance reduction in anchor capacity is necessary:

Reduction factor in tension

$$e_{\min} := 2.75 \cdot \text{in} \quad e_{\max} := 6.25 \cdot \text{in} \quad e := 5.5 \cdot \text{in} \quad r_t := \frac{e + e_{\max} - 2 \cdot e_{\min}}{2 \cdot (e_{\max} - e_{\min})} \quad r_t = 0.89$$

Reduction factor in shear

$$e_{\min} := 4.125 \cdot \text{in} \quad e_{\max} := 8.25 \cdot \text{in} \quad e := 5.5 \cdot \text{in} \quad r_v := \frac{e + e_{\max} - 2 \cdot e_{\min}}{2 \cdot (e_{\max} - e_{\min})} \quad r_v = 0.67$$

Interaction factor is:

$$IF := \frac{T_b}{r_t T_a} + \frac{V_b}{r_v V_a} \quad IF = 0.22 \quad IF \leq 1.0 \quad \text{ok.}$$

Client: WHC

Subject: Structural analysis of Safety Related
Electrical Equipment Supports.

Location: 241-C-106 & 241-AY-102

WO/Job No.: ER 4319/W 320

Date: 5/05/97

Checked 5/8/97

Revised

By: M.B. Lasota

By: M.R. Custer

By

MLC

Although the maximum load occurs at 600A Safety Switch, the foundation supplied is larger than the rest of the pads.
The second largest load occurs at the rack for TB4&LD-1364/1368. This rack is located on 3'X3' pad.

F_{w_i}
lb/ft
54
121
256
29
121
97
152
121
43
121

F_{p_i}
lb/ft
13
24
35
6
24
16
32
32
10
10

$P_w := F_{w_8}$ $P_w = 121 \cdot \text{lb/ft}$ For simplicity and added conservatism use: $P_w := 130 \cdot \text{lb/ft}$

Seismic load respectively is:

$P_e := F_{p_8}$ $P_e = 32 \cdot \text{lb/ft}$ Use: $P_e := 40 \cdot \text{lb/ft}$

Check the foundation, 3'-0"x3'-0"x0'-8" deep pad: $a := 3 \cdot \text{ft}$ $b := 3 \cdot \text{ft}$ $d := 8 \cdot \text{in}$

Average density of reinforced concrete is: $\gamma := 150 \cdot \text{pcf}$

Dead Weight

$$W_e := W_3 + 2 \cdot 1.9 \cdot \frac{\text{lb/ft}}{\text{ft}} \cdot (2 \cdot 36 \cdot \text{in} + 32 \cdot \text{in}) + 2 \cdot 4.08 \cdot \text{lb/ft}$$

$$W_e = 61.09 \cdot \text{lb/ft}$$

$$W_c := a \cdot b \cdot d \cdot \gamma$$

$$W_c = 900 \cdot \text{lb/ft}$$

Bearing capacity of the soil from UBC Table 29B for sandy gravel is: $p_a := 2000 \cdot \text{psf}$

The load on the footing is:

$$W := W_e + W_c$$

$$W = 961 \cdot \text{lb/ft}$$

say

$$W := 1000 \cdot \text{lb/ft}$$

$$M := P_w \cdot 26 \cdot \text{in}$$

$$M = 282 \cdot \text{ft} \cdot \text{lb/ft}$$

Check for negative soil pressure:

$$p := \frac{W}{a \cdot b} - \frac{6 \cdot M}{a \cdot b^2}$$

$$p = 48.5 \cdot \text{psf}$$

uplift at the end of foundation will not occur.

Find maximum soil pressure:

$$p_{\max} := \frac{W}{a \cdot b} + \frac{6 \cdot M}{a \cdot b^2}$$

$$p_{\max} = 174 \cdot \text{psf}$$

$$p_{\max} \leq p_a$$

soil pressure is acceptable.

Check footing for overturning:

$$M_s := \frac{W \cdot b}{2}$$

$$FS_o := \frac{M_s}{M}$$

$$FS_o = 5.33$$

$$FS_o \geq 1.5$$

stability of the foundation is satisfactory.

Design of the members and connections based on the rack for 600A Safety Switch is still applicable.

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Date: 07/11/1996 By: M.B. Lasota
Checked: 7/12/96 By: R.W Davidson
Revised: By

Evaluation of the conduit support outside Process Building - see Figure 11 pages 41 & 42.

Seismic Load

The weight per foot of the conduit and maximum span for each individual conduit on the governing Support #1 is:

3S019 - 3"	$d_1 = 3.5\text{-in}$	$dw_1 = 14.5\text{-plf}$	$l_1 = 20\text{-ft}$	$W_1 = dw_1 \cdot l_1$	$W_1 = 290.0\text{-lbf}$	$h_1 = 107.75\text{-in}$
3S027 - 2 1/2"	$d_2 = 2.875\text{-in}$	$dw_2 = 10.2\text{-plf}$	$l_2 = 16\text{-ft}$	$W_2 = dw_2 \cdot l_2$	$W_2 = 163.2\text{-lbf}$	$h_2 = 107.44\text{-in}$
3S021 - 1 1/2"	$d_3 = 1.9\text{-in}$	$dw_3 = 4.5\text{-plf}$	$l_3 = 14\text{-ft}$	$W_3 = dw_3 \cdot l_3$	$W_3 = 63.0\text{-lbf}$	$h_3 = 106.95\text{-in}$
3P240 - 1"	$d_4 = 1.315\text{-in}$	$dw_4 = 2.3\text{-plf}$	$l_4 = 12\text{-ft}$	$W_4 = dw_4 \cdot l_4$	$W_4 = 27.6\text{-lbf}$	$h_4 = 106.66\text{-in}$
3P002 - 1"	$d_5 = 1.315\text{-in}$	$dw_5 = 2.3\text{-plf}$	$l_5 = 12\text{-ft}$	$W_5 = dw_5 \cdot l_5$	$W_5 = 27.6\text{-lbf}$	$h_5 = 106.66\text{-in}$
3P006 - 1"	$d_6 = 1.315\text{-in}$	$dw_6 = 2.3\text{-plf}$	$l_6 = 12\text{-ft}$	$W_6 = dw_6 \cdot l_6$	$W_6 = 27.6\text{-lbf}$	$h_6 = 106.66\text{-in}$
3C014 - 1"	$d_7 = 1.315\text{-in}$	$dw_7 = 2.3\text{-plf}$	$l_7 = 12\text{-ft}$	$W_7 = dw_7 \cdot l_7$	$W_7 = 27.6\text{-lbf}$	$h_7 = 106.66\text{-in}$
3S022 - 3/4"	$d_8 = 1.05\text{-in}$	$dw_8 = 1.4\text{-plf}$	$l_8 = 10\text{-ft}$	$W_8 = dw_8 \cdot l_8$	$W_8 = 14.0\text{-lbf}$	$h_8 = 106.53\text{-in}$
3C012 - 3/4"	$d_9 = 1.05\text{-in}$	$dw_9 = 1.4\text{-plf}$	$l_9 = 10\text{-ft}$	$W_9 = dw_9 \cdot l_9$	$W_9 = 14.0\text{-lbf}$	$h_9 = 106.53\text{-in}$
3P007 - 3/4"	$d_{10} = 1.05\text{-in}$	$dw_{10} = 1.4\text{-plf}$	$l_{10} = 10\text{-ft}$	$W_{10} = dw_{10} \cdot l_{10}$	$W_{10} = 14.0\text{-lbf}$	$h_{10} = 106.53\text{-in}$
3S024 - 3/4"	$d_{11} = 1.05\text{-in}$	$dw_{11} = 1.4\text{-plf}$	$l_{11} = 10\text{-ft}$	$W_{11} = dw_{11} \cdot l_{11}$	$W_{11} = 14.0\text{-lbf}$	$h_{11} = 106.53\text{-in}$

Center of gravity of the conduit with respect to grade level is as follows:

$$i = 1, 2 \dots 11 \quad M_x = \sum_i W_i \cdot h_i \quad M_x = 6105.0\text{-ft}\cdot\text{lbf}$$

$$W = \sum_i W_i \quad W = 682.6\text{-lbf}$$

$$y_o = \frac{M_x}{W} \quad y_o = 107.33\text{-in}$$

Assuming, support will develop negligible frame action the overturning moment will be distributed evenly between posts.

$$W_{adj} = W + 2.9\text{-ft} \cdot 6.87 \frac{\text{lbf}}{\text{ft}} + 2.3 \cdot 3.5\text{-ft} \cdot 1.9 \frac{\text{lbf}}{\text{ft}} \quad W_{adj} = 819.56\text{-lbf}$$

$$M_{xadj} = M_x + 2.9\text{-ft} \cdot 6.87 \frac{\text{lbf}}{\text{ft}} \cdot 4.5\text{-ft} + 3.5\text{-ft} \cdot 1.9 \frac{\text{lbf}}{\text{ft}} \cdot 9.4\text{-ft} + 3.5\text{-ft} \cdot 1.9 \frac{\text{lbf}}{\text{ft}} \cdot 106\text{-in} \quad M_{xadj} = 6772.35\text{-ft}\cdot\text{lbf}$$

$$y_{oadj} = \frac{M_{xadj}}{W_{adj}} \quad y_{oadj} = 99.16\text{-in} \quad F = a_h \cdot W_{adj} \quad F = 260.62\text{-lbf} \quad M_c = M_{xadj} \cdot a_h \quad M_c = 25.84\text{-in}\cdot\text{kips}$$

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WO/Job No.: ER 4319/W 320
Date: 07/11/1996 By: M.B. Lasota
Checked: [Signature] By: R.W. Davidson
Revised By

Location: 241-C-106 & 241-AY-102

Wind Load

Wind pressure ref. ASCE 7-93 "Minimum Design Loads for Buildings and Other Structures".

From SDC 4.2 rev. 11 the following are parameters used in wind pressure calculation for Safety Class 3 structures:

Basic wind speed: $V := 70\text{-mph}$

Importance Factor: $I := 1.07$

Exposure Category: C

$K_z := 0.8$

$$q_z := 0.00256 \cdot K_z \left(\frac{I \cdot V}{\text{mph}} \right)^2 \cdot \text{psf} \quad q_z = 11.49 \cdot \text{psf}$$

use $q_z := 11.5 \cdot \text{psf}$

From Table 4. for other structures: $G_h := 1.32$

Calculate wind exposure area.

Length of the conduit exposed to wind is:

$$L_1 := 94\text{-in} + d_2 \quad L_1 = 96.88\text{-in}$$

$$L_2 := 106\text{-in} + d_1 \quad L_2 = 109.50\text{-in}$$

Area of the conduit is: $m := 3, 4, 7$

$$A_c := \left(d_1 + \sum_m d_m + d_{10} \right) \cdot (L_2 - 2\text{-in}) + (d_2 + d_8 + d_9 + d_{11}) \cdot (L_1 - 2\text{-in}) \quad A_c = 12.71 \cdot \text{ft}^2$$

Area of the frame is:

$$A_f := 2 \cdot (L_2 - 2\text{-in} + 3\text{-in}) \cdot 3\text{-in} + 4 \cdot 1.625\text{-in} \cdot 45\text{-in} \quad A_f = 6.64 \cdot \text{ft}^2$$

Solid area is:

$$A_s := A_c + A_f \quad A_s = 19.35 \cdot \text{ft}^2$$

Gross area is:

$$A_g := 45\text{-in} \cdot (L_2 - 2\text{-in}) \quad A_g = 33.59 \cdot \text{ft}^2 \quad \epsilon := \frac{A_s}{A_g} \quad \epsilon = 0.58$$

Average diameter of the round lattice member is:

$$d_{av} := \frac{\sum_i d_i + 2 \cdot 3\text{-in} + 4 \cdot 1.625\text{-in}}{11 + 6} \quad d_{av} = 0.15\text{-ft}$$

$$\frac{d_{av}}{\text{ft}} \sqrt{\frac{q_z}{\text{psf}}} = 0.50 \quad \text{from table 14, } C_f \text{ is: } C_f := 1.5$$

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Checked: 7/12/96 By: R.W. Davidson
Revised By:

Location: 241-C-106 & 241-AY-102

$$A_f := A_g$$

Wind load therefore is:

$$F_w := q_z \cdot G_h \cdot C_f \cdot A_f \quad F_w = 764.93 \cdot \text{lbf}$$

$$M_w := F_w \cdot \frac{L_2 - 2 \cdot \text{in}}{2} \quad M_w = 41.11 \cdot \text{kips} \cdot \text{in} \quad M_w \geq M_c \quad \text{Wind case governs.}$$

Check the foundation, 5'-6"x4'-0"x0'-8" deep pad: $a := 5.5 \cdot \text{ft} \quad b := 4 \cdot \text{ft} \quad d := 8 \cdot \text{in} \quad w_f := 9.42 \cdot \frac{\text{lbf}}{\text{ft}}$

Assume density of concrete to be: $\gamma := 150 \cdot \text{pcf}$

Dead Weight: $W_{adj} = 819.56 \cdot \text{lbf}$

$$W_c := a \cdot b \cdot d \cdot \gamma \quad W_c = 2200.00 \cdot \text{lbf}$$

Bearing capacity of the soil from UBC Table 29B for sandy gravel is:

$$p_a := 2000 \cdot \text{psf}$$

The load on the footing is:

$$W := W_{adj} + W_c \quad W = 3019.56 \cdot \text{lbf}$$

$$M := F_w \cdot \frac{L_2 - 2 \cdot \text{in}}{2} \quad M = 3426.25 \cdot \text{ft} \cdot \text{lbf}$$

Check for negative soil pressure:

$$p := \frac{W}{a \cdot b} - \frac{6 \cdot M}{a \cdot b^2} \quad p = -96.4 \cdot \text{psf} \quad \text{uplift at the end of foundation will occur.}$$

Find actual pressure distribution due to eccentricity:

$$\Sigma M = 0$$

$$b_1 := \left(\frac{b}{2} - \frac{M}{W} \right) \cdot 3 \quad b_1 = 2.60 \cdot \text{ft}$$

$$\Sigma Y = 0$$

$$p_{max} := \frac{2 \cdot W}{a \cdot b_1} \quad p_{max} = 423 \cdot \text{psf} \quad p_{max} \leq p_a \quad \text{soil pressure is acceptable.}$$

Check footing for overturning:

$$M_s := \frac{W \cdot b}{2} \quad FS_o := \frac{M_s}{M}$$

$$FS_o = 1.76 \quad FS_o \geq 1.5 \quad \text{stability of the foundation is satisfactory.}$$

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Date: 07/11/1996 By: M.B. Lasota
Checked: *7/12/96* By: R.W Davidson
Revised By: *MB*

Check TS3X3X0.1875 column for stress.

$$C := \frac{W_{adj}}{2} \quad C = 409.78 \cdot \text{lb} \cdot \text{f}$$

$$M_c := \frac{M}{2} \quad M_c = 20557.49 \cdot \text{lb} \cdot \text{f} \cdot \text{in}$$

Properties of the column are (ASTM A500 Gr B): $F_y := 42 \cdot \text{ksi}$

$$A := 2.02 \cdot \text{in}^2 \quad S := 1.73 \cdot \text{in}^3 \quad r := 1.13 \cdot \text{in} \quad l := 9 \cdot \text{ft} \quad K := 2.0$$

$$S_r := \frac{K \cdot l}{r} \quad S_r = 191.15 \quad C_c = 126.10 \quad S_r \geq C_c$$

$$F_a := \frac{12 \cdot \pi^2 \cdot E}{23 \cdot S_r^2} \quad F_a = 4086.96 \cdot \text{psi} \quad f_a := \frac{C}{A} \quad f_a = 202.86 \cdot \text{psi} \quad \frac{f_a}{F_a} = 0.05$$

$$F_{bx} := 0.6 \cdot F_y \quad F_{bx} = 25.20 \cdot \text{ksi} \quad f_{bx} := \frac{M_c}{S} \quad f_{bx} = 11.88 \cdot \text{ksi}$$

$$IF := \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} \quad IF = 0.52 \quad IF \leq 1.00 \quad O.K.$$

Check Base Plate for capacity. $t_{pl} := 0.625 \cdot \text{in} \quad a_{pl} := 10 \cdot \text{in} \quad b_{pl} := a_{pl}$

Check expansion anchors:

$$l_1 := 2.125 \cdot \text{in} \quad l_{01} := 1.5 \cdot \text{in} + 2 \cdot t_{pl} + 0.5 \cdot a_{pl} - 1.25 \cdot \text{in} \quad l_{01} = 6.50 \cdot \text{in}$$

$$l_2 := 2.125 \cdot \text{in} \quad l_{02} := 1.5 \cdot \text{in} + 2 \cdot t_{pl} + 0.5 \cdot a_{pl} - 1.25 \cdot \text{in} \quad l_{02} = 6.50 \cdot \text{in}$$

$$M := M_c \quad M = 20.56 \cdot \text{kips} \cdot \text{in}$$

$$M = T_{max} \left[l_{01} + \left(\frac{l_1}{l_2} \right)^2 \cdot l_{02} \right]$$

$$T_{max} := \frac{M}{l_{01} + \left(\frac{l_1}{l_2} \right)^2 \cdot l_{02}} \quad T_{max} = 1581.35 \cdot \text{lb} \cdot \text{f} \quad T_{min} := T_{max} \left(\frac{l_1}{l_2} \right)^2 \quad T_{min} = 1581.35 \cdot \text{lb} \cdot \text{f}$$

$$T_b := T_{max} \quad T_b = 1581.35 \cdot \text{lb} \cdot \text{f} \quad V_b := \frac{F_w}{8} \quad V_b = 95.62 \cdot \text{lb} \cdot \text{f}$$

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By: M.B. Lasota

By: R.W Davidson

By

For 5/8" dia HKB II with 2 3/4" min. embedment, installed in $f'_c=4000$ psi concrete, allowable loads are:

$T_a := 2000\text{-lbf}$ $V_a := 2530\text{-lbf}$ Ref. A/E Standard GC-ANCR-01 rev 0 table 21.

Interaction factor is:

$$IF := \frac{T_b}{T_a} + \frac{V_b}{V_a} \quad IF = 0.83 \quad IF \leq 1.0 \quad O.K..$$

Check stress in the base plate (ASTM A36): $F_y := 36\text{-ksi}$

$t_{pl} := 0.625\text{-in}$

$$f_b := \frac{6 \cdot T_b}{t_{pl}^2} \quad f_b = 24.29\text{-ksi} \quad F_b := 0.75 \cdot F_y \quad F_b = 27.00\text{-ksi} \quad f_b \leq F_b \quad O.K..$$

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WO/Job No.: ER 4319W 320

Date: 04/18/1996 By: M.B. Lasota

Checked: H22/96 By: R.W. Davidson

Revised By:

Evaluation of the ECN W-320-251.

LIFTING EYE DETAIL

Determine weld capacity, electrode used E70XX:

$$l_w := 3 \cdot \text{in} - 2 \cdot \frac{3}{16} \cdot \text{in} \quad l_w = 2.62 \cdot \text{in} \quad \text{length of the weld.}$$

$$t_w := \frac{3}{16} \cdot \text{in} \quad \text{effective throat of the weld.}$$

$$SF := 5 \quad F_{uw} := 70 \cdot \text{ksi}$$

$$P_{all} := \frac{2 \cdot l_w \cdot t_w \cdot F_{uw}}{SF} \quad P_{all} = 13.78 \cdot \text{kips}$$

Determine capacity of the plate, material - ASTM A36:

Bearing, ref.: "Steel Structures, Design and Behavior" second edition C. G. Salmon, J. E. Johnson.

$$t_{pl} := 0.5 \cdot \text{in} \quad F_{us} := 58 \cdot \text{ksi} \quad L_e := 2 \cdot \text{in} \quad SF = 5.00$$

$$P_{all} := \frac{F_{us} \cdot t_{pl} \cdot L_e}{SF} \quad P_{all} = 11.60 \cdot \text{kips}$$

Plate tension:

$$F_{ys} := 36 \cdot \text{ksi} \quad SF := 3 \quad b := 4 \cdot \text{in} \quad D := 1 \cdot \text{in}$$

$$P_{all} := \frac{(b - D) \cdot t_{pl} \cdot F_{ys}}{SF} \quad P_{all} = 18.00 \cdot \text{kips}$$

Conclusion: Bearing strength governs thus the capacity of the Lifting Eye is: 11600 lbf.

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Revised By

Foundation for Nitrogen Purge Bottle Station

Determine the Wind Load:

Exposure area:

$$w := 5.25\text{-ft} \quad h := 6.25\text{-ft} \quad A := w \cdot h \quad A = 32.81 \cdot \text{ft}^2$$

From SDC 4.1 rev. 11 the following are parameters used in wind pressure calculation for Safety Class 3 structures:

Basic wind speed: $V := 70\text{-mph}$
Importance Factor: $I := 1.07$
Exposure Category: C

$$K_z := 0.8$$

$$q_z := 0.00256 \cdot K_z \cdot \left(\frac{I \cdot V}{\text{mph}} \right)^2 \cdot \text{psf} \quad q_z = 11.49 \cdot \text{psf}$$

$$\text{use } q_z := 11.5 \cdot \text{psf}$$

From Table 4, for other structures: $G_h := 1.32$

$$\text{and from table 13.} \quad C_f := 1.6 \quad p_w := q_z \cdot G_h \cdot C_f \quad p_w = 24.29 \cdot \text{psf}$$

$$F_w := p_w \cdot A \quad F_w = 0.80 \cdot \text{kips}$$

Dead weight of rack and the equipment is:

$$w_{1x1} := 0.8 \cdot \text{plf} \quad l_{1x1} := 6 \cdot 5.25\text{-ft} \quad l_{1x1} = 31.50\text{-ft} \quad W_{1x1} := w_{1x1} \cdot l_{1x1} \quad W_{1x1} = 25.20 \cdot \text{lbft}$$

$$w_{2x2} := 3.19 \cdot \text{plf} \quad l_{2x2} := (3 \cdot (6.25 + 3 + 1.17) + 5.25 + 3.5) \cdot \text{ft} \quad l_{2x2} = 40.01\text{-ft}$$

$$W_{2x2} := w_{2x2} \cdot l_{2x2} \quad W_{2x2} = 127.63 \cdot \text{lbft}$$

$$W_{n2} := 3 \cdot 150 \cdot \text{lbft} \quad W_{n2} = 450.00 \cdot \text{lbft}$$

Estimating miscellaneous equipment and hardware as 10% of the total weight, the design max weight is:

$$W_{\max} := (W_{1x1} + W_{2x2} + W_{n2}) \cdot 1.1 \quad W_{\max} = 663.12 \cdot \text{lbft}$$

Assume the minimum weight to be 80% of the maximum weight.

In order to satisfy $FS=1.5$ against overturn the foundation width should be no less than the following:

$$d_f := 8\text{-in} \quad \gamma_{rc} := 150 \cdot \text{pcf} \quad l_f := 6.5\text{-ft} \quad b_{fx} \quad FS := 1.5$$

$$\text{let } x := 12\text{-in}, 15\text{-in}..60\text{-in}$$

$$\text{Stabilizing weight is - } W_s(x) := 0.8 \cdot W_{\max} + d_f \gamma_{rc} \cdot l_{fx}$$

$$\text{Stabilizing moment is - } M_s(x) := W_s(x) \cdot \frac{x}{2}$$

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Revised

By: M.B. Lasota

By: R.W. Davidson

By

Overturing moment is:

$$M_o := F_w \frac{h}{2} \quad M_o = 2490.47 \cdot \text{ft} \cdot \text{lbf}$$

$$M(x) := M_s(x) - FS \cdot M_o \quad x := 3.006749 \cdot \text{ft} \quad \text{root}(M(x), x) = 3.006749 \cdot \text{ft} \quad M(x) = 0.00 \cdot \text{lbf} \cdot \text{ft}$$

$$x := 4.0 \cdot \text{ft}$$

$$W_s(x) := 0.8 \cdot W_{\text{max}} + d_f \gamma_{rc} \cdot l_f x \quad W_s(x) = 3130.49 \cdot \text{lbf} \quad M_s(x) = 6260.98 \cdot \text{lbf} \cdot \text{ft}$$

$$b_f := x \quad W_s := W_s(b_f) \quad W_s = 3130.49 \cdot \text{lbf}$$

$$p := \frac{W_s}{l_f b_f} - \frac{6 \cdot M_o}{l_f b_f^2} \quad p = -23.3 \cdot \text{psf} \quad \text{uplift at the end of foundation will occur.}$$

Find actual pressure distribution due to moment:

$$\Sigma M = 0$$

$$b_1 := \left(\frac{b_f}{2} - \frac{M_o}{W_s} \right) \cdot 3 \quad b_1 = 3.61 \cdot \text{ft}$$

$$\Sigma Y = 0$$

$$W_s(x) := W_{\text{max}} + d_f \gamma_{rc} \cdot l_f x \quad W_s(x) = 3263.12 \cdot \text{lbf}$$

$$b_f := x \quad W_s := W_s(b_f) \quad W_s = 3263.12 \cdot \text{lbf}$$

$$p_{\text{max}} := \frac{2 \cdot W_s}{l_f b_1} \quad p_{\text{max}} = 277.9 \cdot \text{psf} \quad p_{\text{max}} \leq p_a \quad \text{soil pressure is acceptable.}$$

Check footing for overturning:

$$FS := \frac{M_s(b_f)}{M_o}$$

$$FS = 2.51 \quad FS \geq 1.5 \quad \text{stability of the foundation is satisfactory.}$$

Provide foundation pad 4'-0" x 6'-6" x 0'-8".

Client: WHC
Subject: Structural analysis of Safety Related
Electrical Equipment Supports.

WO/Job No.: ER 4319/W 320

Date: 05/03/1996

By: M.B. Lasota

Checked: *sp/au*

By: R.W. Davidson

Revised

By

Location: 241-C-106 & 241-AY-102

Evaluation of the anchorage.

Overturning moment caused by wind must be resisted by the anchorage system. In further analysis weight of rack will be neglected for simplicity and added conservatism.

$$M_o = 29.89 \cdot \text{kips} \cdot \text{in}$$

The overturning moment will be resisted by three identical frames. Each frame will resist a portion of the moment proportional to a distance between frames. Middle frame will carry the highest moment that amounts to:

$$l_1 := 19.75 \cdot \text{in} \quad l_2 := 51 \cdot \text{in} - l_1$$

$$M_{om} := M_o \cdot \frac{l_1 + l_2}{2 \cdot (l_1 + l_2)} \quad M_{om} = 14.94 \cdot \text{kips} \cdot \text{in}$$

Tension in expansion anchor is:

$$l_{base} := 17 \cdot \text{in} \quad ed_{bolt} := 1.75 \cdot \text{in} \quad a := 3 \cdot \text{in} \quad - \text{distance to the compression member (base inflection point)}$$

$$T_b := \frac{M_{om}}{l_{base} - a - ed_{bolt}} \quad T_b = 1.22 \cdot \text{kips}$$

$$V_b := \frac{F_w}{2 \cdot 2} \quad V_b = 0.20 \cdot \text{kips}$$

Using HKBII 1/2 ϕ with 3.5 min embedment installed in $f'_c = 4000 \text{ psi}$ concrete, ref GC-ANCR-01 rev. 1 table 21, interaction equation is:

$$T_{all} := 10.0 \cdot \text{kN} \quad T_{all} = 2.25 \cdot \text{kips} \quad V_{all} := 6.53 \cdot \text{kN} \quad V_{all} = 1.47 \cdot \text{kips}$$

$$IF := \frac{T_b}{T_{all}} + \frac{V_b}{V_{all}} \quad IF = 0.68$$

Anchorage is adequate.

Client: WHC
Subject: Structural analysis of Safety Related
Electrical Equipment Supports.
Location: 241-C-106 & 241-AY-102

WO/Job No.: ER 4319/W 320
Date: 7/26/96 By: M.B. Lasota
Checked: 7/29/96 By: R.W. Davidson
Revised By:

Meter and Socket Mounting Rack - Figure 14, page 45, (ref dwg H-2-820752 sheet 3).

The analysis of the rack and its foundation will include an influence of the environmental loads i.e., Wind and Seismic.
The loads are calculated on the basis outlined in UBC.

Wind Load, $i = 1..8$

From observation, the critical wind exposure appears on the frontal area of the rack (maximum Area of exposure).
The following is the calculation of the exposure area and its center of gravity.

Box :	$h_1 := 52\text{-in}$	$w_1 := 36\text{-in}$	$A_1 := h_1 \cdot w_1$	$y_1 := 42\text{-in}$	$A_1 = 13.00 \cdot \text{ft}^2$	$Q_1 := A_1 \cdot y_1$	$W_1 := 200\text{-lbf}$
Uni 1:	$h_2 := 16\text{-in}$	$w_2 := 1.625\text{-in}$	$A_2 := h_2 \cdot w_2$	$y_2 := 8\text{-in}$	$A_2 = 0.18 \cdot \text{ft}^2$	$Q_2 := A_2 \cdot y_2$	$W_2 := 22.2\text{-lbf}$
Uni 2:	$h_3 := 16\text{-in}$	$w_3 := 1.625\text{-in}$	$A_3 := h_3 \cdot w_3$	$y_3 := 8\text{-in}$	$A_3 = 0.18 \cdot \text{ft}^2$	$Q_3 := A_3 \cdot y_3$	$W_3 := 22.2\text{-lbf}$
Uni 3:	$h_4 := 1.625\text{-in}$	$w_4 := 36\text{-in}$	$A_4 := h_4 \cdot w_4$	$y_4 := 5.8125\text{-in}$	$A_4 = 0.41 \cdot \text{ft}^2$	$Q_4 := A_4 \cdot y_4$	$W_4 := 6\text{-lbf}$
Cond1:	$h_5 := 18\text{-in}$	$w_5 := 4.5\text{-in}$	$A_5 := h_5 \cdot w_5$	$y_5 := 9\text{-in}$	$A_5 = 0.56 \cdot \text{ft}^2$	$Q_5 := A_5 \cdot y_5$	$W_5 := 0\text{-lbf}$
Cond2:	$h_6 := 18\text{-in}$	$w_6 := 4.5\text{-in}$	$A_6 := h_6 \cdot w_6$	$y_6 := 9\text{-in}$	$A_6 = 0.56 \cdot \text{ft}^2$	$Q_6 := A_6 \cdot y_6$	$W_6 := 0\text{-lbf}$
Cond3:	$h_7 := 18\text{-in}$	$w_7 := 4.5\text{-in}$	$A_7 := h_7 \cdot w_7$	$y_7 := 9\text{-in}$	$A_7 = 0.56 \cdot \text{ft}^2$	$Q_7 := A_7 \cdot y_7$	$W_7 := 0\text{-lbf}$
Cond4:	$h_8 := 18\text{-in}$	$w_8 := 4.5\text{-in}$	$A_8 := h_8 \cdot w_8$	$y_8 := 9\text{-in}$	$A_8 = 0.56 \cdot \text{ft}^2$	$Q_8 := A_8 \cdot y_8$	$W_8 := 0\text{-lbf}$

$i = 1..8$

$$A = \sum_i A_i \quad A = 16.02 \cdot \text{ft}^2$$

$$Q = \sum_i Q_i \quad Q = 47.63 \cdot \text{ft}^3$$

$$y_o = \frac{Q}{A} \quad y_o = 35.68\text{-in} \quad \text{for wind load computation increase area by 25\%} \quad A_f := 1.25 \cdot A \quad A_f = 20.02 \cdot \text{ft}^2$$

Wind pressure ref. ASCE 7-93 "Minimum Design Loads for Buildings and Other Structures".

From SDC 4.1 rev. 11 the following are parameters used in wind pressure calculation for Safety Class 3 structures:

Basic wind speed: $V := 70\text{-mph}$
Importance Factor: $I := 1.07$
Exposure Category: C

$$K_z := 0.8$$

$$q_z = 0.00256 \cdot K_z \cdot \left(\frac{1 \cdot V}{\text{mph}} \right)^2 \cdot \text{psf} \quad q_z = 11.49 \cdot \text{psf}$$

$$\text{use } q_z = 11.5 \cdot \text{psf}$$

Client: WHC

Subject: Structural analysis of Safety Related
Electrical Equipment Supports.

Location: 241-C-106 & 241-AY-102

WO/Job No.: ER 4319/W 320

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By: M.B. Lasota

By: R.W. Davidson

By

From Table 4, for other structures: $G_h = 1.32$

and from table 13: $C_f = 1.6$

Wind load therefore is:

$$F_w = q_z \cdot G_h \cdot C_f A_f \quad F_w = 486.3 \cdot \text{lb} \cdot \text{ft} \quad M_w = F_w y_o \quad M_w = 17351 \cdot \text{lb} \cdot \text{ft} \cdot \text{in}$$

Seismic load per UBC (36-1)

$$F_p = Z \cdot I \cdot C_p \cdot W_p$$

$$\text{From SDC 4.1 rev 11: } Z = 0.12 \quad I = 1.25 \quad C_p = 2.12 \quad a_h = Z \cdot I \cdot C_p \quad a_h = 0.32$$

Adjusting the location of the center of gravity of the vertical members to reflect the actual mass center of gravity rather than area center of gravity as formerly calculated, and adding the contribution of two horizontal P1001 channels, the mass center of gravity is as follows:

$$y_2 = 35 \cdot \text{in} \quad y_3 = 35 \cdot \text{in}$$

$$\text{Uni 4: } h_9 = 1.625 \cdot \text{in} \quad w_9 = 36 \cdot \text{in} \quad y_9 = 17.8125 \cdot \text{in} \quad W_9 = 11.4 \cdot \text{lb} \cdot \text{ft}$$

$$\text{Uni 5: } h_{10} = 1.625 \cdot \text{in} \quad w_{10} = 36 \cdot \text{in} \quad y_{10} = 66.1875 \cdot \text{in} \quad W_{10} = 11.4 \cdot \text{lb} \cdot \text{ft}$$

The weight of four stubbed-out 4" conduits is indicated as 0, since they are relatively stiff and self-supported.

$j = 1 \dots 10$

$$M_x = \sum_j W_j y_j \quad M_x = 10946.48 \cdot \text{lb} \cdot \text{ft} \cdot \text{in}$$

$$W_x = \sum_j W_j \quad W_x = 273.20 \cdot \text{lb} \cdot \text{ft}$$

$$F_s = a_h \cdot W_x \quad F_s = 86.9 \cdot \text{lb} \cdot \text{ft}$$

$$M_s = a_h \cdot M_x \quad M_s = 3481 \cdot \text{lb} \cdot \text{ft} \cdot \text{in}$$

Compare wind and seismic loads.

$$F_s = 86.9 \cdot \text{lb} \cdot \text{ft} \quad M_s = 3481 \cdot \text{lb} \cdot \text{ft} \cdot \text{in}$$

$$F_w = 486.3 \cdot \text{lb} \cdot \text{ft} \quad M_w = 17351 \cdot \text{lb} \cdot \text{ft} \cdot \text{in}$$

It is evident that the wind load governs.

ICF KAISER HANFORD COMPANY

DESIGN ANALYSIS

HNF-2468, Rev. 0
Calc. No. W320-24-029
Revision 1
Page No. 17g of

Client: WHC
Subject: Structural analysis of Safety Related
Electrical Equipment Supports.
Location: 241-C-106 & 241-AY-102

WO/Job No.: ER 4319/W 320
Date: 7/26/96 By: M.B. Lasota
Checked: 7/29/96 By: R.W. Davidson
Revised By:

From Shannon & Wilson inc. "Final Report of Geotechnical Studies W 320 Waste Retrieval and Sluicing System
200 East Area, Hanford Site", April 1994, soil properties are:

$$\gamma := 115 \cdot \text{pcf} \quad \phi := 37 \cdot \text{deg}$$

Rankine passive lateral earth pressure is:

$$p := \frac{1 + \sin(\phi)}{1 - \sin(\phi)} \cdot \gamma \quad p = 462.62 \cdot \frac{\text{psf}}{\text{ft}}$$

From UBC 2907 6 (g) 2.

$$S_1 := p$$

$$b := 12 \cdot \text{in} \quad h := y_o \quad h = 35.68 \cdot \text{in}$$

$$P := \frac{F_w}{2} \quad P = 0.24 \cdot \text{kips}$$

$$\alpha := 3 \cdot 2.34 \cdot \frac{P}{b \cdot S_1} \quad \alpha = 3.69 \cdot \text{ft}^2$$

Assume: $d := 33.80212 \cdot \text{in}$

$$A_o := \frac{\alpha}{d} \quad A_o = 1.31 \cdot \text{ft}$$

$$u := \frac{A_o}{2} \left(1 + \sqrt{1 + \frac{4.36 \cdot h}{A_o}} \right) \quad u = 33.80212 \cdot \text{in}$$

Existing embedment depth of 5'-0" is adequate

Check P1001 column for stress.

$$C := \frac{W_x}{2} \quad C = 136.60 \cdot \text{lb/ft} \quad M := \frac{M_w}{2} \quad M = 8675.37 \cdot \text{lb/ft-in}$$

Properties of the column are:

$$A := 1.11 \cdot \text{in}^2 \quad S := .572 \cdot \text{in}^3 \quad r := .915 \cdot \text{in} \quad I := 5.8333 \cdot \text{ft} \quad K := 2.0$$

$$E := 29000 \cdot \text{ksi} \quad F_y := 41.67 \cdot \text{ksi} \quad C_c := \sqrt{\frac{2 \cdot \pi^2 \cdot E}{F_y}} \quad C_c = 117.21$$

$$S_r := \frac{K \cdot I}{r} \quad S_r = 153.00 \quad C_c = 117.21 \quad S_r \geq C_c$$

$$F_a := \frac{12 \cdot \pi^2 \cdot E}{23 \cdot S_r^2} \quad F_a = 6378.85 \cdot \text{psi} \quad f_a := \frac{C}{A} \quad f_a = 123.06 \cdot \text{psi} \quad \frac{f_a}{F_a} = 0.02$$

$$F_{bx} := 0.6 \cdot F_y \quad F_{bx} = 25.00 \cdot \text{ksi} \quad f_{bx} := \frac{M}{S} \quad f_{bx} = 15.17 \cdot \text{ksi}$$

$$IF := \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} \quad IF = 0.63 \quad IF \leq 1.00 \quad \text{O.K.}$$

Buckling in the other direction is not critical due to a membrane action of the Box. All other members are adequate.

Client: NHC

Subject: Structural analysis of Safety Related
Electrical Equipment Supports.

Location: 241-C-106 & 241-AY-102

WO/Job No.: ER 4319/W 320

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Revised

By: M.B. Lasota

By: R.W. Davidson

By

Design and analysis of the restraint for the Mass Flow Meter.

**ELEVATION LOOKING SOUTH AT THE EQUIPMENT
ATTACHED TO THE CONCRETE COLLAR**

Client: NHC
Subject: Structural analysis of Safety Related
Electrical Equipment Supports.

Location: 241-C-106 & 241-AY-102

WO/Job No.: ER 4319/W 320

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By: R.W. Davidson

By

SECTION LOOKING EAST AT THE EQUIPMENT ATTACHED TO THE CONCRETE COLLAR

The analysis of the frame and its anchorage will include an influence of the environmental loads i.e., Wind and Seismic. The loads are calculated on the basis outlined in UBC.

Wind Load. $i = 1..2$

From observation, the critical wind exposure appears on the frontal area of the frame (maximum Area of exposure). The following is the calculation of the exposure area and its center of gravity. $Y=0$ is at the elevation of top anchors.

Box :	$h_1 := 12\text{-in}$	$w_1 := 9\text{-in}$	$A_1 := h_1 \cdot w_1$	$y_1 := 47.5\text{-in}$	$A_1 = 0.75 \cdot \text{ft}^2$	$Q_1 := A_1 \cdot y_1$	$W_1 := 20\text{-lbf}$
TS3X3:	$h_2 := 49\text{-in}$	$w_2 := 3\text{-in}$	$A_2 := h_2 \cdot w_2$	$y_2 := 30\text{-in}$	$A_2 = 1.02 \cdot \text{ft}^2$	$Q_2 := A_2 \cdot y_2$	$W_2 := 26.3\text{-lbf}$

Client: NHC

Subject: Structural analysis of Safety Related
Electrical Equipment Supports.

Location: 241-C-106 & 241-AY-102

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By: M.B. Lasota

By: R.W. Davidson

By

$$i := 1..2$$

$$A := \sum_i A_i \quad A = 1.77 \cdot \text{ft}^2 \quad Q := \sum_i Q_i \quad Q = 5.52 \cdot \text{ft}^3$$

$$y_o := \frac{Q}{A} \quad y_o = 37.41 \cdot \text{in} \quad \text{for wind load computation, increase area by 25\%} \quad A_f := 1.25 \cdot A \quad A_f = 2.21 \cdot \text{ft}^2$$

Wind pressure ref. ASCE 7-93 "Minimum Design Loads for Buildings and Other Structures".

From SDC 4.1 rev. 11 the following are parameters used in wind pressure calculation for Safety Class 3 structures:

Basic wind speed: $V := 70 \cdot \text{mph}$

Importance Factor: $I := 1.07$

Exposure Category: C

$$K_z := 0.8$$

$$q_z := 0.00256 \cdot K_z \left(\frac{I \cdot V}{\text{mph}} \right)^2 \cdot \text{psf} \quad q_z = 11.49 \cdot \text{psf}$$

$$\text{use } q_z := 11.5 \cdot \text{psf}$$

From Table 4. for other structures: $G_h := 1.32$

and from table 13: $C_f := 1.6$

Wind load therefore is:

$$F_w := q_z \cdot G_h \cdot C_f \cdot A_f \quad F_w = 53.8 \cdot \text{lbf} \quad M_w := F_w \cdot y_o \quad M_w = 2011 \cdot \text{lbf} \cdot \text{in}$$

Seismic load per UBC (36-1)

$$F_p = Z \cdot I \cdot C_p \cdot W_p$$

From SDC 4.1 rev 11: $Z := 0.12 \quad I := 1.25 \quad C_p := 2.12 \quad a_h := Z \cdot I \cdot C_p \quad a_h = 0.32$

$$j := 1..2$$

$$M_x := \sum_j W_j \cdot y_j \quad M_x = 1739 \cdot \text{lbf} \cdot \text{in} \quad W_x := \sum_j W_j \quad W_x = 46.30 \cdot \text{lbf}$$

$$F_s := a_h \cdot W_x \quad M_s := a_h \cdot M_x$$

$$F_s = 14.7 \cdot \text{lbf} \quad M_s = 553 \cdot \text{lbf} \cdot \text{in}$$

Client: NHC
Subject: Structural analysis of Safety Related
Electrical Equipment Supports.
Location: 241-C-106 & 241-AY-102

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Checked: 5/8/97 By: R.W. Davidson
Revised By

The support frame for the Mass Flow Meter is located in the area where the maintenance work will take place. In order to prevent the frame from collapse when subject to a load resulting from the worker leaning on the frame, a standard horizontal load on the handrail will be applied. This load is equal to 200 lbf.

$$F_I := 200 \cdot \text{lbf} \quad M_I := F_I (y_1 + 7 \cdot \text{in}) \quad M_I = 10900 \cdot \text{lbf} \cdot \text{in}$$

Compare wind, seismic and live loads.

$$F_S = 15 \cdot \text{lbf} \quad M_S = 553 \cdot \text{lbf} \cdot \text{in}$$

$$F_W = 54 \cdot \text{lbf} \quad M_W = 2011 \cdot \text{lbf} \cdot \text{in}$$

$$F_I = 200 \cdot \text{lbf} \quad M_I = 10900 \cdot \text{lbf} \cdot \text{in}$$

It is evident that the live load governs.

Check the member (TS3X3X0.25) stress.

$$S_X := 2.10 \cdot \text{in}^3$$

$$f_b := \frac{M_I}{S_X} \quad f_b = 5.19 \cdot \text{ksi} \quad \text{stress in the TS3X3X0.25 is acceptable.}$$

Determine forces in the top anchors.

$$T_{at} := \frac{M_I + F_I 10.5 \cdot \text{in}}{2 \cdot 10.5 \cdot \text{in}} \quad T_{at} = 619 \cdot \text{lbf}$$

Determine forces in the bottom anchor.

$$T_{ab} := \frac{F_I (y_1 + 7 \cdot \text{in} - 5.5 \cdot \text{in})}{16 \cdot \text{in}} \quad T_{ab} = 613 \cdot \text{lbf}$$

CG of anchor group is:

$$y_o := \frac{A_b \cdot 10.5 \cdot \text{in}}{3 \cdot A_b} \quad y_o := \frac{10.5 \cdot \text{in}}{3} \quad y_o = 3.50 \cdot \text{in}$$

$$\text{Tor} := M_I + F_I y_o \quad \text{Tor} = 11.60 \cdot \text{in} \cdot \text{kips}$$

Shear in the bolt is: $i := 1 \dots 3$

$$J_b = \sum_{i=1}^3 A_b [(x_i)^2 + (y_i)^2] \quad \text{or} \quad J_b = A_b \left[\sum_{i=1}^3 [(x_i)^2 + (y_i)^2] \right]$$

$$x_1 := -4.25 \cdot \text{in} \quad y_1 := y_o$$

$$x_2 := 4.25 \cdot \text{in} \quad y_2 := y_o$$

$$x_3 := 0 \cdot \text{in} \quad y_3 := 7.0 \cdot \text{in}$$

Client: NHC

Subject: Structural analysis of Safety Related
Electrical Equipment Supports.

Location: 241-C-106 & 241-AY-102

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By: M.B. Lasota

By: R.W. Davidson

By

$$V_{b_i} = \sqrt{\left(\frac{F_i}{3} + \frac{\text{Tor} \cdot y_i}{J_b} \cdot A_b\right)^2 + \left(\frac{\text{Tor} \cdot x_i}{J_b} \cdot A_b\right)^2}$$

$$V_{b_i} = \sqrt{\left[\frac{F_i}{3} + \frac{\text{Tor} \cdot y_i}{\sum_{i=1}^3 [(x_i)^2 + (y_i)^2]} \right]^2 + \left[\frac{\text{Tor} \cdot x_i}{\sum_{i=1}^3 [(x_i)^2 + (y_i)^2]} \right]^2}$$

$$V_{b_1} = 627 \cdot \text{lbf}$$

$$V_{b_2} = 627 \cdot \text{lbf}$$

$$V_{b_3} = 807 \cdot \text{lbf}$$

$$T_{b_1} = T_{at}$$

$$T_{b_2} = T_{at}$$

$$T_{b_3} = T_{ab}$$

Try 1/2" Φ HKB II with 3 1/2" embedment. For 3000psi concrete capacities are:

$$T_a = 2.00 \cdot \text{kips}$$

$$V_a = 1.47 \cdot \text{kips}$$

Interaction factors for each bolt are:

$$IF_i = \frac{T_{b_i}}{T_a} + \frac{V_{b_i}}{V_a}$$

$$IF_1 = 0.74$$

$$IF_2 = 0.74$$

$$IF_3 = 0.86$$

OK.

Check stress in the horizontal L3X3X3/8 and the related weld.

DETAIL OF THE JOINT BETWEEN L3X3X3/8 AND TS3X3X0.25

Client: NHC
Subject: Structural analysis of Safety Related
Electrical Equipment Supports.
Location: 241-C-106 & 241-AY-102

WO/Job No.: ER 4319W 320
Date: 3/12/97 By: M.B. Lasota
Checked: 5/8/97 By: R.W. Davidson
Revised By: *MR*

$$V_{bx_i} = \frac{F_i}{3} + \frac{\text{Tor} \cdot y_i}{\sum_{i=1}^3 [(x_i)^2 + (y_i)^2]}$$

$$V_{by_i} = \frac{\text{Tor} \cdot x_i}{\sum_{i=1}^3 [(x_i)^2 + (y_i)^2]}$$

$$V_{bx_1} = 0.44 \cdot \text{kips}$$

$$V_{bx_2} = 0.44 \cdot \text{kips}$$

$$V_{by_1} = -0.45 \cdot \text{kips}$$

$$V_{by_2} = 0.45 \cdot \text{kips}$$

Forces in the joint. $L = 2.25 \cdot \text{in}$

$$T_j = V_{bx_2} \quad V_{jy} = V_{by_2} \quad V_{jz} = T_{b_2}$$

$$M_{jz} = V_{jy} \cdot L \quad M_{jy} = V_{jz} \cdot L \quad \text{Torsion in the joint is negligible.}$$

$$T_j = 0.44 \cdot \text{kips} \quad V_{jy} = 0.45 \cdot \text{kips} \quad V_{jz} = 0.62 \cdot \text{kips}$$

$$M_{jz} = 1.01 \cdot \text{in} \cdot \text{kips} \quad M_{jy} = 1.39 \cdot \text{in} \cdot \text{kips}$$

Stress in the L3X3X3/8 is:

$$A_x = 2.11 \cdot \text{in}^2 \quad S_z = 0.833 \cdot \text{in}^3 \quad S_y = 0.833 \cdot \text{in}^3 \quad A_y = 3 \cdot \text{in} \cdot 0.375 \cdot \text{in} \quad A_y = 1.13 \cdot \text{in}^2 \quad A_z = A_y$$

$$\sigma = \frac{T_j}{A_x} + \frac{M_{jy}}{S_y} + \frac{M_{jz}}{S_z} \quad \sigma = 3.09 \cdot \text{ksi} \quad \text{normal stress in the member is well within allowable.}$$

$$\tau_y = \frac{V_{jy}}{A_y} \quad \tau_y = 0.40 \cdot \text{ksi} \quad \tau_z = \frac{V_{jz}}{A_z} \quad \tau_z = 0.55 \cdot \text{ksi} \quad \text{OK.}$$

Check stress in the weld.

$$w_y = 2.40 \cdot \text{in} \quad w_z = 2.13 \cdot \text{in} \quad A_w = w_y + w_z \quad A_w = 4.53 \cdot \text{in}$$

$$S_{wy} = \frac{w_z^2 \cdot (4 \cdot w_y + w_z)}{6 \cdot (2 \cdot w_y + w_z)} \quad S_{wy} = 1.28 \cdot \text{in}^2 \quad S_{wz} = \frac{w_y^2 \cdot (4 \cdot w_z + w_y)}{6 \cdot (2 \cdot w_z + w_y)} \quad S_{wz} = 1.57 \cdot \text{in}^2$$

$$f_w = \sqrt{\left(\frac{T_j}{A_w} + \frac{M_{jy}}{S_{wy}} + \frac{M_{jz}}{S_{wz}} \right)^2 + \left(\frac{V_{jy}}{w_y} \right)^2 + \left(\frac{V_{jz}}{w_z} \right)^2} \quad f_w = 1.86 \cdot \frac{\text{kips}}{\text{in}}$$

Allowable stress in 3/16" fillet weld using electrode E70XX is: $F_{uw} = 70 \cdot \text{ksi}$

$$F_w = \frac{3}{16} \cdot \text{in} \cdot \frac{1}{\sqrt{2}} \cdot 0.3 \cdot F_{uw} \quad F_w = 2.78 \cdot \frac{\text{kips}}{\text{in}} \quad \text{weld stresses are acceptable.}$$

Client: WHC

Subject: Structural analysis of Safety Related
Electrical Equipment Supports.

Location: 241-C-106 & 241-AY-102

WO/Job No.: ER 4319/W 320

Date: 4/21/97

Checked *5/8/97*

Revised

By: M.B. Lasota

By: M.R. Custer

By

MRC

241-AY-102 - Rack for Building 241-AY-801 Pull-boxes - Figure 17 pages 48 & 49 (ref dwg. H-2-818695 sh 2 & 4).

Wind pressure ref. ASCE 7-93 "Minimum Design Loads for Buildings and Other Structures".

From SDC 4.2 rev. 11 the following are parameters used in wind pressure calculation for Safety Class 3 structures:

Basic wind speed: $V = 70$ mph

Importance Factor: $I = 1.07$

Exposure Category: C

$K_z = 0.8$

$$q_z = 0.00256 \cdot K_z \left(\frac{I \cdot V}{\text{mph}} \right)^2 \cdot \text{psf} \quad q_z = 11.49 \cdot \text{psf}$$

use $q_z = 11.5 \cdot \text{psf}$

From Table 4, for other structures: $G_h = 1.32$

and from table 13.

$C_f = 1.6$

$p_w = q_z \cdot G_h \cdot C_f$

$p_w = 24.29 \cdot \text{psf}$

$F_w = p_w \cdot A$

RACK FOR BLDG 241-AY-801 PULL-BOXES

Client: WHC

Subject: Structural analysis of Safety Related
Electrical Equipment Supports.

Location: 241-C-106 & 241-AY-102

WO/Job No.: ER 4319/W 320

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By: M.R. Custer

By

MLC

Seismic load per UBC (36-1)

$$F_p = Z \cdot I \cdot C_p \cdot W_p$$

From SDC 4.1 rev 11: $Z := 0.12$ $I := 1.25$ $C_p := 2.12$ $a_h := Z \cdot I \cdot C_p$ $a_h = 0.32$

PB-1 $h_1 := 8\text{-in}$ $w_1 := 10\text{-in}$ $A_1 := h_1 \cdot w_1$ $y_1 := 43\text{-in}$ $A_1 = 0.56 \cdot \text{ft}^2$ $W_1 := 10\text{-lbf}$

$F_{w1} := P_w \cdot A_1$ $F_{p1} := a_h \cdot W_1$ $F_{w1} = 13\text{-lbf}$ $F_{p1} = 3\text{-lbf}$

PB-2 $h_2 := 20\text{-in}$ $w_2 := 20\text{-in}$ $A_2 := h_2 \cdot w_2$ $y_2 := 37\text{-in}$ $A_2 = 2.78 \cdot \text{ft}^2$ $W_2 := 30\text{-lbf}$

$F_{w2} := P_w \cdot A_2$ $F_{p2} := a_h \cdot W_2$ $F_{w2} = 67\text{-lbf}$ $F_{p2} = 10\text{-lbf}$

The wind load is:

$P_{w1} := F_{w1}$ $P_{w1} = 13\text{-lbf}$ $P_{w2} := F_{w2}$ $P_{w2} = 67\text{-lbf}$

For simplicity and added conservatism use:

$P_{w1} := 20\text{-lbf}$ $P_{w2} := 70\text{-lbf}$

Seismic load is:

$P_{e1} := F_{p1}$ $P_{e1} = 3\text{-lbf}$ $P_{e2} := F_{p2}$ $P_{e2} = 10\text{-lbf}$

Use:

$P_{e1} := 10\text{-lbf}$ $P_{e2} := 10\text{-lbf}$

Check the foundation, 3'-0"x5'-0"x0'-8" deep pad: $a := 5\text{-ft}$ $b := 3\text{-ft}$ $d := 8\text{-in}$

Average density of reinforced concrete is: $\gamma := 150\text{-pcf}$

Dead Weight $W_e := W_1 + W_2 + 2 \cdot 1.9 \cdot \frac{\text{lbf}}{\text{ft}} \cdot (2 \cdot 49\text{-in} + 46\text{-in} + 24\text{-in}) + 2 \cdot 4.08\text{-lbf}$ $W_e = 101\text{-lbf}$

$W_c := a \cdot b \cdot d \cdot \gamma$ $W_c = 1500.00\text{-lbf}$

Bearing capacity of the soil from UBC Table 29B for sandy gravel is: $p_a := 2000\text{-psf}$

The load on the footing is:

$W := W_e + W_c$ $W = 1601\text{-lbf}$

$M := P_{w1} \cdot y_1 + P_{w2} \cdot y_2$ $M = 287\text{-ft-lbf}$

Increase moment by 20% to account for the unistrut frame: $M := 1.2 \cdot M$ $M = 345\text{-ft-lbf}$

Client: WHC
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By: M.B. Lasota

By: M.R. Custer

By

Location: 241-C-106 & 241-AY-102

Check for negative soil pressure:

$$p := \frac{W}{a \cdot b} - \frac{6 \cdot M}{a \cdot b^2} \quad p = 60.8 \cdot \text{psf} \quad \text{uplift at the end of foundation will not occur.}$$

Check footing for overturning:

$$M_s := \frac{W \cdot b}{2} \quad FS_o := \frac{M_s}{M}$$

$$FS_o = 6.96 \quad FS_o \geq 1.5 \quad \text{stability of the foundation is satisfactory.}$$

Check unistrut section for stress:

Bending about major axis:

$$M_x := \frac{M}{2} \quad M_x = 2070 \cdot \text{lbf} \cdot \text{in}$$

Bending about minor axis:

$$P_{wy1} := 49 \cdot \text{in} \cdot 3.25 \cdot \text{in} \cdot p_w \quad P_{wy1} = 26.9 \cdot \text{lbf} \quad P_{wy2} := 10 \cdot \text{in} \cdot 24 \cdot \text{in} \cdot p_w \quad P_{wy2} = 40.5 \cdot \text{lbf}$$

$$P_{sy} := W_e \cdot a_h \quad P_{sy} = 32.2 \cdot \text{lbf}$$

$$M_y := (P_{wy1} \cdot 49 \cdot \text{in} + P_{wy2} \cdot y_2) \cdot 0.5 \quad M_y = 1407 \cdot \text{lbf} \cdot \text{in}$$

Properties for the channel P1001 are as follows:

$$A := 1.11 \cdot \text{in}^2 \quad S_x := 0.572 \cdot \text{in}^3 \quad S_y := 0.588 \cdot \text{in}^3 \quad r_x := 0.915 \cdot \text{in} \quad r_y := 0.656 \cdot \text{in}$$

$$\sigma_x := \frac{M_x}{S_x} \quad \sigma_x = 3.62 \cdot \text{ksi} \quad \sigma_y := \frac{M_y}{S_y} \quad \sigma_y = 2.39 \cdot \text{ksi}$$

The stresses found above occur at different location of the member, also they are a result of two non-concurrent events, the combination is therefore not required.

Check members for axial load.

$$C := \frac{W_e}{2} \quad C = 50.68 \cdot \text{lbf}$$

$$I_x := 49 \cdot \text{in} \quad K_x := 2 \quad Sr_x := \frac{K_x \cdot I_x}{r_x} \quad Sr_x = 107.1$$

$$I_y := 49 \cdot \text{in} - \frac{20 \cdot \text{in}}{2} \quad K_y := 2 \quad Sr_y := \frac{K_y \cdot I_y}{r_y} \quad Sr_y = 118.9 \quad \text{Maximum slenderness ratio.}$$

$$E := 29000 \cdot \text{ksi} \quad F_y := 33 \cdot \text{ksi} \quad \frac{F_y}{2} = 16.50 \cdot \text{ksi}$$

Client: WHC

Subject: Structural analysis of Safety Related
Electrical Equipment Supports.

Location: 241-C-106 & 241-AY-102

WO/Job No.: ER 4319/W 320

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Revised

By: M.B. Lasota

By: M.R. Custer

By

MRC

$$F_{ey} := \frac{\pi^2 \cdot E}{S_r y^2} \quad F_{ey} = 20.24 \cdot \text{ksi} \quad F_{ey} \geq \frac{F_y}{2} \quad F_n := \left(1 - \frac{F_y}{4 \cdot F_{ey}}\right) \cdot F_y \quad F_n = 19.55 \cdot \text{ksi}$$

$$R := \sqrt{\frac{F_y}{2 \cdot F_{ey}}} \quad R = 0.902784 \quad \Omega_c := \frac{5}{3} + \frac{3}{8} \cdot R - \frac{1}{8} \cdot R^3 \quad \Omega_c = 1.91$$

$$P_n := A \cdot F_n \quad P_n = 21702.93 \cdot \text{lb} \quad P_a := \frac{P_n}{\Omega_c} \quad P_a = 11344 \cdot \text{lb} \quad P_a \geq C \quad \text{O.K.}$$

Check standard Base for capacity.

Unistrut nut slip capacity is (ref Unistrut catalog No. 11 page 9):

$$V_1 := 1500 \cdot \text{lb}$$

$$V_a := 2 \cdot V_1 \quad V_a = 3000.00 \cdot \text{lb}$$

Distance between centers of the bolts for part P 2073 A is: $D := 3.25 \cdot \text{in}$

Shear in the bolt due to moment is:

$$V := \frac{M_x}{D} \quad V = 637 \cdot \text{lb} \quad V \leq V_a \quad \text{Part P 2073 A is adequate.}$$

Check expansion anchors:

$$d := 3 \cdot \text{in} + 0.8 \cdot 1.625 \cdot \text{in} + 2 \cdot 0.25 \cdot \text{in} \quad d = 4.80 \cdot \text{in}$$

$$T_b := \frac{M_x}{d} \quad T_b = 431 \cdot \text{lb} \quad V_b := \frac{P_w}{8} \quad V_b = 16 \cdot \text{lb}$$

For 5/8" dia HKB II with 2 3/4" min embedment, installed in $f'_c = 4000 \text{ psi}$ allowable loads are:

$$T_a := 2000 \cdot \text{lb} \quad V_a := 2530 \cdot \text{lb} \quad \text{Ref. A/E Standard GC-ANCR-01 rev. 0, table 21.}$$

Interaction factor is:

$$IF := \frac{T_b}{T_a} + \frac{V_b}{V_a} \quad IF = 0.22 \quad IF \leq 1.0 \quad \text{O.K.}$$

FLUOR DANIEL
NORTHWEST INC.

DESIGN ANALYSIS

Revision 2
Page No. 27a of

Client: WHC
Subject: Structural analysis of Safety Related
Electrical Equipment Supports.

WO/Job No.: ER 4319/W 320
Date: 1/19/98 By: A. S. Langevin
Checked: 1/20/98 By: N. B. Lasota
Revised: By:
Revised By:

Location: 241-C-106 & 241-AY-102

The far north expansion anchor of the building 241-AY-801 pullbox rack was broken during installation and therefore not installed. (See ECN W-320-709) Need to verify that the remaining expansion anchors will be sufficient to carry the loads.

First check loads due to moments about the y-axis:

Due to frame action about the y-axis the moment at the base will be significantly lower than M_y . However with the anchor bolt missing we need to assume a total lack of fixity at the north post about the minor axis. (It becomes a pinned connection). With the loss of fixity at the north post the moment will be higher at the south post as follows:

$$P_y := P_{wy1} + P_{wy2} \quad P_y = 67.34 \cdot \text{lbf}$$

Based on relative stiffness of the two leg members, $P_{\text{south}} = 2/2.5 P_y = 0.8 P_y$

$$M_{y\text{base}} := \frac{0.8 \cdot P_y \cdot 2.25 \cdot \text{in}}{8} \quad M_{y\text{base}} = 336.70 \cdot \text{lbf} \cdot \text{in}$$

$$M_x = 2070.00 \cdot \text{lbf} \cdot \text{in}$$

$M_{y\text{base}} < 1/2 M_x = M_{x\text{base}}$, Therefore, the moment about the x-axis governs

Check anchor bolt loads due to moment about the x axis:

Due to the missing anchor bolt location, T_b is not affected, because this bolt does not take tension in the moment direction. However V_b is affected. Assume only 6 bolts take the shear instead of 8.

$$T_b = 431.25 \cdot \text{lbf}$$

$$V_b := \frac{P_w}{6} \quad V_b = 21.67 \cdot \text{lbf}$$

$$IF := \frac{T_b}{T_a} + \frac{V_b}{V_a} \quad IF = 0.22$$

$IF \leq 1.0$ O.K.

Client: WHC

Subject: Structural analysis of Safety Related
Electrical Equipment Supports.

Location: 241-C-106 & 241-AY-102

WO/Job No.: ER 4319/W 320

Date: 5/06/97

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By: M.B. Lasota

By: M.R. Custer

By

241-AY-102 - Rack for electrical enclosure for NEMA 3R 60A SAFETY SWITCH (PIT COOLING) and Pneumatic Valve Actuator Control Station - Figure 2, (ref dwg H-2-818695 sh 2 & 4 and H-2-818540 sh 1, 2 and 4)

Wind pressure ref. ASCE 7-93 "Minimum Design Loads for Buildings and Other Structures".

From SDC 4.2 rev. 11 the following are parameters used in wind pressure calculation for Safety Class 3 structures:

Basic wind speed: $V = 70$ mph

Importance Factor: $I = 1.07$

Exposure Category: C

$$K_z = 0.8$$

$$q_z = 0.00256 \cdot K_z \cdot \left(\frac{I \cdot V}{\text{mph}} \right)^2 \cdot \text{psf} \quad q_z = 11.49 \cdot \text{psf}$$

$$\text{use } q_z = 11.5 \cdot \text{psf}$$

From Table 4, for other structures: $G_h = 1.32$

and from table 13.

$$C_f = 1.6 \quad p_w = q_z \cdot G_h \cdot C_f \quad p_w = 24.29 \cdot \text{psf} \quad F_w = p_w \cdot A$$

Seismic load per UBC (36-1)

$$F_p = Z \cdot I \cdot C_p \cdot W_p$$

$$\text{From SDC 4.1 rev 11: } Z = 0.12 \quad I = 1.25 \quad C_p = 2.12 \quad a_h = Z \cdot I \cdot C_p \quad a_h = 0.32 \quad i = 1..2$$

$$\text{NEMA 3R 60A S.S. } h_1 = 20\text{-in} \quad w_1 = 8.5\text{-in} \quad A_1 = h_1 \cdot w_1 \quad y_1 = 52.13\text{-in} \quad A_1 = 1.18\text{-ft}^2 \quad W_1 = 20\text{-lbf}$$

$$F_{w_1} = p_w \cdot A_1 \quad F_{p_1} = a_h \cdot W_1 \quad F_{w_1} = 29\text{-lbf} \quad F_{p_1} = 6\text{-lbf}$$

$$\text{Valve Station } h_2 = 20\text{-in} \quad w_2 = 16\text{-in} \quad A_2 = h_2 \cdot w_2 \quad y_2 = 53.5\text{-in} \quad A_2 = 2.22\text{-ft}^2 \quad W_2 = 75\text{-lbf}$$

$$F_{w_2} = p_w \cdot A_2 \quad F_{p_2} = a_h \cdot W_2 \quad F_{w_2} = 54\text{-lbf} \quad F_{p_2} = 24\text{-lbf}$$

Check unistrut section for stress:

Bending about major axis:

Shear in west direction at the south west column is:

$$V_x = \left[h_2 \cdot 10\text{-in} \cdot \left(\frac{25}{20} \right) \cdot p_w + \frac{1}{2} \cdot (2 \cdot 5.5\text{-ft} \cdot 1.625\text{-in} + 3 \cdot 22\text{-in} \cdot 1.625\text{-in} + 8.5\text{-in} \cdot 20\text{-in}) \cdot p_w \right] \cdot \frac{1}{2} \quad V_x = 42\text{-lbf}$$

$$M_x = V_x \cdot (5.5\text{-ft} - 10\text{-in}) \quad M_x = 2342\text{-lbf-in}$$

Client: WHC
Subject: Structural analysis of Safety Related
Electrical Equipment Supports.

WO/Job No.: ER 4319/W 320

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By: M.B. Lasota
By: M.R. Custer
By:

Location: 241-C-106 & 241-AY-102

Bending about minor axis:

Determine bending moment in the west column due to the wind acting in south direction:

Shear due to wind:

$$A_f := 5.5\text{-ft} \cdot 3.25\text{-in} + \frac{3}{2} \cdot (29\text{-in} - 3.25\text{-in}) \cdot 3.25\text{-in} \quad A_f = 2.36\text{-ft}^2$$

$$V_w := A_f p_w + F_{w2} \cdot \frac{19}{29} \quad V_w = 92.71 \cdot \text{lbf} \quad \text{use} \quad V_w = 100\text{-lbf}$$

Conservatively, apply resultant wind load at the location of the box and amplify the moment by eccentric weight:

$$M_y := V_w \cdot y_2 + W_2 \cdot \left(\frac{10\text{-in}}{2} + 1.625\text{-in} \right) \cdot \frac{19}{29} \quad M_y = 5676\text{-in}\cdot\text{lbf}$$

Properties for the channel P1001 are as follows:

$$A := 1.11\text{-in}^2 \quad S_x := 0.572\text{-in}^3 \quad S_y := 0.588\text{-in}^3 \quad r_x := 0.915\text{-in} \quad r_y := 0.656\text{-in}$$

$$\sigma_x := \frac{M_x}{S_x} \quad \sigma_x = 4.09\text{-ksi} \quad \sigma_y := \frac{M_y}{S_y} \quad \sigma_y = 9.65\text{-ksi}$$

The stresses found above occur at different location of the member, also they are a result of two non-concurrent events, the combination is therefore not required.

Check members for axial load.

$$C := W_2 \cdot \frac{19}{29} + \left[2 \cdot 5.5\text{-ft} + \frac{6 \cdot (29\text{-in} - 3.25\text{-in})}{2} \right] \cdot 1.9 \cdot \frac{\text{lbf}}{\text{ft}} \quad C = 82\text{-lbf}$$

$$I_x := 5.5\text{-ft} \quad K_x := 2 \quad S_{r_x} := \frac{K_x \cdot I_x}{r_x} \quad S_{r_x} = 144.26 \quad \text{Maximum slenderness ratio.}$$

$$I_y := 5.5\text{-ft} - \frac{41\text{-in}}{2} \quad K_y := 2 \quad S_{r_y} := \frac{K_y \cdot I_y}{r_y} \quad S_{r_y} = 138.72$$

$$E := 29000\text{-ksi} \quad F_y := 33\text{-ksi} \quad \frac{F_y}{2} = 16.50\text{-ksi}$$

$$F_{ex} := \frac{\pi^2 \cdot E}{S_{r_x}^2} \quad F_{ex} = 13.75\text{-ksi} \quad F_{ex} \geq \frac{F_y}{2} \quad F_n := \left(1 - \frac{F_y}{4 \cdot F_{ex}} \right) \cdot F_y \quad F_n = 13.20\text{-ksi}$$

$$R := \sqrt{\frac{F_y}{2 \cdot F_{ex}}} \quad R = 1.095332 \quad \Omega_c := \frac{5}{3} + \frac{3}{8} \cdot R - \frac{1}{8} \cdot R^3 \quad \Omega_c = 1.91$$

$$P_n := A \cdot F_n \quad P_n = 14656.52\text{-lbf} \quad P_a := \frac{P_n}{\Omega_c} \quad P_a = 7660.94\text{-lbf} \quad P_a \geq C \quad \text{O.K.}$$

Client: WHC

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By: M.R. Custer

By

Check standard Base for capacity.

Unistrut nut slip capacity is (ref Unistrut catalog No. 11 page 9):

$$V_1 := 1500 \cdot \text{lbf}$$

$$V_a := 2 \cdot V_1 \quad V_a = 3000.00 \cdot \text{lbf}$$

Distance between centers of the bolts for part P 2073 A is: $D := 3.25 \cdot \text{in}$

Shear in the bolt due to moment is:

$$V := \frac{M_x}{D} \quad V = 720.57 \cdot \text{lbf} \quad V \leq V_a \quad \text{Part P 2073 A is adequate.}$$

Check expansion anchors:

$$d := 3 \cdot \text{in} + 0.8 \cdot 1.625 \cdot \text{in} + 2 \cdot 0.25 \cdot \text{in} \quad d = 4.80 \cdot \text{in}$$

$$T_b := \frac{M_y}{d} \quad T_b = 1182 \cdot \text{lbf} \quad V_b := \frac{P_w}{8} \quad V_b = 16 \cdot \text{lbf}$$

For 5/8" dia HKB II with 2 3/4" min embedment, installed in $f'_c = 4000 \text{ psi}$ allowable loads are:

$$T_a := 2000 \cdot \text{lbf} \quad V_a := 2530 \cdot \text{lbf} \quad \text{Ref. A/E Standard GC-ANCR-01 rev. 0, table 21.}$$

Interaction factor is:

$$IF := \frac{T_b}{T_a} + \frac{V_b}{V_a} \quad IF = 0.60 \quad IF \leq 1.0 \quad \text{O.K..}$$

**ICF KAISER HANFORD
COMPANY**
DESIGN ANALYSIS

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By:

FIGURE 1
RACK FOR NEMA 3R 600A SWITCH
SCALE 1:36

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By: R.W. Davidson

By:

SECTION A

SECTION B

FIGURE 2
RACK FOR NEMA 3R 60A SWITCH AND
PNEUMATIC VALVE ACTUATOR CONTROL STATION

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DESIGN ANALYSIS

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 FIGURE 3
RACK FOR 16X20X8 BOX - CTB-0621
SCALE 1/2"=1'-0"

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By:

SECTION B

FIGURE 4
RACK FOR 24X30X8 BOX -- ITB-0621
SCALE 1/8"

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By:

By:

FIGURE 5
RACK FOR 30X24X12 BOX - PTBX-0621
SCALE 1/2"

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DESIGN ANALYSIS

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Subject: Structural analysis of Safety Related
Electrical Equipment Supports

Location: 241-C-106 & 241-AY-102

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Page No.: 36 of

By: M.P. Lesota

By: *DD*

By:

FIGURE 6
PROCESS BLDG ELECTR. RACK #1 ER-1361
SCALE 1/30

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COMPANY**

DESIGN ANALYSIS

Calc. No.: W320-24-029

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Subject: Structural analysis of Safety Related
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By:

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SECTION A

SECTION B

PLAN

FIGURE 7
RACK FOR TWO 30X30X12 BOXES - TB-2 & TB-3
SCALE 1/30

DESIGN ANALYSIS

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Subject: Structural analysis of Safety Related Electrical Equipment Supports

Location: 241-C-106 & 241-AY-102

WO/Job No.: ER4319/W320

Date: 11/09/1995

Checked: 12/14/45

Revised:

By: M.B. Lasota

By:

By:

SECTION B

BASE PLATE
SCALE 1:12

SECTION A

PLAN

FIGURE 8
RACK FOR CCTV EQUIPMENT
SCALE 1:30

**ICF KAISER HANFORD
COMPANY**

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Subject: Structural analysis of Safety Related Electrical Equipment Supports

Location: 241-C-106 & 241-AY-102

WO/Job No.: ER4319/W320

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By:

FIGURE 9
RACK FOR TERMINAL BOX - TB-1

SCALE 1:24

SCAF 1:24

DESIGN ANALYSIS

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Subject: Structural analysis of Safety Related
Electrical Equipment Supports

Location: 241-C-106 & 241-AY-102

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Date: 4/29/1997

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By: M.B. Lasota

By: M.R. Custer

By:

FIGURE 10
RACK FOR TB-4 AND LD-1364/1368
SCALE 1/2"

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COMPANY**

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By: M.B. Lasota

Checked: 7/12/96

By: R.W. Davidson

Location: 241-C-106 & 241-AY-102

Revised:

By:

FIGURE 11
PROCESS BLDG CONDUIT SUPPORT #1
SCALE 1/8"=1'-0"

**ICF KAISER HANFORD
COMPANY**

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By: R.W. Davidson

By:

PLAN

SCALE 1/20

BASE PLATE

SCALE 1/2

**ICF KAISER HANFORD
COMPANY**

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By: M.B. Lasota

By: Ruo

By:

FIGURE 12
PROCESS BLDG CONDUIT SUPPORT #2

SCALE 1/24

**ICF KAISER HANFORD
COMPANY**

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By:

By:

FIGURE 13
PROCESS BLDG CONDUIT SUPPORT #3

SCALE 1/24

**ICF KAISER HANFORD
COMPANY**

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By: M.B. Lasota *MB*
By: R.W. Davidson *RD*
By:

FIGURE 14
METER AND SOCKET MOUNTING RACK
SCALE 1/24

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By: M.R. Custer

By:

SECTION A

SECTION B

PLAN

FIGURE 15
RACK FOR 16X16X6 NEMA4 ENCL. LDK-102-32
SCALE 1/2"

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By: M.B. Lasota *ML*By: M.R. Custer *MAC*

By:

FIGURE 16
RACK FOR LD-1363/1366
SCALE 1/2"

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By: M.B. Lasota *ML*
By: M.R. CUSTER *MRC*
By:

FIGURE 17 - ELEVATION LKG. WEST
RACK FOR BLDG 241-AY-801 PULL BOXES
SCALE 1/36

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By: M.B. Lasota

By: M.R. COSTER

By:

FIGURE 17
RACK FOR BLDG 241-AY-801 PULL BOXES
SCALE 1/8"

**ICF KAISER HANFORD
COMPANY**

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Subject: *Structural analysis of Safety Related
Electrical Equipment Supports.*

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Date: 05/06/1997

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Revised N/A

By: M. B. Lasota

By: R. W. Davidson

By N/A

7. CONCLUSIONS.

The preceding calculations show that the Electrical Racks, Conduit Supports and Nitrogen Bottle Rack as designed and detailed are capable of effectively resisting Dead Load and Environmental (horizontal seismic and wind) Loads.

W320-24-030

Electrical Equipment and Conduit Supports
Process Building

	CALCULATION IDENTIFICATION AND INDEX	Page i of iii
		Date 10/26/95

This sheet shows the status and description of the attached Design Analysis sheets.

Discipline Civil

WO/Job No. ER-6159/W-320

Calculation No. W320-24-030

Project No. & Name PROJECT W-320, TANK 241-C-106 WASTE RETRIEVAL

Calculation Item Electrical Equipment and Conduit Supports - Process Building

These calculations apply to:

Dwg. No. H-2-818455 sht.2

Rev. No. 0

Dwg. No. H-2-818678, Shts 1 and 5

Rev. No. 0,0

Other (Study, CDR)

Rev. No.

The status of these calculations is:

☐ Preliminary Calculations

☒ Final Calculations

☐ Check Calculations (On Calculation Dated)

☐ Void Calculation (Reason Voided)

Incorporated in Final Drawings?

☒ Yes ☐ No

This calculation verified by independent "check" calculations?

☐ Yes ☒ No

Original and Revised Calculation Approvals:

	Rev. 0 Signature/Date	Rev. 1 Signature/Date	Rev. 2 Signature/Date
Originator	<i>Michael R. Custer</i> 7/29/96		
Checked by	<i>R. S. Smith</i> 9/13/96		
Approved by	<i>R. S. Smith</i> 5/13/96		
Checked Against Approved Vendor Data	<i>A. Langervin</i> 7/7/98		

INDEX	
Design Analysis Page No.	Description
i	Calculation Identification and Index
ii	Calculation Cross Index
iii	Design Verification Screening Criteria
1,2	OBJECTIVES; CRITERIA; DATA; ASSUMPTIONS; METHODS; REFERENCES; CALCULATIONS; CONCLUSIONS
3-12	DESIGN/ANALYSIS
APPENDIX A	Figures 1 thru 7 (6 Pages)

CALCULATION CROSS INDEX (typical)

Subject Calculation No. W320-24-030

Page ii of iii[illegible]

SFS No. _____

DESIGN VERIFICATION SCREENING CRITERIA

ILL OF ILL

Project/Document No. W320/W320-24-030

When the design or design change affects hardware, formal design verification must be performed if one or more of the following questions must be answered affirmatively (YES).

YES

NO

_____✓

1. Does the design or design change involved meet the established criteria to be considered Safety Class 1?

_____✓

2. Does this design or design change cause or permit changes to Safety Class 1 instrument or alarm setpoints outside of previously approved operational limits?

_____✓

3. Does this design or design change significantly affect the nuclear or environmental safety consequences of a malfunction or failure of the structure, system, or component?

_____✓

4. Does this design or design change involve or change design that has previously undergone formal design verification?

Assigned Lead Engineer

5/13/96
Date

Responsible Discipline Manager

5/13/96
Date

Original Design Package Distribution:

Project Manager

Design Verification Officer

Engineering Document Control

Design Change Distribution:

Attach to Engineering Change Notice

Calc. No. W320-24-030

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DESIGN ANALYSIS

Client WHC

Subject Electrical Equipment and Conduit
Process Building

Support -

WO/Job No. ER-6159/W-320

Date 10/26/95

By M.A. Custer

Checked 5/13/96

By

Revised

By

Location TANK 241-C-106

OBJECTIVE:

The objective of this calculation is to provide a detailed design of a Safety Class 3 support for electrical panels C106-PP1 and SCP-1361 (Equipment No. ER-1362) and an intermediate support for conduits routed from outside the Process Building into panel IE-1361.

CRITERIA:

1. DOE ORDER 6430.1A (04/06/89)
2. HANFORD PLANT STANDARDS, STANDARD ARCH./CIVIL DESIGN CRITERIA, SDC-4.1, REV.11
3. FUNCTIONAL DESIGN CRITERIA, WHC-SD-W320-FDC-001, REV.2

DATA:

1. The equipment and conduit to be supported are Safety Class 3 per Reference 6, therefore the supports are Safety Class 3.

ASSUMPTIONS:

1. None

METHODS:

Hand calculations

REFERENCES:

1. AISC Manual of Steel Construction, Ninth Edition
2. ICBO, Uniform Building Code (UBC), 1991 Edition
3. American Welding Society (AWS), Standard D1.1, Structural Welding Code
4. ASCE 7-93, Minimum Design Loads for Buildings and Other Structures
5. HANFORD PLANT STANDARDS, Standard Arch./ Civil Design Criteria, SDC-4.1
6. WHC-SD-WM-SEL-033, Rev.0-C, Interim Safety Equipment List for 241-C-106 Waste Retrieval
7. Design of Welded Structures, Blodgett (Eighth Edition)

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DESIGN ANALYSIS

Client WHC

Subject Electrical Equipment and Conduit
Supports - Process Building

Location TANK 241-C-106

WO/Job No. ER-6159/W-320

Date 10/26/95

Checked *SL/4u*

Revised

By M.R. Custer

By *Red*

By

*MRC***CALCULATIONS:**

REFER TO DESIGN/ANALYSIS SECTION

CONCLUSIONS:

Based on the evaluation performed, the designed structures are adequate to support the Seismic Class 3 equipment and conduits.

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DESIGN ANALYSIS

Client WHC

Subject **Electrical Equipment and Conduit
Support - Process Building**

WO/Job No. ER-6159/W-320

Date 10/26/95

By M.R Custer *MRC*Checked *5/13/96* By *Red*

Location TANK 241-C-106

Revised

By

DESIGN/ANALYSIS:

The design and analysis of the equipment and conduit supports involves the development of the loads on the structures and design of the support structures and associated connections to the Prefiltration/ Recirculation equipment skid. The equipment and conduits shall be located inside the Process Building, therefore the design loads included in determining the governing load combination, consists of dead and seismic load components only.

DESIGN EQUIPMENT SUPPORT, ER-1362

(Reference Figures 1 and 2)

DESIGN LOADS:**DEAD LOADS**● **Electrical equipment:**

- Mini-power Panel (C106-PP1), 200 lbs
- SCR Temporary Power Control Panel (SCP-1361), 100 lbs

● **Tributary conduit weights: (Based on the following conduit diameters used)**

- 0.50", 1.3 lbs/ ft
- 0.75", 1.9 "
- 1.00", 2.9 "
- 1.50", 5.8 "
- 2.00", 8.7 "
- 2.50", 13.0 "
- 3.00", 18.5 "
- 3.50", 23.8 "
- 4.00", 29.9 "

Equipment and conduit supports located inside the Process Building, so:

LIVE LOAD, L = 0 (Not Applicable)**SNOW LOAD,** S = 0 (Not Applicable)**ROOF LIVE LOAD,** Lr = 0 (Not Applicable)**RAIN LOAD,** R = 0 (Not Applicable)**WIND LOAD,** W = 0 (Not Applicable)

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DESIGN ANALYSIS

Client WHC

Subject **Electrical Equipment and Conduit
Support - Process Building**

WO/Job No. ER-6159/W-320

Date 10/26/95

By M.R Custer *MLC*

Checked 5/13/96

By *RO*

Location TANK 241-C-106

Revised

By

EARTHQUAKE LOADS (Seismic loads distributed based on mass distribution)

• Lateral Seismic Force Components

 $F_p = ZIC_p W_p$, where:

$$Z = 0.20 \text{ (Zone 2B)}$$

$$I = 1.25$$

$$C_p = 0.75 \times 2 = 1.5$$

$$F_p = (0.20)(1.25)(1.5) W_p = 0.375 W_p$$

$$E_n-s = F_p \text{ (100\% max.)} = 0.375 W_p \text{ (1.0)} = 0.375 W_p$$

$$E_e-w = F_p \text{ (30\% max.)} = 0.375 W_p \text{ (0.30)} = 0.113 W_p$$

GOVERNING LOAD COMBINATION/ MAXIMUM LOADS ON THE EQUIPMENT STRUCTURE:

1) D

2) $D + L + (L_r \text{ or } S \text{ or } R)$; L, L_r , S and R = 0, then D3) $D + (W \text{ or } E)$, W = 0, then D + E4) $D + L + (L_r \text{ or } S \text{ or } R) + (W \text{ or } E)$; L, L_r , S, R and W = 0, then D + EGoverning load combination for the design is: (D + E)**EQUIPMENT/ CONDUIT SEISMIC LOAD COMPONENTS:**

Dead Load (1), Wt of Mini Power Panel = 200 lbs

Seismic Load (1), $E_n-s = 0.375 (200) = 75.0 \text{ lbs}$

$$E_e-w = 0.113 (200) = 23.0 \text{ lbs}$$

Dead Load (2), Wt of Control Panel = 100 lbs

Seismic Load (2), $E_n-s = 0.375 (100) = 38 \text{ lbs}$

$$E_e-w = 0.113 (100) = 11 \text{ lbs}$$

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DESIGN ANALYSISClient **WHC**WO/Job No. **ER-6159/W-320**Subject **Electrical Equipment and Conduit
Support - Process Building**Date **10/26/95**By **M.R Custer** *MRC*Checked *5/13/96*By *RS*Location **TANK 241-C-106**

Revised

By

Dead Load (3), Wt of conduits:

<u>Conduit No.</u>	<u>Conduit size</u>	<u>Tributary Length</u>	<u>Weight, lb/ft</u>	<u>Total Wt.lbs</u>
3C011	3/4"	5 ft	2	10
3C014	3/4"	"	2	10
3P002	1"	"	3	15
3P006	1"	"	3	15
3P007	1"	"	3	15
3P008	3/4"	"	2	10
3P016	3/4"	"	2	10
3P240	1"	"	3	15
3P248	1"	"	3	15
3S023	3/4"	"	2	10
				Total weight = 125 lbs

Seismic Loads (3), $E_n-s = 0.375 (125) = 47 \text{ lbs}$
 $E_e-w = 0.113 (125) = 14 \text{ lbs}$

MOMENTS AND FORCES ON EACH COLUMN: (Equipment loads)

- Moments due to seismic component about the Z-axis of the TS:

$$M_{ze1} = [E_e-w(1) (60.0") + E_e-w(2) (24.0")]/ 2 =$$

$$M_{ze1} = [23 \text{ lbs } (60.0") + 11 \text{ lbs } (24.0")]/ 2 = 822 \text{ in-lbs}$$

- Moments due to seismic component about the X-axis of the TS:

$$M_{xe1} = [E_n-s(1) (60.0") + E_n-s(2) (24.0")]/ 2 =$$

$$M_{xe1} = [75 \text{ lbs } (60.0") + 38 \text{ lbs } (24.0")]/ 2 = 2706 \text{ in-lbs}$$

- Axial Force, P_{ae} on each TS, $P_{ae} = (200 \text{ lbs} + 100 \text{ lbs})/2 = 150 \text{ lbs}$

- Moment, M_{xe2} on each column due to eccentric, dead load = $P \times 7.5"$

$$M_{xe2} = 150 \text{ lb } (7.5") = 1125 \text{ in-lb}$$

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Support - Process Building**Date **10/26/95**By **M.R Custer** *MRC*Checked *5/13/96*By *RS*Location **TANK 241-C-106**

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MOMENTS AND FORCES ON EACH TS COLUMN: (Tributary conduit loads)

Conservatively, 50 % of the load due to the dead load of the conduit is applied, as a concentrated load at the vertical centerline of each TS support column with the horizontal seismic component applied at 84" above the base of the TS columns.

- Moments due to the seismic component about the Z-axis of the TS:

$$Mzc1 = Ee-w (84") / 2 =$$

$$Mzc1 = 14 \text{ lbs } (84") / 2 = 588 \text{ in-lb}$$

- Moments due to the seismic component about the X-axis of the TS:

$$Mxc1 = En-s (84") / 2 =$$

$$Mxc1 = 47 \text{ lbs } (84") / 2 = 1974 \text{ in-lb}$$

- Axial Force, Pac on each TS member, Pac = 125/ 2 = 63 lbs

- Moment, Mxc2 on each column due to eccentric, dead load =

$$Mxc2 = 63 \text{ lb } (4") = 252 \text{ in-lb}$$

COMBINED MOMENTS AND FORCES ON EACH TS COLUMN:

$$Mxt = Mxe1 + Mxe2 + Mxc1 + Mxc2 =$$

$$Mxt = 2706 + 1125 + 1974 + 252 = 6057 \text{ in-lb}$$

$$Mzt = Mze1 + Mzc1 =$$

$$Mzt = 822 + 588 = 1410 \text{ in-lb}$$

$$Pta = 150 \text{ lb} + 63 \text{ lb} = 213 \text{ lbs}$$

DESIGN COLUMN per AISC: (TS 3 X 3 X 0.25 column)**(Reference 1, page 5-54)**

$f_a / F_a < \text{or} = 0.15$, then use AISC, Equation H1-3

$$f_a = P/A = 213 \text{ lb} / 2.59 \text{ in}^2 = 0.082 \text{ ksi}$$

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DESIGN ANALYSIS

Client WHC

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Support - Process Building**

WO/Job No. ER-6159/W-320

Date 10/26/95

By M.R Custer

MRC

Checked 5/13/96

By *Res*

Location TANK 241-C-106

Revised

By

Determine Kl/r and F_a :

$$K = 2.1, \quad l = 7.0 \text{ ft} = 84 \text{ in},$$

$$r = 1.10", \quad S_x = S_z = 2.10 \text{ in}^3, \quad I_x = I_z = 3.16 \text{ in}^4$$

$$Kl/r = 2.10 (84 \text{ in}) / 1.1 = 160.1$$

$$C_c = 126.1, \text{ since } Kl/r > C_c$$

$$\text{then } F_a = 12\pi^2 E / 23 (Kl/r)^2 = 5.83 \text{ ksi}$$

$$f_a / F_a = 0.082 / 5.83 = 0.014 < 0.15, \text{ Use AISC formula H1-3}$$

$$f_a / F_a + f_{bx} / F_{bx} + f_{by} / F_{by} < \text{or} = 1.0$$

$$f_{bx} = 6.1 \text{ k-in} / 2.10 = 2.88 \text{ ksi}$$

$$f_{bz} = 1.4 \text{ k-in} / 2.10 = 0.67 \text{ ksi}$$

$$F_y = 46 \text{ ksi}$$

$$F_b = 0.60 F_y = 27.6 \text{ ksi}$$

$$0.014 + 2.88 / 27.6 + 0.67 / 27.6 = 0.14 < 1.0 \text{o.k}$$

Use TS 3 x 3 x 0.25

DESIGN WELD TS 3 x 3 TO SKID:

$$f_w = [(M_z t / S_{wz} + M_x t / S_{wx})^2 + (F_x / A_w)^2 + (F_z / A_w)^2]^{0.5}$$

$$S_w = bd + d^2/3 = 2(2) + (2)^2/3 = 5.33 \text{ in}^2, A_w = 8" \text{ (conservative)}$$

$$f_w = [(1410 / 5.33 + 6057 / 5.33)^2 + (24/8)^2 + (80/8)^2]^{0.5}$$

$$f_w = 1401 \text{ lb/in} = 1.4 \text{ k/in}$$

Minimum weld size required:

$$S = 21000 \text{ psi} = 21.0 \text{ ksi}$$

$$w = f_w / 0.707 S = 1.4 / 0.707(21.0) = 0.09 = 3/32" \text{ fillet}$$

Use 3/16" fillet weld, all-around TS column

$$\text{Weld capacity} = 0.707(3/16")(21.0 \text{ ksi}) = 2.8 \text{ k/in} > 1.4 \text{ k/in} \text{ ..OK}$$

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Support - Process Building**WO/Job No. **ER-6159/W-320**Date **10/26/95**By **M.R. Custer** *MRC*Checked **5/13/96**By *RS*Location **TANK 241-C-106**

Revised

By

DESIGN CONDUIT SUPPORT No. 1

(Reference Figures 3, 4, and 5)

DESIGN LOADS:**DEAD LOADS:**

Conduit No. No.	Dia. (in.)	Wt/ft (ft.)	Tributary Lgth (ft.)	Total Wt. (lbs.)
3S021	1.5	6.0	4.0	24
3S019	3.0	20.0	"	80
3P240	1.0	3.0	"	12
3P007	0.75	2.0	"	8
3P002	1.0	3.0	"	12
3P006	1.0	3.0	"	12
3C014	1.0	3.0	"	12
3S027	2.5	13.0	"	52
3S024	0.75	2.0	"	8
3C012	0.75	2.0	"	8
3S022	0.75	2.0	"	8

Total Dead Weight of Conduit = $W_p = 236$ lbs**SEISMIC LOADS:** $E_n-s = 0.375$ (100%) $W_p = 0.375$ (236) = 89 lbs $E_e-w = 0.375$ (30%) $W_p = 0.113$ (236) = 27 lbs $F_x = 89 / 2 = 45$ lbs $F_y = 236 / 2 = 118$ lbs $F_z = 27 / 2 = 14$ lbs

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DESIGN ANALYSISClient **WHC**Subject **Electrical Equipment and Conduit
Support - Process Building**WO/Job No. **ER-6159/W-320**Date **10/26/95**By **M.R Custer** *MRC*Checked *5/13/96*By *ED*Location **TANK 241-C-106**

Revised

By

$$M_x = (14 \text{ lb})(48") = 672 \text{ in-lb}$$

$$M_y = 0$$

$$M_z = (45 \text{ lb})(100") = 4500 \text{ in-lb}$$

Design Support Members and Connections:

Since the applied forces and moments on Conduit Support No. 1 are less than those on the Equipment Support No. ER-1362, the same member size and connection welds are adequate for Conduit Support No. 1.

Horizontal cross members shall be provided for attachment of "unistrut" type members connecting the conduits to the TS support. These members shall be provided with fillet welds 2" long on the top and bottom sides of each horizontal TS 3 x 3, typical each end of the TS. Stitch welds, 1/8" - 2" long at 6" o.c will be provided on both the top and bottom to attach the "unistrut" type member to the TS (typ.).

The bracing members which are oriented in the Z-direction, provide additional horizontal support to minimize deflection of the vertical TS members and are adequate by engineering judgement. Welding of the braces to be provided shall be 3/16" fillet (or equivalent), 2" long, two opposite sides at the connection to the vertical column and at the base of the brace.

DESIGN EQUIPMENT SUPPORT No. IE-1361

(Reference Figure 6 and 7)

DESIGN LOADS:**DEAD LOADS:**

Weight of Instrument Cabinet, including contents: 450 lbs

SEISMIC LOADS:

$$E_n-s = 0.375 (100\%) W_p = 0.375 (450) = 169 \text{ lbs}$$

$$E_e-w = 0.375 (30\%) W_p = 0.375 (0.30)(450) = 51 \text{ lbs}$$

The E_n-s seismic component is applied parallel to the 24" dimension at the centerline of the cabinet and 72"(2/3) above the base of the cabinet (conservative) which will result in the worst case loading on the anchorage. The E_e-w component is applied parallel to the 36" dimension at the centerline of the panel and 72"(2/3) above the base of the panel.

The reactions from Conduit Support No. 1 will be added to the reactions from Instrument Panel IE-1361 for the design of the equipment/ conduit support anchorage.

Calc. No. W320-24-030

Revision 0

Page No. 10 of 12

DESIGN ANALYSISClient **WHC**Subject **Electrical Equipment and Conduit
Support - Process Building**WO/Job No. **ER-6159/W-320**Date **10/26/95**By **M.R Custer** *MRC*Checked **5/13/96**By *RS*Location **TANK 241-C-106**

Revised

By

Forces and Moments from Equipment No. IE-1361: (Orientation based on Figure 6)

$$F_x = 51 \text{ lbs}$$

$$F_y = 450 \text{ lbs}$$

$$F_z = 169 \text{ lbs}$$

$$M_x = 72''(2/3)(169 \text{ lbs}) = 8112 \text{ lb-in} = 8.1 \text{ k-in}$$

$$M_y = 0$$

$$M_z = 72''(2/3)(51 \text{ lbs}) = 2448 \text{ lb-in} = 2.5 \text{ k-in}$$

Forces and Moments from Conduit Support No. 1: (orientation based on Figure 6)

$$F_x = 45 \text{ lbs}$$

$$M_x = 672 \text{ in-lb}$$

$$F_y = 118 \text{ lbs}$$

$$M_y = 0$$

$$F_z = 14 \text{ lbs}$$

$$M_z = 4500 \text{ in-lb}$$

Maximum combined Loading on the connection:

$$F_x = 51/4 = 13 \text{ lbs} + 45 \text{ lbs} = 58 \text{ lbs}$$

$$F_y = [2448 \text{ in-lb}/42(2)] + [8112 \text{ in-lb}/18(2)] = 29 \text{ lb} + 225 \text{ lb} = 254 \text{ lb}$$

In addition, F_y due to the overturning moments of the conduit support no. 1 are resolved to vertical reactions on the angle at the centerline of the TS member):

$$F_y = 672 \text{ in-lb}/18'' + 4500 \text{ in-lb}/42'' = 146 \text{ lbs}$$

$$F_y (\text{total}) = 254 + 146 = 400 \text{ lbs}$$

$$F_z = 169/4 = 42 \text{ lbs} + 14 \text{ lbs} = 56 \text{ lbs}$$

Design Instrument Cabinet to Skid connection:

Try (1) connection angle $3 \times 5 \times 0.5$, 6" long (LLH) at each corner of the Instrument Cabinet with (3) - 5/8" dia. A-307 (min.) bolts.

Check $\angle 3 \times 5$: (Bending in angle due to eccentric vertical loading)

$$\text{Moment} = \text{Vert. force} \times \text{distance to the C.G. of the weld configuration (Ny)}$$

Calc. No. W320-24-030

Revision 0

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DESIGN ANALYSIS

Client WHC

WO/Job No. ER-6159/W-320

Subject **Electrical Equipment and Conduit
Support - Process Building**

Date 10/26/95

By M.R Custer

MRC

Checked 5/13/96

By

[Signature]

Location TANK 241-C-106

Revised

By

$$N_y = 5'' - (d^2/b + 2d) = 5 - [5^2/6 + 2(5)] = 3.43'' \text{ (Distance from weld C.G to outside of vertical leg of angle)}$$

$$M_{zt} = (254 \text{ lb})(3.43'') + (58 \text{ lb})(3.0'') = 1045.2 \text{ in-lb} = 1.05 \text{ k-in}$$

$$F_b = 22 \text{ ksi}$$

$$f_b = M/S_x, S_x = bd^2/6 = 6(.5)^2/6 = 0.25 \text{ in}^3$$

$$f_b = 1.05/0.25 = 4.2 \text{ ksi} < 22 \text{ ksi} \dots \text{ok}$$

Check A-307 bolts: (3) bolts per connection

$$\text{Area/ bolts} = A_b = 0.202 \text{ in}^2 \times 3 = 0.606$$

$$\text{Allowable shear load/ bolt group} = 0.606 (10 \text{ ksi}) = 6.1 \text{ k (min. per AISC, 6 k).. ok}$$

$$f_{vz} = P/A_b = 56 \text{ lb}/0.606 = 0.09 \text{ ksi (negligible)}$$

$$f_{vy} = P/A_b = 254 \text{ lb}/0.606 = 0.42 \text{ ksi}$$

$$\text{SRSS stress} = 0.44 \text{ ksi} < F_v = 10 \text{ ksi} \dots \text{ok}$$

Check weld configuration: (fillet weld 3 sides, typ.)

Combined Forces and moments on the weld due to the reactions from the Instrument Cabinet and TS Conduit Support:

$$F_{xc} + F_{ts} = 58 \text{ lbs}$$

$$F_{yc} = 254 \text{ lbs}$$

$$F_{yts} = 700 \text{ in-lb}/18'' = 39 \text{ lbs}$$

$$F_{zc} = 42 \text{ lbs}$$

$$F_{zts} = 14 \text{ lbs}$$

$$M_{xc} = 42 \text{ lbs} (2'') = 84 \text{ in-lb}$$

$$M_{xts} = 0$$

$$M_{yc}, M_{yts} = \text{negligible}$$

$$M_{zc} = 194 \text{ lbs} (3.43'') = 665 \text{ in-lb}$$

$$M_{zts} = 4500 \text{ in-lb}$$

Weld Properties:

$$A_w = 16''$$

$$S_{wx} = db + d^2/6 = 6(5) + 6^2/6 = 36 \text{ in}^2$$

Calc. No. W320-24-030

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Page No. 12 of 12

DESIGN ANALYSISClient **WHC**Subject **Electrical Equipment and Conduit
Support - Process Building**WO/Job No. **ER-6159/W-320**Date **10/26/95**By **M.R Custer** *MRC*Checked *5/13/96*By *DS*Location **TANK 241-C-106**

Revised

By

$$Swz = d^2 (2b + d)/3 (b + d) = 5^2 [2(6) + 5]/3 (6 + 5) = 12.9 \text{ in}^2$$

$$fw = [(Fy/ Aw + Mx/ Swx + Mz/ Swz)^2 + (Fx/ Aw)^2 + (Fz/ Aw)^2]^{1/2}$$

$$fw = [(254 + 39)/ 16 + 84/36 + (4500 + 665)/ 12.9)^2 + (58/16)^2 + (42 + 14)/16)^2 =$$

$$fw = 421 \text{ lbs/in}$$

Try 1/4" fillet weld:

$$\text{Weld capacity} = 1/4" (.707)(21000 \text{ lb/in}) = 3712 \text{ lb/in} > 421 \text{ lb/in} \dots \text{ok}$$

Use $\angle$ 3 x 5, 6" long w/ 3 - 5/8" dia., A-307 bolts (min.) to connect the cabinet to the angle and a 1/4" fillet weld on (3) sides to connect the angle to the skid.

Appendix A

Calc. No.: W320-24-030

Revision: 0

Page No.: 1 of 6

NOTES: CONDUITS ARE FIELD ROUTED.
CONDUIT INSTALLATION MAY DIFFER FROM WHAT IS SHOWN.

FIGURE 1
(ELECTRICAL RACK ER-1362)

Appendix A

Calc. No.: W320-24-030

Revision: 0

Page No.: 2 of 6

NOTES: CONDUITS ARE FIELD ROUTED.
CONDUIT INSTALLATION MAY DIFFER FROM WHAT IS SHOWN.

FIGURE 2
(ELECTRICAL RACK ER-1362)

Appendix A

Calc. No.: W320-24-030

Revision: 0

Page No.: 3 of 6

FIGURE 3

(EQUIPMENT NO. IE-1361/CONDUIT SUPPORT #1)

FIGURE 4

(PROCESS BLDG CONDUIT SUPPORT #1)

FIGURE 5
(PROCESS BLDG CONDUIT SUPPORT #1)

Appendix A

Calc. No.: W320-24-030

Revision: 0

Page No.: 6 of 6

W320-24-031

Process Building Equipment Skid
Adjustable Lifting Beam Frame

	CALCULATION IDENTIFICATION AND INDEX	Page i of iii
		Date 02/12/96

This sheet shows the status and description of the attached Design Analysis sheets.

Discipline **Civil** WO/Job No. **ER-6159/W-320** Calculation No. **W320-24-031**

Project No. & Name **PROJECT W-320, TANK 241-C-106 SLUICING**

Calculation Item **Process Building Equipment Skid - Adjustable Lifting Beam Frame Modifications**

These calculations apply to:

Dwg. No. **H-2-83689, Sh 1** Rev. No. **0,**

Dwg. No. Rev. No.

Other (Study, CDR) Rev. No.

The status of these calculations is:

☐ Preliminary Calculations

☒ Final Calculations

☐ Check Calculations (On Calculation Dated)

☐ Void Calculation (Reason Voided)

Incorporated in Final Drawings? ☒ Yes ☐ No

This calculation verified by independent "check" calculations? ☐ Yes ☒ No

Original and Revised Calculation Approvals:

	Rev. 0 Signature/Date	Rev. 1 Signature/Date	Rev. 2 Signature/Date
Originator	<i>Michael R. Carter</i> 3/7/96		
Checked by	<i>Paul D. Smith</i> 3/13/96		
Approved by	<i>Paul D. Smith</i> 3/13/96		
Checked Against Approved Vendor Data	<i>Ann Langrim</i> 4/7/96		

Design Analysis Page No.	Description
i	Calculation Identification and Index
ii	Calculation Cross Index
iii	Design Verification Screening Form
1,2	OBJECTIVES; CRITERIA; DATA; ASSUMPTIONS; METHODS; REFERENCES; CALCULATIONS; CONCLUSIONS
3-5	DESIGN/ANALYSIS
APPENDIX A	Drawing H-2-83689, Sh 1 (ECN)

CALCULATION CROSS INDEX (typical)

Subject Calculation No. W320-24-031, REV. 0

Page 11 of 111[illegible]

DESIGN VERIFICATION SCREENING CRITERIA

PAGE iii OF iiiProject/Document No. W-320/CALCULATION W320-24-031,
REV. 0

When the design or design change affects hardware, formal design verification must be performed if one or more of the following questions must be answered affirmatively (YES).

YES

NO

 ✓

1. Does the design or design change involved meet the established criteria to be considered Safety Class 1?

 ✓

2. Does this design or design change cause or permit changes to Safety Class 1 instrument or alarm setpoints outside of previously approved operational limits?

 ✓

3. Does this design or design change significantly affect the nuclear or environmental safety consequences of a malfunction or failure of the structure, system, or component?

 ✓

4. Does this design or design change involve or change design that has previously undergone formal design verification?

Assigned Lead Engineer

3/13/94

Date

Responsible Discipline Manager

3/13/94

Date

Original Design Package Distribution:

Project Manager

Design Verification Officer

Engineering Document Control

Design Change Distribution:

Attach to Engineering Change Notice

Page G-3

**KAISER ENGINEERS
HANFORD**

Calc. No. W320-24-031

Revision 0

Page No. 1 of 5

DESIGN ANALYSIS

Client WHC

WO/Job No. ER-6159/W-320

Subject Process Building Equipment Skid -
Rigging Frame

Date 02/12/96

By M.R. Custer *MRC*

Checked 3/13/96

By *Paul Souter*

Location TANK 241-C-106

Revised

By

OBJECTIVE:

The objective of this calculation is to provide an evaluation of modifications to an existing adjustable lifting beam frame configuration, as shown on drawings H-2-83689 Sh 1 thru 4, Rev 0 (References 3 thru 6). The modifications include a replacement of the existing W10 x 49 spreader beam with a W12 x 35, use of fixed-frame dimensions of 12'- 1 1/4" x 14'- 1" and a reduction of the maximum lifted load on the rigging frame.

CRITERIA:

1. DOE ORDER 6430.1A (04/06/89)
2. DOE-RL-92-36, Hanford Rigging and Hoisting Manual, January 15, 1993

DATA:

1. The data consists of the information provided on drawings H-2-83689 Shts 1 thru 4, modified as described in the preceding "OBJECTIVE" section of the calculation.

ASSUMPTIONS:

1. None

METHODS:

Hand calculations

REFERENCES:

1. AISC Manual of Steel Construction, Ninth Edition
2. DOE-RL-92-36, Hanford Rigging and Hoisting Manual, January 15, 1993
3. Drawing H-2-83689, Sh 1, Rev 0
4. Drawing H-2-83689, Sh 2, Rev 0
5. Drawing H-2-83689, Sh 3, Rev 0
6. Drawing H-2-83689, Sh 4, Rev 0
7. Drawing H-2-818455, Sh 1, Rev 0
8. Vendor drawing A-1759 submitted per Specification W-320-P05

**KAISER ENGINEERS
HANFORD**

Calc. No. W320-24-031

Revision 0

Page No. 2 of 5

DESIGN ANALYSIS

Client WHC

WO/Job No. ER-6159/W-320

Subject Process Building Equipment Skid -
Rigging Frame

Date 02/12/96

By M.R. Custer

Checked 3/13/96

By

Location TANK 241-C-106

Revised

By

CALCULATIONS:

Refer to the DESIGN/ANALYSIS section of the calculation.

CONCLUSIONS:

Based on the evaluation performed, the modifications made to the adjustable lifting beam frame, as shown on Appendix A, ECN to drawing H-2-83689, Sh 1 and as referenced to drawings H-2-83689, Shts 1, 2, 3 and 4 is structurally adequate to support the loads applied during the lifting and placement of the Process Building equipment skid.

KAISER ENGINEERS
HANFORD**DESIGN ANALYSIS**

Calc. No. W320-24-031

Revision 0

Page No. 3 of 5 *MRC*

Client WHC

WO/Job No. ER-6159/W-320

Subject Process Building Equipment Skid -
Rigging Frame

Date 02/12/96

By M.R Custer *MRC*

Checked 3/13/96

By *Red Douthett*

Location TANK 241-C-106

Revised

By

DESIGN/ANALYSIS:

The design and analysis of the lifting beam involves the development of the applied loads on the frame and a re-evaluation of the frame assembly, based on modified dimensions and member sizes. The evaluation includes determining the magnitude and distribution of forces resulting from a lifted load of 12,000 lbs applied at each of four pick points for a total lifted load of 48,000 lbs.

The total weight of the equipment skid was reduced from the original design value due to a reduction in the estimated, dry weight of the HEME equipment from 27000 lbs to 8900 lbs, as provided by the vendor drawing submittal of drawing A-1759, Project Specification W-320-P05 (Reference 8). Reference 7, drawing H-2-818455, Sh 1, Note 2 provides the magnitude of the lifted load.

DETERMINE FRAME MEMBER FORCES:

- Vertical load at each of (4) pick points, 12000 lbs
- Dimensions of Rigging Frame, 12'- 10 1/2" x 14'- 1" at points of sling attachment
- Vertical angle of slings: 60° (minimum)
- Length of each sling: 20 ft (minimum)

Determine force components on the members, based the proposed rigging geometry:
(The lifting hook of the crane is located at the center of the frame dimensions)

$$a = 12.88 \text{ ft}/2 = 6.44 \text{ ft.}$$

$$b = 14.08 \text{ ft}/2 = 7.04 \text{ ft.}$$

$$c = (6.44^2 + 7.04^2)^{1/2} = 9.54 \text{ ft}$$

$$\tan \theta = 6.44 / 7.04$$

$$\theta = 42.45^\circ$$

Determine the vertical angle of the sling, based on a 20 ft sling length: ($c = 9.53 \text{ ft}$)

$$\cos \phi = 9.53 \text{ ft}/20.0$$

$$\phi = 61.54^\circ$$

Determine the force components in the members:

- The lifted load, vertical component, $b = 12,000 \text{ lbs}$
- Horizontal component, $d = 12000/\tan 61.54^\circ = 6505 \text{ lbs}$
- Horizontal component, $x = \cos \theta (6505 \text{ lbs}) = 4800 \text{ lbs}$
- Horizontal component, $y = \sin \theta (6505 \text{ lbs}) = 4391 \text{ lbs}$

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**KAISER ENGINEERS
HANFORD**

Calc. No. W320-24-031

Revision 0

Page No. 4 of 5

DESIGN ANALYSIS

Client WHC

WO/Job No. ER-6159/W-320

Subject Process Building Equipment Skid -
Rigging Frame

Date 02/12/96

By M.R. Custer *MRC*

Checked 3/13/96

By *Red Dantona*

Location TANK 241-C-106

Revised

By

CHECK THE LIFTING FRAME MEMBERS:

Members used consist of the following:

MEMBER SIZE	W12 X 72	W12 X 35
LENGTH (ft)	12.88	14.08
LOAD (lbs)	4391	4800
Sx (in ³)	97.4	45.6
Sy (in ³)	32.4	7.47
A (in ²)	21.1	10.3
r _x (in)	5.28	5.25
r _y (in)	3.02	1.54
r _T (in)	3.29	1.74
L _c (in)	152.4	82.8
L _u (in)	366.0	151.2

Check W12 X 35: (Reference 1 and 2)

Axial load = 4800 lbs,

Moment, $M_x = (12.25" + 12.5")/2 (4800\text{lbs}) = 59400 \text{ in-lb} = 59.4 \text{ k-in}$ Moment, $M_y = (13.88")(4800 \text{ lb}) = 66600 \text{ in-lb} = 66.6 \text{ k-in}$ $f_a = P/A = 4800/ 10.3 = 466 \text{ psi}$ $K_l/r_y = 1.0 (14.08)(12)/1.54 = 109.7 < C_c = 126.1$, Use $F_a = 11.67 \text{ ksi}$ $f_a/F_a = 0.466/ 11.67 = 0.040 < 0.15$, Use $f_a/F_a + f_{bx}/F_{bx} + f_{by}/F_{by} \leq 1.0$ $1/r_T = 14.08(12)/1.74 = 97.1$ Use $F_b = [2/3 - F_y(1/r_T)^2 / 1530(10)^3 C_b]F_y = 16.0 \text{ ksi}$ $f_{bx} = 59.4 \text{ k-in/} 45.6 = 1.3 \text{ ksi}$

**KAISER ENGINEERS
HANFORD**

Calc. No. W320-24-031

Revision 0

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DESIGN ANALYSIS

Client WHC

WO/Job No. ER-6159/W-320

Subject Process Building Equipment Skid -
Rigging Frame

Date 02/12/96

By M.R Custer *MRC*

Checked 3/13/96

By *Reda*

Location TANK 241-C-106

Revised

By

$$f_{by} = 66.6 \text{ k-in} / 7.47 = 8.9 \text{ ksi}$$

$$f_{bx}/F_{bx} = 1.3 / 16.0 = 0.08$$

$$f_{by}/F_{by} = 8.9 / 22.0 = 0.41$$

$$0.040 + 0.08 + 0.41 = 0.53 < 1.0 \text{ok}$$

Use W12 X 35

Check W12 X 72:

$$\text{Axial load} = 4391 \text{ lbs}$$

$$\text{Moment, } M_x = (4.63") 12000 \text{ lb} + (12.25"/2 + 2.5") 4391 \text{ lbs} = 93.4 \text{ k-in}$$

$$\text{Moment, } M_y = (13.88") (4800 \text{ lbs}) = 66.6 \text{ k-in}$$

$$f_a = 4391 / 21.1 = 208 \text{ psi}$$

$$Kl/r = 1.0 (12.88)(12)/3.02 = 51 < C_c = 126.1, F_a = 18.3 \text{ ksi}$$

$$f_a/F_a = 0.208 / 18.3 = 0.01 < 0.15, \text{ Use } f_a/F_a + f_{bx}/F_{bx} + f_{by}/F_{by} < / = 1.0$$

$$l/rT = 12.88(12)/3.29 = 46.9$$

$$F_{bx} = F_{by} = 22 \text{ ksi}$$

$$f_{bx} = 93.4 \text{ k-in} / 97.4 = 0.96 \text{ ksi}$$

$$f_{by} = 66.6 \text{ k-in} / 32.4 = 2.06 \text{ ksi}$$

$$f_{bx}/F_{bx} = 0.96 / 22 = 0.04$$

$$f_{by}/F_{by} = 2.06 / 22 = 0.09$$

$$f_a/F_a + f_{bx}/F_{bx} + f_{by}/F_{by} = 0.01 + 0.04 + 0.09 = 0.14 < 1.0 \text{ok}$$

Use W12 X 72

Check Connections:

Based on Engineering Judgement, the original design for the connections, as shown on drawings H-2-83689 Shts 1, 2, 3 and 4 is structurally adequate for the new rigging frame configuration, based on the reduced member spans and magnitude of loads, resulting in lower stresses in the connections.

GENERAL NOTES

- ADD —————
- *REPLACE A325 BOLTS, NUTS AND WASHERS ANNUALLY
 - *GREASE ROTATING PIN AS REQUIRED*

6. THE ASSAY WILL BE RUN 10-15 MIN AFTER RECEIVING AND RECORDING THE ASSAY SHEET. IF THE ASSAY SHEET IS NOT RECEIVED, THE ASSAY WILL BE RUN 15 MIN AFTER THE ASSAY SHEET IS RECEIVED. THE ASSAY SHEET MUST BE RECORDED IN THE RECORDING BOOK. THE RECORDING BOOK MUST BE RECORDED IN THE RECORDING BOOK. THE RECORDING BOOK MUST BE RECORDED IN THE RECORDING BOOK.
7. PART THE FOLLOWING INFORMATION MUST BE RECORDED ON BOTH SIDES OF THE RECORDING BOOK:
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8. THE ASSAY SHEET MUST BE RECORDED IN THE RECORDING BOOK. THE ASSAY SHEET MUST BE RECORDED IN THE RECORDING BOOK. THE ASSAY SHEET MUST BE RECORDED IN THE RECORDING BOOK.
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HNF-2468, Rev.0

APPENDIX A

CALC. W320-24-03

SAT. 1 OF

W320-24-032

Sluice Pit AY-02E Analysis and
Design of the Cover Block

☐ Yes ☒ No

	Rev. 0 Signature/Date	Rev. 1 Signature/Date	Rev. 2 Signature/Date
Originator	Michael B. Lasota 4/16/96		
Checked by	William J. Magruder 4/16/96		
Approved by			
Checked Against Approved Vendor Data			

Description

[illegible]

Subject Calculation No.: W320-24-032

Page ii of iii.[illegible]

DESIGN VERIFICATION SCREENING CRITERIA

Project/Document No. W-320/W320-24-032

When the design or design change affects hardware, formal design verification must be performed if one or more of the following questions must be answered affirmatively (YES).

YES	NO	
<u> </u>	<u> ✓ </u>	1. Does the design or design change involved meet the established criteria to be considered Safety Class 1?
<u> </u>	<u> ✓ </u>	2. Does this design or design change cause or permit changes to Safety Class 1 instrument or alarm setpoints outside of previously approved operational limits?
<u> </u>	<u> ✓ </u>	3. Does this design or design change significantly affect the nuclear or environmental safety consequences of a malfunction or failure of the structure, system, or component?
<u> </u>	<u> ✓ </u>	4. Does this design or design change involve or change design that has previously undergone formal design verification?

 Assigned Lead Engineer

4/18/96

 Date

 Responsible Discipline Manager

4/18/96

 Date

Original Design Package Distribution:

Project Manager
 Design Verification Officer
 Engineering Document Control

Design Change Distribution:

Attach to Engineering Change Notice

ICF KAISER HANFORD COMPANY

Calc. No. W320-24-032

Revision: 0

Page No. 1 of 22

DESIGN ANALYSIS

Client: WHC

Subject: Sluice Pit AY-02E Analysis
and Design of the Cover Block.

Location: 241-AY-02E, Sluice Pit

WO/Job No. ER 4319/W 320

Date: 03/21/1996

Checked: 4/16/96

Revised: N/A

By: M. B. Lasota

By: W. J. Magruder

By: N/A

1. OBJECTIVES

The purpose of these calculations is to determine whether the design of Cover Blocks for the 241-AY-02E Sluice Pit meets or exceeds the criteria outlined in SDC-4.1.

2. CRITERIA AND ASSUMPTIONS

DESIGN CRITERIA

Functional Design Criteria WHC-SD-W320-FDC-001, REV. 2 for Tank 241-C-106 Waste Retrieval Project, W-320. The criteria for evaluation of cover blocks is outlined in SDC 4.1 Rev 11. The cover blocks, although classified as Safety Class 3 are located above and supported from Safety Class 1 structure. In order to protect Safety Class 1 structure, Cover Blocks will be subjected to the design loads applicable to Safety Class 1 structures. The safety classification thus becomes "3 over 1".

METHODS

MATCAD 4.0 (A474) was used for evaluation of the Cover Blocks.

4. REFERENCES

1. SDC-4.1 "Standard Arch-civil Design Criteria, Design Loads for Facilities", Rev 11, 9-6-89.
2. "Uniform Building Code", 1994.
3. ASCE 7-93, "Minimum Design Loads for Buildings and Other Structures".
4. ACI 318-89, rev'd 1992, "Building Code Requirements for Reinforced Concrete".
5. ACI 349-90, "Code Requirements for Nuclear Safety Related Concrete Structures".
6. ACI-340.1R-91, "Design Handbook Volume 1 - Beams, One Way Slabs, Brackets, Footings and Pile Caps".
7. Wang and Salmon, "Reinforced Concrete Design", 3rd Edition.
8. ICF KH calculations W320-33-009 rev 1.
9. UCRL 15910.

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COMPANY**

DESIGN ANALYSIS

Calc. No.: W320-24-032

Revision: 0

Page No.: 2 of 22

Client: WHC

Subject: Sluice Pit AY-02E Analysis
and Design of the Cover Block.

Location: 241-AY-02E (Sluice Pit)

WO/Job No.: ER 4319/W 320

Date: 03/19/1996

Checked: 4/16/96

Revised:

By: Michael B. Lasota *me*By: William J. Magruder *WJM*

By:

FIGURE 1
AY-02E EAST COVER BLOCK
SCALE 1/8" = 1'-0"
(PLAN AT MIDDLE OF THE DEPTH)

REF DWG: H-2-818454 SHEET 2

**ICF KAISER HANFORD
COMPANY**
DESIGN ANALYSIS

Calc. No.: W320-24-032

Revision: 0

Page No.: 3 of 22

Client: WHC

Subject: Sluice Pit AY-02E Analysis

and Design of the Cover Block.

Location: 241-AY-02E (Sluice Pit)

WO/Job No.: ER 4319/W 320

Date: 03/19/1996

Checked: 4/16/96

Revised:

By: Michael B. Lasota *MBL*By: William J. Magruder *WJM*

By:

FIGURE 2
SECTION A-A FROM FIGURE 1
SCALE 1/24

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COMPANY**

DESIGN ANALYSIS

Calc. No.: W320-24-032

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Client: WHC

WO/Job No.: ER 4319/W 320

Subject: Sluice Pit AY-02E Analysis
and Design of the Cover Block.

Date: 03/19/1996

By: Michael B. Lasota *MBL*

Location: 241-AY-02E (Sluice Pit)

Checked: 4/16/96

By: William J. Magruder *WJM*

Revised:

By:

FIGURE 3
SECTION B-B FROM FIGURE 1
SCALE 1/24

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and Design of the Cover Block.

Location: 241-AY-02E (Sluice Pit)

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By:

FIGURE 4
SECTION OF THE WEST COVER BLOCK
SCALE 1/2"

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By: M.B. Lasota *MB*
By: W.J. Magruder *WJM*
By:

$$f(x) := \frac{36 \cdot x}{1 + x^2} - \frac{2 \cdot x}{1 - x^2} \cdot \left(\frac{t}{in} + 2.45 \right)$$

$$g(x) := \frac{d}{dx} f(x) \quad x := 0.507669123 \quad a := \text{root}(g(x), x) \quad a = 0.507669123$$

$$f(a) = 9.469883083$$

$$g(a) = 0.000000000$$

$$\alpha := 2 \cdot \text{atan}(a) \quad \alpha = 53.830996878 \cdot \text{deg}$$

Angle α constitutes the critical angle, at which shielding properties of the above arrangement will be equivalent to 18 inches of the concrete, ref 8. At this angle the least value of the depth of the upper sleeve is reflected in terms of $f(i)$. The value of the depth of upper sleeve used in further design is 13.0". The following is the graph of the function of the sleeve depth with respect to the value of angle α reflected in in terms of the tangent of half-angle.

$$x := 0.0, 0.04 \dots 0.66 \quad t := 1.25 \cdot in$$

$$f(x) := \left[\frac{36 \cdot in \cdot x}{1 + x^2} - \frac{2 \cdot x}{1 - x^2} \cdot (t + 2.45 \cdot in) \right] \cdot \frac{1}{in} \quad g(x) := \frac{d}{dx} f(x) \quad h(x) := 0$$

The distance from the bottom of the lower sleeve to the bottom of the upper sleeve is:

$$h_{min} := 31 \cdot in$$

$$d := h_{min} - f(a) \cdot in \quad d = 21.530116917 \cdot in$$

Use $d=18 in$.

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By: M.B. Lasota *MBL*

By: W.J. Magruder *WJM*

By:

Determine minimum thickness of the plugs lower cylinder.

$$18\text{-in} = \frac{t_2}{\cos(\beta)} + \frac{1.25\text{-in}}{\cos(\beta)} \cdot \frac{\gamma_s}{\gamma_c} + \frac{4.5\text{-in} - (t_2 + 1.5\text{-in}) \cdot \tan\beta}{\sin\beta}$$

$$18\text{-in} = (t_2 + 1.25\text{-in}) \cdot \frac{\gamma_s}{\gamma_c} \cdot \frac{1}{\cos\beta} + \frac{4.5\text{-in}}{\sin\beta} - \frac{t_2 + 1.5\text{-in}}{\cos\beta}$$

Let:

$$\tan\left(\frac{\alpha}{2}\right) = x \quad \text{than:} \quad \sin(\alpha) = \frac{2 \cdot x}{1 + x^2} \quad \cos(\alpha) = \frac{1 - x^2}{1 + x^2} \quad \text{substitution yields:}$$

$$t_2(\beta) := \left[\frac{18\text{-in} \cdot \cos(\beta)}{\frac{\gamma_s}{\gamma_c} - 1} - \frac{1.25\text{-in} \cdot \frac{\gamma_s}{\gamma_c} - 1.5\text{-in}}{\frac{\gamma_s}{\gamma_c} - 1} - \frac{4.5\text{-in} \cdot \cos(\beta)}{\left(\frac{\gamma_s}{\gamma_c} - 1\right) \cdot \sin(\beta)} \right] \cdot \frac{1}{\sin\beta}$$

$$T_2(\beta) := \frac{d}{d\beta} t_2(\beta) \quad \beta := 0.681502382 \quad a := \text{root}(T_2(\beta), \beta) \quad a = 0.681502382$$

$$t_2(a) = 2.580119567 \quad T_2(a) = 0.000000000$$

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By:

$$\beta := 0.4, 0.5 \dots 0.9$$

$$h(\beta) := 0$$

Minimum thickness of the lower cylinder of the plug is:

$$t_{min} := t_2(a) \cdot in \quad t_{min} = 2.58 \cdot in$$

In design use:

$$t_{cyl} := 3 \cdot in$$

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WEIGHT OF WEST COVER BLOCK

For the reference, see Figure 4.

$$\begin{aligned} b_1 &:= 55\text{-in} & h_1 &:= 7.5\text{-in} & l_1 &:= 102\text{-in} & h_{12} &:= 2.5\text{-in} & b_{2u} &:= 45\text{-in} & b_{2m} &:= 44.375\text{-in} \\ b_{2l} &:= 43.75\text{-in} & l_{2u} &:= 82\text{-in} & l_{2m} &:= 80.75\text{-in} & l_{2l} &:= 79.5\text{-in} & h_2 &:= 6.75\text{-in} & b_{3u} &:= 46.5\text{-in} \\ h_{23} &:= 2.5\text{-in} & b_{3m} &:= 47.375\text{-in} & b_{3l} &:= 48.25\text{-in} & l_{3u} &:= 79.5\text{-in} & l_{3m} &:= 78.25\text{-in} & l_{3l} &:= 77\text{-in} \\ h_3 &:= 10.25\text{-in} & h_{34} &:= 2.5\text{-in} & h &:= h_1 + h_2 + h_3 + h_{12} + h_{23} + h_{34} & h &:= 32.000\text{-in} \end{aligned}$$

Volume of the west cover block is:

$$\begin{aligned} V_1 &:= b_1 \cdot l_1 \cdot h_1 & V_1 &:= 24.349 \cdot \text{ft}^3 & V_{12} &:= (b_{2u} + 0.25\text{-in}) \cdot (l_{2u} + 0.5\text{-in}) \cdot h_{12} & V_{12} &:= 5.401 \cdot \text{ft}^3 \\ A_{2u} &:= b_{2u} \cdot l_{2u} & A_{2u} &:= 3690.000 \cdot \text{in}^2 \\ A_{2m} &:= b_{2m} \cdot l_{2m} & A_{2m} &:= 3583.281 \cdot \text{in}^2 & A_{2l} &:= b_{2l} \cdot l_{2l} & A_{2l} &:= 3478.125 \cdot \text{in}^2 \\ V_2 &:= \frac{h_2 \cdot (A_{2u} + 4 \cdot A_{2m} + A_{2l})}{6} & V_2 &:= 13.998 \cdot \text{ft}^3 & V_{23} &:= b_{3u} \cdot l_{3u} \cdot h_{23} & V_{23} &:= 5.348 \cdot \text{ft}^3 \\ A_{3u} &:= b_{3u} \cdot l_{3u} & A_{3u} &:= 3696.750 \cdot \text{in}^2 \\ A_{3m} &:= b_{3m} \cdot l_{3m} & A_{3m} &:= 3707.094 \cdot \text{in}^2 & A_{3l} &:= b_{3l} \cdot l_{3l} & A_{3l} &:= 3715.250 \cdot \text{in}^2 \\ V_3 &:= \frac{h_3 \cdot (A_{3u} + 4 \cdot A_{3m} + A_{3l})}{6} & V_3 &:= 21.987 \cdot \text{ft}^3 & V_{34} &:= b_{3l} \cdot l_{3l} \cdot h_{34} & V_{34} &:= 5.375 \cdot \text{ft}^3 \\ V_w &:= V_1 + V_{12} + V_2 + V_{23} + V_3 + V_{34} & V_w &:= 76.459 \cdot \text{ft}^3 \end{aligned}$$

Volume displaced by a penetration:

$$\begin{aligned} D_p &:= 8.5\text{-in} & V_p &:= \frac{\pi D_p^2}{4} \cdot h & V_p &:= 1.051 \cdot \text{ft}^3 \\ \text{Volume of steel penetration assembly: } & D_{pi} &:= 8\text{-in} & V_{ps} &:= \frac{\pi (D_p^2 - D_{pi}^2)}{4} \cdot h & V_{ps} &:= 0.120 \cdot \text{ft}^3 \end{aligned}$$

The density of normal weight reinforced concrete and steel is 150pcf and 490pcf respectively.

$$\gamma_{rc} := 150\text{-pcf} \quad \gamma_s := 490\text{-pcf}$$

Total weight of concrete cover block is:

$$W_{wcb} := (V_w - V_p) \cdot \gamma_{rc} + V_{ps} \cdot \gamma_s \quad W_{wcb} = 11370 \cdot \text{lb}$$

In design use:

$$W_{wcb} := 11500 \cdot \text{lb}$$

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By: W.J. Magruder
By:

WEIGHT OF EAST COVER BLOCK

For the reference, see Figures 2 @ 3.

$$\begin{aligned} b_{11} &:= 70.75\text{-in} & h_{11} &:= 7.5\text{-in} & l_{11} &:= 102\text{-in} & b_{2u} &:= 60.75\text{-in} & b_{2m} &:= 60.13\text{-in} & b_{2l} &:= 59.5\text{-in} \\ h_{12} &:= 2.5\text{-in} & l_{2u} &:= 82\text{-in} & l_{2m} &:= 80.75\text{-in} & l_{2l} &:= 79.5\text{-in} & h_{21} &:= 3.75\text{-in} & h_{23} &:= 2.5\text{-in} \\ b_{3u} &:= 57\text{-in} & b_{3m} &:= 54.75\text{-in} & b_{3l} &:= 52.5\text{-in} & l_{3u} &:= 79.5\text{-in} & l_{3m} &:= 78.25\text{-in} & l_{3l} &:= 77\text{-in} \\ h_{31} &:= 13.25\text{-in} & h_{34} &:= 2.5\text{-in} & h &:= h_{11} + h_{21} + h_{31} + h_{12} + h_{23} + h_{34} & & & & & h &:= 32,000\text{-in} \end{aligned}$$

Volume of the west cover block is:

$$\begin{aligned} V_{11} &:= b_{11} \cdot l_{11} \cdot h_{11} & V_{11} &:= 31,322 \cdot \text{ft}^3 & V_{12} &:= (b_{2u} + 0.25\text{-in}) \cdot (l_{2u} + 0.5\text{-in}) \cdot h_{12} & V_{12} &:= 7,281 \cdot \text{ft}^3 \\ A_{2u} &:= b_{2u} \cdot l_{2u} & A_{2u} &:= 4981,500 \cdot \text{in}^2 & A_{2m} &:= b_{2m} \cdot l_{2m} & A_{2m} &:= 4855,498 \cdot \text{in}^2 \\ A_{2l} &:= b_{2l} \cdot l_{2l} & A_{2l} &:= 4730,250 \cdot \text{in}^2 \\ V_2 &:= \frac{h_{21} \cdot (A_{2u} + 4 \cdot A_{2m} + A_{2l})}{6} & V_2 &:= 10,537 \cdot \text{ft}^3 & V_{23} &:= b_{2l} \cdot l_{2l} \cdot h_{23} & V_{23} &:= 6,844 \cdot \text{ft}^3 \\ A_{3u} &:= b_{3u} \cdot l_{3u} & A_{3u} &:= 4531,500 \cdot \text{in}^2 & A_{3m} &:= b_{3m} \cdot l_{3m} & A_{3m} &:= 4284,188 \cdot \text{in}^2 \\ A_{3l} &:= b_{3l} \cdot l_{3l} & A_{3l} &:= 4042,500 \cdot \text{in}^2 \\ V_3 &:= \frac{h_{31} \cdot (A_{3u} + 4 \cdot A_{3m} + A_{3l})}{6} & V_3 &:= 32,858 \cdot \text{ft}^3 & V_{34} &:= b_{3l} \cdot l_{3l} \cdot h_{34} & V_{34} &:= 5,849 \cdot \text{ft}^3 \end{aligned}$$

Volume of concrete missing due to penetrations

$$\begin{aligned} OD_{1u} &:= 38.5\text{-in} & A_{p1u} &:= \frac{\pi \cdot OD_{1u}^2}{4} & h_{1u} &:= 14\text{-in} & OD_{1l} &:= 36\text{-in} & A_{p1l} &:= \frac{\pi \cdot OD_{1l}^2}{4} \\ V_{p1} &:= A_{p1l} \cdot (h - h_{1u}) + A_{p1u} \cdot h_{1u} & & & & & V_{p1} &:= 20,035 \cdot \text{ft}^3 \\ OD_{2u} &:= 14.00\text{-in} & A_{p2u} &:= \frac{\pi \cdot OD_{2u}^2}{4} & h_{2u} &:= 4\text{-in} & OD_{2l} &:= 12.75\text{-in} & A_{p2l} &:= \frac{\pi \cdot OD_{2l}^2}{4} \\ V_{p2} &:= A_{p2l} \cdot (h - h_{2u}) + A_{p2u} \cdot h_{2u} & & & & & V_{p2} &:= 2,425 \cdot \text{ft}^3 \\ OD_{3u} &:= 10.75\text{-in} & A_{p3u} &:= \frac{\pi \cdot OD_{3u}^2}{4} & h_{3u} &:= 4\text{-in} & OD_{3l} &:= 8.625\text{-in} & A_{p3l} &:= \frac{\pi \cdot OD_{3l}^2}{4} \\ V_{p3} &:= A_{p3l} \cdot (h - h_{3u}) + A_{p3u} \cdot h_{3u} & & & & & V_{p3} &:= 1,157 \cdot \text{ft}^3 \\ V_{\theta} &:= V_{11} + V_2 + V_3 + V_{12} + V_{23} + V_{34} - V_{p1} - V_{p2} - V_{p3} & & & & & V_{\theta} &:= 71,073 \cdot \text{ft}^3 \end{aligned}$$

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By: W.J. Magruder

Volume of three penetrations steel assemblies is:

$$OD_{1u} := 38.5\text{-in} \quad ID_{1u} := 36\text{-in} \quad A_{p1u} := \frac{\pi(OD_{1u}^2 - ID_{1u}^2)}{4} \quad h_{1u} := 14\text{-in} \quad OD_{1l} := 36\text{-in}$$

$$ID_{1l} := 34.5\text{-in} \quad A_{p1l} := \frac{\pi(OD_{1l}^2 - ID_{1l}^2)}{4} \quad h_{1l} := 19.5\text{-in} \quad V_{s1} := A_{p1l}h_{1l} + A_{p1u}h_{1u}$$

$$V_{s1} = 2.122 \cdot ft^3$$

$$OD_{2u} := 14.00\text{-in} \quad ID_{2u} := 13.124\text{-in} \quad A_{p2u} := \frac{\pi(OD_{2u}^2 - ID_{2u}^2)}{4} \quad h_{2u} := 6.5\text{-in} \quad OD_{2l} := 12.75\text{-in}$$

$$ID_{2l} := 11.938\text{-in} \quad A_{p2l} := \frac{\pi(OD_{2l}^2 - ID_{2l}^2)}{4} \quad h_{2l} := 27.5\text{-in} \quad V_{s2} := A_{p2l}h_{2l} + A_{p2u}h_{2u}$$

$$V_{s2} = 0.321 \cdot ft^3$$

$$OD_{3u} := 10.75\text{-in} \quad ID_{3u} := 10.02\text{-in} \quad A_{p3u} := \frac{\pi(OD_{3u}^2 - ID_{3u}^2)}{4} \quad h_{3u} := 6.5\text{-in} \quad OD_{3l} := 8.625\text{-in}$$

$$ID_{3l} := 7.981\text{-in} \quad A_{p3l} := \frac{\pi(OD_{3l}^2 - ID_{3l}^2)}{4} \quad h_{3l} := 27.5\text{-in} \quad V_{s3} := A_{p3l}h_{3l} + A_{p3u}h_{3u}$$

$$V_{s3} = 0.178 \cdot ft^3$$

$$\text{Recall the volume of the concrete: } V_e = 71.073 \cdot ft^3$$

Total weight of concrete cover block. Not including penetration plugs is:

$$W_{ecb} := V_e \cdot \gamma_{rc} + (V_{s1} + V_{s2} + V_{s3}) \cdot \gamma_s \quad W_{ecb} = 11946 \cdot lbf$$

For design use:

$$W_{ecb} := 12000 \cdot lbf$$

Weight of the plug for the pump penetration.

$$r_i := 5\text{-in} \quad r_o := 6.5\text{-in} \quad t_{bl} := 1.5\text{-in} \quad t_t := 2\text{-in} \quad d_t := 42\text{-in} \quad t_m := 5\text{-in} \quad d_m := 35.5\text{-in} \quad t_b := 6.5\text{-in}$$

$$d_{bl} := 29.5\text{-in} \quad h_{bl} := 6\text{-in}$$

$$V_{plg} := \frac{\pi}{4} \left[(r_o^2 - r_i^2) \cdot t_{bl} + d_t^2 \cdot t_t + d_m^2 \cdot (t_m + t_b) - d_{bl}^2 \cdot t_b \right] + 2 \cdot h_{bl} \cdot t_{bl}^2 \quad V_{plg} = 5.659 \cdot ft^3$$

$$W_{plg} := V_{plg} \cdot \gamma_s \quad W_{plg} = 2772.840 \cdot lbf$$

In further design use:

$$W_{plg} := 2780 \cdot lbf$$

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Cover Block Design Objective: The cover block design will consist of two parts. First, the east cover block will be designed as a system of beams spanning between penetrations. Second, a cover block design will be performed on the west cover block.

PART 1 Design of the East Cover Block.

Dead Load:

$$W_{ecb} = 12.000 \cdot \text{kips} \quad w := 57 \cdot \text{in} \quad l := 79.5 \cdot \text{in}$$

$$w_{d1} := \frac{W_{ecb}}{w \cdot l} \quad w_{d1} = 381.331 \cdot \text{psf} \quad \text{Maximum weight per square foot of cover block.}$$

$$W_{plg1} := 2780 \cdot \text{lb} \quad \text{Weight of steel plug for pump penetration.}$$

$$W_{plg2} := 350 \cdot \text{lb} \quad \text{Weight of the plug for 12" penetration. See H-2-818454, sh. 3}$$

$$W_{plg3} := 190 \cdot \text{lb} \quad \text{Weight of the plug for 8" penetration. See H-2-818454, sh. 3}$$

$$W_{d2} := \frac{W_{plg1}}{2} \quad W_{d2} = 1.390 \cdot \text{kips}$$

$$W_{d3} := \frac{W_{plg2}}{2} \quad W_{d3} = 0.175 \cdot \text{kips}$$

$$W_{d4} := \frac{W_{plg3}}{2} \quad W_{d4} = 0.095 \cdot \text{kips}$$

$$\text{Live Load: } q_{ll} := 50 \cdot \text{psf} \quad w_1 := 70.75 \cdot \text{in} \quad l_1 := 102 \cdot \text{in}$$

$$w_l := q_{ll} \cdot \frac{w_1 \cdot l_1}{w \cdot l} \quad w_l = 79.6 \cdot \text{psf} \quad \text{Floor load is conservative since access on top of pit is restricted.}$$

Seismic Load:

An equivalent static evaluation will be used in lieu of a dynamic analysis for a simplified and conservative approach. The lateral cover block load must be multiplied by 1.5 per reference 9. In the equivalent static evaluation the fundamental frequency is not calculated but is assumed to be such that response acceleration is the peak of the response spectrum.

$$z := 0.47$$

Response Spectrum Acceleration (Ref 1, pg 24, Table 6)
@ 7% of Critical Damping (Conservative)

$$F_{H1} := 1.5 \cdot z \cdot w_{d1} \quad F_{H2} := 1.5 \cdot z \cdot W_{d2} \quad F_{H3} := 1.5 \cdot z \cdot W_{d3} \quad F_{H4} := 1.5 \cdot z \cdot W_{d4}$$

$$F_{H1} = 268.8 \cdot \text{psf} \quad F_{H2} = 0.980 \cdot \text{kips} \quad F_{H3} = 0.123 \cdot \text{kips} \quad F_{H4} = 0.067 \cdot \text{kips}$$

The vertical seismic load is 2/3 of the horizontal load (REF 1).

$$w_{e1} := \frac{2}{3} \cdot F_{H1} \quad w_{e2} := \frac{2}{3} \cdot F_{H2} \quad w_{e3} := \frac{2}{3} \cdot F_{H3} \quad w_{e4} := \frac{2}{3} \cdot F_{H4}$$

$$w_{e1} = 179.2 \cdot \text{psf} \quad w_{e2} = 0.653 \cdot \text{kips} \quad w_{e3} = 0.082 \cdot \text{kips} \quad w_{e4} = 0.045 \cdot \text{kips}$$

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By: W.J. Magruder *WJM*
By:

Environmental Loads:

Ashfall and snow loading will be combined as stated in SDC 4.1, Rev. 11, page 8 and substituted for tornado loads as stated in the SDC and ACI 349, sec. 9.2.1, load case 5. By inspection, a wind blown object as described in SDC 4.1 will not penetrate the cover blocks.

Ashfall loading: $w_{ash} := 24 \cdot \text{psf}$

Snow loading: $w_{sn} := 20 \cdot \text{psf}$

$w_a := (w_{ash} + w_{sn}) \cdot \frac{w_{f1} \cdot l_1}{w \cdot l} \quad w_a = 70.071 \cdot \text{psf} \quad \text{Combined distributed load from snow and ashfall.}$

Combined Loading: The following load cases were determined to be valid load combinations from ACI 349, sec 9.2.1 which will be compared to determine the Ultimate Moment.

$$w_1 := 1.4 \cdot w_{d1} + 1.7 \cdot w_l \quad w_1 = 669.2 \cdot \text{psf} \quad w_7 := w_{d1} + w_l \quad w_7 = 461.0 \cdot \text{psf}$$

$$w_2 := 1.4 \cdot w_{d1} + 1.7 \cdot w_l \quad w_2 = 669.2 \cdot \text{psf} \quad w_8 := w_{d1} + w_l + w_{e1} \quad w_8 = 640.2 \cdot \text{psf}$$

$$w_3 := 1.4 \cdot w_{d1} + 1.7 \cdot w_l + 1.7 \cdot w_a \quad w_3 = 788.3 \cdot \text{psf} \quad w_9 := 1.05 \cdot w_{d1} + 1.3 \cdot w_l \quad w_9 = 503.9 \cdot \text{psf}$$

$$w_4 := w_{d1} + w_l + w_{e1} \quad w_4 = 640.2 \cdot \text{psf} \quad w_{10} := 1.05 \cdot w_{d1} + 1.3 \cdot w_l \quad w_{10} = 503.9 \cdot \text{psf}$$

$$w_5 := w_{d1} + w_l + w_a \quad w_5 = 531.0 \cdot \text{psf} \quad w_{11} := 1.05 \cdot w_{d1} + 1.3 \cdot w_l + 1.3 \cdot w_a \quad w_{11} = 595.0 \cdot \text{psf}$$

$$w_6 := w_{d1} + w_l \quad w_6 = 461.0 \cdot \text{psf}$$

Load combination 3 is a controlling combination, hence ultimate moment can be computed using the following loads:

$$w_{31} := w_3 \quad w_{31} = 788.3 \cdot \text{psf} \quad \text{and} \quad W_{32} := 1.4 \cdot W_{d2} \quad W_{32} = 1.946 \cdot \text{kips}$$

$$W_{33} := 1.4 \cdot W_{d3} \quad W_{33} = 0.245 \cdot \text{kips}$$

$$W_{34} := 1.4 \cdot W_{d4} \quad W_{34} = 0.133 \cdot \text{kips}$$

$$\text{Cross beam 1: } b_1 := \frac{38.5 \cdot \text{in}}{2} + 3.94 \cdot \text{in} + \frac{8.625 \cdot \text{in}}{2} \quad b_1 = 2.292 \cdot \text{ft}$$

$$L := 38.5 \cdot \text{in} \quad L = 3.208 \cdot \text{ft} \quad \text{See figure 1.}$$

$$a := \frac{L}{2} \quad \text{Distance from support to the critical section.}$$

$$M_{umax} := \frac{w_{31} \cdot L^2 \cdot b_1}{8} + \frac{W_{32} \cdot L}{4} + \frac{W_{34} \cdot L}{4}$$

$$M_{umax} = 3.992 \cdot \text{kips-ft} \quad \text{maximum moment in the beam 1.}$$

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Location: Tank 241-AY-102, Sluice Pit.

WO/Job No.: W320/ER4319
Date: 03/08/1996 By: M.B. Lasota
Checked: 4/16/96 By: W.J. Magruder
Revised: By:

$$M_u := M_{umax}$$

$$M_u = 3.992 \cdot \text{ft} \cdot \text{kips}$$

Determine required flexural reinforcement:

$$f_c := 4000 \cdot \text{psi} \quad f_y := 60000 \cdot \text{psi}$$

$$h := 27 \cdot \text{in} \quad d := h - (1.5 \cdot \text{in} + 0.375 \cdot \text{in}) \quad \text{Assume \#6 bars.}$$

$$d = 25.125 \cdot \text{in}$$

$$b := 2.875 \cdot \text{in} \quad \text{Approximate width of beam}$$

$$F := \frac{b \cdot d^2}{12000} \cdot \text{in}^{-3} \quad F = 0.151$$

$$K_n := \frac{M_u}{F} \cdot (\text{ft} \cdot \text{kips})^{-1} \quad K_n = 26.397 \quad \text{Use} \quad K_n := 175 \quad \text{Ref. FLEXURE 2.2. ACI-340.1R-91}$$

$$a_n := 4.37$$

$$A_s := \frac{M_u}{d \cdot a_n} \cdot \frac{\text{in}^3}{\text{ft} \cdot \text{kips}} \quad A_s = 0.036 \cdot \text{in}^2$$

Check minimum steel required ref ACI 10.5

$$A_{smin} := 0.0033 \cdot b \cdot d \quad A_{smin} = 0.238 \cdot \text{in}^2 \quad \text{controls.}$$

Use 1-#6 bar for a steel area of 0.44 sq. inches at the top and bottom of the beam 1 (north of the pump penetration).

Determine shear reinforcement requirements due to shear at the critical section at the pump penetration.

Determine shear and reinforcement requirements due to shear at a distance from the support equal to the depth of the beam.

$$V_{ud1} := \frac{w_3 \cdot L \cdot b_1}{2} + \frac{W_{32}}{2} + \frac{W_{34}}{2} \quad V_{ud1} = 3.938 \cdot \text{kips}$$

Allowable Shear (use width of the section just below the plug crown): $b := 3.94 \cdot \text{in}$

$$V_c := 2 \cdot \sqrt{f_c \cdot \text{psi}} \cdot b \cdot d \quad V_c = 12.522 \cdot \text{kips} \quad \phi := 0.85 \quad V_n := \phi \cdot V_c \quad V_n = 10.643 \cdot \text{kips}$$

$$0.5 \cdot V_n = 5.322 \cdot \text{kips}$$

Since $V_{ud1} < 0.5 \cdot V_n$, Design for shear reinforcement not required per ACI 349, Sec. 11.5.5.1.

Development Length: Development length for number 6 bars is 17 inches. Bars will be extended well past the pump penetration opening.

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Checked: 4/16/96

Revised:

By: M.B. Lasota
By: W.J. Magruder
By:

$$\text{Cross beam 2: } b_1 := \frac{38.5 \cdot \text{in}}{2} + 5.13 \cdot \text{in} + \frac{12.75 \cdot \text{in}}{2} \quad b_1 = 2.563 \cdot \text{ft}$$

$$L := 38.5 \cdot \text{in} \quad L = 3.208 \cdot \text{ft}$$

See figure 1.

$$a := \frac{L}{2}$$

Distance from support to the critical section.

$$M_{\text{umax}} := \frac{w_{31} \cdot L^2 \cdot b_1}{8} + \frac{W_{32} \cdot L}{4} + \frac{W_{33} \cdot L}{4}$$

$$M_{\text{umax}} = 4.357 \cdot \text{kips} \cdot \text{ft}$$

maximum moment in the beam 2.

$$M_u := M_{\text{umax}}$$

$$M_u = 4.357 \cdot \text{ft} \cdot \text{kips}$$

Determine required flexural reinforcement:

$$f_c := 4000 \cdot \text{psi}$$

$$f_y := 60000 \cdot \text{psi}$$

$$h := 27 \cdot \text{in}$$

$$d := h - (1.5 \cdot \text{in} + 0.375 \cdot \text{in})$$

Assume #6 bars.

$$d = 25.125 \cdot \text{in}$$

$$b := 4.5 \cdot \text{in}$$

Approximate width of beam

$$F := \frac{b \cdot d^2}{12000} \cdot \text{in}^{-3}$$

$$F = 0.237$$

$$K_n := \frac{M_u}{F} \cdot (\text{ft} \cdot \text{kips})^{-1}$$

$$K_n = 18.406$$

Use

$$K_n := 175$$

Ref. FLEXURE 2.2. ACI-340.1R-91

$$a_n := 4.37$$

$$A_s := \frac{M_u}{d \cdot a_n} \cdot \frac{\text{in}^3}{\text{ft} \cdot \text{kips}}$$

$$A_s = 0.040 \cdot \text{in}^2$$

Check minimum steel required ref ACI 10.5

$$A_{s \text{ min}} := 0.0033 \cdot b \cdot d$$

$$A_{s \text{ min}} = 0.373 \cdot \text{in}^2$$

controls.

Use 1-#6 bar for a steel area of 0.44 sq. inches at the top and bottom of the beam 1 (south of the pump penetration).

Determine shear reinforcement requirements due to shear at the critical section at the pump penetration.

Determine shear and reinforcement requirements due to shear at a distance from the support equal to the depth of the beam.

$$V_{ud2} := \frac{w_{31} \cdot L \cdot b_1}{2} + \frac{W_{32}}{2} + \frac{W_{33}}{2}$$

$$V_{ud2} = 4.337 \cdot \text{kips}$$

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Checked: 4/16/96

Revised:

By: M.B. Lasota
By: W.J. Magruder
By:

Allowable Shear (use width of the section just below the plug crown): $b = 5.13\text{-in}$

$$V_c := 2 \cdot \sqrt{f_c} \cdot \text{psi} \cdot b \cdot d \quad V_c = 16.304 \cdot \text{kips} \quad \phi := 0.85 \quad V_n := \phi V_c \quad V_n = 13.858 \cdot \text{kips}$$

$$0.5 \cdot V_n = 6.929 \cdot \text{kips}$$

Since $V_{ud2} < 0.5 \cdot V_n$ Design for shear reinforcement not required per ACI 349, Sec. 11.5.5.1.

Development Length: Development length for number 6 bars is 17 inches. Bars will be extended well past the pump penetration opening.

$$\text{Primary beam 1 west side of the cover block: } b_1 := \frac{57 \cdot \text{in}}{2} \quad b_1 = 2.375 \cdot \text{ft}$$

$$L = 79.5 \cdot \text{in} \quad L = 6.625 \cdot \text{ft} \quad \text{See figure 1.}$$

The contributing width of the beam will result in an overlap of the tributary areas for beam 1 & 2 and primary beam 1. Also reactions from beam 1 and 2 are lumped in the center of the primary beam 1, all preceding actions will result in a conservative value of M_u .

$$M_{u\text{max}} := \frac{w_{31} \cdot L^2 \cdot b_1}{8} + \frac{V_{ud1} \cdot L}{4} + \frac{V_{ud2} \cdot L}{4}$$

$$M_{u\text{max}} = 23.977 \cdot \text{kips} \cdot \text{ft} \quad \text{maximum moment in the primary beam 1.}$$

$$M_u := M_{u\text{max}} \quad M_u = 23.977 \cdot \text{ft} \cdot \text{kips}$$

Determine required flexural reinforcement:

$$f_c := 4000 \cdot \text{psi} \quad f_y := 60000 \cdot \text{psi}$$

$$h := 27 \cdot \text{in} \quad d := h - (1.5 \cdot \text{in} + 0.375 \cdot \text{in})$$

$$d = 25.125 \cdot \text{in} \quad \text{Assume \#6 bars.}$$

$$b := 10 \cdot \text{in} \quad \text{Approximate width of beam}$$

$$F := \frac{b \cdot d^2}{12000} \cdot \text{in}^{-3} \quad F = 0.526$$

$$K_n := \frac{M_u}{F} \cdot (\text{ft} \cdot \text{kips})^{-1} \quad K_n = 45.579 \quad \text{Use } K_n = 175 \quad \text{Ref. FLEXURE 2.2. ACI-340.1R-91}$$

$$a_n := 4.37$$

$$A_s := \frac{M_u}{d \cdot a_n} \cdot \frac{\text{in}^3}{\text{ft} \cdot \text{kips}} \quad A_s = 0.218 \cdot \text{in}^2$$

Check minimum steel required ref ACI 10.5

$$A_{s\text{min}} := 0.0033 \cdot b \cdot d \quad A_{s\text{min}} = 0.829 \cdot \text{in}^2 \quad \text{controls.}$$

Use 2-#6 bars for a steel area of 0.88 sq. inches at the top and bottom of the primary beam 1.

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Checked: 4/16/96 By: W.J. Magruder *WJM*
Revised: By:

Determine shear reinforcement requirements due to shear at the critical section at the pump penetration.

Determine shear and reinforcement requirements due to shear at a distance from the support equal to the depth of the beam.

$$V_{ud} := \frac{w_s \cdot L \cdot b_1}{2} + \frac{V_{ud1}}{2} + \frac{V_{ud2}}{2} \quad V_{ud} = 10.339 \cdot \text{kips}$$

Allowable Shear (use average width of the primary beam 1): $b := 10 \cdot \text{in}$

$$V_c := 2 \cdot \sqrt{f_c} \cdot \text{psi} \cdot b \cdot d \quad V_c = 31.781 \cdot \text{kips} \quad \phi := 0.85 \quad V_n := \phi \cdot V_c \quad V_n = 27.014 \cdot \text{kips}$$

$$0.5 \cdot V_n = 13.507 \cdot \text{kips}$$

Since $V_{ud2} < 0.5 \cdot V_n$ Design for shear reinforcement not required per ACI 349, Sec. 11.5.5.1.

Development Length: Development length for number 6 bars is 17 inches. Bars will be extended well past the pump penetration opening.

The primary beam 2, east of penetration is similar to the primary beam 1 with bending moment of the lower magnitude, hence use of the same reinforcement arrangement is justified.

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Checked: 4/16/96

Revised:

By: M.B. Lasota *MBL*
By: W.J. Magruder *WJM*
By:

PART 2 Design of the West Cover Block.

Dead Load:

For the reference, see Figure 4.

$$W_{wcb} = 11.500 \text{ kips}$$

The width of an equivalent rectangular section is:

$$h = 27.000 \text{ in} \quad b_{wcb} := 48.25 \text{ in} - 3 \text{ in} - 2.75 \text{ in} \quad b_{wcb} = 42.500 \text{ in}$$

The following calculations will apply to 1'-0" wide beam strip.

$$W_{wcb} = 11.500 \text{ kips} \quad b_1 := 12 \text{ in} \quad l = 6.625 \text{ ft}$$

$$w_{d1} := \frac{W_{wcb}}{l} \cdot \frac{b_1}{b_{wcb}} \quad w_{d1} = 0.490 \frac{\text{kips}}{\text{ft}}$$

Live Load:

$$q_{ll} := 50 \frac{\text{lb}}{\text{ft}^2} \quad b_1 := 12 \text{ in}$$

$$w_{l1} := b_1 \cdot q_{ll} \quad w_{l1} = 0.050 \frac{\text{kips}}{\text{ft}} \quad \text{Floor load is conservative since access on top of pit is restricted.}$$

Seismic Load:

An equivalent static evaluation will be used in lieu of a dynamic analysis for a simplified and conservative approach. The lateral cover block load must be multiplied by 1.5 per reference 9. In the equivalent static evaluation the fundamental frequency is not calculated but is assumed to be such that response acceleration is the peak of the response spectrum.

$$z := 0.47$$

Response Spectra Acceleration (Ref 1, pg 24, Table 6)
@ 7% of Critical Damping (Conservative)

$$F_{H1} := 1.5 \cdot z \cdot w_{d1} \quad F_{H1} = 0.346 \frac{\text{kips}}{\text{ft}}$$

The vertical seismic load is 2/3 of the horizontal load (REF 1).

$$w_{e1} := \frac{2}{3} \cdot F_{H1} \quad w_{e1} = 0.230 \frac{\text{kips}}{\text{ft}}$$

Environmental Loads:

Ashfall and snow loading will be combined as stated in SDC 4.1, Rev. 11, page 8 and substituted for tornado loads as stated in the SDC and ACI 349, sec. 9.2.1, load case 5. By inspection, a wind blown object as described in SDC 4.1 will not penetrate the cover blocks.

$$\text{Ashfall loading: } w_{ash} := 24 \text{ psf}$$

$$\text{Snow loading: } w_{sn} := 20 \text{ psf}$$

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DESIGN ANALYSIS

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Date: 03/08/1996

Checked: 4/11/96

Revised:

By: M.B. Lasota
By: W.J. Magruder
By:

$$w_a := (w_{ash} + w_{sn}) \cdot b \quad w_a = 0.044 \cdot \frac{\text{kips}}{\text{ft}} \quad \text{Combined distributed load from snow and ashfall.}$$

Combined Loading: The following load cases were determined to be valid load combinations from ACI 349, sec 9.2.1 which will be compared to determine the Ultimate Moment.

$$\begin{aligned} w_1 &:= 1.4 \cdot w_{d1} + 1.7 \cdot w_l & w_1 &= 0.771 \cdot \frac{\text{kips}}{\text{ft}} & w_7 &:= w_{d1} + w_l & w_7 &= 0.540 \cdot \frac{\text{kips}}{\text{ft}} \\ w_2 &:= 1.4 \cdot w_{d1} + 1.7 \cdot w_l & w_2 &= 0.771 \cdot \frac{\text{kips}}{\text{ft}} & w_8 &:= w_{d1} + w_l + w_{e1} & w_8 &= 0.770 \cdot \frac{\text{kips}}{\text{ft}} \\ w_3 &:= 1.4 \cdot w_{d1} + 1.7 \cdot w_l + 1.7 \cdot w_a & w_3 &= 0.846 \cdot \frac{\text{kips}}{\text{ft}} & w_9 &:= 1.05 \cdot w_{d1} + 1.3 \cdot w_l & w_9 &= 0.580 \cdot \frac{\text{kips}}{\text{ft}} \\ w_4 &:= w_{d1} + w_l + w_{e1} & w_4 &= 0.770 \cdot \frac{\text{kips}}{\text{ft}} & w_{10} &:= 1.05 \cdot w_{d1} + 1.3 \cdot w_l & w_{10} &= 0.580 \cdot \frac{\text{kips}}{\text{ft}} \\ w_5 &:= w_{d1} + w_l + w_a & w_5 &= 0.584 \cdot \frac{\text{kips}}{\text{ft}} & w_{11} &:= 1.05 \cdot w_{d1} + 1.3 \cdot w_l + 1.3 \cdot w_a & w_{11} &= 0.637 \cdot \frac{\text{kips}}{\text{ft}} \\ w_6 &:= w_{d1} + w_l & w_6 &= 0.540 \cdot \frac{\text{kips}}{\text{ft}} \end{aligned}$$

Load combination 3 is the controlling combination, hence ultimate moment can be computed using the following loads:

$$w_3 = 0.846 \cdot \frac{\text{kips}}{\text{ft}}$$

$$L := l \quad L = 6.625 \cdot \text{ft} \quad \text{See figure 1.}$$

$$M_{umax} := \frac{w_3 \cdot L^2}{8}$$

$$M_{umax} = 4.641 \cdot \text{kips} \cdot \text{ft} \quad \text{maximum moment in the cover block per one foot of its width.}$$

$$M_u := M_{umax} \quad M_u = 4.641 \cdot \text{ft} \cdot \text{kips}$$

Determine required flexural reinforcement:

$$f_c := 4000 \cdot \text{psi} \quad f_y := 60000 \cdot \text{psi}$$

$$h := 27 \cdot \text{in} \quad d := h - (1.5 \cdot \text{in} + 0.375 \cdot \text{in}) \quad \text{Assume \#6 bars.}$$

$$d = 25.125 \cdot \text{in} \quad b := 12 \cdot \text{in}$$

$$F := \frac{b \cdot d^2}{12000} \cdot \text{in}^{-3} \quad F = 0.631$$

$$K_n := \frac{M_u}{F \cdot (\text{ft} \cdot \text{kips})^{-1}} \quad K_n = 7.352 \quad \text{Use } K_n := 175 \quad \text{Ref. FLEXURE 2.2. ACI-340.1R-91}$$

$$a_n := 4.37 \quad A_s := \frac{M_u}{d \cdot a_n \cdot \text{ft} \cdot \text{kips}} \quad A_s = 0.042 \cdot \text{in}^2$$

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Date: 03/08/1996

Checked: 4/16/96

Revised:

By: M.B. Lasota *ML*
By: W.J. Magruder *WJM*
By:

Check minimum steel required per foot of the width of cover block.

$$A_{smin} := \frac{4}{3} \cdot A_s \quad A_{smin} = 0.056 \cdot \text{in}^2$$

For the average width of the cover block of 3.542' total reinforcement area is:

$$A_s := A_{smin} \cdot \frac{b_{wcb}}{1 \cdot \text{ft}} \quad A_s = 0.200 \cdot \text{in}^2 \text{ controls.}$$

$$A_{smin} := 0.0033 \cdot b_{wcb} \cdot d \quad A_{smin} = 3.524 \cdot \text{in}^2$$

Use #5 bars for a steel area of 2.17 sq. inches.**Determine shear and reinforcing requirements due to shear at the critical section of the pump penetration.**

$$\text{Factored shear at the controlling section is: } w_{d1} = 0.490 \cdot \frac{\text{kips}}{\text{ft}} \quad w_l = 0.050 \cdot \frac{\text{kips}}{\text{ft}}$$

$$V_u := 1.4 \cdot \left(\frac{w_{d1} \cdot L}{2} \right) + 1.7 \cdot \left(\frac{w_l \cdot L}{2} \right) \quad V_u = 2.555 \cdot \text{kips}$$

Allowable Shear:

$$V_c := 2 \cdot \sqrt{f_c \cdot \text{psi} \cdot b \cdot d} \quad V_c = 38.137 \cdot \text{kips} \quad \phi := 0.85$$

$$V_n := \phi \cdot V_c \quad V_n = 32.417 \cdot \text{kips}$$

$$\text{Since } 0.5 \cdot V_n = 16.208 \cdot \text{kips}$$

$$V_u < 0.5 \cdot V_n \quad \text{Design for shear reinforcement not required per ACI 349, Sec. 11.5.5.1.}$$

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COMPANY****DESIGN ANALYSIS**Calc. No. *W320-24-032*Revision: *0*Page No. *22* of *22*Client *WHC*Subject *Sluice Pit AY-02E Analysis*
*and Design of the Cover Block.*Location *241-AY-02E, Sluice Pit*WO/Job No. *ER 4319/W 320*Date *03/21/1996*Checked *4/16/96*Revised *N/A*By *M. B. Lasota* *ML*By *W. J. Magruder* *WJM*By *N/A***6. CONCLUSIONS.**

The Calculation revealed, that the design of the Cover Blocks reflected on ECN W-320-173 and the drawing H-2-818454 meets or exceeds requirements outlined in the Criteria.

W320-24-033

Vehicle Load Over Pipeline

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HANFORD****CALCULATION IDENTIFICATION AND INDEX**Page 1 of 16
Date 4/9/96

This sheet shows the status and description of the attached Design Analysis sheets.

Discipline CIVIL/STRUCT. WO/Job No. P15400 Calculation No. W 320-24-033Project No. & Name W-320Calculation Item VEHICLE LOAD OVER PIPELINE

These calculations apply to:

Dwg. No. H-2-818426 SHTS 142 Rev. No. 0Dwg. No. H-2-818428 Rev. No. 0

Other (Study, CDR) _____

Rev. No. _____

The status of these calculations is:

☐ Preliminary Calculations☒ Final Calculations☐ Check Calculations (On Calculation Dated _____)☐ Void Calculation (Reason Voided _____)

Incorporated in Final Drawings?

☐ Yes☒ No

This calculation verified by independent "check" calculations?

☐ Yes☒ No

Original and Revised Calculation Approvals:

	Rev. 0 Signature/Date	Rev. 1 Signature/Date	Rev. 2 Signature/Date
Originator	<u>R. L. ACKERMAN</u> <u>4/9/96</u>		
Checked by	<u>W. J. Maginn</u> <u>4/11/96</u>		
Approved by	<u>R. L. Ackerman</u> <u>4/11/96</u>		
Checked Against Approved Vendor Data	<u>R. L. Ackerman</u> <u>8/2/96</u>		

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Description

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DESIGN VERIFICATION SCREENING CRITERIA

Project/Document No. W-320/W320-24-033

When the design or design change affects hardware, formal design verification must be performed if one or more of the following questions must be answered affirmatively (YES).

YES	NO	
<u> </u>	<u> ✓ </u>	1. Does the design or design change involved meet the established criteria to be considered Safety Class 1?
<u> </u>	<u> ✓ </u>	2. Does this design or design change cause or permit changes to Safety Class 1 instrument or alarm setpoints outside of previously approved operational limits?
<u> </u>	<u> ✓ </u>	3. Does this design or design change significantly affect the nuclear or environmental safety consequences of a malfunction or failure of the structure, system, or component?
<u> </u>	<u> ✓ </u>	4. Does this design or design change involve or change design that has previously undergone formal design verification?

 Assigned Lead Engineer

4/11/90

 Date

 Responsible Discipline Manager

4/11/90

 Date

Original Design Package Distribution:

Project Manager
 Design Verification Officer
 Engineering Document Control

Design Change Distribution:

Attach to Engineering Change Notice

DESIGN ANALYSIS

W320 -24
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Client	WNC			WO/Job No.	
Subject	VEHICLE LOAD - BEAM OVER			Date	4/9/96
	TRANSFER LIFT			By	R. A. CLEARMAN
				Checked	4/11/96
				By	W.S. Magruder
Location	200E - BUFFALO & SEVENTH STS.			Revised	
				By	

OBJECTIVE

DETERMINE STRESSES IN ENCASEMENT PIPE FROM VEHICLE LOAD ON BEAM. THERE IS PHYSICAL EVIDENCE THAT A VEHICLE DROVE ON BEAM OVER PIPE

DESIGN BASIS

ANALYSIS EFFECT OF SUPERIMPOSED LOAD FROM FILL OVER PIPE FROM FILL AND VEHICLE LOAD

METHOD OF ANALYSIS

HAND CALCULATION

REFERENCES

1. HANDBOOK OF PVC PIPE, UNIBELL PVC PIPE ASSOC, 1991
2. DESIGN AND CONSTRUCTION OF SANITARY AND STORM SEWERS ASCE M&R ENGR. PRACTICE NO. 37, WPCF NO. 9
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4. DWG.

H-2-818426 REV. 0

H-2-818428 REV. 0

SPEC W-320-C1, SECTION 02225

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HANFORD**DESIGN ANALYSIS**

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Client

WO/Job No.

Subject VEHICLE LOAD-BEAM OVER Date 4/5/96 By R. A. OLIVERTRANSFER LINEChecked 4/11/96 By W.J. Maguire

Location

Revised

By

CONCLUSION

STRESSES IN 6" ENCASUREMENT PIPE FROM FILL
AND VEHICLE LOAD IS EXTREMELY LOW.

DESIGN ANALYSIS

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033
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Client	WO/Job No.		
Subject	VEHICLE LOAD-BERM OVER	Date	4/8/96 By R. ACKERMAN
	TRANSFER LANE	Checked	4/11/96 By W.J. Maguire
Location		Revised	By

REFERENCE 2 - PAGE 197

EQUATION 8

WHERE: C_n = LOAD COEFFICIENT
 W = UNIT WT. OF SOL (120 pcf)

$$W_c = C_n \cdot W \cdot B_d^2$$

- RIGID PIPE IN NEGATIVE PROTECTIVE TRENCH

$$B_d = \frac{7 + 0.625 + 6}{12} = 1.03$$

$$p' B_d = 0.5 \quad p' = 0.31$$

$$\frac{H}{B_d} = \frac{3}{1.03} = 1.84$$

PAGE 196, W RIGID CONDUIT NEGATIVE PROTECTION

$$r_{cb} = -0.370 - 0.5$$

$$\text{USE } R_d = 0.3, \quad C_n = 1.8$$

$$W_c = 1.8 (120 \text{ lb/ft}^3) (1.03)^2 = 574 \text{ lb/ft}$$

DESIGN ANALYSIS

Calc. No. W320-24
Revision -033
Page No. 4 of 4

Client <u>WNC</u>	WO/Job No.
Subject <u>VEHICLE LOAD - BERM</u>	Date <u>7/9/96</u> By <u>R. ACKERMAN</u>
<u>OVER TRANSFER LINE</u>	Checked <u>4/11/96</u> By <u>WJ Maguire</u>
Location	Revised

EVEN THOUGH TIRE TRACKS ON BERM LOOKED LIKE A SINGLE PICK-UP 1400 DRIVEN OVER THE BERM - WILL USE H-20 TRUCK LOADS TO DETERMINE PRESSURE ON PIPE

REF. 1, P153 FIG 6.7 - H-20 HIGHWAY LOAD

VERT SOIL PRESS. - 3 FT. OF COVER = 650 #/FT²

PER LINEAL FT PIPE = $(650)(1.63) = 1060 \text{ #/LINEAL FT.}$

TOTAL = $\frac{574 + 1060}{1.634} \text{ #/LINEAL FT.}$

IGNORING SUPPORT FROM SOIL COMPACTED AROUND PIPE, CONSERVATIVE CALCULATION OF STRESS:

REF 3, P160

$$\sigma = \frac{1634(3.125)}{12(.28)} = 1520 \text{ PSI} \rightarrow \text{OK} < 35,000 \text{ PSI}$$

YIELD FOR SCH 40
A-106 B PIPE

6" SCH 40 PIPE

W320-24-034

Process Building Lower Lifting Frame

Subject Calculation No. W320-24-034, Rev. 0

Page 66 of 666[illegible]

DESIGN VERIFICATION SCREENING CRITERIA

PAGE III OF III

Project/Document No. CALL. W320-2A-034, Rev. 0

When the design or design change affects hardware, formal design verification must be performed if one or more of the following questions must be answered affirmatively (YES).

YES	NO	
☐	☒	1. Does the design or design change involved meet the established criteria to be considered Safety Class 1 or 2?
☐	☒	2. Does this design or design change cause or permit changes to Safety Class 1 or 2 instrument or alarm setpoints outside of previously approved operational limits?
☐	☒	3. Does this design or design change significantly affect the nuclear safety consequences of a malfunction or failure of the structure, system, or component?
☐	☒	4. Does this design or design change involve or change design that has previously undergone formal design verification?

Assigned Lead Engineer

Date

Responsible Discipline Manager

Date

Original Design Package Distribution:

Project Engineer
Chief Design Engineer
Engineering Document Control

Design Change Distribution:

Attach to Engineering Change Notice

DESIGN ANALYSIS

Calc. No. W320-24-034

Revision 0

Page No. 1 of 6

Client WHC

Subject Process Building Lower Lifting Frame

Location TANK 241-C-106

WO/Job No. ER-6159/W-320

Date 05/21/96

Checked *ulifpu*

Revised

By M.R. Custer

By *RCS*

By

*MRC***OBJECTIVE:**

The objective of this calculation is to provide a design for a lifting frame to be used in the placement of the Process Building onto the foundation in the C-Tank Farm.

CRITERIA:

1. DOE ORDER 6430.1A (04/06/89)
2. DOE-RL-92-36, Hanford Rigging and Hoisting Manual, January 15, 1993

DATA:

1. The data consists of the information provided on sketches prepared by field construction a shown in Appendix A.

ASSUMPTIONS:

1. None

METHODS:

Hand calculations

REFERENCES:

1. AISC Manual of Steel Construction, Ninth Edition
2. DOE-RL-92-36, Hanford Rigging and Hoisting Manual, January 15, 1993
3. AISC Manual of Steel Construction, 9th Edition
4. ES drawing per Appendix A.

Calc. No. W320-24-034

Revision 0

Page No. 2 of 6

DESIGN ANALYSIS

Client WHC

Subject Process Building Lower Lifting Frame

Location TANK 241-C-106

WO/Job No. ER-6159/W-320

Date 05/21/96

Checked *W. H. H. H.*

Revised

By M.R. Custer

By *D. S.*

By

*MRC***CALCULATIONS:**

Refer to DESIGN/ ANALYSIS section.

CONCLUSIONS:

Based on the evaluation performed, the design of the Safety Class 3, Process Building Lifting Frame is structural adequate.

Calc. No. W320-24-034

Revision 0

Page No. 3 of 6

DESIGN ANALYSISClient **WHC**Subject **Process Building Lower Lifting Frame**WO/Job No. **ER-6159/W- 320**Date **05/21/96**By **M.R Custer MRC**Checked **WJH**By **Red**Location **TANK 241-C-106**

Revised

By

DESIGN/ ANALYSIS:

The design and analysis of the lower lifting frame involves determining the applied load(s) on the frame, evaluating the available framing members, design of the member connections and design of a typical lifting lug. The frame will be utilized as a temporary lifting device to place the pre-constructed Process Building into the C-Tank Farm onto the existing foundation. The loads applied to the frame consists of the dead load of the building. The design of the frame will consider only the dead loads of the building and frame , since the frame is temporary.

DETERMINE FRAME MEMBER FORCES:

- Weight of the Process Building, 5000 lb (6000 lb is used in the evaluation)
- Dimensions of the lower lifting frame, 15' - 0" x 25' - 0"
- Vertical angle of the slings which provide a 20-30 degree (max.) offset
- Weight of the Lifting Frame, $W_f = 17.0' \times 2 \times 35 \text{ lb/ft} + 24.3' \times 2 \times 33 \text{ lb/ft} = 2794 \text{ lb}$, Use 3000 lb

DETERMINE FRAME MEMBER FORCES:

The weight of the building is distributed at the four corners of the lifting frame = $6000 \text{ lb} / 4 = 1500 \text{ lb}$

The building is supported at the corners and the weight is distributed to each the W10 X 33 and the W12 x 35 beams with a maximum moment in the beam is as follows:

• **W10 X 33 :**

- Maximum vertical reaction, 645 lb
- Maximum horizontal reaction, 1160 lb
- Maximum vertical moment , $M_x \text{ max} :$
 $M_x \text{ max} = 18" (645 \text{ lb}) = 11610 \text{ in-lb}$
- Maximum horizontal moment, $M_y \text{ max} :$
 $M_y \text{ max} = 18" (1160 \text{ lb}) = 20880 \text{ in-lb}$

• **W12 X 35:**

- Maximum vertical reaction, $855 \text{ lb} + 645 \text{ lb} + 415 = 1915 \text{ lb}$
- Maximum horizontal reaction, 1299 lb

FIGURE 1 **FIGURE 2** **FIGURE 3**

Calc. No. **W320-24-034**Revision **0**

Page No. 4 of 6

DESIGN ANALYSISClient **WHC**Subject **Process Building Lower Lifting Frame**WO/Job No. **ER-6159/W- 320**Date **05/21/96**By **M.R. Custer****MRC**

Checked

6/14/96 By**RD**Location **TANK 241-C-106**

Revised

By

- Maximum vertical moment, M_x max:

$$- M_x \text{ max} = 16'' (1915 \text{ lb}) + 14'' (855 \text{ lb}) = 42610 \text{ in-lb}$$

- Maximum horizontal moment, M_y max:

$$M_y \text{ max} = 16'' (1299 \text{ lb}) = 20784 \text{ in-lb}$$

- Check combined stresses in the W10 x 33 and W12 x 35:

W10 X33: $f_{bx}/F_{bx} + f_{by}/F_{by} \leq 1.0$

$$f_{bx} = 11610 \text{ in-lb} / 35 \text{ in}^3 = 332 \text{ psi} < 11200 \text{ psi}$$

($L_b = 22.0 \text{ ft}$, $L_c = 8.4 \text{ ft}$,
 $L_u = 16.5 \text{ ft}$, $I/r_T = 123$, so
 use $F_{bx} = 11200 \text{ psi}$)

$$f_{by} = 20880 \text{ in-lb} / 9.2 \text{ in}^3 = 2270 \text{ psi} < 27000 \text{ psi}$$

($F_{by} = .75 F_y = 27000 \text{ psi}$)

$$332 / 11200 + 2270 / 27000 = 0.03 + 0.08 = 0.11 < 1.0 \quad \dots \text{OK}$$

W12 x 35: $f_{bx}/F_{bx} + f_{by}/F_{by} \leq 1.0$

$$f_{bx} = 42610 \text{ in-lb} / 45.6 \text{ in}^3 = 934 \text{ psi} < 18300 \text{ psi}$$

($L_b = 12.0 \text{ ft}$, $L_c = 6.9 \text{ ft}$,
 $L_u = 12.6 \text{ ft}$, $I/r_T = 82.8$, so
 use $F_b = 18300 \text{ psi}$)

$$f_{by} = 20784 \text{ in-lb} / 7.47 \text{ in}^3 = 2782 \text{ psi}$$

($F_{by} = .75 F_y = 27000 \text{ psi}$)

$$934 / 18300 + 2782 / 27000 = 0.05 + 0.10 = 0.15 < 1.0 \quad \dots \text{OK}$$

Try (2)- 3/4" DIA. A-325 bolts for all framing connections (typ., unless noted)
 (bearing type connection with threads included in the shear plane)

$$\text{Single shear capacity of } 3/4'' \text{ dia bolt} = (2) 9.3 \text{ kips} = 18.6 \text{ k} > 6.0 \text{ k (AISC min.)} >> 1.06 \text{ k} \quad \dots \text{OK}$$

CHECK SHEAR STRESS IN THE BEAM WEB: (worst case W10 to W10, apply max. Shear = 1.06 kips):

$$f_v = 1.06 \text{ k} / (5/16) (4'' + 1.5'') = 0.62 \text{ ksi} <<< F_v = 0.40 F_y = 14.4 \text{ ksi}$$

Calc. No. W320-24-034

Revision 0

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DESIGN ANALYSIS

Client WHC

Subject Process Building Lower Lifting Frame

WO/Job No. ER-6159/W- 320

Date 05/21/96

By M.R Custer

MRC

Checked

W. H. H. H.

By

R.D.

Location TANK 241-C-106

Revised

By

CHECK COLUMN ATTACHMENT FOR JACK MECHANISM:

- Maximum moment on the W4 X 13 member = $M_{max} = 6" (6.0 \text{ k} / 4) = 9.0 \text{ k-in}$
 $S_x = 5.46 \text{ in}^3$, $f_b = M_{max} / S_x = 9.0 \text{ k-in} / 5.46 \text{ in}^3 = 1.65 \text{ ksi} \lll F_b = 22 \text{ ksi} \dots \text{OK}$
- Check plate bending:
 $S_x (pl) = bd^3/6 = 5(0.5)^3/6 = 0.10 \text{ in}^3$
 $M_{max} = [(7" - 4.12")/2] [(6" \times 1.5 \text{ k})/7"] = 1.9 \text{ k-in}$
 $f_b = M_{max} / S_x (pl) = 1.9 \text{ k-in} / 0.10 \text{ in}^3 = 19.0 \text{ ksi} \lll 22 \text{ ksi} \dots \text{OK}$
- Maximum shear on the W4 X 13 member = 1.5 k
 $V = 1.5 \text{ kips}$, $A (w4) = [4.16" - 2(0.345)] (0.28") = 0.97 \text{ in}^2$
 $f_v = 1.5 \text{ k} / 0.97 \text{ in}^2 = 1.55 \text{ ksi} \lll F_v = 14.4 \text{ ksi} \dots \text{OK}$
- Check the $\frac{1}{2}"$ dia. A-325 bolts:
 $T = (M_{max} / d) / 2 = [(1.5 \text{ k} \times 6" / 5.5") / 2] = 0.82 \text{ k} \ll 22 \text{ ksi} (.393 \text{ in}^2) = 8.6 \text{ k} \dots \text{OK}$
- Check weld between the W14 X 13 and the $\frac{1}{2}"$ baseplate:
 - Tension force on the weld, $T = (6" \times 1.5 \text{ k}) / 4.13" = 2.18 \text{ k}$
 - Weld capacity = $(3/16") (4.06") (15800 \text{ lb/in}^2) = 12.0 \text{ k} \gg 2.18 \text{ k} \dots \text{OK}$

CHECK LIFTING LUG CAPACITY: (The load on the lug is $6000 \text{ lb}/4 + 3000/4 = 2250 \text{ lb} = 2.3 \text{ k}$ in the vertical direction and 1.3 k in the horizontal direction:

- Check shear on capacity of the lug:
 - $A_v = [4"/2 - 1.125/2] 0.5" = 0.72 \text{ in}^2$
 - $f_v = P / A_v = 2.6 \text{ k} / 0.72 \text{ in}^2 = 3.7 \text{ ksi} \lll F_v = 0.4 (36 \text{ ksi}) = 14.4 \text{ ksi} \dots \text{OK}$
 - Factor of Safety = $36 / 3.7 = 9.7 > 3.0 \dots \text{OK}$

Calc. No. W320-24-034

Revision 0

Page No. 6 of 6

DESIGN ANALYSISClient **WHC**Subject **Process Building Lower Lifting Frame**WO/Job No. **ER-6159/W- 320**Date **05/21/96**By **M.R Custer***MRC*Checked *W.H. Custer*By *W.H. Custer*Location **TANK 241-C-106**

Revised

By

- **Check weld capacity:**
 - $fw = 2.3 \text{ k} / .707 (.25") (4" + 4" + .5" + .5") + 1.3 / .707 (.25) (4 + 4) = 2.4 \text{ ksi} << Fw = 15.8 \text{ ksi} \dots \text{OK}$
 - **Factor of Safety** = $70 / 2.4 = 29.1 > 3.0 \dots \text{OK}$
- **Check local flange bending of the Process Building Column:**
 - $tf > 0.4 (Pbf/Fyc) \frac{1}{2} \text{ (no stiffeners required):}$ $tf = 3/16" = 0.188"$
 - $0.4 [(2.18 \text{ k}/2)(5/3) / (36 \text{ ksi})] \frac{1}{2} = 0.09" < 0.188" \dots \text{O.K. , no stiffeners required}$

Title:

STRUCTURAL - PROCESS BUILDING
ERECTION AND LIFTING FRAME

Prepared by:

MD AXUP

Sh of

1 3

Sketch no.

Rev

Checked by:

Michael R. Custer
6/7/96

Cadline

NA

ES-W-320-C1

2

APPENDIX A

CALC. W320-24-034, REV. 0

SHT 1 OF 3

Title

STRUCTURAL - PROCESS BUILDING
ERECTION AND LIFTING FRAME

Prepared by:

MD AXUP

Sh 2 of 3

Sketch no.

ES-W-320-C2

Rev

0

Checked by:

Michael R. Custer
5/23/96

Cadline

NA

APPENDIX A

CALC. W320-24-034, REV.0

SHT. 2 OF 3

W320
5/23/96
MRC

* COPE TOP FLANGE ONLY

DETAIL 1

SECTION A-A

SECTION B-B

W320-24-035

Recirculation/Exhaust Skid
Structural Evaluations

**FLUOR DANIEL
NORTHWEST, INC.**
CALCULATION IDENTIFICATION AND INDEX

Page 1 of 1

Date

March 28, 1997

This sheet shows the status and description of the attached Design Analysis sheets.

Discipline 24 STRUCTURAL WO/Job W-320

Calculation No. W-320-24-035

Project No. & Name W-320 Tank 240-C-106 Sluicing

Calculation Item Recirculation / Exhaust Skid Structural Evaluations

These calculations apply to:

Dwg. No. _____

Rev. No. _____

Dwg. No. _____

Rev. No. _____

Other (Study, CDR) _____

Rev. No. _____

The status of these calculations is:

☐ Preliminary Calculations

☒ Final Calculations

☐ Check Calculations (On Calculation Dated)

☐ Void Calculation (Reason Voided)

Incorporated in Final Drawings?

☒ Yes ☐ No

This calculation verified by independent "check" calculations?

☐ Yes ☒ No

Original and Revised Calculation Approvals:

	Rev. 0 Signature/Date	Rev. 1 Signature/Date	Rev. 2 Signature/Date
Originator	N. D. Ha <i>N. D. Ha</i> 3/28/97		
Checked by	A. Giller <i>A. Giller</i> 3-31-97		
Approved by	<i>P. D. Daniel</i> 4/1/97		
Checked Against Approved Vendor Data	<i>Ann Hanger</i> 7/1/97		

INDEX

 Design Analysis
Page No.

Description

1

OBJECTIVE, CRITERIA, REFERENCES, ASSUMPTIONS, CONCLUSIONS

2-14

HANGER EVALUATIONS

Fluor Daniel Northwest

Calc # W-320-24-035

Revision 0

Page 1 of 14

DESIGN CALCULATION

Client : Project W-320

WO/Job No

Subject : Recirculation/Exhaust skid evaluation

Date 3/28/97

By N. D. Ha *N.D. Ha*

Location : 200 East Area

Checked 3-31-97

By R. A. Giller *R.A. Giller*

Revised

By

Objective:

The Recirculation / Exhaust support skid is a portion of W-320 project. The HVAC skid consists of the steel frame which supports the HVAC equipment and the concrete slab that supports the frame.

Design criteria:

Operation : Deadweight and wind loading are considered.

Seismic :

The HVAC skid structure is designed in accordance with UBC 1994 seismic requirements for low hazard facility (UCRL 15910 safety class 3 structure).

Assumptions:

Assume the Recirculation and the Exhaust skid structures are similar in design

References :

1. Fluor Daniel North-West A/E Standard GC-Load-01
2. Manual of Steel Construction (AISC), 9th Edition
3. Designing Floor Slab On Grade, 1992. Boyd C. Ringo and Robert B. Anderson
4. Concrete Floors on Ground, 2th Edition. Portland Cement Association.
5. Uniform Building Code (UBC), 1994
6. Design and Evaluation Guidelines for DOE Facilities Subjected to Natural Phenomina Hazards (UCRL 15910)
7. ASCE Standard 7-88

Conclusions : The HVAC Skid structure is designed to meet the Hanford standard requirements.

DESIGN CALCULATION

Client : Project W-320

Subject : Recirculation/Exhaust skid evaluation

Location : 200 East Area

WO/Job No

Date 3/28/97

Checked 3-31-97

Revised

By N. D. Ha
By R. A. Giller

Concrete weight	$w_{concrete} := 145 \frac{lb}{ft^3}$	
Concrete compression	$f_c := 3000 \frac{lb}{in^2}$	
Concrete tension allowable	$f_t := 300 \frac{lb}{in^2}$	
Reinforced steel strength	$f_{y.rebar} := 60000 \frac{lb}{in^2}$	
Soil modulus subgrade reaction k (Sand-gravel) k= load (psi)/deflection (.05in)	$k := 300 \frac{lb}{in^3}$	
Poisson ratio for concrete	$\nu := .15$	
Steel properties		
Bending allowable	$F_{y.bending.allow} := 63600 \frac{lb}{in^2}$	
L3x3x1/4	$A_{L3x3} := 1.44 \text{ in}^2$ $S_{L3x3} := .577 \text{ in}^3$	$w_{L3x3} := 4.9 \frac{lb}{ft}$ $r_{L3x3} := .592 \text{ in}$
TS3x3x1/4	$A_{T3x3} := 2.59 \text{ in}^2$ $S_{T3x3} := 2.1 \text{ in}^3$ $W_{TS.vert} := w_{T3x3} \cdot 88 \text{ in}$ $W_{TS.horz} := w_{T3x3} \cdot 45 \text{ in}$	$r_{T3x3} := 1.1 \text{ in}$ $w_{T3x3} := 8.81 \frac{lb}{ft}$
Weld electrode 70	$f_{y.weld.allow} := 70000 \frac{lb}{in^2}$	
Safety class 3 horizontal acceleration (UBC 1994)	$Z = .2$ $a_{horz.ubc} := I \cdot Z \cdot C_p$ $a_{horz.ubc} = 0.225$	$I = 1.5$ $C_p = .75$

Fluor Daniel Northwest

Calc # W-320-24-035

Revision 0

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DESIGN CALCULATION

Client : Project W-320

Subject : Recirculation/Exhaust skid evaluation

Location : 200 East Area

WO/Job No

Date 3/28/97

Checked 3-31-97

Revised

By

N. D. Ha

By

R. A. Giller

By

Recirculation skidProjected areas of
all components

$$A_{T3x3} := 3 \cdot \text{in} \cdot (3 \cdot 88 + 45) \cdot \text{in}$$

$$A_{\text{recirculator}} := 8.5 \cdot 74 \cdot \text{in}^2$$

$$A_{\text{tank.recir}} := 11 \cdot \text{in} \cdot 23 \cdot \text{in}$$

$$A_{\text{tank.recir}} = 253 \cdot \text{in}^2$$

$$A_{\text{elec.box}} := 4 \cdot \text{in} \cdot 4 \cdot \text{in}$$

$$A_{\text{elec.box}} = 16 \cdot \text{in}^2$$

$$A_{\text{pipe}} := 1.66 \cdot \text{in} \cdot (11 \cdot 12) \cdot \text{in}$$

$$A_f = A_{\text{recirculator}} + A_{T3x3} + A_{\text{tank.recir}} + A_{\text{elec.box}} + A_{\text{pipe}}$$

$$A_f = 2.044 \cdot 10^3 \cdot \text{in}^2$$

Exposure coefficient

$$K_z := .8$$

Wind speed

$$V := 70$$

Importance factor(Safety class 3)

$$I := 1.07$$

Velocity pressure q_z
(Evaluate at z height)

$$q_z := .00256 \cdot K_z \cdot (I \cdot V)^2 \cdot \frac{\text{lb}_f}{\text{ft}^2}$$

$$q_z = 11.489 \cdot \frac{\text{lb}_f}{\text{ft}^2}$$

Exposure C

$$G_z := 1.32$$

$$C_f := 2.0$$

Wind Force on structure
(ASCE 1988)

$$F_{\text{wind.recir}} := q_z \cdot G_z \cdot C_f \cdot A_f$$

$$F_{\text{wind.recir}} = 430.568 \cdot \text{lb}_f$$

Use

$$F_{\text{wind.recir}} := 431 \cdot \text{lb}_f$$

Fluor Daniel Northwest

Calc # W-320-24-035

Revision 0

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DESIGN CALCULATION

Client : Project W-320

Subject : Recirculation/Exhaust skid evaluation

Location : 200 East Area

WO/Job No

Date 3/28/97

Checked 3-31-97

Revised

By

By

By

N. D. Ha

R. A. Giller

Area of the Tube steel

$$A_{TS,vert} = 3\text{-in} \cdot 88\text{-in}$$

$$A_{TS,horz} = 3\text{-in} \cdot 45\text{-in}$$

Distance from CG of components to the base

$$y_{TS,vert} = \frac{88\text{-in}}{2}$$

$$y_{TS,horz} = 88\text{-in}$$

$$y_{recir} = 43\text{-in}$$

$$y_{tank,recir} = 62\text{-in}$$

$$y_{pipe} = 37\text{-in}$$

$$L_{recir,arm} = 7.5\text{-in}$$

$$L_{TS} = 18\text{-in}$$

$$y_{recirculator} = \frac{2 \cdot A_{TS,vert} \cdot y_{TS,vert} + A_{TS,horz} \cdot y_{TS,horz} + A_{recirculator} \cdot y_{recir} + A_{tank,recir} \cdot y_{tank,recir} + A_{pipe} \cdot y_{pipe}}{2 \cdot A_{TS,vert} + A_{TS,horz} + A_{recirculator} + A_{tank,recir} + A_{pipe}}$$

$$y_{recirculator} = 48.723\text{-in}$$

Moment due to Deadweight

$$M_{dw,recir} = (W_{recir} + W_{tank,recir} + W_{pipe} + W_{pump} + W_{misc,pumps}) \cdot L_{recir,arm} + (W_{TS,vert} \cdot 3 + W_{TS,horz}) \cdot \frac{15}{3}\text{-in}$$

$$M_{dw,recir} = 3.857 \cdot 10^3 \text{ in-lbf}$$

Tipping about Steel frame

$$M_{tipping,recir} = F_{wind,recir} \cdot y_{recirculator}$$

$$M_{tipping,recir} = 2.1 \cdot 10^4 \text{ in-lbf}$$

Fluor Daniel Northwest

Calc # W-320-24-035

Revision 0

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DESIGN CALCULATION

Client : Project W-320

Subject : Recirculation/Exhaust skid evaluation

Location : 200 East Area

WO/Job No

Date 3/28/97

Checked 3-31-97

Revised

By

N. D. Ha

By

R. A. Giller

By

Exhaust skid

$$A_{\text{exhaust}} = 6.125 \cdot 32 \cdot \text{in}^2$$

$$A_{T3 \times 3} = 3 \cdot \text{in} \cdot (3 \cdot 88 + 45) \cdot \text{in}$$

$$A_{\text{tank.exhaust}} = 11 \cdot \text{in} \cdot 15.5 \cdot \text{in}$$

$$A_{\text{tank.exhaust}} = 170.5 \cdot \text{in}^2$$

$$A_{\text{pipe}} = 1.66 \cdot \text{in} \cdot (11 \cdot 12) \cdot \text{in}$$

$$A_{f.\text{exhaust}} = A_{\text{exhaust}} + A_{T3 \times 3} + A_{\text{tank.exhaust}} + A_{\text{elec.box}} + A_{\text{pipe}}$$

$$A_{f.\text{exhaust}} = 1.529 \cdot 10^3 \cdot \text{in}^2$$

$$A_{\text{exhaust}} = (6.125 \cdot 32.5) \cdot \text{in}^2$$

$$A_{\text{tank.exhaust}} = (11 \cdot 15.5) \cdot \text{in}^2$$

$$L_{\text{exhaust.arm}} = 6.5 \cdot \text{in}$$

$$y_{\text{exhaust}} = 43 \cdot \text{in}$$

$$y_{\text{tank.exhaust}} = 67 \cdot \text{in}$$

$$y_{\text{pipe}} = 37 \cdot \text{in}$$

$$y_{\text{exhaust}} = \frac{2 \cdot A_{TS.\text{vert}} \cdot y_{TS.\text{vert}} + A_{TS.\text{horz}} \cdot y_{TS.\text{horz}} + A_{\text{exhaust}} \cdot y_{\text{exhaust}} + A_{\text{tank.exhaust}} \cdot y_{\text{tank.exhaust}} + A_{\text{pipe}} \cdot y_{\text{pipe}}}{2 \cdot A_{TS.\text{vert}} + A_{TS.\text{horz}} + A_{\text{exhaust}} + A_{\text{tank.exhaust}} + A_{\text{pipe}}}$$

$$y_{\text{exhaust}} = 50.494 \cdot \text{in}$$

Moment due to Deadweight

$$M_{dw.\text{exhaust}} = (W_{\text{exhaust}} + W_{\text{tank.exhaust}} + W_{\text{pipe}} + W_{\text{pump}} + W_{\text{misc.pumps}}) \cdot L_{\text{exhaust.arm}} + (W_{TS.\text{vert}}^3 + W_{TS.\text{horz}})$$

$$M_{dw.\text{exhaust}} = 2.441 \cdot 10^3 \cdot \text{in} \cdot \text{lbf}$$

$$M_{dw.\text{total}} = M_{dw.\text{exhaust}} + W_{\text{slab}} \cdot L_{\text{exhaust.arm}}$$

$$M_{dw.\text{total}} = 2.153 \cdot 10^4 \cdot \text{in} \cdot \text{lbf}$$

Fluor Daniel Northwest

Calc # W-320-24-035

Revision 0

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DESIGN CALCULATION

Client : Project W-320

Subject : Recirculation/Exhaust skid evaluation

Location : 200 East Area

WO/Job No

Date 2/28/97

Checked 3-31-97

Revised

By

N. D. Ha

By

R. A. Giller

By

$$F_{wind} := q_z \cdot G_z \cdot C_{FA} \cdot f_{exhaust}$$

$$F_{wind} = 321.984 \cdot \text{lbf}$$

$$F_{wind,exhaust} = 322 \cdot \text{lbf}$$

Tipping about Steel frame

$$M_{tipping,exhaust} := F_{wind,exhaust} \cdot Y_{exhaust}$$

$$M_{tipping,exhaust} = 1.626 \cdot 10^4 \cdot \text{in} \cdot \text{lbf}$$

Check steel frameSeismic loading

$$W_{total,recir} := [W_{recir} + W_{tank,recir} + W_{pipe} + W_{pump} + W_{misc,pumps} + (W_{TS,vert}^3) + W_{TS,horz}]$$

$$W_{total,recir} = 589.878 \cdot \text{lbf}$$

$$W_{total,exhaust} := [W_{exhaust} + W_{tank,exhaust} + W_{pipe} + W_{pump} + W_{misc,pumps} + (W_{TS,vert}^3) + W_{TS,horz}]$$

$$W_{total,exhaust} = 427.878 \cdot \text{lbf}$$

$$F_{x,seis} := W_{total,recir} \cdot a_{horz,ubc}$$

$$F_{x,seis} = 132.722 \cdot \text{lbf}$$

$$F_{z,seis} = F_{x,seis}$$

$$F_{y,seis} := W_{total,recir} \cdot \frac{2}{3} \cdot a_{horz,ubc}$$

$$F_{y,seis} = 88.482 \cdot \text{lbf}$$

Since the seismic load is much less than wind load, wind loading will be a critical load case for this structure.

Fluor Daniel Northwest

Calc # W-320-24-035

Revision 0

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DESIGN CALCULATION

Client : Project W-320

Subject : Recirculation/Exhaust skid evaluation

Location : 200 East Area

WO/Job No

Date 3/28/97

Checked 3-31-97

Revised

By

N. D. Ha

By

R. A. Giller

By

Deadweight and wind loading

L3x3x1/4

$$F_{\text{wind.recirculator}} = q_z \cdot G_z \cdot C_f \cdot A_{\text{recirculator}}$$

$$F_{\text{wind.recirculator}} = 132.491 \cdot \text{lb} \cdot \text{ft}$$

$$\sigma_{L3x3} = \frac{(W_{\text{recir}} + F_{\text{wind.recirculator}}) \cdot 11.5 \cdot \text{in}}{S_{L3x3}^2}$$

$$\sigma_{L3x3} = 3.543 \cdot 10^3 \cdot \frac{\text{lb} \cdot \text{ft}}{\text{in}^2}$$

Vertical TS3x3

$$\sigma_{T3x3.\text{recir}} = \frac{M_{\text{tipping.recir}}}{2 \cdot S_{T3x3}}$$

$$\sigma_{T3x3.\text{recir}} = 5 \cdot 10^3 \cdot \frac{\text{lb} \cdot \text{ft}}{\text{in}^2}$$

3/8" Plate (7ft x 2.25ft)

$$t_{\text{plate}} = .375 \cdot \text{in}$$

$$b_{\text{plate}} = 7 \cdot \text{ft}$$

$$S_{\text{plate}} = \frac{b_{\text{plate}} \cdot t_{\text{plate}}^2}{6}$$

$$S_{\text{plate}} = 1.969 \cdot \text{in}^3$$

$$P_{\text{frame}} = (W_{\text{recir}} + W_{\text{tank.recir}} + W_{\text{pipe}} + W_{\text{pump}} + W_{\text{misc.pumps}})$$

$$L_{\text{short}} = 2.25 \cdot \text{ft}$$

$$M_{\text{plate}} = (M_{\text{tipping.recir}}) + \frac{P_{\text{frame}} \cdot L_{\text{short}}}{8}$$

$$\sigma_{\text{plate}} = \frac{M_{\text{plate}}}{S_{\text{plate}}}$$

$$\sigma_{\text{plate}} = 1.129 \cdot 10^4 \cdot \frac{\text{lb} \cdot \text{ft}}{\text{in}^2} < F_{y.\text{bending.allow}} ==> \text{OK}$$

Fluor Daniel Northwest

Calc # W-320-24-035

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DESIGN CALCULATION

Client : Project W-320

Subject : Recirculation/Exhaust skid evaluation

Location : 200 East Area

WO/Job No

Date 3/28/97 By N. D. Ha *N.D. Ha*Checked 3-31-97 By R. A. Giller *RA Giller*

Revised By

Lifting bail (5/8" dia)

$$D_{rod} := \frac{5}{8} \text{ in}$$

$$A_{rod} := \pi \frac{D_{rod}^2}{4}$$

$$f_{shear.allow} := .436000 \frac{\text{lbf}}{\text{in}^2}$$

$$w_{steel} := 490 \frac{\text{lbf}}{\text{ft}^3}$$

$$w_{channel} := 5.4 \frac{\text{lbf}}{\text{ft}}$$

$$w_{L1.5 \times 1.5} := 1.8 \frac{\text{lbf}}{\text{ft}}$$

$$W_{plate} = \left(2.25 \cdot \text{ft} \cdot 7.33 \cdot \text{ft} \cdot \frac{3}{8} \cdot \text{in} \right) \cdot w_{steel}$$

$$W_{channel} = (2 \cdot 7.33 \cdot \text{ft} + 2 \cdot 2.25 \cdot \text{ft}) \cdot w_{channel}$$

$$W_{L1.5 \times 1.5} = (2 \cdot 7.33 \cdot \text{ft} + 2 \cdot 2.25 \cdot \text{ft}) \cdot w_{L1.5 \times 1.5}$$

$$P_{lift} = W_{total.recir} + W_{plate} + W_{channel} + W_{L1.5 \times 1.5}$$

$$P_{lift} = 980.371 \cdot \text{lbf}$$

$$\tau_{rod} := \frac{P_{lift}}{4 \cdot A_{rod}}$$

$$\tau_{rod} = 798.878 \frac{\text{lbf}}{\text{in}^2} < f_{shear.allow} \implies \text{OK}$$

Roark, Table 17, case 1
(circular ring)

$$I_{rod} := \frac{\pi \cdot D_{rod}^4}{64}$$

$$\text{Radius hook bend} := 1.75 \cdot \text{in}$$

$$\alpha = \frac{I_{rod}}{A_{rod} \cdot \text{Radius hook bend}^2}$$

$$\alpha = 0.008 \quad k_2 := 1 - \alpha$$

$$M_{hook.bending} := .32 \cdot \text{Radius hook bend} \cdot k_2 \cdot \frac{P_{lift}}{4}$$

$$f_{b.hook} := \frac{M_{hook.bending} \cdot D_{rod}}{I_{rod} \cdot 2}$$

$$f_{b.hook} = 5.681 \cdot 10^3 \frac{\text{lbf}}{\text{in}^2} < F_{bending.allow} \implies \text{OK}$$

Fluor Daniel Northwest

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DESIGN CALCULATION

Client : Project W-320

Subject : Recirculator skid evaluation

Location : 200 East Area

WO/Job No

Date 3/28/97

Checked 3-31-97

Revised

By

N. D. Ha

By

R. A. Giller

By

Weld checks

Between L3x3 and TS3x3

$$d_{\text{weld}} := 3 \cdot \text{in}$$

$$b_{\text{weld}} := 3 \cdot \text{in}$$

$$A_{\text{weld}} := b_{\text{weld}} + d_{\text{weld}}$$

$$S_{\text{weld}} := \frac{d_{\text{weld}}^2 \cdot (4 \cdot b_{\text{weld}} + d_{\text{weld}})}{6 \cdot (2 \cdot b_{\text{weld}} + d_{\text{weld}})}$$

$$S_{\text{weld}} = 2.5 \cdot \text{in}^2$$

$$P_{y,\text{weld}} := \frac{W_{\text{recir}}}{2}$$

$$M_{x,\text{weld}} := W_{\text{recir}} \cdot 11.5 \cdot \text{in}$$

$$f_{\text{weld}} := \sqrt{\left(\frac{P_{y,\text{weld}}}{A_{\text{weld}}}\right)^2 + \left(\frac{M_{x,\text{weld}}}{S_{\text{weld}}} + \frac{F_{\text{wind},\text{recir}}}{2 \cdot A_{\text{weld}}}\right)^2}$$

$$f_{\text{weld}} = 1.062 \cdot 10^3 \cdot \frac{\text{lb}}{\text{in}}$$

$$f_{\text{electrode,allow}} := 70000 \cdot \frac{\text{lb}}{\text{in}^2} \cdot .3$$

$$\text{Weld size} := \frac{f_{\text{weld}}}{.707 \cdot f_{\text{electrode,allow}}}$$

$$\text{Weld size} = 0.072 \cdot \text{in} < 3/16" \Rightarrow \text{OK}$$

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DESIGN CALCULATION

Client : Project W-320

Subject : Recirculator skid evaluation

Location : 200 East Area

WO/Job No

Date 3/28/97

Checked 3-31-97

Revised

By

N. D. Ha

By

R. A. Giller

By

Weld between TS3x3 and 9"x9" plate

$$d_{\text{weld}} := 3 \cdot \text{in}$$

$$b_{\text{weld}} := 3 \cdot \text{in}$$

$$A_{\text{weld}} := (b_{\text{weld}} + d_{\text{weld}}) \cdot 2$$

$$S_{\text{weld}} = b_{\text{weld}} \cdot d_{\text{weld}} + \frac{d_{\text{weld}}^2}{3}$$

$$S_{\text{weld}} = 12 \cdot \text{in}^2$$

$$P_{y,\text{weld}} := \frac{W_{\text{total.recir}}}{3} + \frac{M_{\text{tipping.recir}}}{18 \cdot \text{in}}$$

$$P_{x,\text{weld}} := \frac{F_{\text{wind.recir}}}{3}$$

$$f_{\text{weld}} = \sqrt{\left(\frac{P_{y,\text{weld}}}{A_{\text{weld}}}\right)^2 + \left(\frac{P_{x,\text{weld}}}{A_{\text{weld}}}\right)^2}$$

$$f_{\text{weld}} = 114.234 \cdot \frac{\text{lb}}{\text{in}}$$

$$\text{Weld size} := \frac{f_{\text{weld}}}{.707 \cdot f_{\text{electrode.allow}}}$$

$$\text{Weld size} = 0.008 \cdot \text{in} < 3/16" \implies \text{OK}$$

Weld between platform and 9"x9" plate is adequate by comparison

Fluor Daniel Northwest

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DESIGN CALCULATION

Client : Project W-320

Subject : Recirculator skid evaluation

Location : 200 East Area

WO/Job No

Date 3/28/97

Checked 3-31-97

Revised

By N. D. Ha

By R. A. Giller

By

Concrete slab 4.25ftx9ftx6in

The steel frame skid is supported by the concrete slab, assuming the loads are distributed uniformly on the skid rectangular frame. Therefore, we can use the wall loading on slab method to find the slab thickness.

$$k_{slab} = 1000 \frac{lb}{in^2}$$

Concrete slab thickness

$$t_{slab} = 6 \text{ in}$$

Section modulus of 12" strip concrete

$$SM = \frac{12 \cdot t_{slab}^2}{6}$$

Modulus of subgrade reaction

$$k = 300 \frac{lb}{in^3}$$

Modulus of elasticity of the concrete

$$E_c = 57000 \cdot \sqrt{f_c} \left(\frac{lb}{in^2} \right)^{.5}$$

$$E_c = 3.122 \cdot 10^3 \text{ ksi}$$

Slab stiffness

$$Stiffness = \frac{E_c \cdot t_{slab}^3}{12 \cdot (1 - \nu^2)}$$

Radius of relative stiffness

$$L = \left[\frac{E_c \cdot t_{slab}^3}{12 \cdot (1 - \nu^2) \cdot k} \right]^{\frac{1}{4}} \quad L = 20.923 \text{ in}$$

With concrete compressive strength of 3000 psi, Subgrade modulus of 300 pci

Per table 5 reference 3 the wall loading allowable for 6" thick slab is 1497 lb/ft

 d_b : diameter of rebar

 Hook bend radius is $4d_b$

Concrete slab

Fluor Daniel Northwest

Calc # W-320-24-035

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DESIGN CALCULATION

Client : Project W-320

Subject : Recirculator skid evaluation

Location : 200 East Area

WO/Job No

Date 3/28/97

Checked 3-31-97

Revised

By

By

By

N. D. Ha

R. A. Giller

Dimension of steel frame

$$L_{\text{long}} := 7\text{-ft}$$

$$L_{\text{short}} := 2.25\text{-ft}$$

Load on frame

$$\text{Wall load} := \frac{\frac{M_{\text{tipping,recir}}}{18\text{-in}} + P_{\text{lift}}}{2 \cdot L_{\text{long}} + 2 \cdot L_{\text{short}}}$$

$$\text{Wall load} = 116.054 \frac{\text{lb}}{\text{ft}} < \text{Wall load allowable } 1497 \text{ lb/ft} \Rightarrow \text{OK}$$

Shrinkage and temperature

Friction between slab and subgrade
use 1.5 per AASHTO guide

$$\mu = 1.5$$

Concrete self weight

$$W_{\text{concrete}} := \frac{w_{\text{concrete}} \cdot (1.1 \cdot 5) \cdot \text{ft}^3}{\text{ft}^2}$$

$$W_{\text{concrete}} = 72.5 \frac{\text{lb}}{\text{ft}^2}$$

$$L_{\text{slab, long}} := 7\text{-ft}$$

Tensile force in lb/ft width of slab

$$T := \frac{W_{\text{concrete}} \cdot \mu \cdot L_{\text{slab, long}}}{2}$$

$$T = 380.625 \frac{\text{lb}}{\text{ft}}$$

$$A_s := \frac{T \cdot \text{in}}{f_{y,\text{rebar}}}$$

$$A_s = 5.286 \cdot 10^{-4} \cdot \text{in}^2$$

$$A_{s,\text{min}} := \frac{200 \cdot 12 \cdot \text{in} \cdot 6 \cdot \text{in}}{f_{y,\text{rebar}}}$$

$$A_s = 5.286 \cdot 10^{-4} \cdot \text{in}^2$$

Use #4 at 12" o.c. each way or W4 wire mesh with 4" wire spacing. Places bars/wire mesh near top face of slab, minimum cover is 1 1/2" or at least 3" from the bottom of slab (minimum cover for concrete exposes to earth ACI 7.7)

FINAL:

Provide 4' x 8' x 6" thk slab (soil supported, non-structural)

As (provided) w/ due consideration to design forces > 1.33 As (req'd by analysis)
WWF 6x6 - W 2.9 x W 2.9 is adequate.

Fluor Daniel Northwest

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DESIGN CALCULATION

Client : Project W-320

Subject : Recirculator skid evaluation

Location : 200 East Area

WO/Job No

Date 3/18/97

Checked 3-31-97

Revised

By

N. D. Ha *N.D. Ha*

By

R. A. Giller *R.A. Giller*

By

Bearing pressure

$$L_{\text{slab.short}} := 4.25 \cdot \text{ft}$$

$$L_{\text{slab.long}} := 9 \cdot \text{ft}$$

$$A_{\text{concrete}} := L_{\text{slab.short}} \cdot L_{\text{slab.long}}$$

$$P_{\text{bearing}} := \frac{W_{\text{total.recir}}}{A_{\text{concrete}}} + \frac{M_{\text{tipping.recir}} \cdot \frac{L_{\text{slab.short}}}{2}}{L_{\text{slab.short}} \cdot \text{ft}^3}$$

$$P_{\text{bearing}} = 890.398 \cdot \frac{\text{lb}}{\text{ft}^2} < \text{UBC (Table 18-I-A allowable pressure : 1500 lb/ft}^2 \text{)} \Rightarrow \text{OK}$$

Check anchor bolts (4 of 1/2" anchor bolt)

Because the tipping moment is greater than the deadweight moment, the steel frame need to be anchored to the concrete slab.

$$M_{\text{dw.total.recir}} := M_{\text{dw.recir}} + W_{\text{slab}} \cdot L_{\text{recir.arm}}$$

$$M_{\text{dw.total.recir}} = 2.588 \cdot 10^4 \cdot \text{in} \cdot \text{lb} > M_{\text{tipping.recir}} \Rightarrow \text{Bolts are needed}$$

$$\text{Force on Bolts} \quad F_{\text{bolt.tension}} := \frac{M_{\text{tipping.recir}}}{18 \cdot \text{in}^2}$$

$$F_{\text{bolt.tension}} = 583.317 \cdot \text{lb}$$

$$F_{\text{bolt.shear}} := \frac{F_{\text{wind.recir}}}{4}$$

$$F_{\text{bolt.shear}} = 107.75 \cdot \text{lb}$$

1/2" Carbon Steel Kwik Bolt II allowable working load (3.5" embedded)

$$B_{\text{bolt.tension}} := 2130 \cdot \text{lb}$$

$$B_{\text{bolt.shear}} := 1600 \cdot \text{lb}$$

$$I := \frac{F_{\text{bolt.shear}}}{B_{\text{bolt.tension}}} + \frac{F_{\text{bolt.shear}}}{B_{\text{bolt.shear}}}$$

$$I = 0.118 < 1 \Rightarrow \text{ok}$$

Concrete slab detail

d_b : diameter of rebar

Hook bend radius is $4d_b$

Concrete slab

W320-24-036

Exhaust Skid Platform Modification

Fluor Daniel Northwest	CALCULATION IDENTIFICATION AND INDEX	Page 1 of <i>iii</i>
		Date May 8, 1998

This sheet shows the status and description of the attached Design Analysis sheets.

Discipline **Structural** WO/Job No. **ER4319/W-320** Calculation No. **W320-24-036**

Project No. & Name **Project W-320, Tank 241-C-106 Sluicing**

Calculation Item **Exhaust Skid Platform Modification**

These calculations apply to:

Dwg. No. **H-2-818470** Rev. No. _____

Dwg. No. _____ Rev. No. _____

Other (Study, CDR) _____ Rev. No. **N/A**

The status of these calculations is:

☐ Preliminary Calculations

☒ Final Calculations

☐ Check Calculations (On Calculation Dated)

☐ Void Calculation (Reason Voided)

Incorporated in Final Drawings? ☒ Yes ☐ No

This calculation verified by independent "check" calculations? ☐ Yes ☒ No

Original and Revised Calculation Approvals:

	Rev. 0 Signature/Date	Rev. 1 Signature/Date	Rev. 2 Signature/Date
Originator	Ann S. Langevin <i>5/8/98</i>		
Checked by	<i>5/12/98</i>		
Approved by	<i>5/12/98</i>		
Checked Against Approved Vendor Data	NA		

<u>INDEX</u>	
Design Analysis Page No.	Description
i	Index
1, 2	Objective, Design Inputs, Conclusion
3 - 15	Analysis
Att A	Pl spec pages
Att B	Air Monitor Calc pages
Att C	Material Certs

CALCULATION CROSS INDEX (Technical)

Subject Calculation No. W320-24-036

Page ii of iii[illegible]

DESIGN VERIFICATION SCREENING CRITERIA

HNF-2468, Rev.0

Calculation

Project/Document No. W320-24-036

When the design or design change affects hardware, independent design verification must be performed if one or more of the following questions must be answered affirmatively (YES).

YES

NO

_____✓

1. Does the design or design change involved meet the established criteria to be considered Safety Class?

_____✓

2. Does this design or design change cause or permit changes to Safety Class instrument or alarm setpoints outside of previously approved operational limits?

_____✓

3. Does this design or design change significantly affect the nuclear or environmental safety consequences of a malfunction or failure of the structure, system, or component?

_____✓

4. Does this design or design change involve or change design that has previously undergone formal design verification?

Ann S. Langerin
Assigned Lead Engineer

5/12/98
Date

J. K. Smith
Engineering Manager

5/13/98
Date

Original Design Package Distribution:

Project Manager

Engineering Manager

Technical Document Control

Design Change Distribution:

Attach to Design Change Notice

Fluor Daniel Northwest

Calc. No. W320-24-36

Revision 0

Page No. 1 of 15

DESIGN ANALYSIS

Client Numatec

WO/Job No. W-320

Subject Exhaust Skid Platform Modification

Date 4/14/98

By A.S. Langevin

Checked 5/11/98

By

Location 241-C

Revised

By

OBJECTIVE

Design an extension to the existing exterior platform for accessing the sample probe enclosures at the exhaust skid stack.

The original design provided access for the lower enclosure but did not provide access for the upper enclosure. Changes in the access frequency now necessitate the raising of the platform so that both enclosures are accessible.

DESIGN INPUTSCRITERIA

The platform and the exhaust skid are General Services. Design the platform for the same criteria and referenced codes as given in the exhaust skid procurement spec W-320-P1. (see attachment A):

Snow Load: 20 psf

Wind Loads: per ASCE 7, Section 6, Wind speed = 70mph, I = 1.07, Exp Cat C

$$\begin{aligned}
 q_z &= 0.00256 K_z (IV)^2 \\
 &= 0.00256 (0.8 \text{ from table 6}) (1.07 \times 70)^2 \\
 &= 11.49 \\
 p &= q_z G_h C_f \\
 &= (11.49) (1.32 \text{ from table 8}) (1.33 \text{ from table 12 for square with } h/D = 3) \\
 &= 20 \text{ psf}
 \end{aligned}$$

Seismic:

for the stack and other equipment as per P1 spec:

$$F_p = 0.375 W_p$$

for the platform, a nonbuilding structure, section 1632 of UBC applies:

$$\begin{aligned}
 V &= Z I (C/R_w) W \\
 &= (0.2) (1.25) (0.4) W \\
 &= 0.1 W
 \end{aligned}$$

Live Load (includes snow load): 100 psf, (UBC Table 16-A pedestrian walkways)

Fluor Daniel Northwest

Calc. No. W320-24-36

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DESIGN ANALYSIS

Client Numatec

WO/Job No. W-320

Subject Exhaust Skid Platform Modification

Date 4/14/98

By A.S. Langevin

Checked 5/11/98

By

Location 241-C

Revised

By

REFERENCES

- 1) W-320-P1 Procurement Specification Exhaust Skid Ventilation Air Cleanup Trains
- 2) W-320-P41 Procurement Specification Isokinetic Air Sampling System
- 3) FDNW Practice 134 215 5130 Miscellaneous Typical Steel Ladder and Safety Gate Details
- 4) Air Monitor Corporation Design Calculation Report for an Air Monitor Supplied Instrument & Isokinetic Control Cabinet and Stack Section (dated 12/20/95)

CODES AND STANDARDS

AISC Manual of Steel Construction, 9th Edition

Uniform Building Code 1994

ASCE 7-88, Minimum Design Loads for buildings and other structures

METHODS OF ANALYSIS

Hand Calculations for platform design. Used standard details of FDNW Practice 134 215 5130 for the design of the ladder and handrails.

ASSUMPTIONS/UNCERTAINTIES

The existing stack and support system, together with the exhaust skid platform, are structurally sound and capable of resisting current design loads in accordance with DOE 6430.1A and UBC requirements. Comprehensive structural analysis of the stack and exhaust skid platform, to ascertain their structural integrity, is not part of the scope of work and therefore not addressed in this ECN. Only cursory review and design check of the pertinent elements as required for interface purposes shall be performed.

The existing skid checkered plate receiving the probe enclosure's access platform support legs is assumed to be adequate requiring no additional reinforcement.

CONCLUSIONS

See ECN 647838 and these calculations.

DESIGN ANALYSIS

Client NumatecWO/Job No. W-320Subject Exhaust SkidDate 4/29/98 By W. LangstonPlatform ModChecked 5/11/98 By T. WallLocation 241-C

Revised By

Determine Surface Areas and Wind Loads in Worst Case Direction:

WIND LOAD

$$\text{Railing: } \frac{1.5''}{12} [(4' \times 2) + (3.5' \times 2)] = 1.88$$

$$C5: \left(\frac{5''}{12}\right)(4') = 1.67$$

$$TS \ 3 \times 2: \left(\frac{3''}{12}\right)(4')(4') = 4.00$$

2 legs, horiz L & crosspiece

$$C4: \left(\frac{4''}{12}\right)(3)(6') = 6.00$$

$$\text{Toeboard: } \left(\frac{4''}{12}\right)(4') = 1.33$$

$$\text{Ladder: } \frac{3/8''}{12}(2)(12') + \frac{3/4''}{12}(1.5')(9) = 1.59$$

$$\text{Tubing Bundle } (1')(4') = 4.00$$

$$\text{Total Surface Area} = 20.47 \text{ ft}^2$$

Platform Wind Load

$$20.47 \text{ ft}^2 (20 \text{ psf}) = 410 \text{ lb.}$$

Wind Load

DESIGN ANALYSIS

Client NumatecWO/Job No. W-320Subject Exhaust SkidDate 4/29/98 By AS Langdon

Platform Mod

Checked 5/11/98 By J. P. MillerLocation 241-C

Revised By

Determine Dead Loads and Seismic Loads:

SEISMIC LOAD

$$\text{Ladder} : (3.19 \text{ lb/ft}) (12' \times 2) + (1.5 \text{ lb/ft}) (9' \times 1.5') = 98 \text{ lb}$$

$$\text{Railing} : (2.27 \text{ lb/ft}) [(9' \times 2) + (3.5' \times 6)] = 89 \text{ lb}$$

$$\text{Toeboard} : (3.4 \text{ lb/ft}) (14') = 47 \text{ lb}$$

$$\text{C5} \times 6.7 : (6.7 \text{ lb/ft}) (14') = 94 \text{ lb}$$

$$\text{Grating} : (5.5 \text{ psf}) (3' \times 9') = 166 \text{ lb}$$

(1" x 1/2" beading bars)

$$\text{TS } 3 \times 2 \times \frac{1}{4} : (7.11 \text{ lb/ft}) [(4' \times 4) + (3' \times 4)] = 199 \text{ lb}$$

legs cross pieces

$$\text{C4} \times 5.4 : (5.4 \text{ lb/ft}) (6' \times 8) = 259 \text{ lb}$$

legs & cross pieces

$$\text{L3} \times 3 \times \frac{1}{4} : (4.9 \text{ lb/ft}) (3 \times 4') = 58 \text{ lb}$$

$$\text{Conduit} - \text{assume} = 50 \text{ lb}$$

$$\text{TOTAL DL} = 960 \text{ lb}$$

$$\text{Platform Seismic} = (1.30)(0.1)(960) = 125 \text{ lb}$$

(Orthogonal
affects
see p. A3)

Seismic

FLUOR DANIEL NORTHWEST, INC.

Calc. No. W320-24-036Revision 0Page No. 5 of 15

DESIGN ANALYSIS

Client	<u>Numatec</u>	WO/Job No.	<u>W-320</u>
Subject	<u>Exhaust Skid</u>	Date	<u>4/29/98</u>
	<u>Platform Mod</u>	By	<u>Jeff Angewin</u>
Location	<u>241-C</u>	Checked	<u>5/11/98</u>
		By	<u>J. Wade</u>
		Revised	
		By	

Top Platform - Determine DL & LL

$$DL = \text{Railing} + \frac{\text{Ladder}}{2} + \text{Toeboard} + C5 + \text{Grating}$$

$$= 89 + \frac{98}{2} + 47 + 94 + 66$$

$$DL = 345 \text{ lb} \rightarrow \frac{345}{3' \times 4'} = 28 \text{ psf}$$

$$LL = 100 \text{ psf}$$

Allowable Uniform LL for 1" x 1/8" Grating
= 305 psf (Irving Grating Catalog)

$$DL + LL = 128 \text{ psf}$$

Use 1" x 1/8"
Grating

$$\underline{C5 \times 6.7} : 128 \text{ psf} (1.5') = 200 \text{ lb/ft} = w$$

$$M = \frac{w}{8l^2} (l+a)^2 (l-a)^2 \quad (\text{AISC p. 2-304})$$

$$= \frac{200 \text{ lb/ft}}{8(3')^2} (3+1)^2 (3-1)^2$$

$$= 178 \text{ ft-lb}$$

$$f_b = \frac{M}{S} = \frac{178 (12)}{3.00 \text{ in}^3} = 712 \text{ psi} = 0.02 F_y$$

bending stresses very small $\therefore$ Use C5 x 6.7
to support grating

Reactions at Supports

$$R_1 = \frac{w}{2l} (l^2 - a^2) = \frac{200}{6} (3^2 - 1^2) = 267 \text{ lb} \text{ say } 270 \text{ lb}$$

$$R_2 = \frac{w}{2l} (l+a)^2 = \frac{200}{6} (3+1)^2 = 533 \text{ lb} \text{ say } 540 \text{ lb}$$

FLUOR DANIEL NORTHWEST, INC.

Calc. No. W320-24-036Revision 0Page No. 6 of 15

DESIGN ANALYSIS

Client Numatec
 Subject Exhaust Skid
Platform Mod
 Location 241-C

WO/Job No. W-320
 Date 4/29/98 By J. Langwin
 Checked 5/11/98 By J. Billed
 Revised By

If grating is fully loaded, reactions are both downward, so no uplift.

Check reactions due to concentrated load on cantilevered end. - say 300 lb

$$R_1 = \frac{w}{2l}(l^2 - a^2) - \frac{Pa}{l} \quad (\text{AISC p. 2-304, 2-305})$$

$$= \frac{37}{2(3')}(3'^2 - 1'^2) - \frac{300(1')}{3'}$$

$$= 49 - 100$$

$$= -51 \text{ lb}$$

Use uplift load in design of connection (see page 15)

Determine R_2 due to DL alone

$$R_2 = \frac{w}{2l}(l+a)^2 = \frac{37}{2(3')}(4')^2 = 99 \text{ lb}$$

DESIGN ANALYSIS

Calc. No. W320-24-03

Revision 0

Page No. 7 of 15

Client Numatec
 Subject Exhaust Skid
Platform Mod
 Location 241-C

WO/Job No. W-320
 Date 5/7/98 By A. Langner
 Checked 5/11/98 By Tommy
 Revised By

Seismic - For stack load see B-2
 (conservative - stack length was
 reduced by ECN 368)

Seismic Left to Rt

Moment @ Int flange

$$\begin{aligned} M_1 &= 244 \times 108 - (80 \times 14 + 55 \times 12) \\ &= 26,352 - 1780 \\ &= 24,572 \text{ in-lb} \end{aligned}$$

Moment at exst lateral support

$$\begin{aligned} M_2 &= 24,572 + 244 \times 12 \\ &= 27,500 \text{ in-lb} \end{aligned}$$

Assume stack support at
top of steel plate box

$$R_1 = \frac{244 \times 162}{42} - \frac{1780}{42} = 899 \text{ lb}$$

$$R_2 = 655 \text{ lb}$$

DESIGN ANALYSIS

Calc. No. W320-24-031Revision 0Page No. 8 of 15

Client Numatec
 Subject Exhaust Skid
Platform Mod
 Location 241-C

WO/Job No. W-320
 Date 5/7/98 By A. Langevin
 Checked 5/11/98 By P. Hall
 Revised By

Seismic Right to Left

$$M_1 = 244 \times 108 + 80 \times 14 + 55 \times 12$$

$$= 28132 \text{ in-lb}$$

$$M_2 = 28132 + 244 \times 12$$

$$= 31060 \text{ lb}$$

$$R_1 = \frac{244 \times 162}{42} + \frac{1780}{42} = 983 \text{ lb}$$

$$R_2 = 739 \text{ lb}$$

CHECK STACK DEFLECTION DUE TO SEISMIC

Stack deflection due to 244 lb alone (deflection due to 1780 in-lb moment will be small)

$$\Delta = \frac{P a^2}{3EI} (l+a) = \frac{244 \text{ lb} (120)^2}{3(29 \times 10^6)(10.18)} (162) = 0.642''$$

deflection at top of
 stack - OK

$$I = \pi R^3 t = \pi (3'')^3 (0.12'') = 10.18 \text{ in}^3$$

→ assume 11 ga wall

DESIGN ANALYSIS

Calc. No. W320-24-03

Revision 0

Page No. 9 of 15

Client	<u>Numatec</u>	WO/Job No.	<u>W-320</u>
Subject	<u>Exhaust Skid Platform Mod</u>	Date	<u>5/7/98</u> By <u>W. Langston</u>
Location	<u>241-C</u>	Checked	<u>5/11/98</u> By <u>W. Langston</u>
		Revised	

WIND - For stack load see p. B-1

Wind Left to Right

$$\Sigma F_w = 281.3 \text{ lb}$$

$$\Sigma M = 23,386 \text{ in.-lb}$$

$$\frac{\Sigma M}{\Sigma F_w} = \bar{y} = \frac{23,386}{281.3} = 83 \text{ in. above int. flange}$$

$$M_1 = 281 \times 83 - (80 \times 14 + 55 \times 12) = 23,323 - 1,780 = 21,543 \text{ in.-lb}$$

$$M_2 = 21,543 + 281 \times 12 = 24,915 \text{ in.-lb}$$

$$R_1 = \frac{281 \times 137}{42} - \frac{1,780}{42}$$

$$R_1 = 874 \text{ lb}$$

$$R_2 = 593 \text{ lb}$$

Wind Right to Left

$$M_1 = 23,323 + 1,780 = 25,103 \text{ in.-lb}$$

$$M_2 = 25,103 + 281 \times 12 = 28,475 \text{ in.-lb}$$

$$R_1 = \frac{281 \times 137}{42} + \frac{1,780}{42} = 959 \text{ lb}$$

$$R_2 = 678 \text{ lb}$$

FLUOR DANIEL NORTHWEST, INC.

Calc. No. W320-24-031

Revision 0

Page No. 10 of 15

DESIGN ANALYSIS

Client	<u>Numatec</u>	WO/Job No.	<u>W-320</u>
Subject	<u>Exhaust Skid Platform Mod</u>	Date	<u>5/7/98</u> By <u>Alkanalun</u>
		Checked	<u>5/11/98</u> By <u>Fahed</u>
Location	<u>241-C</u>	Revised	

TOTAL LOADS DUE TO PLATFORM AND STACK REACTION @ R₁

Wind = 410 lb + 959 lb = 1369 lb

Seismic = 125 lb + 983 lb = 1108 lb

∴ Wind Controls, but look at both.

CASE I SEISMIC

Assume all stack seismic taken by one frame that is at edge of skid and closest to stack

NOTE

99 lb = DL alone
540 lb = full DL + LL
125/2 = full platform seismic conserv to apply at top = 63

F = 2338 lb comp.

DESIGN ANALYSIS

Client	Numatec	WO/Job No.	W-320
Subject	Exhaust Skid Platform Mod	Date	5/8/98 By <i>A. Langman</i>
Location	241-C	Checked	5/11/98 By <i>Samuel</i>
		Revised	By

CASE II - WIND

Assume all stack wind load taken by one frame that is at edge of skid and closest to stack.

Apply platform wind load at top of platform. This is conservative, since load would be at mid height so reduce by using factor of 0.6

$$\frac{410 \text{ lb}}{2 \text{ frames}} (0.6) = 125 \text{ lb}$$

FLUOR DANIEL NORTHWEST, INC.

Calc. No. W320-24-03Revision 0Page No. 12 of 15

DESIGN ANALYSIS

Client	Numatec	WO/Job No.	W-320
Subject	Exhaust Skid Platform Mod	Date	4/29/98
Location	241-C	By	A. Hangerman
		Checked	5/11/98
		By	T. G. B. L. L.
		Revised	
		By	

FRAME MEMBERS

Worst case axial load:

2833 lb compression with $l = 8.25'$ Look at C4x5.4
Existing memberThe TS 3x2 is stiffer when comparing the
weakest direction.

Add in Dead Load of lower frame and ladder

$$2833 \text{ lb} + \frac{199 + 259 + 58}{4 \text{ legs}} = 2962 \text{ — Say } 3010 \text{ lb}$$

$$f_a = \frac{3010 \text{ lb}}{1.59 \text{ in}^2} = 1893 \text{ psi} = 0.05 F_y$$

Determine F_a :

$$\frac{b}{t} = \frac{1.584}{0.296} = 5.35 < \frac{65}{\sqrt{F_y}} = 10.8 \therefore \text{Compact Section per AISC Table B5.1}$$

$$\frac{Kl}{r} = \frac{1.0 (8.25' \times 12)}{0.449} = 220 > C_c = 126 \rightarrow \text{Exceeds 200 per AISC B7. Use Eq. E2-2}$$

$$F_a = \frac{12 \pi^2 E}{23 (Kl/r)^2} = \frac{12 \pi^2 (29 \times 10^6)}{23 (220)^2} = 3085 \text{ psi} > f_a = 1893 \text{ psi}$$

Frame Members C4x5.4 & TS 3x2x1/4 OK

DESIGN ANALYSIS

Client Numatec
 Subject Exhaust Skid
Platform Mod.
 Location 241-C

WO/Job No. W-320
 Date 5/7/98 By A. Langerin
 Checked 5/11/98 By Reviewed
 Revised By

FRAME MEMBERS NOT COCENTRIC

LOOK at bending of C4x5.4 combined with axial

$$M_b = 125 \text{ lb}(16) + 983(3)$$

$$= 4949 \text{ in-lb}$$

$$f_b = \frac{M}{S} = \frac{4949}{1.93}$$

$$= 2564 \text{ psi}$$

$$F_b = \frac{12 \times 10^3 C_b}{L_d A_f} = \frac{12 \times 10^3}{(8.25 \times 12)(0.52)}$$

$$= 14.2 \text{ ksi} \quad (\text{Eq. F1-8})$$

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{f_a}{F_e}\right) F_b} \quad (\text{Eq. H1-1})$$

$$= \frac{1893}{3085} + \frac{2564}{\left(1 - \frac{1893}{3085}\right) 14,200}$$

$$= 1.08 \cong 1.0 \quad \text{OK}$$

(Conserv - did not include $\frac{1}{3}$ increase for EQ)

$$\frac{f_a}{0.6 F_y} + \frac{f_b}{F_b} = \frac{1893}{0.6(36,000)} + \frac{2564}{14,200}$$

$$(\text{Eq. H1-2})$$

$$= 0.24 < 1.0 \quad \text{OK}$$

DESIGN ANALYSIS

Client NumatecWO/Job No. W320Subject Exhaust Skid
Platform ModDate 5/7/98 By A. LangermanChecked 5/11/98 By TD/MLLocation 241-C

Revised By

Look at bending in weak axis direction
Loading in this direction similar to other direction

$$M_b = 125(12) - 983(3) = -1449 \text{ in-lb}$$

$$f_b = \frac{M}{S} = \frac{1449}{0.283}$$

$$f_b = 5120 \text{ psi}$$

$$F_b = \frac{12 \times 10^3 C_b}{2d A_f} = \frac{1000}{8.5(8.52)} = 13.8 \quad (F1-3)$$

$$\frac{f_a}{F_a} + \frac{c_m f_b}{\left(1 - \frac{f_a}{F_a}\right) F_b}$$

(Include allowable
1/3 increase in
stress due to
seismic)

$$\frac{1893}{1.33(3085)} + \frac{5120}{\left(1 - \frac{1893}{1.33(3085)}\right)(1.33 \times 1380)}$$

$$= 0.46 + 0.518$$

$$= 0.978 < 1.0 \quad \text{OK}$$

$$\frac{f_a}{0.6 F_y} + \frac{f_b}{F_b} = \frac{1893}{1.33(36,000)(0.6)} + \frac{5120}{133(1380)}$$

$$= 0.34 < 1.0 \quad \text{OK}$$

C4 x 5.4 OK for bending due to eccentricity

DESIGN ANALYSIS

Client NurmatecWO/Job No. W-320Subject Exhaust SkidDate 4/29/98 By g. LangerPlatform ModChecked 5/11/98 By J. WhiteLocation 241-C

Revised By

CONNECTIONSPlatform C5x6.7 to TS 3x2x1/4:

$$\text{Shear (due to wind)} = \frac{125 \text{ lb}}{2 \text{ bolts}} = 63 \text{ lb/bolt}$$

$$\text{Tension (due to uplift)} = \frac{41 \text{ lb}}{2 \text{ bolts}} = 21 \text{ lb/bolt}$$

Loads are very small $\therefore$

Use (2) 1/2" ϕ A307
bolts for connection
of C5x6.7 to TS 3x2

Platform Leg SpliceTS 3x2x1/4 to exst C4x5.4

Shear due to axial load:

$$V_i = \frac{2833 \text{ lb}}{2 \text{ bolts}} = 1417 \text{ lb/bolt} < V_a = 3.1 \text{ K}$$

(AISC Table I-D)

Use (2) 5/8" ϕ A307 bolts
at connection of
TS 3x2x1/4 to C4x5.4

- 2.3.3 Air Movement and Control Association (AMCA)
 99-83 Standards Handbook
 Revised 1986
 210-85 Laboratory Methods of Testing Fans for Rating Purposes
- 2.3.4 American National Standards Institute (ANSI)
 S2.19-1989 Mechanical Vibration - Balance Quality Requirement of Rigid Motors, Part 1, Determination of Residual Unbalance
- 2.3.5 American Society of Civil Engineers Standards (ASCE) ←
 ASCE 7-88 Minimum Design Loads For Buildings and Other Structures
- 2.3.6 American Society of Mechanical Engineers (ASME)
 AG-1-1991 Code on Nuclear Air and Gas Treatment
 B16.5-1988 Pipe Flanges and Flanged Fittings
 N509-1989 Nuclear Power Plant Air-Cleaning Units and Components
 N510-1989 Testing of Nuclear Air-Cleaning Systems
 NQA-1-1994 Quality Assurance Program Requirements for Nuclear Facilities
- 2.3.7 American Society for Nondestructive Testing (ASNT)
 SNT-TC-1A Recommended Practice (Dec 1988 Edition)
- 2.3.8 American Society of Testing and Materials (ASTM)
 A 36-91 Structural Steel
 A 193-91a Alloy-Steel and Stainless Steel Bolting Materials for High Temperature Service
 A 194-91 Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service
 A 240-91a Heat-Resisting Chromium and Chromium-Nickel Stainless Plate, Sheet and Strip for Pressure Vessels
 A 325-92a High-Strength Bolts for Structural Steel Joints
 A 563-92a Carbon and Alloy Steel Nuts

- A 786-89 Rolled Steel Floor Plates
F 436-92 Hardened Steel Washers
- 2.3.9 American Welding Society (AWS)
D1.1-92 Structural Welding Code-Steel
D9.1-90 Sheet Metal Welding Code
- 2.3.10 Federal Specifications (FS)
RR-G-661 Grating, Metal, Bar Type (Floor, Except For Naval Vessels)
- 2.3.11 International Conference of Building Officials (ICBO)
UBC Uniform Building Code (1994) ←
- 2.3.12 Institute of Electrical and Electronics Engineers (IEEE)
519-1992 Recommended Practices and Requirements for Harmonic Control in Electrical Power Systems
- 2.3.13 Instrument Society of America (ISA)
S5.1-1984 (1992) Instrumentation Symbols and Identification
- 2.3.14 National Electrical Manufacturers Association (NEMA)
250-1991 Enclosures for Electrical Equipment
ICS 6-1993 Enclosures for Industrial Controls and Systems
MG 1-1993 Motors and Generators
- 2.3.15 National Fire Protection Association (NFPA)
70 (1993) National Electrical Code (NEC)
- 2.3.16 Underwriters Laboratories (UL)
Electrical Appliance and Utilization Equipment 1994
Directory
- 2.3.17 Washington Administrative Code (WAC)
Title 29 Department of Labor and Industries
Chapter 296-24 General Safety and Health Standards

- 3.2 Environmental Conditions: The Air Cleanup Train will be located outside at an elevation of 700 feet above sea level. Equipment must operate without failure at ambient conditions of -20 to 115°F, and 0 to 100% relative humidity, in rain, sleet, snow, extreme blowing dust, and intense solar radiation.
- 3.3 Structural Criteria
- 3.3.1 Structural steel supports and components shall be fabricated in accordance with the AISC ASD, S329 and S335, and the ICBO UBC.
- 3.3.1.1 The skid shall have a heavy duty, structural steel base that provides structural integrity for the entire ACT system assembly. The arrangement of longitudinal and cross beams shall provide the required strength and stiffness for the base, and support for equipment installed.
- 3.3.1.2 Deflection of the entire base shall be minimized so as not to cause damage to container elements or equipment installed. Base construction shall take into account loads imposed on the structure during seismic, lifting, shipping, etc. Permanent deformations to the base shall not impair performance of the structure.
- 3.3.1.3 Lifting attachments for the skid shall be engineered with a safety factor of 5. The manufacturers' name or identification mark shall be forged in raised letters on the lifting attachment. Include any special instructions needed for a safe lift along with manufacturers' data on lifting attachment.
- 3.3.1.4 The base shall be constructed and detailed for attachment to a concrete pad. Anchorage locations shall be provided.
- 3.3.1.5 The base and stack support shall be fabricated to handle static and dynamic loads imposed by the Buyer provided 14-foot long upper stack and rain cap assembly. The Buyer provided upper stack section shall be subjected to the loading stated in 3.3.2 and shall transmit those loads to the Seller's lower stack section. Thus, the lower stack section shall be fabricated to support the loads imposed by the upper section. See Sketch ES-1.
- 3.3.2 Structural Loads: Loads shall include live, dead, wind, snow, sleet, and seismic loads, including normal, operating loads. Other loads, either static or dynamic shall be considered auxiliary loads. Equipment function shall be maintained during load conditions.
- 3.3.2.1 Wind loads: Loads shall be in accordance with ASCE 7, Section 6, with wind assumed to come from any horizontal direction. Fabrication of the structure and components shall be based on the following:
- Basic wind speed: 70 mi/h.
 - Importance factor: $I = 1.07$.
 - Exposure category: C.

- 3.3.2.2 Snow Loads: In accordance with ASCE 7, Section 7. Use 20 lb/ft².
- 3.3.2.3 Seismic loads: Seismic loads shall be in accordance with the ICBO UBC, Sections 2336 and 2337 (Zone 2B), where $F_p = 0.375 W_p$.
- a. The lateral force, F_p shall be distributed in proportion to the mass distribution of the element or component. The distributed forces shall be applied so that the most critical loadings result. These loadings shall be used in the fabrication of the elements or components and anchorages.
- b. Orthogonal effects, as described in the ICBO UBC, shall be satisfied by designing such elements for 100% of the seismic forces in one horizontal direction plus 30% of the seismic forces in the perpendicular horizontal direction.
- 3.4 Material
- 3.4.1 Materials shall be new, and conform to a national standard such as ASTM or ASME. Pressure boundary material shall be in accordance with ASME N509, Sections 5.6 and 5.10, as applicable.
- 3.4.1.1 Structural steel shapes, plate, and bars: ASTM A 36.
- 3.4.1.2 Stainless steel sheet: ASTM A 240.
- 3.4.1.3 Floor plate for skid base: ASTM A 786, using ASTM A 36 material. Diamond Tread.
- 3.4.1.4 Bolts: ASTM A 325, Type 1 or 2, plain (noncoated), or ASTM A 193, as applicable.
- 3.4.1.5 Nuts: ASTM A 563, Grade C, or ASTM A 194, Grade 8F, plain, heavy hex, as applicable.
- 3.4.1.6 Washers: ASTM F 436, circular.
- 3.4.1.7 Metal Grating: FS RR-G-661, Type I, Class 1, Material S, hot-dip galvanized. Grating shall be plain surface type with end banding bars. Provide manufacturer's standard clips for attachment to framing.
- 3.4.2 Pressure boundary materials in contact with the air stream shall be 300 series stainless steel unless noted otherwise in this Specification.
- 3.4.3 Tape and adhesives leachable chloride content shall not exceed 15 ppm, and leachable fluoride content 10 ppm; 3M Company preservation sealing tape No. 481, 9 mil thick, or Kendall Company Polyken Division No. 226. In addition, materials which contact stainless steel during fabrication and shipping shall not exceed these limits.
- 3.5 Fabrication and Assembly
- 3.5.1 Base Construction: The base shall be constructed from fabricated hot or cold rolled or cold formed steel shapes, welded on suitable jigs or alignment beds. Welding shall be in accordance with AWS D1.1.

DESIGN CALCULATION WORKSHEET

Project: WHC W320-P41

W.O. No: 25255

Contract No: P.O. 80166

3.2 CONT. CALCULATE WIND LOADS (STACK SECTION)

WIND LOAD IN "X" DIRECTION

ITEM	G _w S _f (lb/ft ²)	C _F	A _P (FT ²)	F _w (lbF)	Y (IN)	M _w (IN-lb)
TOP ENCLOSURE	17.3	1.2	1.67	34.7	112"	3,886
LOWER ENCLOSURE	17.3	1.2	1.39	28.6	80"	2,288
6" STACK W/ 1" INSULATION	17.3	1.2	5.56	115.4	72"	8,309

 $\Sigma = 8.6$
 $\Sigma = 178.7$
 $\Sigma = 14,483$

WIND LOAD IN "Z" DIRECTION

ITEM	G _w S _f (lb/ft ²)	C _F	A _P (FT ²)	F _w (lbF)	Y (IN)	M _w (IN-lb)
TOP ENCLOSURE	17.3	1.2	3.33	69.1	112"	7,739
LOWER ENCLOSURE	17.3	1.2	2.22	46.1	80"	3,688
6" STACK W/ 1" INSULATION	17.3	1.2	8.0	160.1	72"	11,959

 $\Sigma = 13.6$
 $\Sigma = 281.3$
 $\Sigma = 23,386$ TOTAL MOMENT AT STACK BASE

FIND VERTICAL CENTROID OF WIND FORCE ACTION

$$Y_{TOTAL} = \frac{\Sigma A_i Y_i}{\Sigma A_i} = \frac{(1.67)(112) + (1.39)(80) + (5.56)(72)}{8.6} = Y_c = 83 \text{ IN ABOVE FLANGE}$$

$$Y_{TOTAL} = \frac{\Sigma A_i Y_i}{\Sigma A_i} = \frac{(3.33)(112) + (2.22)(80) + (8.0)(72)}{13.6} = Y_c = 83 \text{ IN ABOVE FLANGE}$$

THEREFORE USE $Y = 83"$

pared By: Scott Minich

Title: M.E.H. ENGR.

Date: 11-7-95

Verified By: Parash Dave

Title: MECH. APPL. ENGR.

Date: 11-18-95

DESIGN CALCULATION WORKSHEET

Project: WHC W320-P41

W.O. No: 25255

Contract No: P.O. 80166

S.O SEISMIC FORCES ON STACK COMPONENTS

ASSEMBLED STACK WEIGHT $\approx$ 500 lb = W_p

$F_p = 0.375 W_p$ (FROM SPEC. SECTION 3.3.1.4)

$F_p = 0.375(500 \text{ lb}) \Rightarrow F_p = 188 \text{ lb}$

SEISMIC FORCE $F_s = 188 \text{ lb} + 0.3(188 \text{ lb}) \Rightarrow F_s = 244 \text{ lb}$ ←

TOTAL FORCE $F_T = \text{SEISMIC} + \text{DEADWEIGHT} \Rightarrow F_T = 500 \text{ lb} + 244 \text{ lb}$

$\Rightarrow F_T = 744 \text{ lbs}$

THE ENTIRE LOAD WILL BE DISTRIBUTED OVER THE
~~6" 15076 FLANGES~~ 8-3/4" ASTM A-325 HEX HEAD HIGH
STRENGTH CAP SCREWS. SHEAR + TENSILE STRESSES
WILL BE CALCULATED BASED ON THE APPLIED
LOAD PER BOLT. $F_T = 744 \text{ lb} / 8 \text{ bolts} \Rightarrow F = 93 \text{ lb/bolt}$

S.1 CALCULATE SHEAR PER BOLT

SHEAR AREA (A_s) = $\frac{\pi (0.75)^2}{4} = \frac{\pi (75)^2}{4} \Rightarrow A_s = .4418 \text{ in}^2$

YIELD STRENGTH OF BOLT = 92 Kpsi (REF 1 PG 8-30)

SAFETY FACTOR = 3 (FROM SPEC SEC 3.3.1.1)

ALLOWABLE LOAD = $\frac{92 \text{ Kpsi}}{3} \Rightarrow T_a = 30.6 \text{ Kpsi}$

ALLOWABLE SHEAR = $0.5 T_a \Rightarrow T_u = 15.3 \text{ Kpsi}$

$\tau = \frac{F}{A_s} \Rightarrow \frac{93 \text{ lb}}{.4418 \text{ in}^2} \Rightarrow \tau = 210 \text{ lb/in}^2 \ll 15.3 \text{ Kpsi} \Rightarrow \text{S.F.} = 73$

Prepared By: Scott Munsie

Title: MECH ENG'G

Date: 12-20-95

Verified By: Parish Dave

Title: Mgt. Appl. Engrg

Date: 12-20-95

DESIGN CALCULATION WORKSHEET

Project: WMC W320-P41
Contract No: P.O. 80166

W.O. No: 25255

3.2 CALCULATE WIND LOAD (STACK SECTION)

NOTES: DRAWING NOT TO SCALE.
1" INSULATION ASSUMED ON STACK

SECTION C-C

PROJECTED AREAS:

$$A_{P_x} = 8''(20'' + 10'' + 70'') + (24'' \times 10'') + (20'' \times 10'') \Rightarrow 1240 \text{ IN}^2 = 8.6 \text{ FT}^2$$

$$A_{P_z} = 8''(144'') + (24'' \times 20'') + (20'' \times 16'') \Rightarrow 1952 \text{ IN}^2 = 13.6 \text{ FT}^2$$

C_s TO BE CALCULATED ON A COMPONENT BASIS

FOR TOP ENCLOSURE $V_1 = \frac{\text{HEIGHT}}{\text{WIDTH}} = \frac{24''}{10''} = 2.4 < 3 \Rightarrow C_{F1} = 1.2$

$$V_2 = \frac{24''}{20''} = 1.2 < 3 \Rightarrow C_{F2} = 1.2$$

Prepared By: Scott Munster
Verified By: Patrick Dave

Title: MECH. ENGR
Title: M&R. APPL. ENGR

Date: 11/7/95

Date: 11-16-95

PO BOX 321168
BIRMINGHAM, AL 35232-1168
(205) 592-8981

CERTIFIED TEST REPORT

The following tests conform to the requirements of the specifications listed.

Rect. 172384
Line # 100
Job #0701

WE HEREBY CERTIFY THAT THE FOLLOWING DATA IS A TRUE COPY FROM TESTS PERFORMED IN OUR LABORATORY.

Trish Lauer
TRISH LAUER
QUALITY CONTROL MANAGER

1 / 4

SHIP NO S97611
BOL NO B46915
CUST PO P51116TL008
INV NO V126768
INV DATE 01/03/96

S 297750
O RIVERFRONT STEEL
L P.O. BOX 141494
D CINCINNATI,
T USA

OH
452500149

S RIVR
H RIVERFRONT STEEL
I 884 MEHRING WAY
P CINCINNATI
T USA

OH
45203

HEAT NO.	SECTION	SPECIFICATION	T #	YIELD PSI	TENSILE PSI	ELONG % IN	RA %	BEND TST DIAM	RSL	DATE ROLLED	U
952739	L 3X3X1/4	20	ASTM A36-93a	1 51500	72500	33.0 8				061195	
953741	C 4X5.4	20	ASTM A36-93a	2 51000	72000	33.0 8				090395	
953742	C 4X5.4	20	ASTM A36-93a	1 49700	70000	27.0 8				090395	
956458	W 8X13.0	45	ASTM A36-93/A572-93-50	2 49500	71000	28.0 8				010296	
956468	W 8X13.0	50	ASTM A36-93/A572-93-50	1 50500	70500	30.0 8				010296	
956472	W 8X13.0	50	ASTM A36-93/A572-93-50	2 50500	70500	28.0 8				010296	
				1 53000	72500	28.0 8					
				2 52500	72500	28.0 8					
				1 56000	75500	28.0 8					
				2 56000	76000	28.0 8					
				1 57000	76500	26.0 8					
				2 55500	75500	26.0 8					

HEAT NO.	C	MN	P	S	SI	CU	CR	NI	MO	CB	V	AL	CE	SN	BHN	CHARPY V-NOTCH IMPACT RESULTS
952739	.14	0.74	.012	.039	.20	.39	0.07	0.15	.041	.000	.0000	.000	.32	.016		SIZE (MM) TEMP. FT.-LBS. FT.-LBS. FT.-LBS.
953741	.13	0.69	.008	.036	.21	.45	0.09	0.18	.053	.001	.0000	.000	.31	.013		
953742	.13	0.62	.007	.022	.20	.48	0.08	0.15	.046	.000	.0000	.000	.30	.014		
956458	.15	0.81	.007	.026	.15	.21	0.04	0.09	.020	.000	.0200	.001	.32			
956468	.16	0.86	.008	.032	.23	.35	0.06	0.12	.040	.001	.0190	.002	.35			
956472	.15	0.81	.009	.019	.22	.31	0.08	0.12	.040	.000	.0200	.002	.34			

Approved E & WQA

Name date

REMARKS: 100% MELTED AND MANUFACTURED IN THE USA AND FREE FROM MERCURY CONTAMINATION IN THE PROCESS

W320-24-037

Elec. Equip. Rack Check Calculations

CALCULATION SHEET

Revision 0

Page No. 1 of 8

Client	Numatec Hanford Corporation	Contract/Job No.	W-320-98-005 (NHC-9853435)
Subject	W-320 Tank 241-C-106 Shuicing	Originated By	L.A. VALLE <i>LA</i>
	Elec. Equip. Rack Check Calculations	Checked By	W.J. MAGRUDER <i>WJM</i>
Location	241-AY/200E	Revised By	

Date 6/23/98
Date 6/23/98

Executive Summary: The Tank Waste Remediation System (TWRS) Initial Tank Retrieval System (ITRS) will provide feed for processing plants, dispose of processed waste, consolidate solid tank waste, and provide for mixing of waste types. This process will backup space available and support the close down of other facilities and remediation of the Hanford Site.

Objective: 1) Provide check calculations to verify the structural adequacy of the existing AY-102 Electrical Equipment Rack, FIT-0622 and its foundation.

2) Provide design and sketches to reinforce the rack and/or its foundation, as required to mount the new electrical equipment FIT-0622.

Design Criteria:

- 1) DOE Order 6430.1A and UCRL-15910, Design and Evaluation Guidelines for the Design of DOE Facilities
- 2) Uniform Building Code (ICBO) 1994
- 3) ASCE 7-95 Minimum Design Loads for Buildings and other Structures
- 4) Manual of Steel Construction (ASC) 9th Edition
- 5) GC-LOAD-01; Rev. 1; Design Loads for Facilities
- 6) Safety Class: The support system subject of these calculations is classified as Safety Class 3.

References:

- 1) EDT 604381 "Structural Evaluation of Electrical Equipment Rack of the Annulus Ventilation System"; dated 1/18/96
- 2) ECN No: 647828 showing additional elec. equip. to be supported by the exist. rack AY-102, FIT 0622
- 3) UNISTRUT and Hilti Kwik Bolt II catalog information

Conclusions:

- 1) Check calculations indicate that the existing electrical equipment rack system is adequate to support the electrical equipment required to support the replacement of Tank AY-102 Annulus Air Flow monitoring instruments.
- 2) Analysis also indicates that one of the existing footings requires modification to satisfy Natural phenomena hazards code requirements. See pages 19 & 20 of ECN 647828 for required modifications.

CALCULATION SHEET

Revision 0

Page No. 2 of 5

Client	Numatec Hanford Corporation	Contract/Job No.	W-320-98-005 (NHC-9853435)
Subject	W-320, Tanks: 241-C-106 Sluicing	Originated By	L.A. VALLE <i>LA</i>
	Elec. Equip. Rack FIT-0622 - Check Calcs	Checked By	WJ M <i>WJ</i>
Location	241-AY/200E	Revised By	
		Date	6/23/98
		Date	6/23/98

ELEC EQUIP SUPPORT PAD

PLAN

1" = 1' = 0"

* FIELD FURNISHED DIMENSIONS

Client Numatec Hanford Corporation

Contract/Job No.

W-320-98-005 (NHC-9853435)

Subject W-320, Tank, 241-C-106 Shuicing

Originated By

L.A. VALLE

Date 6/23/98

Elec. Equip. Rack FIT-0622 - Check Calcs.

Checked By

WJ

Date 6/23/99

Location 241-A/Y/200E

Revised By

Date

SECTION

1" = 1' = 0"

Client NUMATEC HANFORD OFF Contract/Job No. 65100501 EP-511/H...
 Subject VI-320, TANK 241-C-106 Originated By LAVALLE Date 6/22/96
SLUICING - ELEC. EQUIP. RACK Checked By WJ Mign Date 6/23/98
 Location FIT-0622 - CHECK CALCS. Revised By Date

ASSUME FULL WIND ON FRAME
SETTING FOR WIND CONTROL

$$P = C_e C_d I$$

$$C_e = 1.31$$

$$C_d = 1.3$$

$$I = 1.05$$

$$(60 \text{ LBS} \cdot \text{FT}^{-2})$$

$$(90 \cdot 3)$$

$$P = 1.11 \cdot 1.3 \cdot 1.05$$

$$= 1.51 \text{ LBS} \cdot \text{FT}^{-2}$$

$$V_w = (1.7 \times 53.3) \cdot 20.7$$

$$= 1.36 \text{ LBS} \cdot \text{FT}^{-2}$$

$$L_{AT} \approx 5.5' + 1.1'$$

$$\text{LATERAL} = 5.5' \text{ FROM BASE}$$

$$M_y = P_y = 345 \cdot 6' = 2070 \text{ LBS} \cdot \text{FT}$$

$$V_y = 436 \times 5.5' = 2398 \text{ LBS}$$

$$\text{MAX TENSION IN TRACE} = (1526 + 275) \cdot \frac{1}{1.1}$$

$$= 1750 \text{ LBS}$$

$$F_{BRACE} = 1526 \cdot \frac{1}{1.1} = 1387 \text{ LBS (TENS)}$$

$$\text{MAX COMPRESSIVE EQUIP. SUPPORT} = 348 \text{ LBS} + 755 \text{ LBS}$$

$$= 1103 \text{ LBS}$$

WIND FROM RIGHT TO LEFT

$$R = (1526 + 275) \cdot \frac{1}{1.1} = 1750 \text{ LBS}$$

$$F_{BRACE} = 1750 \cdot \frac{1}{\cos 15^\circ} = 1825 \text{ LBS (COMP)}$$

$$R = 1750 \cdot \frac{1}{\cos 15^\circ} = 1825 \text{ LBS}$$

$$F_{BRACE} = 1825 \cdot \frac{1}{\cos 15^\circ} = 1900 \text{ LBS}$$

FIGURE 1 - SIDE VIEW

FIGURE 2 - FRONT VIEW

FIG. 1 ELECTRICAL EQUIPMENT RACK (REF.)

$$\Sigma W = 265 \text{ LBS (EXT EQUIP)}$$

$$80 \text{ LBS} + (\text{NEW PANEL})$$

$$345 \text{ LBS} = P$$

CALCULATION SHEET

Calc. No. VI-320-24-2

Revision 0

Page No. 5 of 8

Client	HUMATEC HANFORD CORP	Contract/Job No.	VI-320-24-2 (VI-320)
Subject	W-320, TANK 241-C-106	Originated By	LAVALLE
	SLUICING - ELEC. EQUIP RACK	Checked By	WJ Magn
Location	FIT-0622 - CHECK CALC	Revised By	

$$R_1 = [436(3.5 - 3.24) + 255] / 3.24 \times 2$$

$$= 52 \#$$

$$R_2 = 112 \#$$

$$= 27 \#$$

$$MOMENT = 436(3.5 - 3.24) + 255$$

$$= 112 \# \times 3.24 = 345.4 \# = 414 \#$$

$$P1000 \text{ UNISTRUT CAP } P_{ALLOW} = 3590 \# (3.25') - \text{UNISTRUT CATALOG \#12}$$

$$M_{ALLOW} = 5050 \#$$

COME.

$$\frac{7100}{3510} + \frac{414}{5080} = 0.58 + 0.516 = 1.2 < 1.33 \text{ OK}$$

CHECK EX. ANCHORS (1/2" ϕ R.D. II), $F_v = 1640 \#$, $F_t = 1230 \#$

$$\frac{P_v}{2} = \frac{313}{2} = 157 \#, \quad \frac{P_t}{2} = 75 \#$$

COMB.: $\frac{104}{1840} + \frac{75}{1230} = 0.06 + 0.612$
 $\frac{104}{1840} = 0.062 < 1.0 \text{ OK}$

UNISTRUT P1000 EX. CAP = $\frac{3520}{2.5 \times 5.5} = 1400 \# > 1120 \#, \text{ OK}$
 (SEE APP. X B)

ANGULAR FITTING W/ 1/2" NUT (P1000) ALLOWABLE TENSILE ST = $2800 \#$
 (SEE APP. X E) $> 780 \# \text{ OK}$

CALCULATION SHEET

Calc. No. VI

Revision

Page No. 6 of 6

Client NUMATEC HANFORD CORP

Contract/Job No. 65100501 (ER4219/W-220)

Subject W-220 TANK 241-C-106

Originated By LA TALLE Date 2/22/11

SLUICING - ELEC. EQUIP RACK

Checked By WJ Mlyn Date 6/23/98

Location FIT-02 - CHECK CALC

Revised By

Date

CHECK EXT. EQUIP. RACK

$$W_{COND} = 1.57 \times 52 \times 0.150$$

$$= 12.5 \text{ K}$$

32"

PLAN

CHECK SAFETY FACTOR AGAINST

O.T.

$$M_1 = \frac{0.18933 \times 1100 \times 9 \times 0.55}{2 \times 11} + \frac{1100 \times 9 \times 0.55}{12} \times 218 \#$$

$$= 0.24 + 0.722 = 0.94 \text{ K}$$

14"

P₁₁ x = 753 #P₁₁ y = 1100 #

$$M_2 = 0.145 \times 1100 \times 9 \times 0.55 + 0.722 \times 1.1 \times 11$$

$$= 0.146 + 1.442 = 1.588$$

W = 150 #

23" (1.917)

$$S.F. = \frac{0.940}{1.59} = 0.591 < 1.5 \text{ (NOT GOOD)}$$

NEED TO REVISE FIT TO SATISFY SAFETY FACTOR AGAINST O.T.

SEE P. 1

CALCULATION SHEET

Calc. No. W-320-24-03

Revision 0

Page No. 7 of

Client NUMATEC HANCOCK CO.

Contract/Job No. 65100501

Subject VIBRO-TITE - C-106

Originated By LAVALLE Date 6/22/95

SLIDING-BLE FEED PUCK

Checked By WJ Meyer Date 6/23/98

Location FIT.

Revised By Date

Date _____

Page M-8

Ref Dwg	Sh	Rev	Prepared By	Checked By	ECN No	Page
H-2-92970	1	3	J. Valle	W. M. M.	647828	19

ELEC EQUIP SUPPORT PAD

PLAN

1" = 1' = 0"

*FIELD FURNISHED DIMENSIONS

Ref Dwg H-2-92970	Sh 1	Rev 3	Prepared By J. Valle	Checked By WJ Meyer	ECN No 647828	Page 20
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A SECTION

P. 19

1" = 1" = 0"

FLUOR DANIEL NORTHWEST

CALCULATION SHEET

Calc. No. _____
Revision _____
Page No. _____ of _____

Client	Contract/Job No.	
Subject	Originated By	Date
	Checked By	Date
Location	Revised By	Date

ELEC EQUIP SUPPORT PAD

PLAN

FIELD H1
Ref

JAN 18 1996

ENGINEERING DATA TRANSMITTAL

HNF-2468, Rev. 0

Page 1 of 14
1. EDT 604381

2. To: (Receiving Organization) Controlled Clean and Stable Engineering		3. From: (Originating Organization) TPSECS/Analytical Services (OHJ48)		4. Related EDT No.: N/A	
5. Proj./Prog./Dept./Div.: Tank Waste Remediation System		6. Cog. Engr.: M. M. Jennings		7. Purchase Order No.: N/A	
8. Originator Remarks: This correspondence contains a structural evaluations of the Electrical Equipment Rack of the Annulus Ventilation System AY-101 and AY-102 under seismic and wind load conditions.				9. Equip./Component No.: N/A	
				10. System/Bldg./Facility: AY-101 and AY-102.	
11. Receiver Remarks: <i>EDT file (GRS. 23.5a)</i>				12. Major Assm. Dwg. No.: N/A	
				13. Permit/Permit Application No.: N/A	
				14. Required Response Date: N/A	

15. DATA TRANSMITTED				(F)	(G)	(H)	(I)
(A) Item No.	(B) Document/Drawing No.	(C) Sheet No.	(D) Rev. No.	(E) Title or Description of Data Transmitted	Impact Level	Reason for Transmittal	Originator Disposition
1	N/A	<i>MR 1/3/96</i> <i>14</i>	N/A	STRUCTURAL EVALUATION OF ELECTRICAL EQUIPMENT RACK OF THE ANNULUS VENTILATION SYSTEM	N/A	2	1

16. KEY			
Impact Level (F)	Reason for Transmittal (G)		Disposition (H) & (I)
1, 2, 3, or 4 (see MRP 5.43)	1. Approval 2. Release 3. Information	4. Review 5. Post-Review 6. Dist. (Receipt Acknow. Required)	1. Approved 2. Approved w/comment 3. Disapproved w/comment 4. Reviewed no/comment 5. Reviewed w/comment 6. Receipt acknowledged

(G)	(H)	17. SIGNATURE/DISTRIBUTION (See Impact Level for required signatures)								(G)	(H)
Reason	Disp.	(J) Name	(K) Signature	(L) Date	(M) MSIN	(J) Name	(K) Signature	(L) Date	(M) MSIN	Reason	Disp.
1	1	Cog. Eng. W. W. Chen		1/3/96	H5-57					1	
1	1	Cog. Mgr. F. C. Han		1/3/96	H5-72						
2	1	M. M. Jennings		1/3/96	R2-88						
2	1	S. R. Pierce		1/3/96	S5-05						
2	1	G.N. HANSON		1/3/96	S5-05						
2	1	J. WELLS		1/3/96	S5-05						

18. W. W. Chen Signature of EDT Originator		19. M. H. Jennings Authorized Representative for Receiving Organization		20. <i>[Signature]</i> Cognizant Project Engineer's Manager		21. DOE APPROVAL (if required) Ltr. No. () Approved () Approved w/comments () Disapproved w/comments	
<i>[Signature]</i> 1/3/96		<i>[Signature]</i> 1/3/96		<i>[Signature]</i> 1/3/96			

BD-7400-172-2 (07/91) GEF097

EDT 604381
ITEM #1
13 TOTAL PAGES

Structural Evaluation of Annulus Ventilation System
Electrical Equipment Rack

OBJECTIVE

The objective of this analysis is to perform a structural evaluations of the electrical equipment rack of the annulus ventilation system (AY-101 and AY-102) under the seismic and wind load conditions. All the attached components and the system evaluated meet the requirements for dead load and natural phenomena hazard loads (seismic and wind) and are structurally qualified.

This report contains all the calculations to qualify the structure under hazard loads.

INTRODUCTION

The electrical equipment rack system is free-standing and suitable for concrete pad mounting as shown in Figure 1 (Ref 1). The rack is assembled using P1000 unistrut and other assorted hardware as described in Figure 2 (Ref. 1). The component and total system weight of the rack is tabulated in Table 1 (Ref 2). The unistrut channel and combinations are roll formed from 12 gauge (0.105") strip steel.

Under non-reactor Safety Class 3 category, the rack was designated as low hazard/important system or component as described in Ref. 3. This low hazard (safety Class 3) structure or facilities may be evaluated by elastic dynamic analysis. For simple structures with non-uniform mass distributions, the equivalent-static load analysis may be used.

Earthquake input excitation to the elastic dynamic analysis is given site-specific median response spectra anchored to a maximum ground acceleration (Ref. 3). Reference 5 stated that 0.12g site-specific safe shutdown earthquake (SSE) has been maintained for the non-reactor structures. This load may be amplified by multiplying the structure, equipment, or component masses by an acceleration equal to 1.5 times the peak acceleration of the application response spectrum (Ref. 4).

The basic wind speed is 70 mph as determined from regional climate data (Ref. 10). Since the structure height is only slightly over 6', the factors used for velocity pressure calculations are highly conservative.

The structural components connected to the unistrut structure are qualified based on the design data specified in Reference 8. Anchor bolts embedded in the concrete pad are qualified based on the HILTI system technical guide (Ref. 9) and design guide SDC- 4.2 (Ref. 6).

CONCLUSION

A seismic and wind load analyses of the electrical equipment rack system have been completed. The analyses were based on the Standard Design Criteria (Ref. 5 and 6) and ASCE standard reference (Ref. 4 and 10). The Hilti Kwik-Bolt II (Ref. 9) and Unistrut catalogue (Ref. 8) were used to qualify the structural components.

The results indicate that the electrical equipment rack system are qualified to resist the earthquake and wind load requirement. The edge distance and embedment depth for the expansion anchor and the anchor spacing meet the design criteria. Mounting equipments on the rack meet the structural design requirements.

REFERENCES

1. WHC-S-0443, 1995, "Specification for Annulus Ventilation System Electrical Equipment Rack," System No. AY-101 and AY-102.
2. cc\mail from Stenkamp, D. M., dated 12/13/95.
3. UCRL-15910, 1990, "Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Hazard Phenomena Hazards," R.P.Kennedy, et al, June 1990.
4. ASCE Standard Reference 4-86, 1986, "Seismic Analysis of Safety Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety Related Nuclear Structures," ASCE, Sept 1986.
5. SDC-4.1, 1989, "Standard Arch-Civil Design Criteria, Design Loads for Facilities," Rev. 11, Sept 1989.
6. SDC-4.2, 1992, "Standard Arch-Civil Design Criteria, Design and Installation of Expansion Anchors" Rev. 0, Sept 1992.
7. UBC, 1994, "Uniform Building Code," Vol. 2, Structural Engineering Design Provisions.
8. UNISTRUT, Unistrut All Purpose Metal Framing Systems, General Engineering Catalogue, Unistrut Corporation, Wayne MI., 1988.
9. HILTI, Fastening Systems, Technical Guide, Tulsa, OK., 1987.
10. ASCE Standard Reference 7-93, "Minimum Design Loads for Buildings and Other Structures," Revision of ANSI/ASCE 7-88, ASCE, December 1993.

Table 1. Component Weights of the Rack *

<u>ITEM #</u>	<u>DESCRIPTION</u>	<u>WEIGHT</u>
2	COMBINATION STARTER	50 LBS
11	P1000 UNISTRUT 465 LINEAR INCHES - AT 190 LBS/C FT =	74 LBS
12-24	UNISTRUT FITTINGS/HARDWARE	49 LBS
3	12 X 12 X 6 TERMINAL BOX	15 LBS
4	12 X 10 X 5 TERMINAL BOX	10 LBS
8	6 X 6 X 48 WIRE WAY	30 LBS
5	PRESSURE SWITCH W/ENCLOSURE	20 LBS
6	PRESSURE SWITCH W/ENCLOSURE	<u>20 LBS</u>
		268 LBS TOTAL PER RACK

* Reference 2.

FIGURE 1 - SIDE VIEW

FIGURE 2 - FRONT VIEW

FIG. 1 ELECTRICAL EQUIPMENT RACK (REF. 1)

PARTS LIST

101 QTY	102 QTY	ITEM NO.	DESCRIPTION	MATERIAL/REFERENCE	DWG NO.
1	1	1	ELECTRICAL EQUIPMENT RACK		FIGURE 1 & 2
1	1	2	COMBINATION MOTOR STARTER	PARAGRAPH 3.2.4	
1	1	3	TERMINAL BOX	HOFFMAN, A-1212CHNF W/ A-12P12	
1	1	4	TERMINAL BOX	HOFFMAN, A-1210CHNF W/ A-12P10	
1	1	5	DIAPHRAGM PRESSURE SWITCH W/ WEATHERPROOF HOUSING, OPERATING RANGE, INCHES, .05 TO .25	DWYER, SERIES 1630, MODEL NO. 1638-OWP	
1	1	6	DIAPHRAGM PRESSURE SWITCH W/ WEATHERPROOF HOUSING, OPERATING RANGE, INCHES, 2.0 TO 6.0	DWYER, SERIES 1630, MODEL NO. 1638-SWP	
1	1	7	POWER DISTRIBUTION TERMINAL BLOCK	SQUARE "D", CLASS 9080, LBA362104	
1	1	8	WIREWAY	HOFFMAN, F-66W48SS	
AR	AR	9	CONDUIT, 1", W/ MISC. FITTINGS	GALVANIZED RIGID STEEL	
AR	AR	10	CONDUIT, 1/2", W/ MISC. FITTINGS	GALVANIZED RIGID STEEL	
AR	AR	11	CHANNEL, 1 5/8	UNISTRUT, P1000	
2	2	12	FLAT PLATE FITTING	UNISTRUT, P1356	
2	2	13	FLAT PLATE FITTING	UNISTRUT, P1334	
8	8	14	FLAT PLATE FITTING	UNISTRUT, P1065	
2	2	15	FLAT PLATE FITTING	UNISTRUT, P1063	
2	2	16	NINETY DEGREE ANGLE FITTING	UNISTRUT, P2626	
4	4	17	U-SHAPE FITTING	UNISTRUT, P1044	
6	6	18	U-SHAPE FITTING	UNISTRUT, P1046 A *	
2	2	19	ANGULAR FITTING	UNISTRUT, P2103	
2	2	20	ANGULAR FITTING	UNISTRUT, P2106	
24	24	21	HEX HEAD CAP SCREW, 3/8-16UNC-2A X 1 1/4 L	UNISTRUT, HHCS037125EG	
52	52	22	HEX HEAD CAP SCREW, 1/2-13UNC-2A X 1 1/2 L	UNISTRUT, HHCS050150EG	
24	24	23	CHANNEL NUT WITH SPRING, 3/8-16UNC-2B	UNISTRUT, P1008	
52	52	24	CHANNEL NUT WITH SPRING, 1/2-13UNC-2B	UNISTRUT, P1010	

* ENLARGE 9/16" DIA HOLE TO 5/8" DIA FOR HILTI BOLT CLEARANCE.

Subject ELECTRICAL EQUIPMENT RACKOriginator W. ChenDate 12/14/95Checker ETHDate 1/3/96

1. SEISMIC ANALYSIS

LOADINGS:

FOR CLASS 3 STRUCTURE

THE HORIZONTAL g-LOAD

IS (REF. 5) 0.12 g

WHILE THE VERTICAL

g-LOAD IS $\frac{2}{3} \times 0.12 = 0.08 g$.

LOAD WILL BE COMBINED

USING SQUARE-ROOT-SUM (SRSS).

METHOD.

ANALYSIS:

X-DIRECTION (HORIZONTAL)

$$1.5 \times 0.12 g = 0.12 \times 268 = 48 \text{ lbs}$$

$$\text{SHEAR} = \frac{48}{4} = 12 \text{ lb/Bolt}$$

$$\text{TENSION} = 48 \left(\frac{37}{47} \right) \frac{1}{2} = 19 \text{ lb/Bolt}$$

Z-DIRECTION (HORIZONTAL)

$$\text{SHEAR} = \frac{48}{4} = 12 \text{ lb/Bolt}$$

$$\begin{aligned} \text{TENSION} &= 48 \left(\frac{37}{10.42} \right) \frac{1}{2} \\ &= 85 \text{ lb/Bolt} \end{aligned}$$

ANALYTICAL CALCULATIONS

Page 7 of 13Subject ELECTRICAL EQUIPMENT RACKOriginator W. CHENDate 12/15/95Checker D. L.Date 1/3/96

Y-DIRECTION (VERTICAL)

$$1.5 \times \frac{2}{3} \times \text{HORIZ} = \frac{2}{3} \times .12 \times 268 = 21.4 \#$$

$$\text{TENSION} = \frac{21.4}{4} = 5.1 \#$$

USING SRSS FOR DIRECTIONAL LOAD COMBINATIONS PER BOLT,

$$\text{SHEAR} = \sqrt{12^2 + 12^2} = 17 \#$$

$$\text{TENSION} = \sqrt{19^2 + 85^2 + 5.1^2} = 87 \#$$

CONSIDER CONCRETE MINIMUM STRENGTH OF 3000 PSI.
FOR CARBON STEEL HILTI BOLT II OF $\frac{1}{2}$ ", THE MAXIMUM
ALLOWABLE FOR (REF. 6):

TENSION: 1,100 PSI / BOLT

SHEAR 1,450 PSI / BOLT

ASSUMING MINIMUM EMBEDMENT OF $2\frac{1}{4}$ " FOR EACH BOLT.
THE DESIGN SHALL NOT CONSIDER ANCHOR PRETENSION.
BASED ON THE INTERACTION REQUIREMENTS:

$$\frac{\text{SHEAR APPLIED}}{\text{SHEAR ALLOWABLE}} + \frac{\text{TENSION APPLIED}}{\text{TENSION ALLOWABLE}} \leq 1.0$$

SO

$$\frac{17}{1,100} + \frac{87}{1,450} = 0.08 < 1.0 \quad \text{OK.}$$

ANALYTICAL CALCULATIONS

ITEM #1, 13 TOTAL
PAGES

Page 8 of 13

Subject ELECTRICAL EQUIPMENT RACKOriginator W. CHENDate 12/18/95Checker F. LinDate 1/3/96

2. EDGE DISTANCE

FOR THE CARBON STEEL HILTI KWIK-BOLT AND KWIK-BOLT II, THE MINIMUM DISTANCE FROM THE EXPANSION ANCHOR CENTER LINE TO THE EDGE OF THE CONCRETE NOT LESS THAN 10 NOMINAL DIAMETERS. FOR $\frac{1}{2}$ " BOLT, THE EDGE DISTANCE REQUIRED FOR (TABLE 1, REF. 6):

$$\text{SHEAR} = 6\frac{3}{4}"$$

$$\text{TENSION} = 5"$$

THIS DESIGN IS 5.8", SO LESS THAN $6\frac{3}{4}"$. EVEN IF CONSIDER THE EDGE DISTANCE LOAD REDUCTION EFFECT OF 50%, THE ANCHOR STILL MEETS THE REQUIREMENT OF INTERACTION LOADING FACTOR.

3. ANCHOR SPACING:

ANCHOR SPACE BETWEEN ANCHORS IS 12 NOMINAL DIAMETER, $12 \times \frac{1}{2}" = 6"$. FOR ANCHORS BETWEEN A-B AND C-D IS 47", A-C AND B-D IS 10.42". AS SEEN FROM FIGURES IN PAGE 4. IT MEETS THE ANCHOR SPACING REQUIREMENTS.

ANALYTICAL CALCULATIONS

TEAM #1
13 TOTAL PAGES

Page 9 of 13

Subject ELECTRICAL EQUIPMENT RACK

Originator W. CHEN

Date 12/20/95

Checker E. Han

Date 1/3/96

5. EMBEDMENT DEPTH

FOR $\frac{1}{2}$ " KWK BOLT II, THE MINIMUM EMBEDMENT DEPTH IS $2\frac{1}{4}$ ", THEREFORE THE BASE MATERIAL THICKNESS SHOULD BE AT LEAST GREATER THAN $2\frac{1}{4}$ ". OR FOR SAFETY, CONCRETE SHOULD BE 4" THICK.

6. MOUNTING EQUIPMENT SEISMIC CHECK.

FOR SEISMIC CLASS-3 EQUIPMENT, USE REF. 7 FOR TOTAL LATERAL SEISMIC DESIGN FORCE, F_p , FOLLOWING FORMULA MAY BE USED (REF. 7)

$$F_p = Z I_p C_p W_p$$

WHERE Z = SEISMIC ZONE FACTOR $2A = 0.15$ (TABLE 16-I)

I_p = IMPORTANCE FACTOR $= 1.25$ (TABLE 16-K)

C_p = HORIZONTAL FORCE FACTOR $= 0.75$ (TABLE 16-N)

W_p = WEIGHT OF EQUIPMENT,

SO

$$F_p = (0.15)(1.25)(0.75)W_p = 0.14W_p$$

FOR COMBINATION STARTER $W_p = 50 \#$, THE DESIGN FORCE IS

$$F_p = 0.14 \times 50 = 7 \#$$

THE $\frac{3}{8}$ -16 UNC HEX CAP SCREEN SHOULD HOLD THIS FORCE WITHOUT PROBLEM.

ANALYTICAL CALCULATIONS

Item #1
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Subject ELECTRICAL EQUIPMENT RACKOriginator W. CHENDate 12/21/95Checker F. LeeDate 1/3/96

So pressure on the top is

$$P_{tp} = 11.5(-1.0) - 11.5(0.75) = -20.13 \text{ psf}$$

$$= 11.5(-1.0) - 11.5(-0.25) = -8.63 \text{ psf}$$

WIND LOAD

$$\text{LATERAL AREA} = 47'' [5'' + 6'' + 15'' + 7'']$$

$$+ 31'' \times 6'' = 1551 + 186$$

$$= 1737 \text{ in}^2 = 12.1 \text{ ft}^2$$

LATERAL LOAD

$$= 15(12.1) + 16.2(12.1)$$

$$= 377.5 \text{ lbs CONTROL OVER SEISMIC}$$

VERTICAL LOAD

$$= 20.13 \left(\frac{1.625' \times 47''}{144} \right)$$

$$= 10.7 \text{ lb.}$$

THE WIND LOAD ON THE LATERAL SURFACE (FIG. 2) IS SMALL DUE TO ITS SURFACE AREA.

So THE CRITICAL WIND LOAD SHOULD BE ANALYZED FROM THE CONDITION OF FIGURE 1. HOWEVER, THE LATERAL AREA WAS CALCULATED ASSUMING MORE SOLID AREA CONTACTING AGAINST WIND. THEREFORE, THE LATERAL FORCE IS CONSERVATIVE.

Fig. 1

Fig. 2

ANALYTICAL CALCULATIONS

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Item #1, 13 TOTAL PAGES

Subject ELECTRICAL EQUIPMENT RACKOriginator W. CHENDate 12/21/95Checker F. HanDate 1/3/967. WIND LOADTHE VELOCITY PRESSURE q_z AT HEIGHT z MAY BE CALCULATED BY (REF 10)

$$q_z = 0.00256 K_z (ZV)^2$$

WHERE K_z : Vel. Press Exposure Coeff, 0.80 (EXPOSURE C)
(TABLE 6, REF. 10) I = IMPORTANCE FACTOR: 1.07 (REF. 5, P. 8) V = BASIC WIND SPEED: 70 mph (REF. 5, P. 8)

So

$$q_z = 0.00256 (0.8) (1.07)^2 (70)^2 = 11.5 \text{ lb/ft}^2$$

PRESSURE MAY BE CALCULATED (REF. 11, EQ. 3) BY:

$$P = q_z (G_h C_p) - q_h (G C_{pi})$$

WHERE $G_h = 1.32$; $q_h = q_z$; GUST RESPONSE FACTOR $C_p = 0.8$ FOR WINDWARD DIRECTION } WALL PRESS. COEFF.
 $= -0.5$ FOR LEENARD $G C_{pi} = 0.75$ TOWARD SURFACE } INT. PRESS. COEFFICIENTS
 $= -0.25$ AWAY FROM SURFACEWINDWARD

$$P_{\text{WINDWARD}} = 11.5 (1.32) (0.8) - 11.5 (0.75) = 3.5 \text{ psf}$$

$$= 11.5 (1.32) (0.8) - 11.5 (-0.25) = 15.0 \text{ psf}$$

LEENARD

$$P_{\text{LEENARD}} = 11.5 (1.32) (-0.5) - 11.5 (0.75) = -16.2 \text{ psf}$$

$$= 11.5 (1.32) (-0.5) - 11.5 (-0.25) = -4.7 \text{ psf}$$

TOP OF THE RACK $G_h C_p = -1.0$ FOR HEIGHT LESS THAN 60'

ANALYTICAL CALCULATIONS

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 Subject ELECTRICAL EQUIPMENT RACK

 Originator W. CHEN

 Date 12/21/95

 Checker E. Lee

 Date 1/3/96

TAKE MOMENT ABOUT B,

$$\sum M_B = R_A (10.42) + 377.5 (37)$$

$$R_A = -1,341 \text{ lbs COMP}$$

$$R_B = 1341 + 268 + 10.17$$

$$= 1,619.17 \text{ lbs TENSION}$$

 SINCE THERE ARE 2 ANCHORS,
 THE ACTUAL REACTION FORCES
 ARE

$$R_A = \frac{1341}{2} = 671 \text{ lb. COMP}$$

$$R_B = \frac{1,619.17}{2} = 809.6 \text{ lb TENSION}$$

 THE SHEAR AND TENSION FORCES ON ANCHORS DUE TO
 WIND LOAD ARE:

$$\text{SHEAR} = \frac{377.5}{4} = 94.4 \text{ lb}$$

$$\text{TENSION} : 809.6 \text{ lb.}$$

CHECK THE INTERACTIVE REQUIREMENTS

$$\frac{94.4}{1,100} + \frac{809.6}{1,450} = 0.086 + 0.558 = 0.644 < 1.0 \text{ OK.}$$

 WHERE 1,100 lbs AND 1,450 lbs ARE THE SHEAR AND TENSION
 ALLOWANCE FOR HILTI KWIK-BOLT II (REF. 6, TABLE 9)

ANALYTICAL CALCULATIONS

 Item #1
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 Subject ELECTRICAL EQUIPMENT RACK
 Originator W. CHEN Date 12/22/95
 Checker E. Chen Date 1/3/96

8. QUALIFICATION OF UNISTRUT LOAD.

FOR 12 GAUGE (0.105"), P1000 UNISTRUT CHANNEL, WITH BOTH END SUPPORTED 90° FITTING CONNECTION, THE DESIGN LOAD IS 5,000 LBS. EVEN WITH CONSERVATIVE CALCULATIONS, REACTION FORCE AT B (FIGURE AT P.10), IT IS WITHIN THE ALLOWABLE (P.28, REF. 8).

9. BEAM CHECK.

$$+\circlearrowleft M_A = 20 \times 12 + 10 \times 25 + 15(40) - 47 R_B$$

$$R_B = 23 \text{ lbs}$$

$$R_A = 22 \text{ lbs}$$

FLAT PLATE FITTING (UNISTRUT P1356) AT R_A , BEAM UNIFORM LOAD ALLOWED IS 680 LBS AT 48" SPAN (REF. 8). FOR EQUIPMENT LOADED ON THE SPAN OF 47" SHOULD HAVE NO PROBLEM.

STRENGTH OF P1010 NUTS USED IN P1000 CHANNEL IS:

RESISTANCE TO SLIP = 1500 LBS PER BOLT.

PULL OUT STRENGTH = 2000 LBS PER BOLT.

(INCLUDES A MINIMUM SAFETY FACTOR OF 3).

ICBO Evaluation Service, Inc.

5360 WORKMAN MILL ROAD • WHITTIER, CALIFORNIA 90601-2299

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EVALUATION REPORT

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ER-4627

Reissued July 1, 1996

Filing Category: FASTENERS—Concrete and Masonry Anchors (066)

KWIK BOLT-II

HILTI, INC.

5400 SOUTH 122ND EAST AVENUE

TULSA, OKLAHOMA 74146

I. Subject: Kwik Bolt-II Concrete Anchors.

II. Description: A. General: Kwik Bolt-II (KB-II) concrete anchors consist of a stud, wedge, nut and washer. The stud is manufactured from a carbon or stainless steel material. The carbon steel Kwik Bolt-II stud is made from AISI 1038 or AISI 1144 materials. The wedges are made of AISI 1010 steel, except that KB-II $\frac{3}{4}$ -by-12, KB-II 1-by-6, KB-II 1-by-9, and KB-II 1-by-12 have AISI 304 (stainless steel) wedges. All carbon steel components are zinc-plated. Each component of the stainless steel Kwik Bolt-II is made from either AISI 304 or AISI 316 material.

The stud consists of a high-strength steel rod threaded at the upper end. The tapered mandrel has an increasing diameter toward the anchor base and is enclosed by a three-section wedge which freely moves around the mandrel. In the vertical direction, the wedge movement is restrained by the mandrel taper at the bottom and by a collar at the top of the mandrel. When subjected to torque, the wedge is forced against the wall of the predrilled hole to provide the anchorage. Allowable tension and shear values in normal-weight concrete are shown in Tables 3 and 4 for the carbon steel and stainless steel anchors, respectively. Allowable tension and shear values for lightweight concrete are shown in Table 5.

B. Post Nut Kwik Bolt-II: Post Nut Kwik Bolt-II concrete anchors consist of a carbon (AISI 1144) or stainless (Type 304) steel stud and post nut. The stud threaded end is fabricated to accept a threaded post nut whose outside diameter is equal to the nominal diameter of the stud. The post nut has a countersunk head configuration. Allowable tension and shear values in 3,000 psi (20.7 MPa) normal-weight concrete for carbon and stainless anchors are shown in Table 7.

C. Installation: 1. Kwik Bolt-II: The Kwik Bolt-II anchor is installed in concrete with a predrilled hole of approximately the same size as the nominal bolt diameter. The drilled holes must exceed the depth of anchor embedment by at least two anchor diameters to permit overdriving of anchors and to provide a dust-free area. The anchor must be hammered into the predrilled hole until at least six threads are below the surface. The nut must be tightened against the washer until the torque values specified in Table 1 are attained. Minimum embedment depths, and edge and spacing requirements, are set forth in Table 1.

2. Post Nut Kwik Bolt-II: The Post Nut Kwik Bolt-II anchor is installed in concrete in a predrilled hole approximately the same size as the nominal bolt diameter. The drilled holes must exceed the depth of anchor embedment by approximately two anchor diameters to permit overdriving and to provide a dust-free area. The anchor is tapped into the hole until the post nut head touches the material to be fastened. The post nut is then loosened by two complete turns and the anchor tapped again until the

post nut is again in contact with fastened material. The post nut is then tightened.

D. Special Inspection: Where special inspection is required, compliance with Section 1701.5 of the code is necessary. The special inspector must be on the jobsite continuously during anchor installation to verify anchor type, anchor dimensions, concrete type, concrete compressive strength, predrilled hole dimensions, anchor spacing, edge distances, slab thickness, anchor embedment and tightening torque.

E. Identification: The anchors are identified in the field by dimensional characteristics and packaging. The packaging label indicates the manufacturer's name and address and the size and type of anchor. A length identification code letter is stamped on the threaded end of the bolt. See the length identification system in Table 6.

III. Evidence Submitted: Reports of tests in accordance with the ICBO ES Acceptance Criteria for Expansion Anchors in Concrete and Masonry Elements (AC01), dated January 1993.

Findings

IV. Findings: That the Kwik Bolt-II concrete anchors described in this report comply with the 1994 *Uniform Building Code*, subject to the following conditions:

1. Installation dimensions and installation torques are as noted in Table 1.
2. Allowable tension and shear values are as noted in Tables 2 through 5.
3. Calculations justifying that the applied loads comply with this report are submitted to the building official for approval.
4. Special inspection is provided as set forth in Section II D of this report.
5. Anchors are limited to installation in uncracked concrete, which is concrete subjected to tensile stresses not exceeding 170 psi (483 kPa) as induced by external loads, deformations and interior exposures.
6. Use of the anchors in resisting wind and seismic forces is beyond the scope of this report.
7. Use of the anchors in resisting vibratory or shock loads, such as those present in supports for reciprocating engines or crane loads, is beyond the scope of this report.
8. Anchors are limited to nonfire-resistive construction.
9. Anchors are manufactured by Hilti, Inc., at 5400 South 122nd East Avenue, Tulsa, Oklahoma, with quality control inspections by Underwriters Laboratories Inc. (AA-637).

This report is subject to re-examination in one year.

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P1000 & P1001 CHANNELS

FOR 1-5/8" (41 MM) WIDTH SERIES CHANNEL

COLUMN LOADING DATA

Unbraced Height		Channel	Max. Allowable Load at Slot Face		Maximum Column Load Applied at C.G.							
					K = .65		K = .80		K = 1.0		K = 1.2	
In	mm		Lbs	kN	Lbs	kN	Lbs	kN	Lbs	kN	Lbs	kN
24	610	P1000	3400	15.1	9600	42.7	9500	42.3	9320	41.5	9100	40.5
		P1001	6360	28.3	23820	106.0	23560	104.8	23130	102.9	22610	100.6
36	914	P1000	3000	13.3	7640	34.0	7400	32.9	7000	31.1	6490	28.9
		P1001	6190	27.5	23190	103.2	22610	100.6	21640	96.3	20460	91.0
48	1219	P1000	2570	11.4	5910	26.3	5530	24.6	4960	22.2	4430	19.7
		P1001	5970	26.6	22310	99.2	21270	94.6	19560	87.0	17460	77.7
60	1524	P1000	2230	9.9	4780	21.3	4390	19.5	3850	17.1	3330	14.8
		P1001	5690	25.3	21180	94.2	19560	87.0	16870	75.0	13590	60.5
72	1829	P1000	1970	8.8	4090	18.2	3680	16.4	3140	14.0	2650	11.8
		P1001	5360	23.8	19790	88.0	17460	77.7	13590	60.5	9570	42.6
84	2134	P1000	1760	7.8	3600	16.0	3170	14.1	2630	11.7	2160	9.6
		P1001	4970	22.1	18150	80.7	14980	66.6	10130	45.1	7030	31.3
96	2438	P1000	1580	7.0	3220	14.3	2770	12.3	2240	10.0	1800	8.0
		P1001	4510	20.1	16270	72.4	12120	53.9	7750	34.5	5380	23.9
108	2743	P1000	1430	6.4	2910	12.9	2450	10.9	1930	8.6	..	..
		P1001	4030	17.9	14120	62.8	9570	42.6	6130	27.3	4250	18.9
120	3048	P1000	1290	5.7	2640	11.7	2180	9.7	..	..	..	..
		P1001	3610	16.1	11750	52.3	7750	34.5	4960	22.1	..	..

**KL
T > 200**ELEMENTS OF SECTION**

Channel	Areas of Section		Axis 1 - 1						Axis 2 - 2					
			I		S		r		I		S		r	
	In ²	cm ²	In ⁴	cm ⁴	In ³	cm ³	In	cm	In ⁴	cm ⁴	In ³	cm ³	In	cm
P1000	5.56	3.6	185	7.7	202	3.3	5.77	1.5	236	9.8	290	4.7	6.51	1.7
P1001	11.12	7.2	930	38.7	572	9.4	9.15	2.3	472	19.6	580	9.5	6.51	1.7

I - Moment of Inertia

S - Section Modulus

r - Radius of Gyration

P1000® & P1001 CHANNELS FOR 1-5/8" (41 MM) WIDTH SERIES CHANNEL

P1000

Pierced channels are found on pages 60 and 61.

Weight: 190 Lbs/C Ft (283 kg/100 m)

P1001

Weight: 380 Lbs/C Ft (566 kg/100 m)

BEAM LOAD*

*Maximum allowable uniform load.

Channel	Weight		Allowable Moment		Material Thickness		Standard Lengths		Finishes				Other Materials	
	Lbs/Ft	kg/m	In-Lb	N·m	In	mm	10'	20'	PL	GR	HG	PG	SS	EA
P1000	1.90	2.8	5,080	570	.105	2.7	■	■	■	■	■	■	■	■
P1001	3.80	5.7	14,390	1630	.105	2.7	■	■	■	■	■	■	■	■

Nominal thickness of 12 gage strip steel is .105 inches.

1-9/8" Channels

Nuts & Hardware

General Fittings

Pipe/Conduit Supports

Electrical Fittings

Concrete Inserts

1-1/4" Framing System

13/16" Framing System

Spec. Metals & Fiberglass

Index

BEARING LOADS FOR CHANNEL & COMBINATIONS

FOR 1-5/8" (41 MM) WIDTH SERIES CHANNEL

BEARING LOADS ON UNISTRUT CHANNELS

	Bearing Length 1-5/8" (41 mm)		Bearing Length 1-5/8" (41 mm)		Bearing Length 3-1/4" (92 mm)

Channel	Maximum Allowable Loads		Maximum Allowable Loads		Maximum Allowable Loads	
	Lbs	kN	Lbs	kN	Lbs	kN
P1000	5000	22.2	3500	15.6	8000	35.6
P1100	3500	15.6	2500	11.1	5500	24.5
P2000	2000	8.9	1500	6.7	3000	13.3
P3000	5000	22.2	3500	15.6	8000	35.6
P3300	6000	26.7	4000	17.8	9000	40.0
P4000	2200	9.8	1700	7.6	3500	15.6
P4100	3400	15.1	2600	11.6	4800	21.4
P5000	4000	17.8	2000	8.9	5500	24.5
P5500	5000	22.2	3500	15.6	8000	35.6
P9000	5000	22.2	3500	15.6	8000	35.6
P9200	5000	22.2	3500	15.6	8000	35.6

Safety Factor — 2-1/2

HNF-2468, Rev.0

1-5/8" Channels

Nuts & Hardware

General Fittings

Pipe/Conduit Supports

Electrical Fittings

Concrete Inserts

1-1/2" Framing System

13/16" Framing System

Spec Metals & Fittings

Index

CHANNEL NUT LOAD DATA

FOR 1-5/8" (41 MM) WIDTH SERIES CHANNEL

MAXIMUM ALLOWABLE PULL-OUT AND SLIP LOADS

Channel Nut Size/Thread	Gage	Channel	Allowable Pull-Out Strength		Resistance to Slip		Torque	
			Lbs	kN	Lbs	kN	Ft Lbs	N•m
3/4" - 10	12	P1000 P3000 P5000 P5500	2500	11.1	1700	7.6	125	170
5/8" - 11			2500	11.1	1500	6.7	100	135
1/2" - 13			2000	8.9	1500	6.7	50	70
7/16"- 14			1400	6.2	1000	4.4	35	50
3/8" - 16			1000	4.4	800	3.6	19	25
5/16"- 18			800	3.6	500	2.2	11	15
1/4" - 20			600	2.7	300	1.3	6	8
1/2" - 13	12	P3300	1500	6.7	1500	6.7	50	70
3/8" - 16			1000	4.4	800	3.6	19	25
5/16"- 18			800	3.6	500	2.2	11	15
1/4" - 20			600	2.7	300	1.3	6	8
1/2" - 13	14	P1100 & P4100	1400	6.2	1000	4.4	50	70
3/8" - 16			1000	4.4	750	3.3	19	25
5/16"- 18			800	3.6	400	1.8	11	15
1/4" - 20			600	2.7	300	1.3	6	8
1/2" - 13	16	P2000 & P4000	1000	4.4	1000	4.4	50	70
3/8" - 16			1000	4.4	750	3.3	19	25
5/16"- 18			800	3.6	400	1.8	11	15
1/4" - 20			600	2.7	300	1.3	6	8

* May require 3/8" or 1/2" thick fitting.

Nut design loads include a minimum safety factor of 3.

Note: Refer to the Channel Nut Selection Chart on the following two pages for the part number.

1-3/8" Channel

Nuts & Hardware

General Filings

Pipe/Conduit Supports

Electrical Filings

Concrete Inserts

1-1/2" Framing System

13/16" Framing System

Spec Metals & Fiberglass

Index

W320-24-038

DBE Analysis for the 1000 CFM
Portable Exhausters (VTP-POR-007 & 008)

DESIGN VERIFICATION CHECKLIST

Document Reviewed: Project W-320, Tank 241-C-106 Waste Retrieval, DBE analysis for the 1000 CFM Exhausters (VTP-POR-007 & 008) calculation No. W-320-24-038

Author S. A. Fargo

Yes No N/A

☒	☐	☐	Problem completely defined.
☒	☐	☐	Necessary assumptions explicitly stated and supported.
☐	☐	☒	Computer codes and data files documented.
☒	☐	☐	Data used in calculations explicitly stated in document.
☒	☐	☐	Data checked for consistency with original source information as applicable.
☒	☐	☐	Mathematical derivations checked, including dimensional consistency of results.
☐	☐	☒	Models appropriate and used within range of validity or use outside range of established validity justified.
☒	☐	☐	Hand calculations checked for errors.
☐	☐	☒	Code run streams correct and consistent with analysis documentation.
☐	☐	☒	Code output consistent with input and with results reported in analysis documentation.
☒	☐	☐	Acceptability limits on analytical results applicable and supported. Limits checked against sources.
☒	☐	☐	Safety margins consistent with good engineering practices.
☒	☐	☐	Conclusions consistent with analytical results and applicable limits.
☒	☐	☐	Results and conclusions address all points required in the problem statement.
☐	☐	☐	Software QA Log Number.

M. A. Islam
Reviewer

7/14/98
Date

PROJECT W-320
DESIGN BASIS EARTHQUAKE ANALYSIS
FOR THE 1000 CFM PORTABLE EXHAUSTER
(Calculation No. W-320-24-038)

OBJECTIVE

The objective of this engineering evaluation is to analyze the existing 1000 CFM portable exhauster design (VTP-PRO-007 & 008) to determine whether the exhauster will overturn and/or slide during a DBE event for Non-Reactor Safety Class 1, per SDC 4.1 Rev.12.

DESIGN INPUT

This evaluation is based on the following input:

- DBE ground acceleration for Non-Reactor Safety Class facility as specified in SDC 4.1 Rev. 12.
- The exhausters are not restrained and can be resting on sand or concrete floor.
- Exhauster assembly including the skid were designed and qualified by Trentec, Inc. (Ref. 5).
- Weight of the assembly is taken from reference No. 5 and modified to include the new fan shaft mounting base.

ASSUMPTIONS

The coefficient of friction between steel and concrete is normally taken to be 0.70 however, the exhauster jack pads will be resting on smooth concrete surface indoors and loose sand and gravel surface while the unit is outdoors therefore, it is assumed that the coefficient of friction for both surfaces to be 0.50. All other assumptions are identified in the attached calculations.

SUMMARY/CONCLUSIONS

The analysis of the portable exhauster unit concluded that the exhauster will not overturn during a DBE event of 0.20g ZPA horizontal acceleration however, the exhauster assembly unit will slide during the DBE event. The sliding distance of the exhauster unit during the DBE event was calculated by equating the maximum Kinetic Energy to the maximum Potential Energy generated during the seismic event; the maximum distance the unit will travel was found to be 3.92 inches.

Calculation No. W-320-24-038

REFERENCES

1. American Concrete Institute, Building Code Requirements For Reinforced Concrete, ACI 318/318R-160.
2. HNF-SD-W320-FDC-001, Rev. 5, Functional Design Criteria for Tank 241-C-106 Waste Retrieval, Project W-320.
3. JABE-WADCO-02 Rev. 2, Seismic Design Criteria For The Fast Flux Test Facility, July 1975.
4. Mario Paz, "Structural Dynamics Theory And Computation," Chapter 8, 1980.
5. Trentec, Inc., Definitive Design Report 96675.2, Rev. 2, "Portable Exhauster System For Waste Tank Ventilation," October 1997.
6. Farwell & Hendricks, Inc. Vendor Drawings Numbers 96675-0V1 Rev. 5, 96675-A01 Rev. 1, & 96675-0V2 Rev. 4.

CALCULATION SHEET

Client LUMC

Contract/Job No. W-320

Subject Project W-320

Originated By S. Jorgensen

Date 6/98

DBE analysis for portable exhaust

Checked By M.A. Islam

Date 7/14/98

Location 200E

Revised By

Date

Reference:

Vendor DWG.

No. 96675-0V1 Rev.5

The C.G. is taken at 35" above the leveling Jack base (the center of the inlet-outlet duct). The C.G. estimate of 35" is based on a survey of the different Exhauster components and their location & weight.

The moment arm is the distance between the center of the Jacks (no credit is taken for the Jack pads as they are considered non-rigid; used for bearing only.)

CALCULATION SHEET

Calc. No. W-320-24-038Revision 0Page No. 2 of Client LMHCContract/Job No. W-320Subject Project W-320Originated By S. SargunDate 6/98DRE analysis for Portable Exhauster Checked By M.A. IslamDate 7/14/98Location ZOE ERevised By Date

$$\Sigma M_0$$

$$F_h(b) = F(a) \quad \text{--- ①}$$

$$\Sigma F_{\text{horiz}}$$

$$F_h = F_f \quad \text{--- ②}$$

Substituting

$$F_f(b) = F(a)$$

$$\mu W(1+g_v)(b) = W(1+g_v)(a)$$

$$\mu = \frac{a}{b}$$

For the System to overturn, $\mu > \frac{a}{b}$

$$\frac{a}{b} = 1.04 > \mu = 0.5 \quad \text{System Will not Overturn}$$

Note: $\mu = 0.70$ for concrete & Smooth bar (Ref 1)For this application, Steel on Smooth concrete Surface or gravel Use $\mu = 0.50$

CALCULATION SHEET

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Subject Project W-320

Originated By S. Fajon

Date 6/98

DBE analysis for Portable Exhauster

Checked By M.A. Islam

Date 7/14/98

Location 200E

Revised By

Date

Check Sliding -

Sliding will take place if $F_H > F_f$

$$Wg_H > \mu W(1 \pm g_v)$$

$$\mu < \frac{g_H}{1 \pm g_v}$$

$$g_H = 1.5 \sqrt{g_{Hx}^2 + g_{Hz}^2}$$

The design basis for W320 is SDC 4.1 Rev. 12 (Ref 2)
For Non-Reactor SC-1, the max horizontal acceleration
is 0.42 & max vertical is 0.28

$$g_H = 1.5 \sqrt{(0.42)^2 + (0.42)^2} = 0.891$$

$$g_v = 0.28$$

$$\mu = \frac{0.891}{1 - 0.28} = 1.24 > \mu = 0.5$$

The unit will slide; to find the distance the unit
will slide set the Kinetic Energy KE_{max} = the
Potential Energy PE_{max}

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Conservative Assumptions:

- DBE produces 10 cycles of maximum response (Ref 3)
- Horizontal Response, is constant for all 10 cycles.
- Vertical Response, always acts upward.
- Forcing Function is sinusoidal.

The maximum absolute acceleration - Spectral acceleration
($S_a = \ddot{y}_{max}$)

The maximum relative Velocity - Spectral Velocity ($S_v = \dot{y}_{max}$)

The maximum relative displacement - Spectral displacement
($S_D = y_{max}$)

$$S_v = \omega S_D = \frac{S_a}{\omega} = \frac{a g_H}{2\pi f} \quad (\text{Ref 4})$$

The maximum Kinetic Energy

$$\begin{aligned} KE_{max} &= \frac{1}{2} m v^2 = \frac{1}{2} \left(\frac{W}{a} \right) (S_v)^2 \\ &= \frac{1}{2} \left(\frac{W}{a} \right) \left(\frac{a^2 (g_H)^2}{4\pi^2 f^2} \right) = \frac{W a (g_H)^2}{8\pi^2 f^2} \end{aligned}$$

The maximum Potential Energy

$$PE_{max} = F_f * y_{max} = W U(1 - g_v) y_{max}$$

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$$K E_{\max} = P E_{\max}$$

$$\frac{W a (g_H)^2}{8 \pi^2 f^2} = W \mu (1 - g_v) y_{\max}$$

$$y_{\max} = \frac{a (g_H)^2}{8 \pi^2 f^2 \mu (1 - g_v)} \quad (a = 386.4 \text{ in/sec}^2)$$

For 10 cycles of maximum response

$$y_{\max} = \frac{5 a (g_H)^2}{4 \pi^2 f^2 \mu (1 - g_v)}$$

Considering the first mode natural frequency

$$f = 5.2363 \quad (\text{Table 5.3 Ref 5})$$

NOTE: The frequencies of Table 5.3 are based on 0.12g ZPA and total exhauster assembly weight = 8855 lb (Ref 5). This evaluation is based on 0.20g ZPA and total weight of 9000 lb (145 lb added for the modified Fan Shaft - estimated per field inspection). This evaluation is concerned with the Skid (more rigid than the rest of the components) also the first mode of $f = 5.2363 \text{ Hz}$ is used (conservative)

$$y_{\max} = \frac{5(386.4)(0.891)^2}{4 \pi^2 (5.24)^2 (0.5)(1 - 0.28)} = 3.92 \text{ in}$$

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