Florida Power
CORPORATION

June 11, 1984
3F0684-03

Mr. H. R. Denton, Director
Office of Nuclear Reactor Regulation
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Subject: Crystal River Unit 3
Docket No. 50-302
Operating License No. DPR-72
Control of Heavy Loads - Phase I (NUREG-0612)

Dear Mr. Denton:

This letter is in response to the Nuclear Regulatory Commission's concerns regarding the control of heavy loads, phase I (NRC to FPC letter 3N-0583-17, dated May 19, 1983). Florida Power Corporation (FPC) with consultation from the Stone & Webster Engineering Corporation (SWEC) reviewed these concerns and has identified actions necessary to ensure that the intent of NUREG - 0612 is satisfied.

There are ten (10) copies of the SWEC's report including applicable analyses attached. The following is a summary together with FPC's position .

NRC Concern: FPC should resolve the longitudinal stiffener and trolley bumper deficiencies on the fuel (auxiliary) building crane and reactor building polar crane.

FPC Position: The attached analysis shows that the longitudinal stiffeners are not required to prevent girder buckling. Therefore FPC has no plans to modify the crane with regards to longitudinal stiffeners.

FPC will add trolley bumpers to both the reactor building polar crane and auxiliary building crane. It is anticipated that these bumpers will be added during Refuel V scheduled for Spring 1985.

NRC Concern: FPC should complete the design verification of the intake structure crane.

FPC Response: The design was verified to meet the requirements of CMAA 70 and ANSI B30-2-0. A copy of the design Verification is attached.

Two analyses were deferred from the "six-month" effort to be included in the "nine-month" work scope. FPC (via SWEC) has completed these analyses and has transmitted them under separate covers (November 23, 1983). The following positions reflect interim corrective actions FPC will implement until such time as the NRC staff has accepted the "nine-month" analyses.

A033 1/10 Limited Distribution

A. Potential drop of Spent Fuel Divider Gate

1. FPC will prohibit the movement of the spent fuel pool divider gate until after fifty (50) days following transfer of spent fuel to the spent fuel pools.

2. Only spent fuel shall be placed in the vicinity of the gate handling areas.

B. Potential drop of Reactor Building Tendon Jack

1. As shown in Appendix G, Calculation 8 of the NUREG-0612 Nine Month Report, the Spent Fuel Pool Missile Shields will not fail due to the drop of the Reactor Building Tendon Jack. Therefore, no interim restrictions on the movement of the jack is deemed necessary.

One additional change to our previous status requires further discussion. The Reactor Vessel Head and Internals Special Lifting Device (tripod) was removed from the Reactor Building between Refuel III and Refuel IV. Non-destructive examinations identified some surface indications. Research into the tripod's construction and pre-service testing failed to produce adequate documentation of welding methodology and load testing. Therefore FPC performed both documented weld repairs and a load test. This subject was addressed in LER 83-17 by FPC and Information Notice No. 83-71 by the NRC. Region II and Babcock & Wilcox (tripod supplier) have reviewed the repairs/tests. FPC considers our tripod to be a substantially upgraded component as a result. Since the Nine-Month analyses demonstrate the ability of the plant to be shut down following a postulated reactor vessel head drop, the tripod is not considered safety-related. Furthermore, the difficulty in performing adequate non-destructive exams (NDE) on a prior-to-use basis overweighs any increased assurance of its structural integrity. Therefore FPC proposes to perform the following in-service tests:

a) A 10 minute hold immediately after the head has been lifted to a height of less than one foot with documented visual exam of the tripods' performance on each head lift.

b) A thorough NDE every third refueling cycle (e.g., prior to Refuel VII, X, etc.). This interval should equal approximately 4.5 to 6 years.

Our project manager requested we idenify the status of FPC's actions with regard to Generic letter 83-42. FPC has forwarded a copy of Generic Letter 83-42 to the A/E firm working on the design of our upgrade/replacement of FHCR-5 which FPC is making single failure proof. Its concerns will be addressed in the appropriate specification(s).

Sincerely,

G. R. Westafer

G. R. Westafer
Manager
Nuclear Operations Licensing and Fuel Management

GRW/ddl

Attachments

cc: Mr. J. P. O'Reilly, Regional Administrator
U.S. Nuclear Regulatory Commission, Region II
101 Marietta Street, Suite 2900
Atlanta, GA 30303

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EXAMINATIONS
CONSULTING
ENGINEERING

Florida Power Corporation
Attention Mr. E. E. Froats
Nuclear Project Management Engineer
3201 34th Street, South
St. Petersburg, FL 33733

August 1, 1983
J.O.No. 14235.17
SWCR- 150

SWEC TASK .17
CONTROL OF HEAVY LOADS
CRYSTAL RIVER - UNIT 3

Reference: FPC Letter No. WPN83-0615 dated June 8, 1983

SWEC has reviewed the Franklin Research Center's (FRC's) Technical Evaluation Report (TER) submitted by the NRC to Florida Power Corporation (FPC) as Enclosure 1 to NRC Letter No. 3N-0583-17 dated May 19, 1983. The following information is presented to address the NRC's concerns of the control of heavy loads - Phase 1, as identified by FRC in its TER. SWEC believes that this information will allow the NRC to complete its evaluation of Phase 1.

The TER identifies the following areas in which additional licensee action is necessary to ensure that the overall intent of NUREG-0612, Section 5.1.1, is satisfied.

1a. FPC should resolve the longitudinal stiffener and trolley bumper deficiencies on the fuel (auxiliary) building crane and reactor building polar crane.

2a. FPC should complete the design verification of the intake structure crane.

With respect to Item 1a, SWEC has performed a detailed analysis to evaluate the potential for buckling of the auxiliary building crane's main girder. The results of the analysis show that the longitudinal stiffeners, while not meeting the minimum moment of inertia required by CMAA 70, are not required to prevent girder buckling. The analysis is shown in Enclosure 1.

Further, SWEC has determined that trolley bumpers can be added to both the reactor building polar crane and the auxiliary building crane with little impact. The trolley bumpers for the auxiliary building crane can be conveniently added during the upgrade of the crane to single-failureproof criteria.

SWEC has completed the design verification of the intake structure crane, as required by Item 2a. In summary, the crane meets the design requirements of CMAA 70 and ANSI B30-2-0. A detailed design verification is enclosed as Enclosure 2.

In addition, the TER identifies the following areas in which additional licensee interim action in accordance with NUREG-0612, Section 5.3, is necessary. This is to ensure that handling of heavy loads will be performed in a safe manner until implementation of the general guidelines of NUREG-0612, Section 5.1, is complete.

1b. FPC should prohibit the movement of the pool divider gate over spent fuel in the spent fuel pool.

2b. FPC should limit the use of the 5-ton jack over the fuel building until the issue of impact on spent fuel is resolved and appropriate actions are completed.

In response to Item 1b, SWEC has prepared calculations (Enclosure 3), which show that if the gate is dropped on the fuel pool floor, the pool floor will not fail, thereby preventing loss of cooling water. Note that the liner is penetrated, thereby allowing some seepage through the concrete, which would be compensated for by the borated water makeup capability. However, because of potential damage to spent or partially spent fuel (Enclosure 4), SWEC believes the following precautions should be included in CR3 operating procedures.

- o Only totally spent fuel shall be placed in the vicinity of the gate handling area.
- o The gate may be moved only after 50 days following transfer of spent fuel to the spent fuel pools.

For Item 2b, SWEC has performed calculations (Enclosure 5), which show that if the 5-ton jack (weight 7,000 lb) is dropped over the fuel building, the spent fuel pool missile shields will prevent the jack from falling into the pool. Based on this, SWEC believes that movement of the jack can be unrestricted.

Thomas J. Lanza

T. J. Lanza
Project Engineer

Enclosures

EHM:ACD

ENCLOSURE 1

FHCR-5 CRANE GIRDER BUCKLING ANALYSIS

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION TITLE PAGE

***SEE INSTRUCTIONS ON REVERSE SIDE**

5010.64 (FRONT)

CLIENT & PROJECT	PAGE 1 OF 33
Florida Power Corp. - Crystal River - 3	including attachments.

CALCULATION TITLE (Indicative of the Objective):	QA CATEGORY (✓)
FHCR-5 Auxilary Bldg Crane Crane Grider Buckling Analysis	☒ I - NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ______ OTHER

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CURRENT CALC. NO.	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
14235..	Structural	14235.17-C03	—	—

* APPROVALS - SIGNATURE & DATE			REV. NO. OR NEW CALC NO.	SUPERSEDES * CALC. NO. OR REV. NO.	CONFIRMATION * REQUIRED (✓)	
PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)			YES	NO
R. D. Salter 7/25/83	[illegible] 7/25/83 [illegible] 7/29/83	Kenneth Wislocky 7/29/83	NEW	—	✓	

DISTRIBUTION *

GROUP	NAME & LOCATION	COPY SENT (✓)	GROUP	NAME & LOCATION	COPY SENT (✓)
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Historical Data Rev

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FHCR-5 - Auxiliary Building Crane

Crane Girder Buckling Analysis.

Objective of Calculation.

The purpose of this calculation is to check the main girders of the Auxiliary Building Crane (FHCR-5) for buckling. This analysis is required due to the determination of the Tera Corporation report (Jan., 1982) which shows insufficient moment of inertia for the longitudinal stiffeners.

Calculation Method/ Assumptions.

For the purpose of the analysis a detailed finite element computer model was prepared using "Ansys" rev. 3 (ST-360). One of the crane girders was completely modelled with the exception of the longitudinal stiffeners. These stiffeners were left out inorder to determine if the girder was adequate without them.

Two different loading locations were analyzed. These locations were judged to be the worst cases for the girders. The loads were provide by Whiting Corporation.

Calculation Method / Assumptions. (con't)

and represent the maximum wheel loads from the trolley for both girders.

Confirmation Req'd. → 96,000 lb rear main hook wheels
79,000 lb Aux. hook wheels.

The original design specification (refer: # 4) section 4.00 states that this crane is a class A crane as classified by CMAA-70. (refer: 5). Accordingly, all class A cranes are required to take 2½ % of the live load for lateral loading (section 3.3.2.1.2, CMAA-70). This would represent 2.4 kips and 1.975 kips at the trolley wheel locations. These loads were assumed to have very little contribution to this analysis but were included by providing a simple static calculation.

The results of the computer analysis provide the element stresses for the applied loadings. The acceptable stresses for the prevention of buckling are provided in the body of this calculation and are based on the AISC steel design code and other established references.

References & Sources of Information

1.) Manual of Steel Construction 8th ed. AISC.

2.) Steel Structures Design and Behavior, Salmon and Johnson, 1971, International Textbook Company.

3) Ansys - BCS rev. 3 ST-360 Update 67L
Swanson Analysis System, Inc

4) "Detailed Specification for Cranes - Crystal River Plant - Unit No 3 Florida Power Corporation" # GAI-RO-2690, Gilbert Assoiates, Inc. 9-12-68

5) "CMAA Specification # 70" 1975 Revision.

6) Crystal River - Unit No 3 - Flordia Power Corp
Vendor Drawings:

a) Whiting Corp U-62238
b) " " U-62279
c) " " U-63718

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Note: see method and discussion on pages 4, 4.1, & 4.2

DESIGN CHECKLIST

LOAD CONSIDERATIONS	✓
DEAD	
LIVE	
EQUIPMENT	
PIPE	
WIND	
TORNADO	
INTERNAL PRESSURE	
EXTERNAL PRESSURE	
SEISMIC	
THERMAL	
HYDROSTATIC	
EARTH PRESSURES	
MISSILE	
CONSTRUCTION	
SNOW AND ICE	

MATERIAL	GRADE/ STRENGTH
CONCRETE	
REINFORCEMENT	
STRUCTURAL STEEL	A36
HIGH STRENGTH BOLTS	
COMMON BOLTS	
WELD RODS	
ANCHOR BOLTS	
STUDS	

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RECORD OF CONFIRMATIONS

PAGE NO.	DESCRIPTION	REFERENCE	CONFIMATION DATE
4.1.8	Max. trolley wheel loads		

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Computer Index

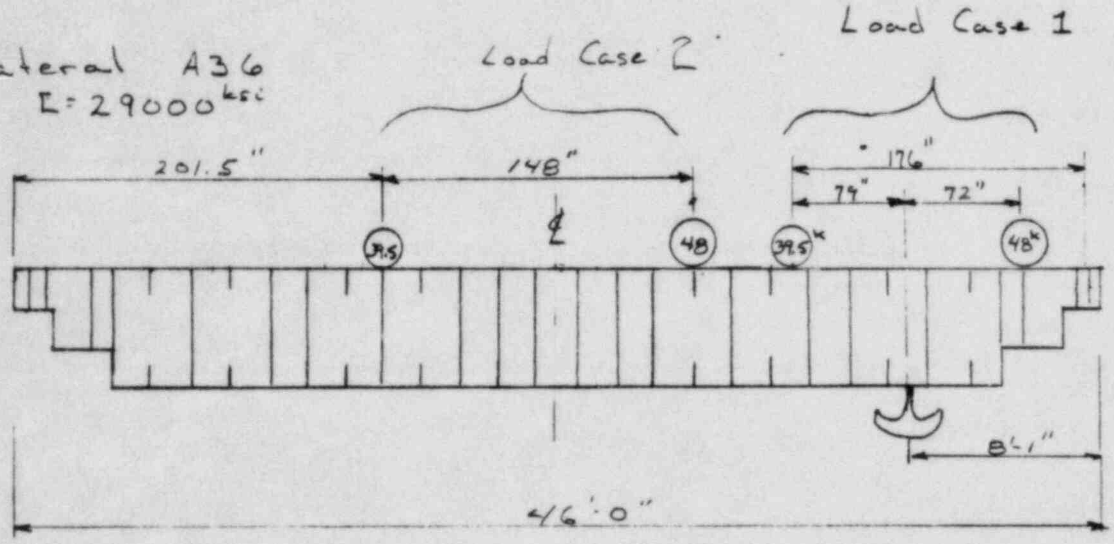
Charge No.
S&W
CDC
BCS SEWCHB

Status
G: Good Run — Micro fiche
V: Void Run — Micro film

Run No.	Job No.	Date	Status*	Prepared By	Checked By	Description
A		7/25/83	G	R.D. Salter	[illegible]	Ansys Prep 7, for Girder geometry, nodes & elements
B		7/25/83	G	R.D. Salter	[illegible]	Ansys static analysis of Crane Girder.

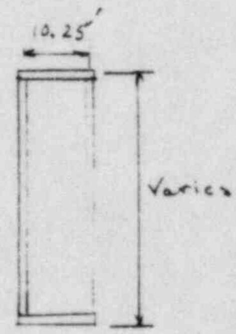
Computer Input

Materal A36
E = 29000 ksi



Type of Analysis - Linear Static (KAN=0)

Ansys Element Type - stif63



Typical cross section

Confirmation Req'd.

Loading

	Nodes	Load
Case I	164	48 k
	364	48 k
	122	39.5 k
	322	39.5 k
Case II	110	48
	310	48
	62	39.5
	262	39.5

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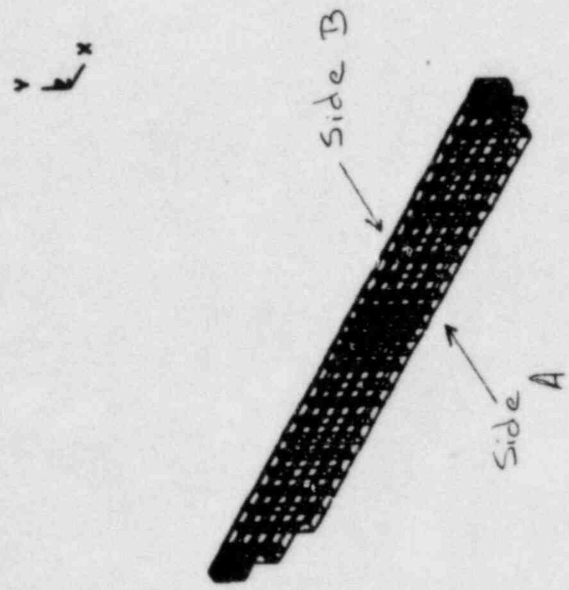
Computer Plot of Crane Girder

EPLT ANSYS 1



D3PLOT COMMAND-
1>

FPC AUX BLDG CRANE GIRDER

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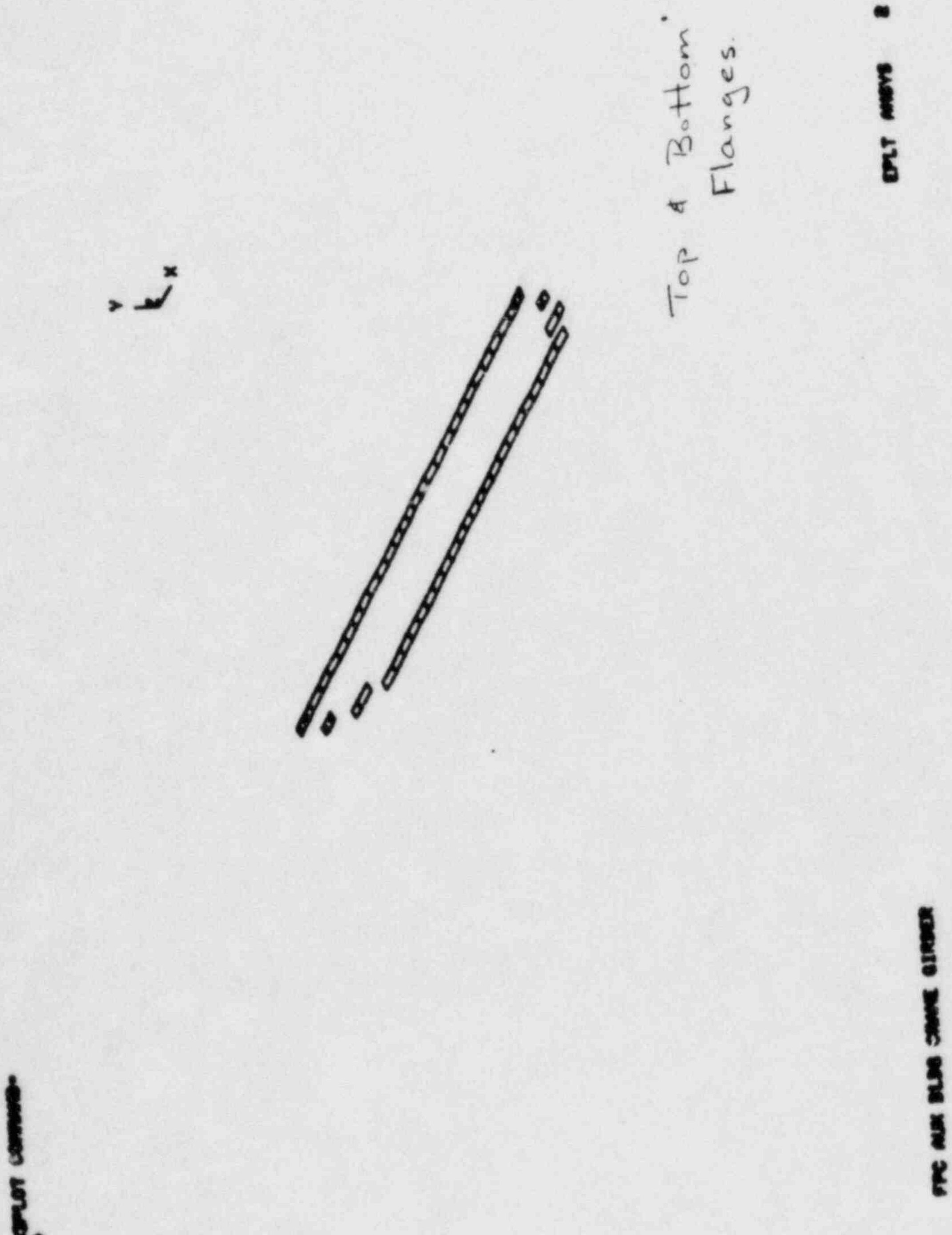
		QA CATEGORY/CODE CLASS I/NSR	
SUBJECT/TITLE FHCR-5 Crane Buckling Buckling Analysis			



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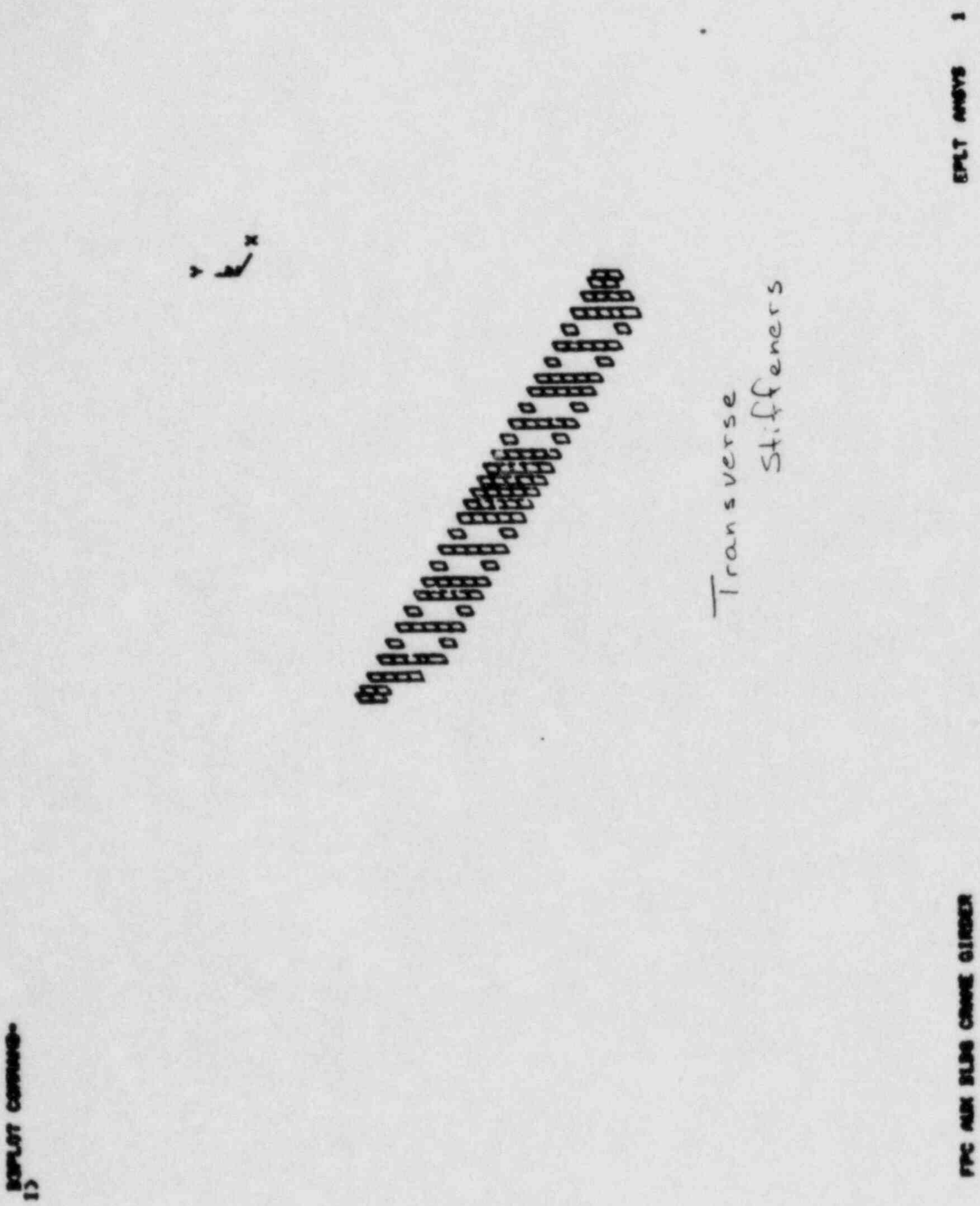

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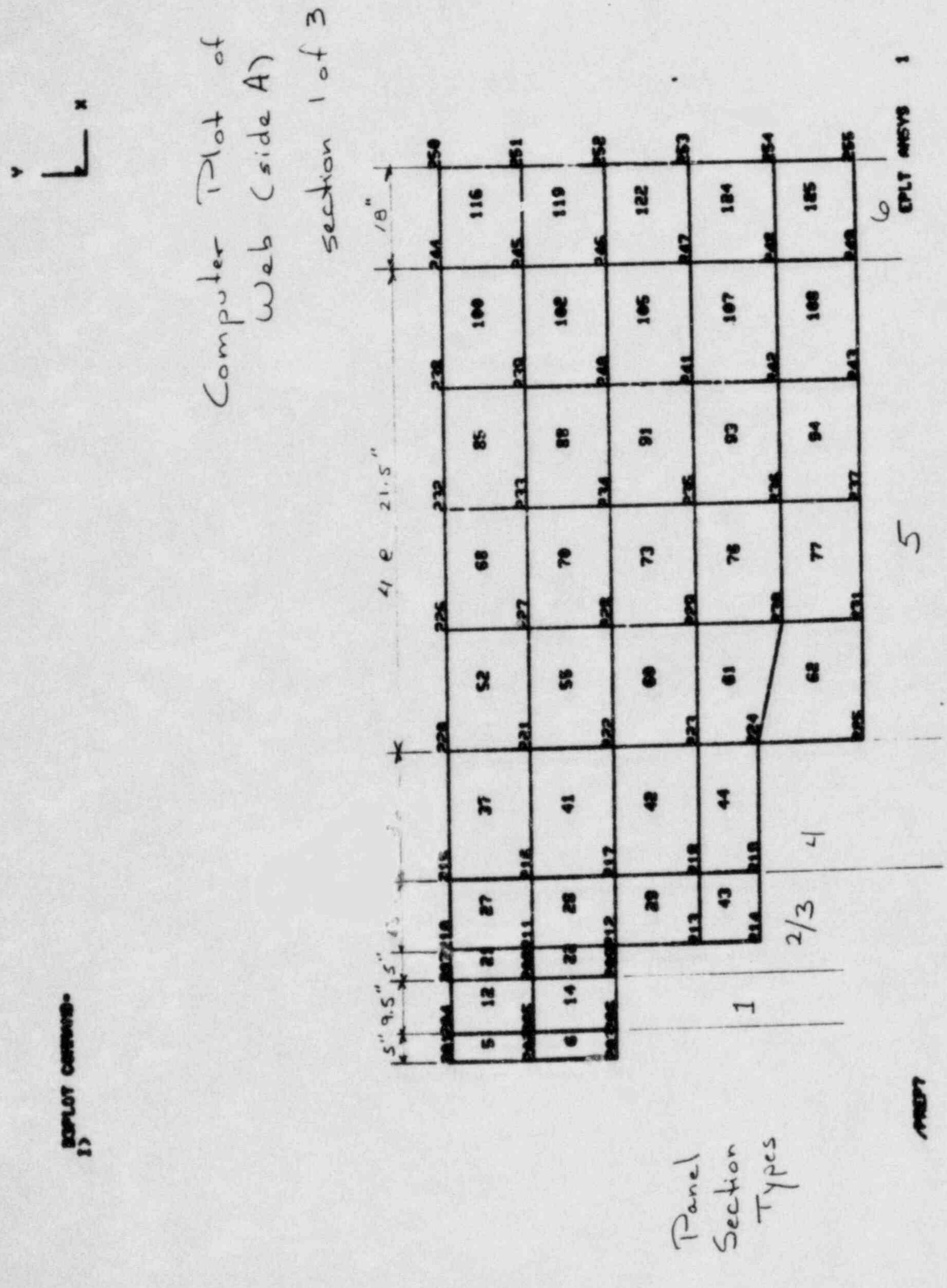
FHCR-7 Crane Girder Buckling Analysis	I/NSR



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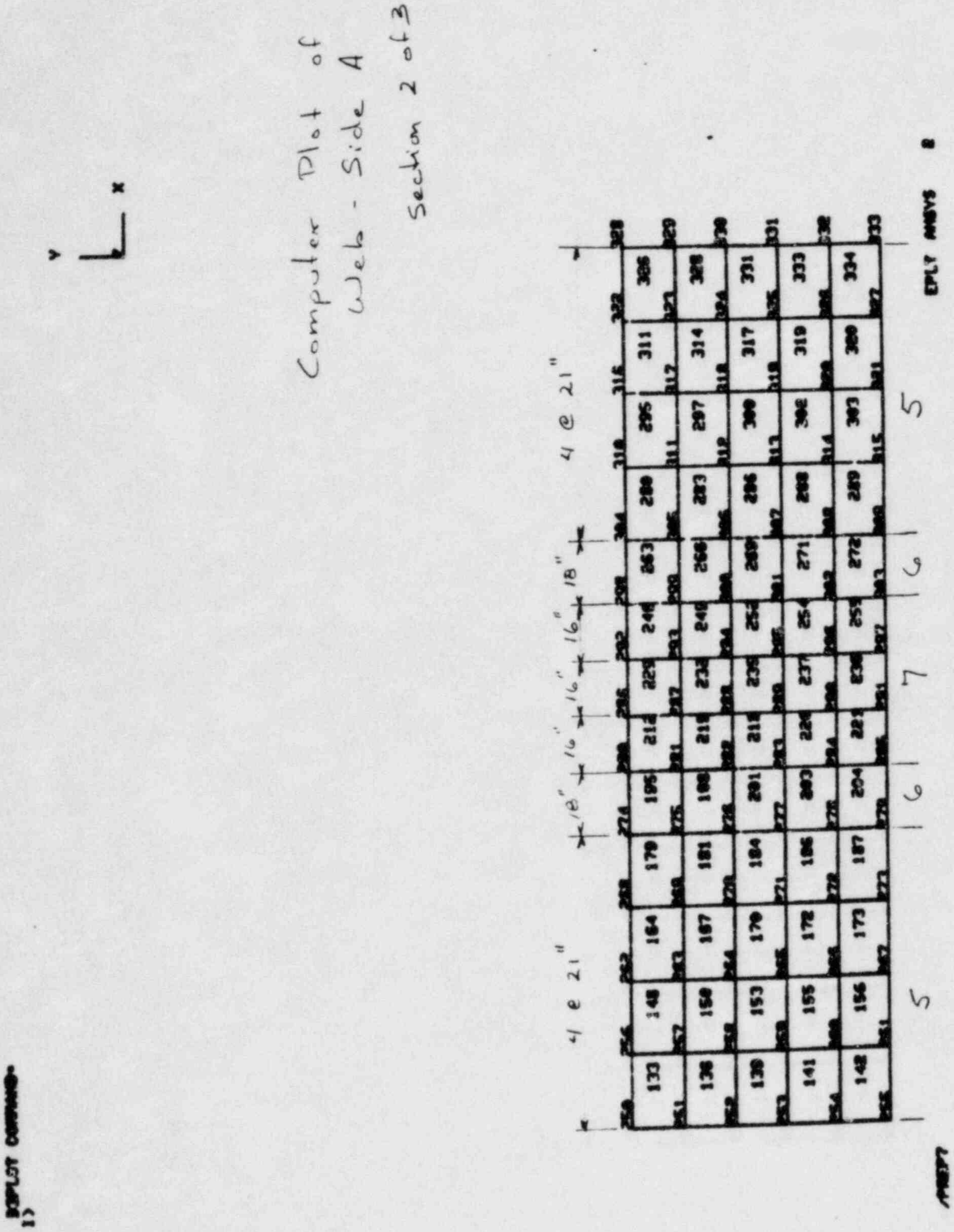

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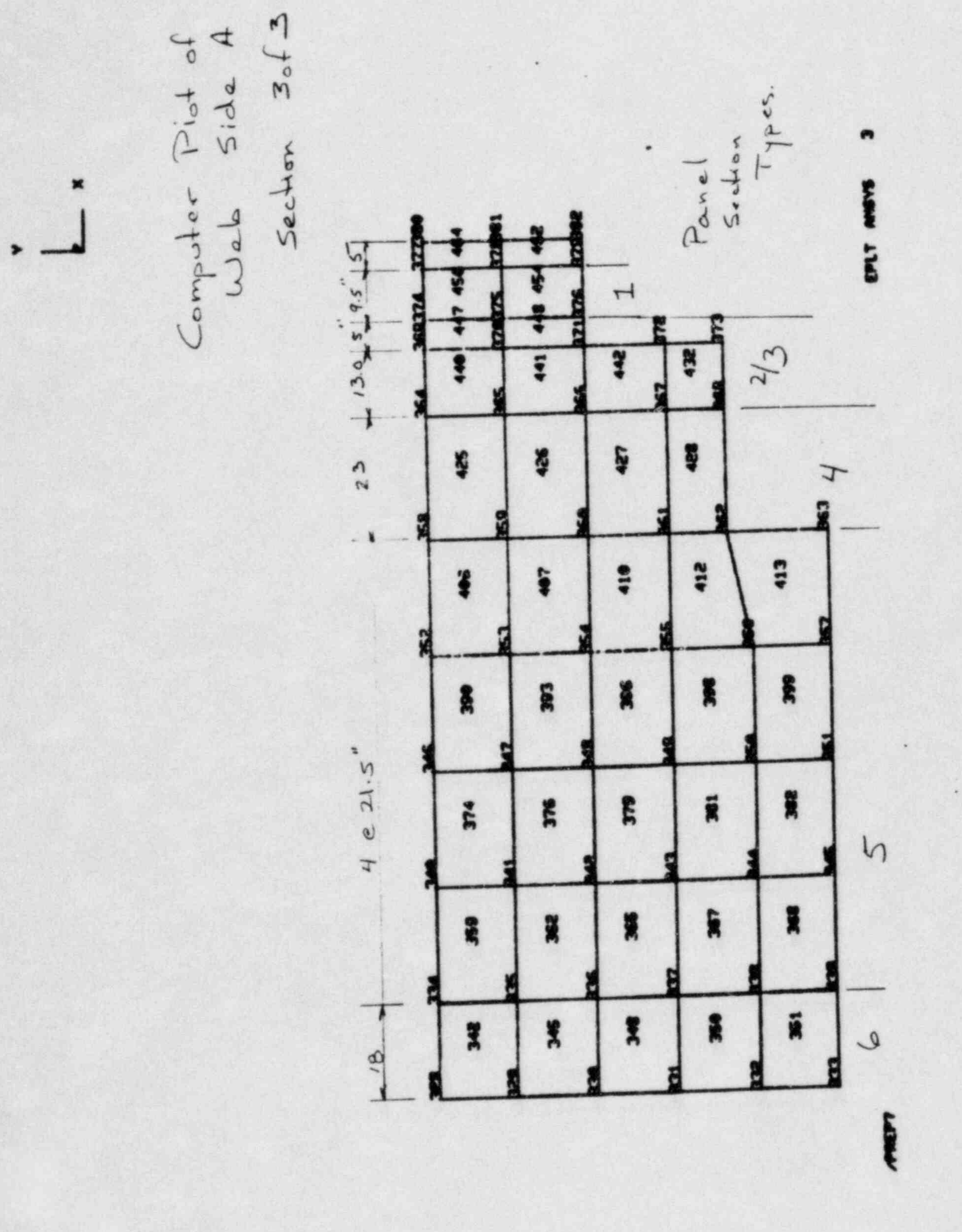

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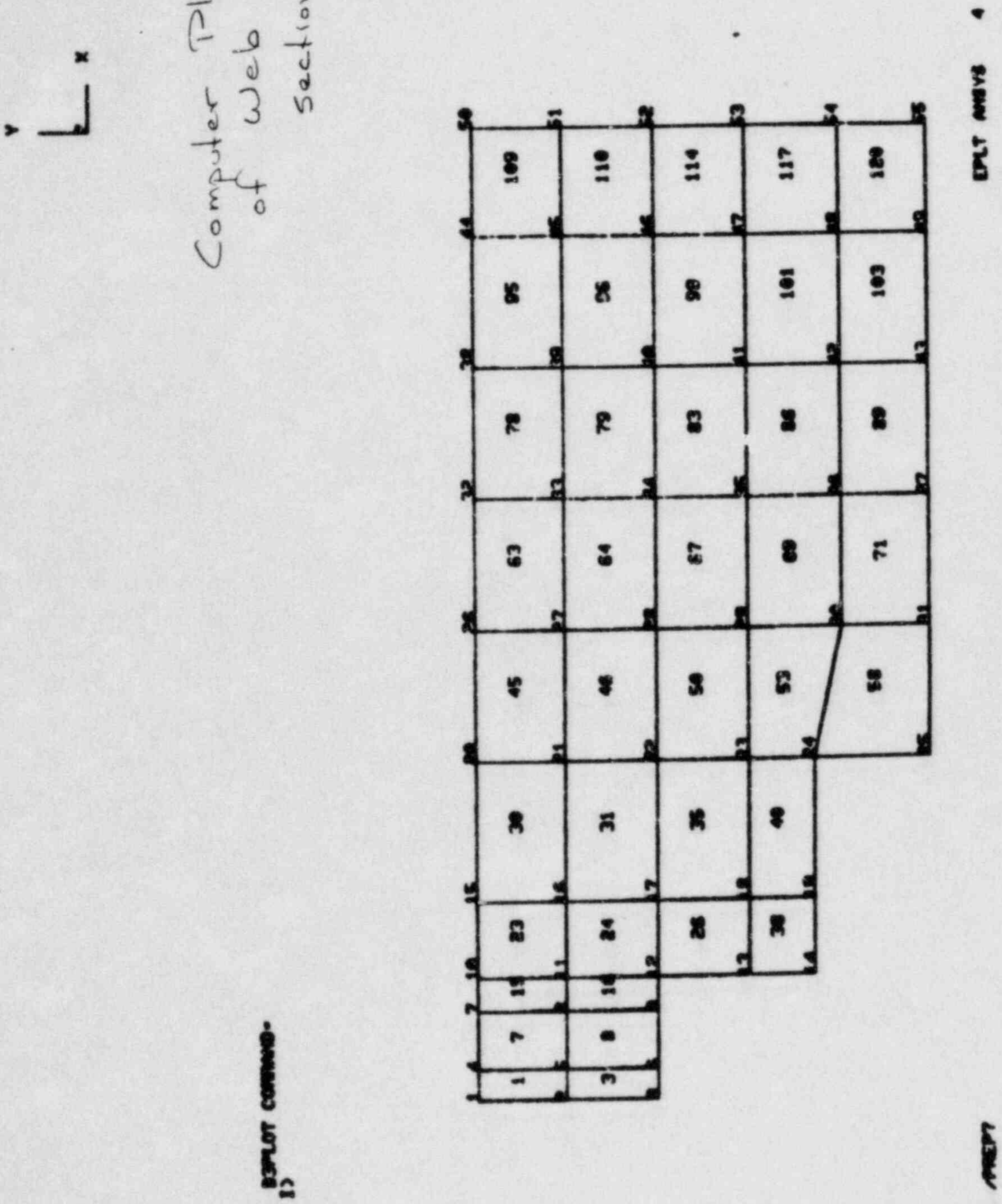
Computer Plot of Web Side B Section 1 of 3



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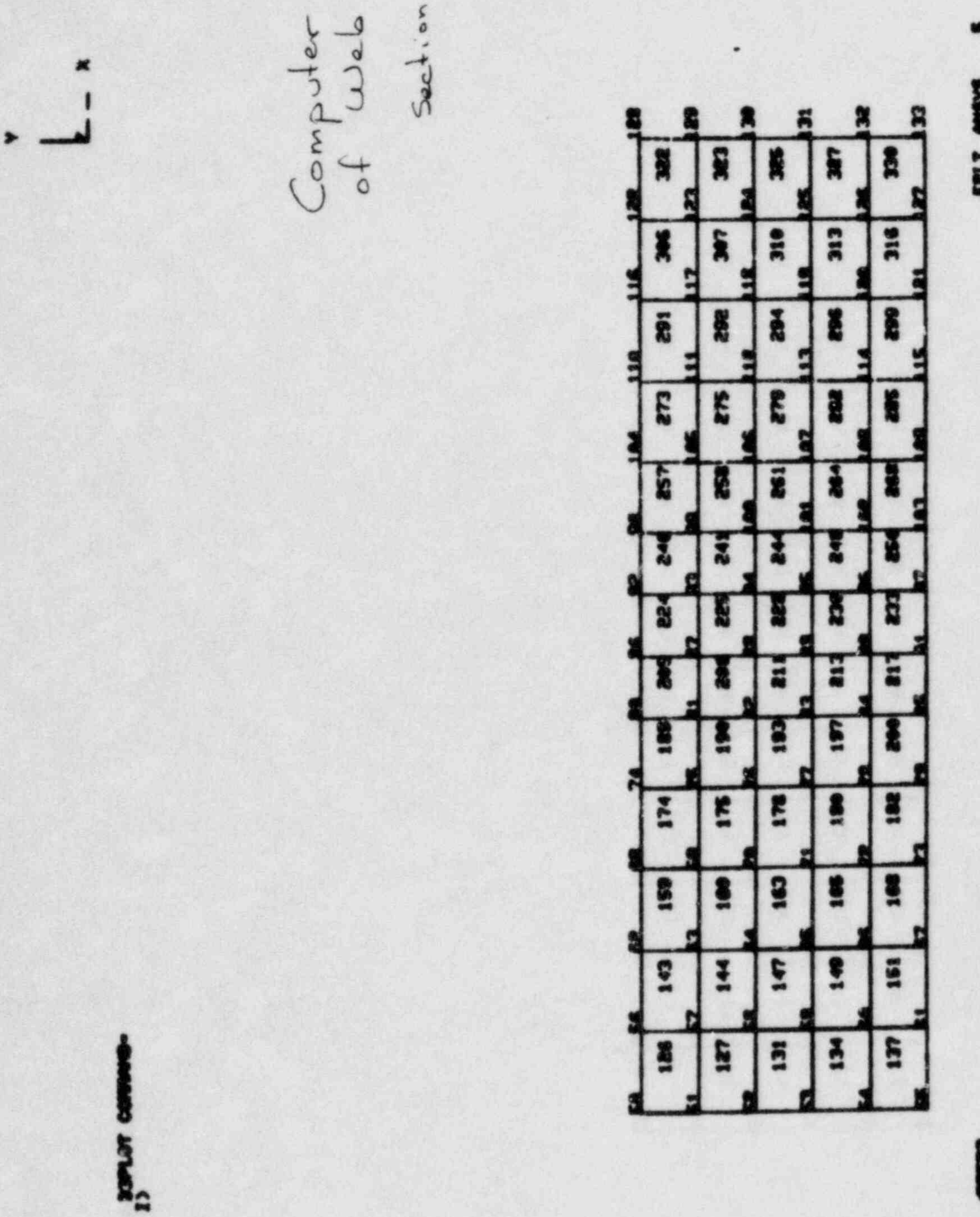
Computer Plot of Web side B
Section 2 of 3



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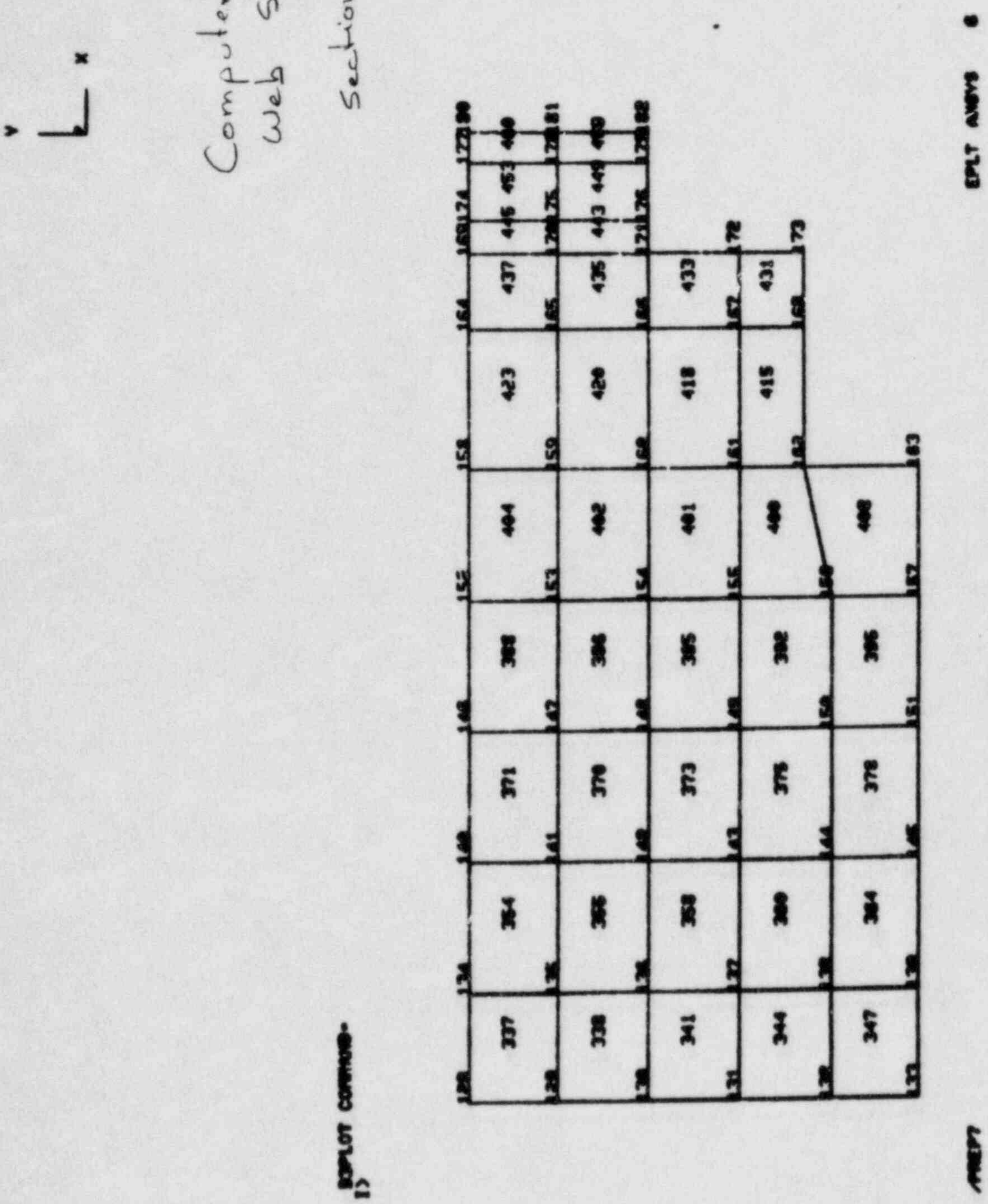
Computer Plot of Web Side B

Section 3 of 3



Element Stress Requirements for Crane Girder

Reference.

Required distance between flanges to web thickness ratio h/t (no stiffeners) (For Vertical Buckling) — AISC sec 1.10.2 & Salmon & Johnson page 567

no stiffeners $h/t \; \dfrac{14000}{\sqrt{F_y(F_y+16.5)}} = \dfrac{14000}{\sqrt{36(36+16.5)}} = 322$

with stiffeners (max spacing, $a = 1\tfrac{1}{2} \times h$.)

$$h/t = \frac{2000}{\sqrt{36}} = 333.$$

check depth-to thickness ratio of the webs — AISC Commentary page 5-131 & sec 1.10.5.3

$$d \le 260t = 260(5/16) = 81.25''$$

$$a/h \le \left[\frac{260}{(h/t)}\right]^2 = 1.17 \qquad \text{for } h = 75''$$

Shear — AISC 1.10.5.2

$$F_V = \frac{F_y}{2.89}(C_V) \le 0.4F_y \qquad (\text{eq } \# \, 1)$$

for

$C_V = \dfrac{45{,}000\,k}{F_y\,(h/t)^2}$ when $C_V < .8$ or $C_V = \dfrac{190}{h/t}\sqrt{\dfrac{k}{F_y}}$ for $C_V > .8$

$k = 4.00 + \dfrac{5.34}{(a/h)^2}$ for $a/h < 1.0$ or $k = 5.34 + \dfrac{4.00}{(a/h)^2}$ for $a/h > 1.0$

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Reference.

Shear (con't.)

Critical Shear — Salmon & Johnson pg 539

$$\tau_{CR} = \frac{\pi^2 E K}{12(1-\mu^2)(h/t)^2}$$

Bending in Plane of Web. — Salmon & Johnson page 541

$$F_{CR} = \frac{954(10^3)}{(h/t)^2}$$

$F_b = .6 F_{CR}$ Factor of saftey = 1.67 — AISC sec 1.5.1.4.4

Combined Bending & Shear — Salmon & Johnson page 542

$$\left[\frac{f_b}{F_{CR}\text{ (pure bending)}}\right]^2 + \left[\frac{v}{\tau_{CR}\text{ (pure shear)}}\right]^2 < 1$$

use only when $\tau_{actual} > .4\,\tau_{CRITICAL}$

Transverse Compression. — Salmon & Johnson page 543

$$F_{CR} = k_c \frac{\pi^2 E}{12(1-\mu^2)(h/t)^2}$$

where $k_c = \left[\frac{4}{(a/h)^2} + 5.5\right]$ — page 545

$$F_t = \frac{F_{CR}}{1.67}$$

Check Top & Bottom Flanges

Lateral Torsional Buckling.

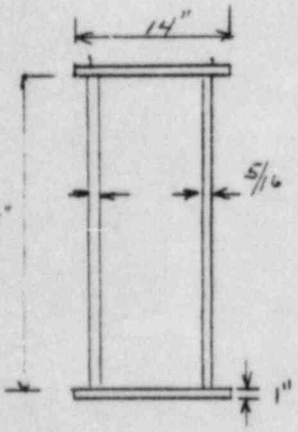
"Lateral torsional buckling need not be investigated for a box section whose depth is less than 6 times is width."

Reference: AISC 8th Sec 1.5.1.4.4

$d = 77 \qquad w = 14$

$6(14) = 84 > 77 \quad OK$

Bending Properties of Girder

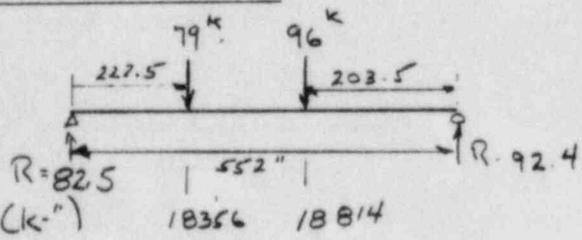


$A = 2(14) + 2(5/16)75 = 75.\ in^2$

$I_o = 2\left[\frac{5/16(75)^3}{12}\right] + 2\left[\left(\frac{75}{2}\right)^2 1(14)\right]$

$I_o = 21972 + 39375 = 61350$

Load Case 2



$f_{by} = \frac{Mc}{I} = \frac{18814(75)}{61350(2)} = 11.22\ ksi$

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SUBJECT/TITLE	QA CATEGORY/CODE CLASS
FHCR-5 Crane Girder Buckling Analysis	I/NSR

Reference.

Flange Compression Buckling & Allowable Bending Stress — AISC 8th sect. 1.9.2.2 subparagraph 3

$b/t \le \frac{190}{\sqrt{F_y}} = 31.6 > \frac{10}{1''}$ — " subparagraph 4

f_a = negligible $d/t = 75/\tfrac{5}{16} = 240 > \frac{640}{\sqrt{F_y}} = 106$ — AISC 8th

$b/t = \frac{238}{\sqrt{36}} = 39.6 > 10''$ — sec 1.5.1.4.4, sec 1.9

$\therefore \quad F_b = .6\,F_y = 21.6$ ksi

Check Web Crippling. (worst case is panel section 4) — AISC 1.10.10.2

Actual Applied.

$$f_{web\ cripp} = \frac{48^k}{\tfrac{5}{16}(23)} = 6.6^{ksi}$$

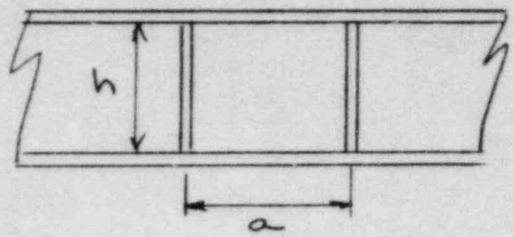
23" = maximum panel span for web crippling

Allowable Web Crippling stress.

$$F_{allow} = \left[5.5 + \frac{4}{(23/56)^2}\right]\frac{10{,}000}{(179)^2} = 9.11^{ksi} > 6.6^{ksi}$$

Summary of Allowable Stresses for Web

Constants: $t = 5/16''$
$E = 29000$ ksi
$F_y = 36$ ksi
$\mu = 0.3$



Section Type	h	a	a/h	h/t	Shear: k	Shear: C_v	Shear: F_v	Shear: σ_{CR}	Bending: F_b ksi	Transverse Compression: k_c	Transverse Compression: F_A
1	30	9	0.3	96	63.3	2.62	14.4	N.R.	21.6	49.9	21.6
2	30	18	0.6	96	18.8	1.43	14.4	N.R.	21.6	16.6	21.6
3	56	18	0.32	179	56.1	1.32	14.4	N.R.	21.6	44.5	21.6
4	56	23	.411	179	35.6	1.05	13.1	N.R.	21.6	29.2	21.6
5	75	43	.57	240	20.2	.443	5.51	9.2	9.94	17.7	8.05
6	75	18	0.24	240	96.7	1.30	14.4	N.R	9.94	75.0	21.6
7	75	16	0.21	240	125.1	1.45	14.4	N.R	9.94	96.2	21.6

N.R. - not required

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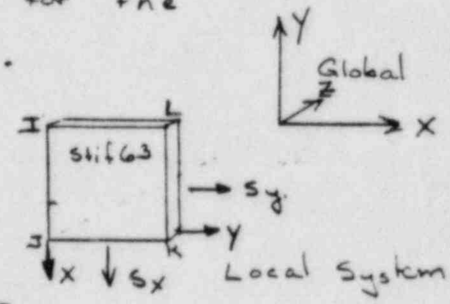
R. D. [illegible] 7/25/83	[illegible] 7/25/83	[illegible] 7/29/83

SUBJECT/TITLE	QA CATEGORY/CODE CLASS
FHCR-5 Crane Girder Buckling Analysis	I/NSR

Summary of Results.

The abbreviated list below represent the worst case sections of elements for the particular load case.

(UNITS: KSI)



Load Case 1

Section	Element #	Sx	Sy	Shear τ_{xy}
1	449	0.55	-0.4	2.6
	453	-0.71	0	1.8
	454	0.18	0.32	2.6
	456	0.48	0	1.8
2.	443	-9.6	1.08	-0.49
	445	-1.9	-1.0	-2.0
	447	-1.0	1.3	0.19
	448	-8.3	.0.83	0.29
3	431	1.0	0.83	-1.6
	433	4.8	1.1	-3.8
	435	-8.1	-1.7	-6.1
	437	-4.0	0	-4.1
	440	-3.8	0.33	-3.2
	441	-7.4	-1.4	-4.8
	442	3.2	0.7	2.7
	432	0.7	0.62	-1.1
4	425	-3.2	-1.6	0
	426	-1.2	-0.67	-0.68
	427	-.77	·0.56	-0.87
	428	.32	1.6	-1.3
	423	-3.2	-2.0	-0.29
	420	-1.0	-0.7	-0.8
	418	-0.6	0.8	-1.11
	415	0.7	2.0	-1.8

By Inpection the stress in section 5, 6, & 7 are minor

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R. D. Salter 7/25/83	[illegible] 7/25/83	[illegible] 7/29/83

SUBJECT/TITLE	QA CATEGORY/CODE CLASS
FHCR-5 Crane Girder Buckling Analysis	I/NSR

Load Case 2

(Units: ksi)

Section	Element #	Sx	Sy	Sxy
1	453(456)	-0.18	0.24	4.9
	449(454)	-2.77	-0.47	7.7
	7 (12)	-0.18	0.23	4.9
	8 (14)	2.78	-0.47	7.7
2	477(445)	-1.85	3.4	1.8
	448(443)	-12.7	-2.7	6.2
	21 (15)	-1.87	3.4	1.8
	22 (16)	-12.7	-2.7	6.2
3	440(437)	-1.4	2.7	-.51
	441 (435)	-9.9	-4.8	-5.9
	442 (433)	3.18	-1.9	-1.9
	432 (431)	0.8	1.1	-1.4
	27 (23)	-1.4	2.7	-0.55
	28 (24)	-10.0	-4.8	-5.9
	29 (26)	3.2	-1.9	-2.0
	43 (38)	0.8	1.1	-1.4
4	425(423)	-0.30	-0.68	-3.5
	426(420)	0.73	-2.6	-3.0
	427 (418)	0.17	-1.1	-1.5
	428 (415)	0.83	1.5	-1.7
	37 (30)	-0.30	-0.73	3.5
	41 (31)	0.75	-2.6	3.0
	42 (35)	0.19	-1.1	1.6
	44 (40)	0.87	1.6	1.7
5	273(280)	-3.57	-9.6	1.6
	285(289)	0.6	9.2	-0.4
	291 (295)	-3.62	-9.1	-2.84
	299 (303)	0.53	8.7	-0.85
	306(311)	-0.41	-7.8	-1.3
	316 (320)	0.52	7.8	-1.4
	359 (354)	-0.39	-4.7	-1.3
	368(364)	0.37	4.7	-1.5
	358(365)	0	0	2.2

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R. D. Salter 7/25/83	[signature] 7-25-83	[signature] 7/29/83

SUBJECT/TITLE	QA CATEGORY/CODE CLASS
FHCR-5 Crane Girder Buckling Analysis	I/NSR

Load Case 2 (con't)

Section	Element*	Sx ksi	Sy ksi	Txy ksi
	174 (179)	-0.5	-8.7	.29
	182 (187)	0.66	8.8	.24
5	159 (164)	-3.0	-8.8	-.93
	168 (173)	0.52	8.5	.46
	143 (148)	-2.9	-8.2	2.5
	151 (156)	0.48	7.9	1.16
	126 (133)	-0.35	-6.9	1.33
	137 (142)	0.5	6.9	1.4
	95 (116)	-0.4	-4.8	1.4
	103 (125)	0.4	4.8	1.6
6	257	-0.6	-9.23	0
	268	0.7	9.3	0.10
	189	-.78	-8.8	0.22
	200	0.77	8.9	0.11
	337	-.46	-5.7	-1.3
	347	-.47	5.7	-1.5
	109	-.50	-5.8	1.3
	120	.47	5.9	1.55
7	224	-.84	-9.0	.10
	233	0.9	9.1	.20
	240	-0.86	-9.0	.343
	250	0.86	9.2	.22
	205	-0.82	-8.9	.16
	217	-0.87	9.0	.15

Addition Stress Due to Dead Weight of Girder

Density of Steel = $.2833 \times 10^{-3}$ k/in³

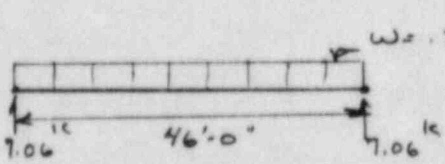
X-sec Area: 75 in²

Approx number of Stiffeners = 30 - say all are 75 × 10 × 3/8

wt of stiffeners = $30(75 \cdot 10 \cdot 3/8) \cdot .2833 \times 10^{-3} = 2.39^{k}$.

equivalent wt/ft for stiffeners = $\frac{2.39}{46'} = .052$ k/ft.

Total uniform dead load wt = $75(12)(.2833 \times 10^{-3}) + .052 = .307$ k/ft.



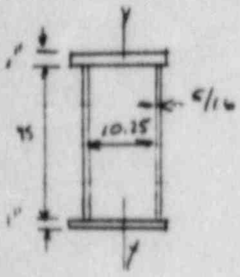
$M_{max} = 81.2^{k\text{-}ft} = 974^{k\text{-}in}$.

$V_{max} = 7.06^{k}$

% of Max Moment = $\frac{974^{k\text{-}"}}{18814^{k\text{-}"}} = .05$ or 5%
(see page 20)

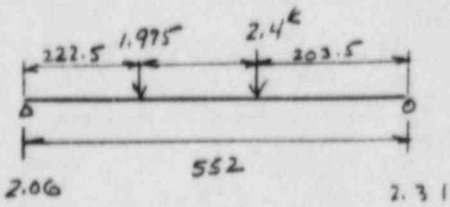
Maximum Compressive Stress on Web Due to Weak Axis.

For lateral loads on a Class A crane use 2½% of Applied Concentrated loads (CMAA #70)



$I = 2\left[\frac{1(4)^3}{12}\right] + \frac{75(10.875^3 - 10.25^3)}{12} = 1765$

$M_{max} = 470^{k\text{-}"}$



$S_{yy} = \frac{470(10.875)}{1765(2)} = 1.42^{ksi}$

Evaluation of Elements Stresses

(included a 5% increase for dead wt to all element stress plus and addition 1.42^{ksi} for weak axis compression in the web.)

Load Case 1

By inspection all of the element stresses for this load case are well below the allowable values for each of the sections.

Load Case 2 - By inspection accept all stresses but the following:

Section 5 elements

$F_b = 9.94^{ksi} \quad F_V = 5.51^{ksi}$

Combined shear & Bend $\left[\frac{f_b}{F_{CR}}\right]^2 + \left[\frac{f_v}{\tau_{CR}}\right]^2 \leq 1$

$\tau_{CR} = 9.2^{ksi}$

$F_{CR} = \frac{954(10)^3}{(240)^2} = 16.56^{ksi}$

Element 273(280)

$S_y = 1.42 + 1.05(9.6) = 11.5^{ksi} \qquad \tau_{factor} = 1.05(1.6) = 1.68^{ksi}$

$\underset{16.56}{F_{CR}} > \underset{11.5^{ksi}}{f_b} > \underset{9.94^{ksi}}{F_b}$ margin of saftey = 1.44 Accept.

Element 291(295) $S_y = 1.42 + 1.05(9.1) = 11.0 \qquad \tau_{xy} = 1.05(2.84) = 3.0^{ksi}$

Check Bending. $\underset{16.56}{F_{CR}} > \underset{11.00}{f_b} > F_b = 9.94^{ksi}$ margin of Saftey = 1.5 accept

check combined is not required $.4(9.2) = 3.68^{ksi} > 3.12^{ksi}$

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SUBJECT/TITLE	QA CATEGORY/CODE CLASS
F/HCR-5 Crane Guider Buckling Analysis.	I/NSR

Conclusions.

The results of the computer analysis show that the webs of the guiders are adequate against buckling without the longitudinal stiffeners. Although the comparison of element stress with allowable stresses show that the second load case has a small margin of acceptance (1.44 minimum) but these critical elements and stresses are highly localized and close to the flanges. The allowable stresses represent overall average cross-sectional stresses and don't take into consideration that the stresses will vary across the section.

The first load case shows some high shear stresses at the supports but these are well below the allowables.

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SUBJECT/TITLE	QA CATEGORY/CODE CLASS
FHCR-5 Crane Girder Buckling Analysis	I/NSR

List of Attachments

1.) Check calculation for Independent Reviewer Load case 3

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STONE & WEBSTER ENGINEERING CORPORATION

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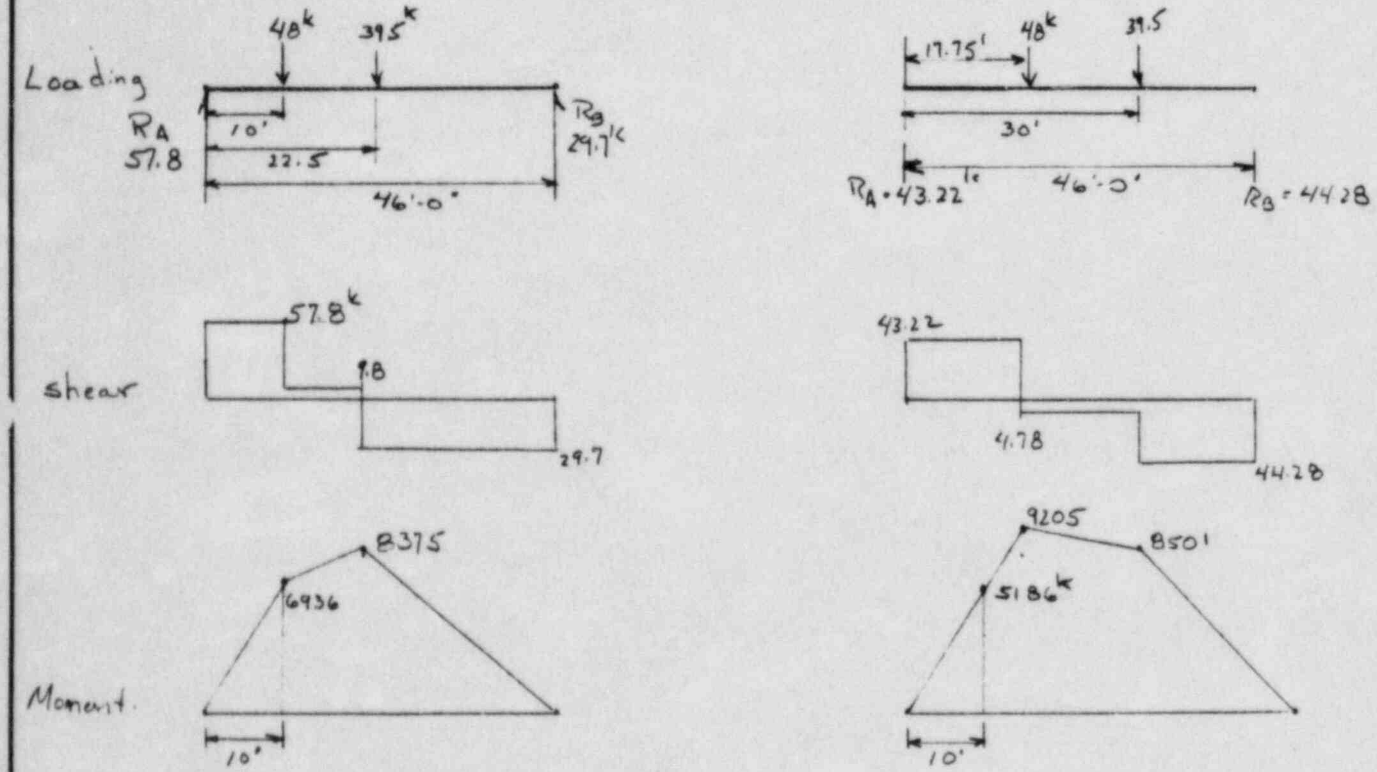
ALTernate check for R SALter calc "position of LOAD"	

ATTACHMENT 1
CALC. NO. C01
JO 14235.17
PAGE 1 OF 4

Load Case 3

Comparison of Load Case 2 & 3

Case 3 Case 2



A comparison of the above two load cases shows that a section between the point loads of load case two (load location for maximum load on the girder.) and the support can have a higher combination of shear and compressive stress than provided by load case two.

In order to locate the worst section for combined moment and shear

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SUBJECT/TITLE	QA CATEGORY/CODE CLASS
Alternate check of SALTERO Calc "Position of Load"	

a strudl computer run was done (computer run C) which provides a set of influence lines using 5 ft sections along the girder.

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CALC. NO. C03
JO 14235.17
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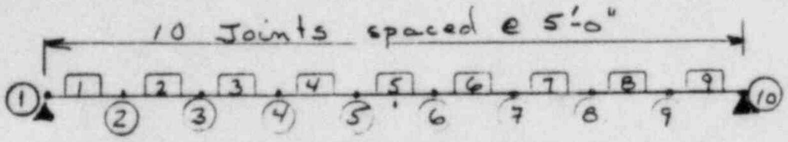
Strudl Model



The 48ᵏ and 39.5ᵏ loads are rolled across the girder and the force on each of the members is calculated for each of the load locations.

Loading Steps

#	48^k @ Joint	39.5^k @ Joint
2	1	3
3	2	4
4	3	5
5	4	6
6	5	7
7	6	8
8	7	9

10 - Dead load of Girder.

Results of Strudl Influence lines.

A comparison of the Influence lines with the shear and moment diagram on page 1 of the attachment shows that member 2 under load step 3 could produce a higher combination of shear and moment

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Alternate check of SACTES code "position of load"	

ATTACHMENT 1
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on this section than was considered in the first two original load cases. This load case is more accurately shown on page 1 of this attachment and shall be considered therein as load case 3.

Due to the similarity of load step 5 & 6 of the strudl model with that of load case 2 the member forces and moments where neglected.

Elements Stresses for load Case 3

Inorder to check the elements stress for this section the stresses provided in the Ansys Run load case 2 will be proportion up by the differences between the calculated values shown of page 1 of this attachment.

$$\% \text{ increase for Moment} = \frac{6936-5186}{5186} \times 100 = 34\%$$

$$\% \text{ increase for shear} = \frac{57.8-43.22}{43.22} = 34\%$$

Therefore increase all shear and normal stresses by 34% in the element between 5 & 15 feet from the support.

ATTACHMENT 1
CALC. NO. C03
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The elements in consideration are list below:

Section	Elements	Load Case 2 Sx	Load Case 2 Sy	Load Case 2 τ_{xy}	Load Case 3 Sx	Load Case 3 Sy	Load Case 3 τ_{xy}
	326(322)	-.524	-6.66	-1.27	0.7	-8.9	-1.7
	328 (323)	0	-3.17	-2.11	0	-4.24	-2.83
5	331 (325)	0	-.30	-2.28	0	-.402	-3.05
	333 (327)	0	3.18	-2.11	0	4.26	-2.83
	334 (330)	.524	6.77	-1.49	0.70	9.07	-2.00
	342 (337)	-.46	-5.7	-1.35	-.616	-7.6	-1.8
6	345 (338)	-.21	-2.9	-1.9	-.28	-3.9	-2.5
	348 (341)	0	-.15	-2.2	0	-.201	-2.9
	350 (344)	0.2	2.7	-2.1	0.27	3.6	-2.8
	351 (347)	0.47	5.7	-1.5	0.63	7.6	-10.2
	359 (354)	-.39	-4.7	-1.3	.52	6.3	1.7
	362 (355)	-.13	-2.3	-1.9	.17	3.1	2.5
5	365 (358)	-0.0	0.0	-2.17	0	0	2.9
	367 (360)	0.0	2.3	-2.1	0	3.1	2.8
	368 (364)	.37	4.7	-1.55	.50	6.3	2.08
	374 (371)	-.28	-3.7	-1.26	.37	5.0	1.69
	379 (373)	-.124	.186	-2.22	.17	0.25	2.97
	382 (378)	.244	3.56	-1.6	.33	4.77	2.14
	390 (388)	-.15	-2.7	-1.4	.20	3.6	1.9
	396 (385)	0.22	.48	-1.81	.30	0.64	2.412
	399 (395)	-.72	2.4	-1.9	1.0	3.2	2.5
	406 (404)	0.29	-1.7	-2.0	0.4	2.3	2.68
	410 (401)	1.1	-.20	-1.6	1.5	0.27	2.1
	413 (408)	1.0	2.2	2.22	1.34	3.0	2.97

Conclusion.

The stress produced in the above element are not as critical as those shown it Load case 2 for similar section pa els.

<u>ENCLOSURE 2</u>

EVALUATION OF THE INTAKE GANTRY
CRANE TO CMAA-70 AND ANSI B30.2.0

J.O. No. 14235.17

EVALUATION OF THE INTAKE GANTRY CRANE TO CMAA-70 AND ANSI B30.2.0

CRYSTAL RIVER - UNIT NO. 3

FLORIDA POWER CORPORATION

REVISION 0, June 28, 1983

STONE & WEBSTER ENGINEERING CORPORATION

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1.00 INTRODUCTION

The Crystal River Unit No. 3 Intake Gantry Crane was designed in 1970 and built in 1971. The NRC's letter of December 22, 1980, Enclosure 3, Paragraph 2.1-3.e requires verification that the crane design complies with the guidelines of CMAA Specification 70, (dated 1975) and Chapter 2-1 of ANSI B30.2.0-1976, Both of these documents were published after the design for the Intake Gantry Crane was established.

The manufacturer's industry specification to which this crane was designed to is EOCI #61, dated 1961. The national safety standard to which the crane was designed to is USAS B30.2.0 dated 1967.

Section 2.00 of this report compares CMAA-70 with EOCI-61, and specifies the extent of compliance of the Crystal River Intake Gantry Crane with CMAA-70. Section 3.00 of this report compares Chapter 2-1 of ANSI B30.2.0 with Chapter 2-1 of USAS B30.2.0, and specifies the extent of compliance of the Crystal River Intake Gantry Crane with ANSI B30.2.0.

Section 4.00 of this report summarizes the findings of the Intake Gantry Crane comparison to CMAA-70 and ANSI B30.2.0. Section 5.00 privides Stone & Webster's recommendations with regard to these findings.

2.00 CMAA vs EOCI AND THE CRYSTAL RIVER CRANE

2.01 General

The CMAA No. 70 specification is basically a rewrite of the EOCI #61 Specification. Much of the criteria is therefore the same or very similar, with some areas becoming less stringent. Additionally, the CMAA specification addresses areas of crane design that are considered standard in the industry but were not addressed in the original EOCI document.

In addition to the EOCI document, the Intake Gantry Crane was designed to the criteria of the Whiting Crane Company a member of the CMAA organization, and to the technical requirements of the procurement specification, No. RO-2872. With the addition of these requirements, the design of the Intake Gantry Crane in many cases surpasses the requirements of CMAA-70.

Areas of major difference between the CMAA and EOCI documents are listed under paragraph 2.02, with a description of these differences and the extent of Intake Gantry Crane compliance to CMAA being covered under paragraph 2.03.

2.02 Paragraph Listing of Major Differences in Industry Specifications.

Topic	CMAA Paragraph No.	EOCI Paragraph No.
a. Material Requirements	3.1	16.A
b. Welding	3.2	17.A
c. Impact Allowance	3.3.2.1.1.3	18.B.1.a(3)
d. Laterial Forces	3.3.2.1.2	18.B.1.b
e. Torsioral Forces	3.3.2.1.3.2&3	18.B.1.c(2)&(3
f. Bending Stress	3.3.2.2	18.B.2
g. Longitudinal Stiffeners	3.3.3.1.1	18.C.1.a
h. Allowable Compressive Stress	3.3.3.1.3	18.c.1.b
i. Fatigue Considerations	3.3.3.1.3	(not covered)
j. Hoist Rope Requirements	4.2.1	24.A
k. Drum Design	4.4.1	26.A
l. Drum Groove Design	4.4.3.1&2	(not covered)
m. Gear Design	4.5.2	27.
n. Bridge Brake Design	4.7.2.2	31.A.1.b
o. Hoist Brake Design	4.7.4.2	31.B.1
p. Bumpers	4.12	(not covered)
q. Static Control Systems	5.4.6	(not covered)
r. Undervoltage/Restart Protection	5.6.2&3	38.A

2.03 Comparison of Differences & Extent of Crane Compliance.

a. Material Requirements

CMAA-70 specifies ASTM-A36 as the basic structural steel. EOCI-61 specifies ASTM-A7 as the basic structural steel.

The Intake Gantry Crane complies with CMAA-70 in that ASTM-A36 steel was used.

b. Welding

CMAA-70 specifies welding to be in accordance with AWS D14.1. EOCI-61 specifies welding to be in accordance with AWS recommendations.

The Intake Gantry Crane complies with CMAA-70 in that welding was in accordance with the Whiting crane company procedures which in turn meet the requirements of AWS D14.1.

c. Impact Allowance

CMAA-70, requires that crane design calculations include an impact allowance of 0.5% of the load per foot per minute of hoisting speed but not less than 15%. EOCI-61 specifies only a minimum allowance of 15%. Therefore, for cranes with hoist speeds in excess of 30 feet per minute, it is possible that the impact allowance applied under EOCI-61 will be less than that required by CMAA-70.

The Intake Gantry Crane complies with CMAA-70 in that the maximum hoist speed is 13 fpm, resulting in an impact loading condition of 15%.

d. Lateral Forces

CMAA-70 specifies the lateral load due to acceleration or deceleration to be 2½% the live load and crane bridge (exclusive of end trucks and ties) for Class A cranes and 5% for Class B, C and D cranes. Additionally, CMAA-70 specifies the lateral load, due to wind, to be 5 pounds per square foot of projected bridge girder area, multiplied by 1.6 when the distance between exposed girder surfaces is greater than the

depth of the girder.

EOCI-61 specifies the lateral load due to acceleration or deceleration to be 5% of the above loads, with the laterial load, due to wind, being 10 pounds per square foot of projected girder area.

The Intake Gantry Crane complies with CMAA-70 in that the requirements of EOCI are more conservative than CMAA. Additionally, this crane was designed for a lateral wind loading of 30 pounds per square foot in the non-operating condition.

e. Torsioral Forces

CMAA-70 specifies that the following forces causing twisting moments; overhanging loads and lateral forces acting eccentric to the horizontal neutral axis, be multiplied by the distance from the force to the shear center of the girder section. EOCI-61 requires that these forces be multiplied by the distance from the force to the center of gravity of the girder.

The Intake Gantry Crane complies with CMAA-70 in that there are no substantial over hanging loads, and in that the box girders are symmetrical about the neutral axis. Note, that the loads imposed by the 7,700 lb bridge walkway are distributed over the cranes 105 ft. girder length resulting in a negligible twisting moment.

f. Bending Stress

CMAA-70 requires that the combined bending stress include a wind load of 5 pounds per square foot for outdoor cranes. EOCI-61 does not address the addition of this wind load for outdoor cranes.

The Intake Gantry Crane complies with CMAA-70 in that the girder structural calculations include a wind force of 10 pounds per square foot in the operating mode and a wind force of 30 pounds per square foot in the non-operating mode.

g. Longitudinal Stiffeners

CMAA-70 gives criteria for the h/t web plate ratio based upon the number of longitudinal stiffeners used in the compression area of the web plate, either none, one or two.

EOCI-61 gives a slightly different criteria for the h/t web plate ratio, but only for the case where longitudinal stiffeners are not used in the compression area of the web plates.

The Intake Gantry Crane has been supplied with one longitudinal stiffener, as indicated below, complying with the requirements of CMAA-70.

h(depth of web) = 71"   t(thickness of web)= 5/16"

Per CMAA-70:

The h/t ratio of the web plate, when provided with transverse stiffeners or diaphragms is limited by the use of longitudinal stiffeners, as follows:

The h/t ratio of the web shall not exceed:

$$\frac{h}{t} = C(k + 1)\sqrt{\frac{17.6}{f_c}}$$ nor shall it exceed M

Where: The coefficients C and M are as tabulated below:

Longitudinal stiffeners	C	M
None	81	188*
One	162	376
Two	243	564

h/t	$C(k + 1)\sqrt{17.6/fc}$	M	No. of Stf. by CMMA-70	No. Stf. Provided
227	324	376	1	1

Note that the most conservative appreach has been used for a symmetrical girder, which is the case where the maximum stresses are assumed to equal the basic allowable stresses. Using this approach the CMAA-70 $c(k + 1)\sqrt{17.6/fc}$ equation governing the longitudinal web plate sitffeners reduces to 2C, which is the smallest possible value for this equation.

h. Allowable Compressive Stress

CMAA-70 requires a reduction in the maximum allowable compressive stress (17,600 psi for A36 steel) if the b/c ratio of the girder exceeds 38.

EOCI-61 requires a similar reduction in the maximum allowable compressive stress (16,000 psi for A7 steel) however, this reduction is required only if the b/c ration exceeds 41.

The Intake Gantry Crane complies with CMAA-70 in that A36 steel was used with a maximum allowable compressive stress of 17,600, and in that b(distance between web plates) = 21½ inches, and c(thickness of top cover plate) = 3/4 inch, resulting in a b/c of 28.7.

i. Fatigue Considerations

CMAA-70 provides substantial guidance with respect to fatigue failure by indicating allowable stress ranges for various structural members in joints under repeated loads. EOCI-61 does not address fatigue failure.

The Intake Gantry Crane inherently complies with CMAA-70 since the number of heavy lifts made by the crane is far less than 20,000 and therefore, fatigue is not a consideration.

j. Hoist Rope Requirements

CMAA-70 requires that the rated capacity load plus the bottom block, divided by the number of parts of rope, not exceed 20% of the published rope breaking strength. EOCI-61 requires that only the rated capacity load divided by the number of parts of rope not exceed 20% of the published rope breaking strength.

The Intake Gantry Crane complies with CMAA-70 in that it has 8 parts of 1 inch 6 x 37 fiber center hoisting rope. Per the Third Edition of The Whiting Handbook, the breaking strength of one part of rope is 39.8 tons. With a safety factor of five and eight parts of rope, the reeving system capacity is 63.7 tons. The hoists rated capacity is 50 tons.

k. Drum Design

CMAA-70 requires that the drum be designed to withstand combined crushing and bending loads. EOCI-61 requires only that the drum be designed to withstand maximum bending and crushing loads with no stipulation that these loads be combined.

The Intake Gantry Crane complies with CMAA-70 in that the drum design was based upon a combination of crushing and bending loads.

l. Drum Groove Design

CMAA-70 provides a recommendation for the minimum drum groove depth and minimum drum groove pitch. EOCI-61 provides no similar guidance.

The Intake Gantry Crane complies with the recommendations of CMAA-70 as indicated below:

Hoisting Rope Diam(in)	Drum Groove Depth (in)	Drum Groove Pitch(in)	CMAA-70 Recommended Min. Depth (in)	CMAA-70 Recommended Min. Pitch(in)
1	13/32	1 1/8	12/32	1 1/8

m. Gear Design

CMAA-70 requires that gearing horsepower ratings be based on certain American Gear Manufacturers Association (AGMA) standards and provides a method for determining allowable horsepower. EOCI-61 addresses gears, however similar design guidance is not provided.

The Intake Gantry Crane complies with CMAA-70 in that the cranes gearing was based upon the AGMA standards referenced in CMAA-70.

n. Bridge Brake Design

CMAA-70 requires that bridge brakes, for cranes that are cab controlled with cab on trolley, have a torque rating of at least 75% of the bridge motor. EOCI-61 has a similar requirement, except that the torque rating of the bridge brake is only a minimum of 50% of the bridge motor.

The Intake Gantry Crane inherently complies with CMAA-70 in that the crane is pendant controlled only, and has no cab.

o. Hoist Brake Design

CMAA-70 requires that hoist holding brakes, when used with a method of control braking other than mechanical, have torque ratings no less than 125% of the hoist motor torque. When used with a mechanical braking system, or if two holding brakes are used, the brake torque rating is to be no less 100% of the hoist motor torque

EOCI-61 specifies hoist braking, but only implies that the braking torque be at least equal to the motor torque.

The Intake Gantry Crane complies with CMAA-70 in that the hoist brake has a torque rating of at least 100% of the motor torque, and is used in conjuction with a mechanical l d brake.

p. Bumpers

CMAA-70 provides substantial guidance as to when bridge and trolley bumpers are required, and specifies some specific design requirements. EOCI-61 makes no mention of bumpers.

The Intake Gantry Crane complies with CMAA-70 in that both the bridge and the trolley are provided with double acting spring bumpers. Due to the slow speed of the bridge (75 fpm) and the trolley (50 fpm) the design of these spring bumpers meet the deceleration rate and energy absorbtion capacity specified in CMAA-70.

q. Static Control Systems

CMAA-70 provides substantial guidance for the use of static control systems. EOCI-61 makes no mention of static controls and primarily gives requirements for magnetic controls.

With regard to static controls, the Intake Gantry Crane inherently complies with CMAA-70, since such controls were not used. The controls for this crane are A-C magnetic reversing.

r. Undervoltage/Restart Protection

CMAA-70 requires that cranes not equipped with spring-return controllers or momentary contact pushbuttons be provided with a device that will disconnect all motors upon power failure and will not permit any motor to be restarted until the controller handle is brought to the "OFF" position, or a reset switch or button is operated. Undervoltage protection is required if spring-return controllers or momentary contact push buttons are not used.

EOCI-61 requires undervoltage protection for all control arrangements and requires momentary contact pushbuttons for floor controlled pendant operated cranes. EOCI-61 however, does not address spring-return controllers or the requirement for restart protection.

The Intake Gantry Crane complies with CMAA-70 in that momentary contact pushbuttons were provided for pendant operation. In addition, this crane was also provided with an undervoltage mainline magnetic contractor utilizing a start button for resetting.

3.00 ANSI B30.2.0 VS USAS B30.2.0 AND THE CYRSTAL RIVER CRANE

3.01 General

The ANSI B30.2.0 - 1976 safety standard is basically a rewrite of the USAS B30.2.0-1967 safety standard. Much of the criteria is therefore the same or very similar, with some areas becoming less stringent.

In addition to the USAS document, the Intake Gantry Crane was designed to the criteria of the Whiting Crane Company, a member of the CMAA organization, and to the technical requirements of the procurement specification, No. RO-2872. With the addition of these requirements, the design of the Intake Gantry Crane in many areas surpasses the requirements of ANSI.

Areas of major difference between the ANSI and USAS documents are listed under paragraph 3.02, with a description of these differences and the extent of Intake Gantry Crane compliance to ANSI being covered under pragraph 3.03.

3.02 Paragraph Listing of Major Differences in Safety Standards

Topic	ANSI Paragraph No.	USAS Para. No.
a. Clearances	2-1.2	2-1.2
b. Welded Construction	2-1.4.1	2-1.3.2
c. Girders	2-1.4.2	2-1.3.4
d. Cab Clearances	2-1.5.1.c	2-1.4.1b
e. Toeboards & Handrails	2-1.7.3	2-1.6.3
f. Ladders	2-1.7.4c	2-1.6.4c
g. Egress from Cab	2-1.7.5	(not covered)
h. Bridge & Trolley Bumpers	2-1.8	2-1.7
i. Electrical Equipment	2-1.10.1a	2.1.9.1a
j. Hoisting ropes	2-1.11.2a	2-1.10.2a
k. Hooks	2-1.11.4	2-1.10.4

3.03 Comparison of Differences & Extent of Crane Compliance

a. Clearances

ANSI B30.2.0 does not specify clearance dimensions. USAS B30.2.0 specifies a 3" overhead and 2" laterial clearance from obstructions and a 4" clearance between parallel cranes.

The Intake Gantry Crane complies with the less specific ANSI document as well as with the USAS document.

b. Welded Construction

ANSI B30.2.0 specifies welding to be in accordance with AWS D14.1. USAS B30.2.0 specifies welding to be in accordance with AWS D2.0-66.

The Intake Gantry Crane complies with the ANSI document in that welding was in accordance with the Whiting crane company procedures which in turn meet the requirements of AWS D14.1

c. Girders

ANSI B30.2.0 specifies the girder design to be in accordance with either CMAA #70, AISC, or AISE Std. #6. USAS B30.2.0 specifies the girders to be of adequate design.

The Intake Gantry Crane complies with the ANSI document, in that allowable stresses and main design features of the girders comply with CMAA-70.

d. Cab Clearances

ANSI B30.2.0 does not specify specific clearance dimensions. USAS B30.2.0 specifies that an operators cab have at least 3" clearance from all fixed structures.

The Intake Gantry Crane inherently complies with the less specific ANSI document since this crane does not have an operator's cab.

e. Toeboards & Handrails

ANSI B30.2.0 specifies that toeboards and handrails be in accordance with ANSI 12.1. USAS B30.2.0 specifies that these items be in accordance with USAS A14.3-1956.

The Intake Gantry Crane complies with the intent of the ANSI document in that toeboards and handrails have been provided and meet the major requirements of ANSI A12.1.

f. Ladders

ANSI B30.2.0 specifies that ladders be in accordance with ANSI 14.3 which inturn requires cages on ladders greater than 20 ft. USAS B30.2.0 specifies that ladders be in accordance with USAS A14.3-1956.

The Intake Gantry Crane complies with the ANSI document in that the crane's access ladder is caged.

g. Egress from Cab

ANSI B30.2.0 recommends a means of egress from the operator's cab for emergencies. USAS B30.2.0 does not address emergency cab egress.

The Intake Gantry Crane inherently complies with the ANSI document since this crane does not have an operator's cab.

h. Bridge & Trolley Bumpers

ANSI B30.2.0 requires the use of bumpers on other automatic means of equivalent effect if the bridge or trolley operates near the end of its travel. The trolley bumpers are also required to have an energy absorbing capacity to stop the trolley when traveling at least 50% of rated speed. USAS B30.2.0 only recommends the use of bumpers.

The Intake Gantry Crane complies with the ANSI document in that both the bridge and trolley are provided with double acting spring bumpers designed to provide the required energy absorbtion.

i. Electrical Equipment

ANSI B30.2.0 specifies that wiring and equipment comply with ANSI C-1, Article 610 (NFPA 70) which inturn requires a 30" walkway width in front of electrical enclosures. USAS B30.2.0 specifies that wiring and equipment comply with Article 610 of USAS C1-1965.

The Intake Gantry Crane meets the intent of the ANSI document with regard to electrical wiring and equipment. However, the walkway width is 30 inches total, and therefore a clearance of 30 inches in front of all electrical enclosures may not be provided.

j. Hoisting Ropes

ANSI B30.2.0 requires selection of hoisting ropes based on the rated load plus the weight of the load block. USAS B30.2.0 requires selection of hoisting ropes based only upon the rated load.

The Intake Gantry Crane complies with the ANSI document in that the hoisting cables have sufficient breaking strength to accomodate the rated load plus the weight of the load block with a safety factor of at least five.

k. Hooks

ANSI B30.2.0 requires safety latches unless the application makes the use of the latch impractical. USAS B30.2.0 does not address hook safety latches.

The Intake Gantry Crane complies with the ANSI document in that a hook safety latch is provided.

4.00 CONCLUSIONS

The Crystal River Unit No. 3 Intake Gantry Crane complies with the major design requirements of the industry specification CMAA-70. This crane also complies with the safety standards of ANSI B30.2.0 - 1976, except that some electrical enclosures may not have 30 inches of clearance in front of the enclosure.

5.00 RECOMMENDATIONS

The only areas of non compliance involve the ANSI B30.2.0 requirement on clearance in front of electrical enclosures.

The clearance in front of electrical enclosures is not an item which affects the safe handling of loads, and therefor no change is recommended.

6.00 REFERENCES

6.01 Standards and Specifications

a. CMAA Specification #70 (CMAA-70), 1975 Crane Manufacturers Association of America "Specifications for Electric Overhead Traveling Cranes".

b. EOCI Specification #61 (EOCI-61), 1961, Electric Overhead Crane Institute "Specifications for Electric Overhead Traveling Cranes".

c. ANSI B30.2.0-1976, American National Standards Institute, Safety Standard "Overhead and Gantry Cranes".

d. USAS B30.2.0-1967, United States of America Standards Institute, Safety Code "Overhead and Gantry Cranes"

e. Gilbert Associates' Specification RO-2872, February 18, 1970 "Requirement Outline Intake Gantry Crane, Crystal River-Unit No. 3"

f. Whiting Corporation Engineering Standard L703, February 20, 1970, "Drum Design".

6.02 Letters

Nuclear Regulatory Commission letter, December 22, 1980, "To All Licensees of Operating Plants and Applicants for Operating Licenses and Holders of Construction Permits" Subject: Control of Heavy Loads.

6.03 Handbooks

Whiting Crane Handbook, Third Edition, April 1967 Whiting Corporation (Table 12-Weight And Strength of Wire Ropes - 6 x 37, page 81)

6.04 Drawings

Whiting Drawing No.	Whiting Drawing Title
U-65088	Girder Detail
U-64400	Gen'l Arrgm't of a 2 motor Gantry Bridge
U-64401	Gen'l Arrgm't of a 2 motor Gantry Bridge
U-52976	Gen'l arrgm't of a 2 Motor RW Trolley

ENCLOSURE 3

LOAD DROP ANALYSIS FOR SFHT-7

CALCULATION SHEET

A5010.61

STONE & WEBSTER ENGINEERING CORPORATION

J.O./W.O./CALCULATION NO.	REVISION	PAGE
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SUBJECT/TITLE	QA CATEGORY/CODE CLASS
CRYSTAL RIVER 3 LOAD DROP ANALYSIS	I / NSR

REFERENCES

1) Proceedings of the ASCE, Journal of the Structural Division. Vol 105 № ST3. March 1979. p. 547
"Structural Response of R/C Slabs to Tornado Missles"
by P. McMahon, S. Sen, B. Meyers, K. Buchert

2) USNRC Standard Review Plan. NUREG-0800
Appendix A

3) "Theory of Plates and Shells", by Timoshenko 2nd Edition
McGraw Hill Book Co.

4) ACI 349 APPENDIX C.
"SPECIAL PROVISIONS FOR IMPULSIVE AND IMPACTIVE LOADS"

5) "Yield Line Analysis of Slabs" by L.L. Jones & R.H. Wood
American Elsevier Publishing Co. Inc N.Y 1963

6) "Theory and Analysis of Plates" by Rudolph Szilard
Prentice Hall, Inc. Englewood Cliffs, N.J 1974

7) "Building Code Requirements for Reinforced Concrete" ACI 318-77
American Concrete Institute, Detroit Michigan

8) "Formulas for Stress & Strain" 5th Edition by R. Roark & W Young
McGraw Hill Publishers N.Y. N.Y.

9) NUS Report 3874, 1981 "The Control of Heavy Loads at Crystal River 3, NUREG 0612, Six Month Report

10) Stone & Webster Engineering Corp Computer Program ST-331
"Single Barrier Mass Missle Impact" Version 00 Level 00

11) Stone & Webster Engineering Corp. Topical Report SWECO 7703
"Missle - Barrier Interaction"

12) "Structural Analysis and Design of Nuclear Plant Facilities"
American Society of Civil Engineers 1980

13) "Introduction to Structural Dynamics" by John M. Biggs
McGraw Hill Book Co. NY. 1964

14) GAI DWG SC-421-019 REV 6

15) GAI DWG SC-400-015 REV 4

CALCULATION SHEET

5010 61

STONE & WEBSTER ENGINEERING CORPORATION

J.O./W.O./CALCULATION NO.	REVISION	PAGE
K4235.17/FPC/C01	0	1.1

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SUBJECT/TITLE	QA CATEGORY/CODE CLASS
CRYSTAL RIVER LOAD DROP ANALYSIS	I NSR

REFERENCES (CONT.)

16) "DESIGN OF STRUCTURES FOR MISSILE IMPACT" BECHTEL TOPICAL REPORT BC-TOP-9A REV 2.

ASSUMPTIONS

1. Initial velocity of missle when dropped is zero
2. The missle strikes the target normal to the surface.
3. Any intermediate targets are ignored. Primary target takes full impact.
4. No crushing of the missle is used
5. The load may be dropped at any location in the crane travel area except where physical interference is present
6. If drag forces are present, they may be considered.
7. Lift height is assumed to be the maximum that is physically possible, unless otherwise stated
8. The most critical condition based on target failure is analyzed. All other conditions, if not considered, are less critical.
9. Analysis is based on a bilinear elastic-plastic curve that represents a true stress/strain relationship
10. Initially the effects of existing loads and deflections are ignored until the target passes the drop analysis. Upon passing, the effects of existing loads & deflections will be checked if their effect is critical
11. Compression steel is ignored in determining Ultimate Moment Capacity
12. Concrete Slab and beam boundary conditions are based on available information and conservative assumptions
13. Concrete slabs and beams are assumed to deform plastically at failure, and seperate into segments at the yield lines.
14. Failure is considered when the ductility ratio exceeds that as stated in Ref 2
15. If the energy absorbtion analysis indicates the target is acceptable, a localized check for perforation, scabbing penetration, and spalling will be done.

ASSUMPTIONS

16. Structural member checked as a column with $Kl/r \leq 22$ length effects are ignored, axial loads & deflections only are checked

17. Columns will first be checked ignoring length effect

18. Passing columns will again be checked with length effects if it is critical

19. Initial deflection to be considered with load drop deflections will include dead load and any permanantly fixed load such as equipment. Earthquake, creep, etc. deflections will be neglected.

20. A combination of structures may be present to resist the load drop, however to simplify analysis only one structure may be analyzed

21. When more than one block assembly exists (ie main & auxiliary hook) the more critical load shall be incorporated into the analysis. U.O.S.

22. Structural Steel supporting concrete slabs shall not be included in developing barrier resistance

23. FPC/CR3 FSAR Section 5.4.5.1 states that any vertical seismic response is assumed to be insignificant. Therefore no induced accelerations other than gravity are considered and all load drops will be assumed to have zero velocity at the time of the drop.

24. The ratio $n = E_s/E_c$ is assumed to remain constant during impact & resistance, any increase in E_c due to dynamic increase factor is assumed negligable

METHOD OF ANALYSIS

As presented in Ref.#1, the effective mass-plastic impact method is used in analysing the response of target structures to missles 'dropped' from cranes.

This method evaluates an effective mass for the structural barrier and treats the impact as a plastic collision between the missle and the effective mass. The strain energy of the target at maximum response is used to balance the residual kinetic energy of the target-missle combination.

The impact is modeled as a missle of mass, M_m, and striking velocity, V_s, impacting a spring-backed target mass M_e. The spring is bilinear and a function of the resistance-displacement properties of the target.

For plastic collisions with short duration impacts the target displacements and spring forces are small during impact and can conservatively be neglected. Therefore the missle and target masses attain the same velocity at the end of impact. The strain energy required to stop the target-missle combination is then the sum of the kinetic energy of the missle and the target masses at the end of the duration of impact.

$$E_s = \frac{M_m V_m^2}{2} + \frac{M_e V_t^2}{2}$$

Conservation of momentum gives the velocities of missle M_m & M_e after impact

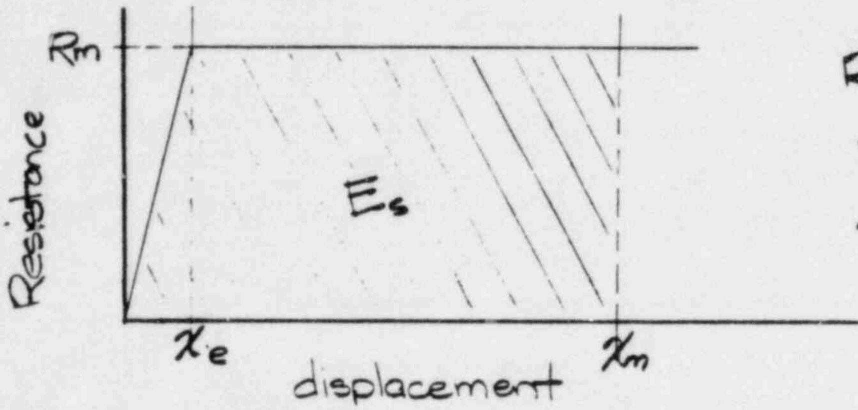
$$M_m V_s = (M_m + M_e) V'$$

(Eq. 1) $$V' = \frac{M_m V_s}{M_m + M_e}$$

Substituting equation 1 into the Energy Equation above gives the required target strain energy of

(Eq. 2) $$E_s = \frac{M_m^2 V_s^2}{2(M_m + M_e)}$$

The target strain energy is defined as the area under the bilinear resistance/displacement curve for the structural target as shown below.



R_m = Plastic resistance

x_e = Yield displacement

x_m = Max. displacement

For elastic/plastic response (without concurrent loads)

$R_{(x)} = Kx \qquad (0 < x \leq x_e)$ K: Elastic Spring Constant: $\frac{R_m}{x_e}$

$R_{(x)} = Kx_e = R_m \quad (x_e < x \leq x_m)$

Therefore the target strain energy is:

$$E_s = R_m(x_m - x_e/2)$$

(Eq. 3) $$x_m = \frac{E_s}{R_m} + \frac{x_e}{2}$$

The corresponding ductility ratio (Ref 1)

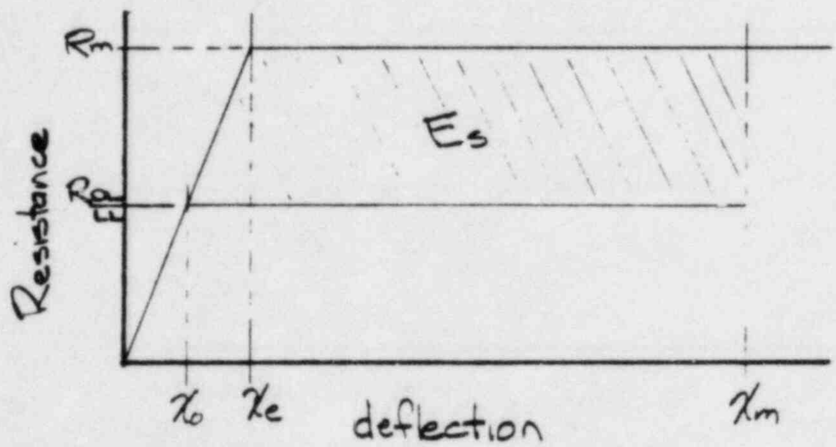
(Eq. 4) $$\mu_n = \frac{x_m}{x_e} = \frac{E_s}{x_e R_m} + \frac{1}{2}$$

This ductility ratio must be less than the limits imposed by Ref. 2

If the ductility ratio of the structural target exceeds the allowables as presented in Ref. 2, then no further analysis is required. The target is insufficient for the crane drop.

If the ductility ratio of the structural target is less than the allowable as presented in Ref 2. a check of existing dead load and live load deflections along with missle deflections must be performed.

$$x' = x_e - x_o$$



$x_o \equiv$ DL/LL deflection

The target strain energy is defined as the area below the Resistance displacement curve but above the dead/live load resistance.

Therefore the target strain energy is:

$$E_s = \tfrac{1}{2}K(x_e - x_o)^2 + K(x_e - x_o)(x_m - x_e)$$

Thus the maximum deflection is:

$$x_m = \frac{E_s}{K(x_e - x_o)} + \frac{x_e + x_o}{2}$$

The ductility ratio

(Eq. 5) $$\mu = \frac{E_s}{R_m(x_e - x_o)} + \frac{1 + x_o/x_e}{2}$$

Load drop target structures that are qualified by allowable ductility ratio for overall structural response will be checked for localized damage to ensure the target's localized capacity for resisting the load drop. (applicable to slabs and beams)

Localized effects include.

Perforation: missle passes through the target
Scabbing: ejection of material from the back face of target
Penetration: displacement of the missle into the target
Spalling: ejection of material from the front face of target

Scabbing effect will be the controlling localized condition because scabbing occurs before perforation; and penetration and spalling pose no risk to systems on the back side of target structure. As per Ref 12 p 318 Punching Shear is implicit in the formulas for penetration, scabbing, etc.

The method of analysis will be that of Ref 11 where the scabbing threshold velocity is calculated and compared to the impact velocity of the load drop. A threshold velocity greater than the impact velocity indicates that scabbing will not occur at that impact velocity and therefore it can be concluded that no localized damage will occur. Acceptable alternate formulas are: The Modified NDRC Formula and The Bechtel Formula as presented in Ref 12

T = Barrier thickness (in)
D = load diameter   equivalent diameter $= \sqrt{\frac{4\,Area}{\pi}}$
t = missle wall thickness
m = mass of missle

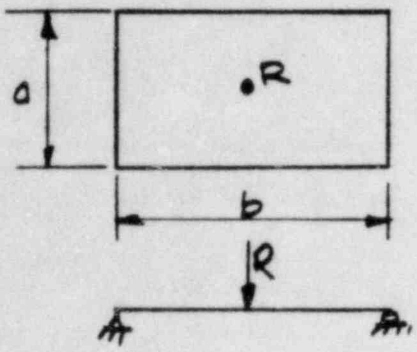
$\frac{2t}{D} = 1.0$ for solid missle

Procedure for checking scabbing:

1. calculate $T^{(in)}/D^{(in)}$
2. calculate $2t^{(in)}/D^{(in)}$
3. using T/D & $2t/D$ in Figure B.3-1 Ref. 11 find KE/T^3 (ksi)
4. calculate KE : $\left[KE = KE/T^3 \times T^3\right]$
5. calculate threshold velocity $V = \sqrt{\frac{KE \times 2}{m}}$
6. Compare threshold velocity with calculated impact velocity
7. If threshold velocity > impact velocity scabbing will not occur
   If threshold velocity < impact velocity reduce height of load drop

SLABS:

simple supports



RESISTANCE[K] Ref5

$$R = 2\pi M_u^+ (DIF)$$

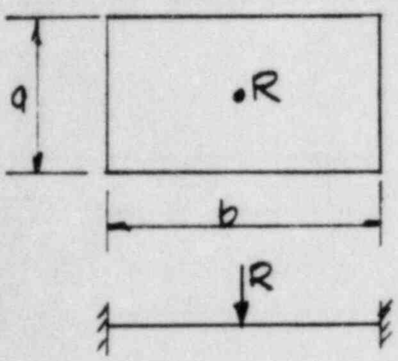
DISPLACEMENT[FT]

$$x_e^{(H)} = C_1 \frac{R a^2}{E I_e}(1-\nu^2)$$

b/a	1.0	1.2	1.4	1.6	1.8	2.0	∞
C_1	0.01120	0.0129	0.0138	0.0142	0.0144	0.01444	0.0145

Ref6

fixed support



RESISTANCE[K] Ref5

$$R = 2\pi (M_u^+ + M_u^-)(DIF)$$

DISPLACEMENT[FT]

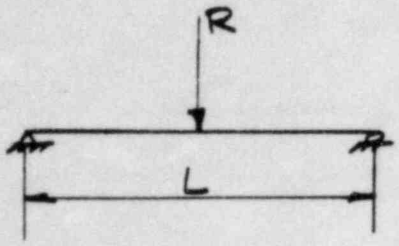
$$x_e^{(H)} = C_1 \frac{R a^2}{E I_e}(1-\nu^2)$$

b/a	1.0	1.2	1.4	1.6	1.8	2.0	∞
C_1	0.00560	0.00647	0.00691	0.00712	0.00720	0.00722	0.00725

Ref6

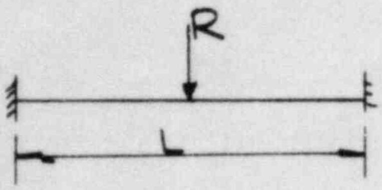
M - FT.K
R - K
E - K/FT²
I - FT⁴/FT

Concrete Beams



$$R = \frac{4M_u^+}{L}(DIF)$$

$$x_e = \frac{RL^3}{48EI}$$



$$R = \frac{4(M_u^+ + M_u^-)}{L}(DIF)$$

$$x_e = \frac{RL^3}{192EI}$$

COLUMNS

Concentrically loaded, short column (height ≤ 3 thickness)

$$R = 0.70\left[0.85 f'_c (A_g - A_s) + A_s f_y\right]$$

$$x_e = \frac{R L}{A_c E_T + A_s E_s}$$

Stone & Webster computer program "Single Based Mass - Missle Impact" ST-331, Version 00, Level 00, will be used to check various "target" capabilities for postulated crane load drops.

The computer analysis approach is similar to the mass-plastic impact method as presented in Ref. 1.

As a means for comparing the two methods a trial check will be done to compare the results of a hand calculation and the computer run.

The trial run will be modeled as follows:

slab: $10^{ft} \times 20^{ft}$, 2^{ft} thick, #11 bars @ 6" EWEF TOP & BOTTOM

$f'_c = 4^{ksi}$, $f_y = 60^{ksi}$, fixed on all sides

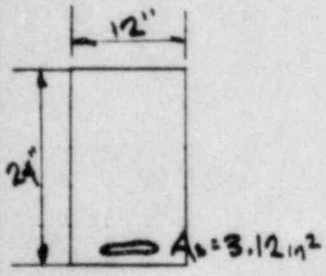
missle: point load, 100^k weight, 10^{ft} drop strikes @ center

$$mass = 100^k / 32.2'/s = 3.1 \frac{ks^2}{'}$$

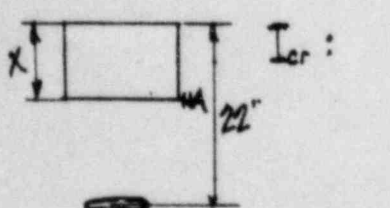
Mass-Plastic Impact Method

Effective Moment of Inertia: based on an effective thickness

REF.
D = depth to tens. steel
I_{cr} = cracked mom of inertia
I_g = gross mom of inertia



$$I_g = \frac{1}{12}(12)(24)^3 = 13824 \, in^4 =$$



I_{cr}:

$$\frac{12x^2}{2} - 24.96(22 - x) = 0$$

$$6x^2 + 24.96x - 549 = 0$$

$$x = 7.7^{in}$$

$$I_{cr} = \frac{1}{3}(12)(7.7)^3 + 24.96(22-7.7)^2 = 6930\,in^4$$

$$I_e = \tfrac{1}{2}(I_{cr} + I_g) = 10377\,in^4 = 0.5004\,ft^4 \qquad \text{(Ref. 4)}$$

Ultimate Moment Capacity of 12" section of slab

$$M_u = \phi A_s f_y (d - a/2) \qquad a = \frac{A_s f_y}{.85 f'_c b} = \frac{3.12(60)}{.85(4)(12)} = 4.59\,in$$

$$M_u = 0.9(3.12)(60\,ksi)(22 - 4.59/2)$$

$$M_u = 3320\,^{\prime\prime K} = 276\,^{ftK}$$

Effective Target Mass : Use 1/6 Volume of Concrete in failure cone as per SBMMI ST-33 (1/6 is the ... of Mass Factor in Ref 13 pgs 2.13, 2.14. Engineer will determine if less conservative Me should be used)

$$M_e = \frac{0.150\,k/ft^3}{6(32.2)} \times \pi(5')^2 \times 2' = 0.1219\,\frac{k\cdot s^2}{ft}$$

Peak Resistance of slab 'R' Ref 5 p 266

$$R = 2\pi(M^+ + M^-)$$

$$R = 4\pi(276\,^{'K}) = 3468^K$$

DYN. INCR FACT = 1.1 $\qquad R = 3815^K$ as per Ref. 4

Yield Displacement $\quad x_e = C_1 \frac{Ra^2}{D}$ Ref 6 p. 654

$$x_e = 0.00722\,\frac{3815^K (10')^2\; 12}{(5.184\times10^5\,k/ft^2)(0.5004\,ft)}(1-.25^2)$$

[Use $\frac{I_e}{ft}$ in place of h^3 for 12]

$$x_e = 0.120\,in = 0.010\,ft$$

IMPACT ANALYSIS

Velocity of Missle:

$$V_m = \sqrt{2gh} = \sqrt{2(32.2)(10)} = 25.4\ '/s$$

Required Target Strain Energy

$$E_s = \frac{M_m^2 V_m^2}{2(M_m + M_e)}$$

$$E_s = \frac{(3.1)^2 (25.4)^2}{2(3.1 + .1219)} = 962^{\prime k}$$

Ductility Ratio

$$\mu_n = \frac{E_s}{x_e R_m} + \frac{1}{2}$$

$$\mu_n = \frac{962^{k}}{0.010^{2}(3815.5)} + \frac{1}{2}$$

$$\mu_n = 25.7$$

Comparison with Computer Run Attachment #1 shows a very good comparison between the hand calculation and the computer run. Therefore it can be concluded that the computer will give an analysis similar to the hand calculation presented here.

Attachment #1 pg 1 of 2
14235.17-C-01
FPC CR1
PREP. R. Shelburger 5/16/83
REV R. D. S. 6/13/83
IR: K Walsh 7/6/83

```
IN    ECHO
          1         2         3         4         5         6         7         8
12345678901234567890123456789012345678901234567890123456789012345678901234567890
TEST RUN FOR THEORTICAL APPROACH---COMPARISON FOR HAND CALCULATION
0.
1.0       3815.     0.010     3815.0    0.10      3815.0    0.50      3.925
0.        0.0       0.        0.0
0.0       0.0       0.0       78.74     100.0
STOP
```

ST-331,SBMMI,VERS-00 LEVEL-00 78.172 13.53.27

TEST RUN FOR THEORTICAL APPROACH---COMPARISON FOR HAND CALCULATION

****

DATA ON MISSILE, BARRIER, AND LOAD COMBINATION EQUATION

****

BARRIER FORCE DISPLACEMENT RELATIONSHIP

KIPS	FEET
3815.0	0.0100
3815.0	0.1000
3815.0	0.5000

0.0 KIPS EQUIVALENT STATIC FORCE ** LOAD 1
0.0 KIPS EQUIVALENT CONSTANT DYNAMIC FORCE ** LOAD 2
0.0 KIP-SEC MISSILE IMPULSE RESISTED BY FORCE AT BARRIER SUPPORT PLUS BARRIER INERTIA DURING MOMENTUM TRANSFER ** LOAD 3
78.740 KIP-SEC MISSILE IMPULSE RESISTED ONLY BY BARRIER INERTIA DURING MOMENTUM TRANSFER ** LOAD 4
24.4 FPS BARRIER INITIAL VELOCITY DUE TO LOAD 4

BARRIER EQUIVALENT HEIGHT	MISSILE WEIGHT LOAD 3	MISSILE WEIGHT LOAD 4	BARRIER PLASTIC FORCE	BARRIER EFFEC. YIELD DEFLECTION	BARRIER PERIOD
KIPS	KIPS	KIPS	KIPS	FT	SEC
3.925	0.000	100.000	3815.0	0.0100	0.0036

****

RESULTS OF TIME HISTORY ANALYSIS FOR MISSILE IMPACT WITH OTHER LOADS

****

1 TIME HISTORY NUMBER	2 DURATION OF LOAD 3	3 MISSILE FORCE LOAD 3	4 FORCE AT BARRIER SUPPORT	5 TIME OF MAX BARRIER DEFLECTION	6 MAXIMUM BARRIER DEFLECTION	7 MAXIMUM BARRIER DUCTILITY	8 MAXIMUM BARRIER VELOCITY	9 FINAL BARRIER RESISTING MECHANISM
	SEC	KIPS	KIPS	SEC	FT		FT/SEC	
0	0.0	0.0	3815.0	0.020838	0.2569	25.69	24.40	SPECIAL BARRIER SPRING

Attachment #1 pg 2 of 2
14235.17-C.01
FPC CR1
PREP R. ... 5/16/83
REV. R. D. ... 6/13/83
I/K ... 7/1/83

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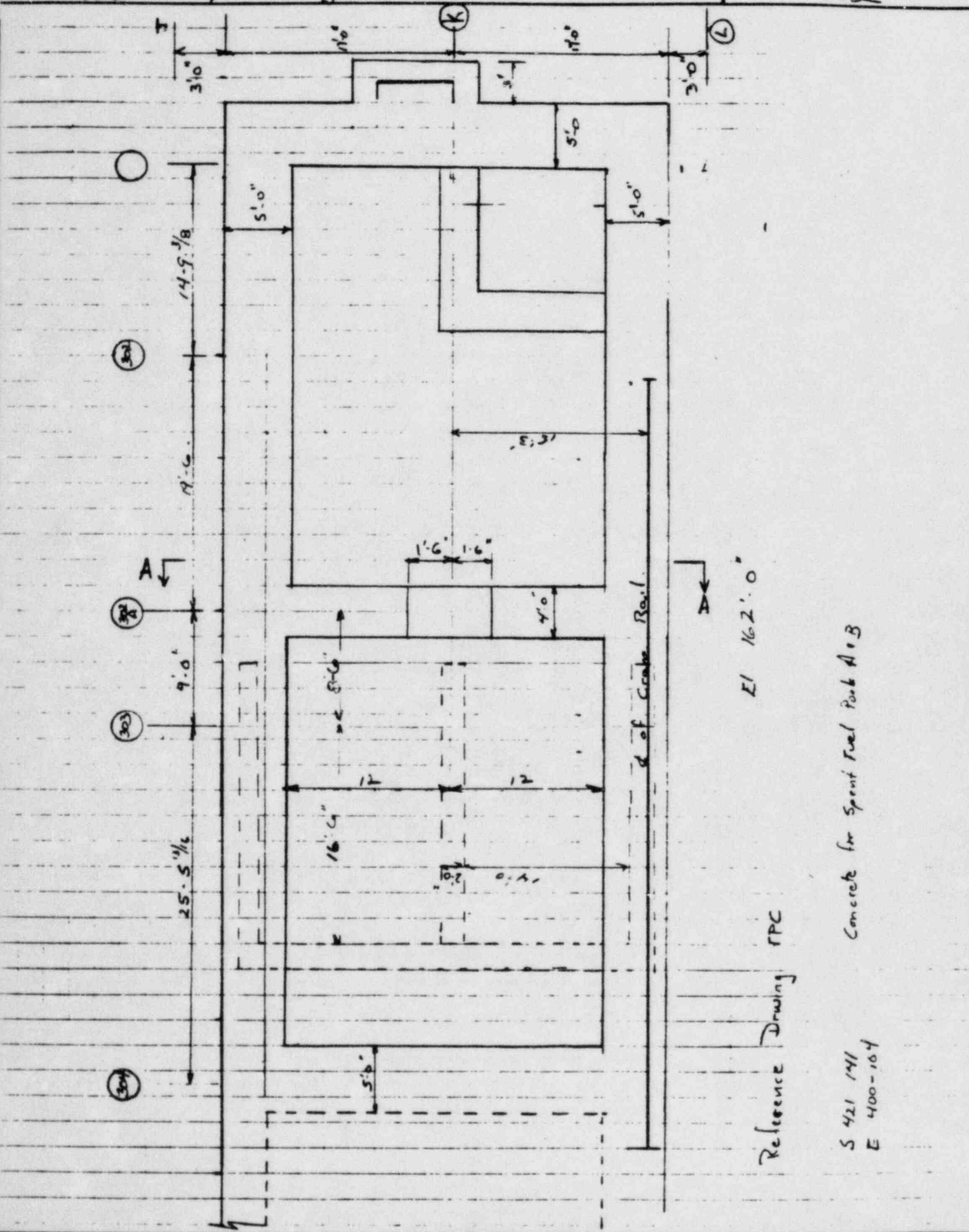
R. D. Salter 6/9/83	[illegible] 6/13/83	[illegible] 9/6/83

SUBJECT/TITLE	QA CATEGORY/CODE CLASS
Load Drop Analysis for SFHT-7	I NSC



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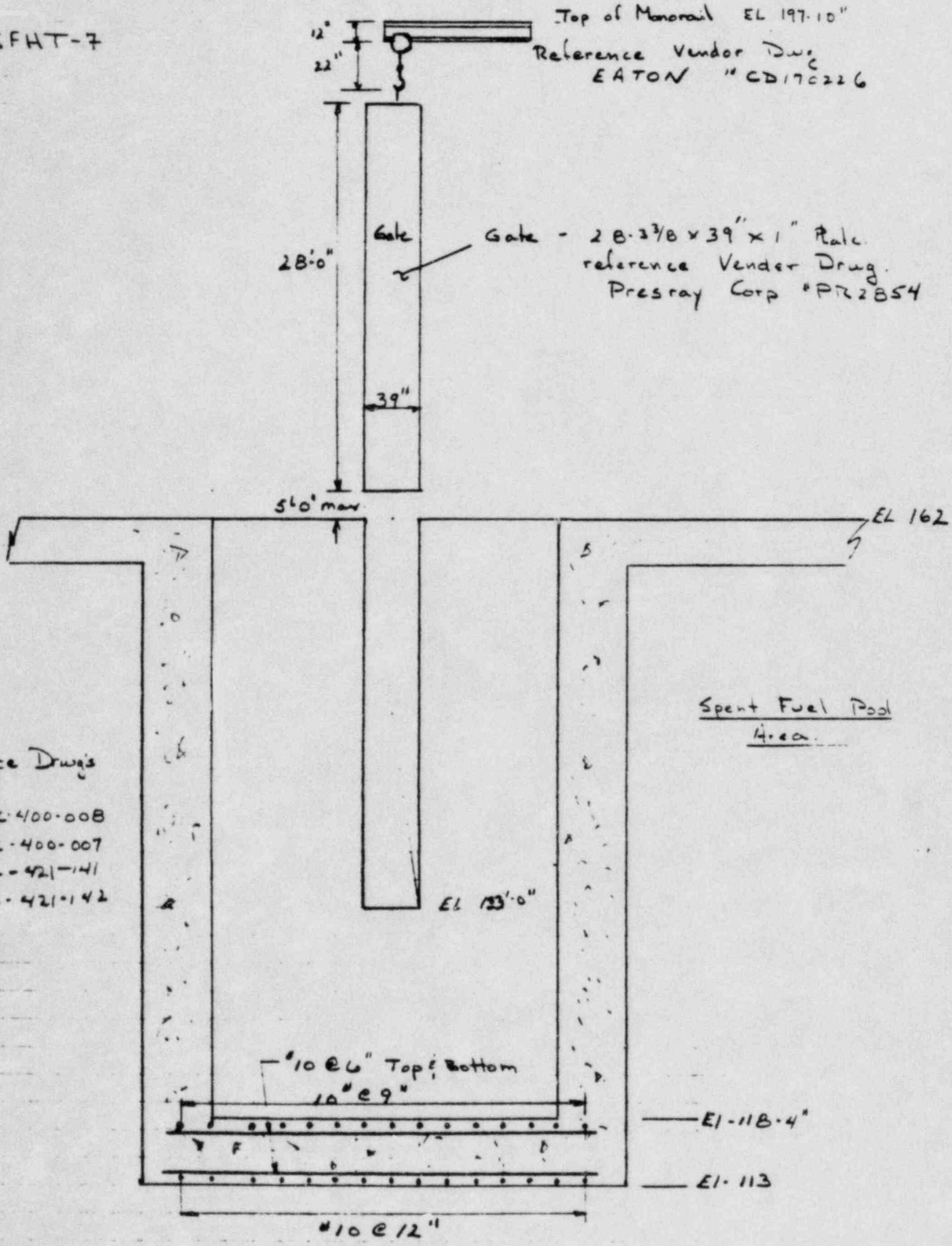
R. D. Balke 6/9/83	R. Seeburger 6-13-83	[signature] 7/6/83

SUBJECT/TITLE	QA CATEGORY/CODE CLASS
Load Drop Analysis for SFHT-7	I NSR

SFHT-7



Reference Drwg's

FPC-SC-400-008
SC-400-007
SC-421-141
SC-421-142

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R. D. Salter 6/9/83	R. Seelinger 6-13-83	Kenneth Nicholas 6/14/83

SUBJECT/TITLE	QA CATEGORY/CODE CLASS
Load Drop Analysis for SFHT-7	I NSR

SFHT-7 Spent Fuel Pool Gate Hoist

Discussion.

This hoist is located on a monorail at EL 197'-10" on column line 302A of the Auxiliary Bldg. The purpose of the hoist is to remove the gate between spent fuel pools A & B. The gate is the only pick-up item for this hoist.

Two load drops case are considered for analysis. The first drop is to allow the gate to fall from maximum lifting height to the bottom of the wall openning. A wall segment equal to the remaining height of the wall and the width of the openning is analysised as a column in compression.

The second drop will analyize the bottom slab of the spent fuel pool. Since the fuel racks are resting in the bottom of the pools the gate can only hit the concrete directly by sliding between the wall and racks. For the ease of calculation the gate is assumed to hit the middle of the slab

Input Data for SBMNI Program (ST-331)

Drop #1 on 4 ft wall between pools.

Gate wt = 3.9^k
Total Drop wt = 3.9

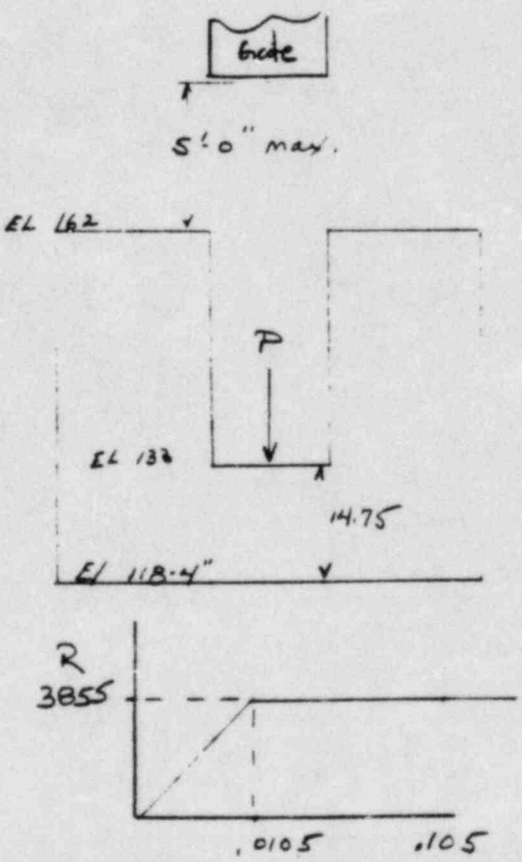
Drop height : 5+(162-133) = 34'
wt of barrier = 1/6 4' × 3' × 14.75' × .15 k/ft² = 4.425 k.

Missile Mass = $\frac{3.9^k}{32.2\ ft/sec^2} = .121\ k\text{-}sec^2/ft$

Velocity @ impact = $V_1 = \sqrt{2gh} = \sqrt{2(32.2\ ft/sec^2)\,34\ ft} = 46.8\ ft/sec$

Momentum @ impact = 46.8 ft/sec × .121 k-sec²/ft = 5.67 k-sec

* Assume a 3 ft effective length



Resistance Curve
* Assume: that the wall steel doesn't act in compression.

$P_u = \phi\,[.85 f'_c A_g]$ $F'_c = 3000$ psi

$\phi = .70$ (ACI-318-77 sec 9.3.2)

$P_u = .7(.85)3(4' \times 3')\,144\ ft^2/in^2 = 3084.^k$

$\Delta = \frac{RL}{AE} = \frac{3855.^k (14.75)'}{12\ ft^2\,(449568\ k/ft^2)} = .0105$

$E = 57000\sqrt{3000} = 3122\ ksi = 449568\ k/ft^2$

$R = DIF \times P_u = 1.25(3084) = 3855^k$

Drop #2 - Bottom slab of Fuel Pools.

Gate wt = 3.9

Total Drop Load = 3.9

Missile Mass = $\frac{3.9}{32.2}$ = .121

Height of Drop = 5' + (162'0" - 118.25) = 48.75'

Velocity @ Impact (no water drag) $V_c = \sqrt{2gh} = \sqrt{2(32.2)48.75} = 56.03$

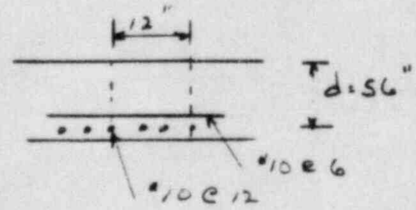
Momentum @ Impact = $V_c \times M_{missile} = 56.03 \times .121 = 6.78$

Assume a 12 ft diameter fan radius for the yeild line circle

Effective Target Wt = $\frac{1}{6}\ .15\ \pi \left(\frac{12}{2}\right)^2 5 = 14.137$

Minimum Ultimate Moment Capacity, M_u

· Use an average of the rebar arrangement to determine A_s



$A_s = \frac{1.27 + 2(1.27)}{2} = 1.905$

$a = \frac{1.905(40)}{.85\ 3\ (12)} = 2.49$

$M_u = \phi A_s f_y (d - a/2) = 0.9(1.905)40\left(56 - \frac{2.49}{2}\right) = 3755\ ^{k\text{-}"}/_{ft} = 313\ ^{k\text{-}'}/_{ft}$

Check Impact Velocity of Missile Through Water (see Reference #16)

$h = 5'$ $W = 3900$ lb $H = 43.75'$
$L = 28'$ $d = 39'' = 3.25'$

$$V_0 = \sqrt{2gh} = \sqrt{2(32.2)5'} = 17.9 \text{ ft/sec}$$

$$R_0 = \frac{V_0 d}{\nu} = \frac{17.9 \cdot 3.25}{.93\times10^{-5}} = 6.25\times10^{6}$$

$\nu = .93\times10^{-5}$ for water viscosity.

$$L/d = \frac{28'}{3.25'} = 8.61 \qquad C_D = 1.33$$

$$A_0 = \frac{39 \times 1}{144} = .271 \text{ ft}^2$$

$$a = \frac{\gamma_w C_D A_0}{2W} = \frac{62.4(1.33).271}{2(3900)} = .0029 \text{ ft}^{-1}$$

$2aL = .1624$
$2aH = .2538$

$$b = \frac{\gamma g}{W} = \frac{62.4(32.2)}{3900} = .5152$$

$$\gamma_m = \frac{3900}{39(28)1/144} = 514 \text{ lb/ft}^2$$

$$V_2 = \left[g(1-\gamma/\gamma_m)/a\right]^{1/2} = \left[32.2\left(1-\frac{62.4}{514}\right)\frac{1}{.0029}\right]^{1/2} = 98.8 \text{ ft/sec.}$$

$$Z_2(H) = V_2^2 + e^{-2aH}\left\{\frac{b A_0}{2a^2}\left[e^{2aL}(1-2aL)-1\right] + V_0^2 + \frac{g}{a}\left(e^{2aL}\frac{\gamma}{\gamma_m}-1\right)\right\}$$

$$Z_2(H) = 98.8^2 + e^{-.2538}\left\{\frac{.5152(.271)}{2(.0029)^2}\left[e^{.1624}(1-.1624)-1\right] + 17.9^2 + \frac{32.2}{.0029}\left(e^{.1624}\frac{62.4}{514}-1\right)\right\}$$

$$Z_2(H) = 9761 + .7758\{-122 + 320.4 - 9518\} = 2530$$

$$V = \sqrt{Z_2(H)} = \sqrt{2530} = 50.3 \text{ ft/sec.}$$

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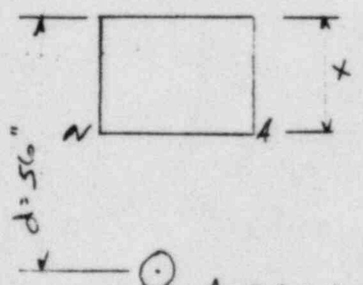
R. D. Salter 6/9/83	R. Seelungan 6-13-83	[illegible]

SUBJECT/TITLE	QA CATEGORY/CODE CLASS
Load Drop Analysis for SFHT-7	I NSR

Momentum @ Impact with water drag $= .121 \times 50.3 = 6.09$ 'k-sec.

Effective Moment of Inertia

$I_g = \frac{1}{12} 12(60)^3 = 216000 \text{ in}^4 = 10.42 \text{ ft}^4$



$nA_s = 9(1.905) = 17.1$

$\frac{E_s}{E_c} = 9$

Solve for x

$\frac{12x^2}{2} - 17.1\ (56 - x) = 0$

$6x^2 + 17.1x - 957 = 0$

$x = 11.28$ in

$I_{cr} = \frac{1}{3} 12(11.28)^3 + 17.1\ (56 - 11.28)^2 = 39940 \text{ in}^4.$

$I_{eff} = \frac{1}{2} (I_{cr} + I_g) = 127970 \text{ in}^4 = 6.17 \text{ ft}^4$

Peak Resistence of slab 'R'

$R = 4\pi (M_u) = 4\pi\ 313\ ^{k'}/ = 3933$

w/ Dyn Incr. Fact = 1.1 $\quad R = 4326$

Yield Displacement

$X_e = C \frac{Ra^2}{D} \qquad X = \frac{.00721\ (4326)(12)^2}{(449571)\ 6.17}(1 - .25^2) = .0015 \text{ ft}$

$D = \frac{Eh^3}{12(1 - \nu^2)}$

let: $C = .00721$ for $B/a = 23/12 = 1.91$

$a = 12$ ft

$h^3 = I_{eff}$.

Deflection Due to Dead Load

Consider slab area above support walls

11' × 23'

Dead wt of water

assume water height = 162 - 118.33 = 43.67'

$62.4 \text{ lb/ft}^2 \times 43.67 \text{ ft} = 2.725 \text{ k/ft}^2$

$a = 23'$ $b = 11'$ $a/b = 2.09$ $\alpha = .0277$

$t_{eff} = \sqrt[3]{I_{eff}} = \sqrt[3]{\frac{127990}{12}} = 4.2$ $y_{max} = \frac{\alpha q b^4}{E t_{eff}^3} = \frac{.0277 (2.725) 11^4}{449568 (4.2)^3} = .00003 \text{ ft.}$

Some additional load & deflection should be included to account for the spent fuel & rack. Even with these additional loads the deflection from the total Dead load on the slab is small when compared to the calculated yeild deflection of .0014 ft.

∴ Neglect deflection due to dead load.

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R. D. Salter 6/9/83	R. Sulinger 6-13-83	Kenneth Wislocky 7/6/83

SUBJECT/TITLE	QA CATEGORY/CODE CLASS
Load Drop Analysis for - SFHT-7	I NSR

Summary of Results

refer: Attach #6

Drop #1 - Gate of Wall

Ductility Ratio 2.06
Max. Deflection 0.0216

Drop #2 - Gate on Slab

	With Water Drag	No Water Drag
Ductility Ratio =	6.04	6.91
Maximum Barrier Deflection =	0.0085	.0104

Penetration & Scabbing NDRC (page 337)
(refer #12)

$$x = \sqrt{4\,k\,N\,W\,d\left(\frac{V_o}{1000\,d}\right)^{1.8}}$$

$$d_e = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4(.271)}{\pi}} = 7.05$$

$K = \frac{180}{\sqrt{F_c}} = 3.28$

$N = .72$

$W = 3900$

$d = 7.05$

$V_o = 6.78$ ft/sec.

$$x = \sqrt{4(3.28).72(3900)7.05\left(\frac{56.03}{1000(7.05)}\right)^{1.8}} = 6.57$$

$$\frac{x}{d} = .0.93$$

$$s/d = 3.91\left(\frac{x}{d}\right) - .718\left(\frac{x}{d}\right)^2$$

or

$$\frac{s}{d} = 3.91(.93) - .718(.93)^2 = 3.0$$

$s_{reqd} = 3.0(7.05) = 21''$ in^2 < 60" OK

Check Liner on bottom of pool for Penetration.

Penetration of Steel

$$T = \frac{\left(\frac{MV^2}{2}\right)^{2/3}}{672\,D}$$ (refer # 16 page 2-3) equation 2-7

V = Velocity of Missile = 56.03 (no water drag)

M: Mass of Missile = 121 lb·sec²/ft (Wt · 3.9ᵏ)

D = Diameter (equivalent) = $\sqrt{\left(\frac{1'' \times 39''}{\pi}\right)4}$ = 7.05

$$T = \frac{\left(\frac{56.03^2(121)}{2}\right)^{2/3}}{672\,(7.05)} = .697$$

required t_p = 1.25 (.697) = .871"

Penetration of Concrete (if no steel is present.)

$$X = 12\,k_p\,A_p\,\log_{10}\left(1 + \frac{V_s^2}{215000}\right)$$ (refer #16 page 2-1)

$f'_c = 3000$

$k_p = .0036$

$A_p = \frac{3900\ \text{lb}}{.271\ \text{sq ft}} = 14391$ psf

$$x = 12(.0036)14391 \log_{10}\left(1 + \frac{56.03^2}{215000}\right)$$

$V_s = 56.03$ ft/sec

$x = 3.9''$

$$\chi_1 = \left[1 + e^{-4(\frac{s}{\chi}-2)}\right]\chi' = 3.9$$

CALCULATION SHEET

STONE & WEBSTER ENGINEERING CORPORATION

A5010 61

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FPC 14235.17 -C01		K.7

PREPARER/DATE	REVIEWER/CHECKER/DATE	INDEPENDENT REVIEWER/DATE
R. D. Salter 6/9/83	R. Seiberger 6/3/83	Kenneth Wiatloy 7/7/83

SUBJECT/TITLE	QA CATEGORY/CODE CLASS
Load Drop Analysis for SFHT-7	I/NSR

Conclusion for Steel Liner

The actual thickness of the stainless steel liner is 3/16" (refer FPC. drwg #521-110 to 1.2) This well below the value required by calculation, 0.871"

The concrete penetration calculation shows that that the gate would penatrate 3.9 inch of concrete if the liner were not present. From the combination of the concrete penetration and the thin steel liner it can be judged that the steel liner is not adequate for the impact of the gate

```
IN[illegible] ECHO
         1         2         3         4         5         6         7         8
12345678901234567890123456789012345678901234567890123456789012345678901234567890
LOAD DROP # 1 ANALYSIS FOR SFHT7--ON WALL                                    ...
0.
1.0       3855.     0.0105    3855.     0.105     3855.     0.50      4.425
0.        0.0       0.        0.0
0.0       0.0       0.0       5.67      3.9
LOAD DROP # 2 ANALYSIS FOR SFHT7--ON SLAB WITHOUT WATER DRAG                 ...
0.
1.0       4326.     0.0015    4326.     0.015     4326.     0.200     14.137
0.        0.0       0.        0.0
0.0       0.0       0.0       6.78      3.9
LOAD DROP # 2 ANALYSIS FOR SFHT7--ON SLAB WITH WATER DRAG                    ...
0.
1.0       4326.     0.0014    4326.     0.0144    4326.     0.200     14.137
0.        0.0       0.        0.0
0.0       0.0       0.0       6.09      3.9
STOP
```

ATTACHMENT 6
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Prep: R J Salter 6/13/83
Rev: [signature] 6/13/83
I.R. [signature] 7/6/83

ST-331,SBMMI,VERS-00 LEVEL-00 78.172 13.53.27

LOAD DROP # 1 ANALYSIS FOR SFHT7--ON WALL ...

****

DATA ON MISSILE, BARRIER, AND LOAD COMBINATION EQUATION

****

BARRIER FORCE DISPLACEMENT RELATIONSHIP

KIPS	FEET
3855.0	0.0105
3855.0	0.1050
3855.0	0.5000

0.0 KIPS EQUIVALENT STATIC FORCE ** LOAD 1
0.0 KIPS EQUIVALENT CONSTANT DYNAMIC FORCE ** LOAD 2
0.0 KIP-SEC MISSILE IMPULSE RESISTED BY FORCE AT BARRIER SUPPORT PLUS BARRIER INERTIA DURING MOMENTUM TRANSFER ** LOAD 3
5.670 KIP-SEC MISSILE IMPULSE RESISTED ONLY BY BARRIER INERTIA DURING MOMENTUM TRANSFER ** LOAD 4
21.9 FPS BARRIER INITIAL VELOCITY DUE TO LOAD 4

BARRIER EQUIVALENT WEIGHT	MISSILE WEIGHT LOAD 3	MISSILE WEIGHT LOAD 4	BARRIER PLASTIC FORCE	BARRIER EFFEC. YIELD DEFLECTION	BARRIER PERIOD
KIPS	KIPS	KIPS	KIPS	FT	SEC
4.425	0.000	3.900	3855.0	0.0105	0.0038

****

RESULTS OF TIME HISTORY ANALYSIS FOR MISSILE IMPACT WITH OTHER LOADS

****

1 TIME HISTORY NUMBER	2 DURATION OF LOAD 3	3 MISSILE FORCE LOAD 3	4 FORCE AT BARRIER SUPPORT	5 TIME OF MAX BARRIER DEFLECTION	6 MAXIMUM BARRIER DEFLECTION	7 MAXIMUM BARRIER DUCTILITY	8 MAXIMUM BARRIER VELOCITY	9 FINAL BARRIER RESISTING MECHANISM
	SEC	KIPS	KIPS	SEC	FT		FT/SEC	
0	0.0	0.0	3855.0	0.001720	0.0216	2.06	21.93	SPECIAL BARRIER SPRING

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CALC. NO. C-01
JO 14235.17
PAGE 2 OF 4
Prep R. D. Sulk 6/13/83
Rev R. [illegible] 6/13/83

LOAD DROP # 2 ANALYSIS FOR SFHT7--ON SLAB WITHOUT WATER DRAG ...

****

DATA ON MISSILE, BARRIER, AND LOAD COMBINATION EQUATION

****

BARRIER FORCE DISPLACEMENT RELATIONSHIP

KIPS	FEET
4326.0	0.0015
4326.0	0.0150
4326.0	0.2000

0.0 KIPS EQUIVALENT STATIC FORCE ** LOAD 1
0.0 KIPS EQUIVALENT CONSTANT DYNAMIC FORCE ** LOAD 2
0.0 KIP-SEC MISSILE IMPULSE RESISTED BY FORCE AT BARRIER SUPPORT PLUS BARRIER INERTIA DURING MOMENTUM TRANSFER ** LOAD 3
6.780 KIP-SEC MISSILE IMPULSE RESISTED ONLY BY BARRIER INERTIA DURING MOMENTUM TRANSFER ** LOAD 4
12.1 FPS BARRIER INITIAL VELOCITY DUE TO LOAD 4

BARRIER EQUIVALENT WEIGHT	MISSILE WEIGHT LOAD 3	MISSILE WEIGHT LOAD 4	BARRIER PLASTIC FORCE	BARRIER EFFEC. YIELD DEFLECTION	BARRIER PERIOD
KIPS	KIPS	KIPS	KIPS	FT	SEC
14.137	0.000	3.900	4326.0	0.0015	0.0025

****

RESULTS OF TIME HISTORY ANALYSIS FOR MISSILE IMPACT WITH OTHER LOADS

****

1 TIME HISTORY NUMBER	2 DURATION OF LOAD 3	3 MISSILE FORCE LOAD 3	4 FORCE AT BARRIER SUPPORT	5 TIME OF MAX BARRIER DEFLECTION	6 MAXIMUM BARRIER DEFLECTION	7 MAXIMUM BARRIER DUCTILITY	8 MAXIMUM BARRIER VELOCITY	9 FINAL BARRIER RESISTING MECHANISM
	SEC	KIPS	KIPS	SEC	FT		FT/SEC	
0	0.0	0.0	4326.0	0.001620	0.0104	6.91	12.10	SPECIAL BARRIER SPRING

Prep: R. D. Salter 6/13/83
Rev: [illegible] 6/13/83
IR: [illegible] 7/7/83

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ST-331,SBHMI,VERS-00 LEVEL-00 78.172 13.53.27

LOAD DROP # 2 ANALYSIS FOR SFHT7--ON SLAB WITH WATER DRAG ...

DATA ON MISSILE, BARRIER, AND LOAD COMBINATION EQUATION

BARRIER FORCE DISPLACEMENT RELATIONSHIP

KIPS	FEET
4326.0	0.0014
4326.0	0.0144
4326.0	0.2000

0.0 KIPS EQUIVALENT STATIC FORCE ** LOAD 1
0.0 KIPS EQUIVALENT CONSTANT DYNAMIC FORCE ** LOAD 2
0.0 KIP-SEC MISSILE IMPULSE RESISTED BY FORCE AT BARRIER SUPPORT PLUS BARRIER INERTIA DURING MOMENTUM TRANSFER ** LOAD 3
6.090 KIP-SEC MISSILE IMPULSE RESISTED ONLY BY BARRIER INERTIA DURING MOMENTUM TRANSFER ** LOAD 4
10.9 FPS BARRIER INITIAL VELOCITY DUE TO LOAD 4

BARRIER EQUIVALENT WEIGHT	MISSILE WEIGHT LOAD 3	MISSILE WEIGHT LOAD 4	BARRIER PLASTIC FORCE	BARRIER EFFEC. YIELD DEFLECTION	BARRIER PERIOD
KIPS	KIPS	KIPS	KIPS	FT	SEC
14.137	0.000	3.900	4326.0	0.0014	0.0024

RESULTS OF TIME HISTORY ANALYSIS FOR MISSILE IMPACT WITH OTHER LOADS

1 TIME HISTORY NUMBER	2 DURATION OF LOAD 3	3 MISSILE FORCE LOAD 3	4 FORCE AT BARRIER SUPPORT	5 TIME OF MAX BARRIER DEFLECTION	6 MAXIMUM BARRIER DEFLECTION	7 MAXIMUM BARRIER DUCTILITY	8 MAXIMUM BARRIER VELOCITY	9 FINAL BARRIER RESISTING MECHANISM
	SEC	KIPS	KIPS	SEC	FT		FT/SEC	
0	0.0	0.0	4326.0	0.001480	0.0085	6.04	10.87	SPECIAL BARRIER SPRING

Prep: R D. Salker 6/13/83
Rev: K Leubuger 6/13/83
I.R. Kubolocky 7/6/83

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ENCLOSURE 4

POSTULATED SPENT FUEL POOL GATE
DROP INTO FUEL POOL

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CALCULATION TITLE PAGE

*SEE INSTRUCTIONS ON REVERSE SIDE

5010.64 (FRONT)

CLIENT & PROJECT	PAGE 1 OF 43
CALCULATION TITLE (Indicative of the Objective): POSTULATED SPENT FUEL POOL GATE DROP INTO FUEL POOL	QA CATEGORY (✓) ☒ I - NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ____ OTHER

CALCULATION IDENTIFICATION NUMBER

J.O. OR W.O. NO.	DIVISION & GROUP	CURRENT CALC. NO.	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
14235.17	NM (C)	M-06		

*APPROVALS - SIGNATURE & DATE [signature] 7-12-83

PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)	REV. NO. OR NEW CALC NO.	SUPERSEDES *CALC. NO. OR REV. NO.	CONFIRMATION *REQUIRED (✓) YES	NO
M. DURKEE 7/5/83	H. H. Kuo 7/6/83	S. M. Feldman 7-12-83	0	—		✓

DISTRIBUTION*

GROUP	NAME & LOCATION	COPY SENT (✓)	GROUP	NAME & LOCATION	COPY SENT (✓)
RECORDS MGT. FILES (OR FIRE FILE IF NONE)	JOB BOOK	KWO			
	BOSTON FIRE FILE	✓			
EMD	S. LIU 6/3RD	✓			
EMD	H. KUO 3C	✓			
EMD	S. FELDMAN 3C	✓			
POWER	E. MICHAELIS 6/3R	✓			

14235.17-NH(C)-M-06

14235.17 M06

STATEMENT OF REVIEW
CALCULATION NUMBER ______________

This calculation has been reviewed in accordance with EMTP-12.8.22-0 and was found to be adequate. The method of review was:

a. Review of Calculation

b. Comparison with a similar previous calculation

	REVIEWER	DATE	METHOD	REV.
1.	Shi Shin Kuo	7/6/83	a	0
2.				
3.				

The statement below applies to Nuclear Safety Related QA Category I calculations only.

This calculation has been INDEPENDENTLY reviewed in accordance with EMTP-12.8.22-0 and was found to be adequate. The method of review was: (list appropriate items)

a. Comparison with prequalified methods and assumptions ______________.
(prequalified document number(s))

b. Addressing the key questions appearing in EAP-5.3, and EAP-3.1

	INDEPENDENT REVIEWER	DATE	METHOD	REV.
1.	Sol M. Feldman	7-12-83	b	0
2.				
3.				

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OBJECTIVE

THE OBJECTIVES OF THIS CALCULATION ARE AS FOLLOWS:

- TO DETERMINE WHETHER THE SPENT FUEL STORAGE RACKS (BOTH STANDARD AND HIGH DENSITY CONFIGURATIONS) ARE CAPABLE OF PROTECTING THE FUEL BUNDLES FROM DAMAGE IN THE EVENT THE FUEL CANAL GATE IS DROPPED ON TOP OF THE RACKS

- IN THE EVENT THE RACKS ARE INCAPABLE OF WITHSTANDING THE DROP, DETERMINING HOW MANY FUEL BUNDLES COULD BE DAMAGED.

- DETERMINE THE FEASABILITY OF A DUAL 3-TON HOIST LIFTING ARRANGEMENT FOR THE PURPOSE OF ELIMINATING THE POSTULATED GATE DROP.

ASSUMPTIONS

FOR SPENT FUEL RACKS

1) MATERIAL PROPERTIES USED ARE THOSE FOR TYPE 304 STAINLESS STEEL @ 150°F.

2) WIDE FLANGE SECTION IS TAKEN TO BE W8x15.

3) MODE OF FAILURE IS DUE TO EXCESSIVE PLASTIC DEFORMATION.

4) BUCKLING IS NOT CONSIDERED

5) EFFECTS OF FUEL BUNDLES ON DROP ARE IGNORED.

FOR HIGH DENSITY FUEL RACKS

1) POISON SHEETS, LAMINATED INTO STORAGE CELL WALLS, ARE NEGLECTED IN THE ANALYSIS.

2) EFFECTS OF FUEL BUNDLES ON DROP ARE IGNORED

FOR GATE DROP

1) BOUANCY AND DRAG EFFECTS ARE NEGLECTED

2) DROP HEIGHT IS TAKEN TO BE 3 FEET ABOVE THE 162' ELEVATION, FOR A TOTAL DROP DISTANCE OF 33 FEET. (SEE PG. 18)

ALLOWABLE STRESS FOR THE DESIGN OF THE LIFTING HOOK OF THE GATE IS $0.6\ \sigma_y$ (REF. 8)

METHOD

AN ENERGY BALANCE APPROACH IS UTILIZED IN THE ANALYSIS OF THE TWO TYPES OF FUEL STORAGE RACKS. THIS METHOD EQUATES THE STRAIN ENERGY OF SPECIFIED SECTIONS OF THE RACKS TO THE KINETIC ENERGY OF THE GATE AT IMPACT. IN THIS WAY THE MAXIMUM DEFORMATIONS OF THE ANALYZED RACK SECTIONS CAN BE DETERMINED. BY NEGLECTING LOCALIZED EFFECTS, THE USE OF AN ENERGY BALANCE WILL GIVE A LOWER BOUND DEFORMATION OF THE SECTIONS ANALYZED.

FOR THE ANALYSIS OF THE DUAL HOIST, ONE OF THE TWO CABLES IS ASSUMED TO SNAP. THE RESULTING MAXIMUM FORCE IN THE REMAINING CABLE IS THEN DETERMINED. THE LIFTING HOOKS OF THE GATE ARE ALSO TO BE ANALYZED TO INSURE THAT THEY ARE CAPABLE OF WITHSTANDING THE MAXIMUM RESULTANT FORCE

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DESIGN INPUT

1) HEIGHT OF GATE DROP : 33 FEET (SEE PG. 18)

2) WEIGHT OF GATE : 4200 lb. (SEE PAGE. 16)

3) CAPACITY OF HOIST : 3 TONS (REF. 3)

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REFERENCES

(1) EMTR 400-A, "MATERIAL PROPERTIES FOR PIPE RUPTURE ANALYSIS", APPR. A.L. VAN SICKEL, 19 NOV 1976, ENGINEERING MECHANICS DIVISION, STONE & WEBSTER ENGINEERING CORP.

(2) VENDOR DRAWING # PR2859, REVISION C, "CANAL GATE", THE PRESRAY CORP.

(3) FPC DRAWING # S-521-114, REVISION C, "AUXILIARY BUILDING SPENT FUEL RACKS", FLORIDA POWER CORPORATION

(4) VENDOR DRAWING # 80 E 1490, REV 3, "STORAGE CELL TYPE 1", AUTOMATION INDUSTRIES INC.

(5) VENDOR DRAWING # 80E 1469, REV 2, "FUEL STORAGE RACK ARRAYS", AUTOMATION INDUSTRIES INC.

(6) DRAWING # 136031 E, REV 4, "MARK B2 FUEL ASSEMBLY DIMENSIONS FOR HANDLING & SHIPPING", BABCOCK & WILCOX COMPANY.

(7) BLAKE, ALEXANDER; "PRACTICAL STRESS ANALYSIS IN ENGINEERING DESIGN", PP. 355-356, MARCEL DEKKER, INC., N.Y., 1982.

(8) "MANUAL OF STEEL CONSTRUCTION - 7TH EDITION", AMERICAN INSTITUTE OF STEEL CONSTRUCTION, 1973.

(9) VENDOR DRAWING, REF # M2871, "3 TON HOIST", EATON CORPORATION.

(10) FPC DRAWING # L-002-003, REV III, "LAYOUT LONGITUDINAL SECTION THROUGH REACTOR BLDG. & SPENT FUEL PIT", FLORIDA POWER CORPORATION.

MATERIALS & ALLOWABLES

RACKS & GATE: TYPE 304 STAINLESS STEEL,

σ_y = 27.5 ksi @ 150°F. (REF 1)

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CONCLUSIONS

(A) IN THE EVENT OF A GATE DROP

1) NEITHER OF THE RACK DESIGNS ARE CAPABLE OF WITHSTANDING THE POSTULATED GATE DROP. THUS, THE GATE DROP WOULD POTENTIALLY RESULT IN DAMAGE TO THE FUEL BUNDLES.

(B) IF A DUAL HOIST ARRANGEMENT IS USED

1) A SINGLE HOIST IS CAPABLE OF TAKING THE MAXIMUM LOAD SHOULD THE SECOND HOIST'S CABLE SNAP. (THIS IS ASSUMING THE DUAL HOIST ARRANGEMENT IS USED). (DETAILED EXPLANATION ON P. 36)

2) THE LIFTING HOOKS ON THE GATE WOULD HAVE TO BE CHANGED TO 1.25" DIAMETER ROD IF THE DUAL HOIST ARRANGEMENT IS USED. (DETAILED SKETCH ON P. 41)

(C) POTENTIAL FUEL DAMAGE IN THE EVENT OF A GATE DROP

1) 64 FUEL BUNDLES COULD BE SHOVED UP AGAINST OTHER FUEL BUNDLES. THIS SITUATION SHOULD BE INVESTIGATED TO DETERMINE WHETHER CRITICALITY OF THE FUEL IS REACHED.

2) FOR THE CASE WHERE THE GATE LIES FLAT ON TOP OF THE RACK, IT COVERS 128 FUEL CELLS. THIS CASE SHOULD BE ANALYZED TO DETERMINE IF ANY THERMAL PROBLEMS EXIST.

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RESULTS SUMMARY

GATE SWING (DUAL HOIST ARRANGEMENT ASSUMING A CABLE SNAP)

1) FOR DUAL HOIST ARRANGEMENT: MAX. LOAD TO SECOND HOIST SHOULD ONE CABLE SNAP IS 6300 lb. (DETAILED ANALYSIS AND EXPLANATION ON PP 35-36).

2) GATE HOOK DIAMETER REQUIRED TO TAKE THE 6300 lb LOAD IS 1.25 INCHES

GATE DROP

SPENT FUEL RACKS

1) MAXIMUM DEFORMATIONS OF TWO SECTIONS ANALYZED: 233 INCHES AND 73 INCHES

HIGH DENSITY FUEL RACKS

1) LOWER BOUND DEFORMATION OF EDGE IMPACT (SMALLEST DIMENSION OF GATE IMPACTING): 22.1 INCHES

2) LOWER BOUND DEFORMATION OF EDGE IMPACT

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RESULTS SUMMARY (CONTINUED)

(LONGEST DIMENSION OF GATE IMPACTING):

2.05 INCHES

3) NUMBER OF CELLS THAT COULD BE PUSHED UP AGAINST OTHER CELLS FROM IMPACT OF GATE.

a) FOR EDGE ON IMPACT, SMALLEST DIMENSION OF GATE: 8 CELLS

b) FOR EDGE ON IMPACT, LONGEST DIMENSION OF GATE: 64 CELLS

4) NUMBER OF CELLS THAT COULD BE COVERED SHOULD THE GATE LAND FLAT ON TOP OF THE RACK:

128 CELLS

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ANALYSIS

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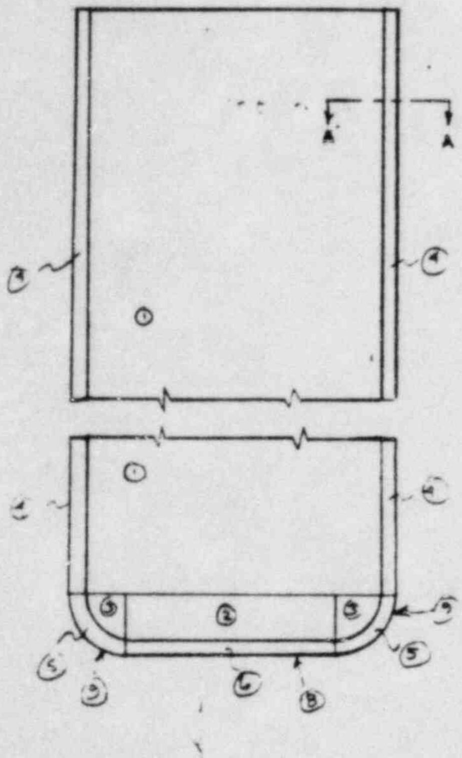
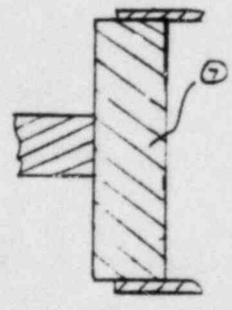
WEIGHT OF CANAL GATE





SECTION A-A

(SEE REF #2 FOR DETAILS CONCERNING DIMENSIONS)

DETERMINE WEIGHT OF GATE (NOTE: USING MAXIMUM TOLERANCES ON ALL DIMENSIONS)

SECTION ①: $332.5'' \times 35.625 \times 1 = 11845.3$ $\gamma_{STEEL} = 0.283\ lb/in^3$

$11845.3\,(0.283) =$ **3352 lb.**

SECTION ②: $24.375 \times 5.625 \times 1 = 137.1$ $137.1\,(0.283) =$ **38 lb.**

SECTION ③: $2\left(\pi(5.625)^2/4\right) = 49.7$ $49.7\,(0.283) =$ **14 lb.**

SECTION ④: $4\left[332.5\,(1.5)\,0.25\right] = 498.7$ $498.7\,(0.283) =$ **141 lb.**

SECTION ⑤: $4\left[\pi\,(7.125^2 - 5.625^2)/4\right]0.25 = 15$ $15\,(0.283) =$ **4 lb.**

SECTION ⑥: $2\,(24.375 \times 0.25 \times 1.5) = 18.3$ $18.3\,(0.283) =$ **5 lb.**

SECTION ⑦: $2\,(3/4 \times 4 \times 332.5) = 1995$ $1995\,(0.283) =$ **564 lb.**

SECTION ⑧: $24.25 \times 0.75 \times 4 = 72.75$ $72.75(0.283) =$ **20 lb.**

SECTION ⑨: $2\left[\pi(6.375^2 - 5.625^2)/4 \times 4\right] =$ $56.5(0.283) =$ **16 lb**

TOTAL WEIGHT OF GATE IS EQUAL TO THE SUM OF THE WEIGHTS OF SECTIONS ① THROUGH ⑨

* TOTAL WEIGHT OF GATE = 1151 lb

≈ 1200 lb.

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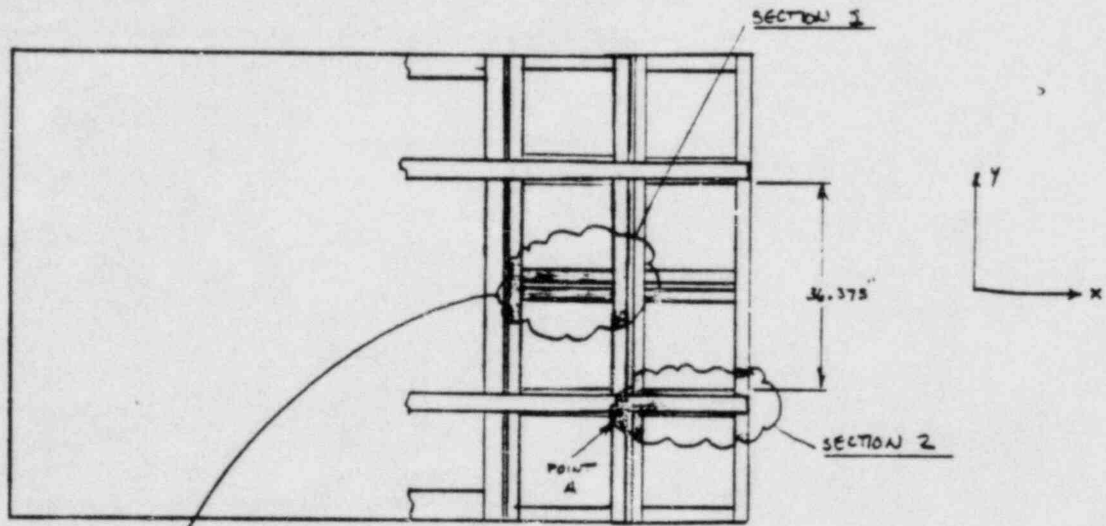
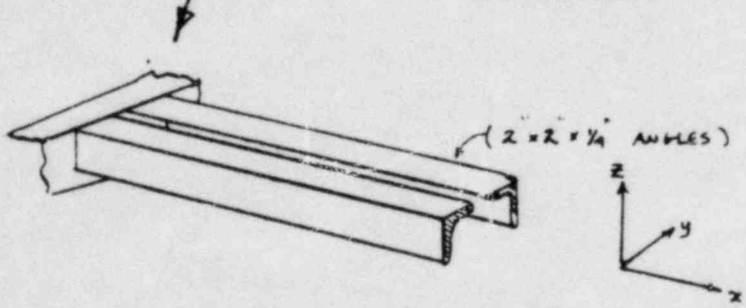
GATE DROP ONTO 1ST SET OF FUEL RACKS

GATE TO BE DROPPED ON SECTIONS OF RACK INDICATED BELOW (SEE REF. #3 FOR RACK DETAILS)



RACK "C" (TOP) (FIGURE I)



AS THE WIDTH OF THE GATE, NEGLECTING THE 1 1/2 × 1/4 SECTIONS AROUND THE PERIMETER, IS 37" AND THE BOTTOM CORNERS ARE ROUNDED, IT CAN BE ASSUMED THAT THE FULL IMPACT WILL BE TAKEN BY THE TWO ANGLES SHOWN ABOVE (SEE FIGURE II, PG. 18). THE GATE IS ASSUMED TO LIE IN THE Y-Z PLANE AND IMPACT THE MID-SPAN OF THE ANGLES.

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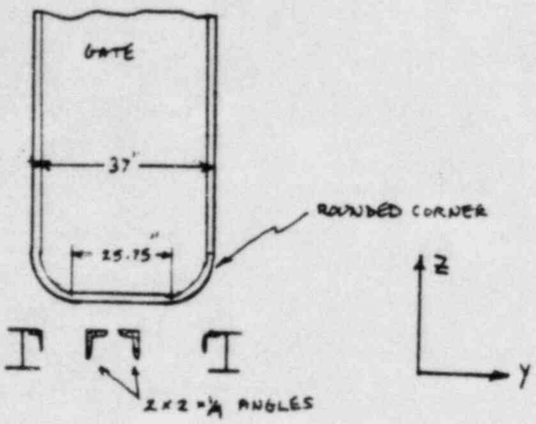

FIGURE II

DROP HEIGHT

FROM REFERENCES (3 & 10) THE HEIGHT FROM THE TOP OF THE FUEL RACKS TO THE TOP OF THE POOL IS FOUND TO BE 30 FEET. AS THE GATE WILL ONLY BE LIFTED OUT OF THE CANAL JUST HIGH ENOUGH TO ALLOW IT TO BE MOVED (~6"), 3 FEET IS ADDED TO THE DROP HEIGHT FOR CONSERVATISM. THIS BRINGS THE TOTAL DROP HEIGHT TO 33 FEET.

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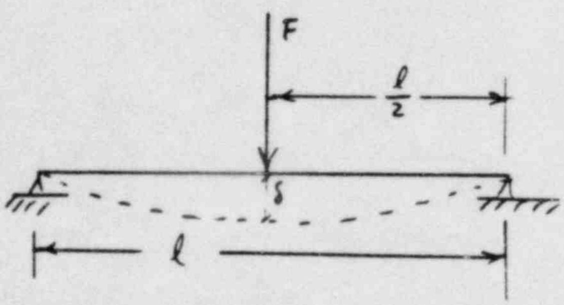
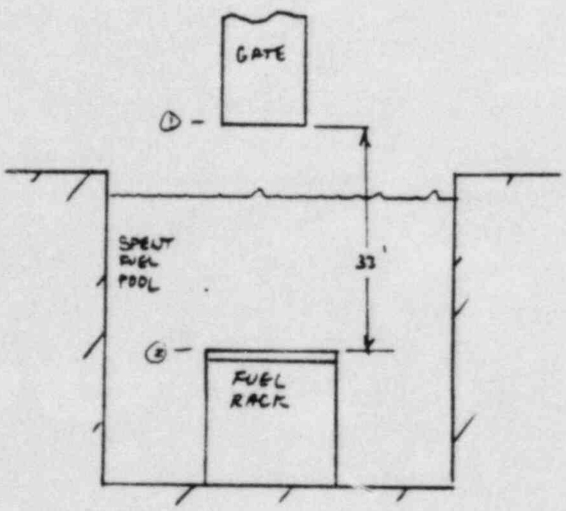
SECTION OF RACK "C" (TOP) WHERE GATE COULD HIT.

MODELED AS A SIMPLY - SUPPORTED BEAM



$l = 15.25''$ (REF # 3)



DROP HEIGHT : 33'

WEIGHT OF GATE: 4200 lb.

(NEGLECT DRAG & BOUANCY EFFECTS OF WATER)

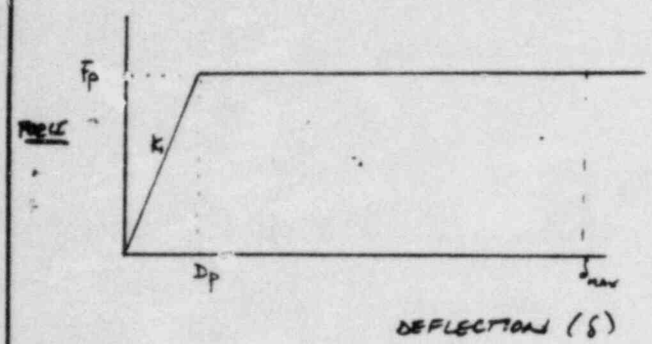
POTENTIAL ENERGY @ ① = KINETIC ENERGY @ ②

$P.E._{①} = mgh = 4200 (33) = \underline{138\ 600\ \text{FT-lb}}$

VELOCITY @ IMPACT $= \sqrt{2gh} = \sqrt{2(32.2)33}$

$= \underline{46\ \text{FT/SEC}}$ (31 mph)

$P.E._{①} = \underline{1.663 \times 10^6\ \text{IN-lb}}$



$K_1 = \frac{F}{\delta}$ $\delta = \frac{F l^3}{48EI}$

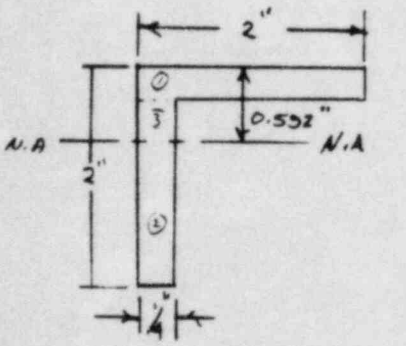
$\therefore K_1 = \frac{48EI}{l^3}$

$Dp = Fp/K_1$

$Fp = 4M_p/l$ (SIMPLY SUPPORTED BEAM)

(FOR A SIMPLY-SUPTED BEAM. $M_{MAX} = \frac{Fl}{4}$. LET $M_{MAX} = M_p$, $F = Fp = 4M_p/l$)

$M_p = \sigma_y \int y\, dA = \sigma_y$ (MOMENT OF AREA ABOUT NEUTRAL AXIS)



MOMENTS OF AREA

SECTION ①:

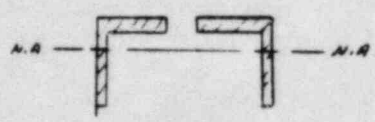
$(0.532 - 0.125)(0.25(2)) = 0.233 \text{ in}^3$

SECTION ②:

$(0.704)(0.25(1.408)) = 0.247 \text{ in}^3$

SECTION ③: $(0.171)(0.342 \times 0.25) = 0.014 \text{ in}^3$

Σ ①+②+③ = 0.233 + 0.247 + 0.014 = 0.494



ACTUAL BEAM CROSS-SECT.

THE ACTUAL BEAM CROSS-SECTION IS MADE UP OF TWO ANGLES, AS SHOWN AT THE LEFT, THEREFORE THE MOMENT OF THE AREA IS TWICE THE ABOVE VALUE.

i.e. $2(0.494) = 0.988^3$

$M_p = \sigma_y (0.988)$ (IN·lb)

$F_p = 4\sigma_y(0.988)/15.25 = 0.259\ \sigma_y$ (lb.)

(ASSUME 304 S.S → 27.5 KSI @150°F)

THEN

$M_p = (27.5 \times 10^3)\ 0.988 = \underline{27,170 \text{ in-lb.}}$

$F_p = 0.259(27.5 \times 10^3) = \underline{7122 \text{ lb.}}$

$K_1 = \dfrac{48 E I}{l^3}$ $\qquad E = 27.9 \times 10^6 \text{ - Psi} \quad (304 @ 152°F)$

$\qquad I = 2(0.384) = 0.768 \text{ in}^4$

$\therefore K_1 = 48(27.9 \times 10^6)\ 0.768 / 15.25^3 = \underline{290\ 000 \ \frac{lb}{in}}$

$D_p = F_p / K_1 = \underline{0.024 \text{ inches}}$

FOR ENERGY BALANCE:

$\frac{1}{2} D_p F_p + F_p (\delta_{max} - D_p) = 1.663 \times 10^6 \text{ in-lb}$

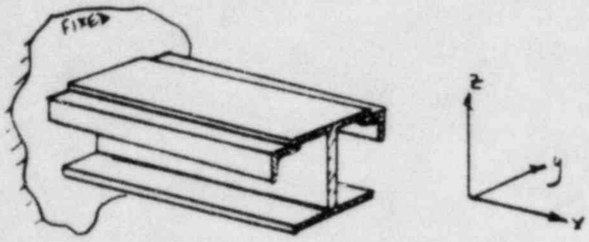
SOLVING FOR δ_{max}

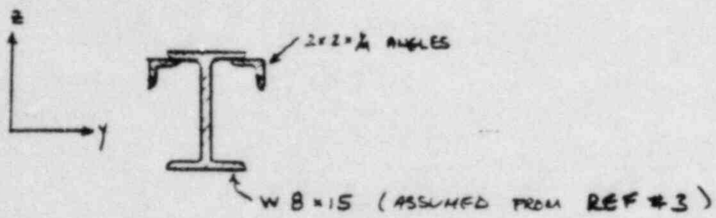
$$\delta_{max} = \frac{1.663 \times 10^6 - \frac{1}{2}(0.024)(7122)}{7122} + 0.02$$

$= \underline{.233 \text{ inches}}$

BECAUSE OF THE EXTREMELY LARGE δ_{max}, A COMPLETE FAILURE OF THE SECTION IN QUESTION CAN BE ASSUMED.

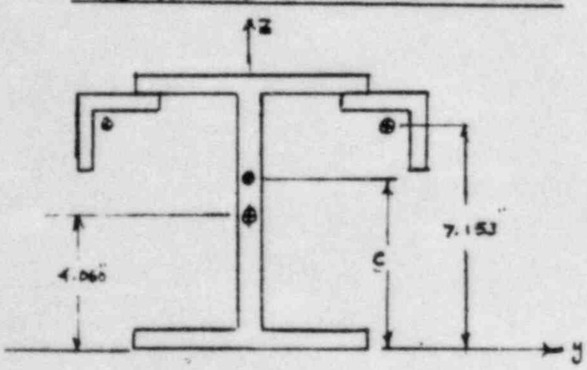
SECTION 2 OF FIGURE I





SECTION 2 IS ANALYZED AS A CANTILEVER BEAM DUE TO THE SUPPORT COLUMN LOCATED AT POINT A IN FIGURE I. LOCAL BUCKLING OF THE BEAM AND BUCKLING OF THE SUPPORT COLUMN ARE IGNORED. ANGLES WELDED TO THE END OF THE BEAM IN THE ± Y DIRECTION ARE IGNORED

FIND CENTROID OF COMPOSITE SHAPE



$$C = \frac{4.06\,(4.43) + 2\,(7.153)\,0.938}{4.43 + 2\,(0.938)}$$

$$= \underline{4.980''}$$

CALCULATE I

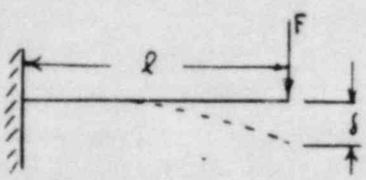
$I_{\text{NEUTRAL AXIS}} = 48.1 + 4.43(0.32)^2 + 2[0.348 + 0.938(2.173)^2]$

$= \underline{61.4 \text{ IN}^4}$

MOMENT OF AREA ABOUT NEUTRAL AXIS

$2[2.173(0.938)] + (0.3125(4.015)2.983) + (2.827)(0.245)(1.913) +$

$4.667(0.245)2.334 + (0.3125(4.015)4.823) = \underline{17.518 \text{ IN}^3}$



$\delta = F\ell^3/3EI$ $\quad \ell = 21.1875''$

$M_p = \sigma_y$ (MOM. OF AREA ABOUT NEUT. AXIS) $= 27.5 \times 10^3 (17.518) = \underline{481\ 745 \text{ in-lb}}$

FOR A CANTILEVER BEAM, THE MAXIMUM MOMENT $M_{max} = F\ell$

LET $M_{max} = M_p$ FOR $F = F_p$

WE HAVE $F_p = M_p/\ell$

$F_p = M_p/\ell$ (CANTILEVER BEAM)

$= 481\ 745/21.1875$

$= \underline{22\ 737 \text{ lb.}}$

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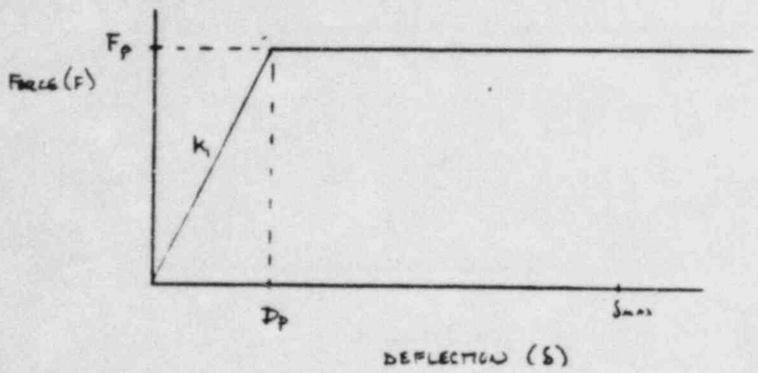

$$K_1 = F/\delta = \frac{3EI}{\ell^3}$$

$$= 3(27.9\times10^6)(61.4)/21.1875^3$$

$$= \underline{540\ 329\ \tfrac{lb}{in}}$$

$$D_p = F_p/K_1 = \frac{22\ 737}{540\ 329}$$

$$= \underline{0.042\ in}$$

FOR ENERGY BALANCE:

$$\tfrac{1}{2} D_p F_p + F_p(\delta_{max} - D_p) = 1.663\times10^6 \ \ in\cdot lb.$$

SOLVING FOR δ_{max}:

$$\delta_{max} = \frac{1.663\times10^6 - \tfrac{1}{2}(0.042)(22\ 737)}{22\ 737} + 0.042$$

$$= \underline{73.2''}$$

AGAIN, DUE TO THE EXTREMELY LARGE δ_{max} CALCULATED A COMPLETE FAILURE OF THE BEAM IS ASSUMED

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ANALYSIS AT THIS POINT SHOWS THAT THE FUEL STORAGE RACKS CANNOT WITHSTAND A DROP FROM THE CANAL GATE. THE TWO SECTIONS OF THE RACK THAT WERE ANALYZED WERE SHOWN TO BE INCAPABLE OF ABSORBING THE IMPACT ENERGY. THESE SECTIONS WOULD EITHER TEAR OUT OR BEND OUT OF THE WAY ;ALLOWING THE GATE TO CONTINUE DOWN THROUGH THE RACK.

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OBJECTIVE

THE OBJECTIVES OF THIS CALCULATION ARE AS FOLLOWS:

- TO DETERMINE WHETHER THE SPENT FUEL STORAGE RACKS (BOTH STANDARD AND HIGH DENSITY CONFIGURATIONS) ARE CAPABLE OF PROTECTING THE FUEL BUNDLES FROM DAMAGE IN THE EVENT THE FUEL CANAL GATE IS DROPPED ON TOP OF THE RACKS

- IN THE EVENT THE RACKS ARE INCAPABLE OF WITHSTANDING THE DROP, DETERMINING HOW MANY FUEL BUNDLES COULD BE DAMAGED.

- DETERMINE THE FEASABILITY OF A DUAL 3-TON HOIST LIFTING ARRANGEMENT FOR THE PURPOSE OF ELIMINATING THE POSTULATED GATE DROP.

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ASSUMPTIONS

FOR SPENT FUEL RACKS

1) MATERIAL PROPERTIES USED ARE THOSE FOR TYPE 304 STAINLESS STEEL @ 150°F.

2) WIDE FLANGE SECTION IS TAKEN TO BE W8×15.

3) MODE OF FAILURE IS DUE TO EXCESSIVE PLASTIC DEFORMATION.

4) BUCKLING IS NOT CONSIDERED

5) EFFECTS OF FUEL BUNDLES ON DROP ARE IGNORED.

FOR HIGH DENSITY FUEL RACKS

1) POISON SHEETS, LAMINATED INTO STORAGE CELL WALLS, ARE NEGLECTED IN THE ANALYSIS.

2) EFFECTS OF FUEL BUNDLES ON DROP ARE IGNORED

FOR GATE DROP

1) BOUANCY AND DRAG EFFECTS ARE NEGLECTED

2) DROP HEIGHT IS TAKEN TO BE 3 FEET ABOVE THE 162' ELEVATION, FOR A TOTAL DROP DISTANCE OF 33 FEET. (SEE PG. 18)

ALLOWABLE STRESS FOR THE DESIGN OF THE LIFTING HOOK OF THE GATE IS $0.6\,\sigma_y$ (REF. 8)

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METHOD

AN ENERGY BALANCE APPROACH IS UTILIZED IN THE ANALYSIS OF THE TWO TYPES OF FUEL STORAGE RACKS. THIS METHOD EQUATES THE STRAIN ENERGY OF SPECIFIED SECTIONS OF THE RACKS TO THE KINETIC ENERGY OF THE GATE AT IMPACT. IN THIS WAY THE MAXIMUM DEFORMATIONS OF THE ANALYZED RACK SECTIONS CAN BE DETERMINED. BY NEGLECTING LOCALIZED EFFECTS, THE USE OF AN ENERGY BALANCE WILL GIVE A LOWER BOUND DEFORMATION OF THE SECTIONS ANALYZED.

FOR THE ANALYSIS OF THE DUAL HOIST, ONE OF THE TWO CABLES IS ASSUMED TO SNAP. THE RESULTING MAXIMUM FORCE IN THE REMAINING CABLE IS THEN DETERMINED. THE LIFTING HOOKS OF THE GATE ARE ALSO TO BE ANALYZED TO INSURE THAT THEY ARE CAPABLE OF WITHSTANDING THE MAXIMUM RESULTANT FORCE

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DESIGN INPUT

1) HEIGHT OF GATE DROP : 33 FEET (SEE PG. 18)

2) WEIGHT OF GATE : 1200 lb. (SEE PAGE. 16)

3) CAPACITY OF HOIST : 3 TONS (REF. 9)

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REFERENCES

(1) EMTR 400-A, "MATERIAL PROPERTIES FOR PIPE RUPTURE ANALYSIS", APPR. A.L. VAN SICKEL, 19 NOV 1976, ENGINEERING MECHANICS DIVISION, STONE & WEBSTER ENGINEERING CORP.

(2) VENDOR DRAWING # PR2859, REVISION C, "CANAL GATE THE PRESRAY CORP.

(3) FPC DRAWING # S-521-114, REVISION C, "AUXILIARY BUILDING SPENT FUEL RACKS", FLORIDA POWER CORPORATION

(4) VENDOR DRAWING # 80 E 1490, REV 3, "STORAGE CELL TYPE 1", AUTOMATION INDUSTRIES INC.

(5) VENDOR DRAWING # 80E 1489, REV 2, "FUEL STORAGE RACK ARRAYS", AUTOMATION INDUSTRIES INC.

(6) DRAWING # 136031 E, REV 4, "MARK B2 FUEL ASSEMBLY DIMENSIONS FOR HANDLING & SHIPPING", BABCOCK & WILCOX COMPANY.

(7) BLAKE, ALEXANDER; "PRACTICAL STRESS ANALYSIS IN ENGINEERING DESIGN", PP. 355-356, MARCEL DEKKER, INC., N.Y., 1982.

(8) "MANUAL OF STEEL CONSTRUCTION - 7TH EDITION", AMERICAN INSTITUTE OF STEEL CONSTRUCTION, 1973.

(9) VENDOR DRAWING, REF # M2871, "3 TON HOIST", EATON CORPORATION.

(10) FPC DRAWING # L-002-003, REV III, "LAYOUT LONGITUDINAL SECTION THROUGH REACTOR BLDG. & SPENT FUEL PIT", FLORIDA POWER CORPORATION.

MATERIALS & ALLOWABLES

RACKS & GATE: TYPE 304 STAINLESS STEEL,

σ_y = 27.5 ksi @ 150°F. (REF 1)

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CONCLUSIONS

(A) IN THE EVENT OF A GATE DROP

NEITHER OF THE RACK DESIGNS ARE CAPABLE OF WITHSTANDING THE POSTULATED GATE DROP. THUS, THE GATE DROP WOULD POTENTIALLY RESULT IN DAMAGE TO THE FUEL BUNDLES.

(B) IF A DUAL HOIST ARRANGEMENT IS USED

1) A SINGLE HOIST IS CAPABLE OF TAKING THE MAXIMUM LOAD SHOULD THE SECOND HOIST'S CABLE SNAP. (THIS IS ASSUMING THE DUAL HOIST ARRANGEMENT IS USED). (DETAILED EXPLANATION ON P.36)

2) THE LIFTING HOOKS ON THE GATE WOULD HAVE TO BE CHANGED TO 1.25" DIAMETER ROD IF THE DUAL HOIST ARRANGEMENT IS USED. (DETAILED SKETCH ON P. 41)

(C) POTENTIAL FUEL DAMAGE IN THE EVENT OF A GATE DROP

1) 64 FUEL BUNDLES COULD BE SHOVED UP AGAINST OTHER FUEL BUNDLES. THIS SITUATION SHOULD BE INVESTIGATED TO DETERMINE WHETHER CRITICALITY OF THE FUEL IS REACHED.

2) FOR THE CASE WHERE THE GATE LIES FLAT ON TOP OF THE RACK, IT COVERS 128 FUEL CELLS. THIS CASE SHOULD BE ANALYZED TO DETERMINE IF ANY THERMAL PROBLEMS EXIST.

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RESULTS SUMMARY

GATE SWING (DUAL HOIST ARRANGEMENT ASSUMING A CABLE SNAP)

1) FOR DUAL HOIST ARRANGEMENT : MAX. LOAD TO SECOND HOIST SHOULD ONE CABLE SNAP IS 6300 lb. (DETAILED ANALYSIS AND EXPLANATION ON pp 35-36).

2) GATE HOOK DIAMETER REQUIRED TO TAKE THE 6300 lb LOAD IS 1.25 INCHES

GATE DROP

SPENT FUEL RACKS

1) MAXIMUM DEFORMATIONS OF TWO SECTIONS ANALYZED: 233 INCHES AND 73 INCHES

HIGH DENSITY FUEL RACKS

1) LOWER BOUND DEFORMATION OF EDGE IMPACT (SMALLEST DIMENSION OF GATE IMPACTING): 22.1 INCHES

2) LOWER BOUND DEFORMATION OF EDGE IMPACT

RESULTS SUMMARY (CONTINUED)

(LONGEST DIMENSION OF GATE IMPACTING):

2.05 INCHES

3) NUMBER OF CELLS THAT COULD BE PUSHED UP AGAINST OTHER CELLS FROM IMPACT OF GATE.

a) FOR EDGE ON IMPACT, SMALLEST DIMENSION OF GATE: 8 CELLS

b) FOR EDGE ON IMPACT, LONGEST DIMENSION OF GATE: 64 CELLS

NUMBER OF CELLS THAT COULD BE COVERED SHOULD THE GATE LAND FLAT ON TOP OF THE RACK:

128 CELLS

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ANALYSIS

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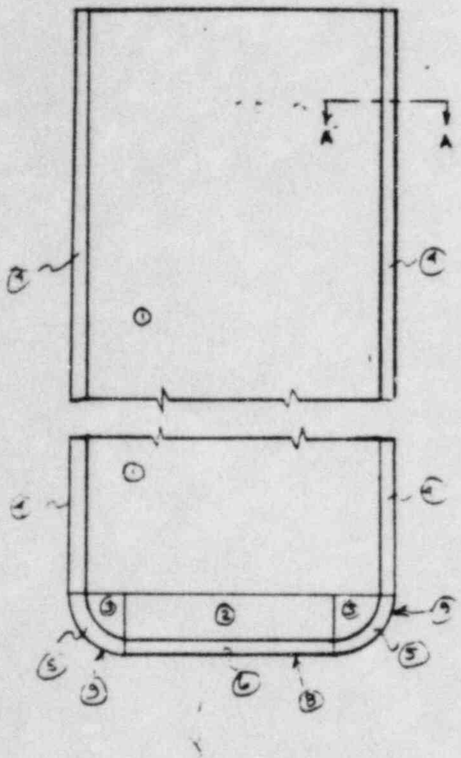
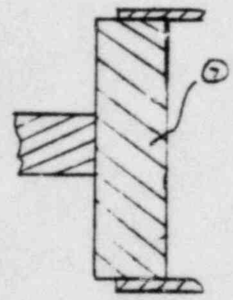
WEIGHT OF CANAL GATE





SECTION A-A

(SEE REF #2 FOR DETAILS CONCERNING DIMENSIONS)

DETERMINE WEIGHT OF GATE (NOTE: USING MAXIMUM TOLERANCES ON ALL DIMENSIONS

SECTION ①: 332.5" × 35.625 × 1 = 11845.3 γ_{STEEL} = 0.283 lb/in³

11845.3 (0.283) = **3352 lb.**

SECTION ②: 24.375 × 5.625 × 1 = 137.1 137.1 (0.283) = **38 lb.**

SECTION ③: 2(π(5.625)²/4) 49.7 49.7 (0.283) = **14 lb.**

SECTION ④: 4[332.5(1.5) 0.25] = 498.7 498.7 (0.283) = **141 lb.**

SECTION ⑤: 4[π (7.125² − 5.625²)/4] 0.25 = 15 15 (0.283) = **4 lb.**

SECTION ⑥: 2(24.375 × 0.25 × 1.5) = 18.3 18.3 (0.283) = **5 lb.**

SECTION ⑦: 2(3/4 × 4 × 332.5) = 1995 1995 (0.283) = **564 lb.**

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SECTION ⑧ : $29.25 \times 0.75 \times 4 = 72.75$ $72.75(0.287) =$ **20 lb.**

SECTION ⑨ : $2\left[\pi\left(6.375^2 - 5.625^2\right)/4 \times 4\right] =$ $56.5(0.283) =$ **16 lb**

TOTAL WEIGHT OF GATE IS EQUAL TO THE SUM OF THE WEIGHTS OF SECTIONS ① THROUGH ⑨

* TOTAL WEIGHT OF GATE = 4151 lb

≈ 4200 lb.

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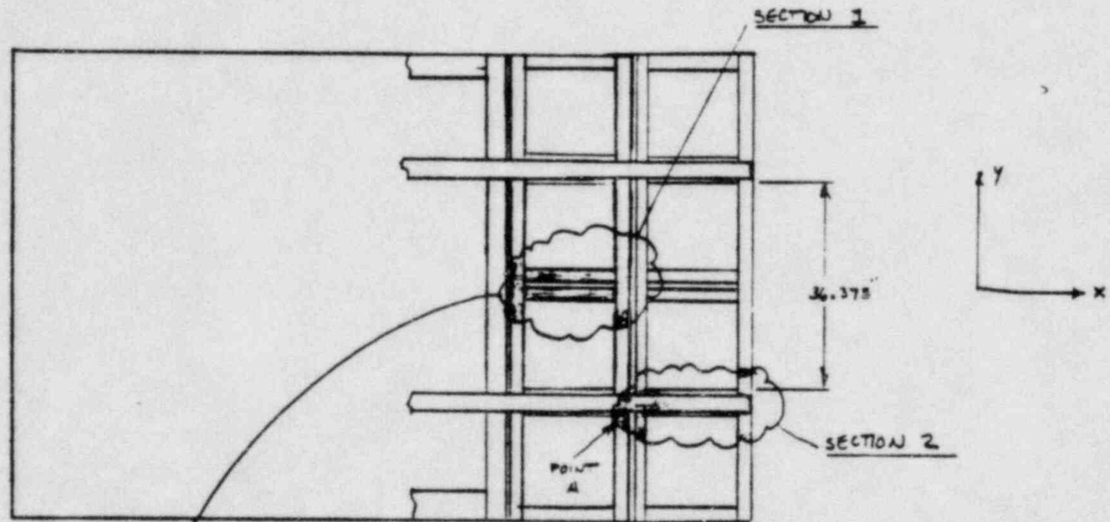
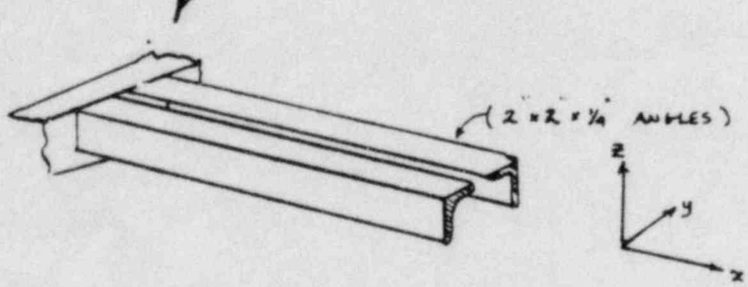
GATE DROP ONTO 1ST SET OF FUEL RACKS

GATE TO BE DROPPED ON SECTIONS OF RACK INDICATED BELOW (SEE REF. #3 FOR RACK DETAILS)



RACK "C" (TOP) (FIGURE I)



AS THE WIDTH OF THE GATE, NEGLECTING THE 1½ × ¼ SECTIONS AROUND THE PERIMETER, IS 37" AND THE BOTTOM CORNERS ARE ROUNDED, IT CAN BE ASSUMED THAT THE FULL IMPACT WILL BE TAKEN BY THE TWO ANGLES SHOWN ABOVE (SEE FIGURE II, PG. (18)). THE GATE IS ASSUMED TO LIE IN THE Y-Z PLANE AND IMPACT THE MID-SPAN OF THE ANGLES.

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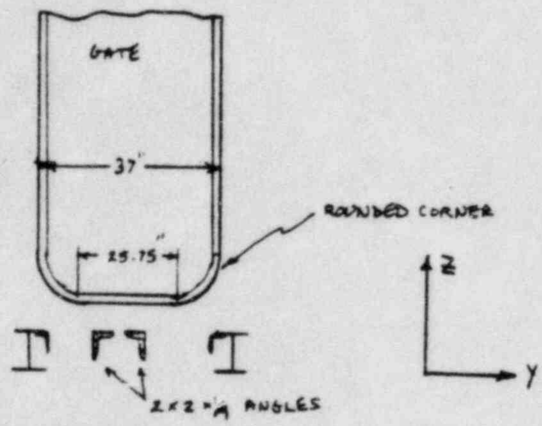

FIGURE II

DROP HEIGHT

FROM REFERENCES (3 & 10) THE HEIGHT FROM THE TOP OF THE FUEL RACKS TO THE TOP OF THE POOL IS FOUND TO BE 30 FEET. AS THE GATE WILL ONLY BE LIFTED OUT OF THE CANAL JUST HIGH ENOUGH TO ALLOW IT TO BE MOVED (~6"), 3 FEET IS ADDED TO THE DROP HEIGHT FOR CONSERVATISM. THIS BRINGS THE TOTAL DROP HEIGHT TO 33 FEET.

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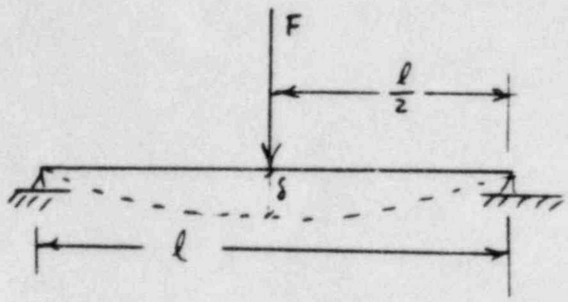
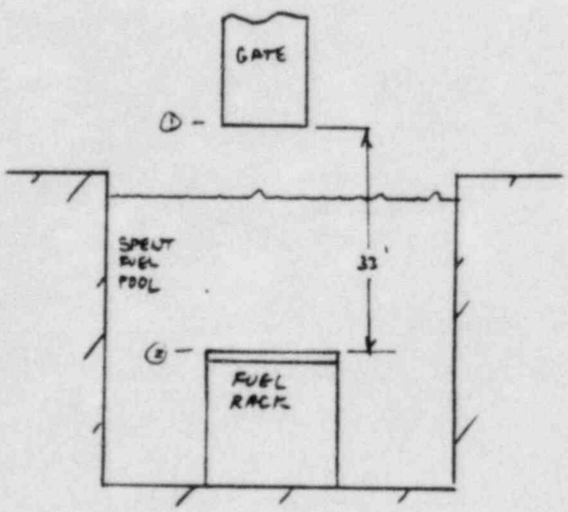
SECTION OF RACK "C" (TOP) WHERE GATE COULD HIT.

MODELED AS A SIMPLY-SUPPORTED BEAM



$l = 15.25''$ (REF # 3)



DROP HEIGHT : 33'

WEIGHT OF GATE : 4200 lb.

(NEGLECT DRAG & BOUANCY EFFECTS OF WATER)

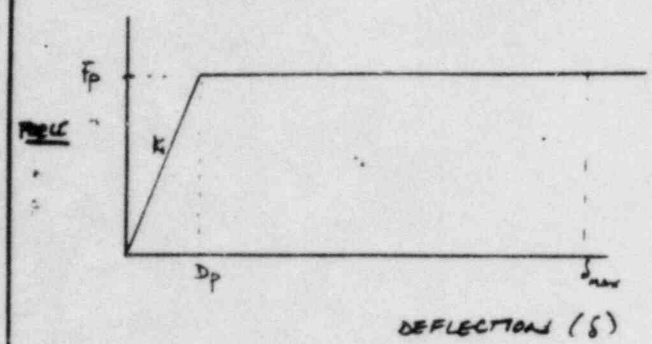
POTENTIAL ENERGY @ ① = KINETIC ENERGY @ ②

$P.E._{①} = mgh = 4200\ (33) = \underline{138\ 600\ \text{FT-lb}}$

VELOCITY @ IMPACT $= \sqrt{2gh} = \sqrt{2(32.2)\,33}$

$= \underline{46\ \text{FT}/\text{SEC}}$ (31 mph)

$P.E._{①} = \underline{1.663 \times 10^{6}\ \text{IN-lb}}$



$K_1 = \frac{F}{\delta}$ $\delta = \frac{F l^3}{48EI}$

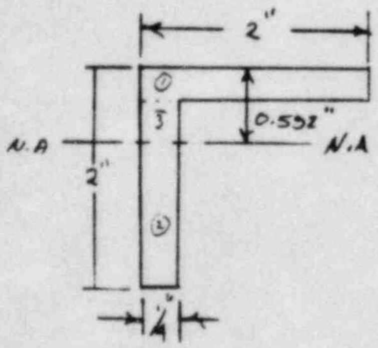
$\therefore K_1 = \frac{48EI}{l^3}$

$D_p = F_p / K_1$

$F_p = 4M_p / l$ (SIMPLY SUPPORTED BEAM)

(FOR A SIMPLY-SUPTED BEAM. $M_{max} = \frac{Fl}{4}$. LET $M_{max} = M_p$, $F = F_p = 4M_p/l$)

$M_p = \sigma_y \int y \, dA = \sigma_y$ (MOMENT OF AREA ABOUT NEUTRAL AXIS)



MOMENTS OF AREA

SECTION ①:

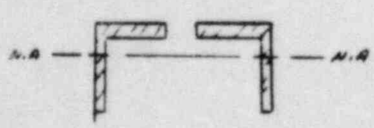
$(0.592 - 0.125)(0.25(2)) = 0.233 \text{ in}^3$

SECTION ②:

$(0.704)(0.25(1.408)) = 0.247 \text{ in}^3$

SECTION ③: $(0.171)(0.342 \cdot 0.25) = 0.014 \text{ in}^3$

Σ ① + ② + ③ = 0.233 + 0.247 + 0.014 = 0.494



ACTUAL BEAM CROSS-SECT.

THE ACTUAL BEAM CROSS-SECTION IS MADE UP OF TWO ANGLES, AS SHOWN AT THE LEFT, THEREFORE THE MOMENT OF THE AREA IS TWICE THE ABOVE VALUE.

i.e. $2(0.494) = \underline{0.988}^{3}$

$M_p = \sigma_y (0.988)$ (IN-lb)

$F_p = 4\sigma_y(0.988)/15.25 = 0.259\ \sigma_y$ (lb.)

(ASSUME 304 S.S → 27.5 KSI @ 150°F)

THEN

$M_p = (27.5 \times 10^3)\ 0.988 = \underline{27,170}$ in-lb

$F_p = 0.259(27.5 \times 10^3) = \underline{7122}$ lb.

$K_1 = \dfrac{48 E I}{l^3}$ $E = 27.9 \times 10^6$ - psi (304 @ 152°F)

$I = 2(0.384) = 0.768$ in^4

$\therefore K_1 = 48(27.9 \times 10^6)\ 0.768 / 15.25^3 = \underline{290\ 000}$ lb/in

$D_p = F_p / K_1 = \underline{0.024}$ inches

FOR ENERGY BALANCE:

$\frac{1}{2} D_p F_p + F_p(\delta_{max} - D_p) = 1.663 \times 10^6$ in-lb

SOLVING FOR δ_{max}

$$\delta_{max} = \frac{1.663 \times 10^6 - \frac{1}{2}(0.024)(7122)}{7122} + 0.024$$

$= \underline{233}$ INCHES

BECAUSE OF THE EXTREMELY LARGE δ_{max}, A COMPLETE FAILURE OF THE SECTION IN QUESTION CAN BE ASSUMED.

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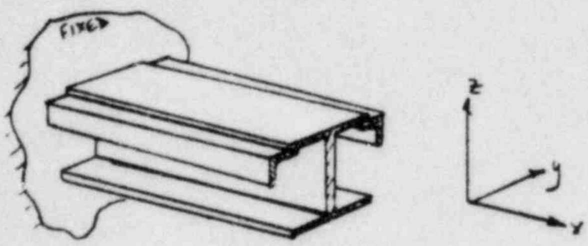
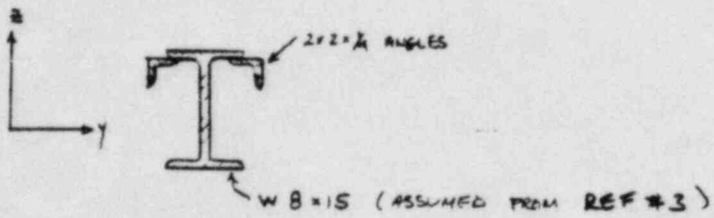
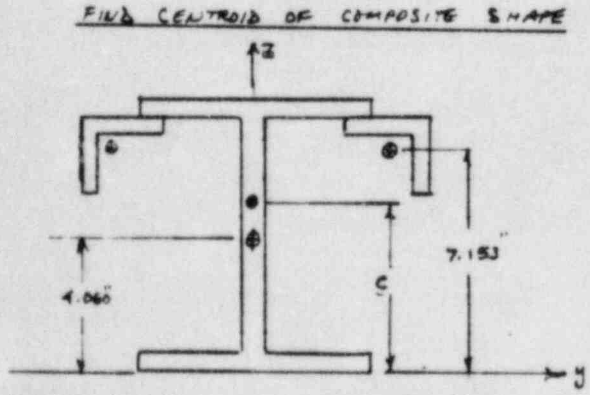
SECTION 2 OF FIGURE I





SECTION 2 IS ANALYZED AS A CANTILEVER BEAM DUE TO THE SUPPORT COLUMN LOCATED AT POINT A IN FIGURE I. LOCAL BUCKLING OF THE BEAM AND BUCKLING OF THE SUPPORT COLUMN ARE IGNORED. ANGLES WELDED TO THE END OF THE BEAM IN THE ±Y DIRECTION ARE IGNORED

FIND CENTROID OF COMPOSITE SHAPE



$$C = \frac{4.06(4.43) + 2(7.153)0.938}{4.43 + 2(0.938)}$$

$$= 4.980''$$

CALCULATE I

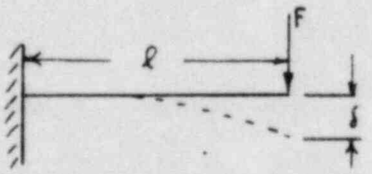
$$I_{\text{neutral axis}} = 18.1 + 4.43\,(0.32)^2 + 2\left[0.348 + 0.938\,(2.173)^2\right]$$

$$= \underline{61.4 \text{ in}^4}$$

MOMENT OF AREA ABOUT NEUTRAL AXIS

$$2\left[2.173\,(0.938)\right] + \left(0.3125\,(4.015)\,2.983\right) + (2.827)(0.245)(1.413) +$$

$$4.667\,(0.245)\,2.334 + \left(0.3125\,(4.015)\,4.823\right) = \underline{17.518 \text{ in}^3}$$



$\delta = Fl^3/3EI \qquad l = 21.1875"$

$M_p = \sigma_y$ (MOM. OF AREA ABOUT NEUT. AXIS) $= 27.5 \times 10^3\,(17.518) = \underline{481\,745 \text{ in-lb}}$

FOR A CANTILEVER BEAM, THE

MAXIMUM MOMENT $M_{max} = Fl$

LET $M_{max} = M_p$ FOR $F \equiv F_p$

WE HAVE $F_p = M_p/l$

$F_p = M_p/l$ (CANTILEVER BEAM)

$= 481\,745/21.1875$

$= \underline{22\,737 \text{ lb.}}$

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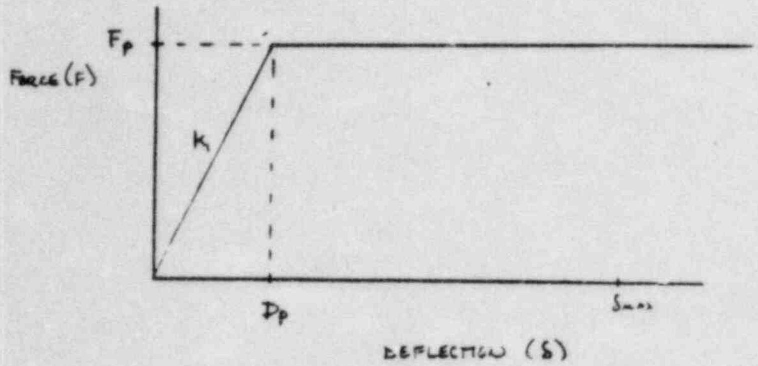

$$K_1 = F/\delta = \frac{3EI}{\ell^3}$$

$$= 3(27.9\times10^6)(61.4)/21.1875^3$$

$$= \underline{540\ 329\ \tfrac{lb}{in}}$$

$$D_p = F_p/K_1 = \frac{22\ 737}{540\ 329}$$

$$= \underline{0.042\ in}$$

FOR ENERGY BALANCE:

$$\tfrac{1}{2} D_p F_p + F_p(\delta_{max} - D_p) = 1.663\times10^6 \text{ in-lb.}$$

SOLVING FOR δ_{max}:

$$\delta_{max} = \frac{1.663\times10^6 - \tfrac{1}{2}(0.042)(22\ 737)}{22\ 737} + 0.042$$

$$= \underline{73.2''}$$

AGAIN, DUE TO THE EXTREMELY LARGE δ_{max} CALCULATED A COMPLETE FAILURE OF THE BEAM IS ASSUMED

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ANALYSIS AT THIS POINT SHOWS THAT THE FUEL STORAGE RACKS CANNOT WITHSTAND A DROP FROM THE CANAL GATE. THE TWO SECTIONS OF THE RACK THAT WERE ANALYZED WERE SHOWN TO BE INCAPABLE OF ABSORBING THE IMPACT ENERGY. THESE SECTIONS WOULD EITHER TEAR OUT OR BEND OUT OF THE WAY; ALLOWING THE GATE TO CONTINUE DOWN THROUGH THE RACK.

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GATE DROP ONTO HIGH DENSITY FUEL RACKS

THE HIGH DENSITY FUEL RACKS ARE MADE UP OF INDIVIDUAL CELLS, WITH EACH CELL MODELED AS A LONG, THIN-WALLED SQUARE TUBE. BECAUSE OF THE THIN WALL'S AND THE TACK WELD CONSTRUCTION (SEE REF'S #4 & #5), IT IS NOT EXPECTED THAT THE RACKS WILL BE ABLE TO WITHSTAND THE IMPACT BY THE GATE. FOR THIS REASON AN ENERGY BALANCE IS USED TO CALCULATE THE TOTAL DEFORMATION OF THE RACKS WITH THE ASSUMPTION THAT NO LOCAL BUCKLING OR TEARING OCCURS. BY NEGLECTING LOCAL CRUSH AND TEARING, AND ONLY CONSIDERING UNIAXIAL DEFORMATION OF THE THIN WALLED SECTIONS, THE DEFORMATION ARRIVED AT THROUGH THE ENERGY BALANCE APPROACH WILL REPRESENT A <u>LOWER BOUND</u> ON THE ACTUAL DEFORMATION.

BY CONSIDERING THE GEOMETRY OF THE GATE (REF. 2) AND THE GEOMETRY OF THE RACK (REF. 5) IT CAN BE SHOWN THAT AN END ON IMPACT, AS SHOWN IN FIGURE (III), WILL AFFECT 4 FUEL CELLS (SEE FIGURE (III)). AS CAN BE SEEN IN FIG (III), 6 WALL SECTIONS OF THE 4 TUBES ARE HIT. WITH THIS INFORMATION WE PROCEED AS FOLLOWS:

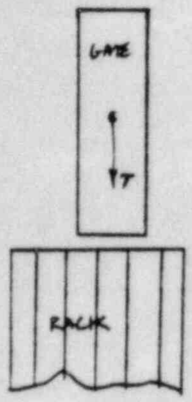

FIGURE (III)

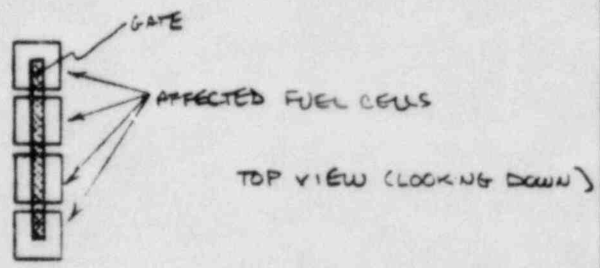

FIGURE (IV)

OF THE SIX SIDES AFFECTED WE ONLY CONSIDER THE MATERIAL DIRECTLY UNDER THE GATE, i.e. THE THICKNESS OF THE GATE (GATE WIDTH = 4" (REF #2)).

THICKNESS OF WALL = 0.12"

TOTAL # OF WALLS AFFECTED = 6

WIDTH OF AFFECTED WALL UNDER CONSIDERATION = 4"

LENGTH OF TUBE = 163"

$$AREA = 0.12(6)4 = 2.88\ in^2$$

∴ EQUIVALENT MODEL FOR IMPACT ANALYSIS IS A ROD OF CROSS-SECTIONAL AREA 2.88 in^2, 163" LONG.

CALCULATE THE YIELD FORCE

$$F_y = \sigma_y A = 27.5\,(2.88) = 79.2\ KIPS$$

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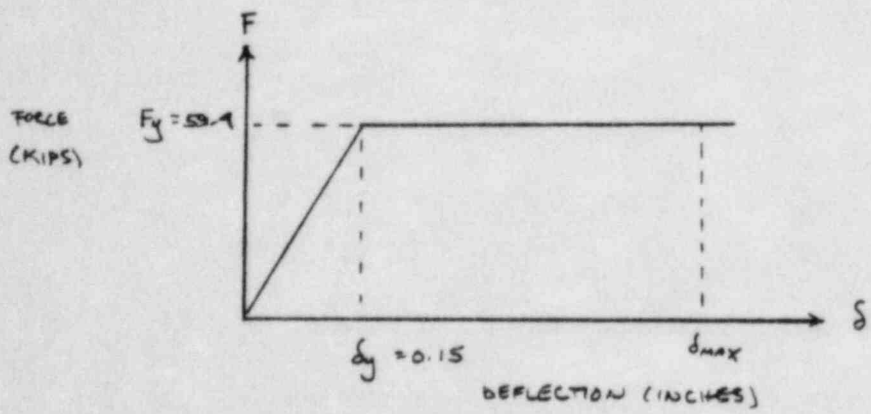
CALCULATE DEFLECTION AT F_y (δ_y)

$$\delta_y = F_y \ell / AE \qquad \ell = \text{LENGTH OF EQUIVALENT ROD} = 163''$$

$$= 79.2(163) / 2.08(30 \times 10^3)$$

$$= \underline{0.15 \text{ INCHES}}$$



SETTING UP THE ENERGY BALANCE EXPRESSION AND SOLVING FOR δ_{max} YIELDS

$$\delta_{max} = \frac{K.E. + \frac{1}{2}\delta_y F_y}{F_y - 9200}$$

$$= \frac{1.663 \times 10^6 + \frac{1}{2}(0.15)\,79200}{79200 - 9200}$$

$$= \underline{22.1 \text{ INCHES}}$$

SIMILARLY, FOR AN EDGE IMPACT LENGTHWISE (WHERE THE LONG-EDGE IMPACTS) IT CAN BE DETERMINED FROM THE GEOMETRY OF THE GATE AND RACK (REF'S (2) & (5)) THAT 32 CELLS COULD BE DIRECTLEY HIT. FOLLOWING THE SAME

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PROCEDURE AS BEFORE :

THICKNESS OF WALL = 0.12"

TOTAL # OF WALLS AFFECTED = 32 (2) = 64

WIDTH OF AFFECTED WALL = 4"

TOTAL LENGTH OF TUBE = 163"

AREA = 0.12 (64) 4 = 30.7 IN^2

∴ EQUIVALENT MODEL FOR IMPACT ANALYSIS IS A ROD OF CROSS-SECTIONAL AREA 30.7 IN^2 , 163 IN. LONG

YIELD FORCE

$$F_y = \sigma_y A = 27.5(30.7) = 849.8 \text{ KIP}$$

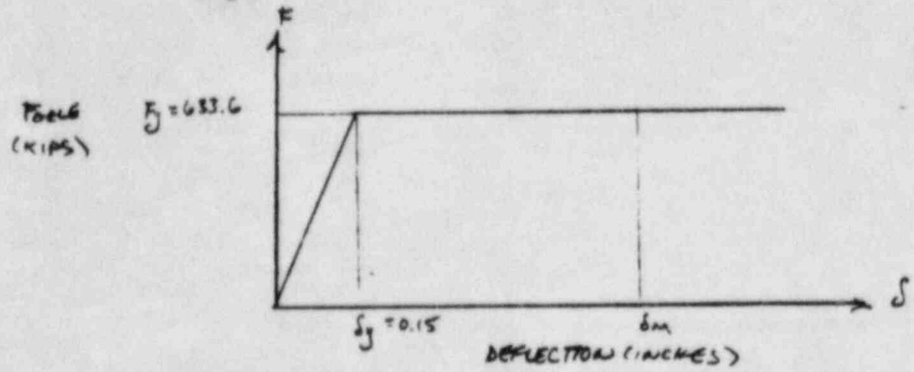
DEFLECTION AT F_y (δ_y)

$$\delta_y = F_y \ell / AE$$

$$= 849.8 (163) / 30.7 (30 \times 10^3)$$

$$= 0.15 \text{ INCHES}$$

THEN



SETTING UP THE ENERGY BALANCE EXPRESION AND SOLVING FOR δ_{max} YIELDS:

$$\delta_{max} = \frac{K.E. + \frac{1}{2}\delta_y F_y}{F_y - 4200}$$

$$= \frac{1.663 \times 10^6 + \frac{1}{2}(0.15)\, 844\,800}{844\,800 - 4200}$$

$$= \underline{2.05''}$$

AS STATED EARLIER, BOTH OF THE DEFORMATIONS REPRESENT LOWER BOUNDS FOR THE ACTUAL DEFORMATIONS AS ALL LOCALIZED EFFECTS HAVE BEEN IGNORED. IN ACTUALLITY, THE MODE OF FAILURE WOULD PROBABLEY BE LOCAL BUCKLING OF THE TUBE SECTION WHICH WOULD RESULT IN MUCH LESS ENERGY DISIPATION AND FAR GREATER DEFORMATIONS.

THE POSSIBILLITY ALSO EXISTS THAT THE GATE COULD IMPACT BETWEEN THE FUEL CELLS, RATHER THAN DIRECTLEY ON TOP OF THEM. THE GATE WEDGING ITSELF BETWEEN THE FUEL CELLS WOULD FORCE THE ADJACENT CELLS UP AGAINST OTHER CELLS. THIS COULD POSSIBLY PRESENT A CRITICALITY PROBLEM. FOR THE SMALL END IMPACT DESCRIBED FIRST, 8 CELLS COULD BE DISPLACED AND FORCED UP AGAINST OTHER CELLS. FOR THE LONG EDGE IMPACT, 64 CELLS WOULD BE DISPLACED.

A THIRD POSSIBILITY EXISTS AS WELL. THAT IS THAT THE GATE COULD LAND FLAT ON TOP OF THE RACK AND BLOCK THE CIRCULATION THROUGH 128 (4 x 32) CELLS.

AS A FURTHER NOTE, A COMPARISON OF THE FUEL BUNDLE LENGTH WITH THE HEIGHT OF THE RACK (SEE REF'S (5) & (6)) SHOWS THAT THE FUEL BUNDLE STICKS OUT OF THE TOP OF THE RACK BY 2".

LENGTH OF FUEL BUNDLE = 165 5/8"

LENGTH OF FUEL CELL = 163 5/8"

THIS FURTHER INCREASES THE LIKELYHOOD OF FUEL BUNDLE DAMAGE DUE TO A DROP, AS THERE IS A HIGH PROBABILITY ANYTHING DROPPED ON THE RACK WILL IMPACT THE FUEL BUNDLES FIRST.

THE ABOVE INFORMATION ON THE GATE IMPACTING THE HIGH DENSITY RACKS WAS TRANSMITTED TO POWER FOR THE PURPOSE OF DETERMINING WHETHER OR NOT THE POSTULATED DAMAGE TO THE FUEL IS ACCEPTABLE.

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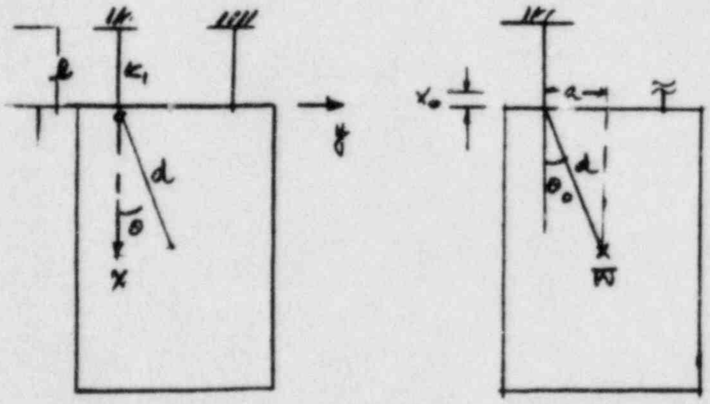
GATE SWING

POSTULATED ACCIDENT : ONE OF THE TWO HOIST CABLES SNAP

OBJECTIVE : TO FIND THE MAXIMUM FORCE IN THE REMAINING CABLE.



$x_0 = \frac{W}{2K_1}$

$\theta_0 = \frac{a}{d}$

INITIAL CONDITION

SINCE $l \ll d$ AND THE MOTION OF THE BODY IS RELATIVELY SMALL, WE ASSUME THE x AND θ MOTIONS CAN BE DECOUPLED.

$W = 4200$ lbs

$a = 13.5$ INCHES *

$d = 168.5$ INCHES *

(1) FOR θ MOTION,

* ALL GATE DIMENSIONS ARE FROM REF. 2

$$I\ddot{\theta} + W d\theta = 0$$

THE INITIAL CONDITIONS ARE

$$\theta(t=0) = \theta_0 \cong a/d$$

$$\dot{\theta}(t=0) = 0$$

WHICH IS A PENDULUM EQUATION. THE MAXIMUM CABLE'S TENSION OCCURS WHEN THE PENDULUM ATTAINS ITS MAXIMUM VELOCITY, i.e.

$$\frac{1}{2} I \dot{\theta}^2 = W h = W d(1-\cos\theta_0) \rightarrow \dot{\theta}^2 = 2 W d(1-\cos\theta_0)/I$$

AND THE MAXIMUM TENSION IN THE CABLE DUE TO CENTRIFUGAL FORCE IS

$$\Delta T = \frac{W}{g} d\dot{\theta}^2 = \frac{2W^2 d^2}{gI}(1-\cos\theta_0).$$ DETAILED CALCULATION FOLLOWS.

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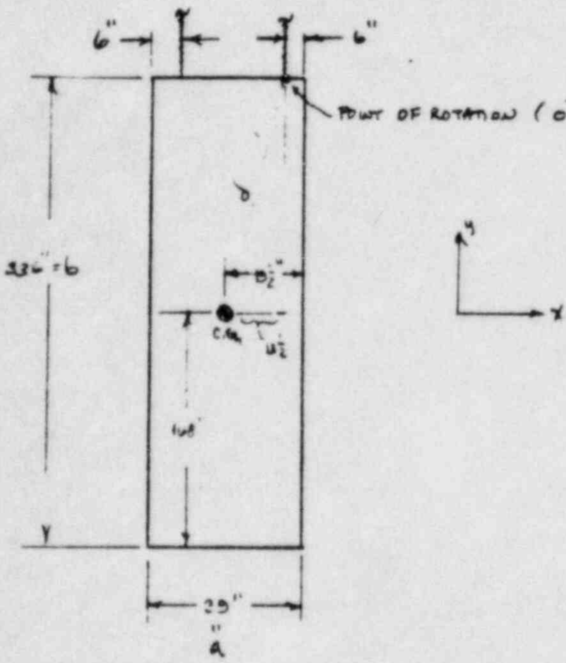
GATE SWING

WEIGHT = 4200 lb.



$d = \sqrt{13.5^2 + 168^2} = 168.5"$

MASS MOMENT OF INERTIA ABOUT POINT OF ROTATION

$I_o = I_{CG_z} + m d^2$

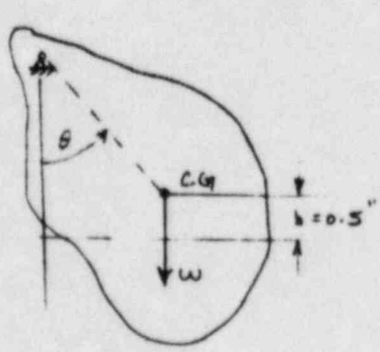
$I_{CG_z} = \frac{1}{12} m (b^2 + a^2)$

$= \frac{1}{12}\left[\frac{4200}{32.2(12)}\right](336^2 + 29^2)$

$= 103\ 638 \quad in \cdot lb\ s^2$

$I_{o_z} = 103\ 638 + \left(\frac{4200}{32.2(12)}\right)(168.5)^2$

$= \underline{412\ 401 \quad in \cdot lb \cdot s^2}$



KINETIC ENERGY = TRANSLATIONAL (K.E.) + ROTATIONAL (K.E.)

$K.E. = \frac{1}{2} m (d\dot{\theta})^2 + \frac{1}{2} I_o (\dot{\theta})^2$

$= \frac{1}{2}(\dot{\theta})^2 (I_o + m d^2)$ WHERE $I = I_o + m d^2$

$K.E. = P.E = wh$

$\therefore \frac{1}{2}(\dot{\theta})^2 I = wh$

$$\therefore \quad \dot{\theta} = \sqrt{2Wh/I} = \sqrt{2\,(4200)\,0.5 / 412\,401}$$

$$= 0.1 \ ^{RAD}/_{S}$$

<u>FORCE DUE TO ROTATION</u>

$$\Delta T = m\,d\,\omega^2$$

$$= \left(\frac{4200}{386.4}\right) 168.5\,(0.1)^2$$

$$= \underline{18.31 \text{ lbs}} \approx 0$$

WHICH IS NEGLEGIBLE.

(2) FOR x MOTION

$$M\ddot{x} + K_1 x = W$$

THE INITIAL CONDITIONS ARE

$$x(t=0) = \frac{\overline{W}}{2K_1}$$

$$\dot{x}(t=0) = 0$$

THE SOLUTION TO THE ABOVE EQUATION IS

$$x = A\cos\omega t + B\sin\omega t + \frac{\overline{W}}{K_1} \quad , \quad \omega = \sqrt{\frac{K_1}{\overline{W}}}$$

FROM $x(t=0) = \frac{\overline{W}}{2K_1}$

$$\frac{\overline{W}}{2K_1} = A + \frac{W}{K_1} \longrightarrow A = -\frac{\overline{W}}{2K}$$

FROM $\dot{x}(t=0) = 0$ $\qquad B = 0$

$$\therefore \quad x = \frac{W}{K_1} - \frac{\overline{W}}{2K_1}\cos\omega t$$

THE MAXIMUM x IS

$$(x)_{max} = \frac{W}{K_1} - \frac{\overline{W}}{2K_1}(-1) = \frac{3\overline{W}}{2K_1}$$

THEREFORE, THE MAXIMUM (CABLE FORCE) DUE TO TRANSLATIONAL MOTION IS

$$R = K_1 (x)_{max} = \frac{3\overline{W}}{2}$$

$\therefore$ $R = \frac{3(4200)}{2} = 6300$ lb.

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THE MAXIMUM FORCE, 6300 lb., IS 300 lb. GREATER THAN THE RATED CAPACITY OF THE HOIST, 6000 lb. BUT, AS HOISTS ARE DESIGNED WITH FACTORS OF SAFETY OF ~5 OR 6, AND THAT THE POSTULATED EVENT IS A ONE-TIME OCCURANCE, IT CAN BE CONCLUDED THAT A SINGLE HOIST CAN SUFFICIENTLY HANDLE THE MAXIMUM LOAD.

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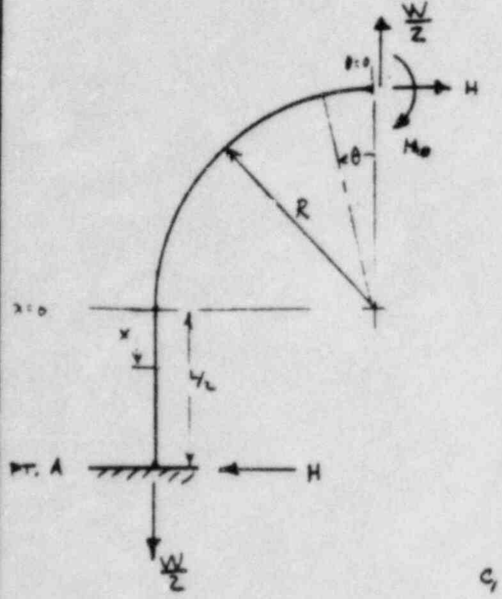
LIFTING HOOKS ON GATE



$$M_\theta = M_0 - \frac{WR}{2}\sin\theta + HR(1-\cos\theta) \quad (1)$$

$$M_x = M_0 - \frac{WR}{2} + H(R+x) \quad (2)$$

WHERE:

$$M_0 = \frac{WRC_1}{2} \quad (3)$$

$$H = \frac{WC_2}{2} \quad (4)$$

$$C_1 = \frac{(k+2)[k^3+6k^2+12k(4-\pi)+48(\pi-3)]}{k^4+4\pi k^3+48k^2+24\pi k+24(\pi^2-8)} \quad (5)$$

[EXPRESSIONS (1)-(7) FROM REF #7]

$$C_2 = \frac{12(k+2)[(\pi-2)k+2(4-\pi)]}{k^4+4\pi k^3+48k^2+24\pi k+24(\pi^2-8)} \quad (6)$$

$$k = L/R \quad (7)$$

W = 6300 lb.

FIRST WE CHECK THE ADEQUACY OF THE EXISTING LIFT HOOKS (REF. 2)

R = 1.75"

L = 2"

DIAMETER = 0.5"

$k = \frac{L}{R} = \frac{2}{1.75} = 1.143$

$$C_1 = \frac{(1.143+2)[1.143^3+6(1.143)^2+12(1.143)(4-\pi)+48(\pi-3)]}{1.143^4+4\pi(1.143)^3+48(1.143)^2+24\pi(1.143)+24(\pi^2-8)}$$

$$= \frac{87.67}{219.18} = 0.400$$

$$C_2 = \frac{12(1.143+2)[(\pi-2)1.143+2(4-\pi)]}{219.18} = 0.580$$

<u>THEN</u>

$$M_0 = \left(\frac{6300}{2}\right) 1.75\ (0.409) = \underline{2254}\ \text{in-lb.}$$

$$H = \left(\frac{6300}{2}\right) 0.580 = \underline{1827\ \text{lb}}$$

THE MAXIMUM STRESS AT $\theta = 0$ OCCURS AT THE INSIDE FIBER.

FROM REF (#7 pg. 333) WE GET

$$\sigma_{\theta=0} = \left(\frac{M_0 C}{I} + \frac{H}{A}\right) \phi_0$$

WHERE: ϕ_0 IS THE CORRECTION FACTOR FOR THE INSIDE FIBER OF A CURVED BEAM.

FROM REF (#7 pg. 334) : $\underline{\phi_0 = 1.14}$

THEN:

$$\sigma_{\theta=0} = \left(\frac{2254\ (0.25)}{(\pi/4)\ (0.25)^3} + \frac{1827}{\pi (0.25)^2}\right) 1.14$$

$$= (45\ 930 + 9304)\ 1.14 = \underline{62\ 967\ \text{psi}}$$

<u>THE OTHER POINT OF CONCERN IS POINT A ($x = 1''$). FOR THIS CASE WE HAVE:</u>

$$M_{x=1} = M_0 - \frac{wR}{2} + H(R+1)$$

$$= 2254 - \frac{6300\ (1.75)}{2} + 1827\ (2.75)$$

$$= \underline{1766\ \text{in-lb}}$$

$$\sigma_{x=1} = \left(\frac{M_{x=1}\ C}{I} + \frac{(P/2)}{A}\right)$$

$$= \left(\frac{1827\ (0.5)}{(\pi/4)\ (0.5)^3} + \frac{6300}{2(\pi)\ (0.5)^2}\right) = \underline{13\ 314\ \text{psi}}$$

USING THE CRITERIA THAT $\sigma_{MAX} < 0.6\,\sigma_y$ (16,500 psi) WE CAN SEE THAT $\sigma_{\theta=0} \gg 16.5$ KSI. THEREFORE WE MUST RESIZE THE HOOKS.

WE WANT TO MAINTAIN A $1\frac{1}{2}''$ INSIDE DIAMETER OF THE HOOK. WE THEN TRY A 1" DIAMETER ROD.

THEN

$$L = 2'' \quad , \quad R = 2''$$

$$k = \frac{L}{R} = 1$$

$$C_1 = \frac{(1+2)\left[1+6+12(4-\pi)+48(\pi-3)\right]}{1+4\pi+48+24\pi+24(\pi^2-8)}$$

$$= \frac{72.292}{181.835} = \underline{0.397}$$

$$C_2 = \frac{12(1+2)\left[(\pi-2)+2(4-\pi)\right]}{181.835}$$

$$= \frac{102.902}{181.835} = \underline{0.566}$$

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$M_0 = \left(\frac{6300}{2}\right) 2 (0.397) = \underline{2501 \text{ in-lb}}$

$H = \left(\frac{6300}{2}\right)(0.566) = \underline{1782 \text{ in-lb}}$

THE EXPRESSION FOR THE STRESS AT $\theta = 0°$ IS GIVEN BY (AS BEFORE)

$\sigma_{\theta=0°} = \left(\frac{M_0 C}{I} + \frac{H}{A}\right) \phi_{0 \text{ inside curve}}$

WHERE: $\phi_{0 \text{ inside curve}}$ IS THE CORRECTION FACTOR FOR THE INSIDE RADIUS OF A CURVED BEAM

THEREFORE, FOR A CIRCULAR CROSS SECTION OF 1" DIAMETER, R = 2" WE HAVE: $\underline{\phi_0 = 1.23}$ (REF #7, PG. 334)

THEN,

$$\sigma_{\theta=0} = \left(\frac{2501 (0.5)}{(\pi/4)(0.5)^3} + \frac{1782}{\pi (0.5)^2}\right) 1.23$$

$$= (12\,737 + 2269)\, 1.23 = \underline{18\,458 \text{ PSI}}$$

THE STRESS AT X = 1" IS GIVEN BY:

$$\sigma_{x=1"} = \left(\frac{M_x C}{I} + \frac{(\frac{W}{2})}{A}\right)$$

$$M_x = M_0 - \frac{wc}{2} + H(R + x)$$

$$= 2501 - \frac{6300(2)}{2} + 2375(3)$$

$$= \underline{3326 \text{ in-lb}}$$

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THEN, THE STRESS AT $x = 1''$ IS :

$$\sigma_{x=1} = \left(\frac{3320\,(0.5)}{\pi/4\,(0.5)^3} + \frac{6300}{2\pi\,(0.5)^2} \right)$$

$$= 16,939 + 4010 = \underline{\underline{20,949 \text{ PSI}}}$$

BASED ON THE CRITERION THAT THE MAXIMUM STRESS MUST BE LESS $0.6\,\sigma_y$ WE HAVE :

$$\sigma_{x=0} + \sigma_{x=1''} > 16,500 \text{ PSI}$$

THEREFORE, FOR $R = 2''$ & $D = 1''$ WE HAVE NOT SATISFIED THE $0.6\,\sigma_y$ CRITERIA.

<u>WE NEXT TRY $D = 1.25''$. WE ALSO WISH TO MAINTAIN AN INSIDE RADIUS OF $1.5''$ $\therefore$ $R = 2.125''$</u>

$$\frac{L}{R} = \frac{2}{2.125} = 0.9411 = k \qquad \phi_0 = 1.33 \text{ (REF \#7, PG. 334)}$$

THEN

$$C_1 = \frac{(0.9411+2)\left[0.9411^3 + 6(0.9411)^2 + 12(0.9411)(4-\pi) + 48(\pi-3)\right]}{0.9411^4 + 4\pi(0.9411)^3 + 48(0.9411)^2 + 24\pi(0.9411) + 24(\pi^2-8)}$$

$$= \frac{56.739}{169.613} = \underline{0.333}$$

$$C_2 = \frac{2(0.9411+2)\left[(\pi-2)\,0.9411 + 2(4-\pi)\right]}{169.613}$$

$$= \frac{98.511}{169.613} = 0.580$$

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THEN, AS BEFORE

$$M_0 = \left(\frac{6300}{2}\right) 2.125\ (0.393) = \underline{2630}\ \text{IN-lb.}$$

$$H = \left(\frac{6300}{2}\right) 0.580 = \underline{1827}\ \text{lb.}$$

<u>THE STRESS AT $\theta = 0^\circ$ IS:</u>

$$\sigma_{\theta=0} = \left(\frac{2630\ (0.625)}{(\pi/4)\ (0.625)^3} + \frac{1827}{\pi (0.625)^2} \right) 1.33$$

$$= (8572 + 1488)\ 1.33$$

$$= \underline{13\ 380\ \text{PSI}}$$

<u>THE STRESS AT $x = 1''$ IS:</u>

$$\sigma_{x=1''} = \left(\frac{M_x\, C}{I} + \frac{(W/2)}{A} \right)$$

$$M_x = 2630 - \frac{6300\ (2.125)}{2} + 1827\ (3.125)$$

$$= 1645\ \text{IN-lb}$$

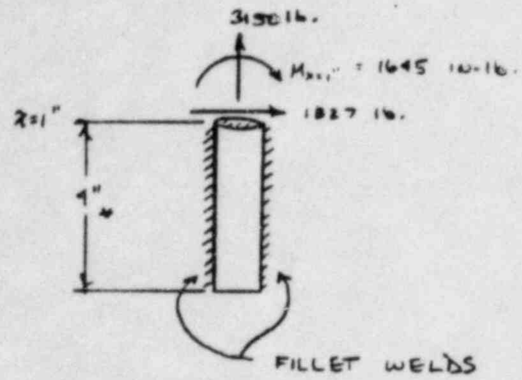
$$\sigma_{x=1''} = \left(\frac{1645\ (0.625)}{\frac{\pi}{4}\ (0.625)^3} + \frac{(6300/2)}{\pi (0.625)^2} \right)$$

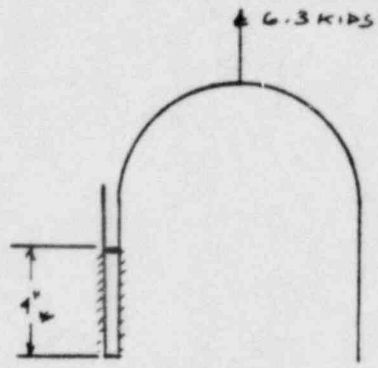
$$= (5363 + 2566)$$

$$= \underline{7929\ \text{PSI}}$$

THE STRESSES ARE BELOW 16.5 KSI $\therefore$ $D = 1.25''$ & $R = 2.125''$ ARE ADEQUATE.

CHECK THE WELDS





* (FROM REF. 2)

FOR 2 FILLET WELDS

$$A = [0.707\,W\,(4)]\,2 = 5.65\,W$$

$$I = \tfrac{1}{12}(0.707)(2)\,w\,(4)^3 + 2\,(0.707\,w)\,(4)\,(0.625)^2$$

$$= 7.54 + 2.2 = 9.54\,w$$

$$f_1 = \frac{3150}{5.65\,w} + \frac{(1645 + 1827(2))\,0.625}{9.54\,w} = \frac{557.5}{w} + \frac{347.1}{w}$$

$$= \frac{904.6}{w}$$

$$f_2 = \frac{1827}{5.65\,w} + \frac{(1645 + 1827(2))\,2}{9.54\,w} = \frac{323}{w} + \frac{1110}{w}$$

$$= \frac{1433}{w}$$

$\sqrt{f_1^2 + f_2^2} = \frac{1694}{w}$, SET THIS EQUAL TO 18×10^3 PSI (REF 8, PG. 5-21)

THEN : $w = 0.094$ INCHES

USE $w = 1/4$"

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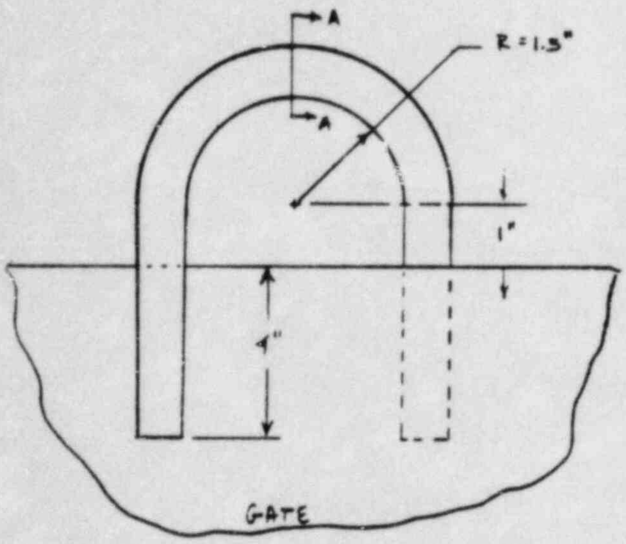
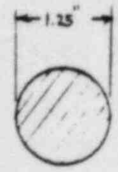
THEREFORE, THE FINAL CONFIGURATION FOR THE HOOK IS AS FOLLOWS:





SECTION A-A

USE 1/4" FILLET WELDS

ENCLOSURE 5

POSTULATED DROP OF 7,000-lb JACK
ONTO SPENT FUEL POOL MISSILE SHIELDS

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION TITLE PAGE

*SEE INSTRUCTIONS ON REVERSE SIDE

5010.64 (FRONT)

CLIENT & PROJECT	PAGE 1 OF 33
FLORIDA POWER / CRYSTAL RIVER - UNIT 3	
CALCULATION TITLE (Indicative of the Objective): POSTULATED DROP OF 7,000 POUND JACK ONTO SPENT FUEL POOL MISSLE SHIELDS	QA CATEGORY (✓) ☒ I - NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ____ OTHER

CALCULATION IDENTIFICATION NUMBER

J.O. OR W.O. NO.	DIVISION & GROUP	CURRENT CALC. NO.	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
14235.17	NM (C)	14235.17-M-07	—	—

* APPROVALS - SIGNATURE & DATE: [signature] 7/21/83

PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)	REV. NO. OR NEW CALC NO.	SUPERSEDES * CALC. NO. OR REV. NO.	CONFIRMATION *REQUIRED (✓) YES	NO
M. DURKEE 7/13/83	H. H KUO 7/15/83	S. M. Feldman 7-20-83	0	14235.17-M-05		—

DISTRIBUTION*

GROUP	NAME & LOCATION	COPY SENT (✓)	GROUP	NAME & LOCATION	COPY SENT (✓)
RECORDS MGT. FILES (OR FIRE FILE IF NONE)	JOB BOOK				
	BOSTON FIRE FILE				
EMD	S. LIU 6/3RD				
EMD	H. H KUO 3G				
EMD	S. FELDMAN 3G				
	E. MICHAELIS 6/3R				
	J. POSUSNEY 50				
	FPC				

STATEMENT OF REVIEW
CALCULATION NUMBER 11235.17-M-07

This calculation has been reviewed in accordance with (EMAG-CH-41-2) and was found to be adequate. The method of review was: (List appropriate items)

a. Review of calculations

b. Comparison with similar calculation (number ____________.)

(c) Number by number check

Hsin-Shien Kuo /7-15-83/0/C
NONINDEPENDENT REVIEWER/DATE/REV./METHOD

NONINDEPENDENT REVIEWER/DATE/REV./METHOD

NONINDEPENDENT REVIEWER/DATE/REV./METHOD

NONINDEPENDENT REVIEWER/DATE/REV./METHOD

NONINDEPENDENT REVIEWER/DATE/REV./METHOD

NONINDEPENDENT REVIEWER/DATE/REV./METHOD

The statement below applies to Nuclear Safety Related QA Category I calculations only.

This calculation has been INDEPENDENTLY reviewed in accordance with EMAG-CH-41-2 and was found to be adequate. The method of review was: (List appropriate items)

a. Comparison with prequalified methods and assumptions

____
prequalified document number(s)

b. Addressing the key questions appearing in Attachment 13.11 of EMAG-CH-41-2

S.M. Feldman /7-20-83/0/b
INDEPENDENT REVIEWER/DATE/REV./METHOD

INDEPENDENT REVIEWER/DATE/REV./METHOD

INDEPENDENT REVIEWER/DATE/REV./METHOD

INDEPENDENT REVIEWER/DATE/REV./METHOD

INDEPENDENT REVIEWER/DATE/REV./METHOD

INDEPENDENT REVIEWER/DATE/REV./METHOD

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CALCULATION SHEET

▲ 5010.65

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OBJECTIVE

THE OBJECTIVE OF THIS CALCULATION IS TO ANALYZE A POSTULATED DROP OF A 7,000 lb. HYDRAULIC JACK, FROM THE MECHANICAL SCAFFOLDING ON TOP OF THE CONTAINMENT BUILDING, ONTO THE SPENT FUEL POOL MISSLE SHIELDS. IT IS DESIRED TO DETERMINE THE ENERGY AT IMPACT, THE POTENTIAL FOR JACK TO DEFORM SHIELD AND FALL INTO THE POOL, AND GENERATE EMBEDMENT LOADS FOR STRUCTURAL EVALUATION.

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ASSUMPTIONS

1) MISSLE SHIELD IS MODELED AS A SIMPLY SUPPORTED BEAM.

2) FOR ENERGY BALANCE APPROACH, BEAM RESPONSE IS ELASTIC- PERFECTLY PLASTIC.

3) THE EFFECTS OF LOCAL BUCKLING ARE NOT CONSIDERED

4) JACK IS MODELED AS A 26" DIAMETER, 5 FOOT LONG CYLINDER.

5) THE JACK IS TAKEN TO IMPACT, ON END, IN THE CENTER OF A SINGLE MISSLE SHIELD.

6) THE ENERGY DISIPATED BY THE JACK PASSING THROUGH THE ROOF OF THE FUEL BUILDING IS NEGLECTED

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<u>METHOD</u>

AN ENERGY BALANCE METHOD, AS A FIRST APPROACH, IS UTILIZED FOR THE MISSLE SHIELD ANALYSIS AS IT IS A SIMPLER, AND MORE CONSERVATIVE APPROACH THAN USING NON-LINEAR LARGE DEFLECTION ANALYSIS. THE ENERGY BALANCE METHOD INVOLVES SETTING THE KINETIC ENERGY AT IMPACT EQUAL TO THE STRAIN ENERGY OF THE TARGET. IN THIS WAY THE MAXIMUM DEFLECTION OF THE MISSLE SHIELD CAN BE DETERMINED. A SLIGHTLY MODIFIED APPROACH IS ALSO UTILIZED WHICH TAKES INTO ACCOUNT STRAIN-RATE EFFECTS AND THE EFFECTIVE MASS OF THE SHIELD.

A NON-LINEAR DYNAMIC ANALYSIS COMPUTER CODE (LIMITA2) IS THEN UTILIZED TO ELIMINATE CONSERVATISM INHERENT IN THE ENERGY BALANCE APPROACH, FOR THE PURPOSE OF MINIMIZING THE MAXIMUM DEFORMATION OF THE SHIELD.

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DESIGN INPUT

1) DROP HEIGHT : ELEVATION 280' (REF. 5)

2) IMPACT HEIGHT: ELEVATION 162' (REF'S. 3 & 4)

3) HYDRAULIC JACK: LENGTH - 5' ; DIAMETER - 26" ; WEIGHT - 7000 lb } PER TELECON WITH FLORIDA POWER

4) MISSLE SHIELD: LENGTH - 29' ; WIDTH - 30" ; DEPTH - 25" ; MATERIAL - A36 STRUCTURAL STEEL } (REF. 3)

REFERENCES

(1) "TOPICAL REPORT - DESIGN OF STRUCTURES FOR MISSLE IMPACT" - SC-TOP-9 REV. 2, BECHTEL POWER CORP., SEPT. 1974.

(2) "THE CONTROL OF HEAVY LOADS AT CRYSTAL RIVER UNIT NO. 3, NUREG-0612 SIX MONTH REPORT," NUS REPORT 3874; AUGUST 24, 1981.

(3) FPC DRAWING NUMBER S-521-116, REV. 1, "AUXILIARY BUILDING SPENT FUEL PIT MISSLE SHIELDING", FLORIDA POWER CORPORATION

(4) FPC DRAWING NUMBER L-002-003, "LAYOUT LONGITUDINAL SECTION THRU REACTOR BLDG. & SPENT FUEL PIT", FLORIDA POWER CORPORATION

(5) VENDOR DRAWING # 3138-1 (FPC REFERENCE # S 0183. SHEET 1, 2, & 3.).

(6) SALMON, C.G., JOHNSON, J.E., "STEEL STRUCTURES - DESIGN AND BEHAVIOR", INTEXT EDUCATIONAL PUBLISHERS, 1971.

(7) EMTR-400A, "MATERIAL PROPERTIES FOR PIPE RUPTURE ANALYSIS", NOV. 13, 1976, STONE & WEBSTER ENGINEERING CORPORATION.

(8) "NONLINEAR DYNAMIC ANALYSIS OF PLANE FRAMES (LIMITA 2): USERS MANUAL", REVISED SEPTEMBER 1980, STONE & WEBSTER ENGINEERING CORPORATION, ST-223.

(9) ROLFE, S.T., BARSOM, J.M., "FRACTURE AND FATIGUE CONTROL IN STRUCTURES: APPLICATIONS OF FRACTURE MECHANICS", PRENTICE-HALL, 1977.

(10) NUREG-0800, STANDARD REVIEW PLAN, SECTION 3.5.3, "BARRIER DESIGN PROCEDURES", U.S. NUCLEAR REGULATORY COMMISION, JULY 1981.

(11) NUREG-0800, STANDARD REVIEW PLAN, SECTION 3.6.2, DETERMINATION OF RUPTURE LOCATIONS AND DYNAMIC EFFECTS ASSOCIATED WITH THE POSTULATED RUPTURE OF PIPING, REV. 1, JULY 1981.

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MATERIALS

1) MISSLE SHIELDS - TYPE: A36 STRUCTURAL STEEL

σ_y : 36 KSI

E : 30×10^3 KSI

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CONCLUSIONS

RESULTS OF THE ANALYSIS SHOWS THAT THE DUCTILITY RATIO IS 13.5, WHICH EXCEEDS THE VALUE OF 10 SPECIFIED IN SRP 3.5.3 (REF 10). HOWEVER, BASED ON THE FACT THAT THE SHIELD HAS NOT FAILED AND ALLOWED THE JACK TO FALL INTO THE POOL, AND BASED ON OTHER ARGUMENTS PRESENTED ON PP. 29-31, IT IS RECOMMENDED THAT THE ABOVE DUCTILITY RATIO OF 13.5 IS ACCEPTABLE AND REDESIGN OF MISSILE SHIELD IS NOT REQUIRED.

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RESULTS SUMMARY

(A) FROM ENERGY BALANCE APPROACH -- A SIMPLIFIED METHOD

1) KINETIC ENERGY OF JACK AT IMPACT = 812 000 FT-lb

2) VELOCITY OF JACK AT IMPACT = 86 FT/SEC

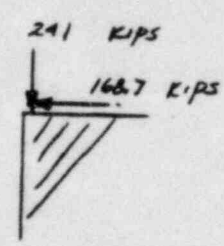
3) DEFLECTION OF SHIELD WHICH WOULD ALLOW THE JACK TO FALL THROUGH = 51 INCHES

(B) FROM A DETAILED NON LINEAR DYNAMIC ANALYSIS

1) MAXIMUM DEFLECTION OF SHIELD:

a.) 16.375" (BASED ON THE USE OF THE LIMITA 2 COMPUTER CODE)

2) DUE TO THE HIGH LOADS DEVELOPED AT THE EMBEDMENTS THEY ARE ASSUMED TO PULL OUT. THE LOADS THEN DEVELOPED ON THE FLOOR ARE:



ANALYSIS

(A) ENERGY BALANCE APPROACH

TWO ENERGY BALANCE APPROACHES WERE USED TO CALCULATE THE MAXMUM DEFLECTION OF THE MISSLE SHIELD. THE FIRST METHOD DOES NOT CONSIDER THE EFFECTIVE MASS OF THE BEAM OR STRAIN RATE EFFECTS. THE SECOND METHOD DOES CONSIDER THESE IN THE ANALYSIS. THIS SECOND METHOD FOLLOWS THE PROCEDURE OUTLINED IN REFERENCE 1.

(B) NONLINEAR DYNAMIC ANALYSIS

THE MISSILE SHIELD IS MODELED AS A LUMPED-MASS BEAM-ELEMENT SYSTEM, AND A BILINEAR STRESS-STRAIN CURVE IS USED TO SIMULATE THE NON-LINEAR BEHAVIOR OF THE MATERIAL. THE VELOCITY OF THE JACK AT IMPACT AND THE GRAVITATIONAL FORCE OF THE JACK IS APPLIED AT THE MID-SPAN OF THE BEAM THROUGH THE USE OF THE LIMITA COMPUTER CODE (REF 8) TO DETERMINE THE STRUCTURAL RESPONSE.

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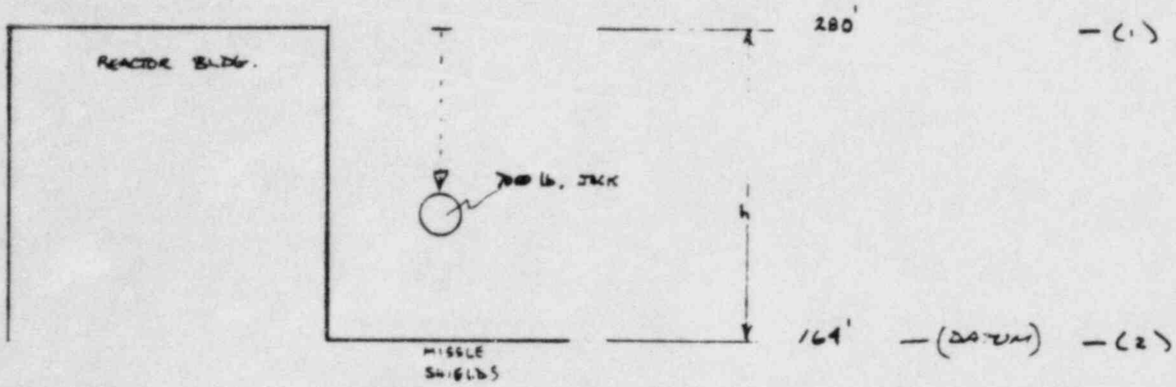
(A) ENERGY-BALANCE APPROACH

POSTULATED DROP SITUATION



280 - 164 = 116' ⟶ HEIGHT THROUGH WHICH THE JACK FALLS BEFORE IMPACTING THE MISSLE SHIELDS

K.E - P.E. = 0

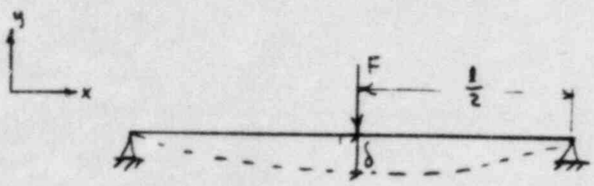
$P.E. = mgh = 7000\,(116) = \boxed{812\,000 \text{ FT-lb}}$

$\frac{1}{2} m v_2^2 - mgh = 0 \longrightarrow v_2 = \sqrt{2gh} = \sqrt{2\,(32.2)\,116}$

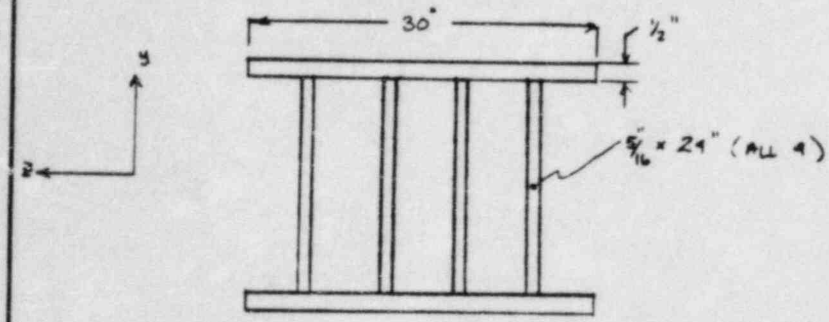
$= \boxed{86 \text{ FT/SEC}}$

$= 1032$ in/SEC

MISSLE SHIELD IMPACT



$l = 27'-8'' = \underline{332''}$



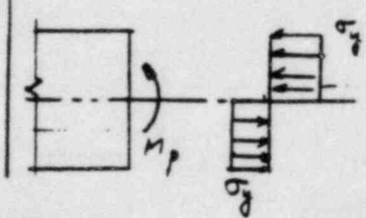
CROSS-SECTION OF MISSLE SHIELD (REF. 3)

$A_x = 60 \text{ in}^2$

MATERIAL: A36, $\sigma_y = 36$ ksi (REF. 3)

$M_p = \sigma_y \int y \, dA$

$= \sigma_y$ (MOMENT OF AREA ABOUT NEUT. AXIS)



MOMENT OF THE AREA

$8\left(12 \times \frac{5}{16}\right)6 + 2\left(\frac{1}{2} \times 30\right)12.25 = \underline{547.5 \text{ in}^3}$

PLASTIC MOMENT & YIELD FORCE

$M_p = \sigma_y (547.5) = 36 \times 10^3 (547.5) = \underline{19.71 \times 10^6 \text{ in-lb.}}$

$F_p = 4 M_p / l = 4(19.71 \times 10^6)/332 = \underline{237\ 470 \text{ lb.}}$

(FOR A SIMPLY-SUPPORTED BEAM LOADED AT MIDSPAN, THE MAX $M = \frac{Fl}{4}$. LET $M_{max} = M_p$, $F \equiv F_p = \frac{4M_p}{l}$)

CALCULATE I

$I = 2\left(\frac{30(0.5)^3}{12} + 12.25^2(15)\right) + 8\left(\frac{5/16(12)^3}{3}\right)$

$= \underline{5942 \text{ in}^4}$

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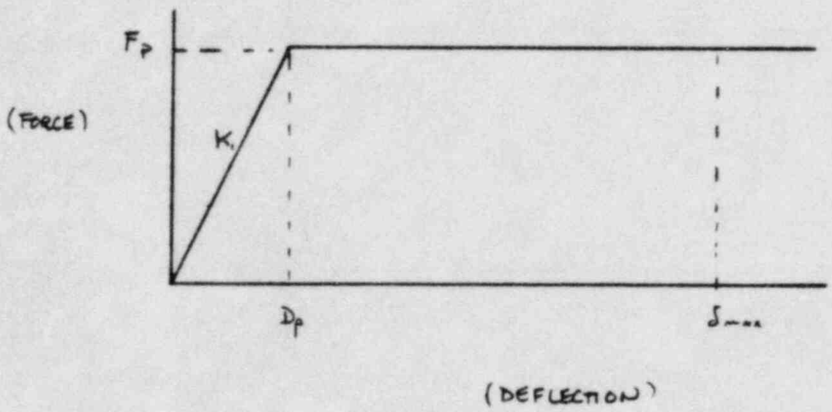
WEIGHT OF JACK = 7000 lb.

DROP HEIGHT = 116'

∴ ENERGY AT IMPACT = 116 (7000) 12 = $\underline{9.74 \times 10^6 \text{ in-lb}}$

{ ASSUME AN ELASTIC - PERFECTLY PLASTIC RESPONSE OF THE BEAM. ASSUME BUCKLING DOES NOT OCCUR.

<u>METHOD I</u>



$$K_1 = F/\delta = 48EI/\ell^3 = 48(30 \times 10^6)\,5942 \,/\, (332)^3$$

$$= \underline{233\,819 \text{ lb/in}}$$

$$D_p = F_p/K_1 = \frac{237\,470}{233\,819} = \underline{1.015} \text{ inches}$$

FOR ENERGY BALANCE:

$$\tfrac{1}{2} D_p F_p + F_p(\delta_{max} - D_p) = 9.74 \times 10^6 \text{ in-lb} + 7000\,\delta_{max}$$

SOLVING FOR δ_{max}:

$$\delta_{max} = \frac{9.74 \times 10^6 + \tfrac{1}{2}(1.015)\,237\,470}{237\,470 - 7000} = 42.8''$$

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CALCULATION SHEET

5010.6

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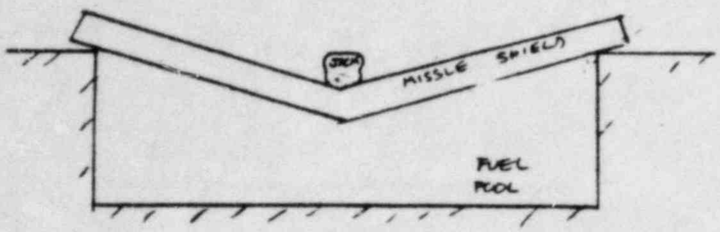
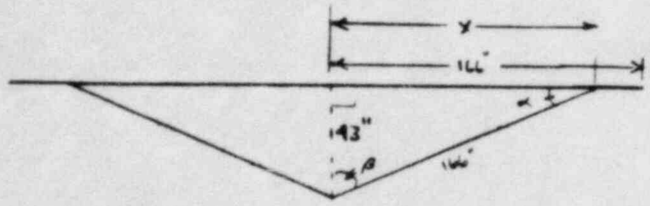


WILL SHIELD FALL INTO POOL?



$$\frac{\sin 90}{166} = \frac{\sin \alpha}{43}$$

$\alpha = 15°$

$\beta = 180 - (90 + 15)$

$= 75°$

THEN: $\sin 90 / 166 = \sin 75 / x$ $\quad$ $\underline{x \approx 160.3"}$

BECAUSE THE X-DIMENSION HAS ONLY SHORTENED BY ~6" AND THE DISTANCE FROM THE END OF THE SHIELD TO THE POOL WALL IS 2'4", IT CAN BE CONCLUDED THAT THE MISSLE SHIELD WILL NOT FALL INTO THE POOL.

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METHOD II:

PROCEDURE AS OUTLINED IN BECHTEL'S REPORT (REFERENCE 1)

EFFECTIVE MASS OF THE STEEL BEAM

$$M_e = (D_x + 2d) M_x$$

WHERE:

D_x = MAX. MISSLE CONTACT DIMENSION IN THE X-DIRECTION (LONGITUDINAL AXIS FOR BEAM)

d = DEPTH OF BEAM

M_x = MASS/UNIT LENGTH OF BEAM

$$M_e = (26 + 2(25))\,0.0438$$

$$= \underline{3.328 \ \frac{lb \cdot s^2}{in}}$$

LET:

$D_x = 26''$

$d = 25''$

$0.283 \times 12 \times (30 + 4 \times 5/16 \times 24)$

$$M_x = \left(\frac{203\ lb}{ft}\right) / 32.2 = 6.32 \ \frac{lb \cdot s^2}{ft^2}$$

$$= 0.0438 \ \frac{lb \cdot s^2}{in^2}$$

TARGET STRAIN ENERGY REQUIRED TO ABSORB THE IMPACT ENERGY

$$E_s = M_m^2 V_s^2 / 2(M_m + M_e)$$

WHERE:

M_m = MASS OF MISSLE

V_s = STRIKING VELOCITY OF MISSLE

$$E_s = (18.083)^2 (1032)^2 / 2(18.083 + 3.328)$$

$$= \underline{8\,132\,674 \ in\text{-}lb}$$

$$(677\,722 \ ft\text{-}lb)$$

LET:

$$M_m = \frac{7000}{32.2} = 217 \ \frac{lb \cdot s^2}{ft}$$

$$= 18.083 \ \frac{lb \cdot s^2}{in}$$

(18.116)

$$V_s = 86 \ ft/s = 1032 \ in/s$$

$$= \sqrt{2gh} = \sqrt{2 \times 32.2 \times 116}$$

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CALCULATE M_p

M_p = DIF (σ_y) (MOMENT OF THE AREA)

WHERE:

DIF = DYNAMIC INCREASE FACTOR (FOR A36 = 1.2) (TABLE 4-1, REF. 1)

σ_y = YIELD STRENGTH (36 KSI)

M_p = 1.2 (36 000) 547.5

= 23.652×10^6 IN-lb

CALCULATE F_p

$$F_p = 4M_p/\ell = 4(23.652 \times 10^6)/332 = 284\ 638 \text{ lb}$$

CALCULATE DISPLACEMENT AT F_p

$$D_p = F_p \ell^3 / 48EI = 284{,}638 (332)^3 / 48 (5942)(30 \times 10^6)$$

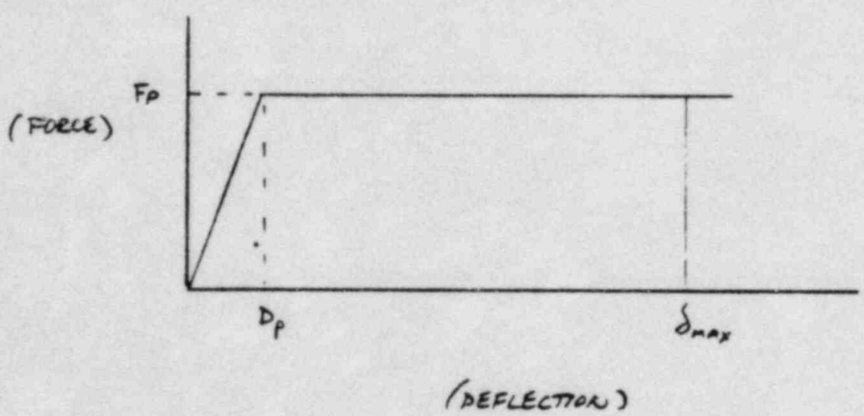
= 1.217 INCHES

MAXIMUM STRAIN ENERGY FOR PURELY ELASTIC RESPONSE:

$$E_e = \tfrac{1}{2} F_p D_p = \tfrac{1}{2} (284\ 638)\ 1.217 = 173\ 202 \text{ in-lb}$$

$E_e < E_s$ ∴ RESPONSE IS ELASTIC-PLASTIC

CALCULATE δ_{max} BASED ON F_p, D_p, AND E_S CALCULATED FOR THIS SECTION (METHOD II)



F_p = 289 638 lb.

D_p = 1.217 INCHES

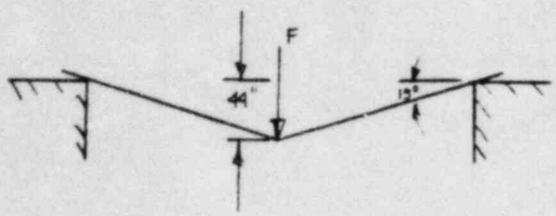
E_s = 8 857 783 in-lb.

$$\tfrac{1}{2} D_p F_p + F_p(\delta_{max} - D_p) = E_S + 7000\,\delta_{max}$$

$$\delta_{max} = \frac{8\,732\,674 + \tfrac{1}{2}(1.218)(284\,638)}{284\,638 - 7000}$$

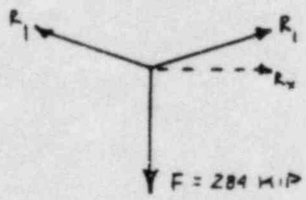
$= 23.9$ INCHES

EMBEDMENT LOADINGS FOR TRANSMITTAL TO STRUCTURAL



F = 284 KIP (LARGEST F)

d = 43 INCHES (LARGEST d)

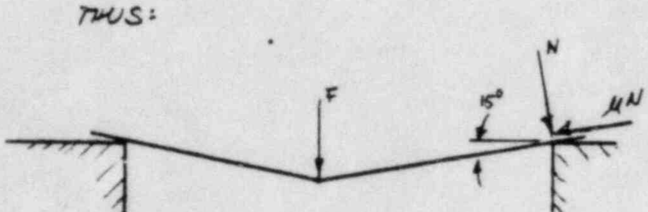


$R_1 = \frac{(284/2)}{\sin 15°} = 548$ KIP

$R_x = 529$ KIP

DUE TO THE LARGE R_x VALUE (529 KIPS), IT IS ASSUMED THAT THE ANCHOR WILL PULL OUT. THE MISSLE SHIELD WILL THEN BE IN CONTACT WITH THE CONCRETE.

THUS:

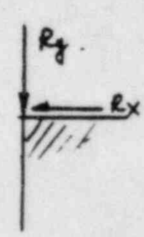


$R_y = (N\cos 15° + \mu N \sin 15°) = \frac{F}{2} = \frac{284}{2} = 142$ kips

$R_x = (\mu N \cos 15° - N \sin 15°)$

$\mu = 0.7$ (STEEL ON CONCRETE)

$N = \frac{142}{\cos 15° + \mu \sin 15°} = 124$ kips



$R_y = 142$ kips

$R_x = (0.7 \times 0.966 - 0.259) \times 124 = 51.8$ kips

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(B) NON-LINEAR DYNAMIC ANALYSIS

THE NRC'S STANDARD REVIEW PLAN 3.5.3, REF. 10, (BARRIER DESIGN PROCEDURES) STIPULATES THAT THE MAXIMUM DUCTILITY RATIO SHOULD NOT BE GREATER THAN 10. BASED ON THE DEFLECTIONS AT YIELD OF THE PREVIOUS TWO ENERGY BALANCE APPROACHES, THIS WOULD ALLOW A MAXIMUM DEFLECTION OF THE SHIELD OF 10.15 TO 12.18 INCHES. AS THE ACTUAL CALCULATED DEFLECTIONS, BASED ON THESE TWO APPROACHES, RANGE FROM 30" TO 43", A MORE ACCURATE APPROACH WILL BE UTILIZED TO REDUCE THG DEFLECTIONS.

A NON-LINEAR DYNAMIC ANALYSIS COMPUTER CODE (LIMITA 2) WILL BE UTILIZED FOR THE ANALYSIS. STRAIN-RATE EFFECTS, WORK HARDENING, AND THE ADJUSTED VELOCITY OF THE MISSLE/BARRIER EFFECTIVE MASS WILL BE ACCOUNTED FOR IN THE ANALYSIS.

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LIMITA ANALYSIS OF MISSLE SHIELD IMPACT

CALCULATE M_p

M_p IS CALCULATED AS BEFORE. HOWEVER, THIS TIME M_p IS MULTIPLIED BY A FACTOR OF 1.2 (REF 1. pg. 4-4) TO ACCOUNT FOR STRAIN RATE EFFECTS. THUS:

$$M_p = 1.2\ \sigma_y \text{ (MOMENT OF AREA)}$$

$$= 1.2\ (36 \times 10^3)\ 547.5$$

$$= \underline{23.652 \times 10^6 \text{ IN-lb}}$$

CALCULATE F_p

FOR THE LIMITA CODE, F_p IS THE AXIAL YIELD FORCE AND IS INCREASED BY THE FACTOR OF 1.2 AS WELL. THUS,

$$F_p = 1.2\ \sigma_y\ A_x \qquad A_x = 60 \text{ in}^2 \text{ (SEE pg. 16)}$$

$$= 1.2\ (36 \times 10^3)\ 60$$

$$= \underline{2.592 \times 10^6 \text{ lb.}}$$

EQUIVALENT VELOCITY OF MISSILE/SHIELD EFFECTIVE MASS JUST AFTER IMPACT

(FROM REF 1, pg. 3-3) FOR A PLASTIC COLLISION, THE VELOCITY

OF THE JACK / SHIELD COMBINATION IS GIVEN BY:

$$v_{EQUIVALENT} = \left(\frac{M_M}{M_M + M_E}\right) v_I$$

WHERE:

M_M = MASS OF MISSLE (JACK)

M_E = EFFECTIVE MASS OF SHIELD (SEE PG. 19)

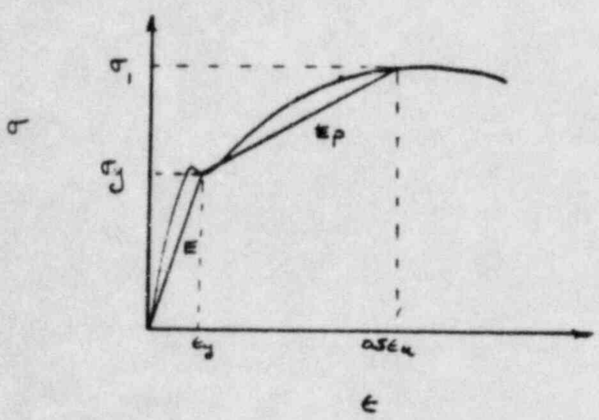
V_I = IMPACT VELOCITY BEFORE CONTACT. (P. 15)

THEN

$$v_{EQUIVALENT} = \left(\frac{18.083}{18.083 + 3.328}\right) 1032$$

$$= \underline{871.6 \ \tfrac{in}{sec}}$$

CALCULATE E_P



FOR A36 STEEL @ 70°F (REF. 6, pg. 32)

$E = 29 \times 10^6$ PSI

$\sigma_y = 36 \times 10^3$ PSI

$\sigma_u = 58 \times 10^3$ PSI

$\epsilon_y = 0.002 + \sigma_y/E = 0.003$

$\epsilon_u = 0.19$

FOLLOWING THE PROCEDURE OF EMTR - 400A (REF. 7):

$E_P = (\sigma_1 - \sigma_y) / (0.5\,\epsilon_u - \epsilon_y)$

$\sigma_1 = \sigma_0 \, (0.5\,\epsilon_u)^n$

$n = \ln(\sigma_u/\sigma_y) / \ln(\epsilon_u/\epsilon_y)$

$= \underline{0.115}$

$\sigma_0 = \sigma_u/\epsilon_u^n = \underline{70.2 \times 10^3 \text{ PSI}}$

$$\sigma_1 = 70.2 \times 10^3 \left[0.5(0.19)\right]^{0.15}$$

$$= \underline{53\,551 \text{ psi}}$$

THEN:

$$E_p = (53\,551 - 36\,000)/(0.5(0.19) - 0.003)$$

$$= \underline{190.8 \times 10^3 \text{ psi}}$$

ROTATIONAL STRAIN HARDENING PARAMETER (HD)

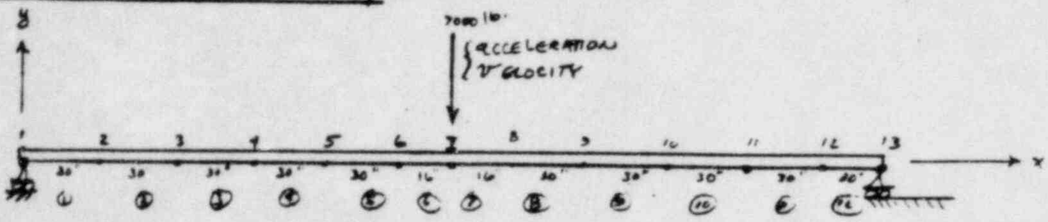
FROM (REF. 8, PG. 19) WE GET:

$$HD = \left[4 I_z / A_x\right]^{1/2}$$

$$= \left[4(5492)/60\right]^{1/2}$$

$$= \underline{19.134 \text{ in}}$$

MEMBER AND JOINT WEIGHTS



MEMBER	WEIGHT (lb)	ROT. WEIGHT (lb)
①	509.4	88652.6
②	509.4	88652.6
③	509.4	88652.6
④	509.4	88652.6
⑤	509.4	88652.6
⑥	271.7	31252.6
⑦	271.7	31252.6
⑧	509.4	88652.6
⑨	509.4	88652.6
⑩	509.4	88652.6
⑪	509.4	88652.6
⑫	509.4	88652.6

NODE	WEIGHT (lb)	ROT. WEIGHT (lb)	NODE COORDINATES (INCHES) (x, y)
1	254.7	44326.3	(0,0)
2	509.4	88652.6	(30,0)
3	509.4	88652.6	(60,0)
4	509.4	88652.6	(90,0)
5	509.4	88652.6	(120,0)
6	390.6	59952.6	(150,0)
7	7271.7	31252.6	(166,0)
8	390.6	59952.6	(182,0)
9	509.4	88652.6	(212,0)
10	509.4	88652.6	(242,0)
11	509.4	88652.6	(272,0)
12	509.4	88652.6	(302,0)
13	254.7	88652.6	(332,0)

$A_x = 60\ in^2$

$I_z = 5432\ in^4$

} SEE PG. (16)

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RESULTS OF COMPUTER ANALYSIS

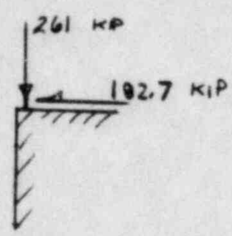
RESULTS SHOW THAT A MAXIMUM DEFLECTION OF 16.920" OCCURS AT NODE 7 (THE CENTER OF THE MISSLE SHIELD)

REACTION FORCES FOR THE BEAM MODELED ARE:

F_y = 261 KIPS

$F_x = \mu F_y$ = 0.7 (261) μ = 0.7 (SEE PG. 22)

= 182.7 KIPS



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DISCUSSION OF RESULTS

AS THE DEFLECTION AT YIELD IS 1.218" AND THE MAXIMUM DEFLECTION IS 16.375", THIS GIVES US A DUCTILITY RATIO OF 16.420/1.218 = 13.5. THIS VALUE IS GREATER THAN 10, THE ALLOWABLE SPECIFIED IN SRP 3.5.3. HOWEVER, THE FOLLOWING ARGUMENTS ARE PRESENTED TO JUSTIFY THE ACCEPTABILITY OF THE 16.375" DEFORMATION.

A) THE GUIDELINES OF THE LATEST SRP ARE BEING USED TO QUALIFY THE SHIELDS(DUCTILITY RATIO = 10). (REF. 10) HOWEVER, AT THE TIME OF THIS PLANTS CONSTRUCTION, THERE WERE NO GUIDELINES ON ALLOWABLE DUCTILITY FOR SUCH DEVICES.

ALSO, IT SHOULD BE EMPHASIZED THAT WE ARE MERELY TRYING TO SHOW THAT THE SHIELDS WILL PREVENT THE JACK FROM FALLING INTO THE FUEL POOL; WE ARE NOT TRYING TO DESIGN A NEW SHIELD.

B) REF (1), PG. 4-7, STIPULATES A MAXIMUM DUCTILITY RATIO, FOR STEEL MEMBERS SUBJECT

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TO FLEXURE, COMPRESSION, AND SHEAR, OF 20. THIS VALUE IS TWICE THAT PRESENTED IN SRP 3.5.3.

C) CONSERVATISM STILL EXISTS, IN THAT THE ENERGY DISSIPATION DUE TO THE JACK GOING THROUGH THE ROOF OF THE AUXILLARY BUILDING IS NOT CONSIDERED. ALSO, ANY CONSTRAINT DUE TO THE EMBEDMENTS HAS NOT BEEN CONSIDERED.

D) LATERAL BUCKLING OF THE SHIELD IS PREVENTED DUE TO THE CONSTRAINT IMPOSED BY THE REMAINING SHIELDS. THIS WOULD PRECLUDE PREMATURE FAILURE OF THE SHIELD AND ALLOW IT TO FAIL BY A HIGHER STRAIN.

E) A MULTIPLIER OF 1.2 WAS USED TO INCREASE σ_y DUE TO THE STRAIN RATE EFFECT. THIS AMOUNTS TO AN INCREASE IN σ_y BY 13.2 KSI. HOWEVER, IT HAS BEEN CITED IN THE LITERATURE (REF. 3, PG. 103) THAT A GOOD APPROXIMATION FOR DYNAMIC TENSILE YIELD STRENGTH IS GIVEN BY $\sigma_{yD} = \sigma_y + (20 \text{ TO } 30 \text{ KSI})$ THE USE OF THIS HIGHER DYANAMIC YIELD STRENGTH WOULD FURTHER REDUCE THE MAXIMUM DEFORMATION.

F) THE PROBABILITY OF THE EVENT OCCURING IS VERY SMALL AS THE SCAFFHOLDING, WHICH CONTAINS THE JACK, IS ONLY IN A POSITION OVER THE POOL ONCE EVERY FIVE YEARS.

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G. MISSILE SHIELD IS DESIGNED FOR THE PROTECTION OF OTHER SAFETY-RELATED SYSTEMS FOR A POSTULATED ACCIDENT, IN THIS CASE FOR A POSTULATED HEAVY WEIGHT DROP ACCIDENT. THE FUNCTIONAL REQUIREMENTS OF A MISSILE SHIELD ARE THEREFORE COMPARABLE TO THAT OF A PIPE RUPTURE RESTRAINT OR A JET IMPINGEMENT SHIELD, WHICH ARE STRUCTURES DESIGNED FOR THE PROTECTION OF OTHER SAFETY-RELATED SYSTEMS IN A POSTULATED PIPE FAILURE.

A 50% OF THE ULTIMATE UNIFORM STRAIN IS ALLOWED FOR THE DESIGN OF THESE PIPE FAILURE PROTECTIVE STRUCTURES ACCORDING TO SRP 3.6.2 (REF. 11). IF THIS SAME CRITERION IS APPLIED TO THE MISSILE SHIELD, THE ALLOWABLE DUCTILITY RATIO WOULD BE AROUND 30 ~ 50.

H. ALTHOUGH THE DEFORMATION IS IN EXCESS OF THAT SPECIFIES IN SRP 3.5.3, THE BOTTOM LINE IS THAT THE SHIELD HAS NOT FAILED AND ALLOWED THE JACK TO FALL INTO THE POOL.

THEREFORE, BASED ON THE ABOVE ARGUMENTS IT IS RECOMMENDED THAT EVEN THOUGH THE DUCTILITY RATIO REQUIREMENTS OF SRP 3.5.3 ARE EXCEEDED THE MISSILE SHIELDS ARE ACCEPTABLE AND NO REDESIGN OF SHIELDS IS REQUIRED.

COMPUTER LOG

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S & W AUTH. NO. 7189
CDC CHARGE NO. ____
CALCULATION NO. 14235.17 - M.07
JOB ORDER NO. 14235.17

RUN NO.	JOB NO. *	FICHE LOC.		PREPARED BY		REVIEWED BY		COMP. **	COMMENTS
		PAGE	SECT	SIGNATURE	DATE	SIGNATURE	DATE		
C52	1235		A	M. Durkee	7/13/82	H H Kuo	7/15/83	S&W	LIMITA 2A - ROLLER/ROLLER SUPPORT

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