SMUD CALCULATION COVER SHEET

SACRAMENTO MUNICIPAL UTILITY DISTRICT ☐ 6201 S Street, P.O. Box 15830, Sacramento CA 95852-1830 (916) 452-3211

ISFSI CONCRETE SLAB DESIGN

(M41.02-12)

CALC. NO. Z-DRY -C1024

FILE NO.

QUALITY CLASS 33

SEISMIC CATEGORY —

SHEET 1 OF _____

RECORD OF ORIGINAL ISSUE AND REVISIONS			SMUD ORIGINATED CALCS			NUCLEAR ENGRG ACCEPTANCE	
REV. NO.	REVISION DESCRIPTION	DATE	ORIG.	CKR.	SUPVY. REVIEW	DE	DGS
0	ISSUE FOR CONSTRUCTION	9-28-93				WHH 9/20/93	DGS

RESULTS OF CHECKER REVIEW

	ITEM DESCRIPTION		ORIG. ISSUE	REVISION NO.					
MUST INITIAL ONE	FINAL RESULT NUMERICAL DIFFERENCES ARE NOT SIGNIFICANT; NO CORRECTIONS NECESSARY.	INITIAL	WHH 9/20/93						
	FINAL RESULT NUMERICAL DIFFERENCES ARE SIGNIFICANT, NECESSARY CORRECTIONS ARE MADE.	INITIAL	WHH 9/20/93						
	CHECK MADE BY ATTACHED ALTERNATE CALCULATIONS. _____ SHEETS	INITIAL	N/A						
		DATE	N/A						
	REVIEWED FOR IMPACT	OPS. PROC.	N/A						
		MAINT. PROC.	N/A						

	PRINTED NAME	SIGNATURE	INITIAL	DISCIPLINE	DATE
ORIGINATOR					
CHECKER					
SUPVY. REVIEW					
DESIGN ENG.	W.H. HAWLEY	W.H. Hawley	WHH	TS/C	9/20/93
DISC. GRP. SUP.	J J FIELD	J J Field	JJF	TS/M	9/28/93

ENG-053 REV. 2

DOCUMENT APPROVAL COVER SHEET

ABB
ASEA BROWN BOVERI
ABB IMPELL CORPORATION

DOCUMENT NUMBER: 0079-00541-2

DOCUMENT TITLE: ISFSI Concrete Slab Design

DOCUMENT TYPE:
- [X] CALCULATION
- [ ] SPECIFICATION
- [ ] REPORT
- [ ] DOCUMENT PER QP-3.11

CLIENT: SMUD

PROJECT: Independent Spent Fuel Storage Installation

PROJECT NUMBER: 0079-00541

SUMMARY DESCRIPTION:

Design of ISFSI Reinforced Concrete Slab.

REVISION	DESCRIPTION:	
A	Issued for Comment	TOTAL NO. OF PAGES 182
ORIGINATOR: Mike David MD		DATE: 4-30-93
VERIFIER: BRAD SENN BS / TOS Terry Stelter		DATE: 4/30/93
APPROVER: N/A		DATE:

REVISION	DESCRIPTION:	
B	Re-Issued for Comment	TOTAL NO. OF PAGES 671
ORIGINATOR: Mike David MD		DATE: 5-18-93
VERIFIER: BRAD SENN BS		DATE: 5-27-93
APPROVER: BRAD SENN BS		DATE: 5-27-93

REVISION	DESCRIPTION: REVISED ~~SHTS~~ 1, 2, 3, 4,	
0	Issued for USE	TOTAL NO. OF PAGES 688
ORIGINATOR: Mike David MD		DATE: 6-8-93
VERIFIER: B. SENN BS		DATE: 6-10-93
APPROVER: B. SENN BS		DATE: 6-10-93

M41.02-12 sub 2

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ABB Impell Corporation

JOB NO 0079-00541

CALC NO 0079-00541-2

Purpose =

The purpose of this calculation is to design the Independent Spent Fuel Storage Installation @ Rancho Seco Nuclear Generating Station.

Methodology =

The reinforced concrete design of the ISFSI slab on grade will be based on a finite element analysis performed by SAP90 version PS.40. The slab(s) will be modeled with quadrilateral plate elements with a thickness to length/width ratio of approximatelly 1:5. The supporting soil will be modeled by springs at each joint with a stiffness based on a modulas of subgrade reaction K = 300 Ksf/ft per reference 11. Various loading conditions will consider dead load, live load, and seismic. The following load combinations and associated load factors will be considered in accordance with reference 3 §2609(c):

(9-1) $U = 1.4D + 1.7L \longrightarrow$ Envelope & use $U = 1.7(D+L)$

(9-2) $U = 0.75[1.4D + 1.7(1.1E)] = 1.05D + 1.40E \rightarrow$ Envelope & Use $U = 1.4(D+E)$

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References

1. Rancho Seco ISFSI Design Criteria
   Rev A, March 10, 1993
2. ABB Impell Letter # 79-541-L1, April 12, 1993
3. Uniform Building Code 1991
4. FAX Transmissions from Dave Middleton & Keith Trommler / Bragg Crane & Rigging Dated 4/12/93, 4/14/93, 5/7/93, & 5/11/93, regarding Link-Belt 200ton wire rope truck crane HC-258. General Specifications and rigging study for 120ton lift.
5. ACI 360 R-91 (Draft)
6. ABB Impell Letter No. 79-541-L3, April 23, 1993
7. Geotechnical Engineering Investigation by GEOCON Inc. dated 8/16/91, File No. 30004-05-01
8. Handbook of Concrete Engineering, 2nd Edition, Mark Fintel
9. SAP90, Version P5.40
10. Manual of Steel Construction, ASD 9th Edition
11. Geotechnical Study for Proposed ISFSI, Rancho Seco Nuclear Generating Station, Job No. ES-519/E306-01, 6-1-93

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0	MD	6-8-93	[illegible]	6/10/93
B	MD	5-18-93	[illegible]	5/27/93
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ABB ASEA BROWN BOVERI
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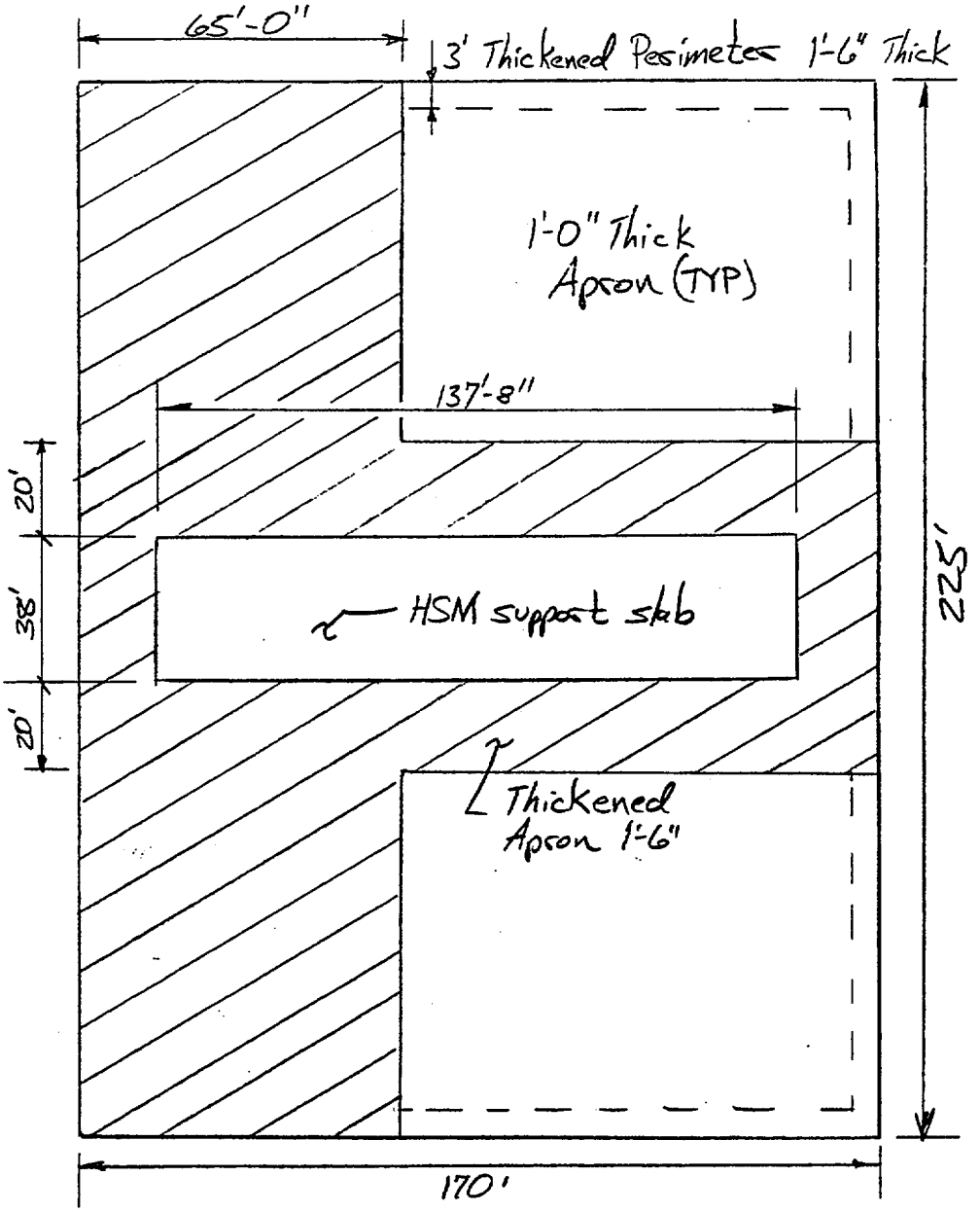
JOB NO 0079-0054I
CALC NO 0079-0054I-2

ISFSI Layout =



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Design Loads: [per Ref 1 & 2]

Transporter:

Max loaded wt = 145 ton = 290K / Unloaded wt = 25t = 50K
Ave Jack load = 74K
Rear Jack max load = 142K Front Jack min Load = 22K
Max wheel load = 20K
Max tire press = 150psi
Four axles w/ 32 tires total
20' long, 12' wide

Horizontal Storage Module (HSM):

Max loaded wt = 250 ton = 500K cg ht = 93" from bottom
Min wt = 130 ton = 260K cg ht = 120" from bottom
Ht = 15'-0" Width = 9'-8" Length = 19'-0"

Dual Purpose Cask:

Max loaded wt = 120 ton = 240K
ht = 17' dia = 8'

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Determine min slab thickness based on punching shear requirements

Transporter

Rear jacks down on apron near HSM's :

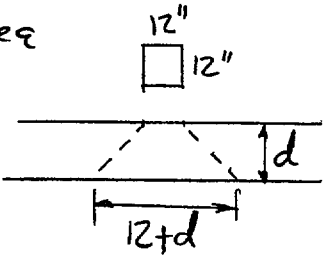
loaded area = 12" square [Ref 2]

max load = 142^k

[Ref 3] § 2611(m) $V_c = \left(2+\frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d \le 4\sqrt{f'_c}\, b_o d$ (11-36)

where $\beta_c = 1$ for square loaded area

critical section $b_o = 4(12+d)$



$\phi V_n = \phi V_c = 0.85(4)\sqrt{4000}(4)(12+d)d$

$\phi V_n = 860d^2 + 10{,}322d$

want $\phi V_n \ge V_u = 1.7(142{,}000)$ ← live load factor

$\therefore \quad 860d^2 + 10{,}322d \ge 1.7(142{,}000)$

$d^2 + 12d - 280.65 = 0$

$d = 11.8$ in

Slab thickness = $11.8 + 3$ (cover) = 14.8" try 18" in this area

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Punching Shear Cont'd:

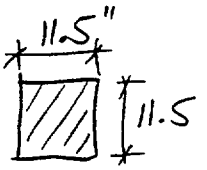
Transporter tires:

Find loaded area → [Per Ref 5] $\frac{\text{Tire load}}{\text{tire press}} = \frac{20{,}000\ lb}{150\ psi} = 133.3\ in^2$

single tire:

approximate as square

side = $\sqrt{133} = 11.5"$



as before $b_o = 4(11.5+d)$

$$\phi V_n \geq V_u$$

$$0.85(4)\sqrt{4000}(4)(11.5+d)\,d \geq 1.7(20{,}000)$$

$$860d^2 + 9891.6d \geq 1.7(20{,}000)$$

$$d^2 + 11.5d - 39.5 = 0 \Rightarrow d = 2.8"$$

look at possibly 2 tires side by side:

equivalent square side = $[2(133.3)]^{1/2} = 16.3"$ on a side

$\therefore\ b_o = 4(16.3+d)$

$$\phi V_n \geq V_u$$

$$0.85(4)\sqrt{4000}(4)(16.3+d)\,d \geq 1.7(2)(20{,}000)$$

$$860d^2 + 14020d - 1.7(2)(40{,}000) = 0$$

$$d^2 + 16.3d - 79.06 = 0$$

$$d = 3.9\ in$$

$\therefore$ min slab thickness = $3.9 + 3_{\text{covers}} = 7"$

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Punching Shear (Cont'd)

Dual Purpose Cask: Max wt = 240^k

find equivalent square bearing area for 8' dia circular area

Area = $\pi(4)^2 = 50.26\ ft^2$ equiv. sq. side = $\sqrt{50}$ = 7' approx

as before $b_o = 4(7\times12+d) = 4(84+d)$

Use 25% impact to account for setting of cask on slap ⇒ "wt" = 1.25(240) = 300^k

$$\phi V_n \geq V_u$$

$$0.85(4)\sqrt{4000}(4)(84+d)d \geq 1.7(300{,}000)$$

$$860d^2 + 72{,}252d \geq 1.7(300{,}000)$$

$$d^2 + 84d - 593.0 = 0$$

$$d = 6.6"$$

∴ min slab thickness = $6.6 + \underset{\text{cover}}{3} =$ <u>9.6" min</u>

Summary:

Min slab thickness required for the thinner part of the apron is 9.6". Try <u>12" slab</u>

Min slab thickness required for thicker part of the apron is 14.8", Try <u>18" slab</u>

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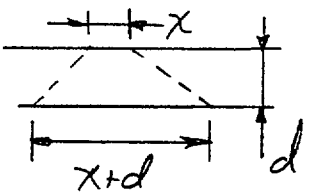
Punching Shear (cont'd)

Check crane outrigger crib pad min dimension reqmts

Crane outrigger max load = 292,310 lb Ref [4]

Determine required crib pad dimensions for 12" & 24" slab:

as before, loaded area is square $\Rightarrow \beta_c = 1$



12" slab: $d = 12 - 3 - 1.5(.5) = 8.25"$ (cover, Bar ø)

18" slab: $d = 18 - 3 - 1.5(.75) = 13.8"$

$\phi V_n = .85(4)\sqrt{f'_c}\, b_o d$

$\phi V_n = .85(4)\sqrt{4000}(4)(x+8.25)(8.25)$ 12" slab

$\phi V_n = 7096.2(x+8.25)$

want $\phi V_n \geq V_u$

$7096.2(x+8.25) \geq 1.7(292,210)$

$x \geq 61.8$ in = 5'-2" square 12" slab

$\phi V_n = 0.85(4)\sqrt{4000}(4)(x+13.8)(13.8)$ 18" slab

$\phi V_n = 11,870(x+13.8)$

$\phi V_n \geq V_u$

$11,870(x+13.8) \geq 1.7(292,210)$

$X \geq 28.1 \simeq 2'-4"$ sq

Use 4'x4' cribbing per Ref 4 on 18" slab OK ✓

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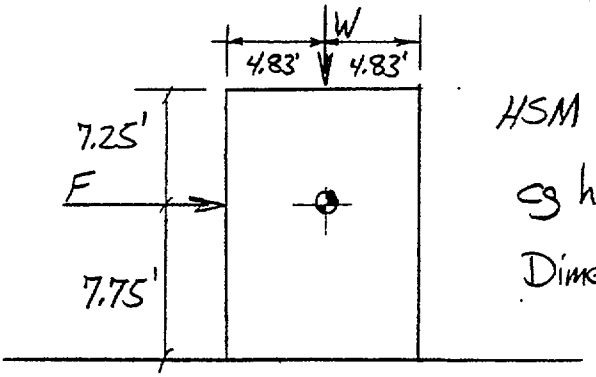
Develop application of loads onto slab:

Fully Loaded HSM:



HSM max wt $W = 500^k$

cg ht = 93" from bottom

Dimensions: 15' x 9.66' x 19'
ht width length

[per Ref 1]

Seismic Base Shear $F = 1.5(ZPA)W$ where ZPA = 0.25g [Per Ref 1]

$= 1.5(0.25)W = 0.375W$ [Per Ref 6]

$= 0.375(500) = 187.5^k$

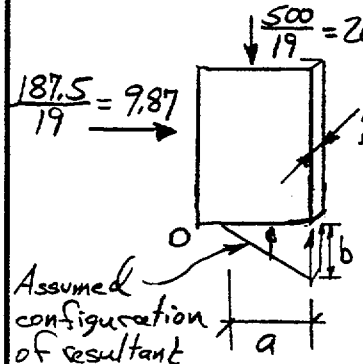
Check OT:

OT moment $= 7.75F = 7.75(187.5) = 1453.1$ k-ft

Rtg moment $= 4.83W = 4.83(500) = 2415$ k-ft

factor of safety $= \frac{2415}{1453} = 1.66$ Good

find distribution of reaction along length of HSM (19' long)



$\Sigma F_{vert} = 0$

$26.3 - \frac{1}{2}ab = 0 \Rightarrow ab = 52.6$

$\Sigma M_O = 0$

$9.87(7.75) + 26.3(4.83) = \frac{1}{2}ab(9.67 - \frac{1}{3}a)$

$203.52 = \frac{1}{2}(52.6)(9.67 - \frac{1}{3}a)$

$a = 5.8$ ft $\Rightarrow b = \frac{52.6}{5.8} = 9.08$ K/ft/ft

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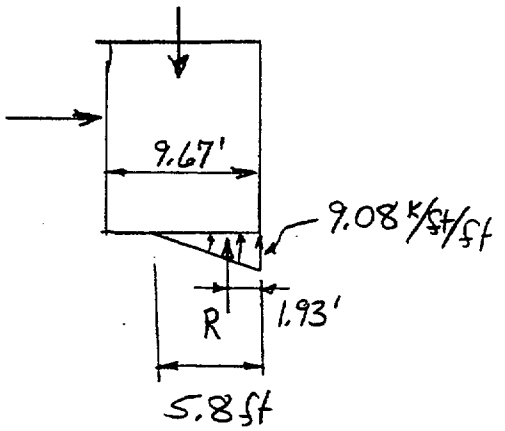
HSM reaction (cont'd)



$R = \frac{1}{2}(5.8)(9.08)$

$R = 26.3$ K/ft along length of HSM

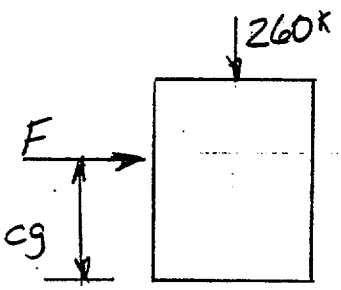
HSM @ max wt.

Empty HSM: wt = 260K cg ht = 120" from bottom

per [ref 2] cg ht is approx.

∴ Look @ 2 cases — if cg is taken @ ht of that for a fully loaded HSM, the OT moment is reduced and therefore the differential between loaded & empty HSM's is greater. This will be compared to the OT affects due to an HSM with a cg @ 120" from the bott.

Case ①: cg = 120" Case ② cg = 93"



$F = 0.375(260) = 97.5^K$

OT moment ① $= 97.5\left(\frac{120}{12}\right) = 975$ k·ft

OT moment ② $= 97.5\left(\frac{93}{12}\right) = 755.6$ k·ft

Rtg moment $= 260(4.83) = 1255.8$ k·ft

$F.S._{①} = \frac{1255.8}{975} = 1.3$ ok $\quad F.S._{②} = \frac{1255.8}{755.6} = 1.66$ ok

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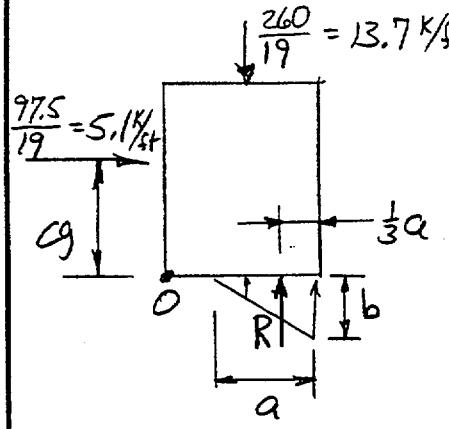
HSM reaction (cont'd)

Distribution of reaction along HSM 19' length:



cg ① = 10' cg ② = 7'-9"

Case ①: $\Sigma F_{vert} = 0$

$13.7 - \frac{1}{2}ab = 0 \Rightarrow ab = 27.4$

$\Sigma M_o = 0$

$5.1(10) + 13.7(4.83) = \frac{1}{2}ab(9.67 - 1/3\,a)$

$117.2 = \frac{1}{2}(27.4)(9.67 - 1/3\,a)$

$a = 3.4$ ft

$b = \frac{27.4}{3.4} = 8.17$ K/ft/ft

$R_① = \frac{1}{2}ab = \frac{1}{2}(3.4)(8.17) = 13.9$ K/ft

Case ②: $\Sigma F_{vert} = 0$

$ab = 27.4$

$\Sigma M_o = 0$

$5.1(7.75) + 13.7(4.83) = \frac{1}{2}(27.4)(9.67 - 1/3\,a)$

$a = 5.86$ ft

$b = \frac{27.4}{5.86} = 4.67$ K/ft/ft

$R_② = \frac{1}{2}(5.86)(4.67) = 13.7$ K/ft

Two cases very close → use ave value of 13.8 K/ft for analysis

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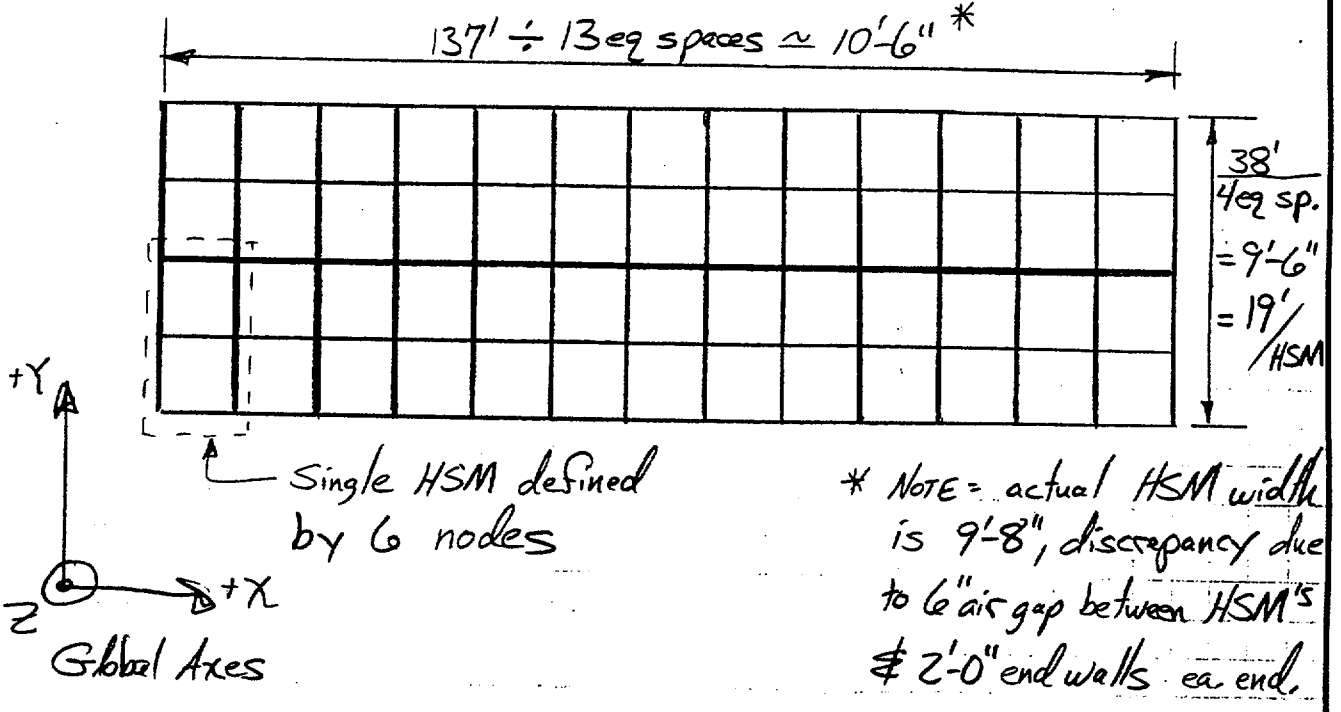
HSM reaction (cont'd)

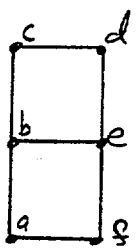
FEM node location = 26 HSM's Total



Single HSM defined by 6 nodes

* NOTE = actual HSM width is 9'-8", discrepancy due to 6" air gap between HSM's & 2'-0" end walls ea end.

Find load per node =



DL only : fully loaded = 500K

empty = 260K

HSM	Loaded	Empty
a, f c, d	62.5K	32.5K
b, e	125K	65K

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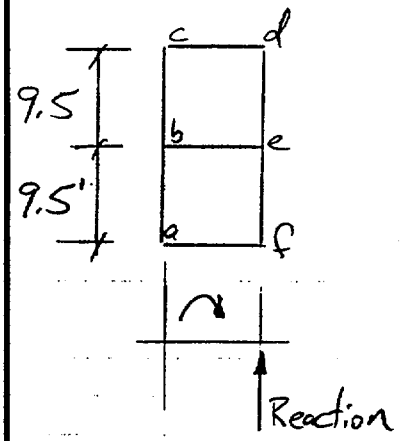
HSM reaction (cont'd)

DL + Seismic Loads: Conservatively apply reaction per tributary length of node to one side only of HSM:

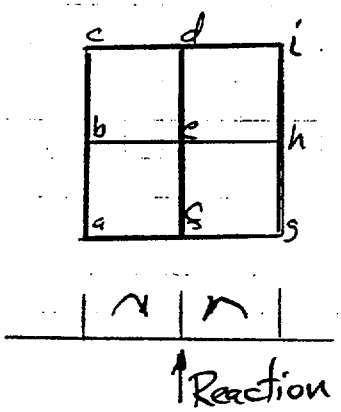
Fully loaded = 26.3 K/f

empty = 13.8 K/ft



HSM	Loaded	Empty
d, f	125 K	65.6 K
e	250 K	131.1 K

Two HSM's rocking toward one-another:



HSM	Both Loaded	Both Empty	One Loaded One Empty
d, f	250 K	131.1 K	190.6 K
e	500 K	262.2 K	381.1 K

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ABB Impell Corporation

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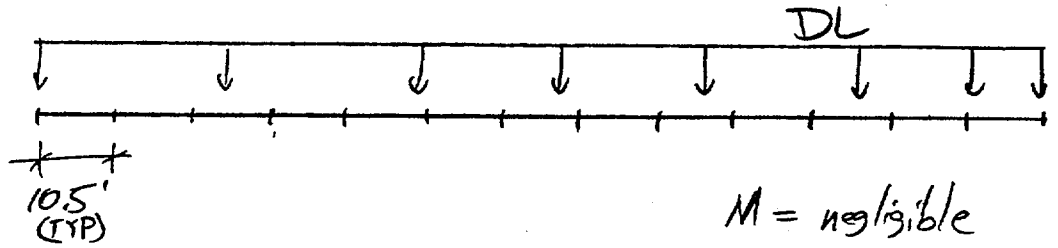
Load Cases – HSM Longitudinal Direction

Values per .FEF files in Appendices

Moments indicated are absolute maximums enveloping pos & neg. Node locations may be representative of several nodes with identical stresses.

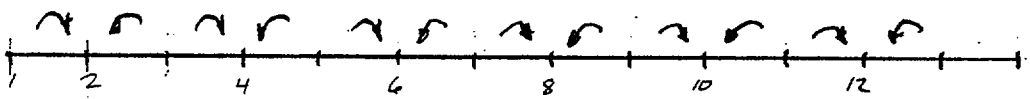
Load Case 1



M = negligible

Load Case 2

HSM's Fully loaded & rocking as shown

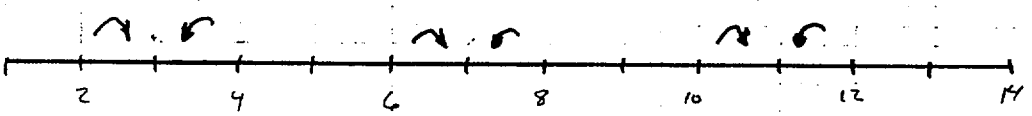


Node 68 $M_y = \frac{321.2 \text{ K-ft}}{4.75'}$

$= 67.6 \text{ K-ft/ft}$

Load Case 3

HSM's Loaded & rocking as shown



Node 46 $M_y = \frac{380.8 \text{ K-ft}}{4.75}$

Some Springs in tension $\Rightarrow M_y = 80.2 \text{ K-ft/ft}$

Tension Springs Cut $\Rightarrow M_y = \frac{381.0}{4.75} = 80.2 \text{ K-ft/ft}$ (approx. no change)

Per Appendix D node #3

REV	BY	DATE	CHECKED	DATE
0	MD	6-8-93	[illegible]	6/10/93
B	MD	5-18-93	[illegible]	5/20/93
A	MD	4-30-93	[illegible]	4/30/93

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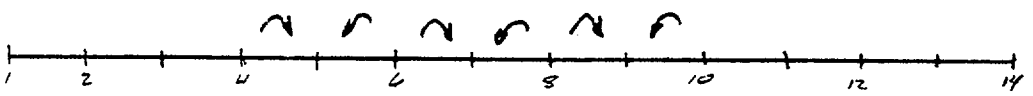
Load Cases - HSM

Load Case 4

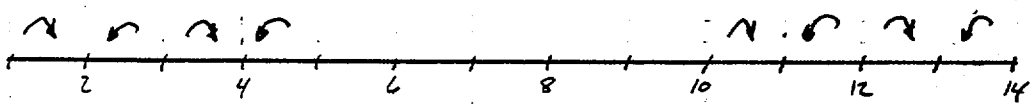
HSM's Fully Loaded & Rocking as Shown



Node 65 $M_y = \frac{322.7}{4.75} = 67.9$ k·ft/ft

Load Case 5

HSM's Fully Loaded & Rocking as Shown



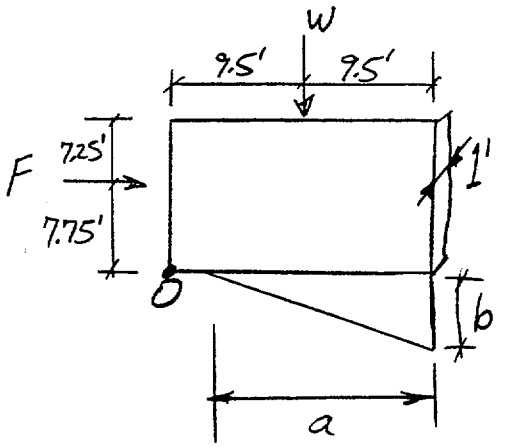
Node 67 $M_y = \frac{316.2}{4.75} = 66.6$ k·ft/ft

Load Case 6

Node 60 $M_y = \frac{384.5}{4.75} = 80.9$ k·ft/ft

REV	BY	DATE	CHECKED	DATE			
B	MG	5-18-93	BSB	5/27/93	ABB ASEA BROWN BOVERI ABB Impell Corporation	JOB NO 0079-0054I	PAGE 17
A	MG	4-30-93	BSB	5/27/93		CALC NO 0079-0054I-Z	CONT ON 18

HSM load transverse direction:



$W = 500^k$ $F = 187.5^k$

Find reaction distribution along HSM width of 9'-8"

For 1' width: $W = \frac{500}{9.67} = 51.7^{k/ft}$

$F = \frac{187.5}{9.67} = 19.4^{k/ft}$

$\Sigma F_{vert} = 0$

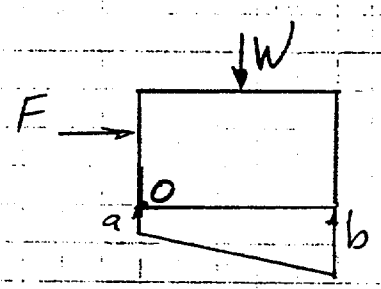
$51.7 - \frac{1}{2}ab = 0 \Rightarrow ab = 103.4^{k/ft}$

$\Sigma M_o = 0$

$51.7(9.5) + 19.4(7.75) = \frac{1}{2}ab\left(19 - \frac{1}{3}a\right)$

$641.5 = \frac{1}{2}(103.4)\left(19 - \frac{1}{3}a\right)$

$a = 19.77\ ft$ ∴ reaction along entire bottom



$\Sigma M_o = 0$

$641.5 = \frac{1}{2}a(19)^2 + \frac{1}{2}(b-a)\frac{2}{3}19^2$

$3.554 = a + \frac{2}{3}(b-a) = \frac{2}{3}b + \frac{1}{3}a$

$10.66 = 2b + a$

$\Sigma F_{vert} = 0$

$51.7 - \frac{1}{2}(a+b)19 = 0$

$a = 5.442 - b$

$10.66 = 2b + 5.442 - b$

$b = 5.22^{k/ft^2}$

$a = 5.442 - 5.22 = 0.22^{k/ft^2}$

REV	BY	DATE	CHECKED	DATE
B	MG	5-18-93	[illegible]	5/27/93
A	MG	4-30-93	[illegible]	4/30/93

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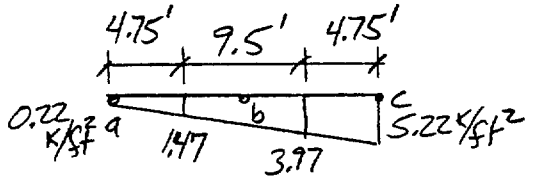
HSM Load transverse direction (cont'd)



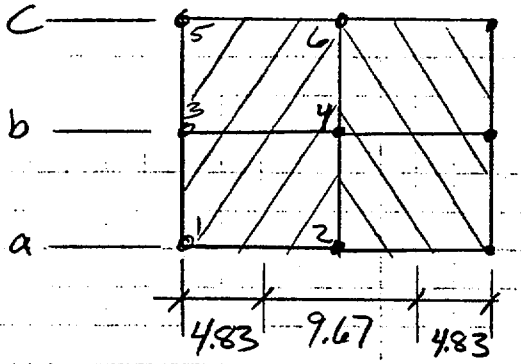
node load for trib distribution

$a = \frac{1}{2}(.22+1.47)4.75 = 4.0\ K/ft$

$b = \frac{1}{2}(1.47+3.97)9.5 = 25.8\ K/ft$

$c = \frac{1}{2}(3.97+5.22)4.75 = 21.8\ K/ft$

HSM FEM mesh :



$1 = 4.83(4\ K/ft) = 19.3\ K$

$2 = 9.67(4) = 38.7\ K$

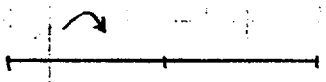
$3 = 4.83(25.8) = 124.6\ K$

$4 = 9.67(25.8) = 249.5\ K$

$5 = 4.83(21.8) = 105.3\ K$

$6 = 9.67(21.8) = 210.8\ K$

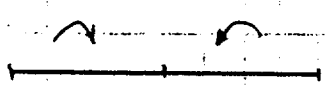
Load Case:
Values per .FEF files in Appendices

#7 

Node 29 : $M_x = \frac{123.2\ k\cdot ft}{5.25'} = 23.5\ k\cdot ft/ft$

#8 

Node 40 : $M_x = \frac{178.0\ k\cdot ft}{5.25'} = 33.9\ k\cdot ft/ft$

#9 

Node 40 $M_x = \frac{177.8\ k\cdot ft}{5.25'} = 33.9\ k\cdot ft/ft$

REV	BY	DATE	CHECKED	DATE
B	MO	5-18-93	[illegible]	5/[illegible]/93
A	MO	4-30-93	[illegible]	4/30/93

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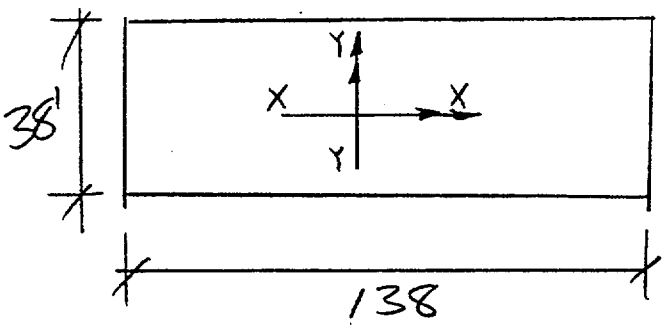
HSM Slab Design



Max moment M_y for all load cases:

Load Case 6 $M_y^+ = M_y^- = 80.9$ k·ft/ft

As defined in the Methodology section

Load due to DL+Seismic ✓ ∴ concrete load factor = 1.4

$\therefore M_u^{+-} = 1.4(80.9) = 113.3$ k·ft/ft

Slab thickness = 24" $\Rightarrow$ $d = 24 - 3 - \frac{1}{2}(.75) = 20.6$ in
(cover, ½ bar ϕ)

Try #6 @ 4" $\Rightarrow$ $\rho = \frac{0.44}{20.6 \times 4} = 0.00534$

$$\phi M_n = \phi A_s f_y d\left(1 - \frac{0.59 \rho f_y}{f'_c}\right)$$

$$\phi M_n = 0.9\left(\frac{12}{4}\right)(.44)(60)(20.6)\left[1 - \frac{.59(.00534)60}{4}\right] = 1400 \frac{\text{k·in}}{\text{ft}} = 116.6 \text{ k·ft/ft}$$

$M_u < \phi M_n$ ✓ OK

REV	BY	DATE	CHECKED	DATE			
B	MB	5-18-93	BDS	5/27/93	ABB Asea Brown Boveri — ABB Impell Corporation	JOB NO 0079-00541	PAGE 20
A	MB	4-30-93	JDL	4/30/93		CALC NO 0079-00541-2	CONT ON 21

HSM Slab Design (Cont'd)

Min shrink & temp steel

gross $A_s = 0.0018(24 \times 12) = 0.52$

Try #5 @ 12" oc T&B

$\therefore$ gross $A_s = 2(.31) = 0.62$ ✓ok

Flexure $d = 24 - 3 - .75 - \frac{1}{2}(.6) \cong 20"$
covers #6 ½ #5

$$\phi M_n = 0.9(.31)60(20)\left[1 - \frac{.59\left(\frac{.31}{12 \times 20}\right)60}{4}\right] = 331 \text{ K·in} = 27.6 \text{ K·ft}$$

Max moment $M_x = 33.9$ K·ft/ft Load Case #8/9

$M_u = 1.4(33.9) = 47.5$ K·ft/ft

Try #6 @ 9" $\Rightarrow \rho = \frac{0.44}{19.8 \times 9} = 0.00247$

$d = 24 - 3 - 1.5(.75) = 19.8"$

$$\phi M_n = 0.9\left(\tfrac{12}{9} \times .44\right)60(19.8)\left[1 - \frac{.59(.00247)60}{4}\right]$$

$\phi M_n = 6.14$ K·in/ft $= 51.1$ K·ft/ft $> M_u$ ✓ok

REV	BY	DATE	CHECKED	DATE			
B	MD	5-18-93	BJB	5/27/93	ABB ASEA BROWN BOVERI ABB Impell Corporation	JOB NO 0079-00541	PAGE 21
A	MD	4-30-93	TOJ	4/30/93		CALC NO 0079-00541-2	CONT ON 22

HSM Soil Pressure

Check soil pressure under HSM slab for following load combinations :

① D+L
② D+E

① D+L : As prev stated, max wt of fully loaded HSM = 500# No. HSM's = 26

Slab dimensions 137'x38'x2' (Approx)

$$\therefore \text{Uniform Soil Pressure} = \frac{26(500)}{137 \times 38}_{\text{HSM's}} + \underbrace{0.15(2)}_{\text{slab DL}}$$

$$= 2.8 \text{ ksf}$$

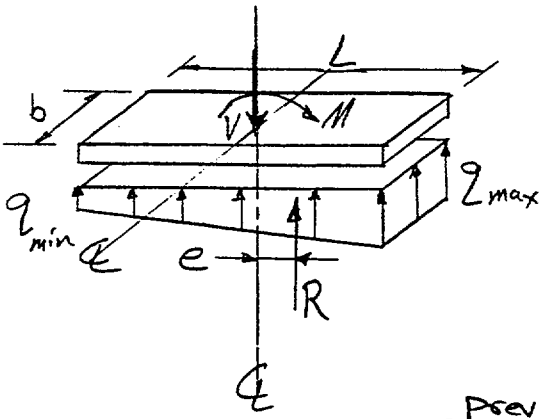
Per Ref 11, allow soil press for D+L = 4ksf ✓ OK

REV	BY	DATE	CHECKED	DATE			
0	MK	6-8-93	BMB	6/10/93	ABB Impell Corporation	JOB NO 0079-00541 CALC NO 0079-0054/-2	PAGE 22 CONT ON 23

HSM Soil Press. (cont'd)

② D+E :

Overturning in Longitudinal Direction :



b = 38'
L = 137'
R = resultant = V
e = eccentricity

M = total overturning moment = 1453.1 K·ft (prev determined) / HSM × 26 HSM's = 37,781 K·ft

V = total Vertical load = 26(500) HSM's + 137 × 38 × 2 × 0.15 slab = 14,562 K

$$e = \frac{M}{V} = \frac{37,781}{14562} = 2.59 \text{ ft}$$

$$\frac{L}{6} = \frac{137}{6} = 22.8 \text{ ft} \quad \therefore \text{ e within kern (middle third)}$$

$$\therefore q = \frac{V}{bL}\left(1 \pm \frac{6e}{L}\right) = \frac{14562}{38 \times 137}\left[1 + \frac{6(2.59)}{137}\right]$$

$$q_{max} = 3.11 \text{ ksf}$$

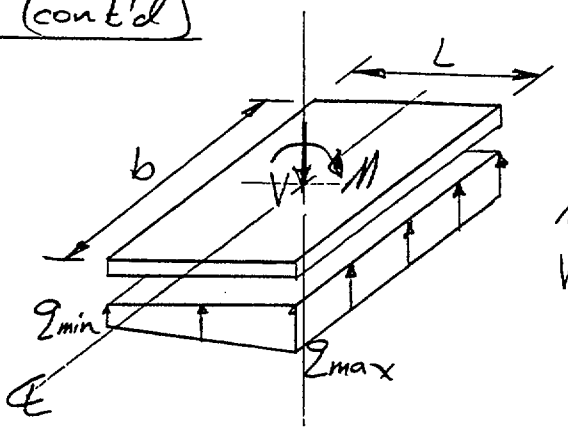
Per Ref 11, allow soil press for D+E = 4/3 (4) = 5.3 ksf ✓ Ok

REV	BY	DATE	CHECKED	DATE		JOB NO	PAGE
					ABB Impell Corporation	CALC NO	CONT ON
0	MAF	6-8-93	PJB	6/10/93		0079-00541-2	24

HSM Soil Press (cont'd)

② D+E

Overturning in Transverse Direction:



b = 137'
L = 38'
M = 37,781 k-ft
V = 14,562 k

as prev determined e = 2.59 ft

$\frac{L}{6} = \frac{38}{6} = 6.3$ ft ∴ e within kern

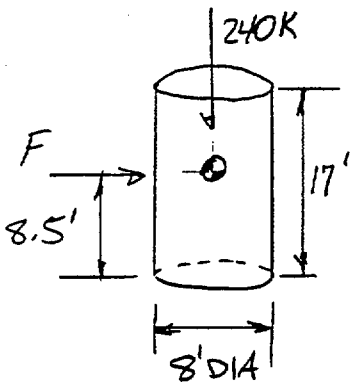
$$\therefore q = \frac{V}{bL}\left(1 \pm \frac{6e}{L}\right) = \frac{14562}{137 \times 38}\left[1 + \frac{6(2.59)}{38}\right]$$

$q_{max} = 3.94$ ksf $< q_{Allow} = 5.3$ ksf ✓OK

REV	BY	DATE	CHECKED	DATE			
0	MA	6-8-93	[illegible]	10/10/93	ABB Impell Corporation	JOB NO 0079-0054/ CALC NO 0079-0054/-2	PAGE 24 CONT ON 25

APRON DESIGN:

Dual Purpose Cask Reaction:



Max wt = 240 K Assume cg @ mid ht

F = 0.375 W (as before)

F = 0.375(240) = 90 K

Check OT:

OT Moment = 90(8.5) = 765 K·ft

Rtg Moment = 240(4) = 960 K·ft

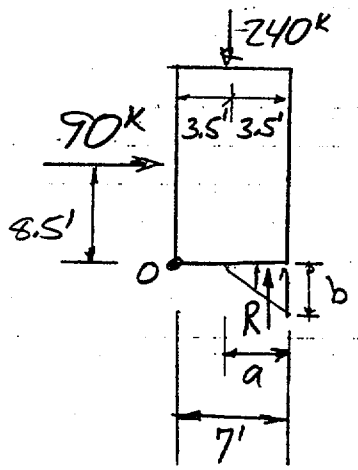
$$F.S. = \frac{960}{765} = 1.25 \quad OK$$

Simplify circular area with a square of equivalent area

Contact area = $\pi 4^2 = 50.3\ ft^2$

Use $7' \times 7' = 49\ ft^2$

Find distribution of reaction



$\Sigma F_{vert} = 0$

$240 - \frac{1}{2}ab = 0 \Rightarrow ab = 480$

$\Sigma M_o = 0$

$90(8.5) + 240(3.5) = \frac{1}{2}ab(7 - \frac{1}{3}a)$

$1605 = \frac{1}{2}(480)(7 - \frac{1}{3}a)$

$a = 0.94\ ft$

$$b = \frac{480}{0.94} = 512\ K/ft$$

$R = 240^K$

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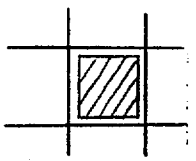
B	MD	5-18-93	BTS	5/27/93	ABB ASEA BROWN BOVERI ABB Impell Corporation	JOB NO 0079-00541	PAGE 25
A	MD	4-30-93	BTS	4/30/93		CALC NO 0079-00541-2	CONT ON 26

Dual Purpose Cask Reaction (cont'd)

Determine pattern of nodes to be loaded by dual purpose cask for DL & seismic conditions:

DL only:

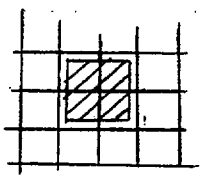
10x10:



Let 4 nodes carry 240^k load

$$\text{load/node} = \frac{240}{4} = 60^k \text{ ea}$$

5x5:



Let 9 nodes carry 240^k load

$$\text{load/node} = \frac{240}{9} = 26.67^k$$

REV	BY	DATE	CHECKED	DATE			
B	MD	5-18-93	BB	5/27/93	ABB ASEA BROWN BOVERI ABB Impell Corporation	JOB NO 0079-00541	PAGE 26
A	MD	4-30-93	BB	4/30/93		CALC NO 0079-00541-2	CONT ON 27

Dual Purpose Cask Reaction (cont'd) :

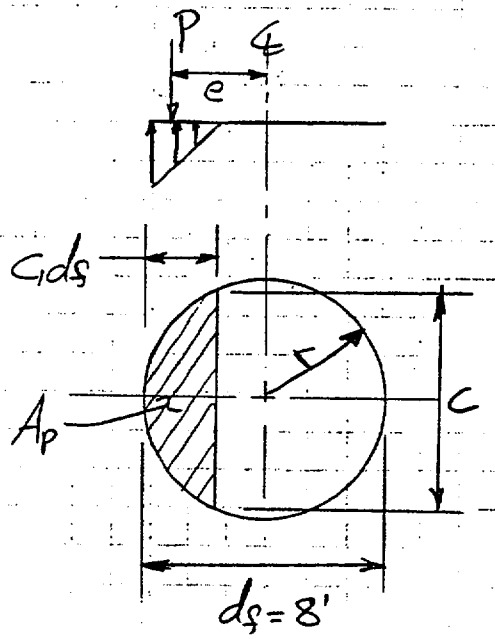
Check punching shear for cask overturning :

Use Reference 8, pg 145 analogy to a circular footing with associated soil bearing pressure to find loaded area under cask with overturning moment. As previously determined, cask is stable and does not overturn.

as prev determined OT moment = 765 k·ft

Cask wt = 240 K , dia = 8 ft

eccentricity $e = \frac{M}{P} = \frac{765}{240} = 3.19$ ft



$\frac{e}{d_s} = \frac{3.19}{8} = 0.3984$

Tables:

e/d_s	C_1	
.375	.295	
.398	.240	← interpolate
.400	.235	

$C_1 d_s = .24(8) = 1.92$ ft

Per Ref 10, pg 6-16

$C = 2\sqrt{2br - b^2}$ where $b = C_1 d_s$

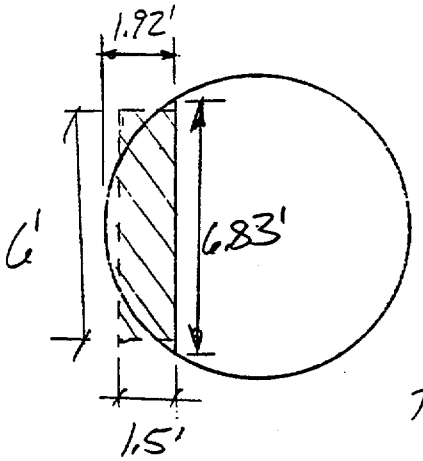
$= 2\sqrt{2(1.92)(4) - 1.92^2}$

$C = 6.83'$

REV	BY	DATE	CHECKED	DATE		JOB NO	PAGE
B	TTO	5-18-93	EPD	4/30/94	ABB Impell Corporation	0079-00541	27
						CALC NO 0079-00541-2	CONT ON 28

Dual Purpose Cask Reaction – Punching Shear (cont'd)

For simplicity, approximate loaded area with a rectangle =



DL+E load factor

$V_u = 1.4(240{,}000) = 336^K$

$\phi V_n = \phi\left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d$

with $\beta_c = \frac{\text{long side}}{\text{short side}} = \frac{6}{1.5} = 4$

$\therefore \phi V_n = \phi 3 \sqrt{f'_c}\, b_o d$

Try $d = 14.6"$ for 18" thick slab

$b_o = (6\times12 + 14.6)2 + (1.5\times12 + 14.6)2$

$b_o = 238.5 \text{ in}$

$\phi V_n = 0.85(3)\sqrt{4000}(238.5)(14.6)$

$\phi V_n = 562^K > V_u$ by factor of $\sim 2\frac{1}{2}$

$\checkmark$ OK which should make up for any non-conservatisms due to approximation of loaded area.

REV	BY	DATE	CHECKED	DATE			
B	[illegible]	5-18-93	[illegible]	4/30/03	ABB Impell Corporation	JOB NO 0079-00541 CALC NO 0079-00541	PAGE 28 CONT ON 29

Transporter Reaction:

Loaded wt = 290^k

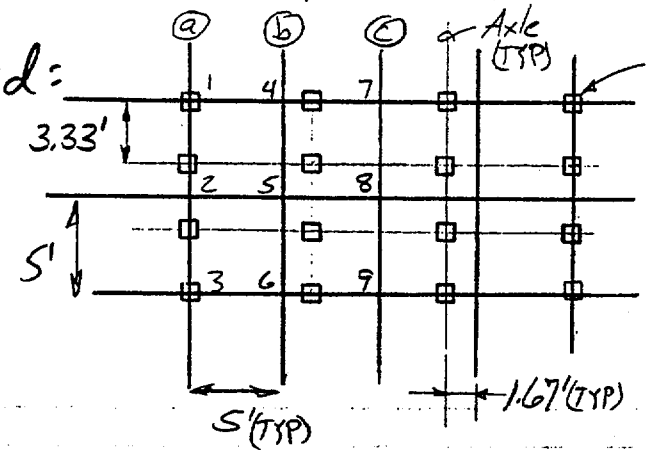
Unloaded wt = 50^k

Wheel Loads:

Length = 20', 4 axles equal space approx, 12' wide

32 tires total, 8 tires/axle

5'x5' Grid:



Possible tire arrangement

Distribute wt as a line load on each axle and apply to nodes per their tributary area

$$load/_{axle} = \frac{290}{4} = 72.5\ ^K/_{axle}$$

load per node line per unit length

a	$72.5^K/10' = 7.25^K/_{ft}$
b	$\frac{3.33}{5}(72.5)/10' = 4.83^K/_{ft}$
c	$2\left[\frac{1.67}{5}(72.5)\right]/10' = 4.83^K/_{ft}$

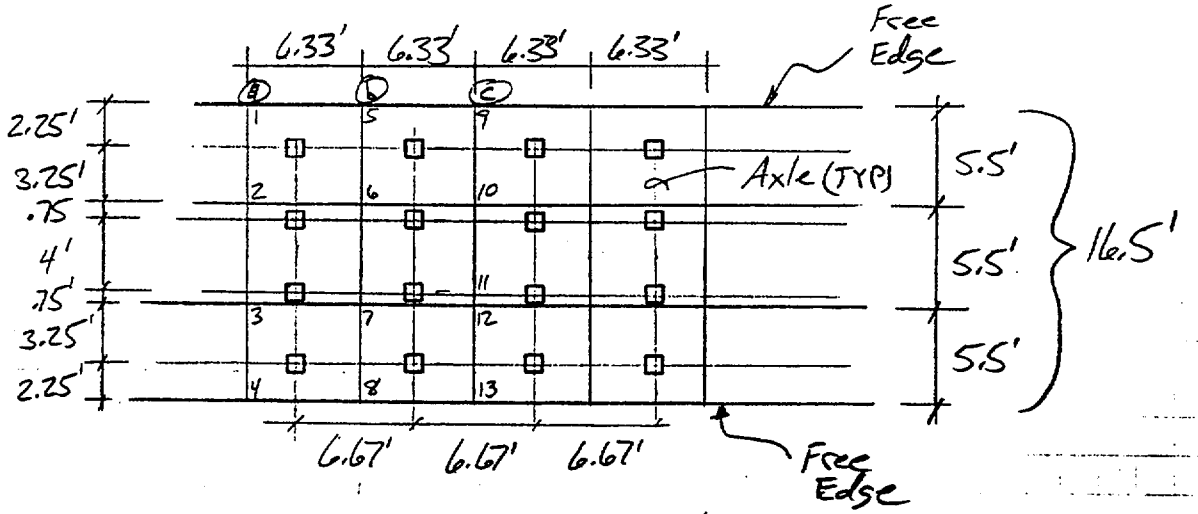
	Loaded Transp.	Unloaded Trans. @ 50/290 factor
1	2.5(7.25) = 18.1 K	3.125
2	5(7.25) = 36.2	6.250
3	= 18.1	3.125
4	2.5(4.83) = 12.1	2.083
5	5(4.83) = 24.2	4.167
6	= 12.1	2.083
7	= 12.1	2.083
8	= 24.2	4.167
9	= 12.1	2.083

Symetrical ↓

REV	BY	DATE	CHECKED	DATE			
B	MD	5-18-93	[illegible]	5/27/93	ABB ASEA BROWN BOVERI ABB Impell Corporation	JOB NO 0079-0054/	PAGE 29
A	MD	4-30-93	[illegible]	4/30/93		CALC NO 0079-0054/-2	CONT ON 30

Transporter Reaction (cont'd)

5'-6" x 6'-4" Grid =



load per axle = 72.5^K/axle

load per node line : $a = \frac{1}{2}\left(\frac{72.5}{16.5}\right) = 2.2^K/ft$

$b = 2(2.2) = 4.4^K/ft$

$c = 4.4^K/ft$

	Load per node
1	$\frac{1}{2}(5.5)(2.2) = 6.05^K$
2	$5.5(2.2) = 12.1^K$
3	12.1^K
4	6.05^K
5	$\frac{1}{2}(5.5)(4.4) = 12.1^K$
6	$(5.5)(4.4) = 24.2^K$
7	24.2^K
8	12.1^K

Symetrical

REV	BY	DATE	CHECKED	DATE
B	MA	5-18-93	[illegible]	5/19/93
A	MA	4-30-93	[illegible]	4/30/93

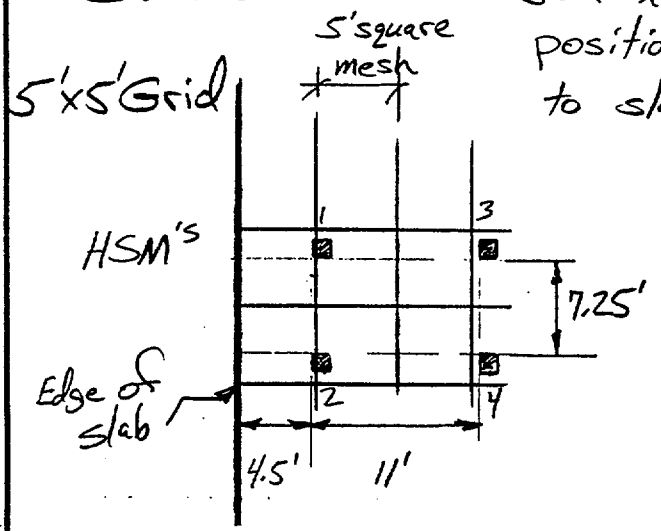
ABB Impell Corporation

JOB NO 0079-00541

CALC NO 0079-00541-2

Transporter Reaction (cont'd)

Jack Loads = Jack locations per [Ref 2]. Jack positions placed as close as possible to slab edge and/or to one-another deemed to be most critical.



Directly apply jack loads to nodes 1-4

Rear Jack = 142^K max

Front Jack = 22^K min

∴

NODE	
1 & 2	= 142^K
3 & 4	= 22^K

Empty Transporter wt = 25 tons = 50^K

equally distributed among 4 jacks = $^{50}/_4$ = 12.5^K/ea

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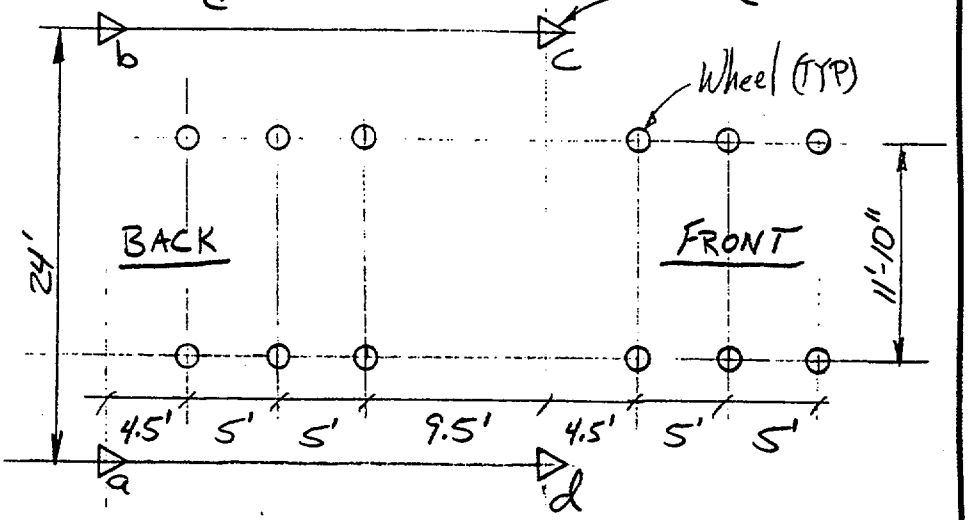
Crane Reaction = [per Ref 4]

Configuration =

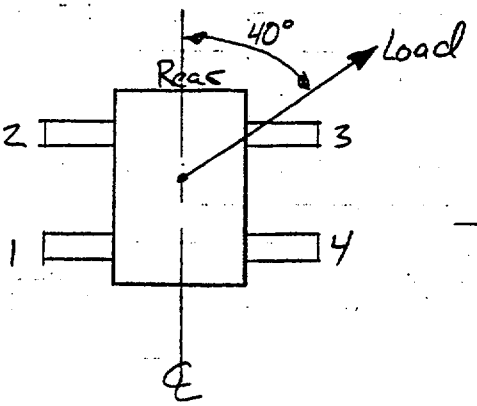


Gross Vehicle Wt = 220.4 K

$$\text{load/tire} = \frac{220.4}{12} = 18.37^{K}$$

Unloaded crane sitting on 4 outriggers → $\frac{\text{load}}{\text{outrigger}} = \frac{220.4}{4} = 55.1\ ^{K}/_{\text{outrigger}}$

Loaded Crane with boom directly over rear outrigger:



Per Reference 4:

Outrigger	LOAD	LOAD/node
1	90 K	÷4 = 22.5 K
2	115 K	÷4 = 28.8 K
3	210 K	÷4 = 52.5 K
4	165 K	÷4 = 41.3 K
	580 K Total	

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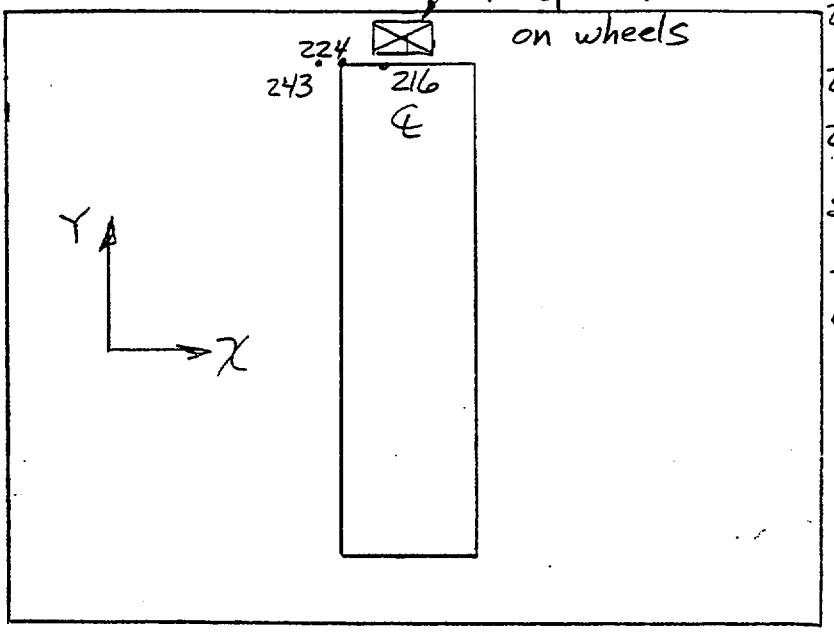
Load Cases - ISFSI

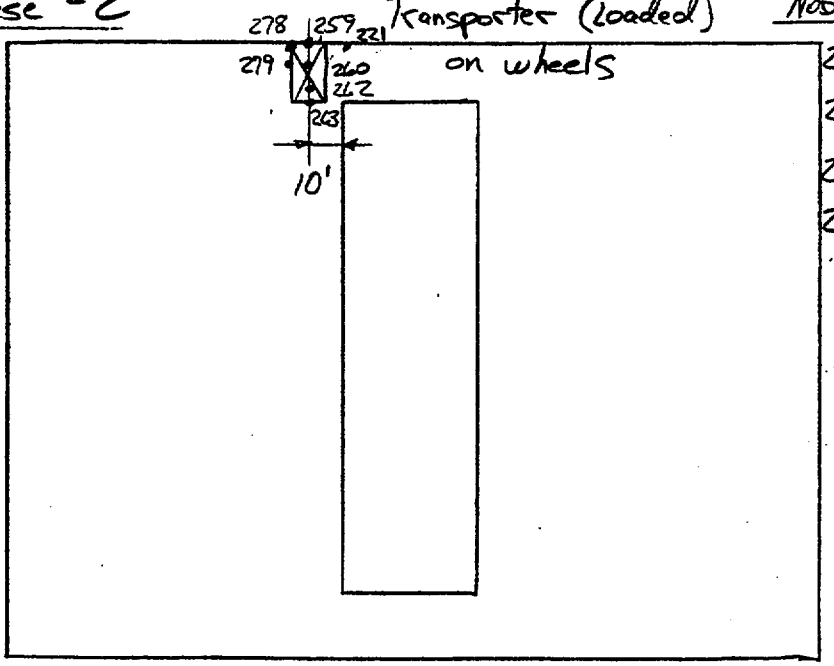
Load Case #1



Node	Max Moments
216	$M_y = +2.7$ K-ft/ft
224	$M_y = -4.2$ K-ft/ft
243	$M_x = -0.3$ K-ft/ft
215	$M_x = +0.4$ K-ft/ft

SEE Appendix for determination of values and locations

Load Case #2



Node	Max Moments
259	$M_y = +15.5$ K-ft/ft
221	$M_y = -6.5$ K-ft/ft
263	$M_x = +6.6$ K-ft/ft
260	$M_x = -3.6$ K-ft/ft

REV	BY	DATE	CHECKED	DATE			
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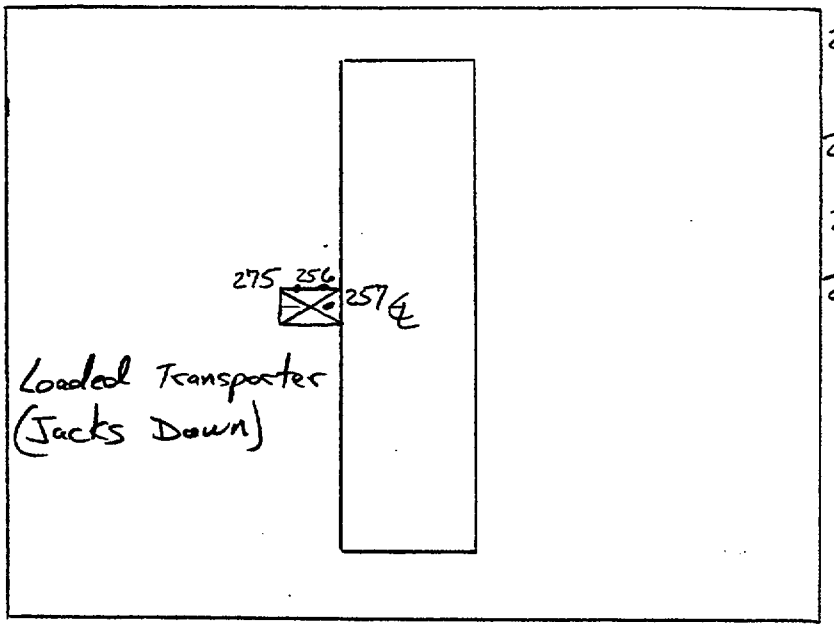
A	MD	4-30-93	[illegible]	4/30/93		CALC NO 0079-00541-2	CONT ON 34

Load Cases – ISFSI (contd)

Load Case #3



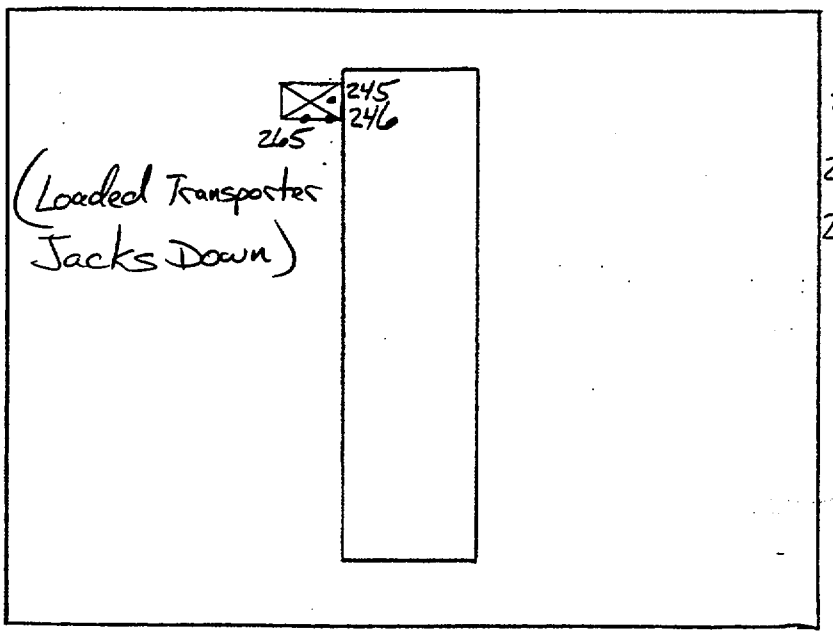
256 $M_x = +26.6$ K-ft/ft

257 $M_x = -8.4$ K-ft/ft

256 $M_y = +25.4$ K-ft/ft

275 $M_y = -7.3$ K-ft/ft

Load Case #4



244 $M_y = +27.2$ K-ft/ft

265 $M_y = -7.2$ K-ft/ft

246 $M_x = +26.9$ K-ft/ft

245 $M_x = -7.9$ K-ft/ft

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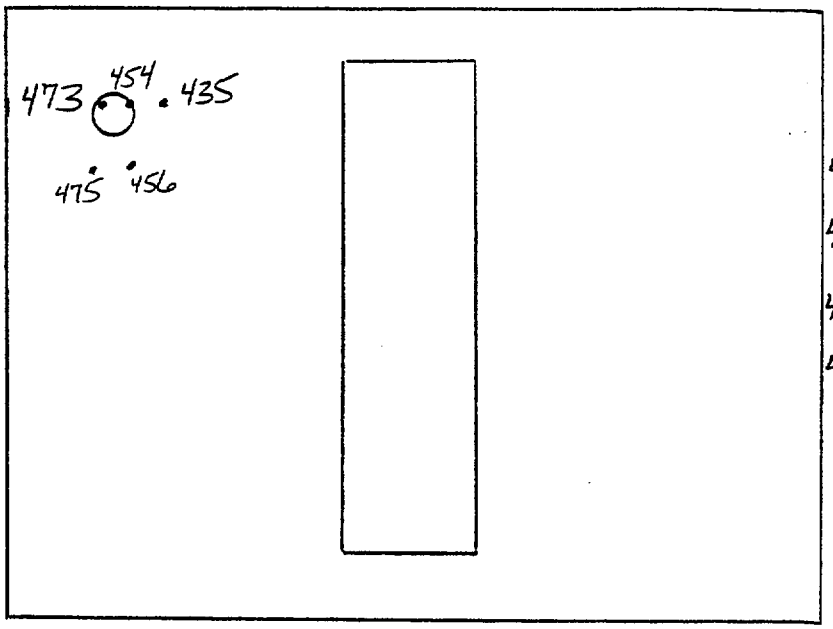
A	MO	4-30-93	[illegible]	4/30/93			

Load Cases – ISFSI (cont'd)

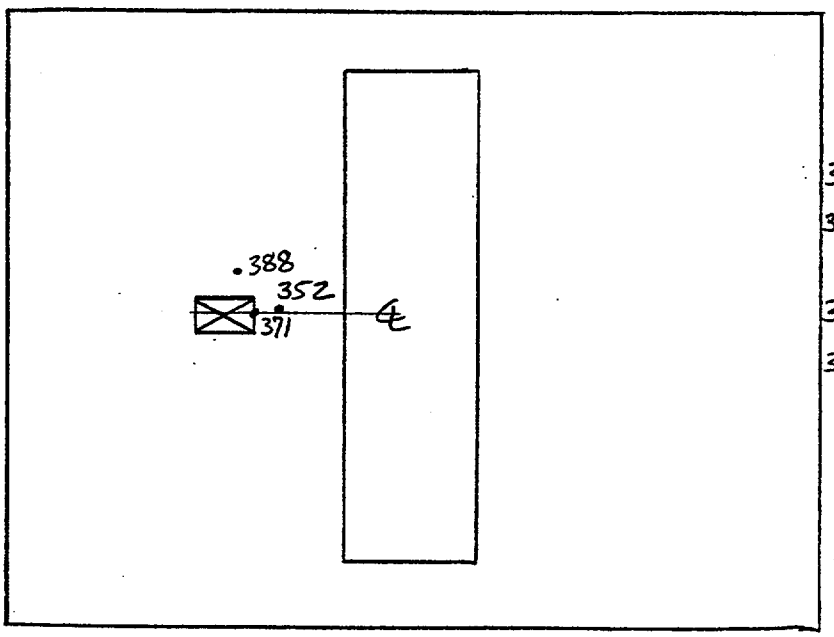
Load Case 5



Dual Purpose Cask w/ Seismic

473 $^{+}M_x$ = 29.6 K-ft/ft
456 $^{-}M_x$ = 4.9 K-ft/ft
473 $^{+}M_y$ = 20.7 K-ft/ft
435 $^{-}M_y$ = 6.6 K-ft/ft

Load Case 6



Transporter (loaded) on wheels

371 $^{+}M_y$ = 4.7 K-ft/ft
352 $^{-}M_y$ = 2.5 K-ft/ft
371 $^{+}M_x$ = 4.7 K-ft/ft
388 $^{-}M_x$ = 1.9 K-ft/ft

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Loaded Crane, Max Outrigger Load

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563	544	525	506	487	468	449	430	411	392	373	354	335	316	297	278	259	240	221	217
564	545	526	507	488	469	450	431	412	393	374	355	336	317	298	279	260	241	222	218
565	546	527	508	489	470	451	432	413	394	375	356	337	318	299	280	261	242	223	219
566	547	528	509	490	471	452	433	414	395	376	357	338	319	300	281	262	243	224	220
567	548	529	510	491	472	453	434	415	396	377	358	339	320	301	282	263	244	225	
568	549	530	511	492	473	454	435	416	397	378	359	340	321	302	283	264	245	226	
569	550	531	512	493	474	455	436	417	398	379	360	341	322	303	284	265	246	227	
570	551	532	513	494	475	456	437	418	399	380	361	342	323	304	285	266	247	228	
571	552	533	514	495	476	457	438	419	400	381	362	343	324	305	286	267	248	229	
572	553	534	515	496	477	458	439	420	401	382	363	344	325	306	287	268	249	230	
573	554	535	516	497	478	459	440	421	402	383	364	345	326	307	288	269	250	231	
574	555	536	517	498	479	460	441	422	403	384	365	346	327	308	289	270	251	232	
575	556	537	518	499	480	461	442	423	404	385	366	347	328	309	290	271	252	233	
576	557	538	519	500	481	462	443												
577	558	539	520	501	482	463	444												
578	559	540	521	502	483	464	445												

LOAD CASE #7

■ Transporter Jack ● Crane Outrigger

436 $+M_y$ = 11.4 K-ft/ft 475 $-M_y$ = 5.4 K-ft/ft

455 $+M_x$ = 11.7 K-ft/ft 458 $-M_x$ = 6.3 K-ft/ft

442 $+M_y$ = 11.4 K-ft/ft 480 $-M_y$ = 6.7 K-ft/ft

441 $+M_x$ = 7.9 K-ft/ft 462 $-M_x$ = 2.3 K-ft/ft

Y
X

APRON
UNDEFORMED SHAPE

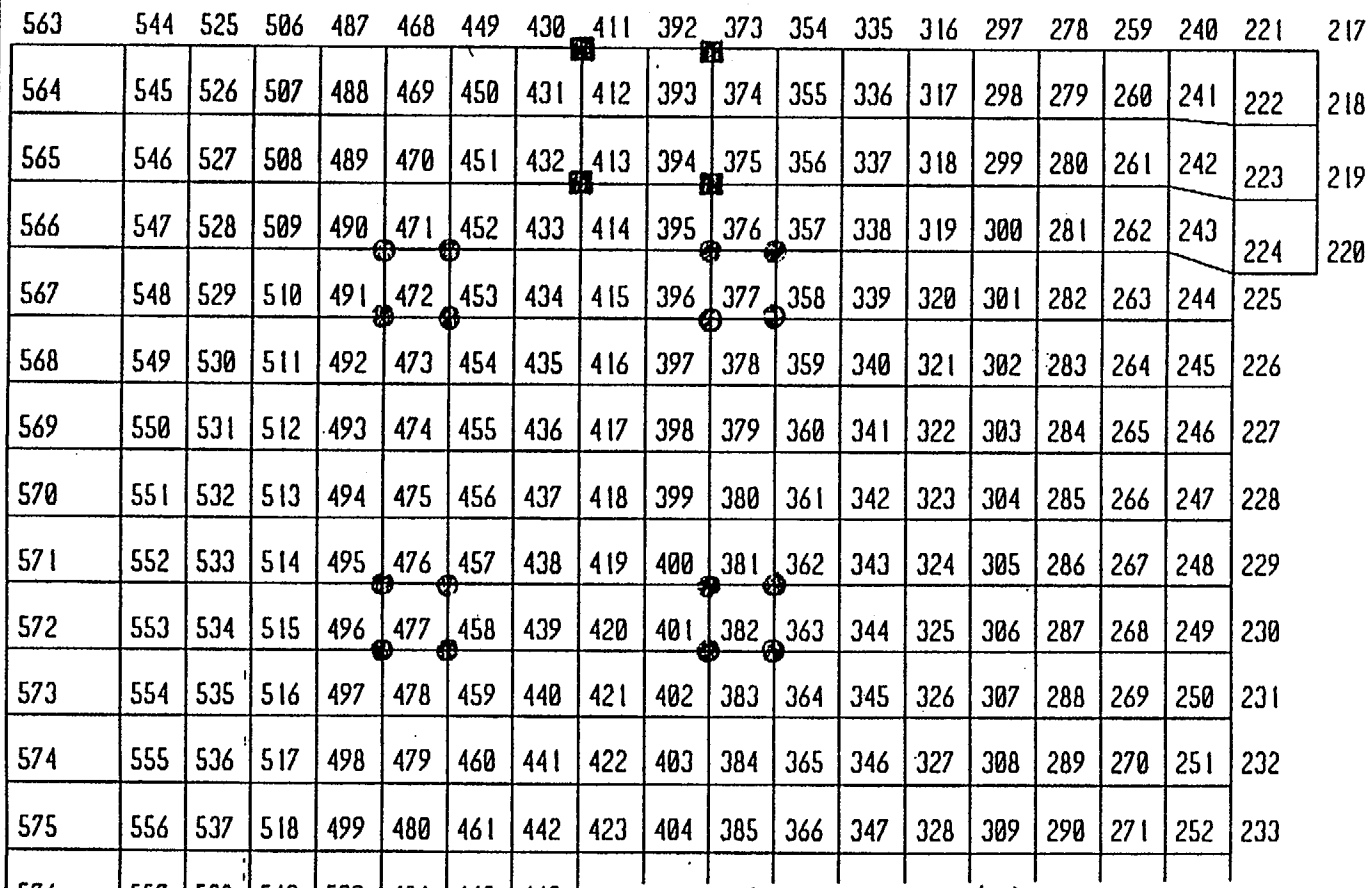
OPTIONS
JOINT IDS
WIRE FRAME

SAP90

38

Z-DRY-CU24

Loaded Transporter, Max Outrigger Load



LOAD CASE #8

■ TRANSPORTER JACK ● CRANE OUTRIGGER

412 $-M_X = 22.4$ K-ft/ft 413 $+M_X = 19.7$ K-ft/ft

411 $+M_Y = 60.5$ K-ft/ft 449 $-M_Y = 11.8$ K-ft/ft

Y
X

APRON

UNDEFORMED SHAPE

OPTIONS

JOINT IDS
WIRE FRAME

SAP90

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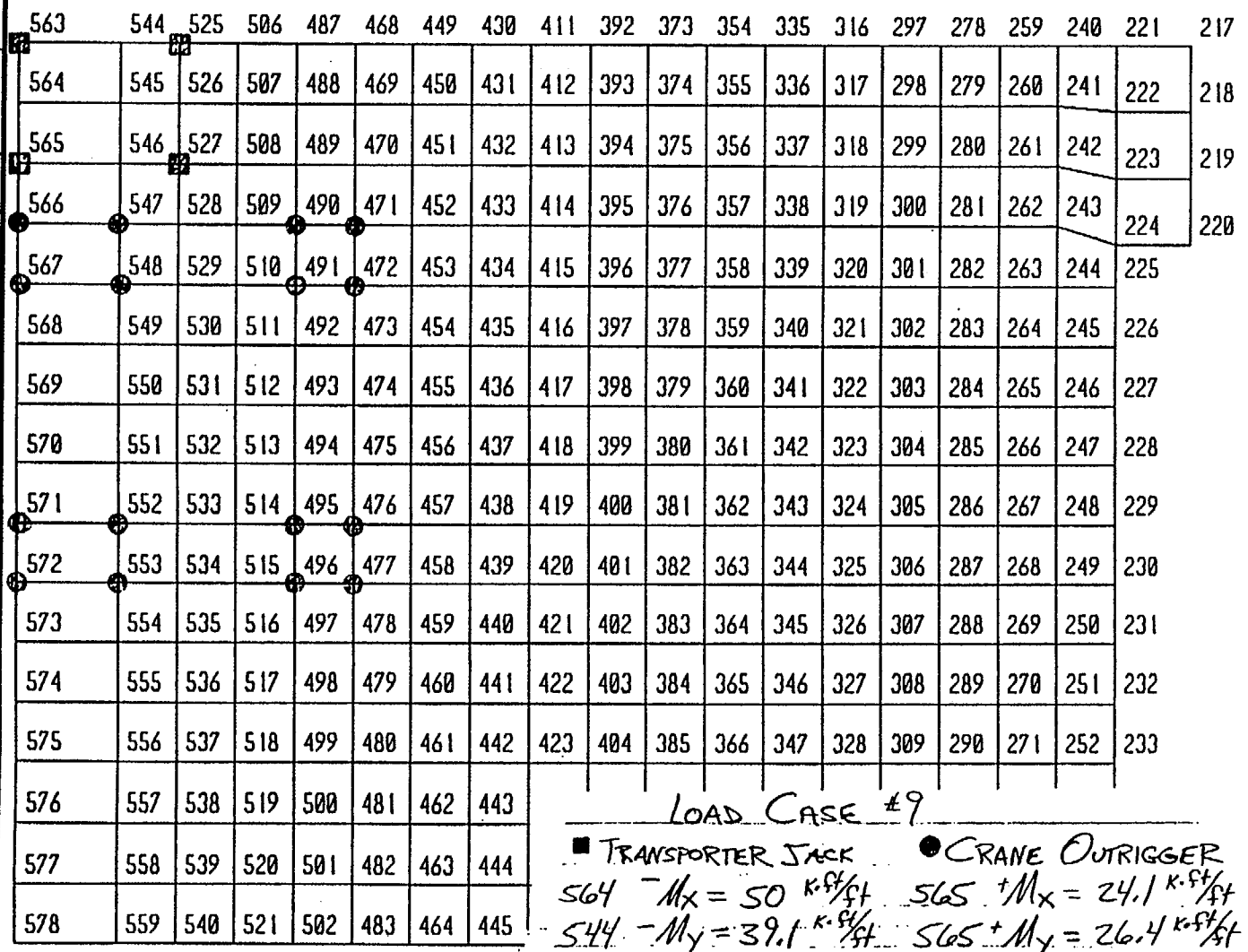
Z-DRY-C1024

Loaded Transporter, Max Outrigger Load



LOAD CASE #9

■ TRANSPORTER JACK ● CRANE OUTRIGGER

564 $^{-}M_x = 50$ K-FT/FT 565 $^{+}M_x = 24.1$ K-FT/FT

544 $^{-}M_y = 39.1$ K-FT/FT 565 $^{+}M_y = 26.4$ K-FT/FT

Y X

APRON

UNDEFORMED SHAPE

OPTIONS

JOINT IDS

WIRE FRAME

SAP90

REV	BY	DATE	CHECKED	DATE
B	MJC	5/18/93	[illegible]	5/27/93

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Z-DRY-C1024

Apron Slab Design

Establish slab capacity with shrink & temp steel only:

1' slab: min steel = $0.0018(12\times12) = 0.259\ in^2/ft$

#4 @ 16" T&B $\Rightarrow A_{s_{gross}} = \frac{2(.2)(12)}{16} = 0.3\ in^2/ft$

#3 @ 10" T&B

#5 @ 28" T&B NG min space @ 18"

Try #4 @ 16" T&B $\quad +d = 12-3-1.5(.5) = 8.25"$ $\quad -d = 12-2-1.5(.5) = 9.25"$

flexure: $\rho^+ = \frac{0.2}{8.25\times16} = 0.00152$ $\quad \rho^- = \frac{0.2}{9.25\times16} = 0.00135$

$$\phi M_n = \phi A_s f_y d\left(1 - \frac{0.59\rho f_y}{f'_c}\right)$$

$$^+\phi M_n = 0.9\left(\tfrac{12}{16}\times0.2\right)60(8.25)\left(1 - \frac{0.59(.0015)60}{4}\right)$$

$$^+\phi M_n = 65.9\ k\cdot in = 5.5\ k\cdot ft$$

$$M_u \le \phi M_n$$

$$1.7 M_{max} \le \phi M_n \Rightarrow M^{(+)}_{max} \le \frac{5.5}{1.7} = 3.23\ k\cdot ft$$

$$-\phi M_n = 0.9\left(\tfrac{12}{16}\times0.2\right)60(9.25)\left[1 - \frac{0.59(.00135)60}{4}\right]$$

$$-\phi M_n = 74\ k\cdot in = 6.2\ k\cdot ft$$

$$M^{(-)}_{max} = \frac{6.2}{1.7} = 3.6\ k\cdot ft$$

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A	MD	4-30-93	TM	4/30/93

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Apron Slab Design (Contd)

Establish capacity with shrink & temp steel only

1'-6" slab:

min steel = $0.0018(12\times18) = 0.389\ in^2/ft$

#3 @ 6½" T&B

#4 @ 12" T&B $\Rightarrow A_s = 2(.2) = 0.4\ in^2/ft$

#5 @ 18" T&B

Try #4@12"

Flexure: $d^+ = 18-3-1.5(.5) = 13.5"$ $d^- = 18-2-1.5(.5) = 15.25"$

$$\rho^+ = \frac{0.2}{12\times13.5} = 0.00123 \qquad \rho^- = \frac{0.2}{12\times15.25} = 0.0011$$

$$(+)\ \phi M_n = 0.9(.2)(60)(13.5)\left[1-\frac{.59(.00123)60}{4}\right] = 144.2\ k\cdot in = 12\ k\cdot ft$$

$$(+)\ M_{max} = \frac{12}{1.7} = 7.1\ k\cdot ft$$

$$(\leftarrow)\ \phi M_n = 0.9(.2)(60)(15.25)\left[1-\frac{.59(.0011)60}{4}\right] = 163\ k\cdot in = 13.6\ k\cdot ft$$

$$(\leftarrow)\ M_{max} = \frac{13.6}{1.7} = 8.0\ k\cdot ft$$

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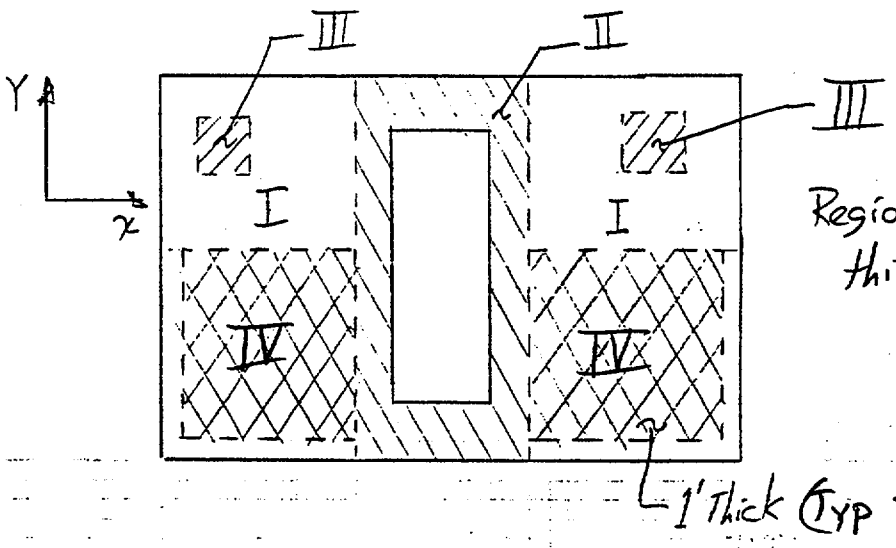
A	MD	4-30-93	TDJ	4/30/93		CALC NO 0079-0054I-2	CONT ON 41

Apron Slab design (cont'd)

Overview of Design Moments:



Regions I, II, III slab thickness 1'-6"

For each load case envelope positive & negative moments between X & Y directions so that slab design will have same steel T&B, EW. Look @ providing different steel for the 4 different regions marked on the Apron

Load Case	Max Moment	Region
1 D+L	4.2 K-ft/ft	II
2 D+L	-15.5 K-ft/ft	II
3 D+L	26.6 K-ft/ft	II
4 D+L	27.2 K-ft/ft	II
5 D+Seismic	29.6 K-ft/ft	III
6 D+L	4.7 K-ft/ft	IV
7 D+L	11.7 K-ft/ft	I
8 D+L	60.5 K-ft/ft	I
9 D+L	50 K-ft/ft	I

Load Cases not used since outriggers will not be allowed w/in 10' of slab perimeter

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Apron Slab Design 1'-6" thick portion:

Region I $\Rightarrow$ $M = 11.7$ k-ft/ft

$M_u = 1.7(11.7) = 19.9$ k-ft/ft

$d = 18 - 3 - 1.5(.75) = 13.8"$
(cover, 1½ x #6 ϕ)

Try #6 @ 14 oc $\Rightarrow$ $\rho_{flex} = \frac{0.44}{13.8 \times 14} = 0.00228$

$$\phi M_n = \phi A_s f_y d \left[1 - \frac{0.59 \rho f_y}{f'_c}\right]$$

$$\phi M_n = 0.9\left(\frac{12}{14} \times .44\right) 60 (13.8) \left[1 - \frac{0.59(.00228)60}{4}\right]$$

$\phi M_n = 275$ k-in/ft $= 23$ k-ft/ft $> M_u$ ✓ ok

Use #6 @ 14" T&B EW

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Apron Slab Design (Cont'd) 1'-6" portion:

Region II:

Critical Design Moment = 27.2 k·ft/ft

$$M_u = 1.7(27.2) = 46.2 \text{ k·ft/ft}$$

(1.7 = live load factor)

$$d = 18 - 3 - 1.5(.75) = 13.8"$$

(3" = cover, 1.5 = 1½ × #6 bar ϕ)

Try #6 @ 6" oc $\Rightarrow$ $\rho_{flex} = \frac{0.44}{6 \times 13.8} = 0.00531$

$$\phi M_n = \phi A_s f_y d \left[1 - \frac{0.59 \rho f_y}{f'_c}\right]$$

$$= 0.9(2 \times .44)\, 60(13.8)\left[1 - \frac{0.59(.00531)60}{4}\right]$$

$$\phi M_n = 625 \text{ k·in/ft} = 52 \text{ k·ft/ft} > M_u \checkmark \text{ OK}$$

use #6 @ 6" top & bottom each way

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Apron Slab Design (cont'd) 1'-6" portion:

Region III = max moment = 29.6 k-ft/ft pos bending

for DL + Seismic $M_u = 1.4(29.6) = 41.4$ k-ft/ft

Since this is pos bending only & region III is within region I, look @ providing same top steel as for region I & just increasing No. of bars on lower mat under dual purpose casks to handle larger pos moment.

Region I bars are #6 @ 14" oc — Try #6 @ 7" for lower mat in Region III: $\rho = \frac{0.44}{13.8 \times 7} = 0.00455$

$$\phi M_n = 0.9\left(\tfrac{12}{7} \times 0.44\right) 60 (13.8)\left[1 - \frac{0.59(0.00455)60}{4}\right]$$

$\phi M_n = 539$ k-in/ft $= 45.0$ k-ft/ft $> M_u$ ✓ ok

use #6 @ 7", lower mat under dual purpose casks

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Apron Slab design (cont'd)

Design 1' thick portion

Load Case 6 : envelope top & bottom, both directions

Design moment = 4.7 k-ft/ft (DL+LL)

$M_u = 1.7(4.7) = 8.0$ k-ft/ft

$d = 12 - 3 - 1.5(.5) = 8.25"$

Try #4@10" $\Rightarrow$ $\rho = \frac{0.2}{8.25 \times 10} = 0.00242$

$\phi M_n = 0.9\left(\frac{12}{10} \times .2\right) 60 (8.25)\left[1 - \frac{.59(.00242)60}{4}\right]$

$\phi M_n = 10.5$ k-in/ft $= 8.7$ k-ft/ft $> M_u$ ✓ OK

use #4@10" over 1' thick portion

@ 1'-6" thick perimeter, min steel req'd for shrink & temp was prev determined as #4@12"

∴ If continuing #4@10"@ perimeter were ok

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Apron Slab Design (cont'd)

Determine lap splice for #6 bar

UBC § 2612 development length $l_d = 0.04 A_b f_y / \sqrt{f'_c}$

with following factors =

Top reinforcement = 1.3 (No Hooks)

1'-6" section only

Use Class B lap splice = $1.3 l_d$

$$\therefore \text{lap splice} = 1.3 l_d = \frac{1.3(1.3)(0.04)(.44)\,60{,}000}{\sqrt{4000}}$$

lap splice = 28.2"

Use 2'-6" = 30"

Soil Pressure =

By comparison to soil pressure analysis for HSM slab and considering the size of the HSM slab and total applied vertical load, the soil pressure for the apron slab is judged to be non critical and well within the allowable values set forth in Ref 11.

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0	MD	6-8-93	[illegible]	6/10/93
B	MD	5-18-93	[illegible]	5/27/93

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