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M. O. MEDFORD
MANAGER, NUCLEAR LICENSING

TELEPHONE
(818) 302-1749

April 26, 1985

Director, Office of Nuclear Reactor Regulation
Attention: J. A. Zwolinski, Chief
Operating Reactors Branch No. 5
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D.C. 20555

Gentlemen:

Subject: Docket No. 50-206
SEP Topic II-3.B, Flooding Potential and Protection Requirements
San Onofre Nuclear Generating Station
Unit 1

References: A. Letter, Walter A Paulson, NRC, to Kenneth P. Baskin, SCE, dated August 27, 1984

B. Letter M. O. Medford, SCE, to D. M. Crutchfield, NRC, dated February 24, 1984

The Reference A letter provided the NRC staff's Safety Evaluation Report on the subject topic. One of the conclusions of that report was that the structural capacity of the Fuel Storage and Ventilation Building roofs required further review. Two issues are associated with the rooftop loading. The first is the appropriate coefficient of discharge used in calculating flow through the scuppers. The second is the appropriateness of using a stress increase factor of 1.6 in the roofdeck stress calculations. In conversations with members of the NRC staff, the information necessary to complete the review was identified. This letter transmits the information necessary to resolve the rooftop ponding issue associated with this topic.

The ponding depths submitted in the Reference B letter were based on calculations that did not consider a coefficient of discharge. The coefficient used in the enclosed calculations is 0.82. This number is greater than the NRC recommended value of 0.6, but is justified by review of the "Handbook of Hydraulics" by King and Brater (6th Edition). The NRC recommended value of 0.6 is an average value for sharp edged circular orifices under medium to high head. This is not an appropriate modeling of the SONGS 1 scuppers. The orifice and head configuration assumed by the NRC acts to maximize the contraction of the jet and results in a low coefficient of discharge. There are three reasons why the scupper configuration has a higher coefficient of discharge than 0.6. The first is the length of the scupper in

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DRAWINGS TO W. PAULSON A028 1/1

relation to its area. The handbook cited above, on page 4-19, defines a standard short tube as a tube whose length is 2 to 3 times its diameter. In such cases the coefficient of discharge generally used is 0.82. Relating the scupper dimensions (2 inch by 8 inch rectangular) to tube diameters would yield length to diameter ratios of 4 to 1 and 1 to 1 respectively, with an average value of 2.5. This value is within the defined range for a standard short tube. If an equivalent diameter of tube is considered based on the area of the scupper, the length to diameter ratio is 1.8, also very near the defined range. Table 4-10 of King's handbook further confirms this coefficient. In that table a coefficient of 0.80 is listed for the orifice dimensions and end conditions of the scuppers. It should be noted that while this table is for submerged tubes, pages 4-9 and 4-10 indicate that submergence does not greatly affect the coefficient. The second reason for a greater than 0.6 coefficient is the rectangular shape. Table 4-5 of King's Handbook indicates that a rectangular orifice similar to the scuppers would have a coefficient 3% greater than for a circular orifice with the same area and head conditions. The third reason for a 0.82 coefficient is due to the low head of the orifice. Figure 4-6 of King's Handbook indicates an increase in the coefficient for low heads.

The revised maximum rooftop ponding depths on the Ventilation Equipment and Fuel Storage Buildings due to the inclusion of this coefficient of discharge were determined to be 5.3 inches and 9.07 inches respectively as opposed to the 5.00 inch and 7.64 inch values previously submitted.

The second issue requiring resolution is the justification for using a stress increase factor of 1.6 in the roof deck calculations. Upon reexamination of the bending calculations to consider the revised ponding depths and to determine the necessity to take credit for the 1.6 stress increase factor, it was determined that the bending stresses resulting from these ponding loads are below the allowable stresses with no credit taken for the 1.6 factor. The allowable stress is in accordance with AISI publication "Specification for the Design of Cold-Formed Steel Structural Members." Enclosure 2 is a copy of the calculations for the roof framing and deck analysis. Also enclosed as requested by members of the NRC staff to assist in reviewing the bases for these conclusions are SCE Design Drawings Nos. 567676, 567677, 567678, 567892, 568139, 568140 and 568145; Specification No. 3246-BSO-258-1-1 by Inland Steel Products Co.; and portions of Inland Floor System Catalog No. 270 indicating properties and material for Type BF 18-16 decking.

This information should be sufficient to conclude your review and confirm our determination that these rooftops are adequately designed to withstand ponding loads resulting from a Probable Maximum Precipitation event at the San Onofre site. If you have any questions, please contact me.

Very truly yours,

M.O. Medford

Enclosures

1. Calculation, "Roof Ponding of Fuel Storage Building and Ventilation Building, Part IVA"
2. Calculation, "Roof Ponding of Fuel Storage Building and Ventilation Building (Structural) Part IVB"
3. SCE Drawing Nos. 567676, 567677, 567678, 567892, 568139, 568140, 568145
4. Inland Steel Products Specification No. 3246-BSO-258-1-1
5. Inland Floor Systems Catalog Page 18

CALCULATION TITLE PAGE

PROJECT SONGS I PMF JOB ORDER NO. 6733 DISCIPLINE CIVIL

SUBJECT: ROOF PONDING OF FUEL STORAGE BUILDING AND VENTILATION BUILDING

CALCULATION NO. DC 99 PART IV A NO. PAGES 8

RESPONSIBLE ENGINEER (signature) T. WANG DATE 12/2/83

INDEPENDENT REVIEW ENG. (signature) D. SCHAFER DATE 12/9/83

ORIGINAL ISSUE

	NAME	DATE	SIGNATURE
GROUP LEADER	C. M. Knarr	2-14-84	
DISCIPLINE SUP. ENGR.	D. H. Johns	2-15-84	
OTHER			

RECORD OF REVISIONS

NO.	REASON FOR REVISION	DATE	RESP. ENGR.	IRE	GL	DSE	OTHER
1	Change coefficient of Discharge for scuppers	8-10-84	T. W.		CMK		

SHEET 1 OF 8 SHEETS

ENGINEERING DEPARTMENT

CALCULATION SHEET

SUBJECT: Roof Ponding of FUEL STORAGE BLDG AND VENTILATION BUILDING. DESIGN CALCULATION NO. DC 99 - IV A

J.O. NO. 6733 MADE BY T. Wang DATE 2/10/84 CHK. BY JS DATE 2/10-84

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DWG. NO.

DESIGN INPUT SHEET

[X] Design Requirements
[ ] Design Input Change

DESIGN CALCULATION NO. DC 99 — IV A

SUBJECT		PROJECT
ROOF PONDING OF FUEL STORAGE BLDG AND VENTILATION BLDG		SONGS 1 PMF
QUALITY CLASS	SEISMIC CLASS	SPECIFICATION REFERENCE
SAFETY RELATED	A	

Design Input

Purpose: Studies the PMP roof top ponding of fuel storage building and ventilation building in case the roofdrains are clogged.

References: 1. SCE DWG's 568145, 567676, 567677, 567678, 567892.
2. "Design of Roadside Drainage Channels", by U.S. Bureau of Public Works.
3. "Handbook of Hydralics", by King, 5th Edition.
4. Probable Maximum Flood Project Design Criteria Manual, SONGS 1, 8/31/77.

Design Assumptions: 1. Time of concentration for design = 5 min.
2. Use Rational Method to determine the peak flow.

RESPONSIBLE ENGINEER	DATE	INDEPENDENT REVIEW ENGINEER	DATE	GROUP LEADER	DATE
Tingz Wang	12/7/83	[signature]	12-9-83	[signature]	2-14-84

ENGINEERING DEPARTMENT

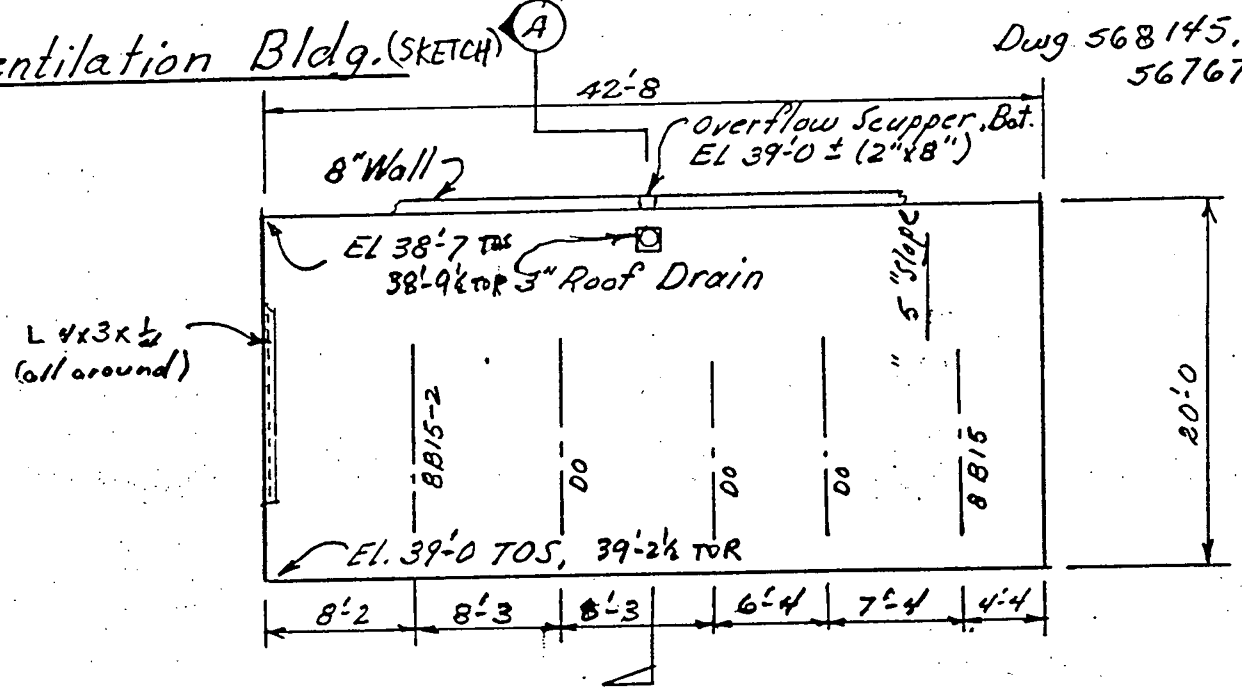
CALCULATION SHEET

SUBJECT: Roof Ponding of Fuel Storage Bldg & Ventilation Bldg. DESIGN CALCULATION NO. DC 99 - IV A

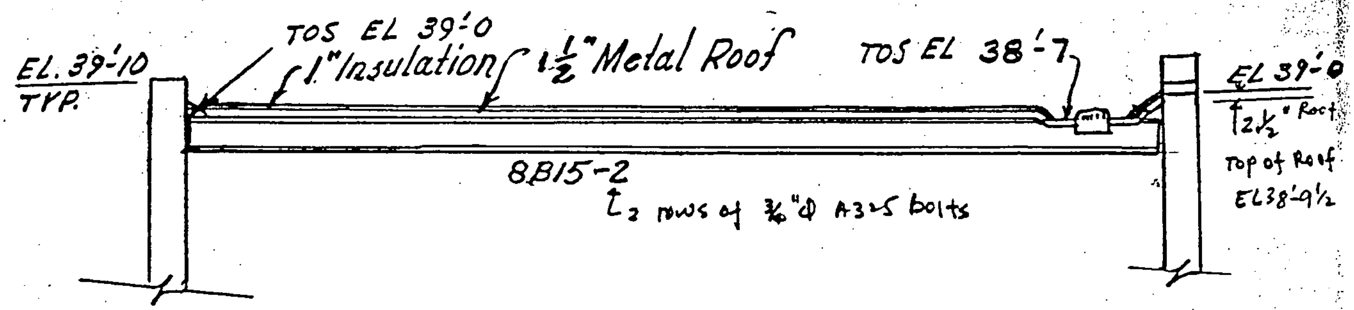
J.O. NO. 6733 MADE BY Dotson/T. Wang DATE 3/30/83 CHK. BY [signature] DATE 5-4-83

I. Ventilation Bldg. (SKETCH)

Dwg 568145, 567676



Roof area = 20 x 42.67 = 853.4 sq. ft.



Section A

DWG. NO. 568145 567676

ENGINEERING DEPARTMENT

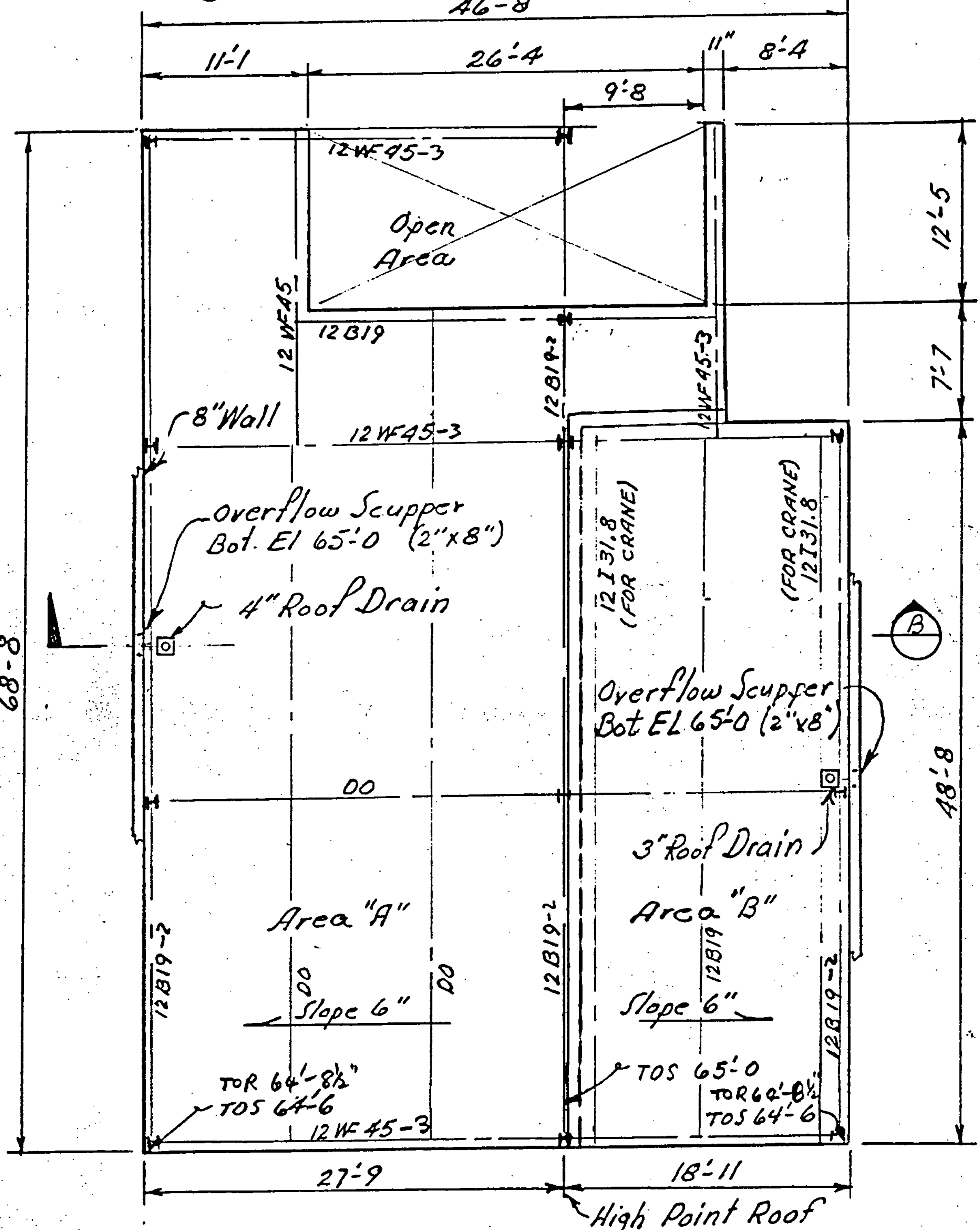
CALCULATION SHEET

SUBJECT: Roof Ponding of Fuel Storage Bldg & Ventilation Bldg DESIGN CALCULATION NO. DC 99 - IV A

J.O. NO. 6733 MADE BY T. Wang/G. Dotson DATE 3/30/83 CHK. BY [signature] DATE 4-1-83

II. Fuel Storage Bldg. (SKETCH)

Dwg. 568145, 567676



Area = 27.75 x 56.25 + 12.42 x 11.08 = 1698.6 □' Area "A"

18.92 x 48.67 + 7.58 x 9.67 + 12.42 x .92 = 1005.6 Sq. ft. Area "B"

Total = 2704.2 Sq. ft.

3/32"

DWG. NO. 568145

SHEET 5 OF 8 SHEETS

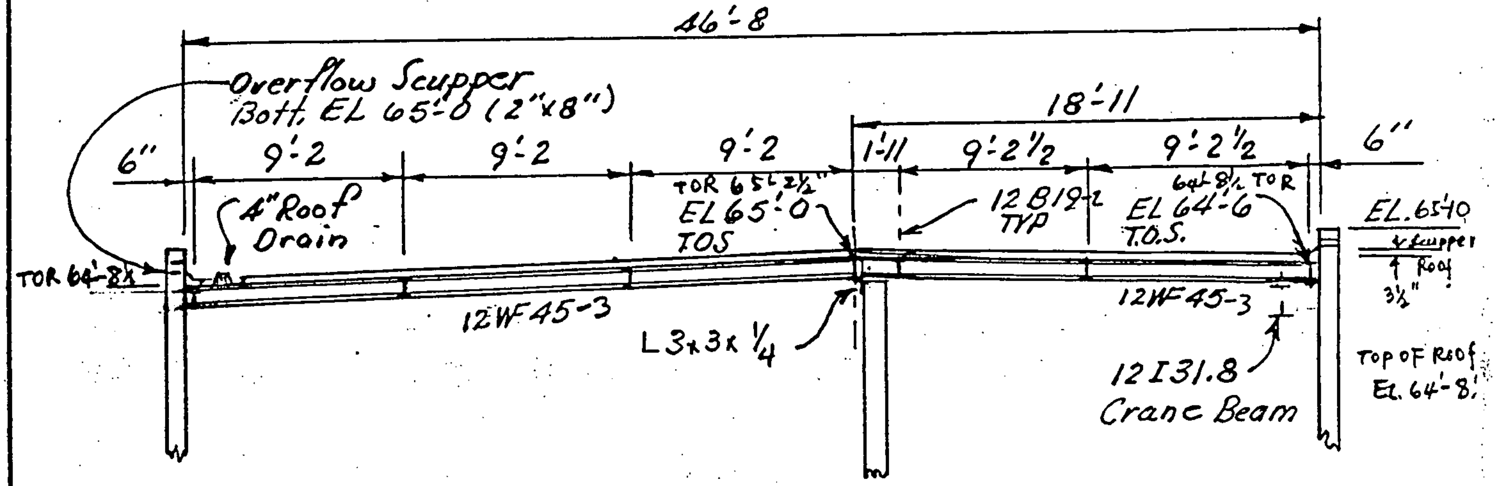
ENGINEERING DEPARTMENT

CALCULATION SHEET

SUBJECT: Roof Ponding of F.S.B & V.B. DESIGN CALCULATION NO. DC 99 - IV A

J.O. NO. 6733 MADE BY T. Wang/L. Dokm DATE 3/10/83 CHK. BY DJ DATE 4-1-83

II. Fuel Storage Bldg. (SKETCH)



Section (B)

DWG. NO. 568145
567676

ENGINEERING DEPARTMENT

CALCULATION SHEET

SUBJECT: Roof Ponding of Fuel Storage Bldgs & Ventilation Bldg DESIGN CALCULATION NO. DC 99 - IV A

J.O. NO. 6733 MADE BY T. WANG DATE 3/30/83 CHK. BY DS DATE 4-1-83

III. Rainfall Intensity for Design:

From Table 1.2-1, Probable Maximum Flood project Design Criteria Manual, SONGS 1

Time (hours)	PMP Total (in)	Intensity (in/hr)
0.25	3.01	12.04
0.50	4.90	9.8
0.75	6.16	8.21
1.00	7.00	7.00
1.25	7.63	6.10
1.50	8.12	5.41
1.75	8.54	4.88
2.00	8.96	4.48

LAGRANGIAN INTERPOLATING POLYNOMIAL

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X(1)= 0.25
Y(1)= 12.04

X(2)= 0.5
Y(2)= 9.8

X(3)= 0.75
Y(3)= 8.21

X(4)= 1
Y(4)= 7

X(5)= 1.25
Y(5)= 6.1

X(6)= 1.5
Y(6)= 5.41

X(7)= 1.75
Y(7)= 4.88

X(8)= 2
Y(8)= 4.48

X= 8.333333333E-02
F(X)= 14.49879449
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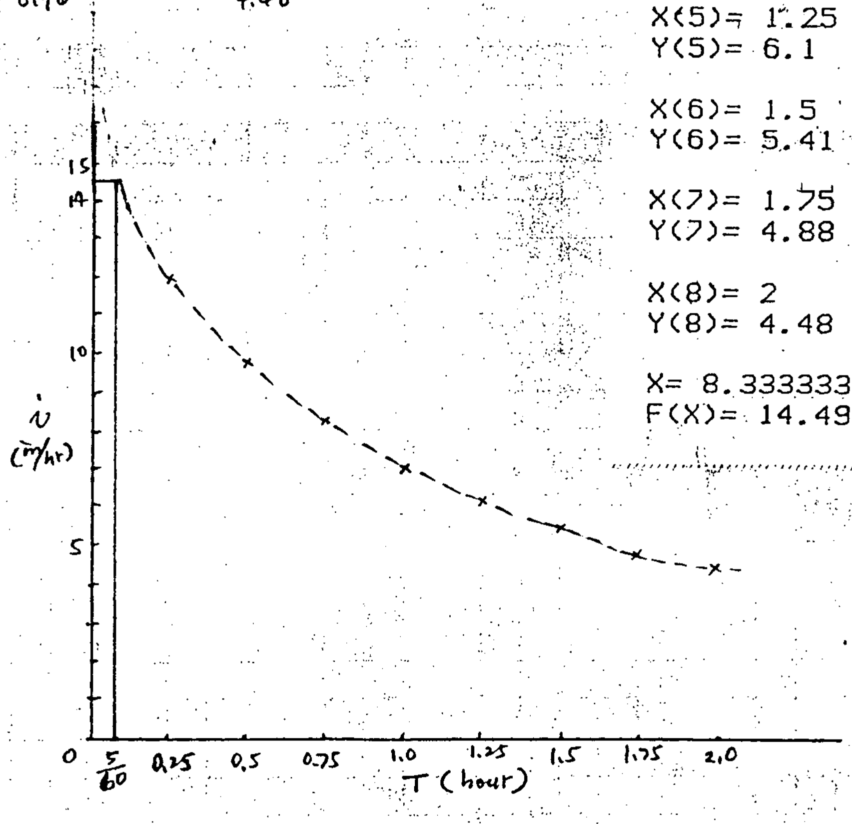


use 14.5 in/hr for 5 minutes of concentration for design.

(Ref. "Design of Roadside Drainage Channels" by U.S. Bureau of Public Works, sec. 2.4-2)

DWG. NO. 568145

ENGINEERING DEPARTMENT

CALCULATION SHEET

SUBJECT: Roof Ponding of FSB & VB — DESIGN CALCULATION NO. DC 99 – IV A

J.O. NO. 6733 MADE BY T. WANG DATE 3/30/83 CHK. BY [initials] DATE 4-1-83

[initials] (Δ1) 8-10-84

IV. Ponding on Ventilation Bldg. Roof

$Q = CIA$ Water accumulated for 5 minutes of time of concentration.

$= 1 \times 14.5 \times \frac{20 \times 42.67}{43,560} = 0.284$ cfs (C ≅ 1 for Roof)

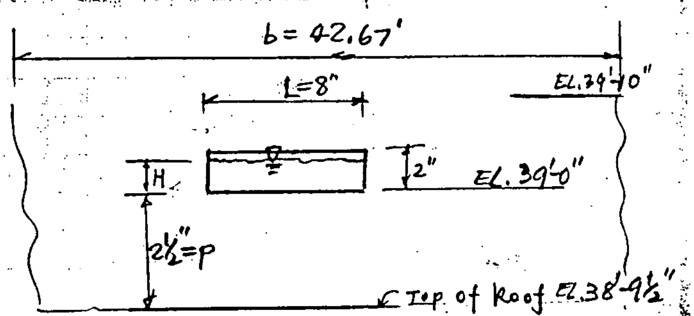
$Q = CLH^{3/2}$ weir discharge (2"×8"), (Ref. King's "Hand Book of Hydralics)

Assume H = 1"

$\frac{H}{P} = \frac{1}{2.5} = 0.40$

$\frac{L}{b} = \frac{8/12}{42.67} = 0.0156$

use C = 3.15 (Fig. 5.3, King's handbook)



$0.284 = 3.15 \times \frac{8}{12} \times H^{3/2}$

$H = \left(\frac{0.284}{3.15 \times \frac{8}{12}}\right)^{2/3} = 0.263' = 3.16"$

∵ H > 2" The water overtops the opening

∴ use Orifice formulas: (4-16, King's handbook)

Δ

Opening $D_{equiv.} = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4 \times 2 \times 8}{\pi}} = 4.5"$

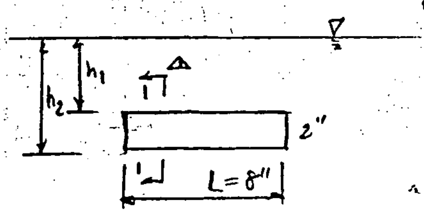
$\frac{B}{D_{equiv.}} = \frac{8}{4.5} = 1.78$

Per pg. 4-19 of King's Handbook, as a short tube. C = 0.82

the C value is close to the number given in Table 4-10 of King's Handbook for all corners square tubes.

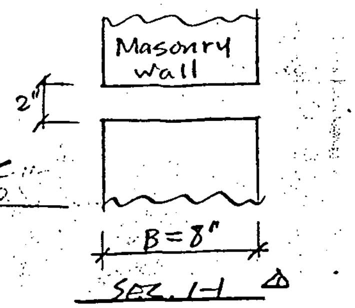
$Q_t = C^{\Delta}\frac{2}{3} L \sqrt{2g} \left(h_2^{3/2} - h_1^{3/2}\right)$

$h_2^{3/2} - h_1^{3/2} = \frac{Q_t}{C^{\Delta}\frac{2}{3} L \sqrt{2g}}$



$h_2^{3/2} - \left(h_2 - \frac{2}{12}\right)^{3/2} = \frac{0.284}{(0.82)^{\Delta}\frac{2}{3} \times \frac{8}{12}\sqrt{2 \times 32.2}} = \cancel{0.080}\ 0.097$

$h \approx \cancel{0.19}\ 0.24' = \cancel{2.28}\ 2.88"$



∴ The ponding to be $H_p = 2.5 + \cancel{2.28}\ 2.88 = \cancel{4.78}\ 5.38"$ ~~Say 5"~~

(The parapet is 12½" from Roof)

Δ REV. 1 CHANGES

DWG. NO. 567676 567677

ENGINEERING DEPARTMENT

CALCULATION SHEET

SUBJECT: Roof Ponding of FSB & VB — DESIGN CALCULATION NO. DC 99 - IV A

J.O. NO. 6733 — MADE BY T. Wang — DATE 3/30/83 — CHK. BY [signature] — DATE 4-1-83

[initials] Rev. 1 8-10-84

V. Ponding on Fuel Storage Building Roof

Area "A".

$Q = CIA$ water accumulated $P = 3.5''$

$= 1 \times 14.5 \times \frac{1698.6}{43,560} = 0.565$ cfs

$Q_t = C\frac{2}{3}\sqrt{2g}\,(h_2^{3/2} - h_1^{3/2})$.

$h_2^{3/2} - (h_2 - \frac{2}{12})^{3/2} = \frac{0.565}{(0.82)\frac{2}{3} \times \frac{8}{12}\sqrt{2 \times 32.2}} = \cancel{0.158''}\ 0.1932$

$h \approx \cancel{0.48'}\ 0.68' = \cancel{5.76''}\ 8.16''$

Ponding $H_p = \overset{h}{\cancel{5.76}}\ 8.16 + \overset{P}{3.5} = \cancel{9.26''}\ 11.66'' > 6''$ (The High Ridge Point of the Roof)

The water will flow over to the other side of roof "B"

Area "B".

$Q = 1 \times 14.5 \times \frac{1005.6}{43,560} = 0.335$ cfs

$h_2^{3/2} - (h_2 - \frac{2}{12})^{3/2} = \frac{0.335}{(0.82)\frac{2}{3} \times \frac{8}{12}\sqrt{2 \times 32.2}} = \cancel{0.0940}\ 0.1145$

$h \approx \cancel{0.23'}\ 0.296 = \cancel{2.76''}\ 3.55$

Ponding $H_p = \cancel{2.76}\ 3.55 + 3.5 = \cancel{6.46''}\ 7.05 > 6''$ (The High Ridge point of the Roof)

The water will also flow over to the other side of roof "A"

Take "A" & "B" as one Area, $A = 1698.6 + 1005.6 = 2704.2^{\square'}$

$Q = 1 \times 14.5 \times \frac{2704.2}{43,560} = 0.90$ cfs

$h_2^{3/2} - (h_2 - \frac{2}{12})^{3/2} = \frac{0.900}{(0.82)\frac{2}{3} \times \frac{2 \times 8}{12}\sqrt{2 \times 32.2}} = \cancel{0.126}\ 0.15386$

$h \approx \cancel{0.345'}\ 0.464' = \cancel{4.14''}\ 5.57''$

Ponding $H_p = \cancel{4.14}\ 5.57 + 3.5 = \cancel{7.64}\ 9.07''$

DWG. NO. 567676

CALCULATION TITLE PAGE

PROJECT SONGS 1 PMF JOB ORDER NO. 6733 DISCIPLINE CIVIL

SUBJECT: ROOF PONDING OF FUEL STORAGE BUILDING AND VENTILATION BUILDING (STRUCTURAL)

CALCULATION NO. DC 99 PART IV B NO. PAGES 26

RESPONSIBLE ENGINEER (signature) Jiz Way DATE 9/18/84

INDEPENDENT REVIEW ENG. (signature) MPA/Y Shiau 11/25/84 DATE 12/3/84

ORIGINAL ISSUE

	NAME	DATE	SIGNATURE
GROUP LEADER	C. M. Knarr	12-5-84	C M Knarr
DISCIPLINE SUP. ENGR.	R ~~D. H. Johns~~ Raymond L Baker	12-6-84	Raymond L Baker
OTHER			

RECORD OF REVISIONS

NO.	REASON FOR REVISION	DATE	RESP. ENGR.	IRE	GL	DSE	OTHER

ENGINEERING DEPARTMENT

CALCULATION SHEET

SUBJECT: Roof Ponding of Fuel Storage Bldg & Ventilation Bldg (STRUCTURES) DESIGN CALCULATION NO. DC 99 - IV B

J.O. NO. 6733 MADE BY T. Wang DATE 9/14/84 CHK. BY [signature] DATE 8/8/84

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DWG. NO.

DESIGN INPUT SHEET

[X] Design Requirements
[ ] Design Input Change

DESIGN CALCULATION NO. DC 99 — IV B

SUBJECT		PROJECT
ROOF PONDING OF FUEL STORAGE BUILDING AND VENTILATION BUILDING (STRUCTURAL)		SONGS 1 PMF
QUALITY CLASS	SEISMIC CLASS	SPECIFICATION REFERENCE
SAFETY RELATED	A	

Design Input

Purpose: Evaluate the roof frame integrity of Ventilation Equipment Building and Fuel Storage Building for PMP rooftop ponding in case the roofdrains are clogged.

References:
1. SCE DWG's 568145, 567676, 567677, 567678, 567892.
2. AISC, "Steel Construction Manual", 6th & 8th Edition.
3. "Steel Deck Institute Design Manual", By SDI.
4. "Floor Systems" Catalog NO. 270, 1963 by INLAND STEEL PRODUCTS CO.
5. "Simplified Steel Design", by AISC, 1975.
6. Bechtel Calculation Book 5, DC 26, SONGS 1
7. Calculation DC 99 Part IV A for PMP calculation.

Design Assumptions:
1. Concrete : $f'_c = 2500$ psi, $f_c = 1125$ psi
2. Structural Steel : $f_y = 36{,}000$ psi (A36)
   Bolt Connections, A325 bolts Friction Type
3. Metal Deck : INLAND B F-18-16 GALV.
   ASTM A446, $f_y = 33{,}000$ psi ; $f_s = 20{,}000$ psi
4. Concrete Block Walls: Hollow Grade A units
   $f'_m = 1350$ psi, $f_m = 450$ psi, $v = 60$ psi (shear wall)
   $f_s = 20{,}000$ psi, $E_s = 30{,}000{,}000$ psi
   $E_m = 1{,}350{,}000$ psi, $n = 22$
5. No seismic, wind, or normal L.L is considered with the PMP condition.

RESPONSIBLE ENGINEER	DATE	INDEPENDENT REVIEW ENGINEER	DATE	GROUP LEADER	DATE
[signature]	7/9/84	Norman W. [signature]	8/8/84	[signature]	12/5/84
		[signature]	11/26/84		

ENGINEERING DEPARTMENT

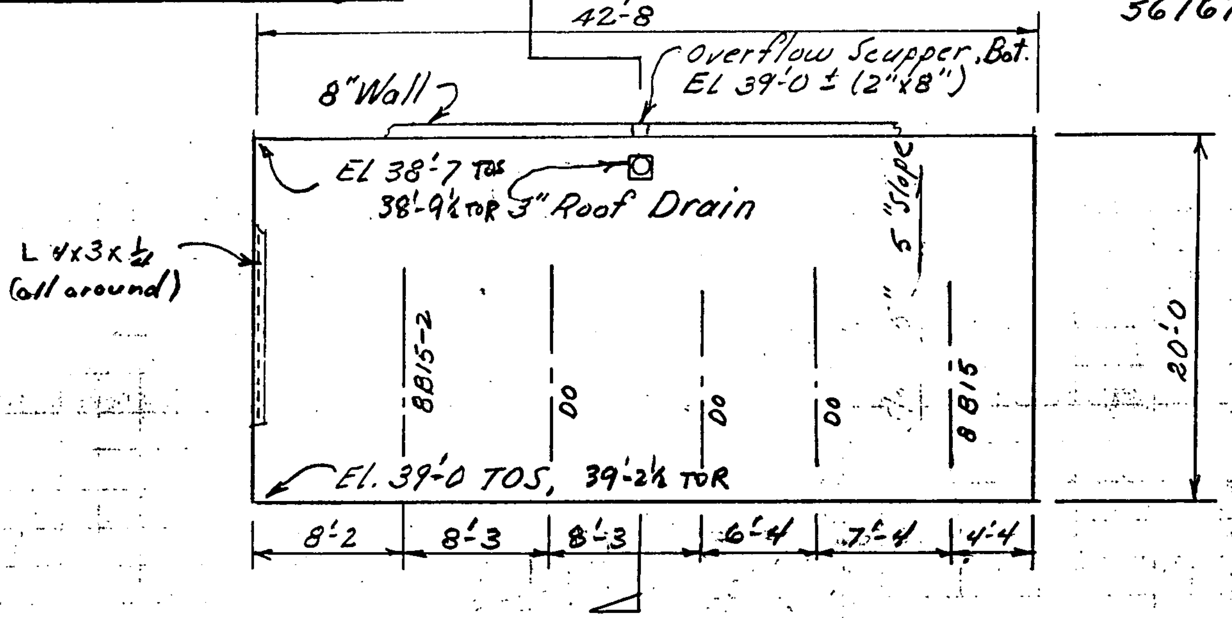
CALCULATION SHEET

SUBJECT: Roof Ponding of Fuel Storage Bldg & Ventilation Bldg (STR) DESIGN CALCULATION NO. DC 99 - IV B

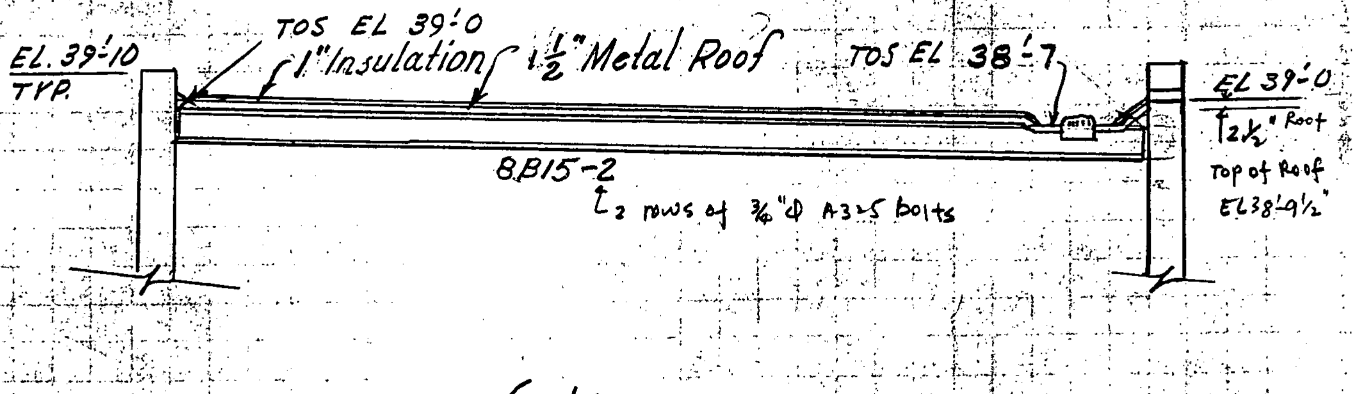
J.O. NO. 6733 MADE BY Dotson/T. Wang DATE 3/30/83 CHK. BY [signature] DATE 5-4-83

I. Ventilation Bldg. (SKETCH)

Dwg 568145, 567676



Roof area = 20 x 42.67 = 853.4 sq. ft.



Section A

DWG. NO. 568145, 567676

ENGINEERING DEPARTMENT

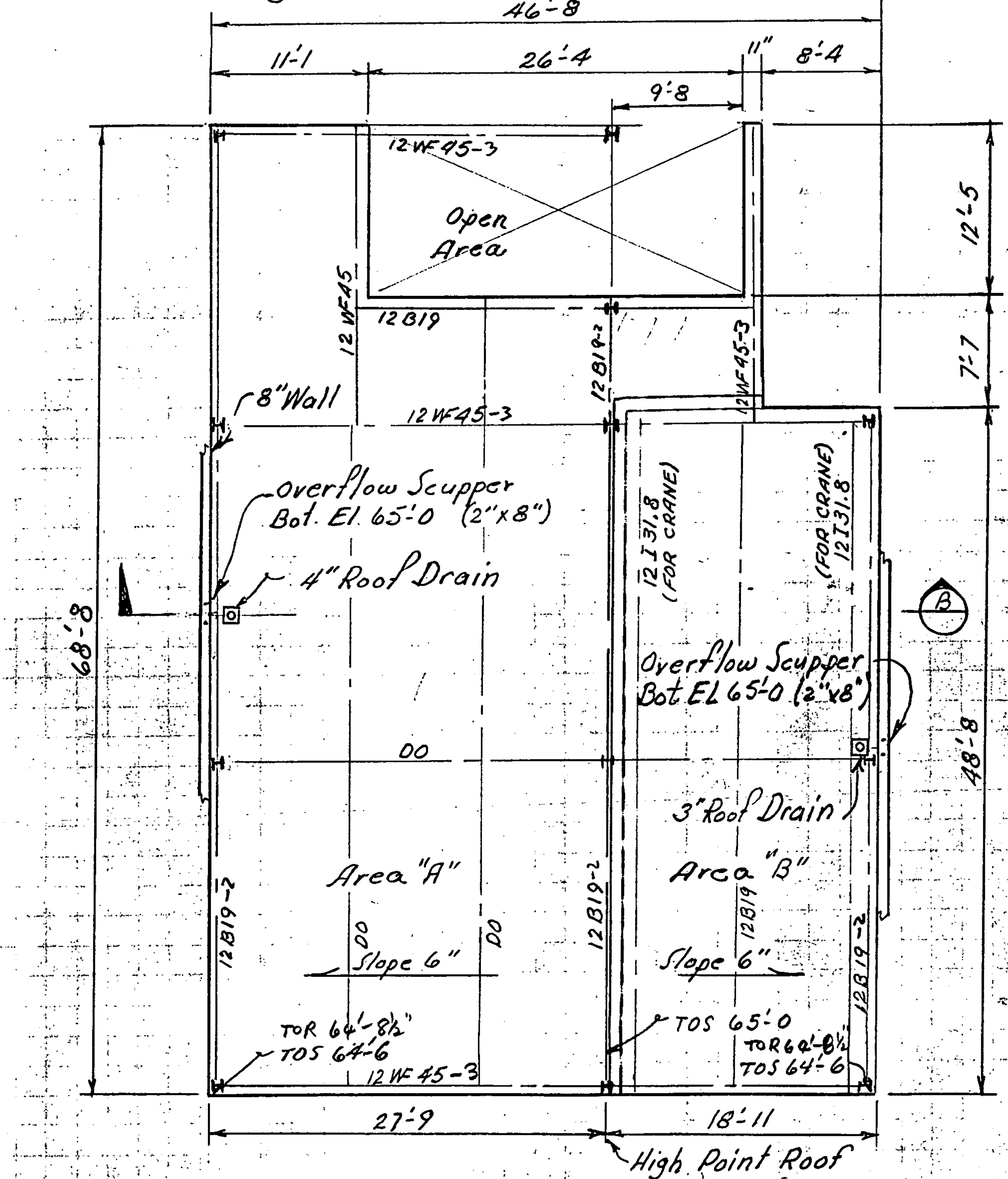
CALCULATION SHEET

SUBJECT: Roof Ponding of Fuel Storage Bldg & Ventilation Bldg (STR) DESIGN CALCULATION NO. DC 99 - IV B

J.O. NO. 6733 MADE BY T. Wang / G. Dotson DATE 3/30/83 CHK. BY [signature] DATE 4-1-83

II. Fuel Storage Bldg. (SKETCH)

Dwg. 568145, 567676



Area = 27.75 x 56.25 + 12.42 x 11.08 = 1698.6 □' Area "A"

18.92 x 48.67 + 7.58 x 9.67 + 12.42 x .92 = 1005.6 Sq. ft. Area "B"

Total = 2704.2 Sq. ft.

3/32"

DWG. NO. 568145 567676

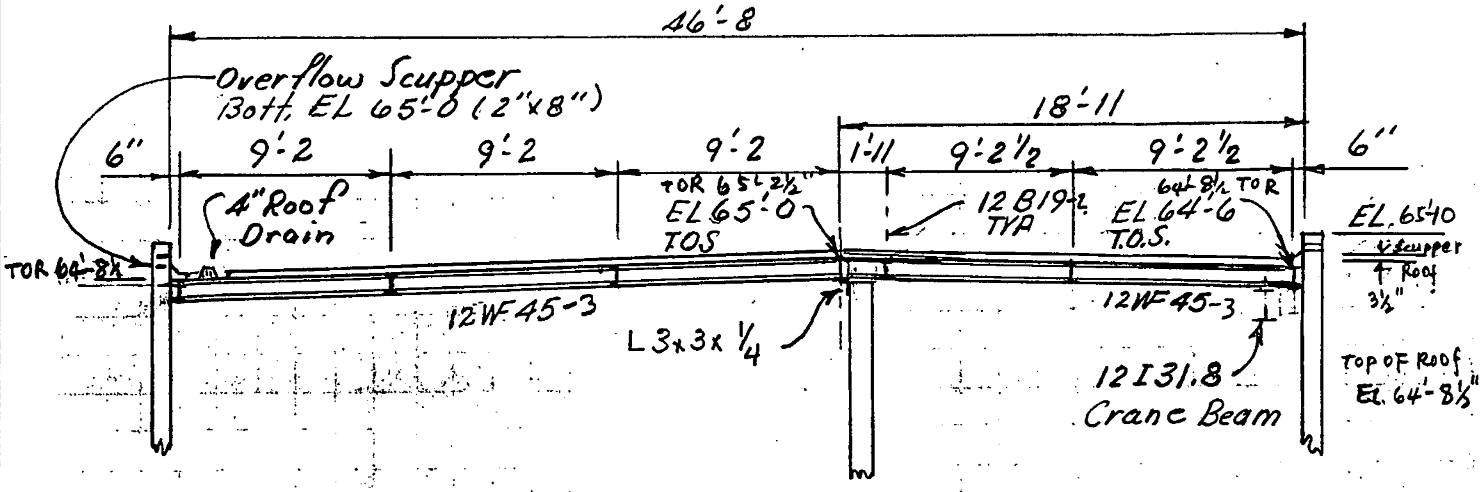
ENGINEERING DEPARTMENT

CALCULATION SHEET

SUBJECT: Roof Ponding of F.S.B & V.B. (STRUCTURAL) DESIGN CALCULATION NO. DC 99 - IV B

J.O. NO. 6733 MADE BY T. Wang/R. Doten DATE 3/30/83 CHK. BY [illegible] DATE 4-1-83

II. Fuel Storage Bldg. (SKETCH)



Section (B)

DWG. NO. 568145
567676

ENGINEERING DEPARTMENT
CALCULATION SHEET

SUBJECT: Roof Ponding of FSB & VB (STRUCTURAL) DESIGN CALCULATION NO. DC 99 - IV B

J.O. NO. 6733 MADE BY T. Wang DATE 9/5/84 CHK. BY [illegible] DATE 12-3-84

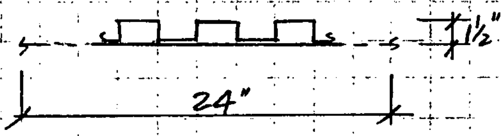
III. CHECK PONDING IN ACCORDANCE WITH AISC SPEC. SEC. 1.13.3

Metal Deck: INLAND BF-18-16 GALV. (SPEC. #BSO-25B)

$I = 0.61^{in^4/'}$, $S = 0.49^{in^3/'}$ (INLAND Floor System Catalog #270, 1963)

$f_y = 33{,}000^{psi}$ (ASTM A 446), $f_s = 20{,}000$ psi

$w = 5.5$ #/□'



Metal Deck

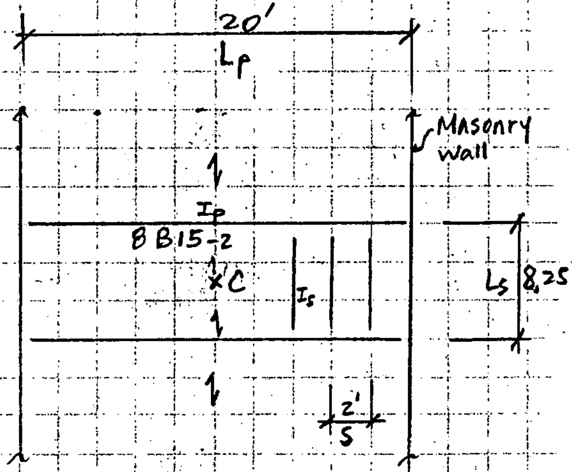
A. VENTILATION BLDG

Beam 8B15,

$A = 4.43^{in^2}$, $d = 8.12"$, $b_f = 4"$

$S_x = 11.8^{in^3}$, $I_x = 48^{in^4}$

Per AISC SPEC. SEC. 1.13.3 commentary, the stability against ponding of a roof consisting of a metal roof deck of relatively slender depth-span ratio may be analyzed by Fig. C1.13.3.1 or C.13.3.2 and constants U_p, U_s, C_p and C_s as specified in the spec. The metal deck may be treated as a secondary member.



Ventilation Bldg Framing

SCE OD 397-B NEW 8/77 (CW)

ENGINEERING DEPARTMENT

CALCULATION SHEET

SUBJECT: Roof ponding of FSB & VB (STRUCTURAL) DESIGN CALCULATION NO. DC 99 - IV B

J.O. NO. 6733 MADE BY T. Wang DATE 9/7/84 CHK. BY [illegible] DATE 12-3-84

III. (Con't.)

A. VENTILATION BLDG (CONT.)

for metal Deck, (as secondary member)

$$M_0 = \frac{13.5(8.25)^2}{8} = 114.9\ ^{1\#/ft\ width}$$ (without ponding) (see pg. 9, $w_D = 13.5\ ^{\#/0'}$)

$$f_0 = \frac{M_0}{S} = \frac{114.9 \times 12}{0.49} = 2{,}813\ ^{psi}$$

$$U_s = \left(\frac{0.80 F_y - f_0}{f_0}\right)_s = \frac{0.80 \times 33{,}000 - 2813}{2813} = 8.39$$

$$C_s = \frac{32\ S L_s^4}{10^7 I_s} = \frac{32(2)(8.25)^4}{10^7(0.61)} = 0.0486$$

for Beam 8B15, (as primary member)

$$M_0 = \frac{126(20)^2}{8} = 6{,}300\ ^{1\#'}$$ (w/o ponding) (see pg. 9, $w_{Dp} = 126\ ^{\#/1}$)

$$f_0 = \frac{M_0}{S} = \frac{6{,}300 \times 12}{11.8} = 6{,}407\ ^{psi}$$

$$U_p = \left(\frac{0.80 F_y - f_0}{f_0}\right)_p = \frac{0.80 \times 36{,}000 - 6407}{6407} = 3.50$$

$$C_p = \frac{32\ L_s L_p^4}{10^7 I_p} = \frac{32(8.25)(20)^4}{10^7(48)} = 0.088$$

From Fig. C1.13.3.1, for $U_p = 3.5$, $C_s = 0.0486$

$$C_p \approx 0.8$$

$$\gg 0.088 = C_p \text{ calculated}$$

∴ Roof ponding is considered ok according to AISC Spec. However, since this is an extreme load case, roof member stresses will be checked. See pgs. 9-10 & 13-15.

DWG. NO. 568145
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ENGINEERING DEPARTMENT

CALCULATION SHEET

SUBJECT: Roof Ponding of FSB & VB (STRUCTURAL) DESIGN CALCULATION NO. DC 99 - IV B

J.O. NO. 6733 MADE BY T. Wang DATE 9/12/84 CHK. BY [illegible] DATE 12-3-84

III. (CONT.)

B. FUEL STORAGE BUILDING

FOR AREA "A" — 12WF45 IS PRIMARY MEMBER, 12B19 IS SECONDARY MEMBER

$S = 9'\text{-}2''$

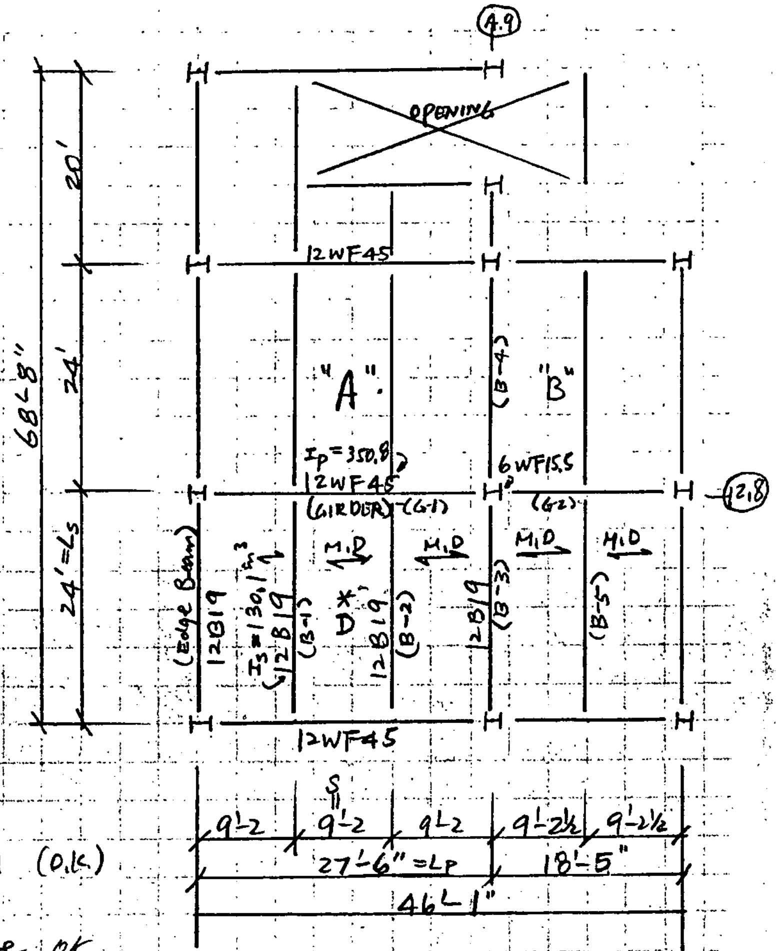
$L_s = 24'$, $L_p = 27'\text{-}6''$

$I_s = 130.1\ \text{in}^4$, $I_p = 350.8\ \text{in}^4$

$$C_p = \frac{32 L_s L_p^4}{10^7 I_p} = \frac{32(24)(27.5)^4}{10^7(350.8)} = 0.125$$

$$C_s = \frac{32 S L_s^4}{10^7 I_s} = \frac{32(9.167)(24)^4}{10^7(130.1)} = 0.075$$

$$C_p + 0.9\,C_s = 0.125 + 0.9 \times 0.075 = 0.200 < 0.25\ (\text{O.K.})$$



for metal Deck,

$$\frac{25 S^4}{10^6} = \frac{25(9.167)^4}{10^6} = 0.177\ \text{in}^4 < 0.61\ \text{in}^4 = I_d\ (\text{O.K.})$$

∴ Roof ponding is considered to be ok according to AISC Spec. However, since this is an extreme load case, roof member stresses will be checked. See pgs. 11-12 & 16-22.

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ENGINEERING DEPARTMENT

CALCULATION SHEET

SUBJECT: Roof Ponding of FSB & VB (Structural) DESIGN CALCULATION NO. DC 99 – IV B

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IV. DESIGN LOADS

Design loads are calculated for accumulation of ponding at low point.

D. L. :

$$w_D = 6.0 + 5.5 + 1.0 + 1.0 = 13.5\ ^{\#/\square'}$$

(Roofing) (Deck) (Insulation) (mechanical)

L.L. : Ponding only (From Ref. 7)

A. VENTILATION BLDG :

preliminary, $H_{PL} = 5.38''$, $w_{Lp} = 62.4 \times \frac{5.38}{12} = 28\ ^{\#/\square'}$ at Low point

$H_{PH} = 5.38 - 5'' = 0.38''$, $w_{Lp} = 62.4 \times \frac{0.38}{12} = 2\ ^{\#/\square'}$ at High point

Deflection accumulation : (At point C, see pg. 6)

Metal Deck, $\Delta_{Lp} = \frac{5}{384} \frac{\left(\frac{0.028+0.002}{2}\right)(8.25)^4(1728)}{29{,}000 \times 0.61} = 0.09''$ (for more than 3 spans)

$\Delta_{Dp} = \frac{5}{384} \frac{0.0135(8.25)^4(1728)}{29{,}000 \times 0.61} = 0.08''$

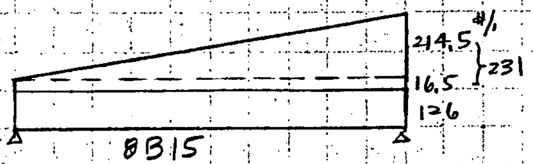
$\Delta_{P_{MD}} = 0.17''$

Beam 8B15,

$w_{dp} = 13.5 \times 8.25 + 15 = 126\ ^{\#/'}$

$w_{Lp} = 28 \times 8.25 = 231\ ^{\#/'}$ (Low point)

$= 2 \times 8.25 = 16.5$ (High point)



$l = 20'$, $I = 48\ in^4$

$W_{Lp} = \frac{(231 + 16.5)(20)}{2} = 2{,}475\ ^{\#}$

$\Delta_{Lp\,max} \cong 0.01304 \frac{WL^3}{EI} = 0.01304 \frac{2475(20 \times 12)^3}{29{,}000{,}000 \times 48} = 0.32''$, (at $X = 0.5193L$ per AISC)

$\Delta_{D\,max} = \frac{5}{384} \frac{0.126(20)^4(1728)}{29{,}000 \times 48} = 0.33''$

$\Delta_{P_B} = 0.65''$

$\Delta_{P_{max}} = 0.17 + 0.65 = 0.82''$

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CALCULATION SHEET

SUBJECT: Roof Ponding of FSB & VB (Structural) DESIGN CALCULATION NO. DC 99 - IV B

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IV. (cont.)

A. (cont.)

At point c,

$$H_p = \frac{5.38+0.38}{2} = 2.88'' \quad \text{Before deflection}$$

$$= 2.88 + 0.82 = 3.70 \quad \text{After deflection}$$

Live load increased by

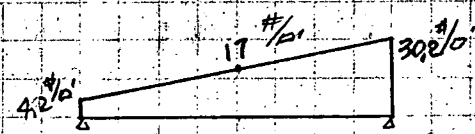
$$W_{La} = 62.4 \times \frac{0.82}{12} = 4.24 \ ^{\#}/_{\text{o}'} \quad \text{at point c,}$$

Take $\frac{1}{2} W_{La} = \frac{4.24}{2} = 2.12 \ ^{\#}/_{\text{o}'}$ say $2.2 \ ^{\#}/_{\text{o}'}$ increase for design, i.e.

DESIGN LOADS (Assume no further secondary deflection occurs)

$$W_L = 28 + 2.2 = \underline{30.2} \ ^{\#}/_{\text{o}'} \quad \text{at Low point}$$

$$= 2 + 2.2 = \underline{4.2} \ ^{\#}/_{\text{o}'} \quad \text{at High point}$$



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CALCULATION SHEET

SUBJECT: Roof Ponding of FSB & VB (Structural) DESIGN CALCULATION NO. DC 99 - IV B

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IV. (cont.)

B. Fuel Storage Bldg

preliminary, $H_{PL} = 9.07''$, $w_{LP} = 62.4 \times \frac{9.07}{12} = 47.2$ #/□'

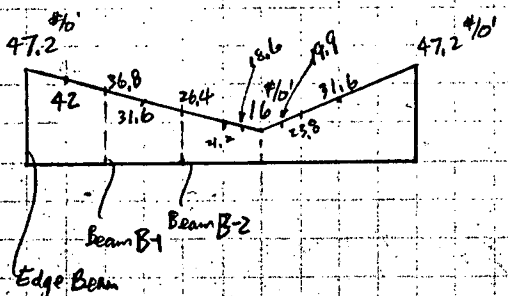
$H_{PR} = 9.07 - 6 = 3.07$, $w_{LP} = 62.4 \times \frac{3.07}{12} = 16.0$ #/□'

Deflection Accumulation : (At point D, see pg. 8)

Metal Deck,

$$\Delta_{P_M} = \frac{5}{384}\frac{wl^4}{EI} = \frac{5}{384} \times \frac{(0.0135+0.0316)(9.167)^4(1728)}{29{,}000\ (0.61)}$$

$$= 0.4''$$



Beam B-1 — 12 B 19, $I_x = 130.1$ in⁴, $l = 24'$

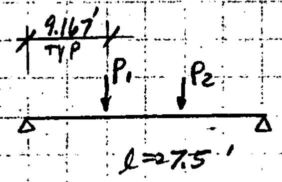
$w_d = 13.5 \times 9.167 + 19 = 143$ #/'

$w_L = 36.8 \times 9.167 = 339$

$w = 482$ #/'

$$\Delta_{P_{B1}} = \frac{5}{384} \times \frac{0.482\,(24)^4(1728)}{29{,}000\ (130.1)} = 0.95''$$

Girder — 12 WF 45, $I_x = 350.8$ in⁴



From B-1, $P_1 = 482 \times \frac{24}{2} = 5784$

$\times 2 = 11{,}568$ #
(2 beams)

From B-2, $w = 26.4 \times 9.2 + 143 = 386$ #/'

$P_2 = 386 \times \frac{24}{2} \times 2 = 9264$ #

$$\Delta_G \cong \frac{\left(\frac{11.568+9.264}{2}\right)}{24} \times \frac{(9.167\times 12)(3\times 27.5^2 - 4\times 9.167^2)(144)}{29{,}000\ (350.8)} + \frac{5}{384}\frac{0.045(27.5)^4(1728)}{29{,}000\ (350.8)}$$

$$= 1.31 + 0.06 = 1.37''$$

$$\Delta_{P_{max}} = 0.4 + 0.95 + 1.37 = \underline{2.72''}$$

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IV. (cont.)

B. (cont.)

At point D,

$$H_p = \frac{9.07+3.07}{2} = 6.07'' \text{ Before deflection}$$

$$= 6.07 + 2.72 \doteq 8.79 \quad \text{After deflection}$$

Live Load increased by

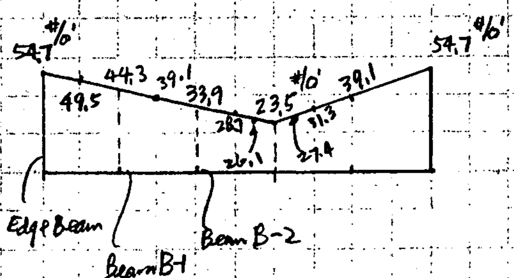
$$w_{La} = 62.4 \times \frac{2.72}{12} = 14.14 \ \#/\square' \text{ at point D,}$$

Take $\frac{1}{2} w_{La} = \frac{14.14}{2} = 7.1 \ \#/\square'$ say $7.5 \ \#/\square'$ increase for design, i.e.

<u>Design Loads</u>: (Assume no further secondary deflection occurs)

$$w_{LL} = 47.2 + 7.5 = \underline{54.7} \ \#/\square' \text{ at Low point}$$

$$w_{LH} = 16.0 + 7.5 = \underline{23.5} \ \#/\square' \text{ at High point}$$



SCE IOD 397-B NEW 8/77 (CW)

ENGINEERING DEPARTMENT
CALCULATION SHEET

SUBJECT: Roof Ponding of FSB & VB (STRUCTURAL) DESIGN CALCULATION NO. DC 99 - IV B

J.O. NO. 6733 MADE BY T. Wang DATE 9/12/84 CHK. BY YS DATE 12-3-84

V. CHECK ROOF DECK AND FRAMING (DL + Ponding Load)*

A. VENTILATION BLDG. (see pg. 6)

(1) ROOF DECK

CHECK TOTAL LOADS:

$$w_{Total} = 13.5 + 30.2 = 43.7 \ ^{\#/\square'}$$

$$M_{\mathcal{C}} = \frac{43.7\,(8.25)^2}{8} = 372 \ ^{'\#/1}$$

$$f_b = \frac{372 \times 12}{0.49} = 9{,}110 \ ^{psi} < 20{,}000 \text{ psi (o.k.)}$$

(2) CHECK BEAM 8B15-2, (8B15-2 = 8B15 Beam with 2 rows of $\frac{3}{4}\phi$ A325 Friction bolts, per Dwg. 567892-0)

For 8B15,

$A = 4.43^{\square''}$, $d = 8.12''$

$S_x = 11.8 \ in^3$, $I_x = 48 \ in^4$

$A_{shear} = 8.12 \times 0.245 = 2^{\square''}$

Loads:

$$w_d = 13.5 \times 8.25 + 15 = 126 \ ^{\#/1}$$

$$w_L = 30.2 \times 8.25 = 249 \ ^{\#/1} \text{ (Low point)}$$

$$= 4.2 \times 8.25 = 35 \ ^{\#/1} \text{ (High point)}$$

* Based on AISC spec. 1.13.3, stresses due to wind or seismic forces need not be included in a ponding analysis.

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V. (Cont.)

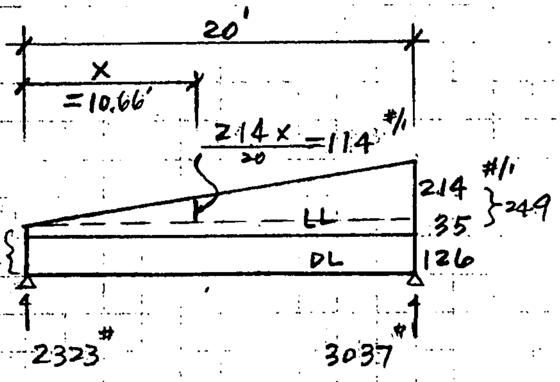
A. (Cont.)

(2) CHECK Beam 8B15-2 (Cont.)

$$R_L = \frac{126\times20}{2} + \frac{214\times20}{3\times2} + \frac{35\times20}{2} = 2323^{\#}$$

$$R_R = \frac{126\times20}{2} + \frac{214\times20\times2}{3\times2} + \frac{35\times20}{2} = 3037$$

$$W_T = 5360^{\#}$$



Bending:

Location of Max. Moment at 0 shear,

$$2323 - (126+35)x - \frac{214\,x}{20}\left(\frac{x}{2}\right) = 0,$$

$$5.35x^2 + 161x - 2323 = 0$$

$$x = \frac{-161 \pm \sqrt{(161)^2 - 4(5.35)(-2323)}}{2\times5.35} = 10.66$$

$$M_{max} = 2323(10.66) - (126+35)(10.66)\left(\frac{10.66}{2}\right) - \frac{114(10.66)}{2}\times\frac{10.66}{3} = 13456^{\prime\#}$$

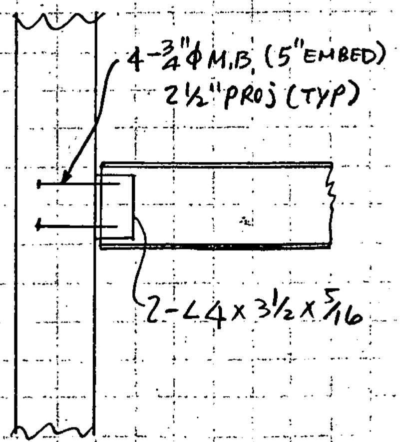
$$f_b = \frac{13{,}456\times12}{11.8} = 13{,}685^{psi} < 0.8\times36{,}000 = 28{,}800 \quad (O.K.)$$

(AISC SEC. 1.13.3)

SHEAR:

$$V_{max} = \frac{3037}{2} = 1518.5^{\#/\square''}$$

$$V_{allow} = 0.40\times36{,}000 = 14{,}400^{\#/\square''} > V_{max} \quad (O.K.)$$



CONNECTION TO WALLS:

$$V_{max} = 3{,}037^{\#}$$

$$V_{allow} = 1{,}100\times4 = 4{,}400^{\#}$$ (4-¾"φ A307 bolts 5 in. Embeded IN Masonry Wall per UBC Table 24-G)

$> V_{max}$ (O.K.)

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CE OD 397-B NEW 8/77 (CW)

ENGINEERING DEPARTMENT
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J.O. NO. 6733 MADE BY T. Wang DATE 9/14/84 CHK. BY YS DATE 12-3-84

V. (cont.)

A. (cont.)

(3) CHECK MASONRY WALLS

Loads :

$P_1' = 3{,}037^{\#}$ @ 6'4" → 8'-3"

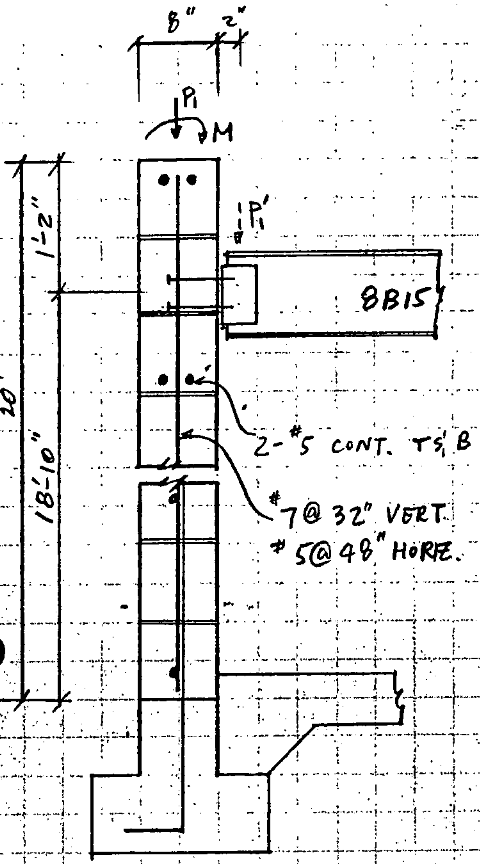
$M = 3{,}037(6) = 18{,}222''^{\#} / 6.33 = \underline{2879}''^{\#/'}$

Let 6.33' be the width of loaded section

$W_B = \frac{3037}{6.33} = 480^{\#/'}$ (Beam Loads)

$W_W = 75^{\#/\square'} \times 20 = \underline{1500}^{\#/'}$ (Wall D.L.)

$W = 1980^{\#/'}$ say $\underline{2000}^{\#/'}$



WALL SECTION

Masonry properties (BPC Cal's Book #5)

Hollow Grade A Units. (For Type of C.B)

$f_m' = 1350^{psi}$ (U.B.C. Table 24-H)

$f_m = 450$ psi max

$f_s = 20{,}000$ psi (Reinf.)

$E_s = 30{,}000{,}000^{psi}$

$E_m = 1{,}350{,}000^{psi}$

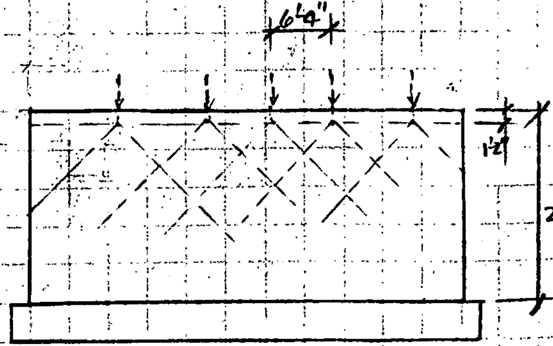
$n = 22$

$w = 75^{\#/\square'}$

$t = 7.625''$

$d = 3.75''$

$v_a = 25^{psi}$ (w/o inspection, Flexural)



BEAM LOAD DISTRIBUTION

DWG. NO.

ENGINEERING DEPARTMENT
CALCULATION SHEET

SUBJECT: Roof ponding of FSB & VB (structural) DESIGN CALCULATION NO. DC 99 - IV B

J.O. NO. 6733 MADE BY T. Wang DATE 9/17/84 CHK. BY J.S. DATE 12-3-84

V. (cont.)

A. (cont.)

(3) (cont.)

Axial stress:

$$f_a = \frac{W}{bt} = \frac{2000}{12(7.625)} = 22 \ \#/\square''$$

$$F_a = 0.2 f'_m \left[1 - \left(\frac{h}{40t}\right)^3\right] = 0.2(1350)\left[1 - \left(\frac{20 \times 12}{40 \times 7.625}\right)^3\right]$$

$$= 138 \text{ psi} > f_a$$

Bending stress: Reinf. vertical #7 @ 32" O.C.

$$p = \frac{A_s}{bd} = \frac{0.60}{32(3.75)} = 0.005$$

$$np = 22(0.005) = 0.11$$

From Table NO. 18-B, "Plan Review Manual" by ICBO, 1977

$k = 0.3718$,

$j = 0.8761$

$$\frac{2}{kj} = 6.14$$

$$f_s = \frac{M}{A_s jd} = \frac{2879}{\frac{0.60}{\frac{32}{12}}(0.8761)(3.75)} = 3895 \text{ psi}$$

$< 20{,}000 \text{ psi} = F_a$

$$f_b = \frac{M}{bd^2}\left(\frac{2}{kj}\right) = \frac{2879}{12(3.75)^2}(6.14) = 105 \text{ psi}$$

$$F_b = 0.166 f'_m = 0.166(1350) = 225 \text{ psi}$$

Combined stress: $\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1.0$

$$\frac{22}{138} + \frac{105}{225} = 0.16 + 0.47 = 0.63 < 1.0 \ (\text{O.K.})$$

DWG. NO.

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ENGINEERING DEPARTMENT

CALCULATION SHEET

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V. (cont.)

A. (cont.)

(4) CHECK DUCT OPENING REINFORCING

LOADS AT TOP OF OPENING:

From P,

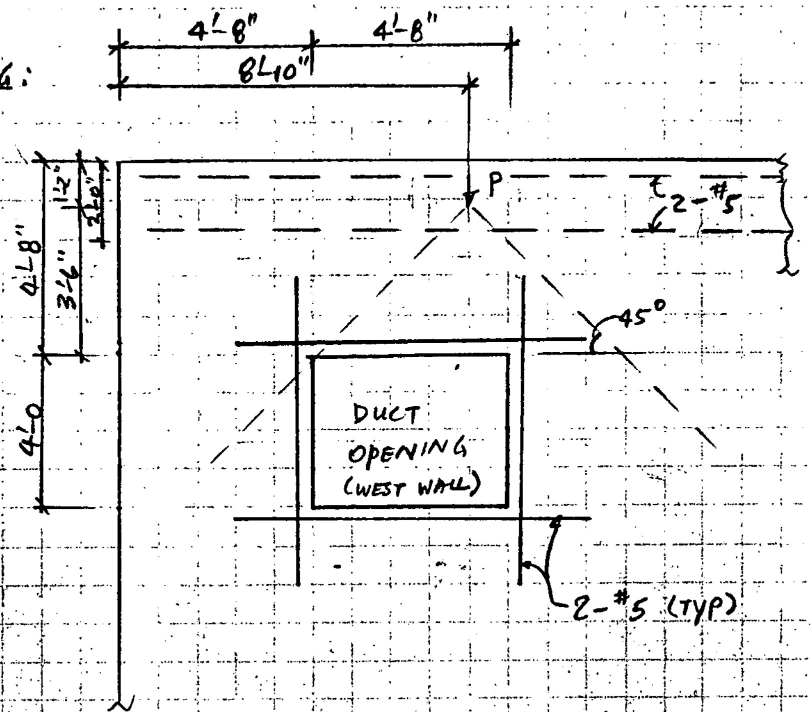
$$w_p = \frac{3037}{3.5 \times 2} = 434^{\#/'}$$

From wall,

$$W_w = 75 \times 4.75 = 356$$

$$w = 434 + 356 = 790^{\#/'}$$

Say $w = 800^{\#/'}$



Check stress:

$V = \frac{1}{2}(800)(4.75) = 1900^{\#}$, Let $d = 3'6"$ (under P) $= 42"$, $b = 7.625"$

$$v = \frac{V}{jdb} = \frac{1900}{0.8761(42)(7.625)} = 6.77^{psi}$$

$< v_a = 25$ (o.k.)

$$M = \frac{800(4.75)^2}{10} = 1805^{'\#} = 1.805^{'k}$$

$$F = \frac{bd^2}{12000} = \frac{7.625(42)^2}{12000} = 1.12,$$

$$K = \frac{M}{F} = \frac{1.805}{1.12} = 1.61$$

From Table E-4, "Reinforced Masonry Engg. Handbook" by J.E. Amrhein

$p < 0.0001$, $A_{s_{req.}} = 0.0001(7.625)(42) = 0.032\square"$

WITH 2-#5, $A_s = 0.62\square" \gg A_{s_{req.}}$ (o.k.)

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ENGINEERING DEPARTMENT
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V. (Cont.)

B. FUEL STORAGE BLDG (SEE Pg. 8)

(1) ROOF DECK

$w_{total} = 13.5 + 49.5 = 63^{\#/\square'}$

$M_{\text{¢}} = \frac{63(9.167)^2}{8} = 662^{'\#}$

$f_b = \frac{662 \times 12}{0.49} = 16,207^{psi} < 20,000 \text{ psi}$ (O.K.)

(2) CHECK BEAM B-1 - 12 B 19-2

$A = 5.62^{in^2}$, $d = 12.16''$

$S_x = 21.4^{in^3}$, $I_x = 130.1^{in^4}$

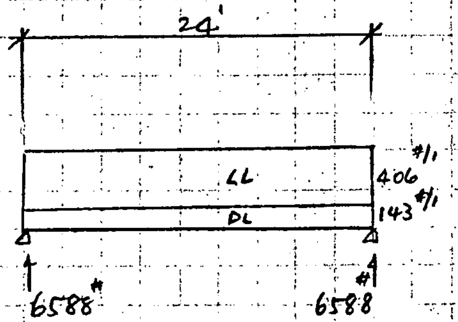
$t_w = 0.240''$

Loads:

$w_d = 13.5 \times 9.167 + 19 = 143^{\#/'}$

$w_L = 44.3 \times 9.167 = \underline{406}$

$w_T = 549^{\#/'}$



$R_L = R_R = 549 \times \frac{24}{2} = 6588^{\#}$

Live Loads only,

$R_{LL} = R_{RL} = 406 \times \frac{24}{2} = 4872^{\#}$

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V. (cont.)

B. (cont.)

(2) (cont.)

<u>Bending</u>:

$$M_{s} = \frac{549(24)^2}{8} = 39{,}528'^{\#}$$

$$f_b = \frac{39{,}528(12)}{21.4} = 22{,}165 < 0.8 \times 36{,}000 = 28{,}800 \text{ psi (O.K.)}$$

<u>Connection to Girder</u>:

2 rows of $\frac{3}{4}''\phi$ A325 bolts, friction Type

$V_{all.} = 26.5^k > 6.588$ (o.k.) (AISC 6th Ed., Connection Table I)

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V. (Cont.)

B. (Cont.)

(3) Reactions on Beam B-2

$w_L = 33.9 \times 9.167 = 311$ #/'

$w = 311 + \underset{(w_D)}{143} \doteq 454$ #/'

$R_L = R_R = 454 \times \frac{24}{2} = 5448$ #

Live load only,

$R_{L_L} = R_{R_L} = 311 \times \frac{24}{2} = 3732$ #

ENGINEERING DEPARTMENT
CALCULATION SHEET

SUBJECT: Roof Ponding of FSB & VB — DESIGN CALCULATION NO. DC 99 – IV B

J.O. NO. 6733 MADE BY T. Wang DATE 9/13/84 CHK. BY [illegible] DATE 12-3-84

V. (cont.)

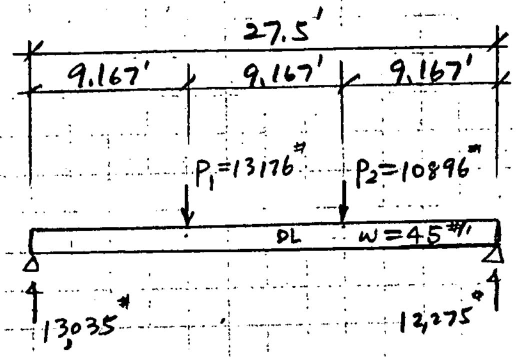
B. (cont.)

(4) CHECK GIRDER: 12WF45-3 (G-1)

$A = 13.24 \square''$, $d = 12.06''$

$S_x = 58.2 \text{ in}^3$, $I_x = 350.8 \text{ in}^4$

$t_w = 0.336''$



Loads:

$w = 45^{\#/'}$

$W = 45 \times 27.5 = 1238^{\#}$

$P_1 = 6588 \times 2 = 13,176^{\#}$

$P_2 = 5448 \times 2 = 10,896^{\#}$

$P_{1L} = 4872 \times 2 = 9,742^{\#}$ } Live load Only

$P_{2L} = 3732 \times 2 = 7,464^{\#}$

$$R_L = \frac{1238}{2} + \frac{13176 \times 2}{3} + \frac{10896}{3} = 13,035^{\#}$$

$$R_R = \frac{1238}{2} + \frac{13176}{3} + \frac{10896 \times 2}{3} = 12,275^{\#}$$

$$W = 25,310^{\#}$$

DWG. NO. 568145
567676

ENGINEERING DEPARTMENT
CALCULATION SHEET

SUBJECT: Roof Ponding of FSB & VB — DESIGN CALCULATION NO. DC 99 - IV B

J.O. NO. 6733 MADE BY T. Wang DATE 9/13/84 CHK. BY [illegible] DATE 12-3-84

IV. (cont.), B. (cont.)

(4) (cont.)

Bending

$$M_{max} = 13035(9.167) - 45\frac{(9.167)^2}{2} = 117{,}601'^{\#}$$

$$f_b = \frac{117{,}601 \times 12}{58.2} = 24{,}248^{psi} < 0.8 \times 36{,}000 = 28{,}800 \ (\text{O.K.})$$

Connection to Column

3 rows of $\frac{3}{4}''\phi$ A 325 bolts,

$V_{all.} = 39.8^{K} > 13.035^{K}$ (AISC 6^{th} Ed., Connection Table I)

DWG. NO. 568145
567676

ENGINEERING DEPARTMENT

CALCULATION SHEET

SUBJECT: Roof Ponding of FSB & VB (Structural) DESIGN CALCULATION NO. DC 99 - IV B

J.O. NO. 6733 MADE BY T. Wang DATE 9/17/84 CHK. BY MPA DATE 12-3-84

V. (Cont.)

B. (cont.)

(5) CHECK CRITICAL COLUMN

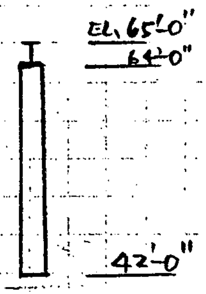
Column (A.9) - (12.8), 6 WF 15.5 (see plan on sht. 8)

$l = 22'$

$A = 4.62\,\square''$, $r_{min} = 1.45$, $k = 1$ (AISC 6th Ed.)

$\frac{kl}{r_{min}} = 182$

$F_a = 4.51^{ksi}$



Loads

From G-1, $P_1 = 12.275^K$

From G-2 : 12 WF 45-3

Reaction on Beam B-5,

$w_L = 39.1 \times 9.21 = 360^{\#}$ (see pg. 11)

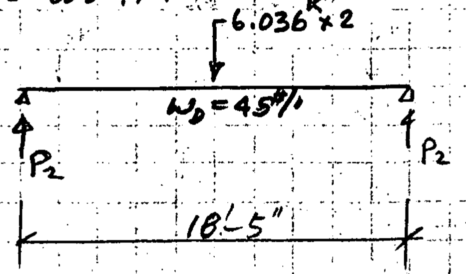
$w = 360 + \underset{(w_D)}{143} = 503^{\#}$ (SEE B-1)

$R_L = R_R = 503 \times \frac{24}{2} = 6036^{\#}$

Load from crane is not included since crane is normally parked at the east end of room, i.e. the probability of crane being on girder at the same time as PMP occurs is very small.

$\therefore P_2 = 6.036 + 0.045(18.42)/2$

$= 6.45^K$



DWG. NO.

ENGINEERING DEPARTMENT

CALCULATION SHEET

SUBJECT: Roof Ponding of FSB & VB (Structural) DESIGN CALCULATION NO. DC 99 – IV B

J.O. NO. 6733 MADE BY T. Wang DATE 9/18/84 CHK. BY MPA DATE 12-3-84

IV. (cont.)

B. (cont.)

(5) (cont.)

Loads (cont.)

Loading from B-3 and B-4 is directly supported by the wall (see sht. 25)

i.e. $w_L = \frac{26.1 + 27.4}{2} \times 9.2 = 246^{\#}$

$w_{B\text{-}3} = 246 + \underset{(w_0)}{143} = 389^{\#}$ (SEE BEAM B-1, sht. 18)

$= w_{B\text{-}4}$

∴ The total load on column A.9-12.8 during PMP condition is

$P = P_1 + P_2 = 12.275 + 6.45 = 18.725^k$

say $P = 19^k$ (including col. weight and MISC conn.)

Stress

$f_a = \frac{19}{4.62} = 4.1^{ksi}$

$\frac{f_a}{F_a} = \frac{4.1}{4.51} = 0.91 < 1.0$ (O.K.)

THE OTHER COLUMNS CARRY LESS LOADING AND THEY ARE O.K. PER INSPECTION.

DWG. NO.

SCE OD 397-B NEW 8/77 (CW)

ENGINEERING DEPARTMENT

CALCULATION SHEET

SUBJECT: Roof Ponding of FSB & UB (Structural) DESIGN CALCULATION NO. DC 99 - IV B

J.O. NO. 6733 MADE BY T. Wang DATE 12/3/84 CHK. BY MPA DATE 12-3-84

V. (cont.)

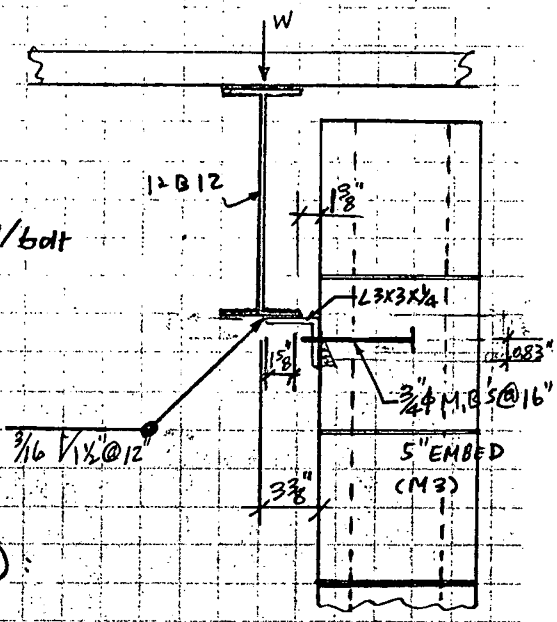
B. (cont.)

(6) CHECK SUPPORTS OF BEAM B-3 AT MASONRY WALL

Shear on bolt (3/4" ϕ M.B. @ 16") :

$V = 389^{\#/'} \times \frac{16}{12} = 519^{\#/\text{bolt}}$

$< 1100^{\#}$ (UBC (79) Table 24-6)
(O.K.)



Bending on $\angle$ 3x3x1/4 :

$M = 389 \times 3\frac{3}{8}'' = 1313^{''\#/'} \times \frac{16}{12} = 1751^{''\#/\text{bolt}}$

$f_b = \frac{M}{S} = \frac{1751}{\frac{12(0.25)^2}{6}} = 14{,}005^{\text{psi}}$

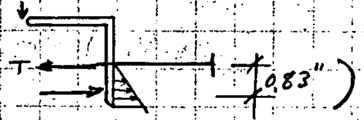
$< 0.75 f_y = 0.75(36{,}000) = 27{,}000$ psi
(O.K.)

Loading on welding (3/16" Fillet weld 1 1/2" @ 12") :

$P = W = 389^{\#/'}$

$P_{allowable} = 0.3(36{,}000)(\frac{3}{16})(1\frac{1}{2}) = 3037.5^{\#} > P$ (O.K.)

Pull out force on Bolt : (Assume



)

$T = \frac{M}{0.83} = \frac{1751}{0.83} = 2110^{\#/\text{bolt}}$

$< 2250^{\#}$ (UBC 79, Table 26-6, Assume embeded in concrete)
(O.K.) (Since the C.B filled with grout of $f'_c = 3000$ psi)

DWG. NO. 568145

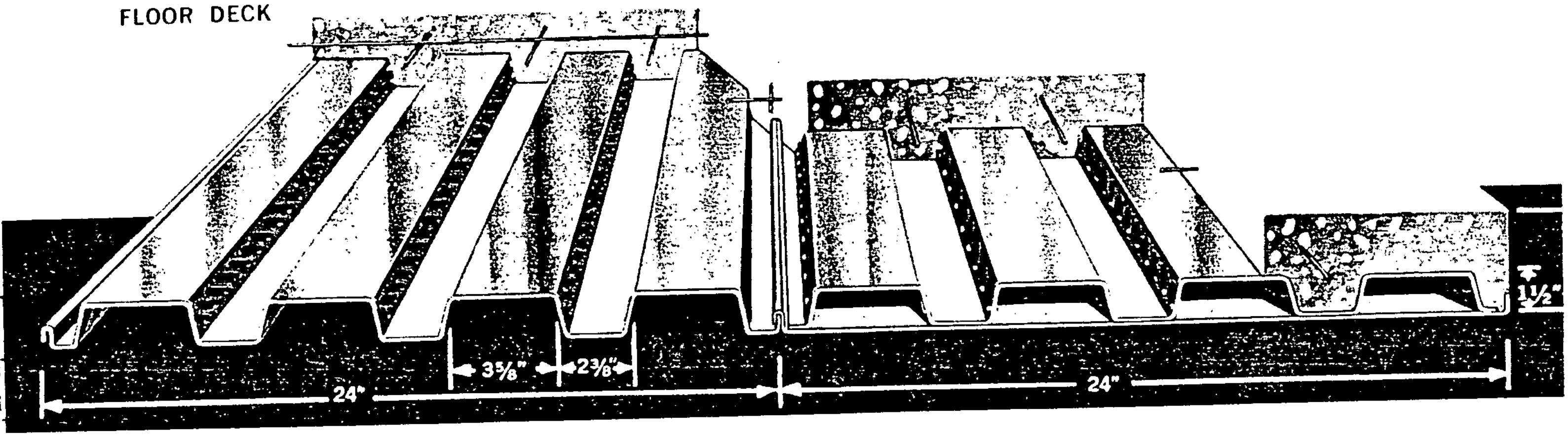
INLAND®

STEEL FLOOR SYSTEMS

Catalog No. 270, 1963

TYPE B HI-BOND FLOOR DECK

TYPE BF HI-BOND CELLUFLOR



TYPE B HI-BOND FLOOR DECK AND TYPE BF HI-BOND CELLUFLOR

Widely used for economy and versatility, these two products are often specified side-by-side for varying requirements on the same building. Used alone, Type BF Celluflor permits complete electrification of a floor with ample cell capacity for many installations. Both types are available in lengths up to 28′6″ in galvanized steel, gauges shown below.

1. SECTION PROPERTIES

Type	Gauge	Weight psf	I in.⁴	S in.³	Area in.²	[illegible] in.²	Y
B	22(2)	2.00	.18	.21	.54	13.25	.870
	20(2)	2.50	.23	.27	.65	13.25	.873
	18	3.20	.34	.40	.86	13.25	.877
	16	3.90	.44	.50	1.07	13.25	.881
	14	4.62	.57	.63	1.32	(1)	.888
	12	6.21	.80	.89	1.76	(1)	.906
BF	18-18	4.96	.58	.48	1.42	13.25	.499
	18-16	5.50	.61	.49	1.57	13.25	.465
	16-16	6.28	.77	.67	1.77	13.25	.540
	16-14	6.54	.83	.68	1.95	13.25	.506
	14-16	7.06	.95	.89	2.02	(1)	.615
	14-14	7.72	1.02	.91	2.20	(1)	.581
	12-14	9.14	1.32	1.33	2.64	(1)	.684
BF₂	18-16	4.90	.572	.48	1.39	13.25	.522
BF₃	18-16	4.20	.514	.47	1.21	13.25	.599

(1) These sections are not available with Hi-Bond feature. All other sections will be furnished as Hi-Bond Floor Deck only if specified.

(2) 22 and 20 gauge material is not recommended for use as floors except as Hi-Bond.

3. TOTAL SUPERIMPOSED LOADS FOR TYPES B and BF HI-BOND FLOOR DECK (Do not use shoring)

Type and Gauge	t	3½"	4"	4½"
	Wt	33	39	45
	M_R	1200	1600	1800
	V	600	700	800
	Span	Superimposed Load, lbs. per sq. ft.		
	6′-0″	200	233	266
B22*	6′-6″	184	215	249
	7′-0″	137	200	229
	7′-6″	160	187	214
B20*	8′-0″	104	175	200
	8′-6″	133	165	188
B18*	9′-0″	119	156	178
	9′-6″		142	160
B16*	10′-0″		128	

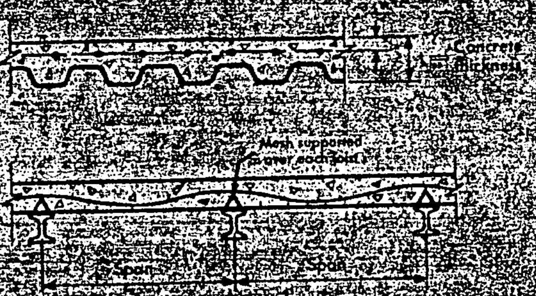
* Or BF, BF₂, BF₃ 18-16 Hi-Bond Floor Deck may be substituted.

NOTE — No continuity is assumed at supports.
See table 20 for minimum temperature reinforcing.

2. ALLOWABLE LOADS — NON COMPOSITE DESIGN

Type	Span Controlled by Span Condition	6′-6″			7′-0″			7′-6″			8′-0″		
		Stress		Defl.	Stress		Defl.	Stress		Defl.	Stress		Defl.
		S or D	T	D or T	S or D	T	D or T	S or D	T	D or T	S or D	T	D or T
	Gauge	UNIFORM LOAD, lbs. per sq. ft.											
B	18	125	156	90	107	134	72	94	118	58	82	103	48
	16	159	199	116	137	171	94	119	149	76	104	130	62
	14	199	249	149	171	214	120	150	188	97	131	164	80
	12	280	350	207	241	301	167	210	263	136	184	230	112
BF	18-18	152	190	151	131	164	121	114	142	100	100	125	81
	18-16	156	195	163	134	167	130	117	146	105	103	129	86
	16-16	211	264	204	181	226	164	158	197	133	139	174	109
	16-14	215	269	219	185	231	176	161	201	143	141	176	117
	14-16	280	350	251	241	301	201	211	264	163	184	230	135
	14-14	288		270	248	310	216	216	270	176	189	236	144
BF₂	18-16	153	191	151	131	164	121	115	143	98	100	126	81
BF₃	18-16	149	186	136	128	160	109	112	140	88	98	123	73

LOAD DATA—REINFORCED SLABS OVER RIBFORM



Note: The concrete slab is designed to carry all dead and live loads after reaching design strength.

1. Negative and Positive Moment — $WL^2/12$ [illegible]
2. Concrete f'_c — [illegible],000 psi f_c = 1,350 psi n = 10 [illegible]
   Steel f_s = 30,000 psi
   Effective depth d [illegible]
3. The weight of the slab has been considered in preparing these tables. All other dead and live loads must be compared with the tabular values.

SPAN	CONCRETE DEPTH (inches)	WELDED WIRE MESH TYPE 6 x 6 10/10	6 x 6 8/8	6 x 6 6/6	4 x 4 10/10	[illegible] 8/8
		SAFE SUPERIMPOSED LOADS (psf)				
2'-0"	2"	175	258	370	275	400
2'-6"	2"	105	157	228	167	248
	2½	166	249	354	261	375
3'-0"	2½	106	164	237	172	251
	3	146	220	322	234	[illegible]
	3½	196	289	403	289	425
3'-6"	2½	70	112	166	118	176
	3	98	152	227	169	249
	3½	133	202	286	202	302
4'-0"	3	67	108	166	116	182
	3½	92	144	209	144	221
	4	108	164	258	183	277
4'-6"	3	45	78	123	84	135
	3½	63	105	155	105	165
	4"	75	119	192	134	207
5'-0"	3	30	56	93	62	103
	3½	44	77	118	77	127
	4"	52	88	148	100	160

RIBFORM ACCESSORIES

Welding Washer

Anchor Washer—This special unit for use with Ribform [illegible] both a welding washer and a support for [illegible] [illegible]

Catalog NO. 270
1963

INLAND SUGGESTED ARCHITECT'S SPECIFICATIONS

HI-BOND, CELLUFLOR AND FLOOR DECK

1. Scope

a. Included are all steel subfloors and their accessories. For location, type, and gage of steel subfloors see structural drawings.

b. Not included under this section of the specification (work included under other sections.):

(1) Structural steel columns, girders, beams, and all miscellaneous bracing or supports of any kind for the steel subfloors.

(2) All reinforcing bars and reinforcing mesh.

(3) The cutting of holes in the subfloors for the passage of all materials for other trades.

(4) The cutting or drilling required for the attachment of materials of other trades to the steel subfloor.

(5) The final placement and attachment of welding access-hole covers and butt closure plates which close cells which are to be electrified.

(6) Electrification of the steel subfloor including headers, jump headers, outlets, and any other materials required to carry wires outside of the cellular panel cells.

(7) Concrete fill (Note: for Hi-Bond composite construction a minimum concrete strength of $f'c$ = 3,000 psi is required. For other steel sub floor construction a low water cement ratio is recommended to control crazing or temperature relief cracks).

(8) Fireproofing on the underside of steel subfloors.

(9) Any additional holes or cutting not indicated on the erection drawings shall be checked with and authorized in writing by the general contractor, as these holes or cut areas may block vital electrical cells or may be of size or shape requiring additional structural supports.

2. Material

Steel subfloors shall be formed from steel sheet conforming to Zinc Coated Steel Sheets of Structural Quality Coils and Cut Lengths, ASTM designation: A446 and Federal Specification QQ-S-775a, Type 2, Class E. Minimum coating 0.5 oz. per sq. ft. Minimum yield strength 33,000 psi.

3. Construction

a. Steel subfloors shall conform to the Inland Steel Products Company's type(s) ..... as to depth, cell area, cell spacing and design. (refer to page .... for specific characteristics of various Inland profiles. If desired the designer should be specific at this point regarding these points rather than making reference to catalog data).

b. When two sections are combined to form a cellular panel, they shall be structurally resistance welded in accordance with (4) Design.

c. (to be inserted if panels are to be used as electrical raceways). Panels shall be listed and labeled by Underwriters' Laboratories, Incorporated.

d. (to be inserted if panels and concrete fill are designed compositely). Deformations shall be provided in all vertical webs of the steel subfloor adequate to structurally