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SHEET NO. ______
JOB NO. ______
DATE 4/8/10
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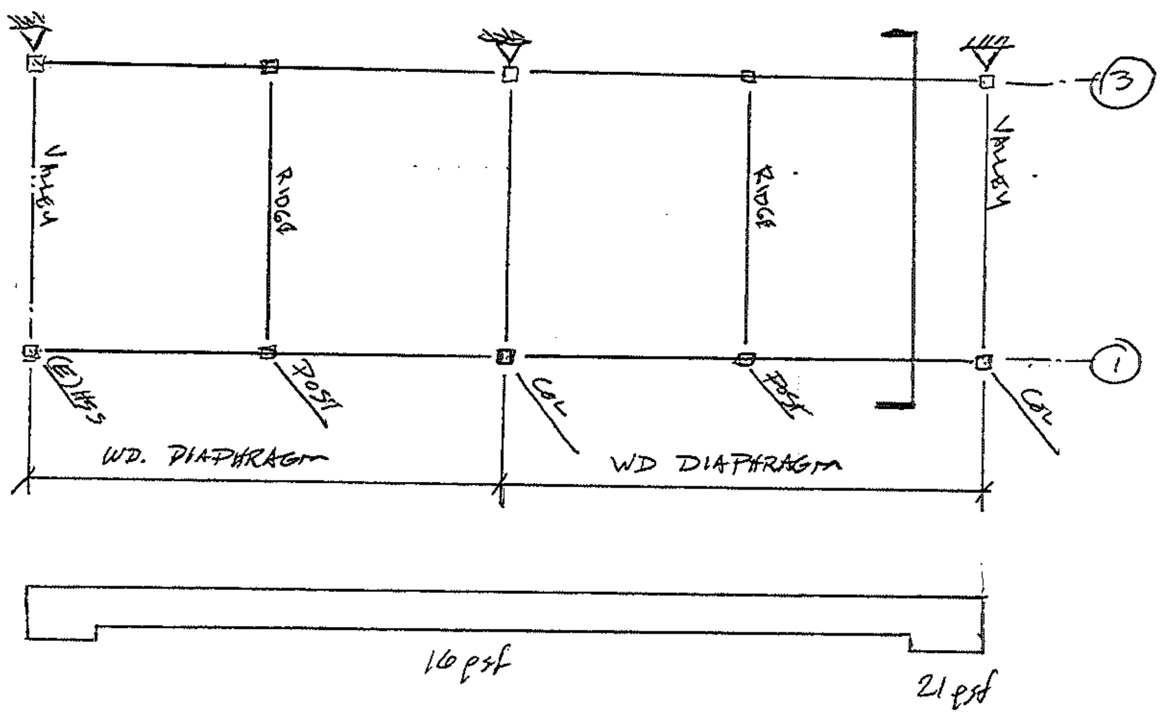
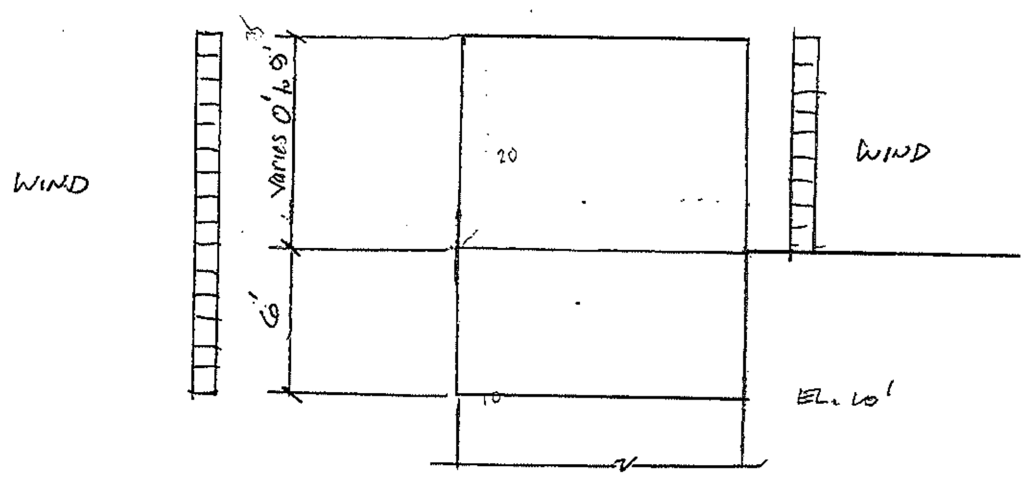
JOB NAME TJ Portland, ME
SUBJECT lateral stability of Canopies





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JOB NAME TJ Portland, ME
SUBJECT wind on canopies

Wind on canopy walls (C&C)

6.5.12.4 $p = q_h [(GC_p) - (GC_{pi})]$ $q_h = 21.8 - 0.94 = 20.5$ psf (Table 6.3)

$GC_p = +0.9$, -1.0 } Fig. 6-11, $A_e = 33\ ft^2$

-1.25 @ 5

$p = 20.5[-1.0 - (0.55)] = 32$ psf (Table 6.5)

stud max span = 15' @ ridge/peak 600S162-43 @ 16" o.c., max = 15'-4" (for 30 psf)

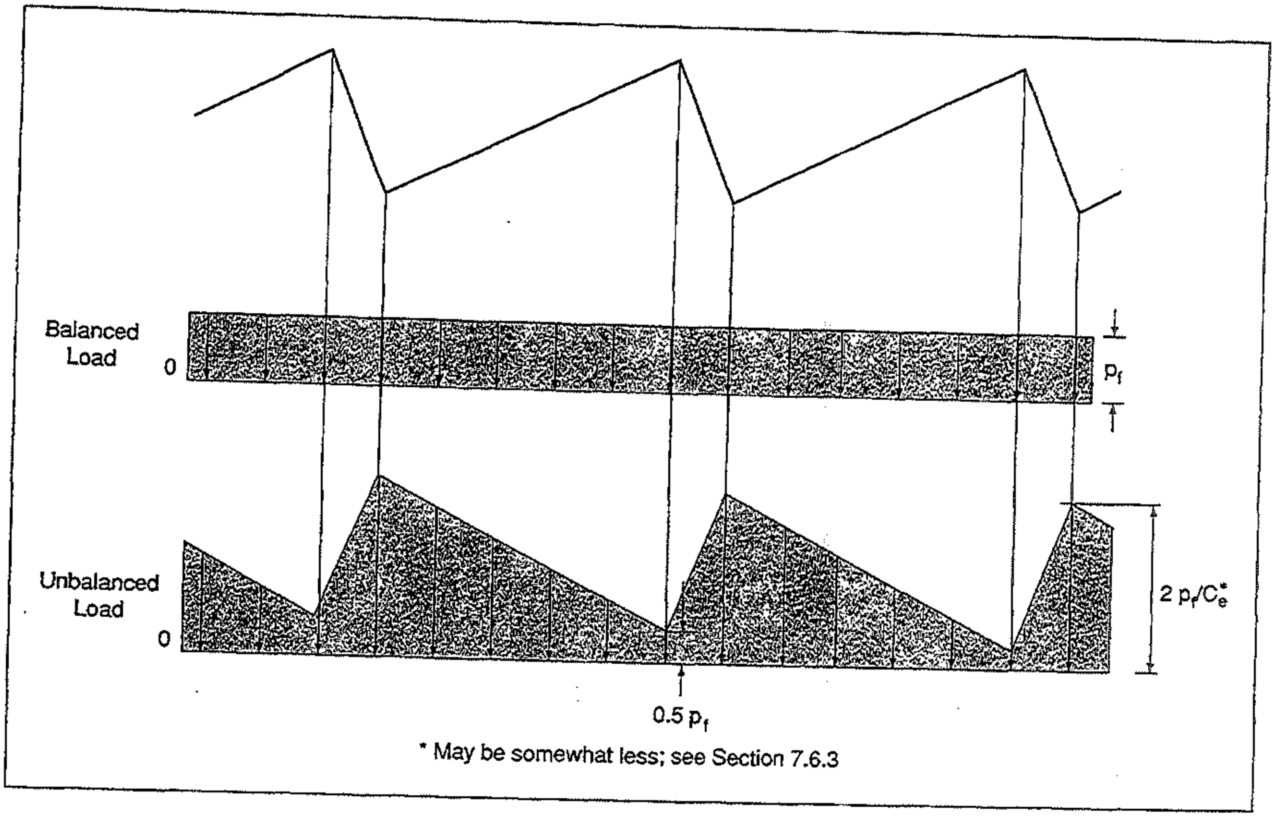

FIGURE 7-6 BALANCED AND UNBALANCED SNOW LOADS FOR A SAWTOOTH ROOF

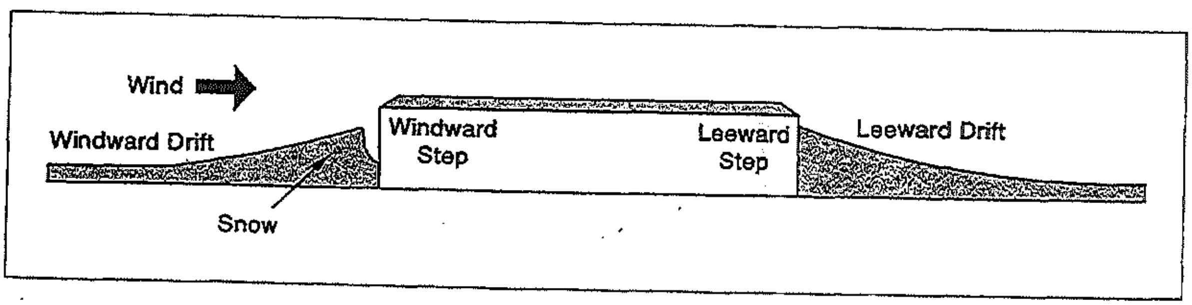

FIGURE 7-7 DRIFTS FORMED AT WINDWARD AND LEEWARD STEPS

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JOB NAME TJ - Portland, ME
SUBJECT Snow Loads

$p_g = 60 psf$

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

C_e per Table 7.2, = 1.0
C_t per Table 7.3, = 1.0 @ bldg
= 1.2 @ front canopy
I per Table 7.4 = 1.0 (category II per Table 1-1)

$p_f = 0.7 \cdot 1.0 \cdot 1.0 \cdot 1.0 \cdot 60 psf = 42 psf$ @ main roof
" " $\cdot 1.2$ " " $\cdot 60 psf = 50 psf$ @ canopy (unheated)

Sawtooth Canopy Roof (Section 7.4.4) $C_s = 1.0 \therefore p_s = p_f$
Fig. 7-6
2 conditions

1. balanced load = p_f = 50 psf
2. unbalanced
   @ ridge $0.5 p_f$ = 25 psf
   @ valley $2 \cdot p_f / C_e$ = $2 \cdot 50 psf / 1.0$ = 100 psf

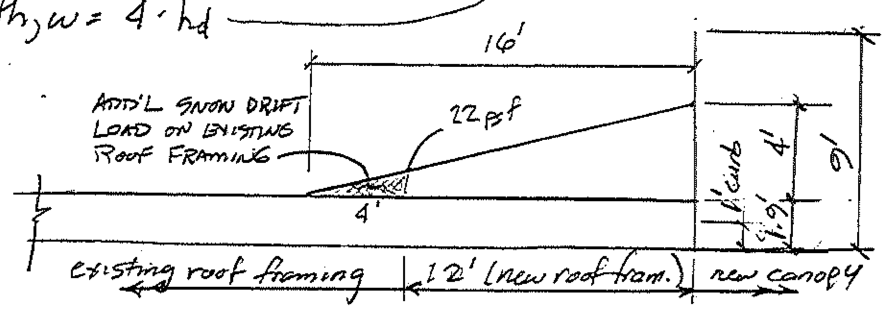
Snow Drift

$p_g = 60 psf$ $\quad l_u = 175'$

Fig. 7-9 $\Rightarrow h_d = 5.3'$ $\quad 0.75 \cdot h_d = 4.0'$

Section 7.8 Roof Projections (parapets), height of drifts = $0.75 h_d$
If parapet height < h_c, drift width, $w = 4 \cdot h_d$
$4 \cdot h_d = 16'$

h_b (balanced snow): $42/21.8 = 1.9'$
γ (density) $= 0.13 p_g + 14 = 0.13(60) + 14$
$= 21.8 pcf$



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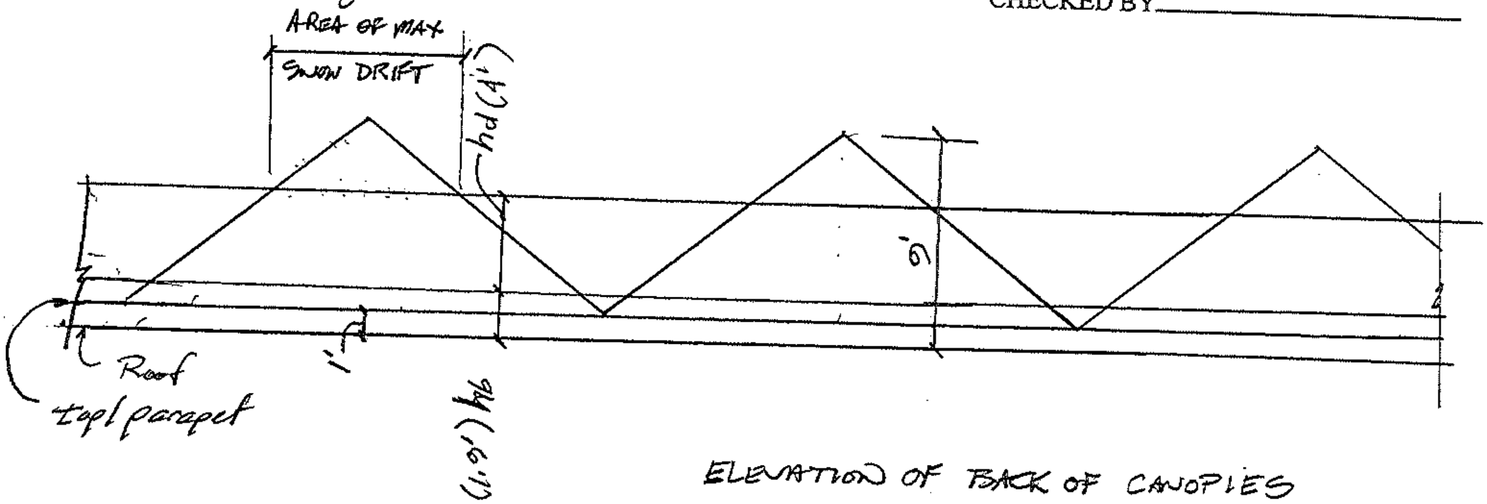
JOB NAME TJ - Portland, ME
SUBJECT Snow load



ELEVATION OF BACK OF CANOPIES

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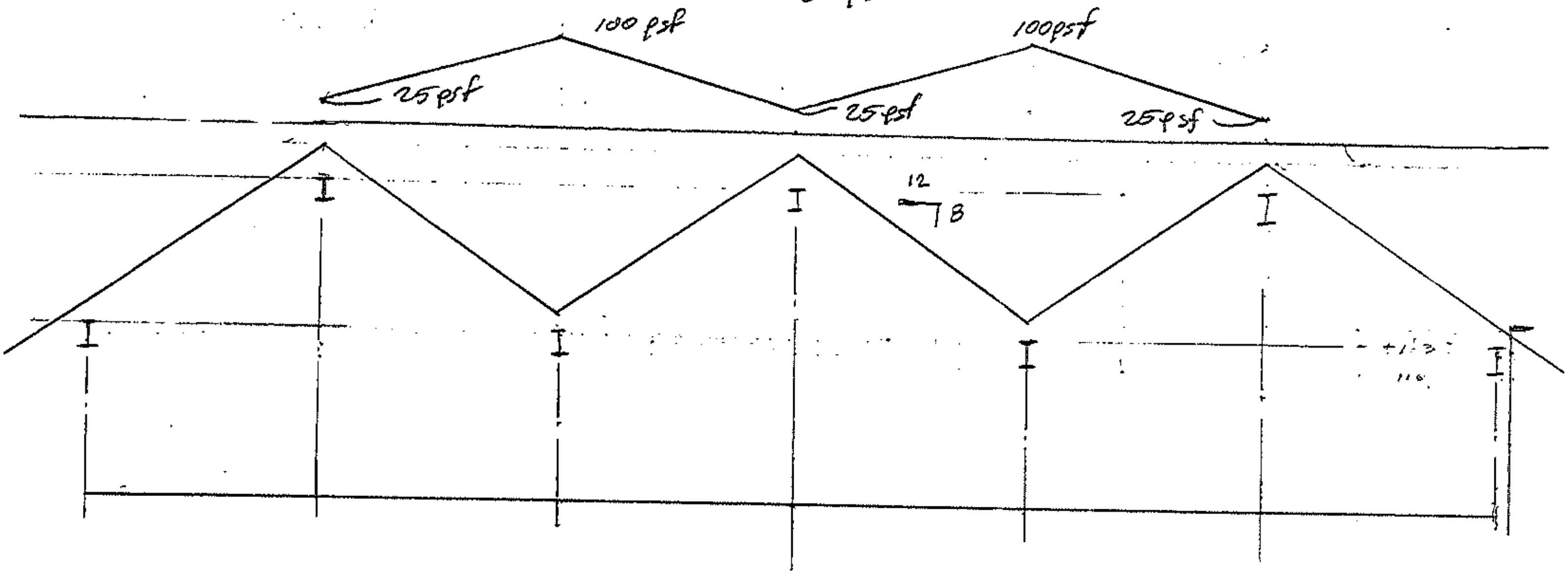
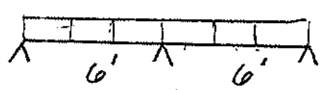
JOB NAME TJ Portland, ME
SUBJECT Front Canopy

versus uniform load = 50 psf



Timber Bms @ 6' o.c.

check planking depth for snow load

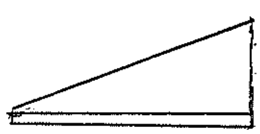


Hem Fir (Commercial Quality) F_b = 1350 psi, E = 1,400,000 psi

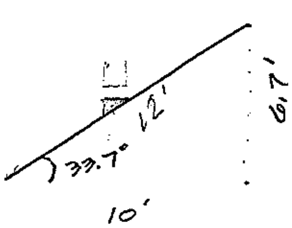
Table 4 of TCM — simple span allowable load is 112 psf.
Table 5 of TCM — $\ell/180$ is 65 psf
$\ell/180$, E = 1,800,000 psi is 83 psf
} 2" Plank (1½" net)

try 3" plank — F_b – okay for 6' span, E okay — Tables 6 & 7

Span will be less than 6' ∴ use 2" plank w/ min. values of $F_b = 1350$ psi, E = 1,600,000 psi



DL		
	Roofing	2
	Sheathing	2
	2" deck	4
	misc	4
		12 psf



unbalanced snow 25 psf, 100 psf

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DATE 3/31/10
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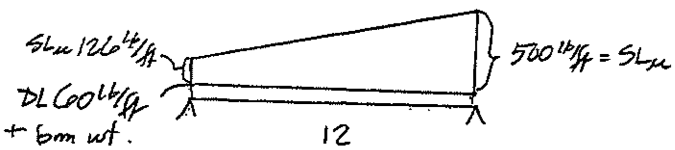
JOB NAME TJ Portland, ME
SUBJECT Front Canopy
TIMBER RAFTER



.832

DL 12 psf · cos 33.7° = 10 psf
SLu 25 psf · " " = 21 psf
100 psf · " " = 83 psf
} · 6'

SL_B = 50 psf · 6' = 300 lb/ft

Reference Enercalc for analysis and design for balanced & unbalanced snow load

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SHEET NO. ______
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DATE 3/20/10
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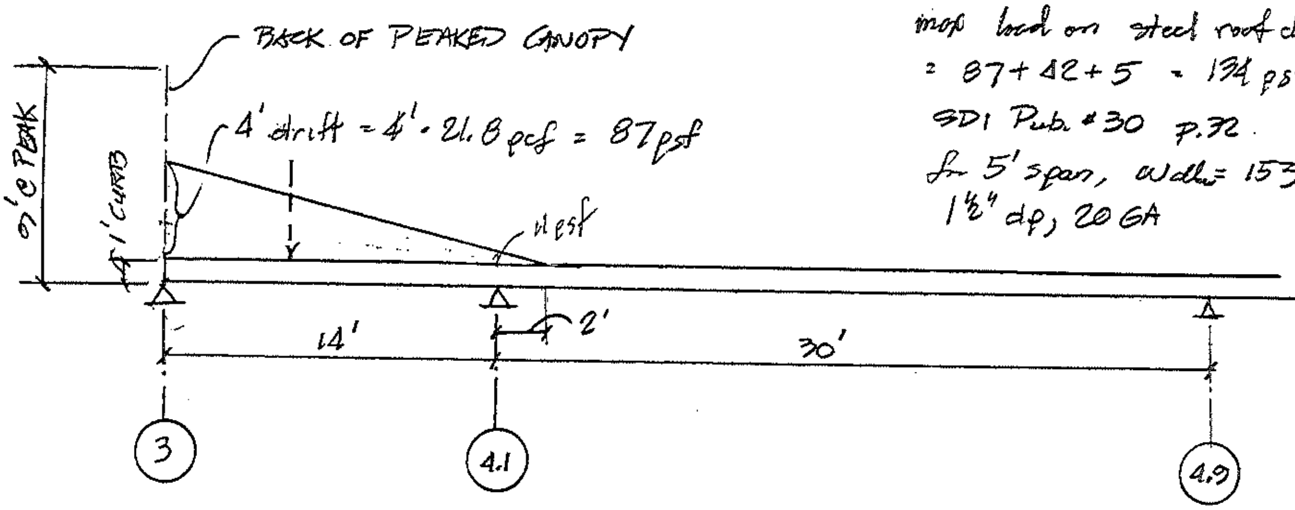
JOB NAME TJ - Portland, ME
SUBJECT Existing Roof Framing



max load on steel roof deck
= 87 + 42 + 5 = 134 psf
SDI Pub. #30 p.32
for 5' span, w_{allow} = 153 psf for 3 span cond.
1½" dp, 20 GA

Snow load = 42 psf
Dead load = 15 psf + bm wt.

Assume load is uniform along C.L. 4.1

$$w_{DL} = \frac{14' + 30'}{2} \cdot 15\,psf + 31\,lb/ft = 361\,lb/ft$$

$$w_{SL} = \frac{14' + 30'}{2} \cdot 42\,psf + \underbrace{\tfrac{1}{2} \cdot 16' \cdot 87\,psf}_{696\,lb/ft} \cdot \frac{16/3}{14'}$$

$$= 924\,lb/ft + 265\,lb/ft$$

$$= 1189\,lb/ft$$

check existing W16×31 F_y = 36 ksi

$$M = (1.19 + 0.36) \cdot 20^2/8 = \underline{\underline{77.5\,k\text{-}ft}}$$

from AISC 9th Ed. M_R = 93 k-ft > 77.5 k-ft okay

$$R_R = R_L = \frac{20'}{2}(1.19 + 0.36) = \underline{\underline{15.5^k}}$$

Column load = 2 · 15.5^k = 31^k @ H-4.1 { Existing Column HSS 5×5×3/8 per S1.2 / Existing FTG Cont wall & FTG }

Column capacity Table 4-4 AISC 13th Ed KL = 16' ⇒ P_n/Ω_c = 84^k >> 31^k OK

check conc. wall & ftg Ref 34/S4.1 HSS 5×5 w/ Base PL ¾" × 10" × 1'-0" OK

Effective wall horiz length (ACI 318, 15.2.4) = 1' + 4(1') = 5' ⇒ okay for 31^k $\frac{31{,}000}{5(12) \cdot 12''}$ = 43 psi

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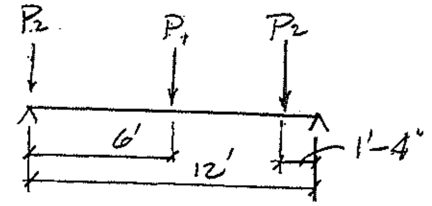
JOB NAME TJ Portland, ME
SUBJECT Beam Loads

Along C.L. 3

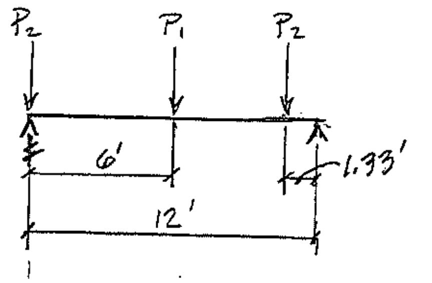
RIDGE BM



$P_{DL_1} = 20\,psf \cdot 12' \cdot 6' = 1440\,lb$

$P_{DL_2} = 20\,psf \cdot 12' \cdot 3' = 720\,lb$

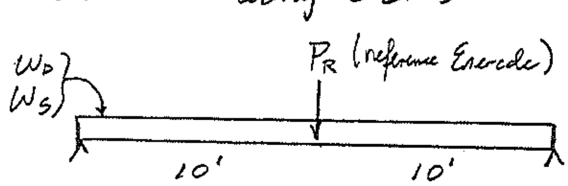
VALLEY BM



$R_D = 1.7^k$
$R_S = 4.8^k$

Ref above for P_D

new roof beam along C.L. 3



$P_{R_D} = 1.7^k$
$P_{R_S} = 3.2^k$ } use reaction @ front (Conservative)

$R_R = R_L = \begin{matrix} R_D = 4.1^k \\ R_S = 10.6^k \end{matrix}$ (Ref. Enercalc)

RIDGE BEAM

2 conditions 1. unbalanced snow
2. balanced snow

Reference elevation of snow loading on previous page.

$W_{snow_b} = 10' \cdot 50\,psf = 500\,lb/ft$

$W_{snow_u} = 10' \cdot 25\,psf + \frac{1}{2} \cdot 10' \cdot (100-25)\,psf \cdot \frac{10/3}{10} \cdot 2$
$= 250\,lb/ft + 250\,lb/ft$
$= 500\,lb/ft$

$P_{1s} = 500\,lb/ft \cdot 6' = 3000\,lb$
$P_{2s} = 500\,lb \cdot 3' = 1500\,lb$

VALLEY BEAM

$W_{snow_u} = 25\,psf \cdot 10' + \frac{1}{2} \cdot 10' (100-25)\,psf \cdot \frac{6.67}{10'} \cdot 2$
$= 250\,lb/ft + 500\,lb/ft = 750\,lb/ft$

$P_{1s} = 750\,lb/ft \cdot 6' = 4500\,lb$
$P_{2s} = 750\,lb/ft \cdot 3' = 2250\,lb$

Reference roof loading section showing snow, snow drift and dead loads

$W_D = \frac{14'}{2} \cdot 20\,psf + 6'_{wall} \cdot 20\,psf$
$= 260\,lb/ft + bm\ wt.$

$W_S = \frac{18'}{2} \cdot 42\,psf + \frac{1}{2} \cdot 16' \cdot 87\,psf \cdot \frac{2/3 \cdot 16'}{14'}$
$= 294\,lb/ft + 530\,lb/ft$
$= 824\,lb/ft$

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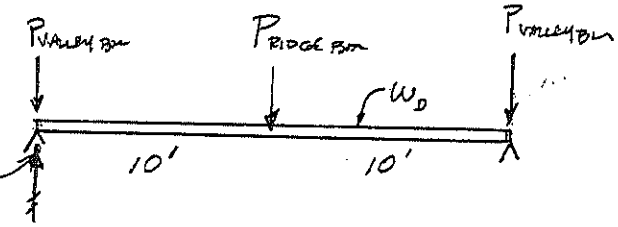
JOB NAME TJ Portland, ME
SUBJECT ____

beam @ front of canopy



$W_D = (10' + 9/2') \cdot 20\,psf = 290\,lb/ft + bm\ wt.$ (wall; avg @ peak)

$P_{RIDGE\ BEAM} = P_D = 1.7^k,\ P_S = 3.2^k$ } ref. enercalc

$P_{uplift} = 4.8^k$ (reference column load @ H-3)

Reference Enercalc for analysis & design

$R_D = 3.21^k$
$R_S = 1.6^k$
$R_W = -2.4^k$

$Uplift = 0.6(3.2^k/{}^{*2} + 1.7^k) + (-2.4^k) \cdot 2 =$
$= 4.9^k$ valley bm $- 4.8^k = +0.1^k$ no uplift ok

Support Reactions @ Canopy Valley beam
$R_D = 1.7^k.\ R_S = 4.8^k.\ R_W = -4.8^k$

Maximum column load @ H-1 $= P_D + P_S = (3.2^k/{}^{D*2\ valley} + 1.7^k) + (1.6^k/{}^{S*2\ valley} + 4.8^k) =$
$= 8.1^k + 8.0^k$
$= 16.1^k$

$16.^k$ + col. wt + brk wt $+ 1^k + 3^k = 20^k$ brk wt $0.67' * 10' \cdot 40\,psf = 2700\,lb$

$20^k / (3')^2 = 2222\,psf$ OK

check for uplift @ column.

$R = 0.6D + W = 0.6(3.2^k + 1.7^k) - 4.8^k = 1.9^k \uparrow$

check gravity load holding down column = brick col wrap + conc. pier + conc. ftg
$= 2.7^k + 2' \times 2' \times 4' (.150\,k/ft^3) +$ ftg wt
$= 2.7^k + 2.4^k +$ ftg wt $= 5.1^k +$ ftg wt $>> 1.9^k$ okay

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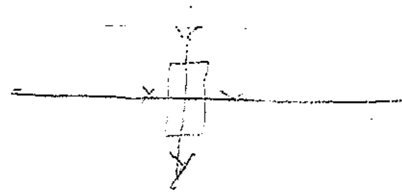

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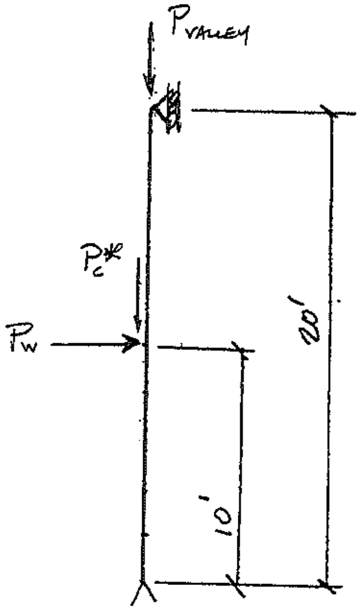
JOB NAME TJ Portland, ME
SUBJECT Column @ H-1



P_{VALLEY}: $P_D = 1.7^k$, $P_S = 4.8^k$, $P_W = -4.8^k$

*2 for beam reaction each side

Canopy Beam Reactions (P_c)
Gravity $P_D = 3.85^k$, $P_S = 1.6^k$, $P_W = -2.4^k$ ↑
↔ P_W 1.9^k

*[P_c centered on column] Note: verify w/ arch.

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JOB NAME TJs Portland, ME
SUBJECT COL H-3

VALLEY BEAM $P_D = 1.7^k$, $P_S = 4.8^k$ } Enercalc.

ROOF BEAM @ C.L. 3
$P_D = 2 \cdot 4.1^k = 8.2^k$
$P_S = 2 \cdot 10.6^k = 21.2^k$

Wind from store front

Ref Fig. 6-3

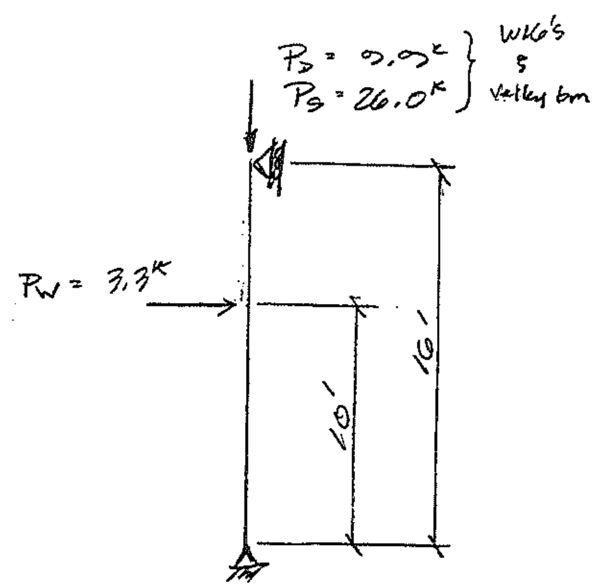
Wind area $= 20' \cdot \frac{16'}{2} = 160\ ft^2$

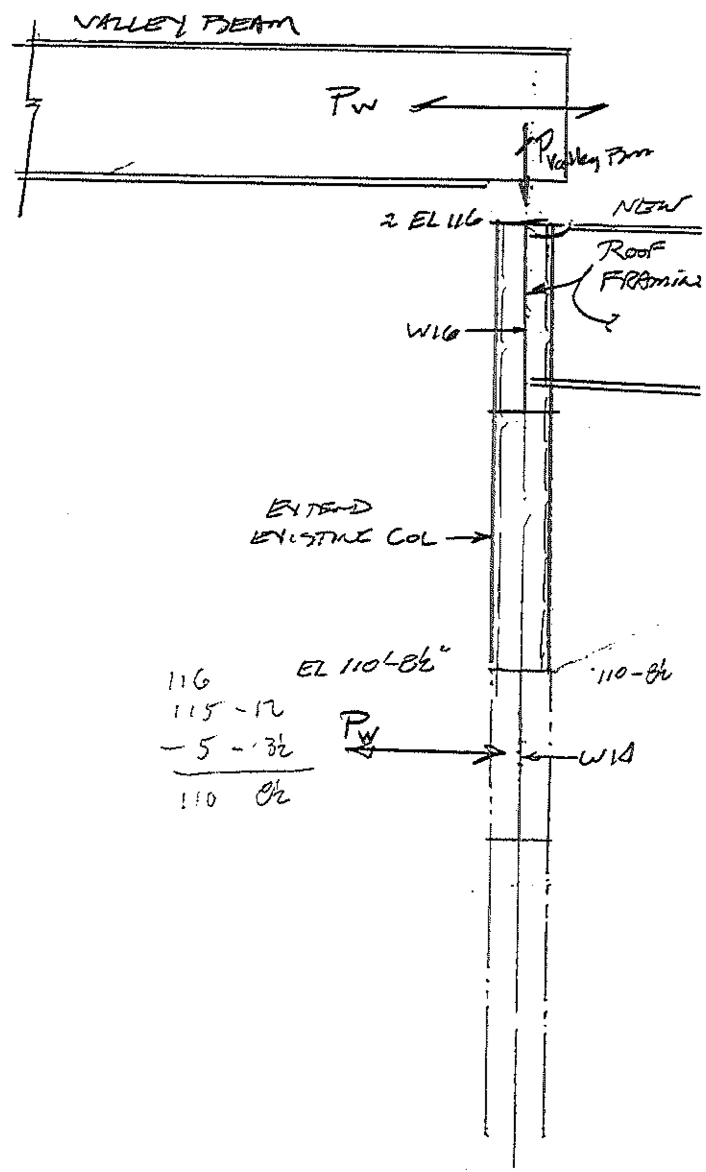
V = 100 mph, Wall @ Zone 4, 100 sf wind area

$P_{net30} = 15.3, -16.8$ psf
$\lambda = 1.23$

$P_{net30\,adj} = +18.8, -20.7$ psf

$P_W = 20.7\text{psf} \cdot 20' \cdot \frac{16'}{2} = 3{,}312$ lb





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DATE 4/12/10
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JOB NAME TJ Portland, ME
SUBJECT beam @ front canopy (CONT)

Wind (C&C) on beam @ front of canopy

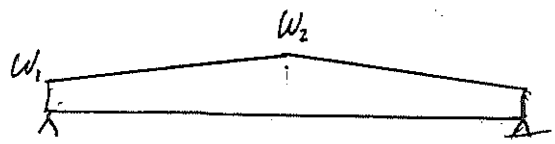
$p = q_h [(GC_p) - (GC_{pi})]$ psf   mean roof height = 22'

$q_h = 21.8 \cdot 0.92 = 20$ psf
Table 6-3

Fig. 6-11A
effective wind area $= \frac{1}{2}[\overset{180}{9' \cdot 20} + \overset{80}{\frac{1}{2} \cdot 10 \cdot 8 \cdot 2}] = 130$ sf   (4) +0.8, −0.92

$p = 20\,[(-0.9) - (0.55)] = 29$ psf

Wind on beam



$w_1 = \frac{9'}{2} \cdot 29\,psf = 131$ lb/ft
$w_2 = \frac{17'}{2} \cdot 29\,psf = 250$ lb/ft

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JOB NAME TJs - Portland, ME

SUBJECT beam @ storefront

Existing W14 x 26

span = 20' = L $\quad w_w = \frac{16'}{2} \cdot 21\,psf = 168\ ^{lb}/_{ft}$

check $\Delta = \frac{.000776 \cdot \overset{.17}{w} \cdot L^4}{I} \leq \frac{L}{600} = \frac{20(12)}{600} = 0.4$ say $\frac{1}{2}$"

$42.2\ in^4 < I$

$I_{W14_{y\text{-}y}} = 8.9\ in^4$

check add'l PL T & B $\quad I_{req} = 42.2 - 8.9 = 33.3\ in^4/2 = 16.65\ in^4$ per PL

$I_{PL} = \frac{t \cdot 6^3}{12} > 16.65 \Rightarrow t > 0.925"$

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JOB NAME TJs Portland, ME
SUBJECT Column / Ftg loads

Along C.L. 3

A-3 Column Load Roof beam $P_D = 2 \cdot 4.1^k = 8.2^k$

$P_S = 2 \cdot 10.6^k = 21.2^k$

Valley beam $P_D = 1.7^k$

$P_S = 4.8^k$

35.9^k

$P_D = 9.9^k$
$P_S = 26.0^k$

per S1.1 ftg = 4'-6" x 3'-0" $36^k / 4.5 \cdot 3.0 = 2667$ psf OK.

Wind uplift load [Reference wind calc.]

Area = $20' \cdot 6' = 120 ft^2$

Reference Fig 6-13 GC_p

① +0.8, −1.1

② +0.8, −1.7 } use avg. $\frac{1.1+1.7}{2} = 1.4$

③ +0.8, −1.7

$p = q_h [(GC_p) - (GC_{pi})]$ psf

$= 20.5 [-1.4 - (0.55)] = 40$ psf

Column uplift Load $120 ft^2 \cdot 40 psf = 4800$ lb

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JOB NO.
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JOB NAME TJs Portland, ME
SUBJECT Canopy

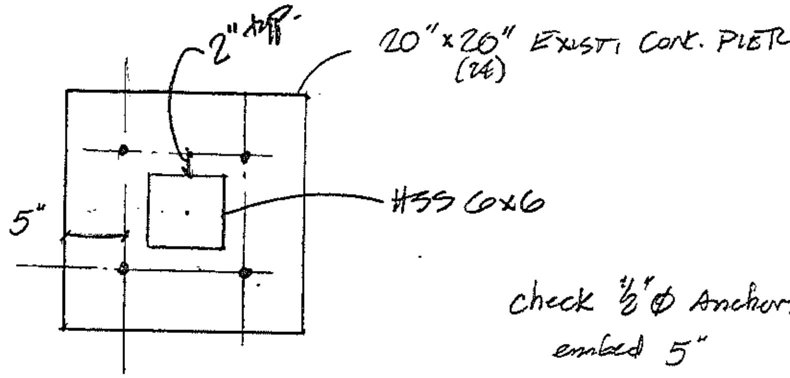
Uplift load @ Col = 2^k. Anchor column to existing 2'x2' pier

Check tensile strength of conc.

(22-2) $5\sqrt{f'_c} = 274$ psi $274 \text{ psi} \cdot 24" \cdot 20" = 131,520$ lb
$\phi \cdot 131,520 = 78.9^k$

$P_u = 1900 \text{ lb} \cdot 1.6 = 3.0^k \ll 79^k$ ok

check anchor pullout



check 1/8" ϕ Anchors embed 5"

ACI 318 D.5.2 – Conc. breakout strength

$\phi N_n \geq N_{ua}$ $\phi = 0.65$ per D.4.4. b)

$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$

$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$

$A_{Nco} = 9 h_{ef}^2 = 9 \cdot (5)^2 = 225$

$A_{Nc} = (5" + 10" + 5") \cdot 2 = 40 \text{ in}^2$

D.5.2.4 $\psi_{ec,N} = 1$ eccentric load on 20"x20" pier

D.5.2.5 $\psi_{ed,N} = 0.7 + 0.3 \frac{C_{a,min}}{1.5 h_{ef}} = 0.7 + 0.3 \cdot \frac{5"}{1.5(5")} = 0.90$

D.5.2.6 $\psi_{c,N} = 1.4$

D.5.2.7 $\psi_{cp,N}$ $C_{ac} = 1.5 h_{ef} = 2.5 \cdot 5 = 12.5$ (D.8.6) $\psi_{cp,N} = \frac{C_{a,min}}{C_{ac}} = \frac{5"}{12.5} = 0.40$
$1.5 \cdot 5/12.5 = 0.6 \Leftarrow$ Controls

Project Information

Project Title : Trader's Joe - Portland, ME

Description :

I.D. : 2010-709

Address : 87 Marginal Way, Portland, ME 04101

Project Leader : Bernard Oles, PE

Phone : 732 302 0300 x211 Fax : 732 302 1020 eMail : boles@barrylevin.com

Project Notes

Client Information

Client Company : Taylor Associates, Inc.
Address : 572 North Broadway, White Plains, NY 10603
Phone : 914 289 0011   Fax : 914 289 0022   eMail :

Contact : Jeffrey Taylor
Alternate Contact : Janet Bade

Client Notes :

Governing Code : IBC 2003 Edition
City Jurisdiction :
Contact Name :
Alternate Contact :

Building Official :
Address : , ,
Phone : Fax : eMail :

Notes :

ASCE 7-05 Seismic Factor Determination

Lic. # : KW-06000246

File: C:\Documents and Settings\boles\My Documents\ENERCALC Data Files\tj portland me.ec6
ENERCALC, INC. 1983-2009, Ver: 6.1.03
License Owner : barry levin & associates, inc.

Occupancy Category

Calculations per IBC 2006 & ASCE 7-05

Occupancy Category of Building or Other Structure : "II" : All Buildings and other structures except those listed as Category I, III, and IV — *ACSE 7-05, Page 3, Table 1-1*

Occupancy Importance Factor = 1 — *ACSE 7-05, Page 116, Table 11.5-1*

Ground Motion, Using USGS Database values

ASCE 7-05 9.4.1.1

Max. Ground Motions, 5% Damping :

S_S = 0.3143 g, 0.2 sec response
S_1 = 0.07687 g, 1.0 sec response

Longitude = 70.258 deg West
Latitude = 43.659 deg North
Location : PORTLAND, ME 4101

Site Class, Site Coeff. and Design Category

Site Classification "D" : Shear Wave Velocity 600 to 1,200 ft/sec		=	D	*ASCE 7-05 Table 20.3-1*
Site Coefficients Fa & Fv	Fa	=	1.55	*ASCE 7-05 Table 11.4-1 & 11.4-2*
(using straight-line interpolation from table values)	Fv	=	2.40	
Maximum Considered Eartquake Acceleration	S_{MS} = Fa * Ss	=	0.487	*ASCE 7-05 Table 11.4-3*
	S_{M1} = Fv * S1	=	0.184	
Design Spectral Acceleration	$S_{DS} = S_{MS}$ * 2/3	=	0.324	*ASCE 7-05 Table 11.4-4*
	$S_{D1} = S_{M1}$ * 2/3	=	0.123	
Seismic Design Category		=	B *(SD1 is most severe)*	*ASCE 7-05 Table 11.6-1*

Resisting System

ASCE 7-05 Table 12.2-1

Basic Seismic Force Resisting System . . . Steel systems not specifically detailed for seismic resistance, excluding cantilever column systems
Steel Systems Not Specifically Detailed for Seismic Resistance (except cantilevers)

Response Modification Coefficient " R "	=	3.00	Building height Limits :	
System Overstrength Factor " Wo "	=	3.00	Category "A & B" Limit:	No Limit
Deflection Amplification Factor " Cd "	=	3.00	Category "C" Limit:	No Limit
			Category "D" Limit:	Not Permitted
NOTE! See ASCE 7-05 for all applicable footnotes.			Category "E" Limit:	Not Permitted
			Category "F" Limit:	Not Permitted

Redundancy Factor

ASCE 7-05 Section 12.3.4

Seismic Design Category of A, B, or C therefore Redundancy Factor " p " = 1.0

Lateral Force Procedure

ASCE 7-05 Section 12.8

Equivalent Lateral Force Procedure

The "Equivalent Lateral Force Procedure" is being used according to the provisions of ASCE 7-05 12.8

Determine Building Period

Use ASCE 12.8-7

Structure Type for Building Period Calculation : Eccentrically Braced Steel Frames

" Ct " value = 0.030 — " hn " : Height from base to highest level = 16.0 ft
" x " value = 0.75

" Ta " Approximate fundemental period using Eq. 12.8-7 : Ta = Ct * (hn ^ x) = 0.240 sec
"TL" : Long-period transition period per ASCE 7-05 Maps 22-15 -> 22-20 — 8.000 sec

Building Period " Ta " Calculated from Approximate Method selected = 0.240 sec

" Cs " Response Coefficient

ASCE 7-05 Section 12.8.1.1

S_{DS} : Short Period Design Spectral Response	=	0.324	From Eq. 12.8-2, Preliminary Cs	=	0.108
" R " : Response Modification Factor	=	3.00	From Eq. 12.8-3 & 12.8-4 , Cs need not exceed	=	0.171
" I " : Occupancy Importance Factor	=	1	From Eq. 12.8-5 & 12.8-6, Cs not be less than	=	0.010

Cs : Seismic Response Coefficient = S_{DS} / (R/I) = 0.1082