2017-06
BTI

HCP ENGINEERING

650 E. Parkridge Ave, CA 92879, Lic. C50389
Ph: (951) 738-0840 Fax: (951) 738-1432

STRUCTURAL CALCULATIONS

Revision Roof Beacon Tower
(3) 2/24/18

FOR

HOME2 SUITES by HILTON

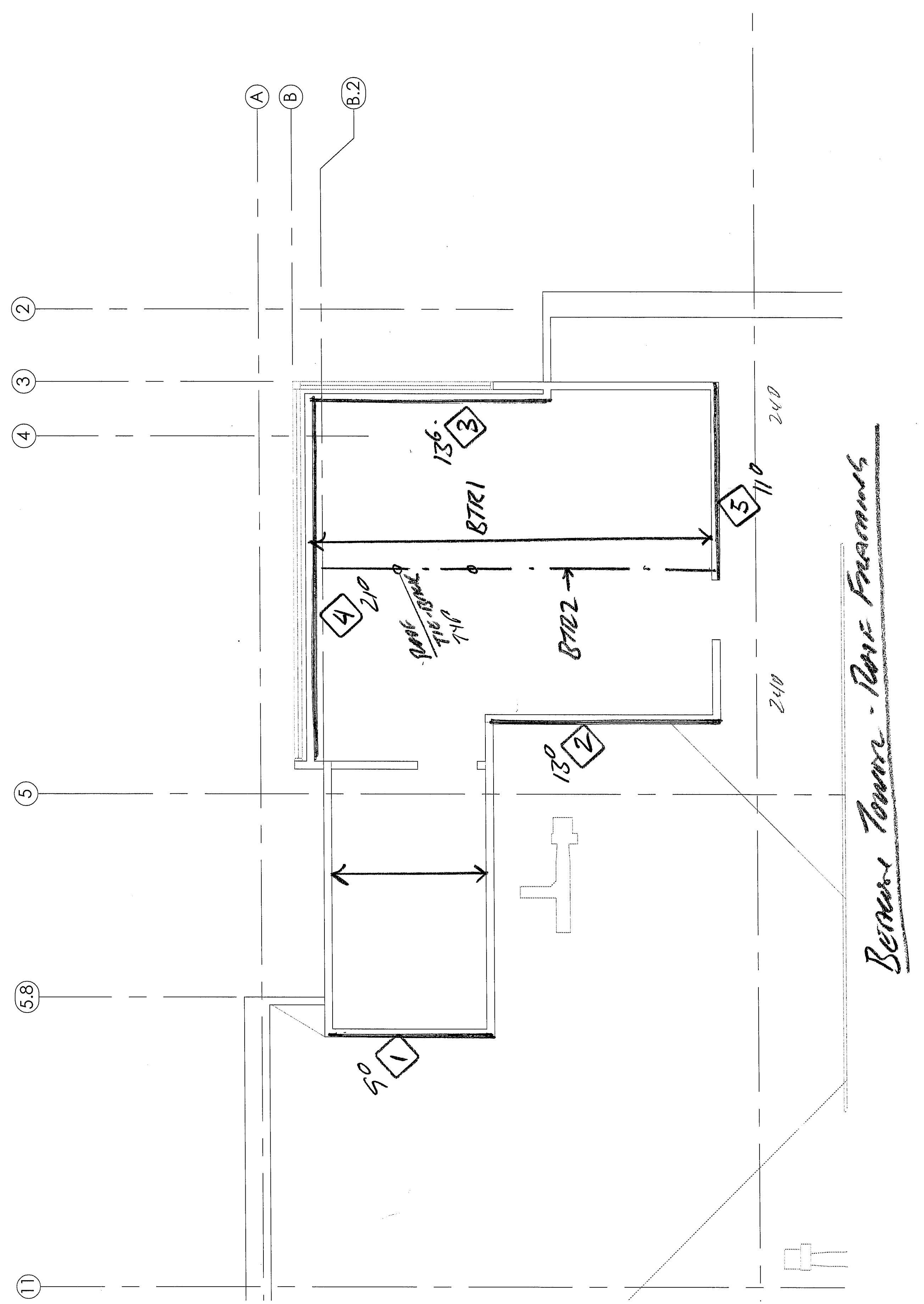
550 Gateway Blvd
South San Francisco, CA
(Project 2017-06)

FOR

ARRIS STUDIO ARCHITECTS
1306 Johnson Ave
San Luis Obispo, CA 93401
Ph: 805-547-2240

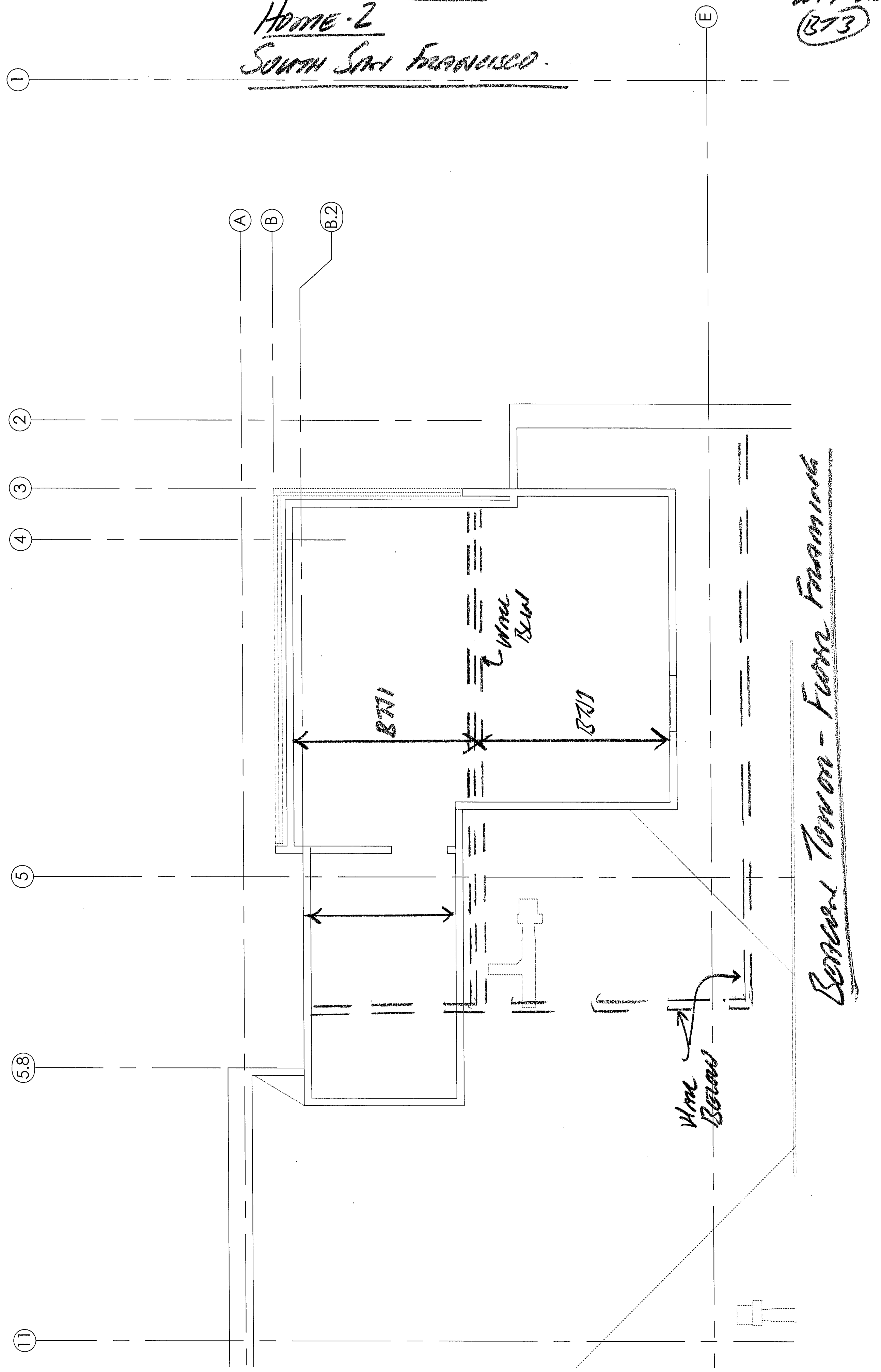
Beacon Tower
Home 2
South San Francisco

2017-06
(BT2)



Beacon Tower
Home-2
South San Francisco.

W17-06
(B73)



ROOF JOIST

BTR1 L = 24' MAX

W_{DL} = 16 PSF (NO SOLAR)

W_{LL} = 20 PSF

W_{TL} = 36 PSF

– USE 14" - TJI-230 @ 16" O.C. (MIN)

OR 16" TJI-230 @ 24" O.C.

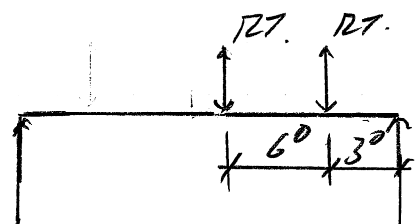
(SEE P.)

BTR2 L = 24' AT ROOF TIE-BACK

W_{DL} = 16 PSF

W_{LL} = 20 PSF

36.0 PSF



RT = 5000# ULTIMATE

$RT_{ASD} = \frac{5000}{1.4} = 3570$#

R_{MAX} = 2.6k

R_{MIN} = -1.4k UP

6.3k

-5.1k UP.

USE ECCQ COL. CAP UPLIFT ALLOW = 5535#
& CMST14 OR HOLDS HDU? TALL = 5646#

M^{+}_{MAX} = 31.62 k-' (DOWNWARD LOAD)

M^{-}_{MAX} = 22.3 k-' (UPWARD LOAD)

5¼" x 16" PSL

(SEE P.)

FLOOR JOIST

BTJ1 L = 17' MAX

W_{DL} = 17 PSF

W_{LL} = 40 PSF

W_{TL} = 57 PSF × 1.33 = 76 #/'

R = 646#

M = 33 k-"

I_{R} = 112 in⁴

2×12 @ 16" O.C.

FORTE®

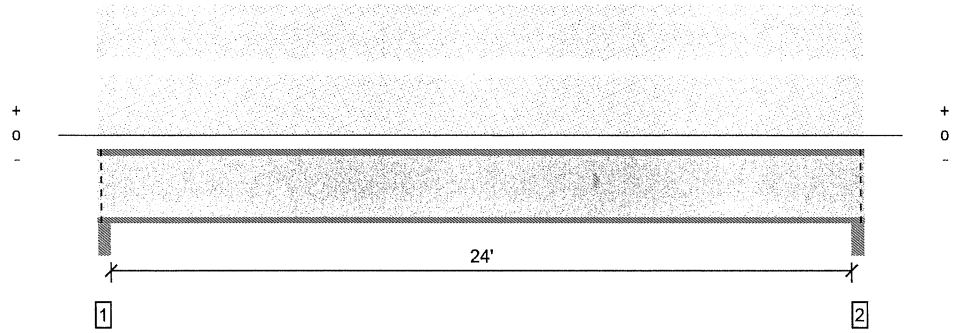
MEMBER REPORT *Level, Roof: Joist BTR1*

1 piece(s) 14" TJI® 230 @ 16" OC

PASSED

2017-06 B75

Overall Sloped Length: 24' 7 3/8"



All locations are measured from the outside face of left support (or left cantilever end).All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	590 @ 2 1/2"	1485 (3.50")	Passed (40%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	576 @ 3 1/2"	1945	Passed (30%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3505 @ 12' 3 1/2"	4990	Passed (70%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.432 @ 12' 3 1/2"	0.806	Passed (L/672)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.778 @ 12' 3 1/2"	1.209	Passed (L/373)	--	1.0 D + 1.0 L (All Spans)

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2009
Design Methodology : ASD
Member Pitch: 0.25/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Top Edge Bracing (Lu): Top compression edge must be braced at 4' 11" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 24' 7" o/c unless detailed otherwise.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Beveled Plate - SPF	3.50"	3.50"	1.75"	262	328	590	Blocking
2 - Beveled Plate - SPF	3.50"	3.50"	1.75"	262	328	590	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 24' 7"	16"	16.0	20.0	

Weyerhaeuser Notes

SUSTAINABLE FORESTRY INITIATIVE

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The product application, input design loads, dimensions and support information have been provided by Forte Software Operator

Forte Software Operator	Job Notes
Hitesh Patel HCP Engineering (951) 738-0840 cantilev@pacbell.net	

2/24/2018 12:09:15 PM
Forte v5.3, Design Engine: V7.0.0.5

FORTE®

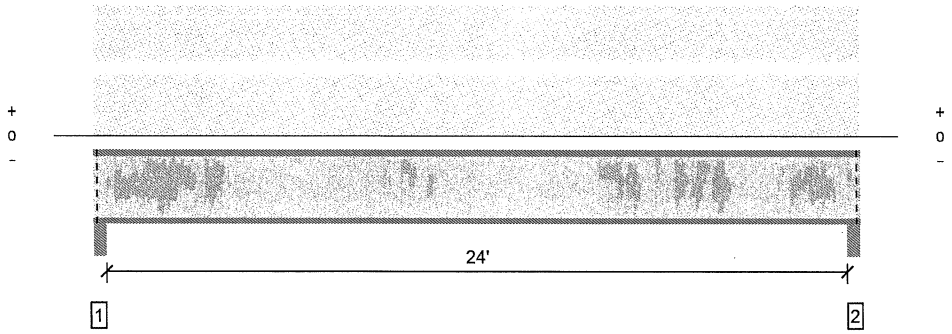
MEMBER REPORT *Level, Roof: Joist BTR1*

1 piece(s) 16" TJI® 230 @ 24" OC

PASSED

2017-06
(876)

Overall Sloped Length: 24' 7 3/8"



All locations are measured from the outside face of left support (or left cantilever end).All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	885 @ 2 1/2"	1485 (3.50")	Passed (60%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	864 @ 3 1/2"	2190	Passed (39%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	5257 @ 12' 3 1/2"	5710	Passed (92%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.483 @ 12' 3 1/2"	0.806	Passed (L/600)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.870 @ 12' 3 1/2"	1.209	Passed (L/333)	--	1.0 D + 1.0 L (All Spans)

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2009
Design Methodology : ASD
Member Pitch: 0.25/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Top Edge Bracing (Lu): Top compression edge must be braced at 4' 3" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 24' 7" o/c unless detailed otherwise.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Beveled Plate - SPF	3.50"	3.50"	1.75"	393	492	885	Blocking
2 - Beveled Plate - SPF	3.50"	3.50"	1.75"	393	492	885	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 24' 7"	24"	16.0	20.0	

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Weyerhaeuser warrants that the sizing of its products will be in accordance with Weyerhaeuser product design criteria and published design values. Weyerhaeuser expressly disclaims any other warranties related to the software. Use of this software is not intended to circumvent the need for a design professional as determined by the authority having jurisdiction. The designer of record, builder or framer is responsible to assure that this calculation is compatible with the overall project. Accessories (Rim Board, Blocking Panels and Squash Blocks) are not designed by this software. Products manufactured at Weyerhaeuser facilities are third-party certified to sustainable forestry standards. Weyerhaeuser Engineered Lumber Products have been evaluated by ICC ES under technical reports ESR-1153 and ESR-1387 and/or tested in accordance with applicable ASTM standards. For current code evaluation reports, Weyerhaeuser product literature and installation details refer to www.weyerhaeuser.com/woodproducts/document-library.

The product application, input design loads, dimensions and support information have been provided by Forte Software Operator

SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
Hitesh Patel HCP Engineering (951) 738-0840 cantilev@pacbell.net	

2/24/2018 12:21:54 PM
Forte v5.3, Design Engine: V7.0.0.5

Wood Beam

File = F:\HCPENG~1\ENGINE~1\ENERCA~1\2017-0~4.EC6
ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.10
Lic. # : KW-06007967 Licensee : HCP ENGINEERING

Description : BTR2 - AT ROOF TIE-BACK (BEACON TOWER) - CASE 1 DOWNWARD LOAD

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

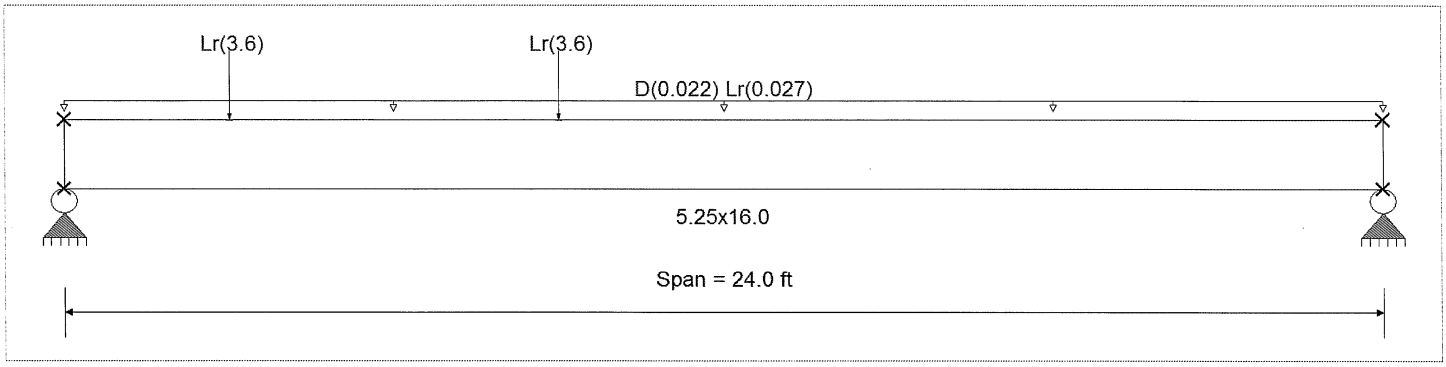
Wood Species : TrussJoist
Wood Grade : Parallam2.0E

Beam Bracing : Completely Unbraced

Fb +	2,900.0 psi
Fb -	2,900.0 psi
Fc - Prll	1,600.0 psi
Fc - Perp	650.0 psi
Fv	290.0 psi
Ft	1,000.0 psi

E : Modulus of Elasticity

Ebend- xx	2,000.0 ksi
Eminbend - xx	1,300.0 ksi
Density	34.150 pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.0220, Lr = 0.0270 , Tributary Width = 1.0 ft

Point Load : Lr = 3.60 k @ 3.0 ft

Point Load : Lr = 3.60 k @ 9.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.525** : 1	Maximum Shear Stress Ratio	=	**0.302** : 1
Section used for this span		**5.25x16.0**	Section used for this span		**5.25x16.0**
fb : Actual	=	1,693.78 psi	fv : Actual	=	109.58 psi
FB : Allowable	=	3,228.31 psi	Fv : Allowable	=	362.50 psi
Load Combination		+D+Lr+H	Load Combination		+D+Lr+H
Location of maximum on span	=	9.022 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.707 in	Ratio =	407 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.794 in	Ratio =	362 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+H					0.969	1.00	1.00	1.00	1.00	0.92			0.00	0.00	0.00	0.00
Length = 24.0 ft	1	0.067	0.031	0.90	0.969	1.00	1.00	1.00	1.00	0.96	3.02	161.69	2417.82	0.45	8.00	261.00
+D+Lr+H					0.969	1.00	1.00	1.00	1.00	0.96			0.00	0.00	0.00	0.00
Length = 24.0 ft	1	0.525	0.302	1.25	0.969	1.00	1.00	1.00	1.00	0.92	31.62	1,693.78	3228.31	6.14	109.58	362.50

Wood Beam

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.10
Lic. # : KW-06007967 Licensee : HCP ENGINEERING

Description : BTR2 - AT ROOF TIE-BACK (BEACON TOWER) - CASE 1 DOWNWARD LOAD

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr+H	1	0.7944	11.124		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	6.227	2.627
Overall MINimum	5.724	2.124
+D+H	0.503	0.503
+D+Lr+H	6.227	2.627
D Only	0.503	0.503
Lr Only	5.724	2.124
H Only		

Wood Beam

Description : BTR2 - AT ROOF TIE-BACK (BEACON TOWER) - CASE 1 UPWARD LOAD

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination ASCE 7-10

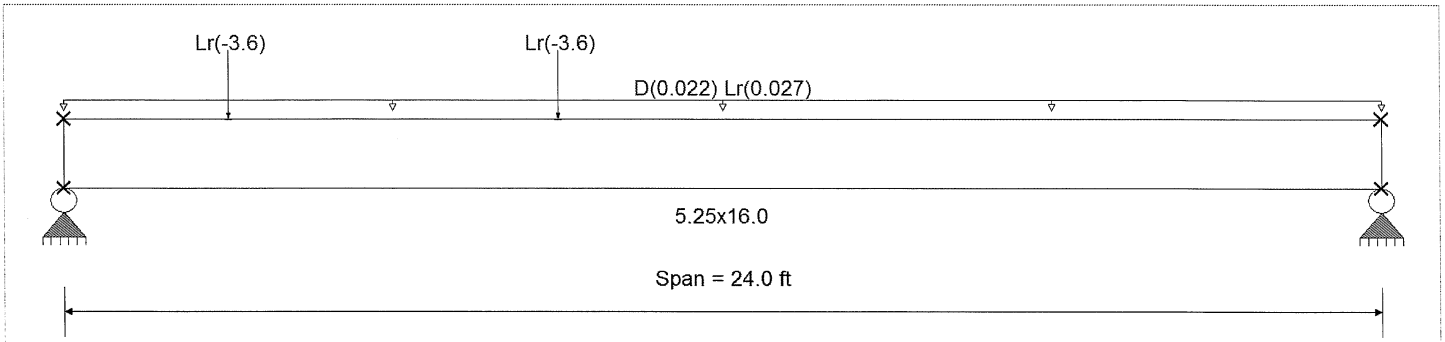
Wood Species : TrussJoist
Wood Grade : Parallam2.0E

Beam Bracing : Completely Unbraced

Fb +	2,900.0 psi
Fb -	2,900.0 psi
Fc - Prll	1,600.0 psi
Fc - Perp	650.0 psi
Fv	290.0 psi
Ft	1,000.0 psi

E : Modulus of Elasticity

Ebend- xx	2,000.0 ksi
Eminbend - xx	1,300.0 ksi
Density	34.150 pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.0220, Lr = 0.0270 , Tributary Width = 1.0 ft

Point Load : Lr = -3.60 k @ 3.0 ft

Point Load : Lr = -3.60 k @ 9.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.370** : 1	Maximum Shear Stress Ratio	=	**0.235** : 1
Section used for this span		**5.25x16.0**	Section used for this span		**5.25x16.0**
fb : Actual	=	1,194.85 psi	fv : Actual	=	85.33 psi
FB : Allowable	=	3,228.31 psi	Fv : Allowable	=	362.50 psi
Load Combination		+D+Lr+H	Load Combination		+D+Lr+H
Location of maximum on span	=	9.022 ft	Location of maximum on span	=	2.978 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <240
Max Upward Transient Deflection	-0.595 in	Ratio =	483 >=240
Max Downward Total Deflection	0.088 in	Ratio =	3279 >=360
Max Upward Total Deflection	-0.509 in	Ratio =	566 >=360

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+H					0.969	1.00	1.00	1.00	1.00	0.92			0.00	0.00	0.00	0.00
Length = 24.0 ft	1	0.067	0.031	0.90	0.969	1.00	1.00	1.00	1.00	0.96	3.02	161.69	2417.82	0.45	8.00	261.00
+D+Lr+H					0.969	1.00	1.00	1.00	1.00	0.96			0.00	0.00	0.00	0.00
Length = 24.0 ft	1	0.370	0.235	1.25	0.969	1.00	1.00	1.00	1.00	0.92	22.30	1,194.85	3228.31	4.78	85.33	362.50

Wood Beam

Description : BTR2 - AT ROOF TIE-BACK (BEACON TOWER) - CASE 1 UPWARD LOAD

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	Lr Only	-0.5952	10.861

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	-5.076	-1.476
Overall MINimum	0.503	0.503
+D+H	0.503	0.503
+D+Lr+H	-4.573	-0.973
D Only	0.503	0.503
Lr Only	-5.076	-1.476
H Only		

7/17-06
(8711)

SHEAR WALL SCHEDULE [1]

TYPE ◇#	SHEATHING MATERIAL	MINIMUM NOMINAL PANEL THICKNESS (in)	MINIMUM FASTENER PENETRATION IN FRAMING (in)	FASTENER TYPE & SIZE	PANEL EDGE FASTENER SPACING (in)	CAPACITY (plf)		ANCHOR BOLT SIZE (in)	ANCHOR BOLT SPACING w/ 3x P.T. PLATE (in)	2x SILL PLATE NAILING AT RAISED FLR. & UPPER FLOORS	TOP PLATE ANCHORS (A35 or LTP4) (in)	MINIMUM STUD SIZE AT ADJOINING PANEL EDGES	PERIODIC [3] INSPECTION REQUIRED
						SEISMIC	WIND						
9	PLYW'D / OSB	3/8	1 3/8"	8d	6	260	365	5/8"	48" [2]	16d @ 6" o.c.	18"	2x.	NO
10	PLYW'D / OSB	3/8	1 3/8"	8d	4	380	533	5/8"	36" [2]	16d @ 5" o.c.	12"	2x.	YES
11	PLYW'D / OSB	3/8	1 3/8"	8d	3	490	685	5/8"	30" [2]	16d @ 4" o.c.	10"	2x.	YES
12	PLYW'D / OSB	3/8	1 3/8"	8d	2	640	895	5/8"	24"	SIMPSON SDS 1/4"x6" @ 6" o.c.	8"	3x.	YES
13	PLYW'D / OSB	15/32	1 1/2"	10d	2	770	1078	5/8"	18"	SIMPSON SDS 1/4"x6" @ 6" o.c.	6 1/2"	3x.	YES
14	PLYW'D / OSB	19/32	1 1/2"	10d	2	870	1217	5/8"	16"	SIMPSON SDS 1/4"x6" @ 5 1/2" o.c.	6"	3x.	YES
15	PLYW'D / OSB BOTH SIDES	3/8	1 3/8"	8d	3	980	1370	3/4"	20"	SIMPSON SDS 1/4"x6" @ 5" o.c.	5"	3x.	YES
16	PLYW'D / OSB BOTH SIDES	3/8	1 3/8"	8d	2	1280	1790	3/4"	14"	SIMPSON SDS 1/4"x6" @ 3 1/2" o.c.	8" BOTH SIDES	3x.	YES
17	PLYW'D / OSB BOTH SIDES	15/32	1 1/2"	8d	2	1540	2156	3/4"	12"	SIMPSON SDS 1/4"x6" @ 3" o.c.	6" BOTH SIDES	3x.	YES
18	PLYW'D / OSB BOTH SIDES	19/32	1 1/2"	10d	2	1740	2435	3/4"	10"	SIMPSON SDS 1/4"x6" @ 2 1/2" o.c.	6" BOTH SIDES	3x.	YES

SPECIAL NOTES:

1. SHEAR WALL CAPACITY IS BASED ON ANSI/AF&PA SDPWS-2008, TABLE 4.3A AND ADJUSTED FOR ASD DESIGN PER 4.3.3. AND 4.3.3.2.
2. 2x FOUNDATION SILL PLATES MAY BE USED. ANCHOR BOLT SPACING SHALL BE REDUCED TO ONE-HALF THE SPECIFIED DISTANCE FOR 3x FOUNDATION SILL PLATES.
3. PERIODIC INSPECTION REQUIRED BY A REGISTERED DEPUTY INSPECTOR.

TYPICAL NOTES:

a. WOOD STRUCTURAL PANELS SHALL CONFORM TO THE REQUIREMENTS FOR ITS TYPE IN DOC PS-1 AND PS-2
b. PANELS SHALL NOT BE LESS THAN 4'x8', EXCEPT AT BOUNDRIES AND CHANGES IN FRAMING. PANELS SHALL BE INSTALLED VERTICAL. FRAMING MEMBERS OR BLOCKING SHALL BE PROVIDED AT THE EDGES OF PANELS.
c. NAILS LOCATED AT LEAST 3/8" FROM EDGES AND ENDS OF PANELS. MAXIMUM NAIL SPACING OF 6" ON CENTER ALONG PANEL EDGES. MAXIMUM NAIL SPACING OF 12" ON CENTER ALONG INTERMEDIATE FRAMING (FIELD NAILING).
d. ALL FRAMING MEMBERS USED FOR SHEAR WALL CONSTRUCTION SHALL BE 2" NOMINAL OR LARGER. MAXIMUM STUD SPACING SHALL BE 16" ON CENTER.
e. SHEAR WALL BOUNDRY ELEMENTS, SUCH AS END POSTS, SHALL BE PROVIDED AT ALL WALL ENDS. USE (2) 2x MINUMUM, UNLESS NOTED OTHERWISE.
f. SHEAR SHEATHING SHALL NOT BE USED TO SPLICE BOUNDRY ELEMENTS.
g. END POST (STUD OR COLUMN) SHALL BE PROVIDED FULL END BEARING.
h. FRAMING MEMBER SHALL BE DOUGLAS FIR.
i. WHERE PLYWOOD IS APPLIED ON BOTH FACES OF WALL AND NAILING IS LESS THAN 6" O.C., PANEL JOINTS SHALL BE OFFSET TO FALL ON DIFFERENT FRAMING MEMBERS, OR FRAMING SHALL BE 3x OR THICKER, NAILS ON EACH SIDE SHALL BE STAGGERED, SILL PLATES SHALL BE 3x MEMBERS.
j. PLYWOOD JOINT AND SILL PLATE NAILING SHALL BE STAGGERED IN ALL CASES.
k. NAILS SHALL BE COMMON OR GALVANIZED BOX. GALVANIZED NAILS SHALL BE HOT-DIPPED OR TUMBLED.
l. ANCHOR BOLTS SHALL BE INSTALLED A MINIMUM OF 4 3/8" AND A MAXIMUM OF 12" FROM PLATE ENDS. A MINIMUM OF TWO (2) BOLTS PER PLATE. ALL ANCHOR BOLTS SHALL ALSO HAVE A 2 1/2"x2 1/2"x1/4" PLATE WASHER. THE PLATE WASHER SHALL EXTEND TO WITHIN 1/2" OF THE EDGE OF THE BOTTOM PLATE ON THE SHEATHING SIDE.
m. ALL ANCHOR BOLTS SHALL BE EMBEDED 7" MIN. (WHERE DUAL POUR CONCRETE FOUNDATION SYSTEM IS USED, ANCHOR BOLTS SHALL BE EMBEDED 7" MIN. INTO CONCRETE FIRST POUR.)
n. PROVIDE PLYWOOD END NAILING TO ALL HOLDOWN POSTS.

SHEAR WALL SCHEDULE | A

2017-06
(B712)

MecaWind Pro v2.2.7.5 per ASCE 7-10

Developed by MECA Enterprises, Inc. Copyright www.mecaenterprises.com

Date	: 12/17/2017	Project No.	:
Company Name	: HCP ENGINEERING	Designed By	:
Address	: 650 E. Parkridge Ave., Suite 1	Description	:
City	: Corona	Customer Name	:
State	: CA	Proj Location	:

File Location: C:\Users\HCP ENGINEERING\AppData\Roaming\MecaWind\Default.wnd

Directional Procedure Simplified Diaphragm Building (Ch 27 Part 2)

Basic Wind Speed(V)	= 110.00 mph				
Structural Category	= II		Exposure Category	=	C
Natural Frequency	= N/A		Flexible Structure	=	No
Importance Factor	= 1.00		Kd Directional Factor	=	0.85
Damping Ratio (beta)	= 0.01				
Alpha	= 9.50		Zg	=	900.00 ft
At	= 0.11		Bt	=	1.00
Am	= 0.15		Bm	=	0.65
Cc	= 0.20		l	=	500.00 ft
Epsilon	= 0.20		Zmin	=	15.00 ft
Pitch of Roof	= 0 : 12		Slope of Roof(Theta)	=	.00 Deg
h: Mean Roof Ht	= 60.00 ft		Type of Roof	=	FLAT
RHt: Ridge Ht	= 60.00 ft		Eht: Eave Height	=	60.00 ft
OH: Roof Overhang at Eave	= .00 ft		Overhead Type	=	No Overhang
Bldg Length Along Ridge	= 187.00 ft		Bldg Width Across Ridge	=	163.00 ft

Gust Factor Calculations

Gust Factor Category I Rigid Structures - Simplified Method

Gust1: For Rigid Structures (Nat. Freq.>1 Hz) use 0.85 = 0.85

Gust Factor Category II Rigid Structures - Complete Analysis

Zm: 0.6*Ht	=	36.00 ft
lzm: Cc*(33/Zm)^0.167	=	0.20
Lzm: l*(Zm/33)^Epsilon	=	508.78 ft
Q: (1/(1+0.63*((B+Ht)/Lzm)^0.63))^0.5	=	0.85
Gust2: 0.925*((1+1.7*lzm*3.4*Q)/(1+1.7*3.4*lzm))	=	0.85

Gust Factor Summary

Not a Flexible Structure use the Lessor of Gust1 or Gust2 = 0.85

Table 26.11-1 Internal Pressure Coefficients for Buildings, GCpi

GCPi : Internal Pressure Coefficient = +/-0.18

Topographic Adjustment

0.33*z	=	1.00
Kzt (0.33*z): Topographic factor at elevation 0.33*z	=	1.00
Vtopo: Adjust V per Para 27.5.2: V * [Kzt(0.33*z)]^0.5	=	110.00 mph

MWFRS Diaphragm Building Wind Pressures per Ch 27 Pt 2

All pressures shown are based upon ASD Design, with a Load Factor of .6

MWFRS Pressures for Wind Normal to 187 ft wall (Normal to Ridge)

WALL PRESSURES PER TABLE 27.6-1

L/B: Bldg Dim in Wind Dir / Bldg Dim Normal to Wind Dir	=	0.87	
h: Height to top of Windward Wall	=	60.00 ft	
ph: Net Pressure at top of wall (windward + leeward)	=	20.76 psf	←
p0: Net Pressure at bottom of wall (windward + leeward)	=	17.64 psf	
ps: Side wall pressure acting away from wall = .54 * ph	=	-11.21 psf	
pl: Leeward wall pressure acting away from wall = .38 * ph	=	-7.89 psf	
pwh: Windward wall press @ top acting toward wall = ph-pl	=	12.87 psf	
pw0: Windward wall press @ bot acting toward wall = p0-pl	=	9.75 psf	

ROOF PRESSURES PER TABLE 27.6-2

h: Mean Roof Height	=	60.000 ft
Lambda: Exposure Adjustment Factor	=	1.000
Slope: Roof Slope	=	.00 Deg

Any slope less than 9.46 Deg is treated as a 'Flat' roof per Table 27.6-2

Zone	Load Case1 psf	Load Case2 psf

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1	.00	.00
2	.00	.00
3	-19.02	.00
4	-16.98	.00
5	-13.92	.00

Note: A value of '0' indicates that the zone/load case is not applicable.

MWFRS Pressures for Wind Normal to 163 ft wall (Along Ridge)

WALL PRESSURES PER TABLE 27.6-1

L/B:	Bldg Dim in Wind Dir / Bldg Dim Normal to Wind Dir	=	1.15
h:	Height to top of Windward Wall	=	60.00 ft
ph:	Net Pressure at top of wall (windward + leeward)	=	20.38 psf
p0:	Net Pressure at bottom of wall (windward + leeward)	=	17.24 psf
ps:	Side wall pressure acting away from wall = .55 * ph	=	-11.31 psf
pl:	Leeward wall pressure acting away from wall = .36 * ph	=	-7.41 psf
pwh:	Windward wall press @ top acting toward wall = ph-pl	=	12.97 psf
pw0:	Windward wall press @ bot acting toward wall = p0-pl	=	9.83 psf

ROOF PRESSURES PER TABLE 27.6-2

h:	Mean Roof Height	=	60.000 ft
Lambda:	Exposure Adjustment Factor	=	1.000
Slope:	Roof Slope	=	.00 Deg

Any slope less than 9.46 Deg is treated as a 'Flat' roof per Table 27.6-2

Zone	Load Case1 psf	Load Case2 psf
1	.00	.00
2	.00	.00
3	-19.02	.00
4	-16.98	.00
5	-13.92	.00

Note: A value of '0' indicates that the zone/load case is not applicable.

Parapet MWFRS Presures:

hp:	Height to Top of Parapet	= 72 ft
php:	Wall Pressure for L/B = 1 at hp (Fig 27.6-1)	= 21.97 psf
pp:	Parapet total pressure (Leeward + windward) - 2.25*php	= 49.44 psf

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BEACON TOWER SHEAR WALLS

PENTHOUSE - TBL 13.5.1

$S_{DS} = 1.253$
$a_p = 2.5$
$R_p = 3.5$
$\Omega_0 = 2.5$
$I = 1.0$
$z = 68.5'$
$h = 60.0'$

$W_p = 16(670) + 15(4)(115) = 17620^{\#}$

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\frac{R_p}{I}} \left(1 + 2\frac{z}{h}\right) \times 0.7$$

$$= \frac{0.4(2.5)(1.253)(17620)\left(1 + 2\left(\frac{68.5}{60}\right)\right)}{\left(\frac{3.5}{1}\right)} \times 0.7$$

$= 14545^{\#}$ $< F_{p_{max}} = 1.6 S_{DS} I W_p \times 0.7$

$= 14545/670 = 22.3$ PSF

WALL 1 $\theta_s = 9°$

$V_w = 20.76(4.2 + 3)(10) = 1455^{\#}$
$V_s = 22.3(100) = 2230^{\#}$

$n_s = 248^{\#/'}$ - TYPE 9

$T = C = 2230^{\#}$
DL = 5 min - CS14 STRAP OR HOLD

USE CASTIN OR HOLD

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Wall 2 $\theta_s = 13^\circ$

$V_w = 1495 + 150(10) = 3000^{\#}$

$V_s = 2230 + 22.3(280) = 7805^{\#}$

$v_s = 600^{\#/ft}$ - Type 12

$T=C = 5403^{\#}$
$DL = 5 min$ ∴ CMST14 or HDU5 Hdr

Drag $v_D = 313^{\#/ft}$

$F_T = 3434^{\#}$ - MST140

Wall 3 $\theta_s = 13^\circ$

$V_w = 150(10) = 1500^{\#}$

$V_s = 22.3(280) = 5625^{\#}$

$v_s = 417^{\#/ft}$ - Type 11

$T=C = 3750^{\#}$
$DL = 5 min$ ∴ CMST14 or HDU5 Hdr

Drag $v_D = 225^{\#}$

$F_T = 2250^{\#}$ MST140

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Wall 4 $\theta_s = 21^\circ$

$V_w = 150(12.5) = 1875^{\#}$

$V_s = 22.3(335) = 7470^{\#}$

$V_s = 356^{\#/ft}$ - Type 11

$T=C = 3202^{\#}$

DL = 5 min - Use CMST14 on HOLES

Wall 5 $\theta_s = 11^\circ$

$V_w = 1875^{\#}$

$V_s = 7470^{\#}$

$V_s = 680^{\#/ft}$ - Type - 14

$T=C = 6111^{\#}$

DL = 5 min Use CMST14 on HOLE