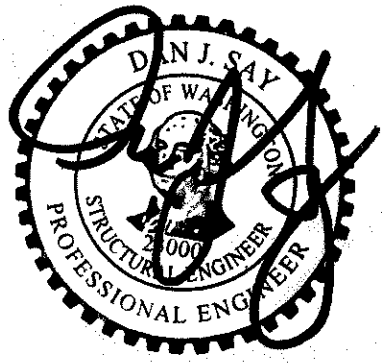
Structural Calculations For:

LBH Residence Site walls & North Elevation Revisions

7450 North Mercer Way
Mercer Island, WA

Architect: **Stillwell Hanson Architects**
46 Etruria Street, Suite 200
Seattle, WA 98109

November 4, 2019



Project # 00834-2018-08

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Criteria Sheet

Codes:

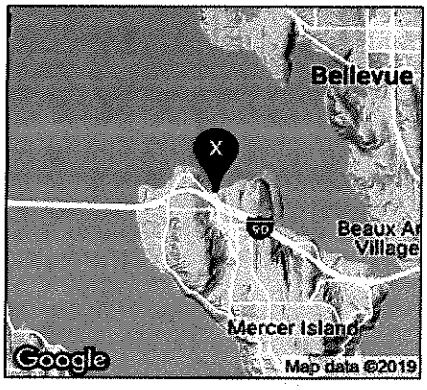
Structural:	IBC 2015
Loading:	ASCE 7-10
Wood:	NDS 2015
Steel:	AISC 360-10
Concrete:	ACI 318-14
Masonry:	TMS 402/602-13

Project Location:

Street & Number:	7450 North Mercer Way
City:	mercer islanc State: WA
ZIP:	98040
Latitude:	47.5919 N
Longitude:	-122.2383 W

Occupancy Category

Risk Category: II ASCE 7 Table 1.5-1



Seismic Load Summary:

Analysis Procedure: Equivalent Lateral Force Procedure

Lateral System: Timber Frames

R:	1.50	C_d=	1.5
Base Shear V =	0 kips	Ω_o=	1.5
S_s=	1.364	S_1=	0.525
S_{DS}=	0.91	S_{D1}=	0.53
C_s=	0.606	I_E=	1.0

Wind Load Summary:

V=	110	K_{ZT}=	1.00
Exposure =	D		

Dead Loads:

Roof

1/4" Glass	4 psf
Roof Framing	5 psf
Misc.	4
	13 psf
Use	15 psf

Floor

2" Conc. Paver	25 psf
3/4" Sheathing	2.7 psf
Joists @ 16" oc	2.2 psf
Misc./Mech.	2 psf
Ceiling Finish	2.8
	34.7 psf
Use	36 psf

FLAT ROOF ADD 5 PSF STANDING WATER LOAD

Live Loads:

Snow 25 psf

Floor 40 psf

60 PSF @ DECK

Soils:

Allowable Bearing 2000 psf

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Seismic Design

ASCE 7-10 Seismic Analysis Equivalent Lateral Force Procedure

Seismic Force Resisting System: Per Table 12.2-1	System:	Cantilevered Column Systems
	Type:	Timber Frames

Seismic Design Cat.	D	
Risk Category	II	I, II, or III, or IV per Table 1.5-1
Site Class	D	per soils report (D assumed, without soils report)
Diaphragm Flexibility	Semi-Rigid/Rigid	

Ω_o	1.5	
S_S	1.364 g	2% in 50 yr, Latitude & Longitude lookup
S_1	0.525 g	2% in 50 yr, Latitude & Longitude lookup
h_n	55.0 ft	
R	1.50	
I_e	1.0	Table 1.5-2
C_e	1.5	
Ct	0.02	Table 12.8-2
x	0.75	Table 12.8-2
T	0.40 sec	Eq. 12.8-7
T_0	0.12 sec	
T_S	0.58 sec	
k	1.000	
Fa	1.00	Table 11.4-1
Fv	1.50	Table 11.4-2
S_{MS}	1.36 g	Eq. 11.4-1
S_{M1}	0.79 g	Eq. 11.4-2
S_{DS}	0.909 g	Eq. 11.4-3
S_{D1}	0.525 g	Eq. 11.4-4
Cs	***0.606 Controls***	Eq. 12.8-2
	0.866	Eq. 12.8-3 need not exceed, $T < T_L$
	0.010	Eq. 12.8-5 or 12.8-6 minimum
Cs, design	0.606	Eq. 12.8-1, Strength Level Base Shear
Cs, ASD	0.424	Eq. 12.8-1 ASD Base Shear

$$T_a = C_t h_n^x \quad \text{Eq. 12.8.7}$$

$$S_{MS} = F_a S_S \quad \text{Eq. 11.4-1}$$
$$S_{M1} = F_v S_1 \quad \text{Eq. 11.4-2}$$
$$S_{DS} = {}^2/_3\, S_{MS} \quad \text{Eq. 11.4-3}$$
$$S_{D1} = {}^2/_3\, S_{M1} \quad \text{Eq. 11.4-4}$$

$$C_S = \frac{S_{DS}}{(R/I_e)} \quad \text{Eq. 12.8-2}$$
$$C_S = \frac{S_{D1}}{T(R/I_e)} \quad \text{Eq. 12.8-3}$$
$$C_S = \frac{S_{D1} T_L}{T^2(R/I_e)} \quad \text{Eq. 12.8-4}$$
$$C_S \geq 0.044 S_{DS} I_e \quad \text{Eq. 12.8-5}$$
$$C_S \geq 0.01 \quad \text{Eq. 12.8-5}$$

$$C_{VX} = w_x h_x^k / \textstyle\sum_{i=1}^{n} w_x h_i^k \quad \text{Eq. 12.8-12}$$

$$F_{px} = {}^{\sum_{i=x}^{n} F_i}/_{\sum_{i=x}^{n} w_i}\, w_{px} \quad \text{Eq. 12.10-1}$$
$$F_{px} \geq 0.2 S_{DS} I_e w_{px} \quad \text{Eq. 12.10-2}$$
$$F_{px} \leq 0.4 S_{DS} I_e w_{px} \quad \text{Eq. 12.10-3}$$

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Wind Design - MWFRS

ASCE 7-10 Chapter 27 - Directional Procedure

Design Method	ASD

Wind Coefficients

Exposure	D	
V=	110	mph
K_d=	0.85	Table 26.6-1
K_h=	1.17	Table 27.3-1
G=	0.85	26.9.4

Location and Building Dimensions

Calculate Kzt?	No	
Kzt	1.00	
Roof Type	Hip	
Roof Angle - Transverse Dir	14	degrees
Roof Angle - Long Dir	14	degrees
Ground to top of roof	34	ft
Bot of roof to top of roof	5	ft
Mean Roof Height, h	31.5	ft
Short Plan Dimension	60	ft
Long Plan Dimension	67	ft
Parapet ?	No	
Ground to top of parapet		ft
Average Parapet Height		ft
Ht of 2nd Level Above Grade	10	ft

Velocity Pressure at Mean Roof Height, q_h =	30.9	psf

Transverse Wind Pressures

L/B = 0.90 h/L = 0.53

Pressure Coefficients from Figure 27.4-1:

Bldg Face	C_p
Windward Wall	0.8
Leeward Wall	-0.50
Windward Roof	-0.76 / -0.18
Leeward Roof	-0.51

Wall Pressures (Unfactored): ASD

Ht	K_z	q_z	$P_{ww\ walls}$	$P_{lwwalls}$	P_{walls} (psf)
0-15	1.03	27.12	18.44	13.12	**18.93**
15-20	1.08	28.44	19.34	13.12	**19.47**
20-25	1.12	29.49	20.05	13.12	**19.90**
25-30	1.16	30.54	20.77	13.12	**20.33**
30-40	1.22	32.12	21.84	13.12	**20.98**
41-50	1.27	33.44	22.74	13.12	**21.51**
51-60	1.31	34.49	23.45	13.12	**21.94**
61-70	1.34	35.28	23.99	13.12	**22.26**
71-80	1.38	36.33	24.71	13.12	**22.69**
81-90	1.4	36.86	25.07	13.12	**22.91**
91-100	1.43	37.65	25.60	13.12	**23.23**

Roof Pressures (Unfactored) ASD

Windward		Leeward	Horiz Proj (psf)
Max	Min		
-4.7	-19.8	-13.3	**4.80**

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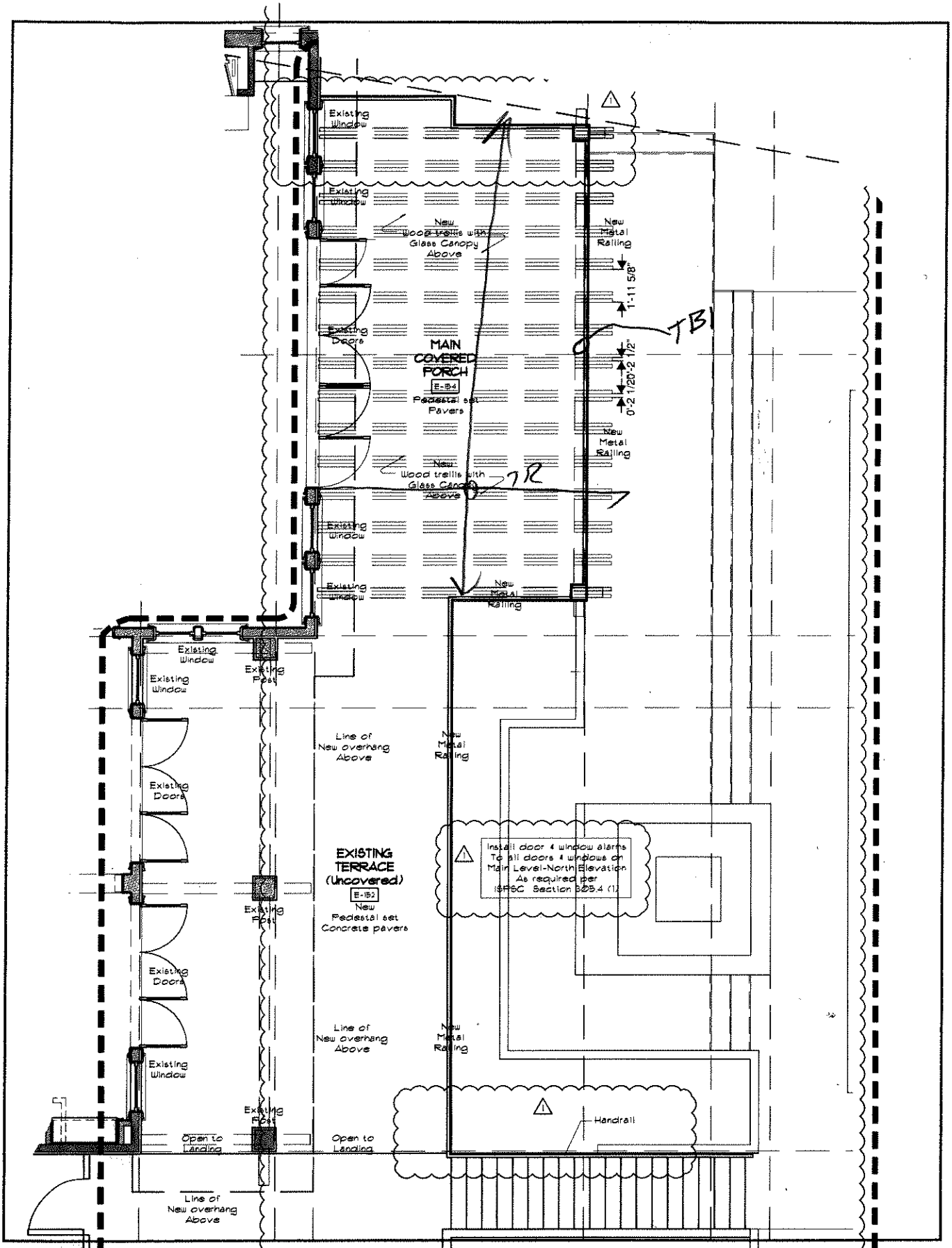

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PROJECT LBH

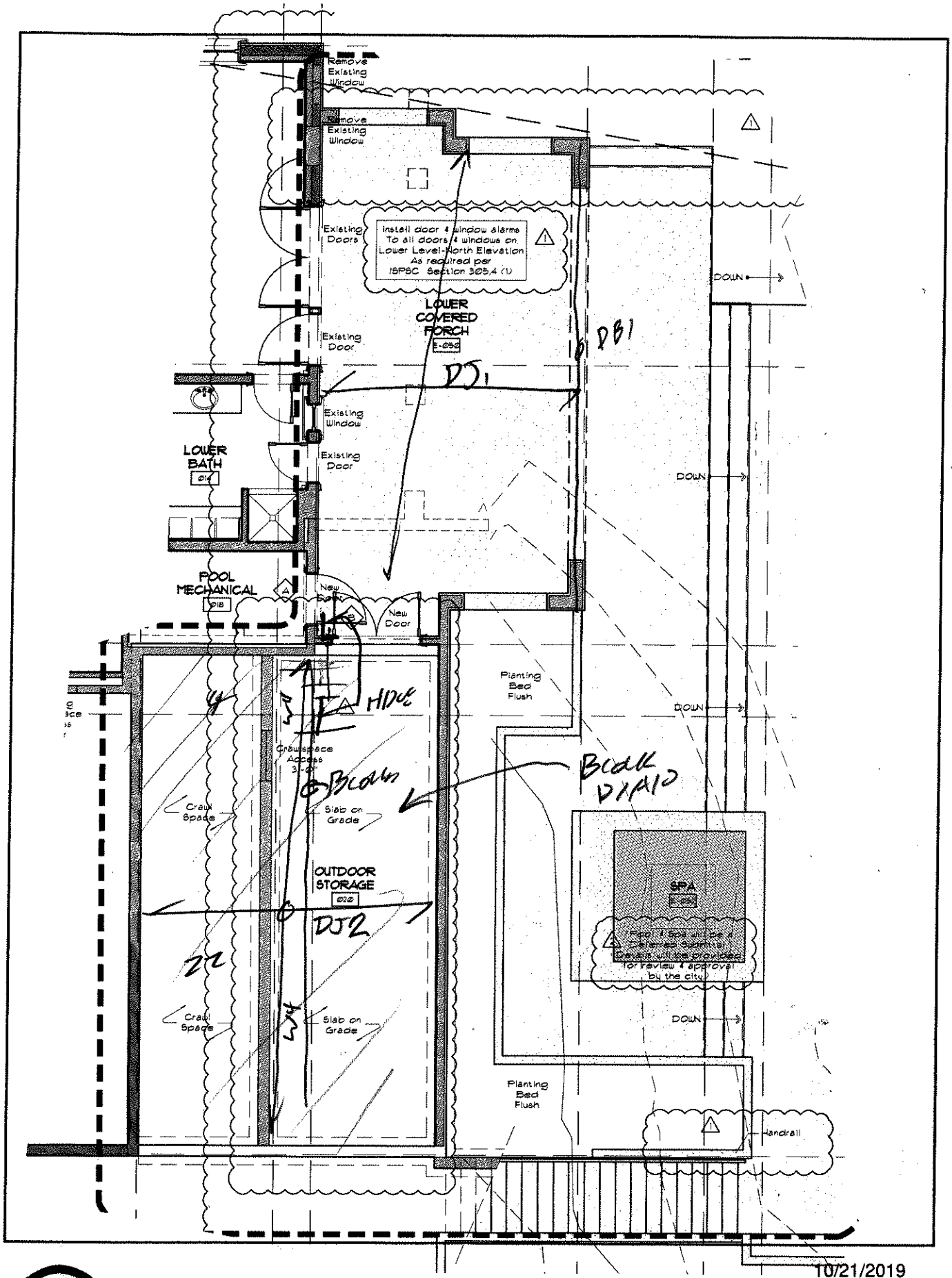
DATE 10/23/2019

00834-2018-08

PROJ. # KMR

DESIGN 5

SHEET



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STRUCTURAL ENGINEERING

PROJECT	LBH
DATE	10/21/2019
PROJ. #	00834-2018-08
DESIGN	KMR
SHEET	6

TRELLIS RAFTERS (TR)   ∴ 5PSF FLAT ROOF

w = (10+15+5) = 80#/FT   => (2) 3x8 @2' => 40#/FT/3x8

SPAN = 16.5'

V = 330 lb

M = 1362 'lb

fb = 745 psi

fv = 25 psi

Δ = .52" = L/377

BEAM TB1

W = (15+25) 10.25 = 410#/FT

SPAN = 20'

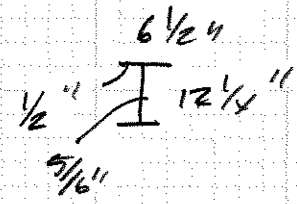
V = 5740#

M = 40180 'lb

Sx REQD = 14.6

Δ = .6" = L/557

W12x35 OK



LATERAL

W = 15 (18) 29 = 7830#   ∴ ENTIRE LOAD IN (2) POST

Vs = .424(7830) = 3320#   => 2 POSTS 1660#/POST

POST HEIGHT = 8'   M = 1660(8) = 13.3'K

P = 5740#   10x10 DF#1

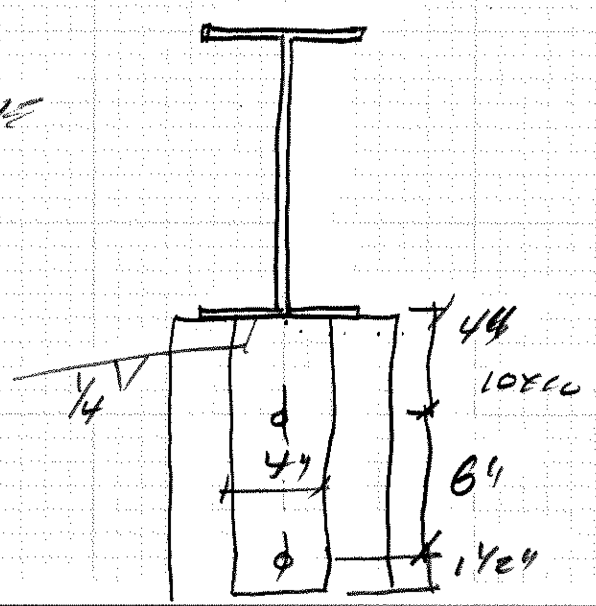
SEE SHEET 8

V@TOP = 1660#

1/2" PLATE 1660/625 = 3"

USE 1/2"x4" PLATE EACH SIDE

W/ (2) 3/4" THROUGH BOLTS



PROJECT LBH

DATE 10/21/19

PROJ. # 234.198

DESIGN Kim

Column Buckling Calculations

NDS 2015

10x10 DF #1

Column Geometry Data

6X Posts Doug Fir - Larch #1		
Hem-Fir Plates		
b	9.5	in
d	9.5	in
Le_1	8.00	ft
Le_2	8.00	ft
$le_{bending}$		ft

Bracing
No Brace
No Brace

0

Column Design Values

F_b	1200	psi
F_c	1000	psi
E'min	580	ksi
Fcperp	405	psi
cb	1.00	

Bearing Area Increase
No

Column Loading

P	6000	lbs
W_1		plf
M1	0	ft-lbs
W_2	0	plf
M2	13300	ft-lbs

Flexural Stress Adjustment Factors

Roof/EQ / Wind - C_D	1.60
Size Factor - C_F	1.00
Repetitive - C_r	1.00

Compressive Parallel Adjustment Factors

Roof/EQ / Wind - C_D	1.00
Size Factor - C_F	1.00

Other Factors

Visually Graded Lumber	
c	0.8
Solid Column	
Kf	0.6
Column: Fixed Pinned	
Ke	0.8

Column Stability Factor Calculation

Strong Axis			Weak Axis		
Fce_1	7295	psi	Fce_2	7295	psi
Fc^*_1	1000	psi	Fc^*_2	1000	psi
Fce_1/Fc^*_1	7.295		Fce_2/Fc^*_2	7.295	
Cp_1	0.582		Cp_2	0.349	

Beam Stability Factor Calculation

Strong Axis			Weak Axis		
Fbe_1	35744	psi	Fbe_2	38,264	psi
Fb'_1	1920	psi	Fb'_2	1920	psi
Fbe_1/Fb'_1	18.6		Fbe_2/Fb'_2	20	
le	15.4	ft			
CL_1	1.00				

Adjusted Allowable Stresses

Strong Axis			Weak Axis		
Fc'_1	582	psi	Fc'_2	349	psi
Fb'_1	1920	psi	Fb'_2	1920	psi

Imposed Column Stresses

Strong Axis			Weak Axis		
fc_1	66	psi	fc_2	66	psi
fb_1	0	psi	fb_2	1117	psi

Perpendicular to Grain Stress Check fcp/Fcp =	***66 / 405***	**OK**
Slenderness Check le/d	**10**	**OK**
Slenderness Check le/b	**10**	**OK**

(1) $$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{b1}}{F_{b1}'\left[1 - f_c/F_{cE}\right]} + \frac{f_{b2}}{F_{b2}'\left[1 - f_c/F_{cE2} - \left(f_{b1}/F_{bE}\right)\right]} \leq 1.0$$

(2) $$\frac{fc}{F_{cE2}} + \left(\frac{f_{b1}}{F_{bE}}\right)^2 < 1.0$$

(3) $$\frac{f_c}{F_{c'}}, \frac{f_{b1}}{F_{b1}'}, \frac{f_{b2}}{F_{b2}'} < 1.0$$

Allowable Stress Interaction Formula	**0.62**	**OK**

A = .507" = 4/376 OK

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Project: LBH Cant. Column

Date: 10/21/2019

Project #: 00834-2018-8

Design: KMR

Sheet: 8

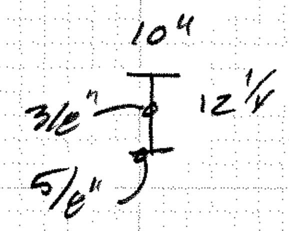
JOIST DJ1

W = (36+60) = 96#/FT
SPAN = 16'

V = 768#
M = 3072'#
fb = 1166 psi
fv = 60 psi
Δ = .15" = L/386

2x12 DF#1 @ 12" oc

DB1



W = 768 #/FT
SPAN = 23'

V = 8832#
M = 50784 'lb
Sx REQD = 12 in⁴
Δ = .35" = L/786

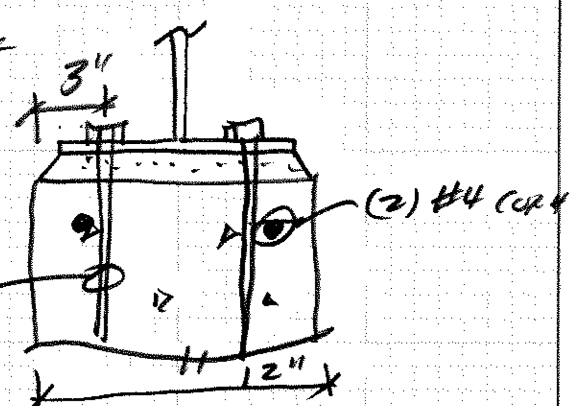
W 12x58

MOMENT FROM COLUMN = 133(2) = 266'K (Ω 1.5)/(1.5) = 40'K
50.78+40 = 90.78
Sx REQD = 33.778 OK

CONNECTION TO CONC.
V = 3320(1.5)(1/2)(1.43) = 3600#/WALL

~~12" WALL x 3'~~
3600/6 = 600#/BOLT

~~(6)~~ 5/8"Ø x 7"
A.B. W/ EC
SPACING



CHECK W/ PALLETS SEISMIC
V = 3600 + [36(8)30] .424 (1.3)(1.3/5) = 5030#
.8 CONC WALLS .165
5030/12 = 420#/BOLT OK
5030/(2.67)22 = 94#/FT CONC WALL OK.

PROJECT LIBH
DATE 10/21/19
PROJ. # 834-1018
DESIGN JMR
SHEET 9

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POST TO BEAM

M = 13.3'k = 160"k
Sx REQD = 4.8 1/2"x6" PLATE EACH SIDE = Sx = 3(2) = 6 OK

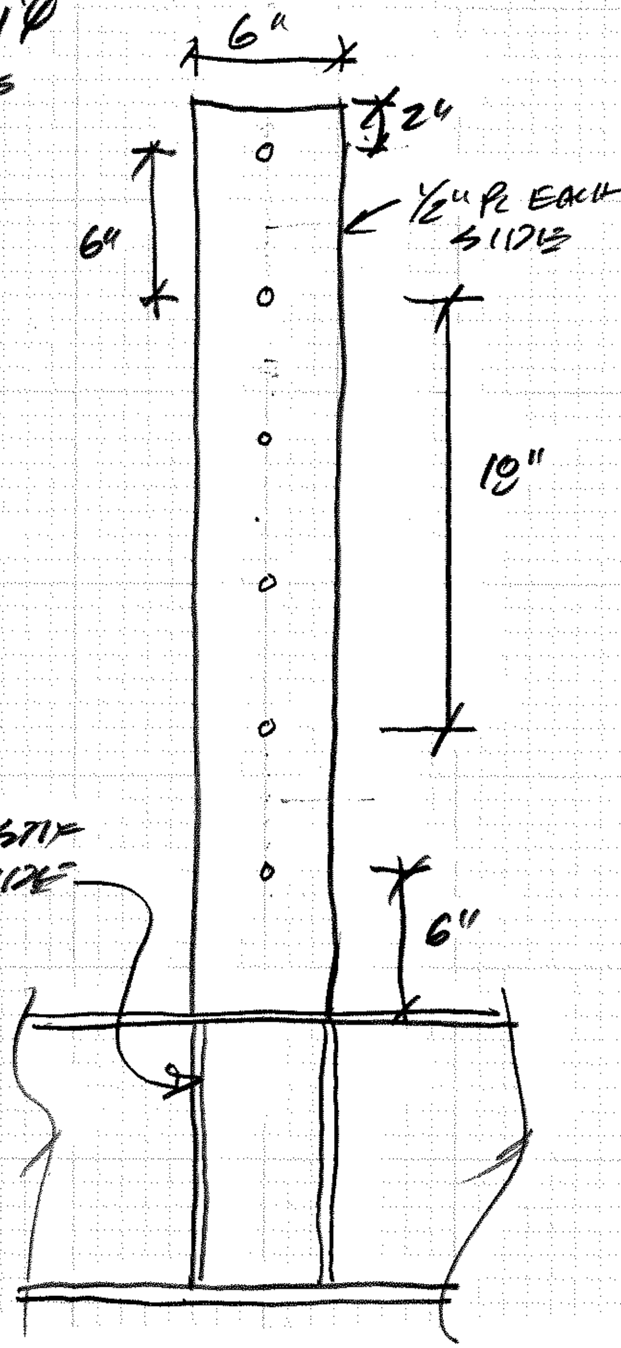
3/4"Ø BOLT DBL SHEAR = 1890# (1.6) = 3024#

13.3/2(3.02) = 2.2'
13.3/4(3.02) = 1.1 = 13.2"
(13.3/1.5)/3.02 = (3) ∴ USE 6 BOLTS @6"

(13.3/1.5) 13 = 2.96k/BOLT, OK

GLOBAL O.T. ON W12x58

40'k/23 = 17.39k
DL = 36(8)(23/2)(.9) = 2980#
∴ O.T. NOT CRIT



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DJ2

W=(36+60) 16/12 = 128#/ft
SPAN = 10.5'
V= 672#
M= 1764'#
fb= 670psi
fv= 56psi
Δ= .16" = L/832

2x12@16"oc

DRAG LOAD FROM (E) BLD INTO NEW CRIPPLE WALL

Vs= 361 (11) = 3970#
Vw= 564 (11) = 6205#

SEISMIC FROM PANEL LOAD = 250(.165) 36 = 1485# = Vs UPPER
570(.165) 36 = 3385# Vs LOWER

DRAG LOAD Vs = 3970+1485 = 5455# ∠ ASSUM ALL LOAD
5455/1.3 = 4196# CMST14, & HD08

30" END LENGTH ∴ (3) BAYS w/ HD08

DIAPH CAPACITY UNBLOCKED = 360/2 = 180# 8d@6&12
BLOCKED 800/2 = 400 8d@4&12

5455/400 = 14'

CRIPPLE WALL SHEAR
5455 + 3385/2 = 7147#/(22+4) = 275#/ft w4.
UPLIFT = [275(4)(4) - .9(10(4) + 36(9.3)) 4/2]/4 = 424#
O.T. NOT CRIT.

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PROJ. # 834-19-19
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