PROJECT NUMBER: **1939-003-191**

DATE: **January 2, 2020**

STRUCTURAL CALCULATIONS
for
CUSTOM RESIDENCE
at
Athens Lot 4
Henderson Nevada
Design is per the 2018 IBC
for
Assured Realty
January 2, 2020
Project #: 1939-003-191

01/02/2020

BY: _____

KENT A. BARBER, P.E., S.E.
SR. VICE PRESIDENT

FOR: **L. R. NELSON CONSULTING ENGINEERS, LLC**

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L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191
DATE 12/6/2019

PROJECT: CUSTOM RESIDENCE **SHEET** **OF**
SUBJECT: DESIGN CRITERIA **DESIGNED** KB **CHECKED** KB

1 Structural Design is based on the International Building Code.

2 Basic Wind and Seismic Load Design Criteria: Risk Category II

a. Wind Speed (3 second Gust) = 100 MPH Exposure = C Gcpi= +/-0.18
b. Seismic Design Category = C
Seismic Factors: Ss = 49% S1 = 16% R = 6.5
Equivalent Lateral Force Sds = 42% Sd1 = 16% Ie= 1.00
Seismic base shear, E = Cs x W Site Class = C Cs = 0.06

3 Roof Trusses to be designed by truss manufacturer. **Truss calculations and truss layout must be provided to the engineer of record prior to construction for review and approval.**

4 Lumber:
a. 2x framing shall be Douglas Fir #2 grade, unless noted otherwise.
b. 4x framing shall be Douglas Fir #2 grade, unless noted otherwise.
c. 6x framing shall be Douglas Fir #1 grade, unless noted otherwise.

5 Glue-Laminated Beams: 24F-V4 DF/DF
(24F-V8 DF/DF at cantilever and continuous beams)
Glue-Laminated Beams: 24F-V4 DF/DF
(24F-V8 DF/DF at cantilever and continuous beams)

6 Wood Structural Panels: Plywood or Oriented Strand Board (O.S.B)

7 Anchor Bolts: ASTM F1554-36

8 Connection Hardware Simpson Strong Tie or ICC approved Equal

9 Concrete: Minimum compressive strength of 2500 psi @ 28 days
Cement for all concrete shall be Type

10 Reinforcing Steel: ASTM A615, Grade 60 or ASTM A706, Grade 60 (welded reinforcing)

11 Foundations are designed for 2500 psf allowable soil bearing pressure.

12 Floor Trusses to be designed by truss manufacturer.
Truss calculations and truss layout must be provided to the engineer of record prior to construction for review and approval.

13 Shear wall materials, nailing, etc, as per attached Shear Wall Schedule.

14 Unless noted otherwise, all rolled steel shapes and plates shall be per ASTM A36 except W shapes shall be ASTM A992.

15 All steel pipe columns shall be per ASTM A53 Grade B. All HSS shapes shall be ASTM A500 Grade C.

16 Welding shall be with E70XX low hydrogen electrodes in an approved fabricating facility.
All field welds shall have constant special inspection.

17 Inspection: As required by governing municipality.

18 Special Inspection: As required per Section 1705 of the IBC.

19 L. R. Nelson Consulting Engineers assumes no liability for the architectural aspects of the provided plans (prepared by others), such as; dimensions, elevations, or sections. All architectural aspects are to be verified by the general contractor.

NEVADA - 6765 West Russell Road, Suite 200 - Las Vegas, NV 89118 - (702) 798-7978 - Fax (702) 451-2296
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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE 12/6/2019

PROJECT: CUSTOM RESIDENCE **SHEET** **OF**
SUBJECT: BUILDING LOADS **DESIGNED** KB **CHECKED** KB

ROOF

BUILT UP ROOFING	5.00
15/32" SHEATHING	1.60
ROOF FRAMING/MFR TRUSSES	3.00
INSULATION	1.50
1/2" GYPSUM	2.20
M, E & MISC.	1.70
DL	15.00
LL	20.00
SEISMIC SNOW 0.00 SNOW	

ALTERNATE LOAD #1

Load Type

M,E & MISC.
DL 0.00
Description of Load

2ND FLOOR (WHERE OCCURS)

FLOOR COVERING - CARPET, VINYL, ET	2.00
1 1/8" SHEATHING	4.00
MF'G FLR TRUSSES /FRAMING	2.80
2X4 @ 16" OC FRAMING	1.41
INSULATION (R30 BATT)	2.70
5/8" GYPSUM	2.75
M, E & MISC.	1.34
DL	17.00
LL	40.00

ALTERNATE LOAD #2

Load Type

M,E & MISC.
DL 0.00
Description of Load

EXTERIOR WALLS

1 COAT STUCCO	3.50
2X4 @ 16" OC FRAMING	1.41
WALL INSULATION	1.00
15/32" SHEATHING	1.60
1/2" GYPSUM	2.20
MISC.	2.29
DL	12.00

OVERFILL (WHERE OCCURS)

15/32" SHEATHING	1.60
OVERFILL RAFTERS	3.40
DL	5.00

ALT. FLOOR LIVE LOAD

LL

BASE DESIGN VALUES

	DFL (STUD)	DFL #2	DFL #1 (TIMBERS)	24F-V4	24F-V8	LSL (1.5E)	LVL	PSL	
F'_B	700	900	1,350	2,400	2,400	2,250	2,600	2,900	PSI
F'_V	180	180	170	240	240	400	285	290	PSI
F'_C	850	1350	925	1650	1650	1,950	2,510	2,500	PSI
$F'_{C(PERP)}$	625	625	625	650	650	775	750	750	PSI
F'_t	450	575	675	1100	1100	1950	2600	2900	PSI
E	1,400,000	1,600,000	1,600,000	1,800,000	1,800,000	1,500,000	1,900,000	2,000,000	PSI

DEFLECTION CRITERIA: $\Delta_{TLallow}$= L/ 240
$\Delta_{LLallow}$= L/ 360

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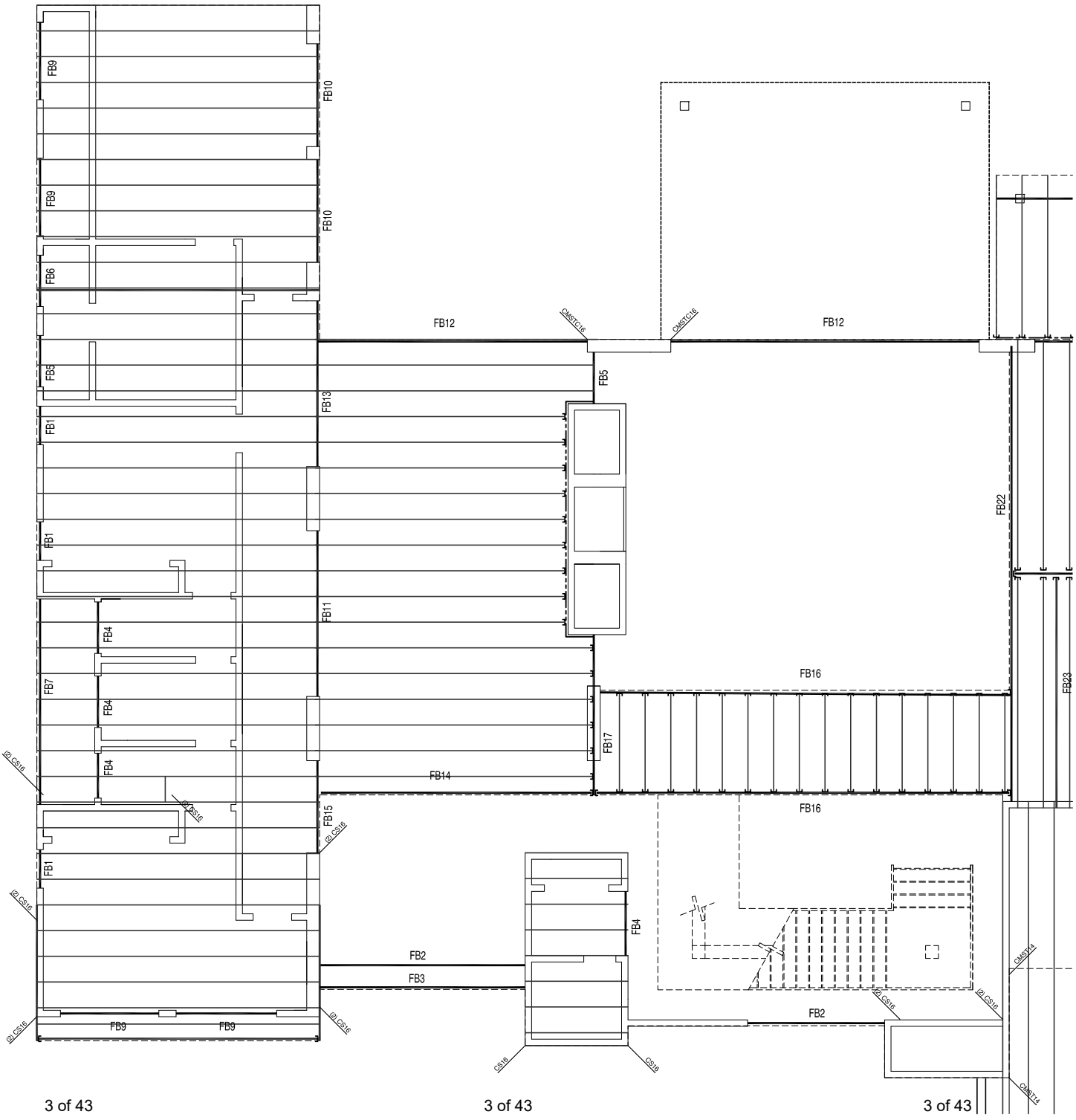
L. R. NELSON CONSULTING ENGINEERS INC.

JOB NO. 1939-003-191

DATE 12/8/19

PROJECT ATHENS LOT 4 SHEET ____ OF ____

SUBJECT FRAMING KEY PLAN DESIGNED KAB CHECKED ____



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L. R. NELSON CONSULTING ENGINEERS INC.

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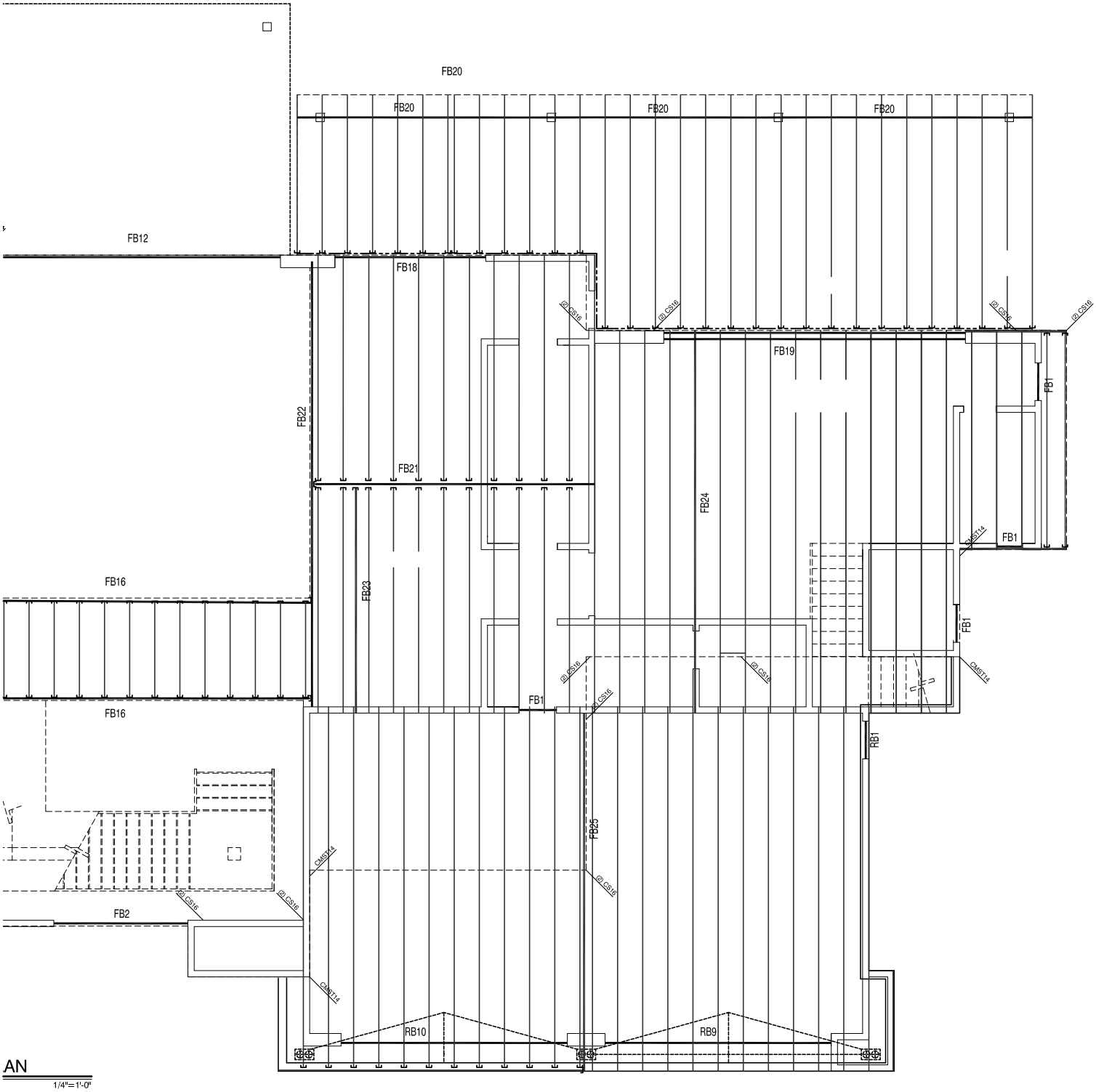
PROJECT ATHENS LOT 4 SHEET ____ OF ____

SUBJECT FRAMING KEY PLAN DESIGNED KAB CHECKED ____



AN

1/4"=1'-0"

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L. R. NELSON CONSULTING ENGINEERS INC.

JOB NO. 1939-003-191

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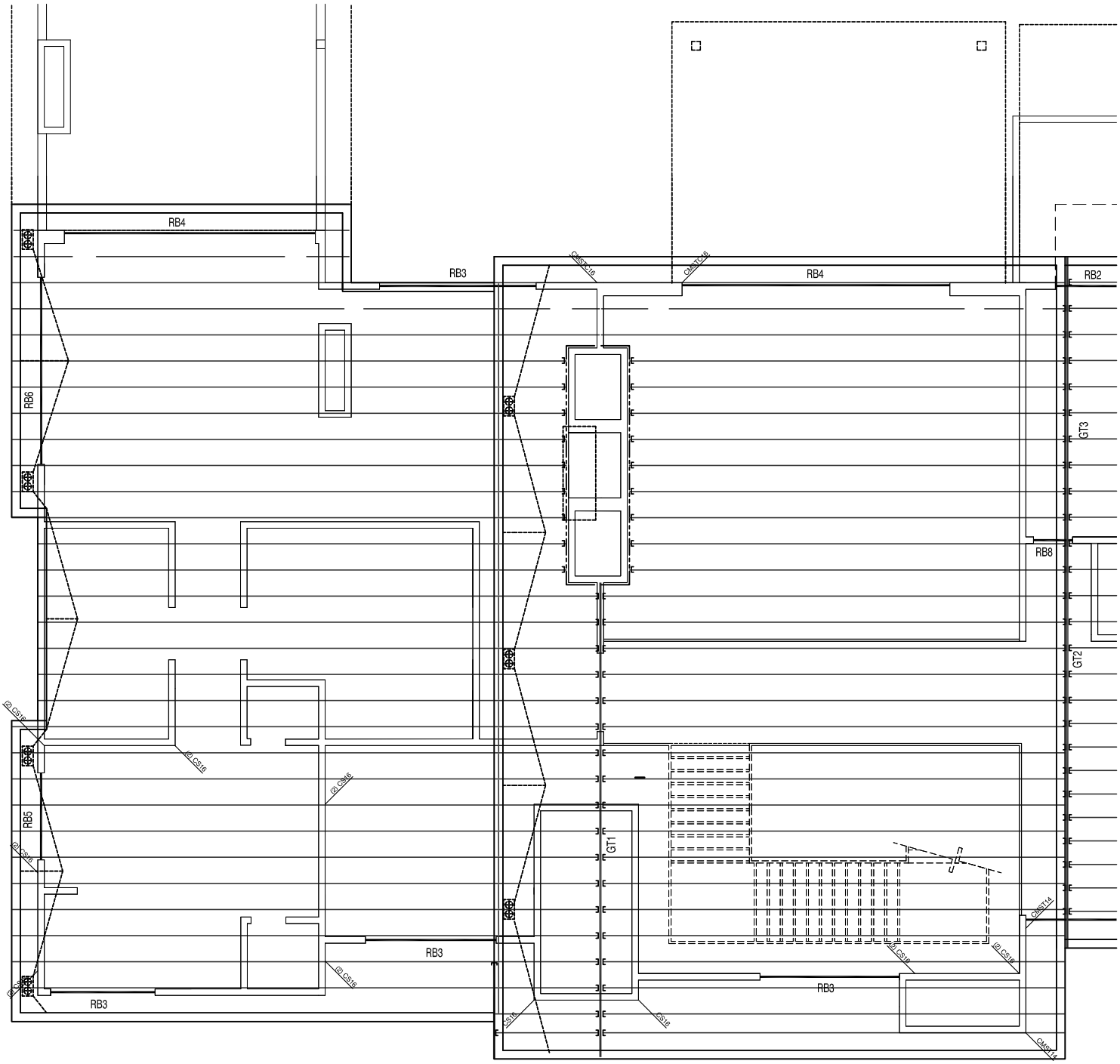
PROJECT ATHENS LOT 4 SHEET ____ OF ____

SUBJECT FRAMING KEY PLAN DESIGNED KAB CHECKED ____



ROOF FRAMING PLAN

1/4"=1'-0"

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L. R. NELSON CONSULTING ENGINEERS INC.

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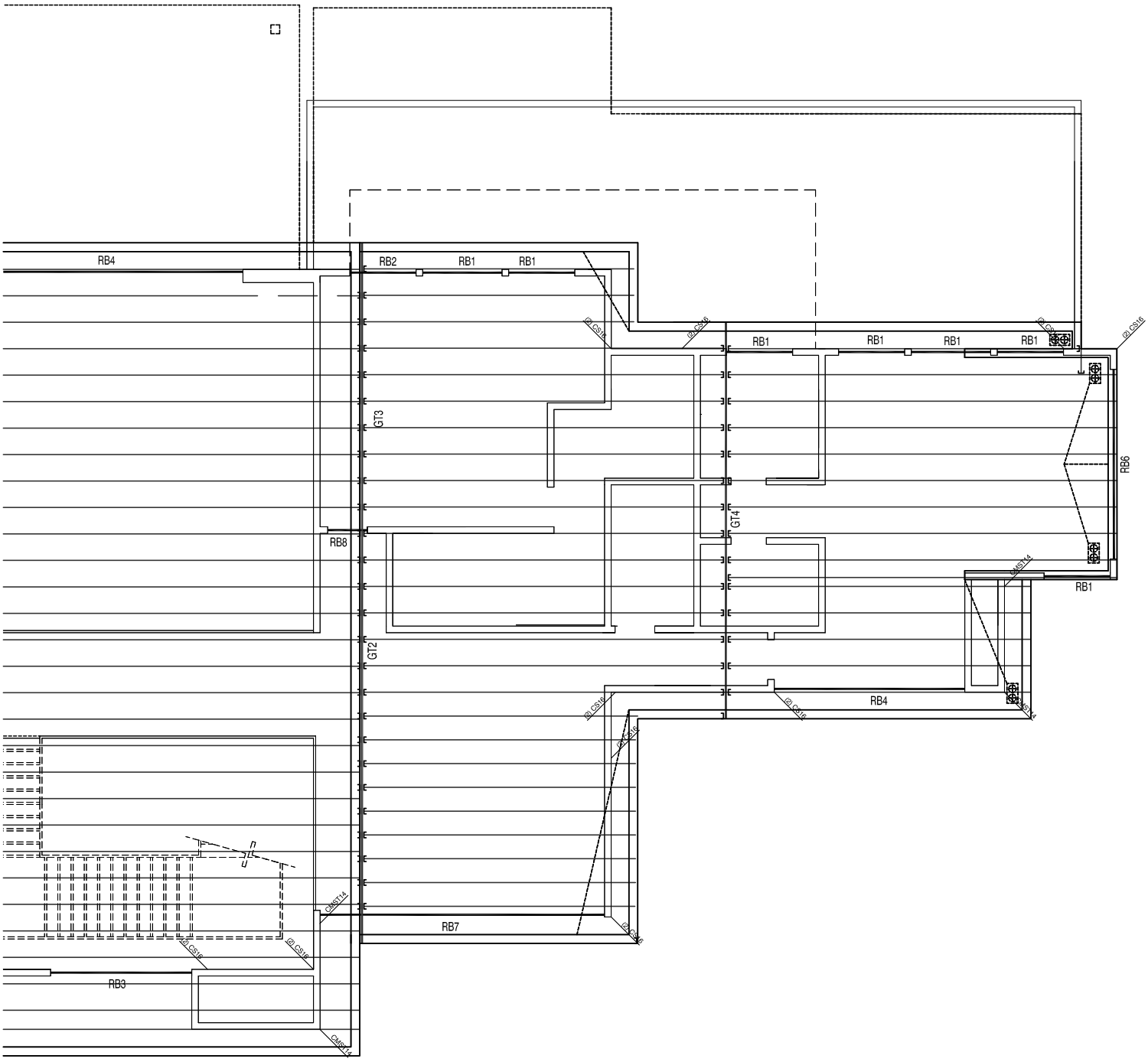
PROJECT ATHENS LOT 4 SHEET ____ OF ____

SUBJECT FRAMING KEY PLAN DESIGNED KAB CHECKED ____



1/4"= 1'-0"

6 of 43
6 of 43

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191

DATE 12/6/2019

PROJECT: CUSTOM RESIDENCE **SHEET** **OF**

SUBJECT: Beams & Headers **DESIGNED** KB **CHECKED** KB

GT1

DESCRIPTION: GT1 | Roof Tributary = 40.00 ft | W_{DL} = 600.00 lb/ft | C_D = 1.00 | $C_{V/L}$ = #N/A
SIZE: | Floor Tributary = 0.00 ft | W_{LL} = 480.00 lb/ft | C_t = 1.00 | C_F = #N/A
GRADE: | Wall Tributary = 0.00 ft | P_{DL} = 0.00 lb | I_u = 2.00
LENGTH (ft)= 36.00 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) 60 %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

	Actual			Req'd			Source:	L/xxx	Alternate Loads #1	#2	LL
S =	#N/A	in^3	#N/A	#N/A	in^3	#N/A	Max LL Deflection = #VALUE!	360			
A =	#N/A	in^2	#N/A	#N/A	in^2	#N/A	Max TL Deflection = #VALUE!	240			
I =	#N/A	in^4	#N/A	#VALUE!	in^4	#N/A	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	19,440	lb	Post:	6X8			DFL#2 (TIMBERS) O.K.		10.0	2.0	ft
Right Reaction =	19,440	lb	Post:	6X8			DFL#2 (TIMBERS) O.K.		2.0	10.0	ft

GT2

DESCRIPTION: GT2 | Roof Tributary = 32.00 ft | W_{DL} = 480.00 lb/ft | C_D = 1.25 | $C_{V/L}$ = #N/A
SIZE: | Floor Tributary = 0.00 ft | W_{LL} = 384.00 lb/ft | C_t = 1.00 | C_F = #N/A
GRADE: | Wall Tributary = 0.00 ft | P_{DL} = 0.00 lb | I_u = 2.00
LENGTH (ft)= 30.00 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) 60 %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

	Actual			Req'd			Source:	L/xxx	Alternate #1	#2	LL
S =	#N/A	in^3	#N/A	#N/A	in^3	#N/A	Max LL Deflection = #VALUE!	360			
A =	#N/A	in^2	#N/A	#N/A	in^2	#N/A	Max TL Deflection = #VALUE!	240			
I =	#N/A	in^4	#N/A	#VALUE!	in^4	#N/A	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	12,960	lb	Post:	6X6			DFL#2 (TIMBERS) O.K.		3.0	2.0	ft
Right Reaction =	12,960	lb	Post:	6X6			DFL#2 (TIMBERS) O.K.		3.0	2.0	ft

GT3

DESCRIPTION: GT3 | Roof Tributary = 32.00 ft | W_{DL} = 480.00 lb/ft | C_D = 1.25 | $C_{V/L}$ = #N/A
SIZE: | Floor Tributary = 0.00 ft | W_{LL} = 384.00 lb/ft | C_t = 1.00 | C_F = #N/A
GRADE: | Wall Tributary = 0.00 ft | P_{DL} = 0.00 lb | I_u = 2.00
LENGTH (ft)= 21.80 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) 60 %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

	Actual			Req'd			Source:	L/xxx	Alternate #1	#2	LL
S =	#N/A	in^3	#N/A	#N/A	in^3	#N/A	Max LL Deflection = #VALUE!	360			
A =	#N/A	in^2	#N/A	#N/A	in^2	#N/A	Max TL Deflection = #VALUE!	240			
I =	#N/A	in^4	#N/A	#VALUE!	in^4	#N/A	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	22,378	lb	Post:	6X10			DFL#2 (TIMBERS) O.K. 12,960		3.0	3.0	ft
Right Reaction =	9,418	lb	Post:	6X6			DFL#2 (TIMBERS) O.K.		3.0	2.0	ft

GT4

DESCRIPTION: GT4 | Roof Tributary = 29.00 ft | W_{DL} = 435.00 lb/ft | C_D = 1.25 | $C_{V/L}$ = #N/A
SIZE: | Floor Tributary = 0.00 ft | W_{LL} = 580.00 lb/ft | C_t = 1.00 | C_F = #N/A
GRADE: | Wall Tributary = 0.00 ft | P_{DL} = 0.00 lb | I_u = 2.00
LENGTH (ft)= 30.00 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.50 ft | Reduction (L) %

	Actual			Req'd			Source:	L/xxx	Alternate #1	#2	LL
S =	#N/A	in^3	#N/A	#N/A	in^3	#N/A	Max LL Deflection = #VALUE!	360			
A =	#N/A	in^2	#N/A	#N/A	in^2	#N/A	Max TL Deflection = #VALUE!	240			
I =	#N/A	in^4	#N/A	#VALUE!	in^4	#N/A	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	15,225	lb	Post:	6X6			DFL#2 (TIMBERS) O.K.		10.0	2.0	ft
Right Reaction =	15,225	lb	Post:	6X6			DFL#2 (TIMBERS) O.K.		10.0	2.0	ft

DESCRIPTION: | Roof Tributary = 0.00 ft | W_{DL} = 0.00 lb/ft | C_D = 1.00 | $C_{V/L}$ = 1.00
SIZE: 4X6 | Floor Tributary = 0.00 ft | W_{LL} = 0.00 lb/ft | C_t = 1.00 | C_F = 1.30
GRADE: DFL#2 | Wall Tributary = 0.00 ft | P_{DL} = 0.00 lb | I_u = 1.00
LENGTH (ft)= 1.00 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.50 ft | Reduction (L) %

	Actual			Req'd			Source:	L/xxx	Alternate #1	#2	LL
S =	17.65	in^3	>	0.00	in^3	O.K.	Max LL Deflection = 0.00 in = N/A	360			
A =	19.25	in^2	>	0.00	in^2	O.K.	Max TL Deflection = 0.00 in = N/A	240			
I =	48.53	in^4	>	0.00	in^4	O.K.	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	0	lb	Post:	2X4			DFLSTUD O.K.		8.0	2.0	ft
Right Reaction =	0	lb	Post:	2X4			DFLSTUD O.K.		8.0	2.0	ft

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191

DATE 12/6/2019

PROJECT: CUSTOM RESIDENCE **SHEET** **OF**

SUBJECT: Beams & Headers **DESIGNED** KB **CHECKED** KB

DESCRIPTION:	RB1	Roof Tributary =	3.00	ft	W_{DL}=	81.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00													
SIZE:	6X4	Floor Tributary =	0.00	ft	W_{LL}=	60.00	lb/ft	C_t =	1.00	C_F =	1.00													
GRADE:	DFL#2	Wall Tributary =	3.00	ft	P_{DL} =	0.00	lb	I_u =	7.00															
LENGTH (ft)=	7.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%														
NOTES:					$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%														
	Actual		Req'd					Source:			L/xxx	Alternate Loads												
S =	11.23	in^3 >	8.77	in^3	O.K.	Max LL Deflection =	0.10 in =	L / 815			360	#1	#2	LL										
A =	19.25	in^2 >	3.02	in^2	O.K.	Max TL Deflection =	0.24 in =	L / 347			240													
I =	19.65	in^4 >	13.60	in^4	O.K.			Addt'l Load to Post			Source			Lx	Ly									
Left Reaction =	494	lb	Post:	2X6		DFLSTUD	O.K.			lb				9.0	2.0	ft								
Right Reaction =	15,719	lb	Post:	6X6		DFL#2 (TIMBERS)	O.K.	15225		lb	GT4			2.0	9.0	ft								

DESCRIPTION:	RB2	Roof Tributary =	3.00	ft	W_{DL}=	81.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	0.99													
SIZE:	5-1/8X12	Floor Tributary =	0.00	ft	W_{LL}=	60.00	lb/ft	C_t =	1.00	C_F =	1.00													
GRADE:	24F-V4	Wall Tributary =	3.00	ft	P_{DL} =	5,160.00	lb	I_u =	5.00															
LENGTH (ft)=	5.00	Additional Load =	0.00	plf	P_{LL} =	4,128.00	lb	Reduction (L_r)		%														
NOTES:					$d_{Point\ Load}$ =	0.80	ft	Reduction (L)		%														
	Actual		Req'd					Source:			L/xxx	Alternate												
S =	123.00	in^3 >	26.14	in^3	O.K.	Max LL Deflection =	0.01 in =	L / 8278			360	#1	#2	LL										
A =	61.50	in^2 >	40.07	in^2	O.K.	Max TL Deflection =	0.02 in =	L / 3665			240													
I =	738.00	in^4 >	48.33	in^4	O.K.			Addt'l Load to Post			Source			Lx	Ly									
Left Reaction =	8,154	lb	Post:	(2)2X6		DFLSTUD	O.K.			lb				9.0	2.0	ft								
Right Reaction =	1,839	lb	Post:	2X6		DFLSTUD	O.K.			lb				9.0	2.0	ft								

DESCRIPTION:	RB3	Roof Tributary =	3.00	ft	W_{DL}=	81.00	lb/ft	C_D =	1.00	$C_{V/L}$ =	0.99													
SIZE:	6X8	Floor Tributary =	0.00	ft	W_{LL}=	60.00	lb/ft	C_t =	1.00	C_F =	1.00													
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	3.00	ft	P_{DL} =	0.00	lb	I_u =	12.00															
LENGTH (ft)=	12.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%														
NOTES:					$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%														
	Actual		Req'd					Source:			L/xxx	Alternate												
S =	51.56	in^3 >	22.72	in^3	O.K.	Max LL Deflection =	0.09 in =	L / 1591			360	#1	#2	LL										
A =	41.25	in^2 >	6.69	in^2	O.K.	Max TL Deflection =	0.21 in =	L / 677			240													
I =	193.36	in^4 >	68.53	in^4	O.K.			Addt'l Load to Post			Source			Lx	Ly									
Left Reaction =	846	lb	Post:	2X6		DFLSTUD	O.K.			lb				9.0	2.0	ft								
Right Reaction =	846	lb	Post:	2X6		DFLSTUD	O.K.			lb				9.0	2.0	ft								

DESCRIPTION:	RB4	Roof Tributary =	3.00	ft	W_{DL}=	81.00	lb/ft	C_D =	1.00	$C_{V/L}$ =	0.98													
SIZE:	6X10	Floor Tributary =	0.00	ft	W_{LL}=	60.00	lb/ft	C_t =	1.00	C_F =	1.00													
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	3.00	ft	P_{DL} =	0.00	lb	I_u =	20.00															
LENGTH (ft)=	20.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%														
NOTES:					$d_{Point\ Load}$ =	0.50	ft	Reduction (L)		%														
	Actual		Req'd					Source:			L/xxx	Alternate												
S =	82.73	in^3 >	63.72	in^3	O.K.	Max LL Deflection =	0.34 in =	L / 699			360	#1	#2	LL										
A =	52.25	in^2 >	11.46	in^2	O.K.	Max TL Deflection =	0.81 in =	L / 297			240													
I =	392.96	in^4 >	317.26	in^4	O.K.			Addt'l Load to Post			Source			Lx	Ly									
Left Reaction =	1,410	lb	Post:	2X6		DFLSTUD	O.K.			lb				9.0	2.0	ft								
Right Reaction =	1,410	lb	Post:	2X6		DFLSTUD	O.K.			lb				9.0	2.0	ft								

DESCRIPTION:	RB5	Roof Tributary =	23.00	ft	W_{DL}=	381.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00													
SIZE:	6X8	Floor Tributary =	0.00	ft	W_{LL}=	460.00	lb/ft	C_t =	1.00	C_F =	1.00													
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	3.00	ft	P_{DL} =	0.00	lb	I_u =	6.70															
LENGTH (ft)=	6.70	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%														
NOTES:					$d_{Point\ Load}$ =	0.50	ft	Reduction (L)		%														
	Actual		Req'd					Source:			L/xxx	Alternate												
S =	51.56	in^3 >	33.56	in^3	O.K.	Max LL Deflection =	0.07 in =	L / 1193			360	#1	#2	LL										
A =	41.25	in^2 >	16.18	in^2	O.K.	Max TL Deflection =	0.12 in =	L / 652			240													
I =	193.36	in^4 >	71.14	in^4	O.K.			Addt'l Load to Post			Source			Lx	Ly									
Left Reaction =	2,817	lb	Post:	2X6		DFLSTUD	O.K.			lb				9.0	2.0	ft								
Right Reaction =	2,817	lb	Post:	2X6		DFLSTUD	O.K.			lb				9.0	2.0	ft								

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191

DATE 12/6/2019

PROJECT: CUSTOM RESIDENCE **SHEET** **OF**

SUBJECT: Beams & Headers **DESIGNED** KB **CHECKED** KB

RB6

DESCRIPTION:	RB6	Roof Tributary =	23.00	ft	W_{DL}=	381.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00		
SIZE:	5-1/8X12	Floor Tributary =	0.00	ft	W_{LL}=	400.20	lb/ft	C_t =	1.00	C_F =	1.00		
GRADE:	24F-V4	Wall Tributary =	3.00	ft	P_{DL} =	0.00	lb	I_u =	2.00				
LENGTH (ft)=	15.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)	87	%			
NOTES:					$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%			

Actual			Req'd			Source:		L/xxx	Alternate Loads			
S =	123.00 in³	>	88.17 in³	O.K.	Max LL Deflection =	0.34 in =	L / 525	360	#1	#2	LL	
A =	61.50 in²	>	25.39 in²	O.K.	Max TL Deflection =	0.67 in =	L / 269	240				
I =	738.00 in⁴	>	659.14 in⁴	O.K.		Addt'l Load to Post		Source	Lx	Ly		
Left Reaction =	5,859 lb	Post:	(2)2X6	DFLSTUD	O.K.		lb		9.0	2.0	ft	
Right Reaction =	5,859 lb	Post:	(2)2X6	DFLSTUD	O.K.		lb		9.0	2.0	ft	

RB7

DESCRIPTION:	RB7	Roof Tributary =	2.00	ft	W_{DL}=	66.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00		
SIZE:	5-1/8X16-1/2	Floor Tributary =	0.00	ft	W_{LL}=	40.00	lb/ft	C_t =	1.00	C_F =	1.00		
GRADE:	24F-V4	Wall Tributary =	3.00	ft	P_{DL} =	7,200.00	lb	I_u =	2.00				
LENGTH (ft)=	22.00	Additional Load =	0.00	plf	P_{LL} =	5,760.00	lb	Reduction (L_r)		%			
NOTES:					$d_{Point\ Load}$ =	3.42	ft	Reduction (L)		%			

Actual			Req'd			Source:	GT2	L/xxx	Alternate			
S =	232.55 in³	>	163.45 in³	O.K.	Max LL Deflection =	0.35 in =	L / 744	360	#1	#2	LL	
A =	84.56 in²	>	59.83 in²	O.K.	Max TL Deflection =	0.82 in =	L / 321	240				
I =	1918.51 in⁴	>	1,434.60 in⁴	O.K.		Addt'l Load to Post		Source	Lx	Ly		
Left Reaction =	12,111 lb	Post:	6X6	DFL#2 (TIMBERS)	O.K.		lb		9.0	2.0	ft	
Right Reaction =	3,181 lb	Post:	2X6	DFLSTUD	O.K.		lb		9.0	2.0	ft	

RB8

DESCRIPTION:	RB8	Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	0.99		
SIZE:	(3)1-3/4X14	Floor Tributary =	0.00	ft	W_{LL}=	0.00	lb/ft	C_t =	1.00	C_F =	0.98		
GRADE:	MICROLLAM LVL (1.9E)	Wall Tributary =	0.00	ft	P_{DL} =	12,432.00	lb	I_u =	3.50				
LENGTH (ft)=	3.50	Additional Load =	0.00	plf	P_{LL} =	9,945.60	lb	Reduction (L_r)		%			
NOTES:					$d_{Point\ Load}$ =	1.00	ft	Reduction (L)		%			

Actual			Req'd			Source:	GT2+3	L/xxx	Alternate			
S =	171.50 in³	>	60.31 in³	O.K.	Max LL Deflection =	0.01 in =	L / 8060	360	#1	#2	LL	
A =	73.50 in²	>	67.30 in²	O.K.	Max TL Deflection =	0.01 in =	L / 3582	240				
I =	1200.50 in⁴	>	80.43 in⁴	O.K.		Addt'l Load to Post		Source	Lx	Ly		
Left Reaction =	15,984 lb	Post:	6X6	DFL#2 (TIMBERS)	O.K.		lb		9.0	2.0	ft	
Right Reaction =	6,394 lb	Post:	6X6	DFL#2 (TIMBERS)	O.K.		lb		9.0	2.0	ft	

RB9

DESCRIPTION:	RB9	Roof Tributary =	19.50	ft	W_{DL}=	328.50	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00		
SIZE:	5-1/8X13-1/2	Floor Tributary =	0.00	ft	W_{LL}=	390.00	lb/ft	C_t =	1.00	C_F =	1.00		
GRADE:	24F-V4	Wall Tributary =	3.00	ft	P_{DL} =	0.00	lb	I_u =	2.00				
LENGTH (ft)=	18.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%			
NOTES:					$d_{Point\ Load}$ =	0.50	ft	Reduction (L)		%			

Actual			Req'd			Source:		L/xxx	Alternate			
S =	155.67 in³	>	116.82 in³	O.K.	Max LL Deflection =	0.49 in =	L / 444	360	#1	#2	LL	
A =	69.19 in²	>	28.29 in²	O.K.	Max TL Deflection =	0.90 in =	L / 241	240				
I =	1050.79 in⁴	>	1,047.58 in⁴	O.K.		Addt'l Load to Post		Source	Lx	Ly		
Left Reaction =	6,467 lb	Post:	(2)2X6	DFLSTUD	O.K.		lb		9.0	2.0	ft	
Right Reaction =	6,467 lb	Post:	(2)2X6	DFLSTUD	O.K.		lb		9.0	2.0	ft	

RB10

DESCRIPTION:	RB10	Roof Tributary =	13.00	ft	W_{DL}=	299.01	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00		
SIZE:	5-1/8X13-1/2	Floor Tributary =	4.00	ft	W_{LL}=	420.00	lb/ft	C_t =	1.00	C_F =	1.00		
GRADE:	24F-V4	Wall Tributary =	3.00	ft	P_{DL} =	0.00	lb	I_u =	2.00				
LENGTH (ft)=	18.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%			
NOTES:					$d_{Point\ Load}$ =	0.50	ft	Reduction (L)		%			

Actual			Req'd			Source:		L/xxx	Alternate			
S =	155.67 in³	>	146.02 in³	O.K.	Max LL Deflection =	0.52 in =	L / 412	360	#1	#2	LL	
A =	69.19 in²	>	35.39 in²	O.K.	Max TL Deflection =	0.90 in =	L / 241	240				
I =	1050.79 in⁴	>	1,048.31 in⁴	O.K.		Addt'l Load to Post		Source	Lx	Ly		
Left Reaction =	6,471 lb	Post:	(2)2X6	DFLSTUD	O.K.		lb		9.0	2.0	ft	
Right Reaction =	6,471 lb	Post:	(2)2X6	DFLSTUD	O.K.		lb		9.0	2.0	ft	

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191

DATE 12/6/2019

PROJECT: CUSTOM RESIDENCE **SHEET** **OF**

SUBJECT: Beams & Headers **DESIGNED** KB **CHECKED** KB

FB1

DESCRIPTION: FB1 | Roof Tributary = 23.00 ft | W_{DL} = 688.02 lb/ft | C_D = 1.25 | $C_{V/L}$ = 1.00
SIZE: 6X6 | Floor Tributary = 11.00 ft | W_{LL} = 900.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: DFL#2 (TIMBERS) | Wall Tributary = 13.00 ft | P_{DL} = 0.00 lb | l_u = 3.25
LENGTH (ft)= 3.25 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

Actual			Req'd			Source:	L/xxx	Alternate Loads		
S = 27.73	in³	>	23.00	in³	O.K.	Max LL Deflection = 0.02 in = L / 1711	360	#1	#2	LL
A = 30.25	in²	>	13.08	in²	O.K.	Max TL Deflection = 0.04 in = L / 970	240			
I = 76.26	in⁴	>	18.87	in⁴	O.K.	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction = 2,581 lb	Post: 2X6		DFLSTUD	O.K.		lb		9.0	2.0	ft
Right Reaction = 2,581 lb	Post: 2X6		DFLSTUD	O.K.		lb		9.0	2.0	ft

FB2

DESCRIPTION: FB2 | Roof Tributary = 2.00 ft | W_{DL} = 186.01 lb/ft | C_D = 1.00 | $C_{V/L}$ = 0.97
SIZE: 5-1/8X12 | Floor Tributary = 0.00 ft | W_{LL} = 40.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: 24F-V4 | Wall Tributary = 13.00 ft | P_{DL} = 0.00 lb | l_u = 16.50
LENGTH (ft)= 16.50 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

Actual			Req'd			Source:	L/xxx	Alternate		
S = 123.00	in³	>	39.46	in³	O.K.	Max LL Deflection = 0.05 in = L / 3943	360	#1	#2	LL
A = 61.50	in²	>	10.24	in²	O.K.	Max TL Deflection = 0.28 in = L / 698	240			
I = 738.00	in⁴	>	253.82	in⁴	O.K.	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction = 1,865 lb	Post: 2X6		DFLSTUD	O.K.		lb		10.0	2.0	ft
Right Reaction = 2,715 lb	Post: 2X6		DFLSTUD	O.K.		850 lb		10.0	2.0	ft

FB3

DESCRIPTION: FB3 | Roof Tributary = 4.00 ft | W_{DL} = 216.01 lb/ft | C_D = 1.00 | $C_{V/L}$ = 0.97
SIZE: 5-1/8X12 | Floor Tributary = 0.00 ft | W_{LL} = 80.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: 24F-V4 | Wall Tributary = 13.00 ft | P_{DL} = 0.00 lb | l_u = 16.50
LENGTH (ft)= 16.50 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

Actual			Req'd			Source:	L/xxx	Alternate		
S = 123.00	in³	>	51.69	in³	O.K.	Max LL Deflection = 0.10 in = L / 1971	360	#1	#2	LL
A = 61.50	in²	>	13.41	in²	O.K.	Max TL Deflection = 0.37 in = L / 533	240			
I = 738.00	in⁴	>	332.43	in⁴	O.K.	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction = 2,442 lb	Post: 2X6		DFLSTUD	O.K.		lb		10.0	2.0	ft
Right Reaction = 2,442 lb	Post: 2X6		DFLSTUD	O.K.		lb		10.0	2.0	ft

FB4

DESCRIPTION: FB4 | Roof Tributary = 0.00 ft | W_{DL} = 211.01 lb/ft | C_D = 1.00 | $C_{V/L}$ = 1.00
SIZE: 6X6 | Floor Tributary = 11.00 ft | W_{LL} = 440.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: DFL#2 (TIMBERS) | Wall Tributary = 2.00 ft | P_{DL} = 0.00 lb | l_u = 4.50
LENGTH (ft)= 4.50 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.50 ft | Reduction (L) %

Actual			Req'd			Source:	L/xxx	Alternate		
S = 27.73	in³	>	22.60	in³	O.K.	Max LL Deflection = 0.04 in = L / 1319	360	#1	#2	LL
A = 30.25	in²	>	10.29	in²	O.K.	Max TL Deflection = 0.06 in = L / 891	240			
I = 76.26	in⁴	>	20.82	in⁴	O.K.	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction = 1,465 lb	Post: 2X4		DFLSTUD	O.K.		lb		9.0	2.0	ft
Right Reaction = 1,465 lb	Post: 2X4		DFLSTUD	O.K.		lb		9.0	2.0	ft

FB5

DESCRIPTION: FB5 | Roof Tributary = 23.00 ft | W_{DL} = 688.02 lb/ft | C_D = 1.00 | $C_{V/L}$ = 1.00
SIZE: 6X8 | Floor Tributary = 11.00 ft | W_{LL} = 900.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: DFL#2 (TIMBERS) | Wall Tributary = 13.00 ft | P_{DL} = 0.00 lb | l_u = 4.25
LENGTH (ft)= 4.25 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.50 ft | Reduction (L) %

Actual			Req'd			Source:	L/xxx	Alternate		
S = 51.56	in³	>	49.17	in³	O.K.	Max LL Deflection = 0.03 in = L / 1940	360	#1	#2	LL
A = 41.25	in²	>	21.02	in²	O.K.	Max TL Deflection = 0.05 in = L / 1100	240			
I = 193.36	in⁴	>	42.20	in⁴	O.K.	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction = 3,375 lb	Post: 2X6		DFLSTUD	O.K.		lb		9.0	2.0	ft
Right Reaction = 3,375 lb	Post: 2X6		DFLSTUD	O.K.		lb		9.0	2.0	ft

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191

DATE 12/6/2019

PROJECT: CUSTOM RESIDENCE **SHEET** **OF**

SUBJECT: Beams & Headers **DESIGNED** KB **CHECKED** KB

FB6

DESCRIPTION: FB6 | Roof Trib. = 2.00 ft | W_{DL} = 373.02 lb/ft | C_D = 1.00 | $C_{V/L}$ = 1.00
SIZE: 6X10 | Floor Trib. = 11.00 ft | W_{LL} = 480.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: DFL#1 (TIMBERS) | Wall Trib. = 13.00 ft | P_{DL} = 2,460.00 lb | I_u = 4.25
LENGTH (ft)= 4.25 | Additional Load = 0.00 plf | P_{LL} = 1,040.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 1.50 ft | Reduction (L) %
Source: GT | L/xxx | Alternate Loads

Actual		Req'd									
S =	82.73 in³	>	45.78 in³	O.K.	Max LL Deflection =	0.01 in =	L / 5283	360	#1	#2	LL
A =	52.25 in²	>	30.02 in²	O.K.	Max TL Deflection =	0.02 in =	L / 2160	240			
I =	392.96 in⁴	>	43.66 in⁴	O.K.							

						Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	4,077 lb	Post:	2X6	DFLSTUD	O.K.		lb		9.0	2.0	ft
Right Reaction =	3,048 lb	Post:	2X6	DFLSTUD	O.K.		lb		9.0	2.0	ft

FB7

DESCRIPTION: FB7 | Roof Trib. = 22.00 ft | W_{DL} = 520.01 lb/ft | C_D = 1.00 | $C_{V/L}$ = 1.00
SIZE: 5-1/8X15 | Floor Trib. = 2.00 ft | W_{LL} = 520.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: 24F-V4 | Wall Trib. = 13.00 ft | P_{DL} = 0.00 lb | I_u = 2.00
LENGTH (ft)= 16.50 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %
Source: | L/xxx | Alternate

Actual		Req'd									
S =	192.19 in³	>	177.53 in³	O.K.	Max LL Deflection =	0.33 in =	L / 592	360	#1	#2	LL
A =	76.88 in²	>	45.50 in²	O.K.	Max TL Deflection =	0.67 in =	L / 296	240			
I =	1441.41 in⁴	>	1,167.97 in⁴	O.K.							

						Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	8,580 lb	Post:	(2)2X6	DFLSTUD	O.K.		lb		10.0	2.0	ft
Right Reaction =	9,430 lb	Post:	(2)2X6	DFLSTUD	O.K.	850	lb		10.0	2.0	ft

FB8

DESCRIPTION: FB8 | Roof Trib. = 0.00 ft | W_{DL} = 70.00 lb/ft | C_D = 1.00 | $C_{V/L}$ = 1.00
SIZE: 6X6 | Floor Trib. = 2.00 ft | W_{LL} = 80.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: DFL#2 (TIMBERS) | Wall Trib. = 3.00 ft | P_{DL} = 0.00 lb | I_u = 10.00
LENGTH (ft)= 10.00 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %
Source: | L/xxx | Alternate

Actual		Req'd									
S =	27.73 in³	>	25.71 in³	O.K.	Max LL Deflection =	0.18 in =	L / 661	360	#1	#2	LL
A =	30.25 in²	>	6.01 in²	O.K.	Max TL Deflection =	0.34 in =	L / 352	240			
I =	76.26 in⁴	>	51.92 in⁴	O.K.							

						Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	750 lb	Post:	2X6	DFLSTUD	O.K.		lb		10.0	2.0	ft
Right Reaction =	750 lb	Post:	2X6	DFLSTUD	O.K.		lb		10.0	2.0	ft

FB9

DESCRIPTION: FB9 | Roof Trib. = 0.00 ft | W_{DL} = 223.01 lb/ft | C_D = 1.00 | $C_{V/L}$ = 1.00
SIZE: 6X8 | Floor Trib. = 11.00 ft | W_{LL} = 440.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: DFL#2 (TIMBERS) | Wall Trib. = 3.00 ft | P_{DL} = 0.00 lb | I_u = 6.50
LENGTH (ft)= 6.50 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.50 ft | Reduction (L) %
Source: | L/xxx | Alternate

Actual		Req'd									
S =	51.56 in³	>	48.02 in³	O.K.	Max LL Deflection =	0.07 in =	L / 1109	360	#1	#2	LL
A =	41.25 in²	>	15.36 in²	O.K.	Max TL Deflection =	0.11 in =	L / 736	240			
I =	193.36 in⁴	>	63.03 in⁴	O.K.							

						Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	2,155 lb	Post:	2X6	DFLSTUD	O.K.		lb		9.0	2.0	ft
Right Reaction =	2,155 lb	Post:	2X6	DFLSTUD	O.K.		lb		9.0	2.0	ft

FB10

DESCRIPTION: FB10 | Roof Trib. = 0.00 ft | W_{DL} = 223.01 lb/ft | C_D = 1.00 | $C_{V/L}$ = 0.99
SIZE: 6X10 | Floor Trib. = 11.00 ft | W_{LL} = 440.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: DFL#2 (TIMBERS) | Wall Trib. = 3.00 ft | P_{DL} = 0.00 lb | I_u = 8.50
LENGTH (ft)= 8.50 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.50 ft | Reduction (L) %
Source: | L/xxx | Alternate

Actual		Req'd									
S =	82.73 in³	>	82.55 in³	O.K.	Max LL Deflection =	0.10 in =	L / 1008	360	#1	#2	LL
A =	52.25 in²	>	20.23 in²	O.K.	Max TL Deflection =	0.15 in =	L / 669	240			
I =	392.96 in⁴	>	140.94 in⁴	O.K.							

						Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	2,818 lb	Post:	2X6	DFLSTUD	O.K.		lb		9.0	2.0	ft
Right Reaction =	2,818 lb	Post:	2X6	DFLSTUD	O.K.		lb		9.0	2.0	ft

NEVADA - 6765 West Russell Road, Suite 200 - Las Vegas, NV 89118 - (702) 798-7978 - Fax (702) 451-2296
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L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191

DATE 12/6/2019

PROJECT: CUSTOM RESIDENCE **SHEET** **OF**

SUBJECT: Beams & Headers **DESIGNED** KB **CHECKED** KB

FB11

DESCRIPTION:	FB11	Roof Tributary =	3.00	ft	W_{DL}=	443.02	lb/ft	C_D =	1.00	$C_{V/L}$ =	0.97
SIZE:	5-1/8X15	Floor Tributary =	22.00	ft	W_{LL}=	940.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	24F-V4	Wall Tributary =	2.00	ft	P_{DL} =	0.00	lb	I_u =	13.50		
LENGTH (ft)=	13.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%	

Actual			Req'd			Source:		L/xxx	Alternate Loads		
S =	192.19 in³	>	162.14 in³	O.K.	Max LL Deflection =	0.27 in =	L / 598	360	#1	#2	LL
A =	76.88 in²	>	47.54 in²	O.K.	Max TL Deflection =	0.40 in =	L / 407	240			
I =	1441.41 in⁴	>	867.28 in⁴	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	9,335 lb	Post:	2X4		DFLSTUD	N.G.		lb		9.0	2.0 ft
Right Reaction =	9,335 lb	Post:	2X4		DFLSTUD	N.G.		lb		9.0	2.0 ft

FB12

DESCRIPTION:	FB12	Roof Tributary =	2.00	ft	W_{DL}=	220.01	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00
SIZE:	5-1/8X13-1/2	Floor Tributary =	2.00	ft	W_{LL}=	120.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	24F-V4	Wall Tributary =	13.00	ft	P_{DL} =	490.00	lb	I_u =	2.00		
LENGTH (ft)=	21.50	Additional Load =	0.00	plf	P_{LL} =	360.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point\ Load}$ =	0.25	ft	Reduction (L)		%	

Actual			Req'd			Source:	RB3	L/xxx	Alternate		
S =	155.67 in³	>	99.05 in³	O.K.	Max LL Deflection =	0.31 in =	L / 839	360	#1	#2	LL
A =	69.19 in²	>	25.70 in²	O.K.	Max TL Deflection =	0.87 in =	L / 297	240			
I =	1050.79 in⁴	>	850.27 in⁴	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	4,495 lb	Post:	2X6		DFLSTUD	O.K.		lb		10.0	2.0 ft
Right Reaction =	3,665 lb	Post:	2X6		DFLSTUD	O.K.		lb		10.0	2.0 ft

FB13

DESCRIPTION:	FB13	Roof Tributary =	0.00	ft	W_{DL}=	374.02	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00
SIZE:	5-1/8X16-1/2	Floor Tributary =	22.00	ft	W_{LL}=	880.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	24F-V4	Wall Tributary =	0.00	ft	P_{DL} =	3,705.00	lb	I_u =	2.00		
LENGTH (ft)=	13.50	Additional Load =	0.00	plf	P_{LL} =	2,310.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point\ Load}$ =	3.50	ft	Reduction (L)		%	

Actual			Req'd			Source:		L/xxx	Alternate		
S =	232.55 in³	>	201.03 in³	O.K.	Max LL Deflection =	0.23 in =	L / 697	360	#1	#2	LL
A =	84.56 in²	>	69.97 in²	O.K.	Max TL Deflection =	0.38 in =	L / 425	240			
I =	1918.51 in⁴	>	1,083.17 in⁴	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	12,920 lb	Post:	(3)2X6		DFLSTUD	O.K.		lb		10.0	2.0 ft
Right Reaction =	10,024 lb	Post:	(3)2X6		DFLSTUD	O.K.		lb		10.0	2.0 ft

FB14

DESCRIPTION:	FB14	Roof Tributary =	0.00	ft	W_{DL}=	154.01	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00
SIZE:	5-1/8X12	Floor Tributary =	2.00	ft	W_{LL}=	80.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	24F-V4	Wall Tributary =	10.00	ft	P_{DL} =	0.00	lb	I_u =	2.00		
LENGTH (ft)=	21.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point\ Load}$ =	0.50	ft	Reduction (L)		%	

Actual			Req'd			Source:		L/xxx	Alternate		
S =	123.00 in³	>	67.78 in³	O.K.	Max LL Deflection =	0.29 in =	L / 891	360	#1	#2	LL
A =	61.50 in²	>	14.26 in²	O.K.	Max TL Deflection =	0.85 in =	L / 305	240			
I =	738.00 in⁴	>	581.42 in⁴	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	2,516 lb	Post:	HGR		-			lb		9.0	2.0 ft
Right Reaction =	2,516 lb	Post:	HGR		-			lb		9.0	2.0 ft

FB15

DESCRIPTION:	FB15	Roof Tributary =	0.00	ft	W_{DL}=	307.02	lb/ft	C_D =	1.00	$C_{V/L}$ =	0.99
SIZE:	5-1/8X12	Floor Tributary =	11.00	ft	W_{LL}=	440.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	24F-V4	Wall Tributary =	10.00	ft	P_{DL} =	2,000.60	lb	I_u =	7.70		
LENGTH (ft)=	7.70	Additional Load =	0.00	plf	P_{LL} =	1,780.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point\ Load}$ =	3.00	ft	Reduction (L)		%	

Actual			Req'd			Source:	FB14	L/xxx	Alternate		
S =	123.00 in³	>	61.61 in³	O.K.	Max LL Deflection =	0.05 in =	L / 1975	360	#1	#2	LL
A =	61.50 in²	>	27.73 in²	O.K.	Max TL Deflection =	0.09 in =	L / 1048	240			
I =	738.00 in⁴	>	169.07 in⁴	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	5,184 lb	Post:	2X6		DFLSTUD	O.K.		lb		9.0	2.0 ft
Right Reaction =	4,349 lb	Post:	2X6		DFLSTUD	O.K.		lb		9.0	2.0 ft

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191

DATE 12/6/2019

PROJECT: CUSTOM RESIDENCE **SHEET** **OF**

SUBJECT: Beams & Headers **DESIGNED** KB **CHECKED** KB

FB16

DESCRIPTION: FB16 | Roof Tributary = 0.00 ft | W_{DL} = 116.01 lb/ft | C_D = 1.00 | $C_{V/L}$ = 0.92
SIZE: 5-1/8X18 | Floor Tributary = 4.00 ft | W_{LL} = 160.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: 24F-V4 | Wall Tributary = 4.00 ft | P_{DL} = 0.00 lb | I_u = 2.00
LENGTH (ft)= 32.50 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

Actual			Req'd			Source:	L/xxx	Alternate Loads #1	#2	LL
S = 276.75	in³	> 198.22	in³	O.K.	Max LL Deflection = 0.90 in = L / 435		360			
A = 92.25	in²	> 25.44	in²	O.K.	Max TL Deflection = 1.55 in = L / 252		240			
I = 2490.75	in⁴	> 2,368.69	in⁴	O.K.		Addt'l Load to Post	Source	Lx	Ly	
Left Reaction = 4,485 lb	Post: HGR				-	lb		9.0	2.0	ft
Right Reaction = 4,485 lb	Post: HGR				-	lb		9.0	2.0	ft

FB17

DESCRIPTION: FB17 | Roof Tributary = 0.00 ft | W_{DL} = 0.00 lb/ft | C_D = 1.00 | $C_{V/L}$ = 1.00
SIZE: 5-1/8X12 | Floor Tributary = 0.00 ft | W_{LL} = 0.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: 24F-V4 | Wall Tributary = 0.00 ft | P_{DL} = 3,540.70 lb | I_u = 3.00
LENGTH (ft)= 3.00 | Additional Load = 0.00 plf | P_{LL} = 3,460.00 lb | Reduction (L_r) %
NOTES: M=21,000LB-FT, V = 7001LB | $d_{Point\ Load}$ = 3.00 ft | Reduction (L) %

Actual			Req'd			Source:	L/xxx	Alternate #1	#2	LL
S = 123.00	in³	> 0.00	in³	O.K.	Max LL Deflection = 0.00 in = N/A		360			
A = 61.50	in²	> 43.75	in²	O.K.	Max TL Deflection = 0.00 in = N/A		240			
I = 738.00	in⁴	> 0.00	in⁴	O.K.		Addt'l Load to Post	Source	Lx	Ly	
Left Reaction = 0 lb	Post: 2X4	DFLSTUD	O.K.			lb		9.0	2.0	ft
Right Reaction = 7,001 lb	Post: (2)2X6	DFLSTUD	O.K.			lb		10.0	2.0	ft

FB18

DESCRIPTION: FB18 | Roof Tributary = 2.00 ft | W_{DL} = 407.02 lb/ft | C_D = 1.00 | $C_{V/L}$ = 1.00
SIZE: 5-1/8X12 | Floor Tributary = 13.00 ft | W_{LL} = 560.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: 24F-V4 | Wall Tributary = 13.00 ft | P_{DL} = 0.00 lb | I_u = 2.00
LENGTH (ft)= 12.50 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

Actual			Req'd			Source:	L/xxx	Alternate #1	#2	LL
S = 123.00	in³	> 94.68	in³	O.K.	Max LL Deflection = 0.23 in = L / 648		360			
A = 61.50	in²	> 31.73	in²	O.K.	Max TL Deflection = 0.40 in = L / 375		240			
I = 738.00	in⁴	> 472.18	in⁴	O.K.		Addt'l Load to Post	Source	Lx	Ly	
Left Reaction = 14,198 lb	Post: 6X6	DFL#2 (TIMBERS)	O.K.			8,154 lb		9.0	2.0	ft
Right Reaction = 6,044 lb	Post: (2)2X6	DFLSTUD	O.K.			lb		9.0	2.0	ft

FB19

DESCRIPTION: FB19 | Roof Tributary = 2.00 ft | W_{DL} = 358.01 lb/ft | C_D = 1.25 | $C_{V/L}$ = 0.99
SIZE: 5-1/8X24 | Floor Tributary = 12.00 ft | W_{LL} = 520.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: 24F-V4 | Wall Tributary = 7.00 ft | P_{DL} = 6,525.00 lb | I_u = 2.00
LENGTH (ft)= 24.50 | Additional Load = 40.00 plf | P_{LL} = 8,700.00 lb | Reduction (L_r) %
NOTES: USE 2 BEAMS W/ 5/8" THROUGH BOLTS AT 12" OC | $d_{Point\ Load}$ = 0.50 ft | Reduction (L) %

Actual			Req'd			Source:	L/xxx	Alternate #1	#2	LL
S = 492.00	in³	> 280.87	in³	O.K.	Max LL Deflection = 0.42 in = L / 695		360			
A = 123.00	in²	> 119.57	in²	O.K.	Max TL Deflection = 0.72 in = L / 410		240			
I = 5904.00	in⁴	> 3,452.07	in⁴	O.K.		Addt'l Load to Post	Source	Lx	Ly	
Left Reaction = 25,670 lb	Post: 6X10	DFL#2 (TIMBERS)	O.K.			lb		2.0	9.0	ft
Right Reaction = 11,066 lb	Post: 6X6	DFL#2 (TIMBERS)	O.K.			lb		9.0	2.0	ft

FB20

DESCRIPTION: FB20 | Roof Tributary = 0.00 ft | W_{DL} = 209.51 lb/ft | C_D = 1.00 | $C_{V/L}$ = 1.00
SIZE: 5-1/8X13-1/2 | Floor Tributary = 9.50 ft | W_{LL} = 380.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: 24F-V4 | Wall Tributary = 4.00 ft | P_{DL} = 0.00 lb | I_u = 2.00
LENGTH (ft)= 18.50 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.50 ft | Reduction (L) %

Actual			Req'd			Source:	L/xxx	Alternate #1	#2	LL
S = 155.67	in³	> 126.46	in³	O.K.	Max LL Deflection = 0.53 in = L / 419		360			
A = 69.19	in²	> 29.94	in²	O.K.	Max TL Deflection = 0.82 in = L / 270		240			
I = 1050.79	in⁴	> 933.14	in⁴	O.K.		Addt'l Load to Post	Source	Lx	Ly	
Left Reaction = 10,906 lb	Post: 6X6	DFL#2 (TIMBERS)	O.K.			5,453 lb		10.0	10.0	ft
Right Reaction = 5,453 lb	Post: 6X6	DFL#2 (TIMBERS)	O.K.			lb		9.0	2.0	ft

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191

DATE 12/6/2019

PROJECT: CUSTOM RESIDENCE **SHEET** **OF**

SUBJECT: Beams & Headers **DESIGNED** KB **CHECKED** KB

FB21

DESCRIPTION: FB21 | Roof Tributary = 0.00 ft | W_{DL} = 342.02 lb/ft | C_D = 1.25 $C_{V/L}$ = 1.00
SIZE: 5-1/8X18 | Floor Tributary = 18.00 ft | W_{LL} = 720.00 lb/ft | C_t = 1.00 C_F = 1.00
GRADE: 24F-V4 | Wall Tributary = 3.00 ft | P_{DL} = 8,880.00 lb | I_u = 2.00
LENGTH (ft)= 13.70 | Additional Load = 0.00 plf | P_{LL} = 7,100.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 3.50 ft | Reduction (L) %
Source: GT2+3 | L/xxx 360 240 | Alternate Loads #1 #2 LL

	Actual		Req'd		
S =	276.75	in³ >	243.46	in³	O.K.
A =	92.25	in² >	87.90	in²	O.K.
I =	2490.75	in⁴ >	1,529.44	in⁴	O.K.

Max LL Deflection = 0.23 in = L / 713
Max TL Deflection = 0.42 in = L / 391

						Addt'l Load to Post		Source	Lx	Ly
Left Reaction =	19,172 lb	Post:	HGR		-		lb		2.0	10.0 ft
Right Reaction =	11,357 lb	Post:	6X6	DFL#2 (TIMBERS)	O.K.		lb		9.0	2.0 ft

FB22

DESCRIPTION: FB22 | Roof Tributary = 0.00 ft | W_{DL} = 25.50 lb/ft | C_D = 1.25 $C_{V/L}$ = 0.89
SIZE: 8-3/4X30 | Floor Tributary = 1.50 ft | W_{LL} = 60.00 lb/ft | C_t = 1.00 C_F = 1.00
GRADE: 24F-V4 | Wall Tributary = 0.00 ft | P_{DL} = 14,495.00 lb | I_u = 2.00
LENGTH (ft)= 36.00 | Additional Load = 0.00 plf | P_{LL} = 15,663.00 lb | Reduction (L_r) %
NOTES: SEE SEPARATE CALCULATION FOR BASE SIZE | $d_{Point\ Load}$ = 14.00 ft | Reduction (L) %
Source: L/xxx 360 240 | Alternate #1 #2 LL

	Actual		Req'd		
S =	1312.50	in³ >	1,212.28	in³	O.K.
A =	262.50	in² >	114.88	in²	O.K.
I =	19687.50	in⁴ >	15,631.74	in⁴	O.K.

Max LL Deflection = 0.76 in = L / 569
Max TL Deflection = 1.43 in = L / 302

						Addt'l Load to Post		Source	Lx	Ly
Left Reaction =	23,189 lb	Post:	6X10	DFL#2 (TIMBERS)	O.K.	3220	lb		2.0	10.0 ft
Right Reaction =	13,267 lb	Post:	6X8	DFL#2 (TIMBERS)	O.K.		lb		2.0	10.0 ft

FB23

DESCRIPTION: FB23 | Roof Tributary = 0.00 ft | W_{DL} = 34.00 lb/ft | C_D = 1.25 $C_{V/L}$ = 1.00
SIZE: 5-1/8X18 | Floor Tributary = 2.00 ft | W_{LL} = 80.00 lb/ft | C_t = 1.00 C_F = 1.00
GRADE: 24F-V4 | Wall Tributary = 0.00 ft | P_{DL} = 8,880.00 lb | I_u = 2.00
LENGTH (ft)= 18.00 | Additional Load = 0.00 plf | P_{LL} = 7,100.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 1.50 ft | Reduction (L) %
Source: L/xxx 360 240 | Alternate #1 #2 LL

	Actual		Req'd		
S =	276.75	in³ >	93.94	in³	O.K.
A =	92.25	in² >	77.52	in²	O.K.
I =	2490.75	in⁴ >	688.73	in⁴	O.K.

Max LL Deflection = 0.13 in = L / 1717
Max TL Deflection = 0.25 in = L / 868

						Addt'l Load to Post		Source	Lx	Ly
Left Reaction =	15,674 lb	Post:	HGR	DFLSTUD	-		lb		9.0	2.0 ft
Right Reaction =	2,358 lb	Post:	(3)2X6	DFLSTUD	O.K.		lb		9.0	2.0 ft

FB24

DESCRIPTION: FB24 | Roof Tributary = 0.00 ft | W_{DL} = 34.00 lb/ft | C_D = 1.00 $C_{V/L}$ = 0.97
SIZE: 6-3/4X19-1/2 | Floor Tributary = 2.00 ft | W_{LL} = 80.00 lb/ft | C_t = 1.00 C_F = 1.00
GRADE: 24F-V4 | Wall Tributary = 0.00 ft | P_{DL} = 6,525.00 lb | I_u = 2.00
LENGTH (ft)= 30.00 | Additional Load = 0.00 plf | P_{LL} = 8,700.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 4.50 ft | Reduction (L) %
Source: L/xxx 360 240 | Alternate #1 #2 LL

	Actual		Req'd		
S =	427.78	in³ >	332.27	in³	O.K.
A =	131.63	in² >	90.41	in²	O.K.
I =	4170.87	in⁴ >	3,202.18	in⁴	O.K.

Max LL Deflection = 0.69 in = L / 519
Max TL Deflection = 1.15 in = L / 313

						Addt'l Load to Post		Source	Lx	Ly
Left Reaction =	14,651 lb	Post:	6X6	DFL#1 (TIMBERS)	O.K.		lb		9.0	2.0 ft
Right Reaction =	3,994 lb	Post:	HGR		-		lb		9.0	2.0 ft

FB25

DESCRIPTION: FB25 | Roof Tributary = 2.00 ft | W_{DL} = 50.00 lb/ft | C_D = 1.00 $C_{V/L}$ = 1.00
SIZE: 5-1/8X15 | Floor Tributary = 0.00 ft | W_{LL} = 40.00 lb/ft | C_t = 1.00 C_F = 1.00
GRADE: 24F-V4 | Wall Tributary = 0.00 ft | P_{DL} = 1,845.00 lb | I_u = 2.00
LENGTH (ft)= 26.50 | Additional Load = 20.00 plf | P_{LL} = 1,335.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 12.80 ft | Reduction (L) %
Source: L/xxx 360 240 | Alternate #1 #2 LL

	Actual		Req'd		
S =	192.19	in³ >	144.74	in³	O.K.
A =	76.88	in² >	17.03	in²	O.K.
I =	1441.41	in⁴ >	1,310.56	in⁴	O.K.

Max LL Deflection = 0.52 in = L / 617
Max TL Deflection = 1.20 in = L / 264

						Addt'l Load to Post		Source	Lx	Ly
Left Reaction =	2,837 lb	Post:	(2)2X6	DFLSTUD	O.K.		lb		10.0	2.0 ft
Right Reaction =	2,729 lb	Post:	(2)2X6	DFLSTUD	O.K.		lb		10.0	2.0 ft

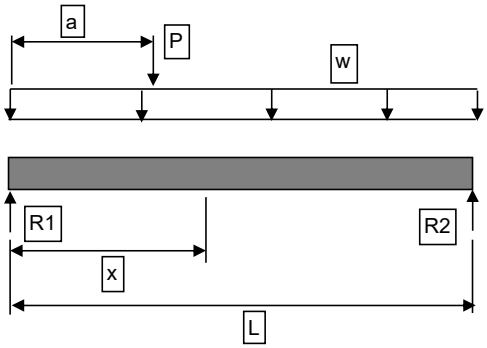
L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE:

PROJECT Assured Development **SHEET** **OF**
SUBJECT BEAMS AND HEADERS **DESIGNED** KB **CHECKED** KB

(SIMPLY SUPPORTED BEAM)

DESCRIPTION:	FB22	NOT Used for base size only		
SIZE:	**6-3/4X33**		C_D =	1.25
GRADE:	24F-V4		C_M =	1.00
LENGTH =	36.00	ft	C_t =	1.00
			C_L =	1.00
F_b=	2,700	psi	C_r =	1.00
F_v=	300	psi	C_c =	1.00
E=	1,800,000	psi	C_f =	1.00
			C_V =	0.90
M_{MAX} =	270,251	lb-ft	C_F =	1.00
V_{MAX} =	23,189	lb	C_{fu} =	1.00
			I_{actual}=	20,214.56
R_1 =	16,853.68	lb		
R_2 =	23,188.85	lb		
S_{req}=	1,201.12	IN^3		
A_{req}=	113.88	IN^2		
$Delta_{LLALLOW.}$ =	1.20	in		
$Delta_{TLALLOW.}$ =	1.80	in		



CHECK:

S_{actual}=	1225.13	in^3	>	1,201.12	in^3	**O.K.**
A_{actual}=	222.75	in^2	>	113.88	in^2	**O.K.**
Delta LL =	0.75	in	<	1.20	in	**O.K.**
Delta TL =	1.50	in	<	1.80	in	**O.K.**

LL => L/? = 576

1.5xDL Deflection = **1.13** inches TL => L/? = 287

UNIFORM LOADS

W_{DL}=	20.00	lb/ft	W_{LL}=	50.00	lb/ft
W_{DL}=	80.00	lb/ft	W_{LL}=	0.00	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft

POINT LOADS

			a (ft)				a (ft)
P_{DL} =	8,954.20	lb @	17.40	P_{LL} =	10,218.13	lb @	17.40
P_{DL} =	3,655.00	lb @	19.78	P_{LL} =	2,845.00	lb @	19.78
P_{DL} =	1,885.10	lb @	26.77	P_{LL} =	2,600.00	lb @	26.77
P_{DL} =	1,885.10	lb @	34.64	P_{LL} =	2,600.00	lb @	34.64
P_{DL} =	0.00	lb @	0.00	P_{LL} =	0.00	lb @	0.00

UNIFORM LOADS PARTIALLY DISTRIBUTED

			a (ft)	b (ft)				a (ft)	b (ft)
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00

NOTE: b IS THE DISTANCE FROM THE START TO THE END OF THE UNIFORMLY DISTRIBUTED LOAD.

16 of 43
"BEAMANAL.xls" Program
Version 2.3

SINGLE-SPAN BEAM ANALYSIS and AISC ASD CODE CHECK

For Simple, Propped, Fixed, or Cantilever Beams

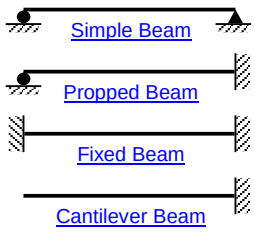
Using AISC W, S, C, or MC Shapes Subjected to X-Axis Bending Only

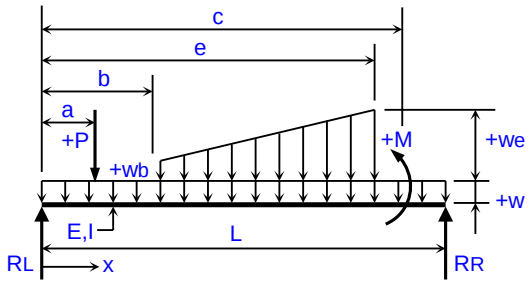
Job Name:		Subject:			
Job Number:		Originator:		Checker:	

Input Data:

Beam Data:

Span Type?	Simple	
Span, L =	36.0000	ft.
Modulus, E =	29000	ksi
Inertia, Ix =	1830.00	in.^4
Beam Size =	W24x68	
Yield, Fy =	36	ksi
Length, Lb =	4.0000	ft.
Coef., Cb =	1.00	





Nomenclature

Beam Loadings:

Full Uniform:

w = 0.2000 kips/ft.

Distributed:	Start b (ft.)	Wb (kips/ft.)	End e (ft.)	We (kips/ft.)
#1:				
#2:				
#3:				
#4:				
#5:				
#6:				
#7:				
#8:				

Point Loads:	a (ft.)	P (kips)
#1:	17.4000	19.17
#2:	19.7800	6.50
#3:	26.7700	4.49
#4:	34.6400	4.49
#5:		
#6:		
#7:		
#8:		
#9:		
#10:		
#11:		
#12:		
#13:		
#14:		
#15:		

Moments:	c (ft.)	M (ft-kips)
#1:		
#2:		
#3:		
#4:		

Results:

End Reactions:

RL =	17.75	kips	RR =	24.09	kips
MxL =	N.A.	ft-kips	MxR =	N.A.	ft-kips

Maximum Moments:

+Mx(max) =	278.64	ft-kips	@ x =	17.40	ft.
-Mx(max) =	0.00	ft-kips	@ x =	0.00	ft.

Maximum Deflections:

-Δ(max) =	-1.067	in.	@ x =	18.19	ft.
+Δ(max) =	0.000	in.	@ x =	0.00	ft.
Δ(ratio) =	L/405				

AISC Code Check for X-Axis Bending:

Lc =	9.47	ft.
Lu =	10.24	ft.
Lb/rt =	21.24	
fbx =	21.71	ksi
Fbx =	23.76	ksi
Mrx =	304.92	ft-kips
S.R. =	0.914	= fbx/Fbx

AISC Code Check for Gross Shear:

fv =	2.45	ksi
Fv =	14.40	ksi
S.R. =	0.170	= fv/Fv

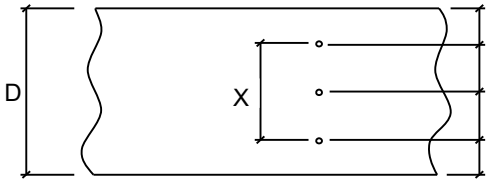
L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE: December 6, 2019

PROJECT CUSTOM RESIDENCE **SHEET** **OF**

SUBJECT Wood Ledger **DESIGNED** KB **CHECKED** KB

Minimum Nail Spacing - Per NDS



(d - Nail Diameter)

For 16d Sinkers: d = 0.148"

2x6 D = 5.5 inches
X = 5.5" - 2(2.5 x 0.148") = 5.5" - 0.74" = 4.76"
4.76 / 1.50 = 3.2 Spaces 2 Spaces @ 1.5" = 3.00" Use (3) Nails Maximum

2x8 D = 7.25 inches
X = 7.25" - 0.74" = 6.51" 4 Spaces @ 1.5" = 6.00" Use (5) Nails Maximum

2x10 D = 9.25 inches
X = 9.25" - 0.74" = 8.51" 5 Spaces @ 1.5" = 7.50" Use (6) Nails Maximum

2x12 D = 11.25 inches
X = 11.25" - 0.74" = 10.51" 7 Spaces @ 1.5" = 10.50" Use (8) Nails Maximum

Per Row
2 or 3 rows possible
where ledger nails to
end of 4x floor truss

DFL No. 2 Ledger to DFL Studs
Allowable Load for 1 1/2 Side Plate = 118 #/Nail

DFL No. 2 Ledger to SPF Studs or SPF Trusses
Allowable Load for 1 1/2 Side Plate = 100 #/Nail)

DFL VALUES
Capacity of 16d Sinkers @ 118 #/nail

	100% (Floor)	125% (Roof)
3 x 118 =	354 #	443 #
4 x 118 =	472 #	590 #
5 x 118 =	590 #	738 #
6 x 118 =	708 #	885 #
7 x 118 =	826 #	1033 #
8 x 118 =	944 #	1180 #

SPF VALUES
Capacity of 16d Sinkers @ 100 #/nail

	100% (Floor)	125% (Roof)
3 x 100 =	300 #	375 #
4 x 100 =	400 #	500 #
5 x 100 =	500 #	625 #
6 x 100 =	600 #	750 #
7 x 100 =	700 #	875 #
8 x 100 =	800 #	1000 #

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE: December 6, 2019

PROJECT CUSTOM RESIDENCE **SHEET** **OF**

SUBJECT Wood Ledger **DESIGNED** KB **CHECKED** KB

Ledgers Supporting Roof Loads — Total Allowable Roof Load

DFL Frmg = 70psf SPF Frmg = 50 psf

Supported Span	Member	Nailing	Min Hanger UNO
8ft or Less	2x6	(3) 16d at 16" o.c.	LUS24/JUS24 at16" oc
	2x8	(5) 16d at 24" o.c.	LUS24/JUS24 at 24" oc
16ft or Less	2x8	(5) 16d at 16" o.c.	LUS26/JUS26 at16" oc
	2x12	(8) 16d at 24" o.c.	LUS28/JUS28 at 24" oc
24ft or Less	2x12	(8) 16d at 16" o.c.	LUS28/JUS28 at16" oc
	2x12	(2) columns of (6) 16d at 24" o.c.	HUS28/HUS28 at 24" oc

Ledgers Supporting Floor Loads — Total Allowable Floor Load

DFL Frmg = 89psf SPF Frmg = 70 psf

Supported Span	Member	Nailing	Min Hanger UNO
10ft or Less	2x8	(5) 16d at 16" o.c.	LUS26/JUS26 at16" oc
	2x10	(2) columns of (6) 16d at 24" o.c.	LUS26/JUS26 at 24" oc
20ft or Less	2x10	(2) columns of (6) 16d at 16" o.c.	LUS28/JUS28 at16" oc
	2x12	(2) columns of (8) 16d at 24" o.c.	HUS210/HUS210 at 24"oc

NOTES:
1. Minimum hangers are listed by manufacturer as Simpson Strong-tie/USP
2. Listed hangers are minimum hangers where not otherwise noted on the structural drawings
3. Two columns of nails require min. 3", (2)2x or 4x2 trusses for supporting members receiving nailing

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE:

PROJECT: CUSTOM RESIDENCE SHEET OF
DESCRIPTION: 1ST FLOOR DESIGNED KB CHECKED KB

STUD WALL DESIGN - ASCE 7 WIND PROVISIONS (BASIC LOAD COMBINATIONS)

Roof Dead Load =	15.00 psf
Floor Dead Load =	17.00 psf
Wall/Misc. Dead Load =	12.00 psf

Roof Live Load =	20.00 psf
Floor Live Load =	40.00 psf
Roof Snow Load =	0.00 psf

Wall Height =	12.00 ft
Roof Tributary Width =	21.00 ft
Floor Tributary Width =	11.00 ft
Wall/Misc. DL Width =	6.00 ft

Wind Zone (Interior or End) =	End

Stud Size =	2X6
Stud Spacing =	16 in o.c.
Lumber Grade =	DFL STUD

Wind Speed =	100 mph
Exposure =	C
Mean Roof Height, h =	25.00 ft
Topo. Factor, K_{zt} =	1.00
Wind Dir. Factor, K_d =	0.85
GCpi +/- =	0.18

INTERACTION RESULTS FOR POSSIBLE CONTROLLING LOAD COMBINATIONS		
DL + FLL =	**0.338**	**OK**
DL + RLL =	**0.311**	**OK**
DL + SL =	**0.191**	**OK**
DL + 0.6*WIND =	**0.606**	**OK**
DL + 0.75*(FLL + RLL) =	**0.382**	**OK**
DL + 0.75*(FLL + SL) =	**0.283**	**OK**
DL + 0.75*(0.6*WIND + FLL + RLL) =	**0.673**	**OK**
DL + 0.75*(0.6*WIND + FLL + SL) =	**0.555**	**OK**
DEFLECTION L/? =	**581.3**	**O.K.****

** Deflection Determined Using 0.42 x Wind Load

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

F_b =	700 psi
F_c =	850 psi
E =	1,400,000 psi

Factors	
C_F (Bending) =	1.00
C_F (Compression) =	1.00
C_r =	1.15
C_D (DL) =	1.00
C_D (FLL) =	1.00
C_D (RLL) =	1.25
C_D (SL) =	1.00
C_D (Wind) =	1.60

Wall Uniform Loads	
Dead Load, DL =	574 lb/ft
Roof Live Load, RLL =	420 lb/ft
Floor Live Load, FLL =	440 lb/ft
Snow Load, SL =	0 lb/ft

Ultimate Design Pressure =	27.61 psf	
Ultimate Max. Deflection =	0.59 in	
Max. Deflection x 0.42 =	0.25 in	
Max. Deflection x 0.60 =	N/A	
L/d =	26.18	< 50 (OK)

Check Bearing Stress on Wall Plate		
Wall Plate Lumber Grade =	DFL#2	
Max. Stud Reaction on Wall Plate =	1,912 lbs	
Bearing Stress on Wall Plate =	231.76 psi	(OK)
Allow. Bearing Stress on Wall Plate =	625 psi	

Ultimate Wall Pressures (psf)				
Zone	Interior Zone (Zone 4)		(Zone 5)	
Positive/Negative	Positive	Negative	Positive	Negative
Pressure (psf)	21.78 psf	-23.83 psf	21.78 psf	-27.61 psf
qh (psf)	20.46 psf	20.46 psf	20.46 psf	20.46 psf
GCp	0.885	-0.985	0.885	-1.169
Gcpi	0.180	-0.180	0.180	-0.180

Studs Part of Fire Assembly? No

Stud Section Properties		
Area =	8.25	in^2
Section Modulus =	7.56	in^3
Moment of Inertia =	20.80	in^4
Depth =	5.50 in	
Width =	1.50 in	

Axial Stresses				
	DL	FLL	RLL	SL
f_a =	92.77 psi	71.11 psi	67.88 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	663 ft-lb
f_b =	1051.32 psi

Allowable Bending Stress		
F'_b =	1,288 psi	

Allowable Axial Stresses					
c =	0.80	Overwrite c =			
$(L/d)^2$ =	685.49				
COV_E =	0.25	Overwrite COV_E =			
E_{min}' =	511,432 psi				
F_{cE} =	613.28				
	DL	FLL	RLL	SL	WIND
F_c^* =	850.00	850.00	1,062.50	850.00	1,360.00
F_{cE}/F_c^* =	0.722	0.722	0.577	0.722	0.451
C_P =	0.570	0.570	0.486	0.570	0.398
F'c =	484.68	484.68	515.90	484.68	541.60 psi

Note: All wall studs in this calculator are braced in the weak direction.

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UTAH - 51 West 9000 South - Sandy, UT 84070 - (801) 565-8580 - Fax (801) 565-9340

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE:

PROJECT: CUSTOM RESIDENCE **SHEET** **OF**
DESCRIPTION: 1ST FLOOR **DESIGNED** KB **CHECKED** KB

STUD WALL DESIGN - ASCE 7 WIND PROVISIONS (BASIC LOAD COMBINATIONS)

Roof Dead Load =	15.00 psf
Floor Dead Load =	17.00 psf
Wall/Misc. Dead Load =	12.00 psf

Roof Live Load =	20.00 psf
Floor Live Load =	40.00 psf
Roof Snow Load =	0.00 psf

Wall Height =	12.00 ft
Roof Tributary Width =	0.00 ft
Floor Tributary Width =	21.00 ft
Wall/Misc. DL Width =	6.00 ft

Wind Zone (Interior or End) =	End

Stud Size =	2X6
Stud Spacing =	16 in o.c.
Lumber Grade =	DFL STUD

Wind Speed =	100 mph
Exposure =	C
Mean Roof Height, h =	25.00 ft
Topo. Factor, K_{zt} =	1.00
Wind Dir. Factor, K_d =	0.85
GCpi +/- =	0.18

INTERACTION RESULTS FOR POSSIBLE CONTROLLING LOAD COMBINATIONS		
DL + FLL =	**0.423**	**OK**
DL + RLL =	**0.134**	**OK**
DL + SL =	**0.143**	**OK**
DL + 0.6*WIND =	**0.569**	**OK**
DL + 0.75*(FLL + RLL) =	**0.423**	**OK**
DL + 0.75*(FLL + SL) =	**0.423**	**OK**
DL + 0.75*(0.6*WIND + FLL + RLL) =	**0.609**	**OK**
DL + 0.75*(0.6*WIND + FLL + SL) =	**0.609**	**OK**
DEFLECTION L/? =	**581.3**	**O.K.****

** Deflection Determined Using 0.42 x Wind Load

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

F_b =	700 psi
F_c =	850 psi
E =	1,400,000 psi

Factors	
C_F (Bending) =	1.00
C_F (Compression) =	1.00
C_r =	1.15
C_D (DL) =	1.00
C_D (FLL) =	1.00
C_D (RLL) =	1.25
C_D (SL) =	1.00
C_D (Wind) =	1.60

Wall Uniform Loads	
Dead Load, DL =	429 lb/ft
Roof Live Load, RLL =	0 lb/ft
Floor Live Load, FLL =	840 lb/ft
Snow Load, SL =	0 lb/ft

Ultimate Design Pressure =	27.61 psf	
Ultimate Max. Deflection =	0.59 in	
Max. Deflection x 0.42 =	0.25 in	
Max. Deflection x 0.60 =	N/A	
L/d =	26.18	< 50 (OK)

Check Bearing Stress on Wall Plate		
Wall Plate Lumber Grade =	DFL#2	
Max. Stud Reaction on Wall Plate =	1,692 lbs	
Bearing Stress on Wall Plate =	205.09 psi	(OK)
Allow. Bearing Stress on Wall Plate =	625 psi	

Ultimate Wall Pressures (psf)				
Zone	Interior Zone (Zone 4)		(Zone 5)	
Positive/Negative	Positive	Negative	Positive	Negative
Pressure (psf)	21.78 psf	-23.83 psf	21.78 psf	-27.61 psf
qh (psf)	20.46 psf	20.46 psf	20.46 psf	20.46 psf
GCp	0.885	-0.985	0.885	-1.169
Gcpi	0.180	-0.180	0.180	-0.180

Studs Part of Fire Assembly? No

Stud Section Properties		
Area =	8.25	in^2
Section Modulus =	7.56	in^3
Moment of Inertia =	20.80	in^4
Depth =	5.50 in	
Width =	1.50 in	

Axial Stresses				
	DL	FLL	RLL	SL
f_a =	69.34 psi	135.76 psi	0.00 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	663 ft-lb
f_b =	1051.32 psi

Allowable Bending Stress		
F'_b =	1,288 psi	

Allowable Axial Stresses					
c =	0.80	Overwrite c =			
$(L/d)^2$ =	685.49				
COV_E =	0.25	Overwrite COV_E =			
E_{min}' =	511,432 psi				
F_{cE} =	613.28				
	DL	FLL	RLL	SL	WIND
F_c^* =	850.00	850.00	1,062.50	850.00	1,360.00
F_{cE}/F_c^* =	0.722	0.722	0.577	0.722	0.451
C_P =	0.570	0.570	0.486	0.570	0.398
F'c =	484.68	484.68	515.90	484.68	541.60 psi

Note: All wall studs in this calculator are braced in the weak direction.

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L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191
DATE 12/6/2019

PROJECT: CUSTOM RESIDENCE **SHEET** **OF**
SUBJECT: Allowable Connector Loads **DESIGNED** KB **CHECKED** KB

16d SINKERS

**Use 16d sinker nails (0.148" Diameter × 3 ¼")*

**Assume all nails penetrate through a 1½" plate, 23/32" floor sheathing and 1½" of solid blocking.*

$Z_{16d\ sinker}$ = 103 lbs/nail C_D = 1.6 C_t = 1.0

$Z'_{16d\ sinker}$ = $(Z_{16d\ sinker})(C_D)(C_M)(C_t)$ SG = 0.50 C_M = 1.0

= (103lbs/nail)(1.6)(1.0)(1.0)

= 165 lbs/nail

16d sinkers at	8 in o.c.	⇒	165 *lbs/nail* x	1.5	nails/ft =	**247**	***lbs/ft***
16d sinkers at	5 in o.c.	⇒	165 *lbs/nail* x	2.4	nails/ft =	**396**	***lbs/ft***
16d sinkers at	4 in o.c.	⇒	165 *lbs/nail* x	3	nails/ft =	**494**	***lbs/ft***
16d sinkers at	3 in o.c.	⇒	165 *lbs/nail* x	4	nails/ft =	**659**	***lbs/ft***
16d sinkers at	2 in o.c.	⇒	165 *lbs/nail* x	6	nails/ft =	**989**	***lbs/ft***

16d SINKERS

**Use 16d sinker nails (0.148" Diameter × 3 ¼")*

**Assume all nails penetrate through a 1½" plate into 1½" (min) plate.*

$Z_{16d\ sinker}$ = 118 lbs/nail C_D = 1.6 C_t = 1.0

$Z'_{16d\ sinker}$ = $(Z_{16d\ sinker})(C_D)(C_M)(C_t)$ SG = 0.50 C_M = 1.0

= (118lbs/nail)(1.6)(1.0)(1.0)

= 189 lbs/nail

16d sinkers at	8 in o.c.	⇒	165 *lbs/nail* x	1.5	nails/ft =	**283**	***lbs/ft***
16d sinkers at	5 in o.c.	⇒	165 *lbs/nail* x	2.4	nails/ft =	**453**	***lbs/ft***
16d sinkers at	4 in o.c.	⇒	165 *lbs/nail* x	3	nails/ft =	**566**	***lbs/ft***
16d sinkers at	3 in o.c.	⇒	165 *lbs/nail* x	4	nails/ft =	**755**	***lbs/ft***
16d sinkers at	2 in o.c.	⇒	165 *lbs/nail* x	6	nails/ft =	**1133**	***lbs/ft***

WOOD SCREWS

**Assume all screws penetrate through a 1½" plate, 23/32" floor sheathing and 1½" of solid blocking.*

$Z_{1/4"\ x\ 6\ WS}$ = 135 lbs/nail C_D = 1.6 C_t = 1.0

$Z'_{1/4"\ x\ 6\ WS}$ = $(Z_{1/4"x6\ WS})(C_D)(C_M)(C_t)$ SG = 0.50 C_M = 1.0

= (135lbs/screw)(1.6)(1.0)(1.0)

= 216 lbs/screw

1/4" x 6 WS	6 in o.c.	⇒	216 *lbs/screw* x	2	screws/ft =	**432**	***lbs/ft***
1/4" x 6 WS	4 in o.c.	⇒	216 *lbs/screw* x	3	screws/ft =	**648**	***lbs/ft***
1/4" x 6 WS	3 in o.c.	⇒	216 *lbs/screw* x	4	screws/ft =	**864**	***lbs/ft***
1/4" x 6 WS	2 in o.c.	⇒	216 *lbs/screw* x	6	screws/ft =	**1296**	***lbs/ft***

A35

**Assume full penetration of nails as required by manufacturer for full load*

SG = 0.50

A35 = 695 lbs

A35	32 in o.c.	⇒	695 *lbs/clip* x	0.4	clips/ft =	**261**	***lbs/ft***
A35	18 in o.c.	⇒	695 *lbs/clip* x	0.7	clips/ft =	**463**	***lbs/ft***
A35	12 in o.c.	⇒	695 *lbs/clip* x	1	clips/ft =	**695**	***lbs/ft***
A35	10 in o.c.	⇒	695 *lbs/clip* x	1.2	clips/ft =	**834**	***lbs/ft***
A35	8 in o.c.	⇒	695 *lbs/clip* x	1.5	clips/ft =	**1043**	***lbs/ft***

LTP4

**Assume full penetration of nails as required by manufacturer for full load*

SG = 0.50

LTP4 = 670 lbs

LTP4	32 in o.c.	⇒	670 *lbs/clip* x	0.4	clips/ft =	**251**	***lbs/ft***
LTP4	18 in o.c.	⇒	670 *lbs/clip* x	0.7	clips/ft =	**447**	***lbs/ft***
LTP4	12 in o.c.	⇒	670 *lbs/clip* x	1	clips/ft =	**670**	***lbs/ft***
LTP4	10 in o.c.	⇒	670 *lbs/clip* x	1.2	clips/ft =	**804**	***lbs/ft***
LTP4	8 in o.c.	⇒	670 *lbs/clip* x	1.5	clips/ft =	**1005**	***lbs/ft***

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE: December 6, 2019

PROJECT CUSTOM RESIDENCE **SHEET** **OF**

SUBJECT Athens Lot 4 **DESIGNED** KB **CHECKED** KB

LOCATION: STAIR TO RCB PASSAGE

BENDING CAPACITY OF RECTANGULAR CONCRETE BM					SHEAR CAPACITY OF RECTANGULAR CONCRETE BM			
M_u =	6,665	ft-lb			V_u =	2000 lb		
f_c' =	2,500	psi			Φ =	0.75		
Width (b) =	12	in			**Without Shear Reinforcement**			
Depth (d) =	8.00	in			$\Phi V_c/2$ =	**3600 lb**		**OK**
fy =	60,000	psi			**With Shear Reinforcement**			
d_t =	6.00	in			Spacing =	4.00	in o.c.	
Bar Size	No. 5				$A_{v\ (min)}$ =	0.04	in^2	
# of Bars	2				Shear Bars	No. 4	A_v =	0.20 in^2
a =	1.44	in			# of Legs	1		
β1 =	0.85		A_s =	0.61 in^2	ΦV_c =	7200 lb		
Φ =	0.90		A_s (min) =	0.00 in^2	ΦV_s =	17671 lb		
ΦMn =	**20,096**	ft-lb	**OK**		ΦV_n =	**24871 lb**		**OK**

NOTES:

LOCATION:

BENDING CAPACITY OF RECTANGULAR CONCRETE BM					SHEAR CAPACITY OF RECTANGULAR CONCRETE BM			
M_u =	0	ft-lb			V_u =	0 lb		
f_c' =	2,500	psi			Φ =	0.75		
Width (b) =	12	in			**Without Shear Reinforcement**			
Depth (d) =	7.80	in			$\Phi V_c/2$ =	**3510 lb**		**OK**
fy =	60,000	psi			**With Shear Reinforcement**			
d_t =	7.80	in			Spacing =	3.90	in o.c.	
Bar Size	No. 5				$A_{v\ (min)}$ =	0.04	in^2	
# of Bars	2				Shear Bars	No. 3	A_v =	0.11 in^2
a =	1.44	in			# of Legs	1		
β1 =	0.85		A_s =	0.61 in^2	ΦV_c =	7020 lb		
Φ =	0.90		A_s (min) =	0.00 in^2	ΦV_s =	9940 lb		
ΦMn =	**19,544**	ft-lb	**OK**		ΦV_n =	**16960 lb**		**OK**

NOTES:

LOCATION:

BENDING CAPACITY OF RECTANGULAR CONCRETE BM					SHEAR CAPACITY OF RECTANGULAR CONCRETE BM			
M_u =	0	ft-lb			V_u =	0 lb		
f_c' =	3,000	psi			Φ =	0.75		
Width (b) =	12	in			**Without Shear Reinforcement**			
Depth (d) =	12.00	in			$\Phi V_c/2$ =	**5915 lb**		**OK**
fy =	60,000	psi			**With Shear Reinforcement**			
d_t =	12.00	in			Spacing =	6.00	in o.c.	
Bar Size	No. 5				$A_{v\ (min)}$ =	0.06	in^2	
# of Bars	2				Shear Bars	No. 3	A_v =	0.11 in^2
a =	1.20	in			# of Legs	1		
β1 =	0.85		A_s =	0.61 in^2	ΦV_c =	11831 lb		
Φ =	0.90		A_s (min) =	0.00 in^2	ΦV_s =	9940 lb		
ΦMn =	**31,473**	ft-lb	**OK**		ΦV_n =	**21771 lb**		**OK**

NOTES:

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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **SECTION**
SUBJECT SEISMIC LOADS **DESIGNED** KB **CHECKED** KB

SEISMIC LOADS (Main Lateral Force Resisting System)

Risk Category =	II
Seismic Design Category =	C

See below

Site Class:	C
R =	6.5
S_s =	0.498
S_1 =	0.162
F_a =	1.3
F_v =	1.5
S_{MS} =	0.65
S_{M1} =	0.24
S_{DS} =	0.432
S_{D1} =	0.162

SDC =	C
SDC =	C

N =	2	(number of stories)
C_T =	0.02	
h_n (ft) =	24.00	(to highest level)
T_a =	0.22	
x =	0.75	
T_L =	6	

Redundancy, ρ =	1.00

I_E =	1.00
C_{SMAX} =	0.115
C_S =	0.066
C_{SMIN} =	0.010
C_{SMIN} =	0

$C_{SCONTROL}$=	**0.066**
ρ*0.7*$C_{SCONTROL}$=	**0.046**

$E = \rho Q_E + 0.2 S_{DS} D$ =	0.066	x W +	0.086	x D
$E = \rho Q_E - 0.2 S_{DS} D$ =	0.066	x W -	0.086	x D
$0.7E = 0.7(\rho Q_E + 0.2 S_{DS} D)$ =	**0.046**	x W +	**0.060**	x D
$0.7E = 0.7(\rho Q_E - 0.2 S_{DS} D)$ =	**0.046**	x W -	**0.060**	x D

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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **SECTION**
SUBJECT VERTICAL REDISTRIBUTION **DESIGNED** KB **CHECKED** KB

VERTICAL REDISTRIBUTION OF SEISMIC FORCES

Number of Stories =	2

Seismic Coefficient, C_S =	0.066
0.7 x C_S =	**0.046**

Building Period, T =	0.22	<	0.5	=> k =	1

LEVEL	H (ft)	W (lb or psf)	ADJUSTED SEISMIC FACTOR AT LEVEL	SEISMIC ADJUSTMENT FACTOR AT LEVEL
2nd FLOOR	12.00	170,810 lbs	**0.032**	**0.696**
ROOF	24.00	132,841 lbs	**0.065**	**1.391**
N/A	0.00	0 lbs	N/A	N/A
N/A	0.00	0 lbs	N/A	N/A
N/A	0.00	0 lbs	N/A	N/A

CALCULATED VALUES:				Vertical Distribution of Shear	% of Base Shear
Level (i)	Height (h_i)	Weight (w_i)	$h_i^k * w_i$		
2nd FLOOR	12.00	170,810	2,049,719	39.13%	100.00%
ROOF	24.00	132,841	3,188,176	60.87%	60.87%
N/A	0.00	0	0	0.00%	0.00%
N/A	0.00	0	0	0.00%	0.00%
N/A	0.00	0	0	0.00%	0.00%
	Σ =	303,651	5,237,895	100.00%	

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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE: 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **OF**
SUBJECT WIND LOADS (Main Lateral Force Resisting System) **DESIGNED** KB **CHECKED** KB

Directional Procedure: Rigid Buildings of All Heights **Parallel to Ridge**

$p = qGC_p - q_i(GC_{pi})$ psf

☐ FLEXIBLE STRUCTURE (natural frequency < 1 Hz)

V(mph) =	100 mph	Mean Roof Height, h (ft) =	25	ω =	1.0
Exposure =	C	Width of Building, B (ft) =	136		
Grnd Elev Factor, Ke =	0.95	Depth of Building, L (ft) =	60		
G =	0.85	Topo. Factor, K_{zt} =	1		
Roof Angle =	1.19	Wind Dir. Factor, K_d =	0.85		
Roof Slope =	0.25 /12	h/L =	0.42		

HEIGHT (ft)	K_Z	q_z (psf)	q_h (psf)	PRESSURE (psf) Walls (Windward)	Walls (Leeward)	Roof (Windward, Positive, Normal)	Roof (Windward, Negative, Normal)	Roof (Leeward, Normal)
C_p =				0.80	-0.50	0.00	0.00	-0.42
0-15	0.85	17.58		**12.0**				
20	0.9	18.61		**12.7**				
25	0.94	19.44	19.44	**13.3**	**-8.3**	**0**	**0**	**0**
30	0.98	20.26		**13.8**				
40	1.04	21.50		**14.7**				
50	1.09	22.54		**15.4**				
60	1.13	23.36		**15.9**				
70	1.17	24.19		**16.5**				
80	1.21	25.02		**17.1**				
90	1.24	25.64		**17.5**				
100	1.26	26.05		**17.8**				
120	1.31	27.09		**18.5**				
140	1.36	28.12		**19.2**				
160	1.39	28.74		**19.6**				
180	1.43	29.57		**20.2**				
200	1.46	30.19		**20.6**				
250	1.53	31.63		**21.6**				

HEIGHT (ft)	Pressure (psf) Side Walls	Roof (Normal w/ Slope Less Than 10 Degrees or Parallel to Ridge) 0 to h/2	h/2 to h	h to 2h	> 2h	Internal +/- Enclosed
C_p =	-0.70	-0.90	-0.90	-0.50	-0.30	0.18
0-15						
20						
25	**-11.6**	**-14.9**	**-14.9**	**-8.3**	**-5.0**	**3.5**
30						
40						
50						
60						
70						
80						
90						
100						
120						
140						
160						
180						
200						
250						

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L. R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191
DATE: 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **OF**
SUBJECT WIND LOADS (Main Lateral Force Resisting System) **DESIGNED** KB **CHECKED** KB

Directional Procedure: Rigid Buildings of All Heights **Perpendicular to Ridge**
$p = qGC_p - q_i(GC_{pi})$ psf ☐ FLEXIBLE STRUCTURE (natural frequency < 1 Hz)

V(mph) =	100 mph	Mean Roof Height, h (ft) =	25	ω =	1.0
Exposure =	C	Width of Building, B (ft) =	60		
Grnd Elev Factor, Ke =	0.95	Depth of Building, L (ft) =	136		
G =	0.85	Topo. Factor, K_{zt} =	1		
Roof Angle =	1.19	Wind Dir. Factor, K_d =	0.85		
Roof Slope =	0.25 /12	h/L =	0.18		

HEIGHT (ft)	K_Z	q_z (psf)	q_h (psf)	PRESSURE (psf) Walls (Windward)	Walls (Leeward)	Roof (Windward, Positive, Normal)	Roof (Windward, Negative, Normal)	Roof (Leeward, Normal)
C_p =				0.80	-0.30	0.00	0.00	-0.30
0-15	0.85	17.58		**12.0**				
20	0.9	18.61		**12.7**				
25	0.94	19.44	19.44	**13.3**	**-5.0**	**0**	**0**	**0**
30	0.98	20.26		**13.8**				
40	1.04	21.50		**14.7**				
50	1.09	22.54		**15.4**				
60	1.13	23.36		**15.9**				
70	1.17	24.19		**16.5**				
80	1.21	25.02		**17.1**				
90	1.24	25.64		**17.5**				
100	1.26	26.05		**17.8**				
120	1.31	27.09		**18.5**				
140	1.36	28.12		**19.2**				
160	1.39	28.74		**19.6**				
180	1.43	29.57		**20.2**				
200	1.46	30.19		**20.6**				
250	1.53	31.63		**21.6**				

HEIGHT (ft)	Pressure (psf) Side Walls	Roof (Normal w/ Slope Less Than 10 Degrees or Parallel to Ridge) 0 to h/2	h/2 to h	h to 2h	> 2h	Internal +/- Enclosed
C_p =	-0.70	-0.90	-0.90	-0.50	-0.30	0.18
0-15						
20						
25	**-11.6**	**-14.9**	**-14.9**	**-8.3**	**-5.0**	**3.5**
30						
40						
50						
60						
70						
80						
90						
100						
120						
140						
160						
180						
200						
250						

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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **SECTION**
SUBJECT LATERAL LINE LOADS **DESIGNED** KB **CHECKED** KB

Seismic Line Loads

Label	Level	Roof Trib (ft)	Floor Trib (ft)	Wall Trib (ft)	Other (lb/ft)	Total Weight (lb/ft)	Total Force (lb/ft)	Redist Factor	Revised Force (lb/ft)
ω1	ROOF	62		12		1074	50	1.391	69
ω2	2nd FLOOR		62	20		1294	60	0.696	42
ω3	ROOF	44		12		804	37	1.391	52
ω4	2nd FLOOR		44	20		988	46	0.696	32
ω5	ROOF	62		12		1074	50	1.391	69
ω6	2nd FLOOR		62	20		1294	60	0.696	42
ω7	ROOF	93		12		1539	72	1.391	100
ω8	2nd FLOOR		60	20		1260	59	0.696	41
ω9						0	0	1.000	0
ω10						0	0	1.000	0
ω11						0	0	1.000	0
ω12						0	0	1.000	0
ω13						0	0	1.000	0
ω14						0	0	1.000	0
ω15						0	0	1.000	0
ω16						0	0	1.000	0
ω17						0	0	1.000	0
ω18						0	0	1.000	0
ω19						0	0	1.000	0
ω20						0	0	1.000	0
ω21						0	0	1.000	0

Roof DL	15.00	psf
Seismic Snow	0.00	psf
Floor DL	17.00	psf
Wall DL	12.00	psf
Seis Coeff	0.046	* V

Wind Line Loads **PROCEDURE:** Envelope: Not Used Directional: X

Label	Roof Pitch /12	Mean Height for Windward Pressure	Direction of Wind Parallel to Ridge?	Windward Wall (ft)	Leeward Wall (ft)	Windward Roof (ft)	Leeward Roof (ft)	Ultimate Wind Force (plf)		Ultimate 16 psf wall & 8 psf roof min. (plf)	Allowable Wind Force (plf)		Allowable 9.6 psf wall & 4.8 psf roof min. (plf)
ω1	0.25/12	25	Yes	12	12			259.20		192.00	155.52		115.20
ω2	0.25/12	12	Yes	12	12			243.60		192.00	146.16		115.20
ω3	0.25/12	25	No	12				159.60		192.00	95.76		115.20
ω4	0.25/12	12	No	12	12			204.00		192.00	122.40		115.20
ω5	0.25/12	25	Yes	12	12			259.20		192.00	155.52		115.20
ω6	0.25/12	12	Yes	12	12			243.60		192.00	146.16		115.20
ω7	0.25/12	25	No	12				159.60		192.00	95.76		115.20
ω8	0.25/12	12	No	12	12			204.00		192.00	122.40		115.20
ω9	0.25/12							0.00		0.00	0.00		0.00
ω10	0.25/12							0.00		0.00	0.00		0.00
ω11	0.25/12							0.00		0.00	0.00		0.00
ω12	0.25/12							0.00		0.00	0.00		0.00
ω13	0.25/12							0.00		0.00	0.00		0.00
ω14	0.25/12							0.00		0.00	0.00		0.00
ω15	0.25/12							0.00		0.00	0.00		0.00
ω16	0.25/12							0.00		0.00	0.00		0.00
ω17	0.25/12							0.00		0.00	0.00		0.00
ω18	0.25/12							0.00		0.00	0.00		0.00
ω19	0.25/12							0.00		0.00	0.00		0.00
ω20	0.25/12							0.00		0.00	0.00		0.00
ω21	0.25/12							0.00		0.00	0.00		0.00

Note: "16 psf wall & 8 psf roof min. (plf)" check is sometimes set to zero due to the line load being windward only or leeward only (16 psf wall & 8 psf roof min. (plf) check is for windward + leeward)

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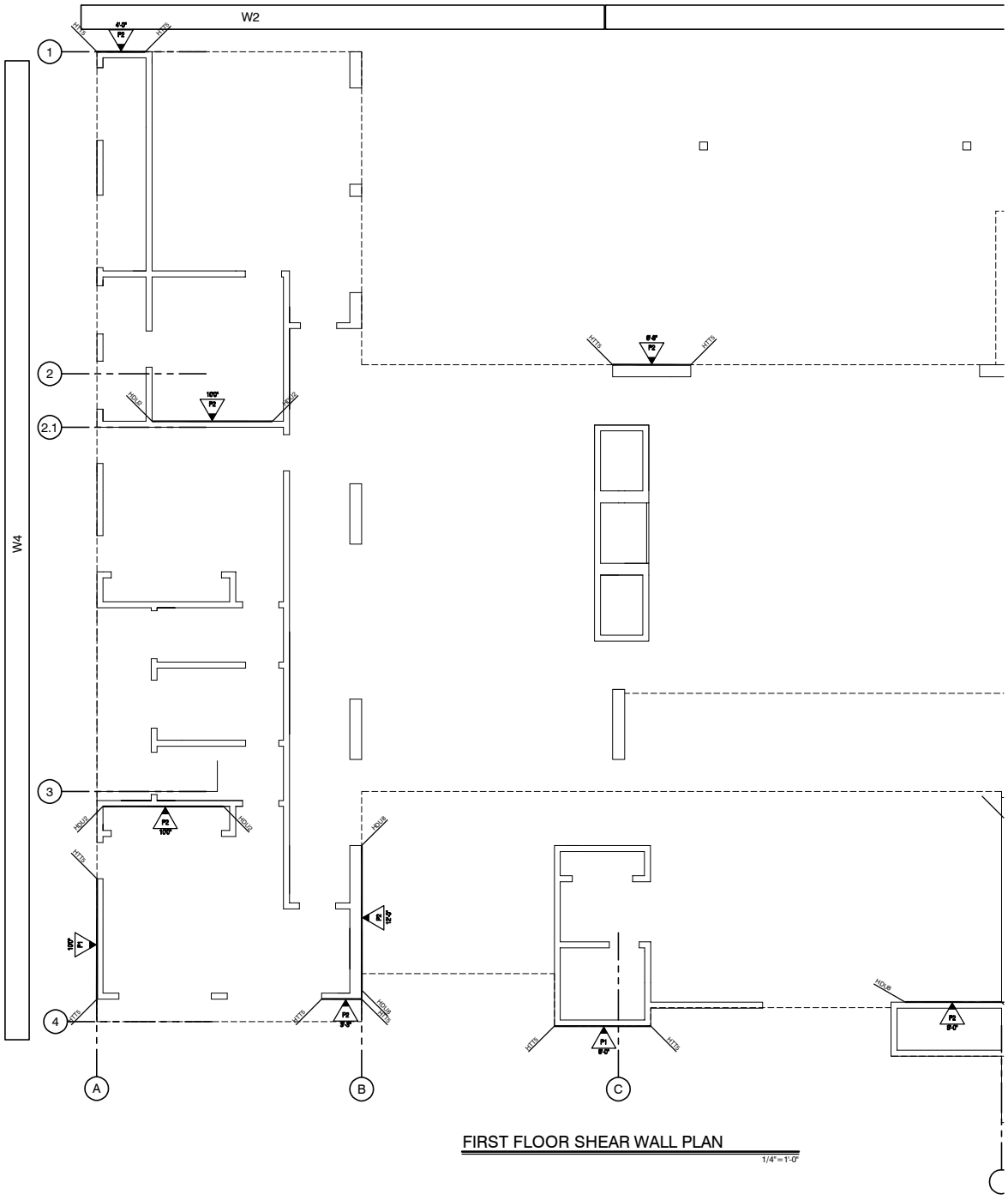
L. R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191

DATE 12/8/19

PROJECT ATHENS LOT 4 SHEET ____ OF ____

SUBJECT SHEAR WALL KEY PLAN DESIGNED KAB CHECKED ____



FIRST FLOOR SHEAR WALL PLAN

1/4" = 1'-0"

28 of 43
28 of 43

29 of 43

L. R. NELSON CONSULTING ENGINEERS, INC.

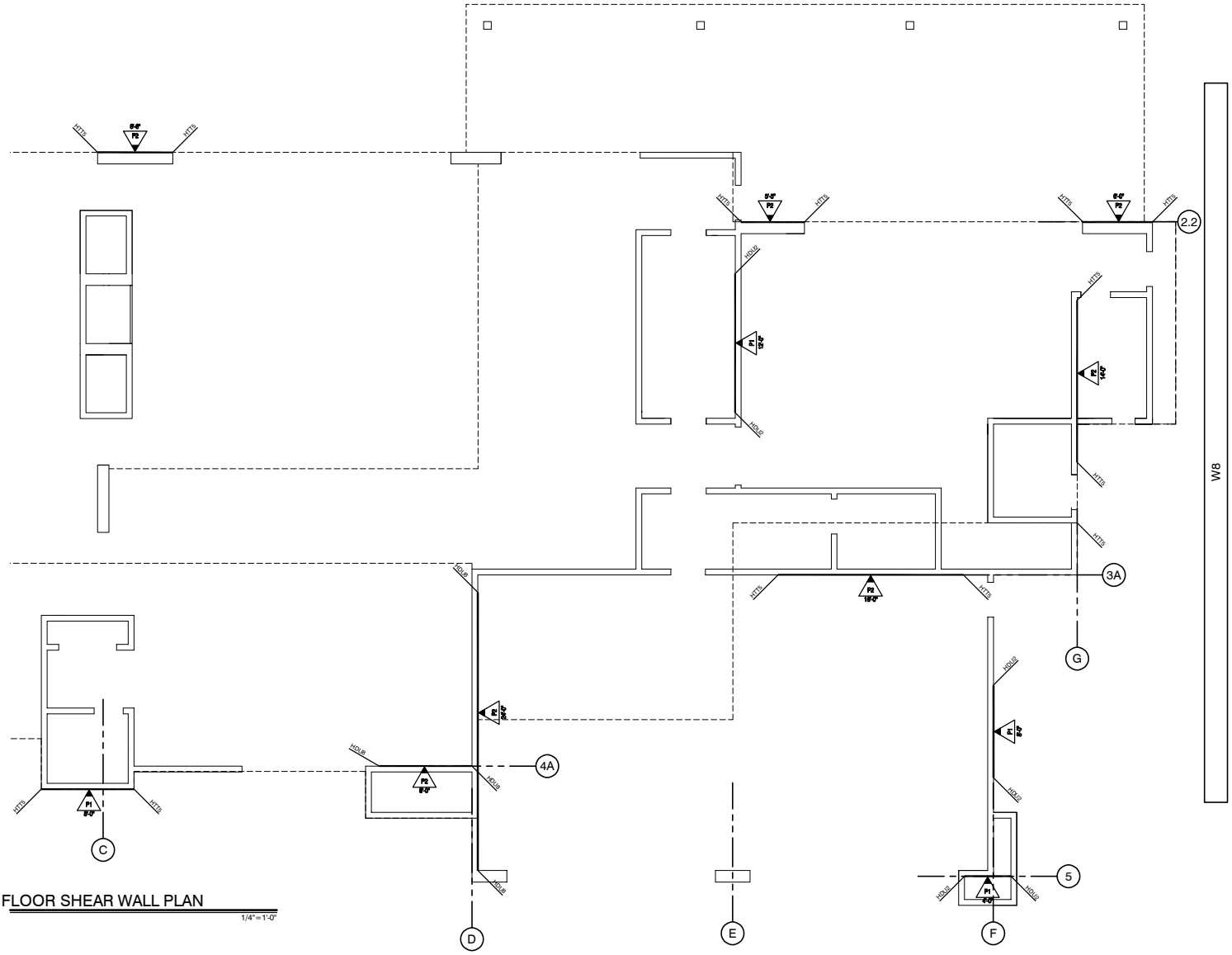
JOB NO. 1939-003-191

DATE 12/8/19

PROJECT ATHENS LOT 4 SHEET ____ OF ____

SUBJECT SHEAR WALL KEY PLAN DESIGNED KAB CHECKED ____

W6



FLOOR SHEAR WALL PLAN

1/4"=1'-0"

SEE SHEET S1.1 FOR FOUNDATION NOTES AND HOLDOWN SCHEDULE

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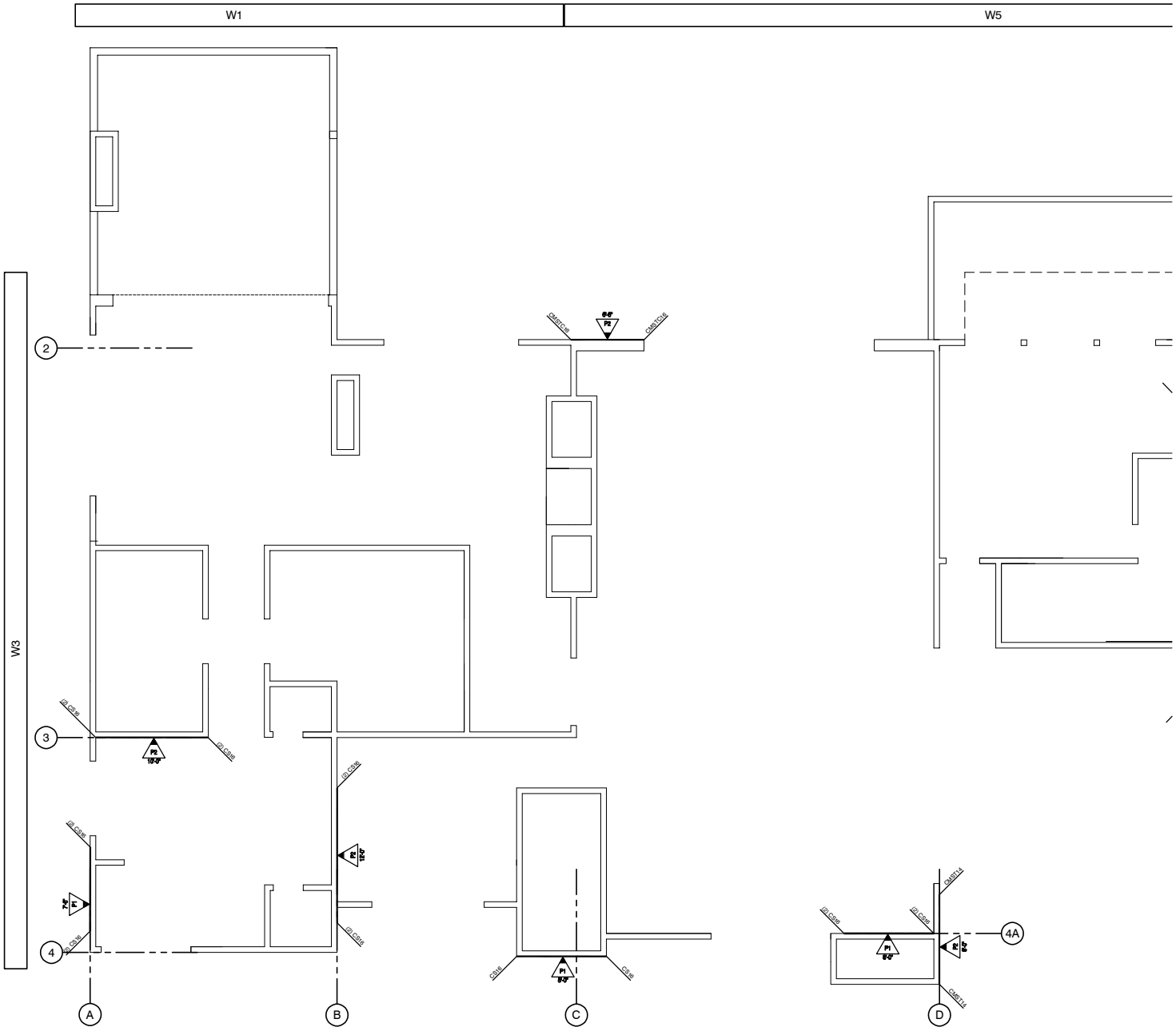
L. R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191

DATE 12/8/19

PROJECT ATHENS LOT 4 SHEET ____ OF ____

SUBJECT SHEAR WALL KEY PLAN DESIGNED KAB CHECKED ____



SECOND FLOOR SHEAR WALL PLAN

1/4" = 1'-0"

30 of 43
30 of 43

31 of 43

L. R. NELSON CONSULTING ENGINEERS, INC.

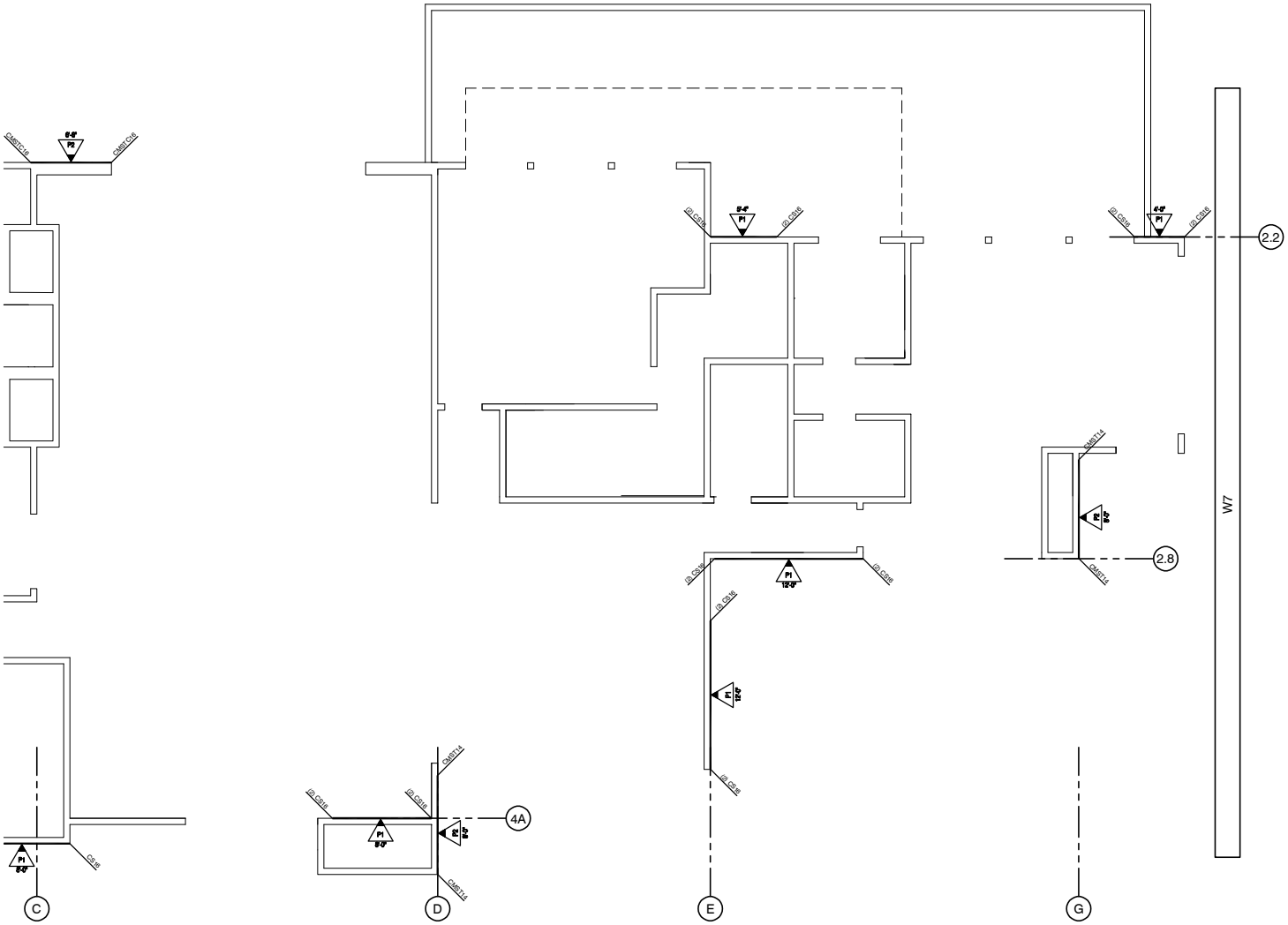
JOB NO. 1939-003-191

DATE 12/8/19

PROJECT ATHENS LOT 4 SHEET ____ OF ____

SUBJECT SHEAR WALL KEY PLAN DESIGNED KAB CHECKED ____

W5



R SHEAR WALL PLAN

1/4" = 1'-0"

SEE SHEET S1.1 FOR FOUNDATION NOTES AND HOLDOWN SCHEDULE

31 of 43
31 of 43

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **SECTION**
SUBJECT SHEARWALLS **DESIGNED** KB **CHECKED** KB

Roof DL (psf)= 15
Floor DL (psf)= 17
Wall DL (psf)= 12

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: 1 1ST STORY Total Length (ft)= 4 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω4	13.00	31.9	115.2	122.4						0							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	
		0.0	0.0	0.0						0							Total=	1591	415

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
4.00				15,912	4,153	576			P2	HTT5	Corner	398	426	104	304	3,834	894	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: 2 2ND STORY Total Length (ft)= 6.5 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω3	4.00	52.0	115.2	95.8						0							Wind (lb)	Seis (lb)
ω3	17.00	52.0	115.2	95.8						0							above=	
		0.0	0.0	0.0						0							Total=	2419	1092

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
6.50				24,192	10,919	1,521			P2	CMSTC16	Full	372	533	168	380	3,488	1,446	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: 2.1 1ST STORY Total Length (ft)= 10 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω4	13.00	31.9	115.2	122.4						0							Wind (lb)	Seis (lb)
ω4	17.00	31.9	115.2	122.4						0							above=	
		0.0	0.0	0.0						0							Total=	3672	958

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
10.00				36,720	9,584	3,600			P2	HDU2	Full	367	533	96	380	3,312	598	N.G.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: 3 2ND STORY Total Length (ft)= 10 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω3	9.00	52.0	115.2	95.8						0							Wind (lb)	Seis (lb)
ω3	17.00	52.0	115.2	95.8						0							above=	
		0.0	0.0	0.0						0							Total=	2995	1352

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
10.00				29,952	13,519	3,600			P2	(2) CS16	Full	300	533	135	380	2,635	992	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **SECTION**
SUBJECT SHEARWALLS **DESIGNED** KB **CHECKED** KB

Roof DL (psf)= 15
Floor DL (psf)= 17
Wall DL (psf)= 12

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: 3 1ST STORY Total Length (ft)= 10 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω4	9.00	31.9	115.2	122.4						0							Wind (lb)	Seis (lb)
ω4	17.00	31.9	115.2	122.4						0							above=	
		0.0	0.0	0.0						0							Total=	3182	831

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
10.00				31,824	8,306	3,600			P1	HDU2	Full	318	365	83	260	2,822	471	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: 4 2ND STORY Total Length (ft)= 8 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω3	12.00	52.0	115.2	95.8						0							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	
		0.0	0.0	0.0						0							Total=	1382	624

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
8.00				13,824	6,239	2,304			P1	CS16	Full	173	365	78	260	1,440	492	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:
PROVIDE HTT5 AT BASE OF WALL

LINE: 4 1ST STORY Total Length (ft)= 3.25 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω4	9.00	31.9	115.2	122.4						0							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	
		0.0	0.0	0.0						0							Total=	1102	288

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
3.25				11,016	2,875	380			P2	HTT5	Full	339	346	88	247	3,273	768	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: 2.2 2ND STORY Total Length (ft)= 9.33 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω7	16.00	99.5	115.2	95.8						0							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	
		0.0	0.0	0.0						0							Total=	1843	1592

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
4.00				7,902	6,827	576			P1	(2) CS16	Full	198	292	171	208	1,832	1,563	O.K.
5.33				10,530	9,097	1,023			P1	(2) CS16	Full	198	365	171	260	1,784	1,515	O.K.
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **SECTION**
SUBJECT SHEARWALLS **DESIGNED** KB **CHECKED** KB

Roof DL (psf)= 15
Floor DL (psf)= 17
Wall DL (psf)= 12

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: 2.2 1ST STORY Total Length (ft)= 11.33 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =							Wind (lb)	Seis (lb)
ω8	16.00	40.7	115.2	122.4						0								
		0.0	0.0	0.0						0							above=	1843	1592
		0.0	0.0	0.0						0							Total=	3802	2244

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
6.00				20,132	11,885	1,296	1,832	1,563	P2	HTT5	Full	336	533	198	380	4,971	3,328	O.K.
5.33				17,884	10,558	1,023	1,784	1,515	P2	HTT5	Full	336	533	198	380	4,947	3,304	O.K.
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: 2.8 2ND STORY Total Length (ft)= 12 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =							Wind (lb)	Seis (lb)
ω7	24.00	99.5	115.2	95.8						0								
		0.0	0.0	0.0						0							above=	
		0.0	0.0	0.0						0							Total=	2765	2389

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
12.00				27,648	23,886	5,184			P1	(2) CS16	Full	230	365	199	260	1,872	1,558	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: 3A 1ST STORY Total Length (ft)= 16 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =							Wind (lb)	Seis (lb)
ω8	24.00	40.7	115.2	122.4						0								
		0.0	0.0	0.0						0							above=	2765	2389
		0.0	0.0	0.0						0							Total=	5702	3366

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
16.00				57,024	33,665	9,217			P2	HTT5	Full	356	533	210	380	2,988	1,528	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: 4A 2ND STORY Total Length (ft)= 8 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =							Wind (lb)	Seis (lb)
ω7	14.00	99.5	115.2	95.8						0								
		0.0	0.0	0.0						0							above=	
		0.0	0.0	0.0						0							Total=	1613	1393

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
8.00				16,128	13,934	2,304			P1	(2) CS16	Full	202	365	174	260	1,728	1,454	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **SECTION**
SUBJECT SHEARWALLS **DESIGNED** KB **CHECKED** KB

Roof DL (psf)= 15
Floor DL (psf)= 17
Wall DL (psf)= 12

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: 4A 1ST STORY Total Length (ft)= 8 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =									
ω8	14.00	40.7	115.2	122.4						0								Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	1613	1393
		0.0	0.0	0.0						0							Total=	3326	1964

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
8.00				33,264	19,638	2,304	1,728	1,454	P2	HDU8	Full	416	533	245	380	5,598	3,620	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: 5 1ST STORY Total Length (ft)= 4 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =									
ω8	5.00	40.7	115.2	122.4						0								Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=		
		0.0	0.0	0.0						0							Total=	612	204

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
4.00				6,120	2,037	576			P1	HDU2	Full	153	292	51	208	1,386	365	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: A 2ND STORY Total Length (ft)= 7.5 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =									
ω1	11.00	69.5	115.2	155.5						0								Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=		
		0.0	0.0	0.0						0							Total=	1711	764

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
7.50				17,107	7,640	2,025			P1	(2) CS16	Full	228	365	102	260	2,011	749	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: A 1ST STORY Total Length (ft)= 10 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =									
ω2	11.00	41.8	115.2	146.2						0								Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	1711	764
		0.0	0.0	0.0						0							Total=	3318	1224

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
10.00				33,185	12,243	3,600	2,011	749	P1	HTT5	Corner	332	365	122	260	4,969	1,613	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **SECTION**
SUBJECT SHEARWALLS **DESIGNED** KB **CHECKED** KB

Roof DL (psf)= 15
Floor DL (psf)= 17
Wall DL (psf)= 12

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: B 2ND STORY Total Length (ft)= 12 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω1	20.00	69.5	115.2	155.5						0							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	
		0.0	0.0	0.0						0							Total=	3110	1389

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
12.00				31,104	13,891	5,184			P1	(2) CS16	Full	259	365	116	260	2,160	726	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: B 1ST STORY Total Length (ft)= 12 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω2	20.00	41.8	115.2	146.2						0							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	3110	1389
		0.0	0.0	0.0						0							Total=	6034	2226

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
12.00				60,336	22,260	5,184	2,160	726	P2	HDU8	Full	503	533	185	380	6,756	2,148	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: C 2ND STORY Total Length (ft)= 24 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω1	20.00	69.5	115.2	155.5						0							Wind (lb)	Seis (lb)
ω5	16.00	69.5	115.2	155.5						0							above=		
		0.0	0.0	0.0						0							Total=	5599	2500

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
12.00				27,994	12,502	5,184					Full	233		104		1,901	610	
12.00				27,994	12,502	5,184					Full	233		104		1,901	610	
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:
Concrete wall, 6" MIN W/ #5 AT 16" OCEW centered

LINE: C 1ST STORY Total Length (ft)= 24 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω2	20.00	41.8	115.2	146.2						0							Wind (lb)	Seis (lb)
ω5	16.00	69.5	115.2	155.5						0							above=	5599	2500
		0.0	0.0	0.0						0							Total=	11010	4449

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
12.00				55,051	22,243	5,184	5,599	2,500			Full	459		185		9,754	3,922	
12.00				55,051	22,243	5,184					Full	459		185		4,156	1,422	
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:
Concrete wall, 6" MIN W/ #5 AT 16" OCEW centered

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JOB NO. 1939-003-191
DATE 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **SECTION**
SUBJECT SHEARWALLS **DESIGNED** KB **CHECKED** KB

Roof DL (psf)= 15
Floor DL (psf)= 17
Wall DL (psf)= 12

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: D 2ND STORY Total Length (ft)= 8 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω5	27.00	69.5	115.2	155.5						0							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	
		0.0	0.0	0.0						0							Total=	4199	1875

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
8.00				41,990	18,753	2,304			P2	CMST14	Full	525	533	234	380	4,961	2,056	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: D 1ST STORY Total Length (ft)= 24 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω6	27.00	41.8	115.2	146.2						0							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	4199	1875
		0.0	0.0	0.0						0							Total=	8145	3005

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
24.00				81,454	30,050	20,737	4,961	2,056	P2	HDU8	Full	339	533	125	380	7,491	2,444	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: E 2ND STORY Total Length (ft)= 12 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω5	26.00	69.5	115.2	155.5						0							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	
		0.0	0.0	0.0						0							Total=	4044	1806

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
12.00				40,435	18,058	5,184			P1	(2) CS16	Full	337	365	150	260	2,938	1,073	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: E 1ST STORY Total Length (ft)= 12 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω6	26.00	41.8	115.2	146.2						0							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	
		0.0	0.0	0.0						0							Total=	3800	1088

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
12.00				38,002	10,879	5,184			P1	HDU2	Full	317	365	91	260	2,735	475	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **SECTION**
SUBJECT SHEARWALLS **DESIGNED** KB **CHECKED** KB

Roof DL (psf)= 15
Floor DL (psf)= 17
Wall DL (psf)= 12

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: F 1ST STORY Total Length (ft)= 8 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω6	15.00	41.8	115.2	146.2						0							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	
		0.0	0.0	0.0						0							Total=	2192	628

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
8.00				21,924	6,276	2,304			P1	HDU2	Full	274	365	78	260	2,452	497	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: G 2ND STORY Total Length (ft)= 8 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω5	23.00	69.5	115.2	155.5						0							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	
		0.0	0.0	0.0						0							Total=	3577	1597

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
8.00				35,770	15,975	2,304			P2	CMST14	Full	447	533	200	380	4,183	1,709	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: G 1ST STORY Total Length (ft)= 14 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω6	11.00	41.8	115.2	146.2						0							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						0							above=	3577	1597
		0.0	0.0	0.0						0							Total=	5185	2058

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
14.00				51,847	20,577	7,056	1,673	684	P2	HTT5	Full	370	533	147	380	4,873	1,649	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **OF**
SUBJECT DIAPRHAGM FORCES **DESIGNED** KB **CHECKED** KB

ROOF DIAPRAGM SHEAR: Min Diaphragm Width: 22 Eave Ht.

Location	Line Load	End Zone	Governing Force	Dist. Btwn. Shear lines (ft)	Diaph'm Depth (ft)	Diaph'm Shear (lb/ft)	Allowable Shear (lb/ft)	Diaphragm Construction	Case
1	ω1	None	WIND	22	60	28.5	232.3	15/32 shthg, 8d nails @ 6" o.c.	CASES 2-6
2	ω3	None	WIND	34	22	89.0	232.3	15/32 shthg, 8d nails @ 6" o.c.	CASES 2-6
3	ω5	None	WIND	34	25	105.8	232.3	15/32 shthg, 8d nails @ 6" o.c.	CASES 2-6
4	ω7	None	WIND	25	34	42.4	232.3	15/32 shthg, 8d nails @ 6" o.c.	CASES 2-6
5									
6									

Specific Gravity of Framing 0.42

FLOOR DIAPRAGM SHEAR: Min Diaphragm Width: 22 Eave Ht.

Location	Line Load	End Zone	Governing Force	Dist. Btwn. Shear lines (ft)	Diaph'm Depth (ft)	Diaph'm Shear (lb/ft)	Allowable Shear (lb/ft)	Diaphragm Construction	Case
7	ω2	None	WIND	22	60	26.8	276	23/32 shthg, 10d nails @ 6" o.c.	CASES 2-6
8	ω4	None	WIND	34	22	94.6	276	23/32 shthg, 10d nails @ 6" o.c.	CASES 2-6
9	ω6	None	WIND	34	25	99.4	276	23/32 shthg, 10d nails @ 6" o.c.	CASES 2-6
10	ω8	None	WIND	25	34	45.0	276	23/32 shthg, 10d nails @ 6" o.c.	CASES 2-6
11									
12									

Specific Gravity of Framing 0.42

CHORD FORCES:

Location	Tension (lbs)	Number of Plies	Grade	Size	Unity	16d Nails Req'd	Number Provided	Cd
1	156.8	2	DFL#2	2x4	0.01	0.830593	12	1.6
2	756.7	2	DFL#2	2x4	0.05	4.007704	12	1.6
3	898.9	2	DFL#2	2x4	0.06	4.761153	12	1.6
7	147.4	2	DFL#2	2x4	0.01	0.780604	12	1.6
8	803.9	2	DFL#2	2x4	0.06	4.258186	12	1.6
9	844.8	2	DFL#2	2x4	0.06	4.474602	12	1.6

Splice Connection: (12) 16d Sinkers

Minimum Splice per IBC 2304 is (8) 16d common Nails

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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **SECTION**
SUBJECT FOOTINGS - 1-story **DESIGNED** KB **CHECKED** KB

FOOTINGS

Assumed Soil Bearing Pressure

q=	2,000	psf not to exceed ------	2,000	psf
	0	psf/ft width greater than	0	in

Concrete Weight: 150 pcf
Concrete Strength: 2500 psi
Steel Grade 60000 psi

Bearing Pressure Spread Footings

Width (in)	Depth (in)	Loads (lb)	#4 Bars
12	12	1,850	2
18	12	4,163	2
24	12	7,400	3
30	12	11,563	4
36	12	16,650	4
42	12	22,663	5
48	12	29,600	5
54	16	36,450	8
60	16	45,000	9

Continuous Footings:

Width (in)	Loads (plf)	#4 Long. Bars	#4 Trans. Bars
12	1850	1	N.A.
18	2775	2	12 in. o.c.
24	3700	2	9 in. o.c.
30	4625	3	9 in. o.c.
36	5550	3	9 in. o.c.
42	6475	4	9 in. o.c.
48	7400	4	9 in. o.c.
54	8100	6	7 in. o.c.
60	9000	7	7 in. o.c.

(The number of longitudinal bars required as per ACI 318, Section 7.12)

Punching Shear

Footing Width	X_{COLUMN} (in)	Y_{COLUMN} (in)	Effective Depth (in)	A_p (in^2)	P_u (lbs)	v_u (psi)	v_c (psi)	Check
12	3	3.5	8.50	399.5	2960	0.3	150	OK
18	3	3.5	8.50	399.5	6660	9.6	150	OK
24	3	3.5	8.50	399.5	11840	22.5	150	OK
30	3	3.5	8.50	399.5	18500	39.2	150	OK
36	3	3.5	8.50	399.5	26640	59.6	150	OK
42	3	3.5	8.50	399.5	36260	83.7	150	OK
48	3	3.5	8.50	399.5	47360	111.4	150	OK
54	3	3.5	12.50	787.5	58320	67.8	150	OK
60	3	3.5	12.50	787.5	72000	85.1	150	OK

One-Way Shear

Footing Width	Effective Depth (in)	X_{min} (in)	e (in)	P_u (lbs)	q_u (psi)	vu (psi)	v_c (psi)	Check
12	8.5	3	0	2960	20.6	0.0	75	OK
18	8.5	3	0	6660	20.6	0.0	75	OK
24	8.5	3	2	11840	20.6	4.8	75	OK
30	8.5	3	5	18500	20.6	12.1	75	OK
36	8.5	3	8	26640	20.6	19.3	75	OK
42	8.5	3	11	36260	20.6	26.6	75	OK
48	8.5	3	14	47360	20.6	33.9	75	OK
54	12.5	3	13	58320	20.0	20.8	75	OK
60	12.5	3	16	72000	20.0	25.6	75	OK

Bending

Footing Width	Effective Depth (in)	X_{min} (in)	q_u (psi)	M_u (lb-in)	Required A_s (in^2)	Min A_s (in^2)	#4 Bars	#5 Bars
12	8.5	3	20.6	2498	0.01	0.20	2	2
18	8.5	3	20.6	10406	0.02	0.30	2	2
24	8.5	3	20.6	27195	0.06	0.52	3	2
30	8.5	3	20.6	56194	0.13	0.65	4	2
36	8.5	3	20.6	100733	0.23	0.78	4	3
42	8.5	3	20.6	164141	0.38	0.91	5	3
48	8.5	3	20.6	249750	0.59	1.04	5	4
54	12.5	3	20.0	351135	0.56	1.56	8	5
60	12.5	3	20.0	487350	0.78	1.73	9	6

(Minimum reinforcement per ACI 318, Section 10.5)

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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-003-191
DATE 12/06/19

PROJECT CUSTOM RESIDENCE **SHEET** **SECTION**
SUBJECT FOOTINGS 2-story **DESIGNED** KB **CHECKED** KB

FOOTINGS

Assumed Soil Bearing Pressure

q=	2,500	psf not to exceed ------	2,500	psf
	0	psf/ft width greater than	0	in

Concrete Weight: 150 pcf
Concrete Strength: 2500 psi
Steel Grade 60000 psi

Bearing Pressure Spread Footings

Width (in)	Depth (in)	Loads (lb)	#4 Bars
12	12	2,350	2
18	12	5,288	2
24	12	9,400	3
30	12	14,688	4
36	12	21,150	4
42	12	28,788	5
48	12	37,600	5
54	16	46,575	8
60	16	57,500	9

Continuous Footings:

Width (in)	Loads (plf)	#4 Long. Bars	#4 Trans. Bars
12	2350	1	N.A.
18	3525	2	12 in. o.c.
24	4700	2	9 in. o.c.
30	5875	3	9 in. o.c.
36	7050	3	9 in. o.c.
42	8225	4	9 in. o.c.
48	9400	4	9 in. o.c.
54	10350	6	7 in. o.c.
60	11500	7	7 in. o.c.

(The number of longitudinal bars required as per ACI 318, Section 7.12)

Punching Shear

Footing Width	X_{COLUMN} (in)	Y_{COLUMN} (in)	Effective Depth (in)	A_p (in^2)	P_u (lbs)	v_u (psi)	v_c (psi)	Check
12	3	3.5	8.50	399.5	3760	0.4	150	OK
18	3	3.5	8.50	399.5	8460	12.2	150	OK
24	3	3.5	8.50	399.5	15040	28.6	150	OK
30	3	3.5	8.50	399.5	23500	49.8	150	OK
36	3	3.5	8.50	399.5	33840	75.7	150	OK
42	3	3.5	8.50	399.5	46060	106.3	150	OK
48	3	3.5	8.50	399.5	60160	141.6	150	OK
54	3	3.5	12.50	787.5	74520	86.6	150	OK
60	3	3.5	12.50	787.5	92000	108.8	150	OK

One-Way Shear

Footing Width	Effective Depth (in)	X_{min} (in)	e (in)	P_u (lbs)	q_u (psi)	vu (psi)	v_c (psi)	Check
12	8.5	3	0	3760	26.1	0.0	75	OK
18	8.5	3	0	8460	26.1	0.0	75	OK
24	8.5	3	2	15040	26.1	6.1	75	OK
30	8.5	3	5	23500	26.1	15.4	75	OK
36	8.5	3	8	33840	26.1	24.6	75	OK
42	8.5	3	11	46060	26.1	33.8	75	OK
48	8.5	3	14	60160	26.1	43.0	75	OK
54	12.5	3	13	74520	25.6	26.6	75	OK
60	12.5	3	16	92000	25.6	32.7	75	OK

Bending

Footing Width	Effective Depth (in)	X_{min} (in)	q_u (psi)	M_u (lb-in)	Required A_s (in^2)	Min A_s (in^2)	#4 Bars	#5 Bars
12	8.5	3	26.1	3173	0.01	0.20	2	2
18	8.5	3	26.1	13219	0.03	0.30	2	2
24	8.5	3	26.1	34545	0.08	0.52	3	2
30	8.5	3	26.1	71381	0.17	0.65	4	2
36	8.5	3	26.1	127958	0.30	0.78	4	3
42	8.5	3	26.1	208504	0.49	0.91	5	3
48	8.5	3	26.1	317250	0.75	1.04	5	4
54	12.5	3	25.6	448673	0.71	1.56	8	5
60	12.5	3	25.6	622725	1.00	1.73	9	6

(Minimum reinforcement per ACI 318, Section 10.5)

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L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191
DATE 12/6/2019

PROJECT: CUSTOM RESIDENCE SHEET OF
SUBJECT: ALLOWABLE ANCHOR BOLT LOADS DESIGNED KB CHECKED KB

A307 Anchor Bolts - 1/2" Diameter

**Assumes 1/2" diameter A307 anchor bolts in a 1 1/2" Dougleas-Fir larch bottom plate and 6" (min) embed, parallel to grain*
**Assumes wood controls*

Z_{II} = 650 lbs/bolt C_D = 1.6 C_t = 1.0
$Z'_{II} = Z*C_D*C_M*C_t$ SG = 0.5 C_M = 1.0
= (650lbs/nail)(1.6)(1.0)(1.0)
= 1040 lbs/nail

1/2" Anchor Bolts at 72 in o.c.	⇒	1040 *lbs/bolt* ×	0.2	nails/ft =	**173**	***lbs/ft***
1/2" Anchor Bolts at 36 in o.c.	⇒	1040 *lbs/bolt* ×	0.3	nails/ft =	**347**	***lbs/ft***
1/2" Anchor Bolts at 32 in o.c.	⇒	1040 *lbs/bolt* ×	0.4	nails/ft =	**390**	***lbs/ft***
1/2" Anchor Bolts at 28 in o.c.	⇒	1040 *lbs/bolt* ×	0.4	nails/ft =	**446**	***lbs/ft***
1/2" Anchor Bolts at 24 in o.c.	⇒	1040 *lbs/bolt* ×	0.5	nails/ft =	**520**	***lbs/ft***
1/2" Anchor Bolts at 20 in o.c.	⇒	1040 *lbs/bolt* ×	0.6	nails/ft =	**624**	***lbs/ft***
1/2" Anchor Bolts at 16 in o.c.	⇒	1040 *lbs/bolt* ×	0.8	nails/ft =	**780**	***lbs/ft***
1/2" Anchor Bolts at 12 in o.c.	⇒	1040 *lbs/bolt* ×	1	nails/ft =	**1040**	***lbs/ft***
1/2" Anchor Bolts at 9 in o.c.	⇒	1040 *lbs/bolt* ×	1.3	nails/ft =	**1387**	***lbs/ft***

A307 Anchor Bolts - 5/8" Diameter

**Assumes 5/8" diameter A307 anchor bolts in a 1 1/2" Douglas-Fir larch bottom plate and 6" (min) embed, parallel to grain*
**Assumes wood controls*

Z_{II} = 930 lbs/bolt C_D = 1.6 C_t = 1.0
$Z'_{II} = Z*C_D*C_M*C_t$ SG = 0.5 C_M = 1.0
= (930lbs/nail)(1.6)(1.0)(1.0)
= 1488 lbs/nail

5/8" Anchor Bolts at 72 in o.c.	⇒	1488 *lbs/bolt* ×	0.2	nails/ft =	**248**	***lbs/ft***
5/8" Anchor Bolts at 36 in o.c.	⇒	1488 *lbs/bolt* ×	0.3	nails/ft =	**496**	***lbs/ft***
5/8" Anchor Bolts at 32 in o.c.	⇒	1488 *lbs/bolt* ×	0.4	nails/ft =	**558**	***lbs/ft***
5/8" Anchor Bolts at 28 in o.c.	⇒	1488 *lbs/bolt* ×	0.4	nails/ft =	**638**	***lbs/ft***
5/8" Anchor Bolts at 24 in o.c.	⇒	1488 *lbs/bolt* ×	0.5	nails/ft =	**744**	***lbs/ft***
5/8" Anchor Bolts at 20 in o.c.	⇒	1488 *lbs/bolt* ×	0.6	nails/ft =	**893**	***lbs/ft***
5/8" Anchor Bolts at 16 in o.c.	⇒	1488 *lbs/bolt* ×	0.8	nails/ft =	**1116**	***lbs/ft***
5/8" Anchor Bolts at 12 in o.c.	⇒	1488 *lbs/bolt* ×	1	nails/ft =	**1488**	***lbs/ft***
5/8" Anchor Bolts at 9 in o.c.	⇒	1488 *lbs/bolt* ×	1.3	nails/ft =	**1984**	***lbs/ft***

A307 Anchor Bolts - 5/8" Diameter

**Assumes 5/8" diameter A307 anchor bolts in a 2 1/2" Douglas-Fir larch bottom plate and 6" (min) embed, parallel to grain*
**Assumes wood controls*

Z_{II} = 1180 lbs/bolt C_D = 1.6 C_t = 1.0
$Z'_{II} = Z*C_D*C_M*C_t$ SG = 0.5 C_M = 1.0
= (1180lbs/nail)(1.6)(1.0)(1.0)
= 1888 lbs/nail

5/8" Anchor Bolts at 72 in o.c.	⇒	1888 *lbs/bolt* ×	0.2	nails/ft =	**315**	***lbs/ft***
5/8" Anchor Bolts at 36 in o.c.	⇒	1888 *lbs/bolt* ×	0.3	nails/ft =	**629**	***lbs/ft***
5/8" Anchor Bolts at 32 in o.c.	⇒	1888 *lbs/bolt* ×	0.4	nails/ft =	**708**	***lbs/ft***
5/8" Anchor Bolts at 28 in o.c.	⇒	1888 *lbs/bolt* ×	0.4	nails/ft =	**809**	***lbs/ft***
5/8" Anchor Bolts at 24 in o.c.	⇒	1888 *lbs/bolt* ×	0.5	nails/ft =	**944**	***lbs/ft***
5/8" Anchor Bolts at 20 in o.c.	⇒	1888 *lbs/bolt* ×	0.6	nails/ft =	**1133**	***lbs/ft***
5/8" Anchor Bolts at 16 in o.c.	⇒	1888 *lbs/bolt* ×	0.8	nails/ft =	**1416**	***lbs/ft***
5/8" Anchor Bolts at 12 in o.c.	⇒	1888 *lbs/bolt* ×	1	nails/ft =	**1888**	***lbs/ft***
5/8" Anchor Bolts at 9 in o.c.	⇒	1888 *lbs/bolt* ×	1.3	nails/ft =	**2517**	***lbs/ft***

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-003-191
DATE 12/6/2019

PROJECT: CUSTOM RESIDENCE **SHEET** **OF**
SUBJECT: ALLOWABLE ANCHOR BOLT LOADS **DESIGNED** KB **CHECKED** KB

A307 Anchor Bolts - 1/2" Diameter

**Assumes 1/2" diameter A307 anchor bolts in a 1 1/2" Douglas-Fir larch bottom plate and 6" (min) embed, perpendicular to grain*
**Assumes wood controls*

$Z_\perp$ = 380 lbs/bolt C_D = 1.6 C_t = 1.0
$Z'_\perp = Z*C_D*C_M*C_t$ SG = 0.5 C_M = 1.0
= (380lbs/nail)(1.6)(1.0)(1.0)
= 608 lbs/nail

1/2" Anchor Bolts at 72 in o.c.	⇒	608 *lbs/bolt* ×	0.2	nails/ft =	**101**	***lbs/ft***
1/2" Anchor Bolts at 36 in o.c.	⇒	608 *lbs/bolt* ×	0.3	nails/ft =	**203**	***lbs/ft***
1/2" Anchor Bolts at 32 in o.c.	⇒	608 *lbs/bolt* ×	0.4	nails/ft =	**228**	***lbs/ft***
1/2" Anchor Bolts at 28 in o.c.	⇒	608 *lbs/bolt* ×	0.4	nails/ft =	**261**	***lbs/ft***
1/2" Anchor Bolts at 24 in o.c.	⇒	608 *lbs/bolt* ×	0.5	nails/ft =	**304**	***lbs/ft***
1/2" Anchor Bolts at 20 in o.c.	⇒	608 *lbs/bolt* ×	0.6	nails/ft =	**365**	***lbs/ft***
1/2" Anchor Bolts at 16 in o.c.	⇒	608 *lbs/bolt* ×	0.8	nails/ft =	**456**	***lbs/ft***
1/2" Anchor Bolts at 12 in o.c.	⇒	608 *lbs/bolt* ×	1	nails/ft =	**608**	***lbs/ft***
1/2" Anchor Bolts at 9 in o.c.	⇒	608 *lbs/bolt* ×	1.3	nails/ft =	**811**	***lbs/ft***

A307 Anchor Bolts - 5/8" Diameter

**Assumes 5/8" diameter A307 anchor bolts in a 1 1/2" Douglas-Fir larch bottom plate and 6" (min) embed, perpendicular to grain*
**Assumes wood controls*

$Z_\perp$ = 530 lbs/bolt C_D = 1.6 C_t = 1.0
$Z'_\perp = Z*C_D*C_M*C_t$ SG = 0.5 C_M = 1.0
= (530lbs/nail)(1.6)(1.0)(1.0)
= 848 lbs/nail

5/8" Anchor Bolts at 72 in o.c.	⇒	848 *lbs/bolt* ×	0.2	nails/ft =	**141**	***lbs/ft***
5/8" Anchor Bolts at 36 in o.c.	⇒	848 *lbs/bolt* ×	0.3	nails/ft =	**283**	***lbs/ft***
5/8" Anchor Bolts at 32 in o.c.	⇒	848 *lbs/bolt* ×	0.4	nails/ft =	**318**	***lbs/ft***
5/8" Anchor Bolts at 28 in o.c.	⇒	848 *lbs/bolt* ×	0.4	nails/ft =	**363**	***lbs/ft***
5/8" Anchor Bolts at 24 in o.c.	⇒	848 *lbs/bolt* ×	0.5	nails/ft =	**424**	***lbs/ft***
5/8" Anchor Bolts at 20 in o.c.	⇒	848 *lbs/bolt* ×	0.6	nails/ft =	**509**	***lbs/ft***
5/8" Anchor Bolts at 16 in o.c.	⇒	848 *lbs/bolt* ×	0.8	nails/ft =	**636**	***lbs/ft***
5/8" Anchor Bolts at 12 in o.c.	⇒	848 *lbs/bolt* ×	1	nails/ft =	**848**	***lbs/ft***
5/8" Anchor Bolts at 9 in o.c.	⇒	848 *lbs/bolt* ×	1.3	nails/ft =	**1131**	***lbs/ft***

A307 Anchor Bolts - 5/8" Diameter

**Assumes 5/8" diameter A307 anchor bolts in a 2 1/2" Douglas-Fir larch bottom plate and 6" (min) embed, perpendicular to grain*
**Assumes wood controls*

$Z_\perp$ = 610 lbs/bolt C_D = 1.6 C_t = 1.0
$Z'_\perp = Z*C_D*C_M*C_t$ SG = 0.5 C_M = 1.0
= (610lbs/nail)(1.6)(1.0)(1.0)
= 976 lbs/nail

5/8" Anchor Bolts at 72 in o.c.	⇒	976 *lbs/bolt* ×	0.2	nails/ft =	**163**	***lbs/ft***
5/8" Anchor Bolts at 36 in o.c.	⇒	976 *lbs/bolt* ×	0.3	nails/ft =	**325**	***lbs/ft***
5/8" Anchor Bolts at 32 in o.c.	⇒	976 *lbs/bolt* ×	0.4	nails/ft =	**366**	***lbs/ft***
5/8" Anchor Bolts at 28 in o.c.	⇒	976 *lbs/bolt* ×	0.4	nails/ft =	**418**	***lbs/ft***
5/8" Anchor Bolts at 24 in o.c.	⇒	976 *lbs/bolt* ×	0.5	nails/ft =	**488**	***lbs/ft***
5/8" Anchor Bolts at 20 in o.c.	⇒	976 *lbs/bolt* ×	0.6	nails/ft =	**586**	***lbs/ft***
5/8" Anchor Bolts at 16 in o.c.	⇒	976 *lbs/bolt* ×	0.8	nails/ft =	**732**	***lbs/ft***
5/8" Anchor Bolts at 12 in o.c.	⇒	976 *lbs/bolt* ×	1	nails/ft =	**976**	***lbs/ft***
5/8" Anchor Bolts at 9 in o.c.	⇒	976 *lbs/bolt* ×	1.3	nails/ft =	**1301**	***lbs/ft***

ABBREVIATIONS	
AB	ANCHOR BOLT
ABV	ABOVE
ADJ	ADJACENT
AFF	ABOVE FINISH FLOOR
AGGR	AGGREGATE
ALT	ALTERNATE
ALUM	ALUMINUM
APPROX	APPROXIMATELY
ARCH'L	ARCHITECTURAL
BD	BOARD
BLDG	BUILDING
BLK'G	BLOCKING
BM	BEAM
BN	BOUNDARY NAILING
BO	BOTTOM OF
BOTT	BOTTOM
BRG	BEARING
BTWN	BETWEEN
CAM	CAMBER
CANT'L	CANTILEVERED
CTR	CENTERED
CL	CENTERLINE
CLR	CLEAR
CMU	CONC MASONRY UNIT
COL	COLUMN
CONC	CONCRETE
CONN	CONNECTION
CONST	CONSTRUCTION
CONT	CONTINUOUS
d	PENNY
D/DIA	DIAMETER
DBL	DOUBLE
DEPT	DEPARTMENT
DFL	DOUGLAS FIR LARCH
DIAG	DIAGONAL
DTL	DETAIL
DIA	DIAMETER
DIM	DIMENSION
DN	DOWN
DWG	DRAWING
DWL	DOWEL
EA	EACH
EJ	EXPANSION JOINT
ELEV	ELEVATION
ELEC	ELECTRIC
EN	EDGE NAILING
ENCL	ENCLOSED
EOR	ENGINEER OF RECORD
EQ	EQUAL
EQUIP	EQUIPMENT
EXIST	EXISTING
EXP	EXPANSION
EXT	EXTERIOR
EXTD	EXTENDED
FD	FLOOR DRAIN
FND/FDN	FOUNDATION
FF	FINISH FLOOR
FG	FINISH GRADE
FIN	FINISH
FLR	FLOOR
FRMG	FRAMING
FT	FOOT OR FEET
FTG	FOOTING
FURR	FURRING
GA	GAUGE
GLB	GLU-LAM BEAM
GRND	GROUND
GT	GIRDER TRUSS
GYP	GYPSUM WALLBOARD
HDR	HEADER
HGR	HANGER
HGT HT	HEIGHT
HORIZ	HORIZONTAL
HTR	HIP TRUSS
INSUL	INSULATION
INT	INTERIOR
JT	JOINT
LAM	LAMINATE
LAT	LATERAL
LBS	POUNDS
LSL	LAM STRAND LUMBER
LT	LIGHT
LVL	LAM VENEER LUMBER
LW	LONG WAY
MAS	MASONRY
MAX	MAXIMUM
MECH	MECHANICAL
MFR	MANUFACTURER
MFR'D	MANUFACTURED
MIN	MINIMUM
MISC	MISCELLANEOUS
NO	NUMBER
N	COMMON NAIL
NTS	NOT TO SCALE
O/	OVER
OC	ON CENTER
OH	OPPOSITE HAND
OPN'G	OPENING
PFA	POST FROM ABOVE
PL	PLATE
PLYWD	PLYWOOD
PNL	PANEL
PREFAB	PREFABRICATED
PSL	PARALLEL STRAND LUMBER
PSI	LBS PER SQ INCH
PT	POST-TENSIONED
#	POUND
r/RAD	RADIUS
REF	REFERENCE
REINF	REINFORCING
REQ'D	REQUIRED
S	SINKER NAIL
SCHED	SCHEDULE
SF	SQUARE FEET
SECT	SECTION
SHTHG	SHEATHING
SIM	SIMILAR
SPEC	SPECIFICATIONS
SQ	SQUARE
STD	STANDARD
STG'D	STAGGERED
STL	STEEL
STRUCT	STRUCTURAL
SYM	SYMMETRICAL
SWL	SHEAR WALL
SPN	SOLE PLATE NAILING
SW	SHORT WAY
T&G	TONGUE AND GROOVE
T&B	TOP AND BOTTOM
THK	THICK
TO	TOP OF
TRIMM	TRIMMER
TR	TRUSS
TS	TUBE STEEL
TS LSL	TIMBERSTRAND BEAM
TYP	TYPICAL
UNO	UNLESS NOTED OTHERWISE
VERT	VERTICAL
W/ W/O	WITH WITHOUT
WS	WOOD SCREW
WT	WEIGHT

STRUCTURAL GENERAL NOTES

GENERAL

1. ALL WORK SHALL CONFORM TO THE MINIMUM STANDARDS OF THE FOLLOWING CODE: THE INTERNATIONAL BUILDING CODE, 2018 EDITION, OTHER REGULATING AGENCIES WHICH HAVE AUTHORITY OVER ANY PORTION OF THE WORK, AND THOSE CODES AND STANDARDS LISTED IN THESE NOTES AND SPECIFICATIONS.
2. THE STRUCTURAL DRAWINGS AND SPECIFICATIONS REPRESENT THE FINISHED STRUCTURE. THEY DO NOT INDICATE THE METHOD OF CONSTRUCTION. THE CONTRACTOR SHALL PROVIDE ALL MEASURES NECESSARY TO PROTECT THE STRUCTURE DURING CONSTRUCTION. SUCH MEASURES SHALL INCLUDE, BUT NOT BE LIMITED TO, BRACING, SHORING FOR LOADS DUE TO CONSTRUCTION EQUIPMENT, ETC. OBSERVATION VISITS TO THE SITE BY THE STRUCTURAL ENGINEER SHALL NOT INCLUDE INSPECTIONS OF THE ABOVE ITEMS.
3. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS PRIOR TO STARTING CONSTRUCTION. THE ARCHITECT SHALL BE NOTIFIED OF ANY DISCREPANCIES OR INCONSISTENCIES.
4. DIMENSIONS SHALL TAKE PRECEDENCE OVER THE SCALE SHOWN ON DRAWINGS.
5. NOTES AND DETAILS ON PLANS SHALL TAKE PRECEDENCE OVER GENERAL NOTES AND TYPICAL DETAILS.
6. SEE ARCHITECTURAL PLANS FOR THE FOLLOWING UNO:
   - A. SIZE AND LOCATION OF ALL DOOR AND WINDOW OPENINGS.
   - B. SIZE AND LOCATION OF ALL INTERIOR AND EXTERIOR NON-BEARING PARTITIONS.
   - C. SIZE AND LOCATION OF ALL CONCRETE CURBS, FLOOR DRAINS, SLOPES, DEPRESSED AREAS, CHANGES IN LEVEL, CHAMFER, GROOVES, INSERTS, ETC.
   - D. SIZE AND LOCATION OF FLOOR AND ROOF OPENINGS.
   - E. FLOOR AND ROOF FINISHES.
   - F. STAIR FRAMING AND DETAILS.
   - G. DIMENSIONS NOT SHOWN ON STRUCTURAL PLANS.
7. SEE MECHANICAL, PLUMBING, AND ELECTRICAL PLANS FOR THE FOLLOWING:
   - A. PIPE RUNS, SLEEVES, HANGERS, TRENCHES, WALL AND SLAB OPENINGS, ETC., (EXCEPT AS SHOWN OR NOTED).
   - B. ELECTRICAL CONDUIT RUNS, BOXES, OUTLETS IN WALLS AND SLABS.
   - C. CONCRETE INSERTS FOR ELECTRICAL, MECHANICAL, OR PLUMBING FIXTURES.
   - D. SIZE AND LOCATION OF MACHINE EQUIPMENT BASES, OR ANCHOR BOLTS FOR MOUNTS.
   - E. SIZE AND LOCATION OF ALL MECHANICAL UNITS.
8. OPENINGS, POCKETS, ETC. LARGER THAN 6 INCHES SHALL NOT BE PLACED IN SLABS, DECKS, BEAMS, JOISTS, COLUMNS, WALLS, ETC., UNLESS SPECIFICALLY DETAILED ON THE STRUCTURAL PLANS.
9. ASTM SPECIFICATIONS NOTED SHALL BE THE LATEST REVISION
10. THE CONTRACTOR SHALL INVESTIGATE THE SITE DURING CLEARING AND EARTHWORK OPERATIONS FOR FILLED EXCAVATIONS OR BURIED STRUCTURES SUCH AS CESSPOOLS, CISTERNS, FOUNDATIONS, ETC. IF ANY SUCH STRUCTURES ARE FOUND, THE ENGINEER OF RECORD SHALL BE NOTIFIED IMMEDIATELY.
11. CONSTRUCTION MATERIALS SHALL BE SPREAD OUT IF PLACED ON FLOORS OR ROOF. LOAD SHALL NOT EXCEED THE DESIGN LIVE LOAD PER SQUARE FOOT. PROVIDE ADEQUATE SHORING AND/OR BRACING WHERE STRUCTURE HAS NOT ATTAINED DESIGN STRENGTH.
12. WHERE THE LONGEST HORIZONTAL CEILING DIMENSION IS EQUAL TO OR GREATER THAN 20'-0", IT IS RECOMMENDED THAT RESILIENT CHANNEL BE USED TO HELP LIMIT DRYWALL CRACKING.

DESIGN CRITERIA

	LIVE LOAD	DEAD LOAD
FLOOR LOAD	40 PSF	15 PSF
ROOF LOAD	20 PSF	22 PSF
ROOF SNOW LOAD	≤ 10 PSF	22 PSF
STAIR & EXIT LOAD	----	----
STORAGE LOAD	----	----

* SEE FRAMING NOTES FOR SPECIAL LOADING CONDITIONS.

BASIC WIND SPEED	100 MPH (LRFD), 78 MPH (ASD)	
WIND RISK CATEGORY	II	
WIND EXPOSURE CATEGORY	C	
INTERNAL PRESSURE COEFFICIENT	± 0.18	
SEISMIC RISK CATEGORY & IMP0RTANCE	II	I_E = 1.0
MAPPED SPECTRAL RESPONSE ACCELERATIONS	S_S = 0.498 & S_1 = 0.162	
SPECTRAL RESPONSE COEFFICIENTS	S_{DS} = 0.398 & S_{D1} = 0.176	
SITE CLASS	C	
LATERAL FORCE RESISTING SYSTEM	LIGHT FRAME WALLS WITH SHEAR PANELS	R = 6.5
SEISMIC RESPONSE COEFFICIENT, C_S	0.066	
DESIGN BASE SHEAR	C_S x DEAD WEIGHT(W)	
ANALYSIS PROCEDURE	EQUIVALENT LATERAL FORCE	
SEISMIC DESIGN CATEGORY	C	

FOUNDATION

1. THIS IS TO CERTIFY THE FOUNDATION DEPICTED HEREIN HAS BEEN DESIGNED IN ACCORDANCE WITH RECOGNIZED ENGINEERING PRACTICE FOR CONDITIONS AS CLASSIFIED BY THE PROJECT GEOTECHNICAL REPORT:
   CONSULTANT: DU PONT ENGINEERING INC
   PROJECT NO: 18-0414
   DATED: SEPTEMBER 29, 2018
   UPDATED: FEBRUARY 21, 2019
2. FOOTINGS ARE DESIGNED BASED ON AN ALLOWABLE BEARING PRESSURE OF 2000 PSF PER SOIL REPORT.
3. SOILS PREPARATION AND FOUNDATION CONSTRUCTION SHALL CONFORM TO THE REQUIREMENTS OF THE GEOTECHNICAL REPORT.
4. SEE ARCHITECTURAL AND STRUCTURAL DRAWINGS FOR EXACT LOCATION OF BULKHEADS AND OPENINGS, ETC.
5. THE CONTRACTOR SHALL PROVIDE FOR PROPER DE-WATERING OF EXCAVATIONS FROM SURFACE WATER, GROUND WATER, SEEPAGE, ETC. (FOOTINGS SHALL NOT BE PLACED UNDER WATER).
6. FOOTINGS SHALL BE PLACED ACCORDING TO DEPTHS SHOWN ON THE STRUCTURAL PLANS. ALL ABANDONED FOOTINGS, UTILITIES, ETC. THAT INTERFERE WITH NEW CONSTRUCTION SHALL BE REMOVED.
7. CONCRETE PLACEMENT SHALL BE IN ONE CONTINUOUS OPERATION UNLESS OTHERWISE SPECIFIED AND SLAB SURFACE SHALL BE CURED WITH HUNTS COMPOUND OR EQUAL.
8. FOOTING BACKFILL AND UTILITY TRENCH BACKFILL WITHIN BUILDING AREA SHALL BE PER THE REQUIREMENTS OF THE GEOTECHNICAL REPORT. FLOODING WILL NOT BE PERMITTED.
9. STOOPS, PORCHES, OR OTHER ATTACHMENTS SHALL BE CAST INDEPENDENT OF THE CONCRETE FOUNDATION SLAB, UNO.

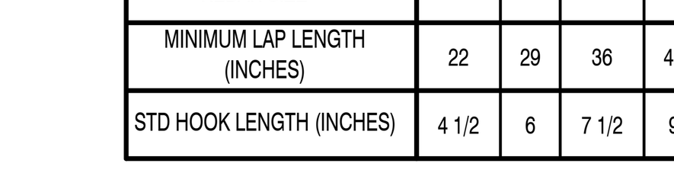
FOUNDATION HARDWARE

1. THICKEN SLAB AS REQUIRED FOR CONCRETE COVERAGE AT ANCHOR BOLTS PER ACI 318 WHERE OCCURS. THE CONCRETE CONTRACTOR SHALL VERIFY LOCATION OF ALL BOLTS, TIE-DOWNS, POST-ANCHORS, ETC. WITH THE ARCHITECTURAL PLANS PRIOR TO COMMENCING WORK AND BE RESPONSIBLE FOR SAME.
2. UNO BOLT SILL PLATES TO THE FOUNDATIONS WITH MIN 1/2" NOMINAL DIA ANCHOR BOLT WITH PLATE WASHERS. BOLTS SHALL BE SPACED NOT MORE THAN 72" O.C. THERE SHALL BE A MIN OF (2) BOLTS PER PIECE. ONE BOLT SHALL BE LOCATED NOT MORE THAN 12" AND NOT LESS THAN 4" FROM EACH END, OR FROM EACH SIDE OF A NOTCH GREATER THAN 1/2 THE WIDTH OF THE PLATE. EMBED BOLTS AT LEAST 7" INTO REINFORCED MASONRY OR CONCRETE. SILL PLATE ANCHOR BOLTS MAY BE "WET SET".
3. PROVIDE 3" X 3" X 0.229" PLATE WASHERS ON ALL ANCHOR BOLTS AT SHEAR WALLS. PLATE WASHERS SHALL EXTEND TO WITHIN 1/2" OF THE EDGE OF THE SILL PLATE ON THE SIDE(S) WITH SHEATHING. PROVIDE STANDARD CUT WASHERS UNDER BOLT HEADS AND NUTS WHEN SLOTTED PLATE WASHERS ARE USED. PROVIDE CUT OR SLOTTED WASHERS AGAINST WOOD AT ALL REMAINING WALLS.
4. ALL 1/2" DIA. ANCHOR BOLTS MAY BE REPLACED WITH:
   - A. 1/2" DIA THREADED ROD EPOXY DOWELED TO FOUNDATION WITH SIMPSON SET-XP EPOXY (ICC REPORT ESR-2508) IN 5/8" DIA HOLE. PROVIDE A MIN. OF 4" EMBEDMENT AND 1 3/4" EDGE DISTANCE FROM ANY SLAB EDGE.
   - B. 1/2" DIA SIMPSON TITEN HD HIGH STRENGTH THREADED ANCHOR (ICC ESR-2713). PROVIDE A MIN. OF 4" EMBEDMENT AND 1 3/4" EDGE DISTANCE FROM ANY SLAB EDGE.
   - C. SIMPSON MASA/MASAP MUDSILL ANCHORS (ICC ESR-2555). INSTALLATION OF THE MUDSILL ANCHOR SHALL BE PER MANUFACTURER'S TYPICAL OR ALTERNATE INSTALLATION METHOD. DO NOT WET-SET THE MUDSILL ANCHOR, INSTALL WITH ONE LEG UP OR INSTALL OVER PLYWOOD OR OSB. FOLLOW ALL MANUFACTURER'S SPECIFICATIONS AND RECOMMENDATIONS DURING INSTALLATION. DO NOT US WHEN ANCHOR SPACING IS < 9 INCHES.
5. AT INTERIOR NON-BEARING AND NON-SHEARWALLS, 1/2" DIA ANCHOR BOLTS MAY BE REPLACED WITH MINIMUM 0.145" DIA X 2 7/8" POWDER DRIVEN FASTENERS AT A MAXIMUM SPACING OF 24" O.C. (HILTI ESR-2379 OR SIMPSON ESR-2138 OR EQUIVALENT)
6. PROVIDE REINFORCING BAR FOR HOLDOWNS PER DETAILS SHEET SD-1, UNO.
7. HOLDOWNS SPECIFIED ON PLANS MAY BE POST INSTALLED (RETRO-FITTED) AS NOTED BELOW. DRILL COMPLETELY THRU THE FOOTING AND PROVIDE THREADED ROD WITH BEARING PLATE, WASHER AND NUT. BACKFILL ALL AROUND THE ROD, WASHER AND NUT TO PROVIDE A MINIMUM OF 3" COVER USING LEAN CONCRETE MIX. THREADED ROD AND BEARING PLATE SHALL BE PER THE TABLE PROVIDED. SEE RETROFIT DETAIL SHEET SD-1.1. THE USE OF POST INSTALLED ANCHORS UNLESS NOTED OTHERWISE, E.G. EXPANSION ANCHORS OR EPOXY ANCHORS, IS NOT ALLOWED WITHOUT PRIOR REVIEW AND AUTHORIZATION BY THE ENGINEER OF RECORD. CONTACT THE ENGINEER OF RECORD FOR REQUEST OF ALTERNATE ANCHORAGE SOLUTIONS.

REINFORCING STEEL

1. REINFORCING BARS SHALL CONFORM TO THE REQUIREMENTS OF ASTM A-615 GRADE 60.
2. WELDED REINFORCING BARS SHALL CONFORM TO THE REQUIREMENTS OF ASTM A-706 GRADE 60.
3. ALL REINFORCING BAR BENDS SHALL BE MADE COLD.
4. WELDED WIRE REINFORCEMENT (WWR) SHALL BE PER ASTM A185 AND SHALL BE SUPPLIED IN SHEETS. WWR FRM ROLLS SHALL NOT BE USED.
5. MINIMUM LAP OF WELDED WIRE FABRIC SHALL BE 6 INCHES OR ONE AND ONE HALF SQUARES, WHICHEVER IS GREATER.
6. ALL BARS SHALL BE MARKED SO THEIR IDENTIFICATION CAN BE MADE WHEN THE FINAL IN-PLACE INSPECTION IS MADE.
7. REBAR SPLICES ARE TO BE CLASS "B" (UNO). MAINTAIN 2 BAR DIA CLEAR SPACE BETWEEN ADJACENT SPLICES.
8. REINFORCING SPLICES SHALL BE MADE ONLY WHERE INDICATED ON THE DRAWINGS.
9. DOWELS BETWEEN FOOTINGS AND WALLS OR COLUMNS SHALL BE THE SAME GRADE, SIZE AND SPACING OR NUMBER AS THE VERTICAL REINFORCING, RESPECTIVELY, UNO.

REBAR SIZE	3	4	5	6	7	8	9	10	11
MINIMUM LAP LENGTH (INCHES)	22	29	36	43	68	78	88	98	107
STD HOOK LENGTH (INCHES)	4 1/2	6	7 1/2	9	10 1/2	12	13 1/2	15 1/2	16 1/2



STRUCTURAL STEEL

1. HOT-ROLLED STRUCTURAL STEEL SHAPES SHALL BE PER ASTM A992, EXCEPT ANGLES AND CHANNELS SHALL BE PER ASTM A36.
2. STRUCTURAL PIPE SHALL BE PER ASTM A53 GRADE B.
3. STRUCTURAL HSS SHAPES SHALL BE PER ASTM A500 GRADE C.
4. STRUCTURAL PLATE STEEL SHALL BE PER ASTM A36.
5. NUTS AND BOLTS IN STRUCTURAL STEEL CONNECTIONS SHALL BE PER ASTM 325N, WITH HARDENED WASHERS UNO. DESIGN IS BASED UPON BEARING TYPE CONNECTIONS WITH THREADS NOT EXCLUDED. PERIODIC SPECIAL INSPECTION REQUIRED.
6. ANCHOR BOLTS SHALL BE PER ASTM F1554-36, U.N.O.
7. DRYPACK/NON-SHRINK GROUT AT COLUMN BASE PLATES SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 4500 PSI.
8. WELDS SHALL BE BY E70XX, LOW HYDROGEN ELECTRODES.
9. ALL FIELD WELDS SHALL HAVE SPECIAL INSPECTION.
10. PROVIDE SPECIAL INSPECTION IN ACCORDANCE WITH IBC 1705.2.1.
11. ALL STEEL EXPOSED TO WEATHER SHALL BE PAINTED.

CONCRETE

1. ALL PHASES OF WORK PERTAINING TO THE CONCRETE CONSTRUCTION SHALL CONFORM TO THE "BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE" (ACI 318 LATEST ADOPTED EDITION), WITH MODIFICATIONS AS NOTED IN THE DRAWINGS AND SPECIFICATIONS.
2. REINFORCED CONCRETE DESIGN IS BY THE "ULTIMATE STRENGTH DESIGN METHOD".
3. SCHEDULE OF STRUCTURAL CONCRETE 28-DAY STRENGTHS AND TYPES:

LOCATION IN STRUCTURE	STRENGTH	PSI TYPE
SLABS ON GRADE	2500 PSI	HARD ROCK
FOOTINGS	2500PSI	HARD ROCK

DESIGN BASED ON 2500 PSI, 28-DAY STRENGTH, THEREFORE SPECIAL INSPECTION IS NOT REQUIRED.

4. CONCRETE MIX DESIGN SHALL BE SUBMITTED TO THE STRUCTURAL ENGINEER FOR APPROVAL WITH THE FOLLOWING REQUIREMENTS:
   - A. COMPRESSIVE STRENGTH AT AGE 28 DAYS AS SPECIFIED ABOVE.
   - B. LARGE AGGREGATE-HARDROCK: 3/4" MAXIMUM SIZE CONFORMING TO ASTM C-33.
   - C. CEMENT: ASTM C-150, TYPE V PORTLAND CEMENT.
   - D. MAXIMUM SLUMP: 5 INCHES.
   - E. NO ADMIXTURES, EXCEPT FOR ENTRAINED AIR, AND AS APPROVED BY THE ENGINEER.
   - F. WATER/CEMENT RATIO SHALL BE AS FOLLOWS:
     - a. f'c = 4500 PSI................w/c = 0.45
     - b. f'c = 4000 PSI................w/c = 0.5
     - c. f'c < 4000 PSI................w/c = PER MIX DESIGN
   - G. FLY ASH: ASTM C618
   - H. SLAG: ASTM C989
   - I. SILICA FUME: ASTM C1240
   - J. LIGHTWEIGHT AGGREGATE: ASTM C330
   - K. AIR ENTRAINING ADMIXTURE: ASTMC260
   - L. WATER REDUCERS: ASTM C494, TYPE A OR F
5. CONCRETE MIXING OPERATIONS, ETC, SHALL CONFORM TO ASTM C-94.
6. PLACEMENT OF CONCRETE SHALL CONFORM TO ACI STANDARD 614 AND PROJECT SPECIFICATIONS.
7. CLEAR COVERAGE OF CONCRETE OVER OUTER REINFORCING BARS SHALL BE AS FOLLOWS:
   - A. CONCRETE POURED DIRECTLY AGAINST EARTH: 3 INCHES CLEAR
   - B. STRUCTURAL SLABS: 3/4 INCHES CLEAR, TOP AND BOTTOM (2" TOP; 3/4" BOTTOM IN CORROSIVE ENVIRONMENTS)
   - C. FORMED CONCRETE WITH EARTH BACKFILL: 2 INCHES CLEAR
8. ALL REINFORCING BARS, HOLD DOWN BOLTS AND STRAPS, AND OTHER CONCRETE INSERTS SHALL BE WELL SECURED IN POSITION PRIOR TO PLACING CONCRETE.
9. PROVIDE SLEEVES FOR PLUMBING AND ELECTRICAL OPENINGS IN CONCRETE BEFORE PLACING. DO NOT CUT ANY REINFORCING WHICH MAY CONFLICT. CORING IN CONCRETE IS NOT PERMITTED EXCEPT AS SHOWN. NOTIFY THE STRUCTURAL ENGINEER IN ADVANCE OF CONDITIONS NOT SHOWN ON THE DRAWINGS.
10. CONDUIT OR PIPE SIZE (O.D.) SHALL NOT EXCEED 30% OF SLAB THICKNESS AND SHALL BE PLACED BETWEEN THE TOP AND BOTTOM REINFORCING, UNLESS SPECIFICALLY DETAILED OTHERWISE. CONCENTRATIONS OF CONDUITS OR PIPES SHALL BE AVOIDED EXCEPT WHERE DETAILED OPENINGS ARE PROVIDED.
11. MODULUS OF ELASTICITY OF CONCRETE, WHEN TESTED IN ACCORDANCE WITH ASTM C-460, SHALL BE AT LEAST THE VALUE GIVEN BY THE EQUATIONS IN SECTION 8.5.1 OF ACI 318 FOR THE SPECIFIED 28-DAY STRENGTH.
12. SEE FOUNDATION DETAILS FOR REINFORCEMENT REQUIRED AT CORNERS AND INTERSECTIONS OF CONCRETE WALLS, CONVENTIONAL FOOTINGS AND GRADE BEAMS.

WOOD

1. FRAMING LUMBER, UNO:
   - A. 2X AND 4X TO BE DOUGLAS FIR LARCH (DFL) NO. 2 GRADE. SEE FRAMING PLANS AND NOTES FOR WALL STUD REQUIREMENTS.
   - B. 6X TO BE DOUGLAS FIR LARCH NO. 1 GRADE.
   - C. ALL LUMBER SHALL HAVE A MOISTURE CONTENT OF LESS THAN 19%.
2. BOLT HOLES SHALL BE 1/16" (MAXIMUM) LARGER THAN THE BOLT SIZE. RE-TIGHTEN ALL NUTS PRIOR TO CLOSING IN.
3. ALL SILLS OR PLATES RESTING ON CONCRETE OR MASONRY SHALL BE PRESSURE TREATED DOUGLAS FIR USING BORON BASED PRESERVATIVES OR LSL TREATED WITH ZINC BORATE.
4. DO NOT NOTCH JOISTS, RAFTERS OR BEAMS, EXCEPT WHERE SHOWN IN DETAILS. OBTAIN ENGINEER'S APPROVAL FOR ANY HOLES OR NOTCHES NOT DETAILED.
5. WHERE A POST OCCURS AT AN UPPER LEVEL, ADD THE SAME POST DIRECTLY BELOW IT ON THE LOWER LEVELS AND IN BETWEEN FLOOR SHEATHING AND LOWER LEVEL WALL TOP PLATES.
6. FACE NAIL EACH PLY OF MULTIPLE 2X POSTS WITH 16D SINKERS AT 6" O.C.
7. PROVIDE MULTIPLE 2X POST AT ALL GIRDER TRUSS AND BEAM BEARING LOCATIONS, WIDTH TO MATCH BEAM OR TRUSS, MIN. (2) 2X UNO.
8. CONNECTION HARDWARE SHALL BE SIMPSON OR EQUAL AND MUST BE I.C.C. APPROVED.
9. FOR CONNECTIONS NOT DETAILED, PROVIDE FASTENERS PER TABLE 2304.9.1 OF THE LATEST ADOPTED EDITION OF THE INTERNATIONAL BUILDING CODE.
10. DO NOT NOTCH TOP PLATES OR STUDS EXCEPT AS SHOWN IN DETAILS. OBTAIN ENGINEER'S APPROVAL FOR ANY HOLES OR NOTCHES NOT DETAILED.
11. NON-BEARING WALLS SHALL HAVE STUDS SPACED AT 24" O.C. (MAX). REFER TO ARCHITECTURAL DRAWINGS FOR SIZE. TOP PLATES SHALL BE SUCH THAT A 1/2" GAP BETWEEN THE TOP OF THE PLATES AND THE BOTTOM OF THE TRUSSES AND/OR BLOCKING PANELS EXISTS AFTER THE ROOF IS LOADED.
12. TYPICAL FASTENER SIZE NOTED IN DRAWINGS:
    8d COMMON NAILS = 0.131" DIA X 2 1/2", 10d COMMON NAILS = 0.148" DIA
    16d COMMON NAILS = 0.162" DIA 16d SINKER NAILS = 0.148" DIA X3 1/4".
13. 16d SINKERS NOTED IN THESE DRAWINGS MAY BE REPLACED WITH 0.131 X 3 1/4" GUN NAILS PER THE FOLLOWING TABLES:

# OF 16d SINKERS	1	2	3	4	5	6	7	8	9	10	11	12
# OF 0.131 X 3 1/4" GUN NAILS	2	3	4	5	6	7	8	9	11	12	13	14

SPACING OF 16d SINKERS	2" OC	4" OC	6" OC	8" OC	10" OC	12" OC
SPACING OF 0.131 X 3 1/4" GUN NAILS	1 1/2" OC	3 1/2" OC	5" OC	7" OC	8 1/2" OC	10 1/2" OC

MANUFACTURED WOOD I JOISTS

1. MANUFACTURED WOOD I-JOISTS SHALL BE DESIGNED BY THE JOIST MANUFACTURER (INCLUDING BRIDGING SIZE AND SPACING), UNO. CONTRACTOR SHALL SUBMIT SHOP DRAWINGS, ERECTION DRAWINGS, AND DESIGN CALCULATIONS, SEALED BY AN ENGINEER, REGISTERED IN THE STATE OF THE GOVERNING JURISDICTION, FOR REVIEW PRIOR TO MANUFACTURE. CALCULATIONS AND SHOP DRAWINGS SHALL SHOW SPECIAL DETAILS REQUIRED AT BEARING POINTS. ALL CONNECTORS SHALL HAVE CURRENT ICC APPROVAL.
2. JOIST MANUFACTURER TO DESIGN DRAG MEMBERS FOR LATERAL LOAD (LAT = XXXX) IN POUNDS, AS SHOWN ON PLANS.
3. ALL JOIST TO JOIST CONNECTORS SHALL BE PER JOIST MANUFACTURER.
4. JOIST MANUFACTURER SHALL LIMIT TOTAL LOAD DEFLECTIONS TO LESS THAN L/240 AND LIVE LOAD DEFLECTIONS TO LESS THAN L/480 AT FLOOR JOISTS AND L/360 AT ROOF JOISTS. DEFLECTION SHALL BE LIMITED SO AS NOT TO CREATE A BEARING CONDITION AT NON-BEARING WALLS. REFER ALSO TO WOOD NOTE 11.

MANUFACTURED BEAMS

1. GLUE LAMINATED BEAMS (GLB)
   - A. GLB SHALL BE 24F-V4, (CONTINUOUS AND CANTILEVERED GLB SHALL BE 24F-V8) AND HAVE THE FOLLOWING MINIMUM PROPERTIES: FB=2400 PSI, FV=240 PSI
     FC (PERPENDICULAR) = 650 PSI
     E=1,800,000 PSI
   - B. ALL BEAMS SHALL BE FABRICATED USING EXTERIOR GLUE. FABRICATION AND HANDLING PER LATEST AITC AND WCCA STANDARDS. BEAMS TO BEAR GRADE STAMP AND AITC STAMP AND CERTIFICATE.
   - C. MOISTURE CONTENT SHALL BE LIMITED TO A MAXIMUM OF 12%.
   - D. ALL GLB SHALL HAVE STANDARD CAMBER, UNO. ON PLANS.
2. LAMINATED STRAND LUMBER (1.3E OR 1.5E LSL)
   - A. LSL BEAMS SHALL HAVE I.C.C APPROVAL AND HAVE THE FOLLOWING MINIMUM PROPERTIES:
     1) LSL (1.3E) - E = 1,300,000 PSI, FB = 1700 PSI, FV = 400 PSI
        FC (PERPENDICULAR) = 680 PSI, FC (PARALLEL) = 1400 PSI
     2) LSL (1.5E) - E = 1,500,000 PSI, FB = 2250 PSI, FV = 400 PSI
        FC (PERPENDICULAR) = 750 PSI, FC (PARALLEL) = 1950 PSI
   - B. ALL MULTI-PLY LSL MEMBERS SPECIFIED ON PLANS MAY BE REPLACED WITH SOLID MEMBERS OF EQUAL OR GREATER PROPERTIES WITH EQUAL OR GREATER WIDTH AND DEPTH WITHOUT FURTHER REVIEW.
3. LAMINATED VENEER LUMBER (LVL)
   - A. LVL BEAMS SHALL HAVE I.C.C APPROVAL AND HAVE THE FOLLOWING MINIMUM PROPERTIES:
     1) LVL (1.7E) - E = 1,700,000 PSI, FB = 2650 PSI, FV = 285 PSI,
        FC (PERPENDICULAR) = 750 PSI, FC (PARALLEL) = 3000 PSI
     2) LVL (1.9E) - E = 1,900,000 PSI, FB = 2600 PSI, FV = 285 PSI,
        FC (PERPENDICULAR) = 750 PSI, FC (PARALLEL) = 2510 PSI
     3) LVL (2.0E) - E = 2,000,000 PSI, FB = 2800 PSI, FV = 285 PSI,
        FC (PERPENDICULAR) = 750 PSI, FC (PARALLEL) = 3000 PSI
   - B. MULTIPLE-PLY LVL BEAMS SHALL BE NAILED TOGETHER AS FOLLOWS:
     1) PROVIDE (2) ROWS OF 16D SINKERS AT 12" O.C.
        FOR BEAMS ≤ 11 7/8" DEEP.
     2) PROVIDE (3) ROWS OF 16D SINKERS AT 12" O.C.
        FOR BEAMS > 11 7/8" DEEP.
   - C. ALL MULTI-PLY LVL MEMBERS SPECIFIED ON PLANS MAY BE REPLACED WITH SOLID MEMBERS OF EQUAL OR GREATER PROPERTIES WITH EQUAL OR GREATER WIDTH AND DEPTH WITHOUT FURTHER REVIEW.
4. PARALLEL STRAND LUMBER (PSL)
   - A. PSL BEAMS SHALL HAVE I.C.C APPROVAL AND HAVE THE FOLLOWING MINIMUM PROPERTIES:
     E = 2,000,000 PSI, FB = 2900 PSI, FV = 290 PSI
     FC (PERPENDICULAR) = 750 PSI, FC (PARALLEL) = 2900 PSI

PREFABRICATED WOOD TRUSSES

1. PREFABRICATED WOOD ROOF TRUSSES SHALL BE AS DESIGNED BY THE TRUSS MANUFACTURER (INCLUDING BRIDGING SIZE AND SPACING) UNO. THE CONTRACTOR SHALL SUBMIT SHOP DRAWINGS, ERECTION DRAWINGS, AND DESIGN CALCULATIONS, SEALED BY AN ENGINEER, REGISTERED IN THE STATE OF THE GOVERNING JURISDICTION, FOR REVIEW PRIOR TO MANUFACTURE. CALCULATIONS AND SHOP DRAWINGS SHALL SHOW ANY SPECIAL DETAILS REQUIRED AT BEARING POINTS. ALL CONNECTORS SHALL HAVE CURRENT I.C.C. APPROVAL.
2. TRUSS MANUFACTURER TO DESIGN TRUSSES FOR LATERAL LOAD (LAT. = XXXX) IN POUNDS, AS SHOWN ON PLANS.
3. UNLESS OTHERWISE NOTED, TRUSSES TO BE DESIGNED FOR LOADS INDICATED IN THE DESIGN CRITERIA AND AS FOLLOWS:
   - A. TOP CHORD DEAD LOAD = XXX + ADDITIONAL 5 PSF AT OVERFRAMING.
   - B. BOTTOM CHORD DEAD LOAD = XXX.
   - C. WIND LOAD: WHEN CALCULATING NET UPLIFT REACTIONS, USE MAXIMUM RESISTING DEAD LOAD = XXX PSF ON TOP CHORD AND XXX PSF ON BOTTOM CHORD
4. TRUSS MANUFACTURER TO DESIGN TRUSSES TO SUPPORT MECHANICAL EQUIPMENT AS REQUIRED. ADDITIONAL TRUSSES MAY BE SUPPLIED AS REQUIRED
5. ALL TRUSS TO TRUSS CONNECTORS PER TRUSS MANUFACTURER.
6. WHERE POST OCCURS ABOVE A MANUFACTURED TRUSS, A VERTICAL WEB SHALL BE PROVIDED UNDER THE POST. THE WEB SECTION AREA SHALL BE EQUAL TO OR LARGER THAN POST SECTION AREA. MULTIPLE 2X4S MAY BE USED FOR THE VERTICAL WEB.
7. THE TOP CHORD OF ALL TRUSSES SHALL HAVE A SPECIES WITH SPECIFIC GRAVITY EQUAL TO OR GREATER THAN 0.42.
8. TRUSS MANUFACTURER SHALL LIMIT TOTAL LOAD DEFLECTIONS TO LESS THAN L/240 AND LIVE LOAD DEFLECTIONS TO LESS THAN L/360. DEFLECTION SHALL BE LIMITED SO AS NOT TO CREATE A BEARING CONDITION AT NON-BEARING WALLS. REFER ALSO TO NOTE 11 OF THE WOOD SECTION.
9. CONNECTION OF MANUFACTURED TRUSSES FOR UPLIFT SHALL BE PER THE TABLES ON SHEET S1.1.
10. TRUSSES SHALL BE DESIGNED TO SUPPORT A 250 POUND CONCENTRATED LOAD AT ANY LOCATION ALONG THE BOTTOM CHORD.

STANDARD TRUSSES(2)

REACTIONS TO THE BEARING WALL OR BEAM BELOW:

TRUSS TYPE & TOTAL LENGTH	HARDWARE REQUIRED	NAILING REQUIRED
JACK TRUSSES ≤ 10'-0"	NONE REQUIRED	UPLIFT IS RESISTED BY EXISTING NAILED CONNECTION OF TRUSS TO PLATE(1)
HIP/COMMON TRUSSES ≤ 30'-0"	NONE REQUIRED	UPLIFT IS RESISTED BY EXISTING NAILED CONNECTION OF TRUSS TO PLATE(1)
HIP/COMMON TRUSS 30'-0" ≤ L ≤ 50'-0"	H1 OR H2.5	(6) 8d x 1 1/2" INTO TRUSS (4) 8d x 1 1/2" INTO PLATE
ALL HIP/COMMON DRAG TRUSSES L ≤ 50'-0"	H1 OR H2.5	(6) 8d x 1 1/2" INTO TRUSS (4) 8d x 1 1/2" INTO PLATE
HIP/COM. GIRDER 8FT SETBACK L ≤ 30'-0"	H10 OR (2) H2.5	(8) 8d x 1 1/2" INTO TRUSS (8) 8d x 1 1/2" INTO PLATE
HIP/COM. GIRDER 8FT SETBACK L ≤ 40'-0"	HTS20	(12) 10d x 1 1/2" INTO TRUSS (12) 10d x 1 1/2" INTO PLATE & STUD BLW
HIP/COM. GIRDER 8FT SETBACK L ≤ 50'-0"	(2) HTS20	(12) 10d x 1 1/2" INTO TRUSS (12) 10d x 1 1/2" INTO PLATE & STUD BLW
ALL OTHER NON-DRAG GIRDER TR L ≤ 40'-0"	H1 OR H2.5	(6) 8d x 1 1/2" INTO TRUSS (4) 8d x 1 1/2" INTO PLATE
ALL OTHER GIRDER TRUSSES L ≤ 50'-0"	H10 OR (2) H2.5	(8) 8d x 1 1/2" INTO TRUSS (8) 8d x 1 1/2" INTO PLATE

AT TRUSSES WHERE THE HARDWARE ABOVE CANNOT BE INSTALLED, PROVIDE AN HTS20 STRAP

GABLE END WALL TRUSSES

HARDWARE	NAILING REQUIRED
LTP4 AT 32" OC UNO	(6) 8d x 1 1/2" INTO TRUSS (4) 8d x 1 1/2" INTO PLATE

NOTES:
(1) SEE DETAIL 1/SD-3 FOR STANDARD NAILING REQUIREMENTS
(2) SEE FRAMING PLANS FOR NON-STANDARD UPLIFT HARDWARE

STRUCTURAL SHEATHING

1. ALL STRUCTURAL SHEATHING SHALL BE C-D INTERIOR SHEATHING WITH EXTERIOR GLUE OR ORIENTED STRAND BOARD (OSB), EXPOSURE I, AND SHALL BEAR THE STAMP OF AN APPROVED TESTING AGENCY. LAY SHEATHING WITH LONG DIMENSION PERPENDICULAR TO SUPPORTS. STAGGER JOINTS AND NAILS.
2. ROOF SHEATHING AT UNBLOCKED DIAPHRAGM (STANDARD, UNO)
   15/32" WOOD STRUCTURAL PANEL: PANEL INDEX = 32/16, UNBLOCKED UNO W/ 8D COMMON NAILS AT 6" O.C. AT ALL BOUNDARIES AND SUPPORTED EDGES, 12" O.C. FIELD. PANELS LESS THAN 2'-0" IN WIDTH SHALL HAVE ALL EDGES BLOCKED.
3. ROOF SHEATHING AT BLOCKED DIAPHRAGM (WHERE NOTED ON PLANS)
   15/32" WOOD STRUCTURAL PANEL: PANEL INDEX = 32/16, BLOCKED. NAIL W/ 8D COMMON NAILS AT 4" O.C. AT ALL BOUNDARIES AND SUPPORTED EDGES, 12" O.C. FIELD, UNO ON PLANS.
4. FLOOR SHEATHING AT UNBLOCKED DIAPHRAGM (STANDARD, UNO)
   23/32" WOOD STRUCTURAL PANEL: T AND G, PANEL INDEX 48/24, UNBLOCKED (UNO.). NAIL WITH 10D COMMON NAILS AT 6" O.C. AT ALL BOUNDARIES AND SUPPORTED EDGES, 12" O.C. FIELD UNO ON PLANS. PANELS LESS THAN 2'-0" IN WIDTH SHALL HAVE ALL EDGES BLOCKED. OPTION: QUIK DRIVE #8 WSNTL WOOD SCREWS MAY BE USED IN LIEU OF 10D COMMON NAILS ABOVE. SPACING SHALL BE SAME AS SHOWN FOR 10D COMMON NAILS PER I.C.C. REPORT ESR-1472.
5. FLOOR SHEATHING AT BLOCKED DIAPHRAGM (WHERE NOTED ON PLANS)
   23/32" WOOD STRUCTURAL PANEL: T AND G, PANEL INDEX = 48/24, BLOCKED. NAIL WITH 16D SINKERS AT 4" O.C. AT ALL BOUNDARIES AND SUPPORTED EDGES, 12" O.C. FIELD UNO ON PLANS. OPTION: QUIK DRIVE #8 X 2 1/4" SCREWS MAY BE USED IN LIEU OF 16D SINKERS ABOVE. SPACING SHALL BE SAME AS SHOWN FOR 10D COMMON NAILS PER I.C.C. REPORT ESR-1472.
6. SHEAR WALL SHEATHING
   PER SHEAR WALL SCHEDULE.

SHOP DRAWINGS

1. SHOP DRAWINGS SHALL BE SUBMITTED FOR ALL STRUCTURAL ITEMS IN ADDITION TO ITEMS REQUIRED BY ARCHITECTURAL SPECIFICATIONS.
2. THE CONTRACTOR SHALL REVIEW ALL SHOP DRAWINGS PRIOR TO SUBMITTAL. ITEMS NOT IN ACCORDANCE WITH CONTRACT DRAWINGS SHALL BE FLAGGED FOR REVIEW.
3. VERIFY ALL DIMENSIONS WITH ARCHITECTURAL DRAWINGS.
4. ANY CHANGES, SUBSTITUTIONS, OR DEVIATIONS FROM ORIGINAL STRUCTURAL DRAWINGS SHALL BE RED-LINED OR FLAGGED BY SUBMITTING PARTIES.
5. THE STRUCTURAL ENGINEER HAS THE RIGHT TO APPROVE OR DISAPPROVE ANY CHANGES TO THE ORIGINAL DRAWINGS AT ANY TIME, BEFORE OR AFTER SHOP DRAWING REVIEW.
6. THE SHOP DRAWINGS DO NOT REPLACE THE ORIGINAL STRUCTURAL DRAWINGS. ITEMS OMITTED OR SHOWN INCORRECTLY BUT NOT FLAGGED BY THE STRUCTURAL ENGINEER OR ARCHITECT ARE NOT TO BE CONSIDERED CHANGES TO THE ORIGINAL CONTRACT DRAWING.
7. THE ADEQUACY OF ENGINEERING DESIGNS AND LAYOUT PERFORMED BY THE OTHERS RESTS WITH THE DESIGNING OR SUBMITTING AUTHORITY.
8. REVIEWING IS INTENDED ONLY AS AN AID TO THE CONTRACTOR IN OBTAINING CORRECT SHOP DRAWINGS. RESPONSIBILITY FOR CORRECTNESS SHALL REST WITH THE CONTRACTOR.
9. THE CONTRACTOR SHALL PROVIDE (1) FILE COPY OF ALL APPROVED SHOP DRAWINGS TO THE STRUCTURAL ENGINEER.

SPECIAL INSPECTIONS

1. PROVIDE SPECIAL INSPECTIONS IN ACCORDANCE WITH SECTION 1705 OF THE BUILDING CODE, AS REQUIRED BY THE PERMITTING AGENCY.
2. WHERE SPECIAL INSPECTION IS REQUIRED, IT SHALL BE PERFORMED BY A REGISTERED DEPUTY INSPECTOR EMPLOYED BY THE OWNER AND APPROVED BY THE GOVERNING JURISDICTION. COPIES OF THE INSPECTION REPORTS SHALL BE SUBMITTED TO THE BUILDING DEPARTMENT AND ARCHITECT/STRUCTURAL ENGINEER FOR REVIEW.
3. ITEMS REQUIRING SPECIAL INSPECTION:

INSPECTION	CODE SECTION
A. POST INSTALLED CONCRETE ANCHORS (PERIODIC)	1705.3
B. NAILING AT P2, P3, P4 AND P5 SHEAR WALLS (PERIODIC)	1705.10.1
C. DRAG TRUSS CONNECTIONS TO SHEAR WALLS (PERIODIC)	1705.10.1
D. SHEARWALL HOLDOWNS (PERIODIC)	1705.10.1
E. STEEL CONSTRUCTION.	

THE FOLLOWING SHALL BE VERIFIED OR INSPECTED IN ACCORDANCE WITH IBC 1705.2, AISC 360, & AISC 341:
1. SHOP WELDING UNLESS PERFORMED ON PREMISES OF AN APPROVED FABRICATOR AS DEFINED IN IBC SECTION 1704.2.5 - PERIODIC.
2. MATERIAL VERIFICATION OF HIGH STRENGTH BOLTS, NUTS, WASHERS - PERIODIC.
3. INSPECTION OF HIGH STRENGTH BOLTING FOR BEARING-TYPE CONNECTIONS.
4. FIELD WELDING - CONTINUOUS WITH THE EXCEPTION OF FILLET WELDS = 5/16", FLOOR AND ROOF DECK WELDS.

DEFERRED SUBMITTAL ITEMS

1. STEEL STAIRS.

SHEET INDEX

JOB NO: 1939-004-191
DESIGNED BY: KAB
DRAWN BY: KAB

ISSUED FOR:
CONSTRUCTION DOCUMENTS
DATE: 12-10-19

SHEET TITLE:
STRUCTURAL GENERAL NOTES

REVISIONS:

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ASSURED DEVELOPMENT

ATHENS LOT 4

PROFESSIONAL ENGINEER - STATE OF NEVADA
KENT A. BARBER
Exp. 12-31-20
CIVIL / STRUCTURAL
No. 017213
01/02/2020

S1

SHEARWALL SCHEDULE[2, 5, 8, 11, 12, 13]

SHEARWALL[7, 17]			ONE SIDED SHEARWALL / TWO SIDED SHEARWALL[9]		SEISMIC	WIND
MARK	MATERIAL[4, 14]	NAILING	UPPER FLOOR SILL PL CONN[1, 18]	ANCHOR BOLTS[3] (UNO ON FOUNDATION PLAN)	ALLOWABLE SHEAR	
P1	3/8" APA SHEATHING[15])	8dN AT 6" OC EDGES, 12" FIELD[16]	16dS AT 4" OC (STG'D) UNO	1/2" DIA X 10" AT 32" OC	260#/FT	365#/FT
P2	3/8" APA SHEATHING[15])	8dN AT 4" OC EDGES, 12" FIELD[16]	16dS AT 3" OC, (STG'D) UNO	1/2" DIA X 10" AT 24" OC	350#/FT	532#/FT
P3	3/8" APA SHEATHING[9, 15])	8dN AT 3" OC EDGES, 12" FIELD[16]	1/4x6 SCREWS AT 4" OC (STG'D), UNO	1/2" DIA X 10" AT 16" OC	490#/FT	685#/FT
P4	3/8" APA SHEATHING[9, 15])	8dN AT 2" OC EDGES, 12" FIELD[16]	1/4x6 SCREWS AT 4" OC (STG'D), UNO	1/2" DIA X 10" AT 12" OC	600#/FT	895#/FT
P5	15/32" APA SHEATHING[10])	10dN AT 2" OC EDGES, 12" FIELD[16]	1/4x6 SCREWS AT 3" OC (STG'D), UNO	1/2" DIA X 10" AT 9" OC	770#/FT	1078#/FT

SCHEDULE NOTES:

(1) SEE DETAIL 7/SD-2 FOR ADDT'L INFO AT UPPER FLOOR SILL PLATE CONNECTION FOR MULTI-STORY PLANS ONLY. DOES NOT APPLY TO SINGLE STORY PLANS. SEE PLAN FOR SILL PLATE AND SHEAR CONNECTIONS AT EXTERIOR WALLS.
(2) OPENINGS IN SHEARWALL SHEATHING SHALL NOT EXCEED 8 INCHES IN ANY DIRECTION FOR A SINGLE OPENING OR THE SUM OF ANY TWO OR MORE OPENINGS ON COMMON OR OVERLAPPING VERTICAL OR HORIZONTAL LINES. OPENINGS NOT GREATER THAN 8 INCHES DO NOT REQUIRE BLOCKING AROUND THE PENETRATION. CONTACT THE ENGINEER OF RECORD FOR REQUIREMENTS AT OPENINGS NOT OTHERWISE DETAILED.
(3) MINIMUM (2) 1/2" DIA ANCHORS PER SHEAR WALL. SEE FOUNDATION HARDWARE NOTES, NOTE 2 ON THE GENERAL NOTES SHEET, S1. ALL ANCHOR BOLTS SHALL HAVE 3" x 3" x 0.229" PLATE WASHERS.
(4) APA RATED (STRUCTURAL II) PLYWOOD OR OSB.
(5) SEE DETAIL 11/SD-2 WHERE WALL FRAMING STEPS OR PERPENDICULAR WALL INTERSECTS SHEAR WALL.
(6) FOR SHEAR PANELS ON TWO SIDES OF WALL, USE ONE-HALF THE SPACING GIVEN IN THE SCHEDULE FOR SILL PLATE CONNECTION AND ANCHOR BOLT SPACING, UNO.
(7) DOUBLE SIDED SHEARWALLS SHALL HAVE VERTICAL PANEL JOINTS OFFSET TO FALL ON DIFFERENT STUDS OR USE SINGLE 3" NOMINAL STUDS (MIN) AT JOINTS. AT THE ENDS OF THE SHEARWALL, 4X NOMINAL MEMBERS ARE REQUIRED. NAILS ON EACH SIDE SHALL BE STAGGERED.
(8) ALL SHEARWALLS REQUIRE DOUBLE 2X TOP PLATES, U.N.O. AT NON-BEARING SHEAR WALLS, SHORTEN STUDS 1/4 INCH TO PROVIDE DEFLECTION CLEARANCE.
(9) P2, P3 AND P4 SHEARWALLS SHALL REQUIRE THE FOLLOWING:
  A. STAGGER NAILING ALONG PLYWOOD JOINTS AND SILL PLATES.
  B. SINGLE 3" NOMINAL MEMBERS AT ALL FRAMING MEMBERS RECEIVING EDGE NAILING FROM ABUTTING PANELS. 3" NOMINAL MEMBERS AT SINGLE SIDED SHEARWALL MAY BE CONSTRUCTED W/ (2) 2X MEMBERS FASTENED TOGETHER W/ (2) ROWS OF 16d SINKERS AT 4" OC.
(10) P5 SHEARWALLS SHALL REQUIRE SINGLE 3" NOMINAL MEMBERS AT ALL FRAMING MEMBERS RECEIVING EDGE NAILING FROM ABUTTING PANELS (MIN). STAGGER JOINT AND SILL PLATE NAILING.
(11) ALL SHEARWALL LENGTHS NOTED ON PLAN ARE MINIMUM REQUIRED AND MAY BE INCREASED WITHOUT REVIEW.
(12) SHEATHING MAY BE PLACED ON EITHER FACE OF DESIGNATED WALL, UNO.
(13) ALLOWABLE SHEAR CAPACITIES ARE IN ACCORDANCE WITH AF&PA SDPWS TABLE 4.3A WITH APPLICABLE OMEGA FACTORS INCLUDED.
(14) APA SHEATHING AND GYPSUM SHEATHING MAY BE INSTALLED WITH THE LONG OR SHORT DIRECTION PERPENDICULAR TO THE FRAMING. WHERE APA SHEATHING IS INSTALLED WITH THE SHORT DIRECTION PERPENDICULAR TO THE FRAMING, THE FRAMING MUST BE 16" ON CENTER (MAX). WHERE GYPSUM SHEATHING IS INSTALLED WITH THE SHORT DIRECTION PERPENDICULAR TO THE FRAMING ALL PANEL EDGES MUST BE BLOCKED AND NAILED.
(15) 3/8" SHEATHING MAY BE REPLACED WITH 7/16" OR 15/32" SHEATHING WITHOUT ADDITIONAL REVIEW.
(16) ALL VERTICAL AND HORIZONTAL PANEL EDGES TO BE BLOCKED AND NAILED.
(17) STUDS SHALL BE 24" OC (MAX). FOR 3/8" AND 7/16" SHEATHING, FIELD NAILING SHALL BE REDUCED TO 6" OC WHERE STUD SPACING IS GREATER THAN 16" OC.
(18) SILL PLATE CONNECTORS SHALL BE PER SHEAR TRANSFER DETAIL SPECIFIED ON PLANS.

HOLDOWN/STRAP SCHEDULE[1, 2]

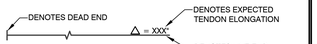
HD/STRAP	EMBED AT FND AND / OR ANCHOR BOLT	CONN TO (2) 2X STUD, UNO[5, 6, 10]
CS16	N/A	EXTEND STRAP 16" MIN. EA. END W/ (13) 8dN TO (2) 2X STUD ABOVE AND BELOW FLOOR FRAMING
(2) CS16	N/A	EXTEND STRAP 16" MIN. EA. END W/ (13) 8dN TO (2) 2X STUD ABOVE AND BELOW FLOOR FRAMING
CMSTC16	N/A	EXTEND STRAP 25' MIN. EA. END W/ (28) 16d SINKERS TO (2) 2X STUD ABOVE AND BELOW FLOOR FRAMING
CMST14	N/A	EXTEND STRAP 32' MIN. EA. END W/ (33) 16dN TO (3) 2X STUD ABOVE AND BELOW FLOOR FRAMING
CMST12	N/A	EXTEND STRAP 40' MIN. EA. END W/ (42) 16dN TO (3) 2X STUD ABOVE AND BELOW FLOOR FRAMING
LSTHD8[4,11])	8" EMBED	(20) 16d SINKERS
STHD10[4,11])	10" EMBED	(24) 16d SINKERS
STHD14[4,11])	14" EMBED	(30) 16d SINKERS
HTT5	SSTB24 W/ 21" EMBED[12])	(26) 16dN X 2 1/2" NAILS
HDU2	SSTB24 W/ 21" EMBED[12])	(6) SDS 1/4 X 2 1/2 SCREWS W/ MIN (2) 2X POSTS
HDU4	SSTB24 W/ 21" EMBED[12])	(10) SDS 1/4 X 2 1/2 SCREWS W/ MIN (2) 2X POSTS
HDU5	SSTB24 W/ 21" EMBED[12])	(14) SDS 1/4 X 2 1/2 SCREWS W/ MIN (2) 2X POSTS
HDU8	SSTB34 W/ 29" EMBED[13])	(20) SDS 1/4 X 2 1/2 SCREWS W/ MIN (3) 2X POSTS
HDU11	1" DIA AB W/ 9" MIN EMBED W/ MIN 28" SQ x 14" DEEP FTG[8])	(30) SDS 1/4 X 2 1/2 SCREWS W/ MIN 4X8 POST[9])
HDU14	1" DIA AB W/ 10" MIN EMBED W/ MIN 30" SQ X 15" DEEP FTG[7])	(36) SDS 1/4 X 2 1/2 SCREWS W/ MIN 4X8 POST[9])

SCHEDULE NOTES:

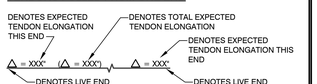
(1) HD/STRAP SHALL BE SIMPSON OR EQUAL W/ ICC APPROVAL. ALL SUBSTITUTES SHALL BE REVIEWED BY THE ENGINEER OF RECORD BEFORE INSTALLATION.
(2) FIXED-LENGTH STRAPS SHALL BE INSTALLED WITH AN EQUAL LENGTH OVERLAPPING CONNECTED MEMBERS AND AN EQUAL NUMBER OF FASTENERS IN EACH MEMBER.
(3) STITCH NAIL EACH STUD AT MULTIPLE 2x STUDS TOGETHER WITH 16d SINKERS AT:
  4" OC FOR P3 AND P4 SHEAR WALLS
  6" OC FOR ALL OTHER SHEAR WALLS
(4) FOR CONCRETE SPALLS LESS THAN 4", THERE IS NO LOAD REDUCTION AND NO FURTHER REVIEW BY EOR IS REQUIRED.
(5) SEE DETAIL 4/SD-2 FOR ADDL CRITERIA AT UPPER FLOOR STRAPS (WHERE OCCURS).
(6) EDGE NAIL SHT'G TO EA MEMBER OF MULTPLE POST, OFFSET 1/2 SPACING BTWN MEMBERS.
(7) ASTM F1554-55 BOLT W/ HEAVY SQUARE NUT OR 1/4 X 1 3/4 X 1 3/4 PLATE WASHER REQUIRED FOR FULL LOAD. REDUCE ALLOWABLE LOAD TO 13180 LBS FOR ASTM GRADE 36 BOLT. MINIMUM EMBEDMENT IS FROM TOP OF FOOTING.
(8) ASTM GRADE 36 BOLT W/ SQUARE OR HEAVY HEX HEAD OR NUT REQUIRED. MINIMUM EMBEDMENT IS FROM TOP OF FOOTING.
(9) PROVIDE 6X8 POST AT 2X6 WALLS, MULTIPLE STUDS NOT ALLOWED.
(10) END POST TO BE FULL HEIGHT MEMBERS, UNO.
(11) STRAPS MAY BE PLACED ON EITHER FACE OF DESIGNATED WALL AND ARE NOT REQUIRED TO OCCUR ON SAME FACE AS SHEATHING, UNO.
(12) AT GARAGE STEMWALL LOCATIONS USE SSTBL.
(13) WHEN SLAB AND FOOTINGS ARE PLACED AS A MONO-POUR, SSTB28 WITH 25" EMBEDMENT MAY BE SUBSTITUTED FOR THE SSTB34 SPECIFIED.

FOUNDATION NOTES:

A. AT INTERIOR BEARING WALLS, WITHOUT DEEPENED FOOTINGS, USE 1/2" DIA TITEN HD HIGH STRENGTH SCREW ANCHORS IN LIEU OF ANCHOR BOLTS.
B. SEE DETAIL 1/SD-1 FOR PERIMETER FOOTING EMBEDMENT DEPTH.
C. VERIFY ALL DIMENSIONS WITH ARCHITECTURAL DRAWINGS.
D. DIMENSIONS BETWEEN POST-TENSIONED CABLES MAY VARY BY 3";
E. POST-TENSIONED CABLE TENDONS:



TENDONS PULLED FROM ONE END



TENDONS PULLED FROM BOTH ENDS

FRAMING NOTES:

1. ALL EXTERIOR WALLS AND INTERIOR BEARING AND SHEAR WALLS TO BE MIN 2x6 AT 16" O.C. DFL STUD GRADE, UNO. SEE FRAMING PLANS FOR NON-TYPICAL STUD SIZE AND SPACING.
2. TRIMMER / KING STUD SCHEDULE, UNO.

	OPENING SPAN (L)	TRIMMERS	KING STUDS
2x4 WALLS	L < 6'-0"	1	1
	6'-0" ≤ L < 10'-0"	2	2
	10'-0" ≤ L < 18'-0"	2	3
2x6 WALLS	L < 8'-0"	1	1
	8'-0" ≤ L < 12'-0"	2	2
	12'-0" ≤ L < 20'-0"	2	3

3. BLOCKED DIAPHRAGM - SEE STRUCTURAL GENERAL NOTES SHEET S1.
4. FOR TYPICAL OVERFILL FRAMING WHERE REQUIRED BY TRUSS SHOP DRAWINGS, SEE DETAILS 4/SD-3 OR 5/SD-3.
5. INTERIOR BEARING WALLS
6. BEAM AND HEADER SIZES INDICATED ON THIS PLAN ARE MINIMUM. LARGER SIZES OR HIGHER GRADE LUMBER MAY BE SUBSTITUTED.
7. TOP PLATE SPLICES PER DETAIL 3/SD-2, UNO.
8. SEE DETAIL 8/SD-2 FOR ADDITIONAL FRAMING REQUIREMENTS.

LEDGER & HANGER SCHEDULE:

UNLESS OTHERWISE NOTED 2X LEDGERS WHERE DETAILED SHALL BE AS FOLLOWS:

		TRUSS SPAN (L)	TRUSS SPACING (MAX)	LEDGER AND NAILING[1,2,6,10,11]	MIN HANGER, UNO [3,4,5,11]
ROOF LOAD		L ≤ 8'-0"	16" O.C.	2X8 W/ (3) 16d AT 16"OC	LUS24/JUS24
			24" O.C.	2X8 W/ (5) 16d AT 24"OC	LUS24/JUS24
		L ≤ 16'-0"	24" O.C.	2X8 W/ (5) 16d AT 16"OC	LUS26/JUS26
				2X12 W/ (8) 16d AT 24"OC	LUS26/JUS26
		L ≤ 24'-0"	24" O.C.	2X12 W/ (8) 16d AT 16"OC	LUS28/JUS28
				2X12 W/ 2 COLUMNS OF (8) 16d AT 24"OC	HUS28/HUS28
FLOOR LOAD	4x MEMBER	L ≤ 10'-0"	16" O.C.	2X8 W/ (5) 16d AT 16"OC	LUS46/JUS46
			24" O.C.	2X10 W/ 2 COLUMNS OF (6) 16d AT 24"OC	LUS46/JUS46
		L ≤ 20'-0"	24" O.C.	2X10 W/ 2 COLUMNS OF (6) 16d AT 16"OC	LUS48/JUS48
			24" O.C.	2X12 W/ 2 COLUMNS OF (8) 16d AT 24"OC	HUS48/HUS48
FLOOR LOAD	2x MEMBER	L ≤ 10'-0"	16" O.C.	2X8 W/ (5) 16d AT 16"OC	LUS26/JUS26
			24" O.C.	2X10 W/ 2 COLUMNS OF (6) 16d AT 24"OC	LUS26/JUS26
		L ≤ 20'-0"	24" O.C.	2X10 W/ 2 COLUMNS OF (6) 16d AT 16"OC	LUS28/JUS28
			24" O.C.	2X12 W/ 2 COLUMNS OF (8) 16d AT 24"OC	HUS28/HUS28
DECK LOAD	2x MEMBER	L ≤ 10'-0"	16" O.C.	2X10 W/ (2) 1/4"x3" SCREWS AT 16" OC	LUS26/JUS26
			24" O.C.	2X10 W/ 2 COLUMNS OF (2) 1/4"x3" SCREWS AT 24" OC	LUS26/JUS26
		L ≤ 15-0"	24" O.C.	2X10 W/ 2 COLUMNS OF (2) 1/4"x3" SCREWS AT 16" OC	LUS28/JUS28
			24" O.C.	2X10 W/ 2 COLUMNS OF (3) 1/4"x3" SCREWS AT 24" OC	HUS28/HUS28

SCHEDULE NOTES:

(1) TWO COLUMNS OF FASTENERS REQUIRE MIN SUPPORTING MEMBER RECEIVING FASTENERS OF 3" OR (2)2X IN WIDTH (ENDS OF 4X2 OR DOUBLE TRUSSES ARE ACCEPTABLE). SPACE FASTENERS MIN 1" APART IN EACH DIRECTION.
(2) SPACING SHOWN EQUALS SPACING OF FRAMING MEMBERS RECEIVING FASTENERS.
(3) HANGERS LISTED IN ORDER ARE BY SIMPSON STRONG-TIE AND USP, RESPECTIVELY.
(4) LISTED HANGERS ARE MINIMUM HANGERS REQUIRED WHERE NOT OTHERWISE NOTED ON THE STRUCTURAL DRAWINGS.
(5) SPACING SHOWN FOR HANGERS IS SPACING OF THE FRAMING MEMBER SUPPORTED BY THE HANGER.
(6) LEDGER MATERIAL SHALL BE DFL #2 OR BETTER UNO. LEDGER AT DECK SHALL BE PRESSURE-PRESERVATIVE TREATED OR NATURALLY DURABLE WOOD.
(7) PROVIDE THA218 (MIN) HANGER FOR ROOF TRUSS TO BEAM CONNECTIONS FOR ALL BEAM DEPTHS GREATER THAN 10 INCHES.
(8) PROVIDE THA418 (MIN) HANGER FOR FLOOR TRUSS TO BEAM CONNECTIONS FOR ALL BEAM DEPTHS GREATER THAN 10 INCHES.
(9) PROVIDE THA422 (MIN) HANGER FOR FLOOR TRUSS TO BEAM CONNECITONS FOR ALL BEAM DEPTHS GREATER THAN 18 INCHES.
(10) ALL SIMPSON SCREWS NOTED SHALL BE STRONG DRIVE SDS SCREWS. ALL USP SCREWS NOTED SHALL BE WS SCREWS.
(11) HANGERS AND FASTENERS WITH EXTERIOR EXPOSURE SHALL BE STAINLESS STEEL, UNLESS NOTED OTHERWISE.

JOB NO: 1939-004-191
DESIGNED BY: KAB
DRAWN BY: KAB

ISSUED FOR:
CONSTRUCTION DOCUMENTS
DATE: 12-10-19

SHEET TITLE:
SCHEDULES & NOTES

REVISIONS:

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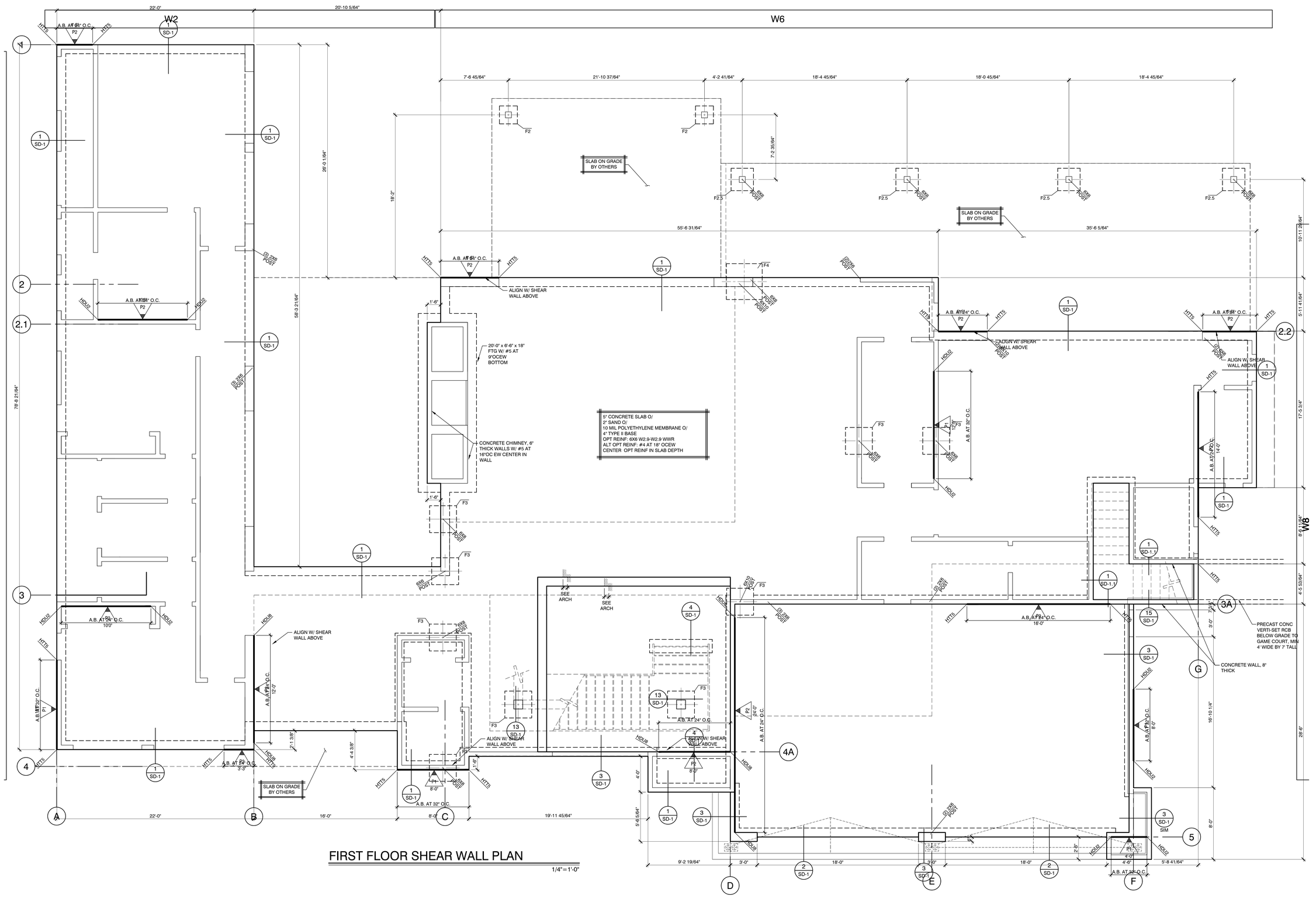
ASSURED DEVELOPMENT
ATHENS LOT 4

01/02/2020

S1.1



FIRST FLOOR SHEAR WALL PLAN

1/4"=1'-0"

JOB NO: **1939-004-191**
DESIGNED BY: **KAB**
DRAWN BY: **KAB**

ISSUED FOR:
CONSTRUCTION DOCUMENTS
DATE: **12-10-19**

SHEET TITLE:
FOUNDATION PLAN

REVISIONS:

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ASSURED DEVELOPMENT
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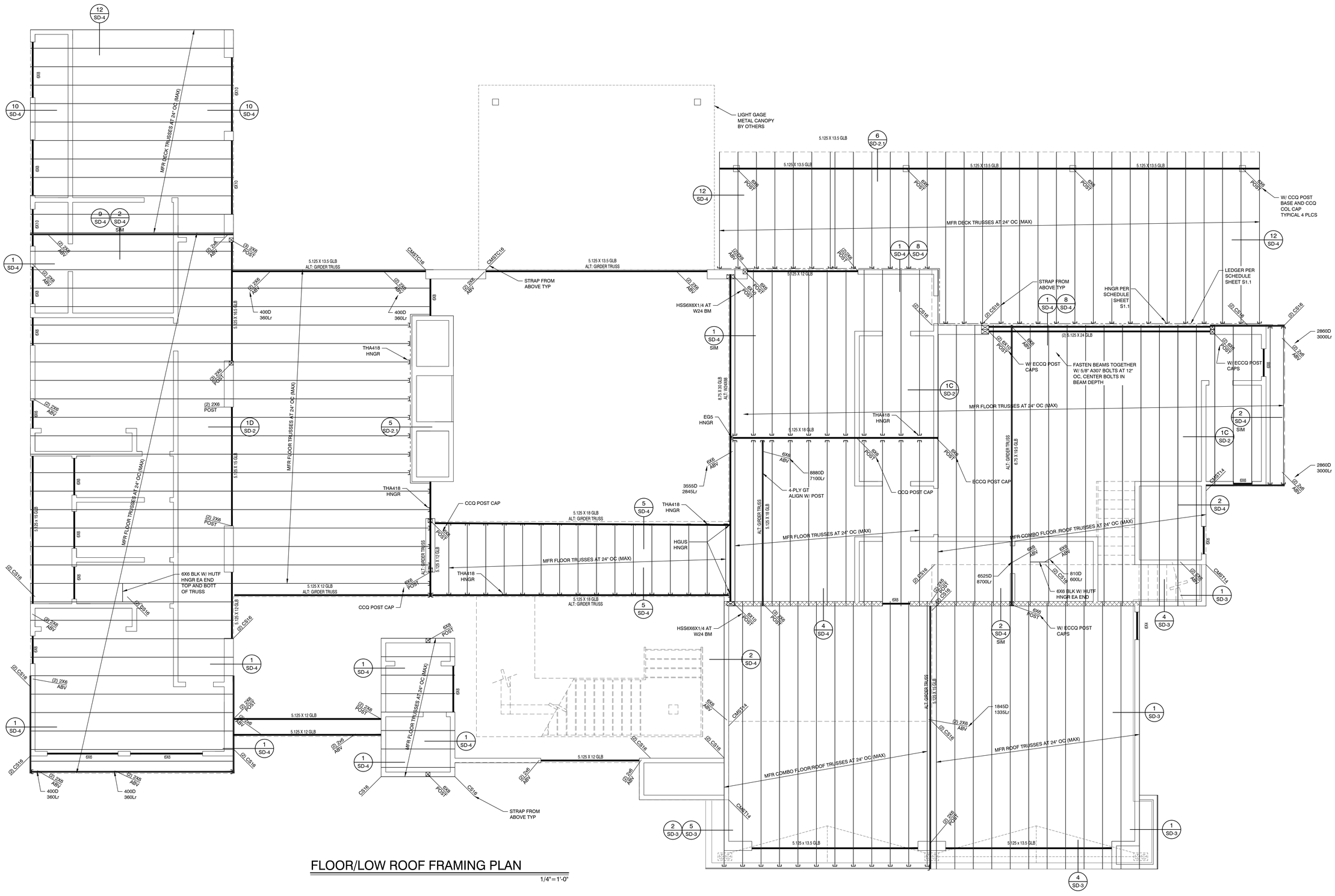
S2



FLOOR/LOW ROOF FRAMING PLAN

1/4" = 1'-0"

SEE SHEET S1.1 FOR FRAMING NOTES

JOB NO: **1939-004-191**
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DRAWN BY: **KAB**

ISSUED FOR:
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SHEET TITLE:
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REVISIONS:

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ASSURED DEVELOPMENT
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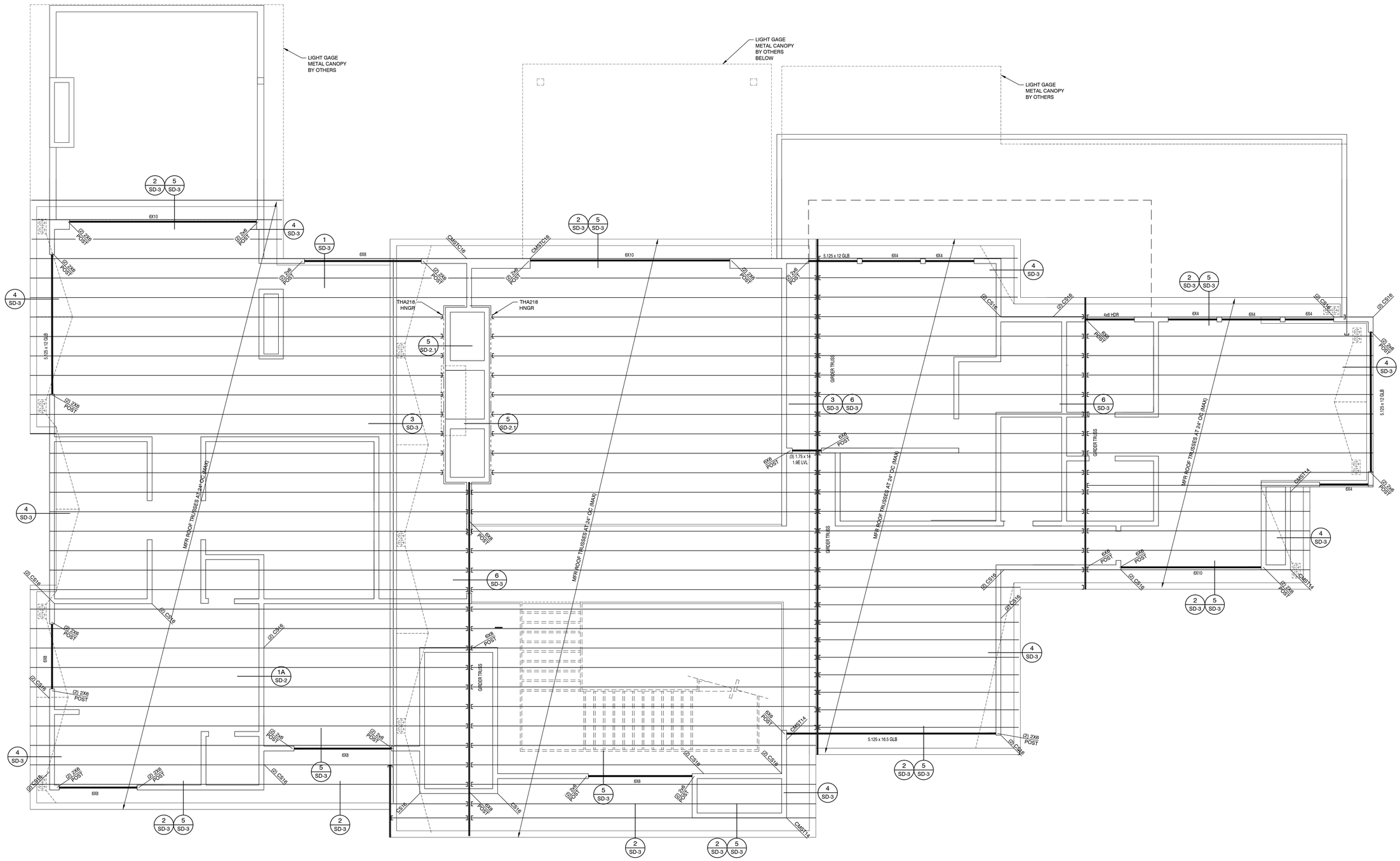
KENT A. BARBER
Exp. 12-31-20
CIVIL / STRUCTURAL
No. 017213
STATE OF NEVADA

01/02/2020

S3



ROOF FRAMING PLAN

1/4"= 1'-0"

SEE SHEET S1.1 FOR FOUNDATION NOTES AND HOLDOWN SCHEDULE

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DRAWN BY: **KAB**

ISSUED FOR:
CONSTRUCTION DOCUMENTS
DATE: **12-10-19**

SHEET TITLE:
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REVISIONS:

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CIVIL / STRUCTURAL
No. 017213
STATE OF NEVADA
PROFESSIONAL ENGINEER
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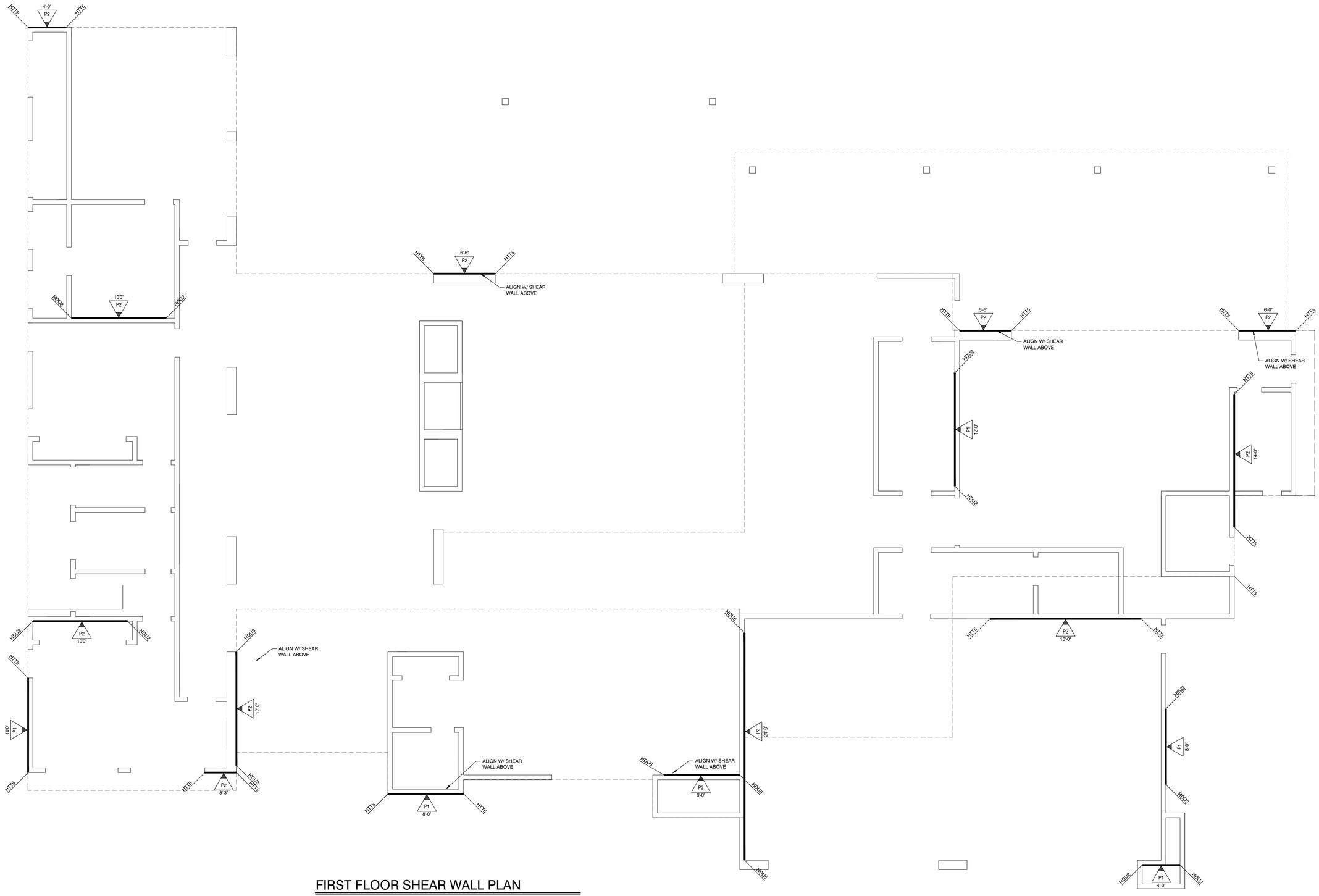
S4



FIRST FLOOR SHEAR WALL PLAN

1/4"=1'-0"

SEE SHEET S1.1 FOR FOUNDATION NOTES AND HOLDOWN SCHEDULE

JOB NO: **1939-004-191**
DESIGNED BY: **KAB**
DRAWN BY: **KAB**

ISSUED FOR:
CONSTRUCTION DOCUMENTS
DATE: **12-10-19**

SHEET TITLE:
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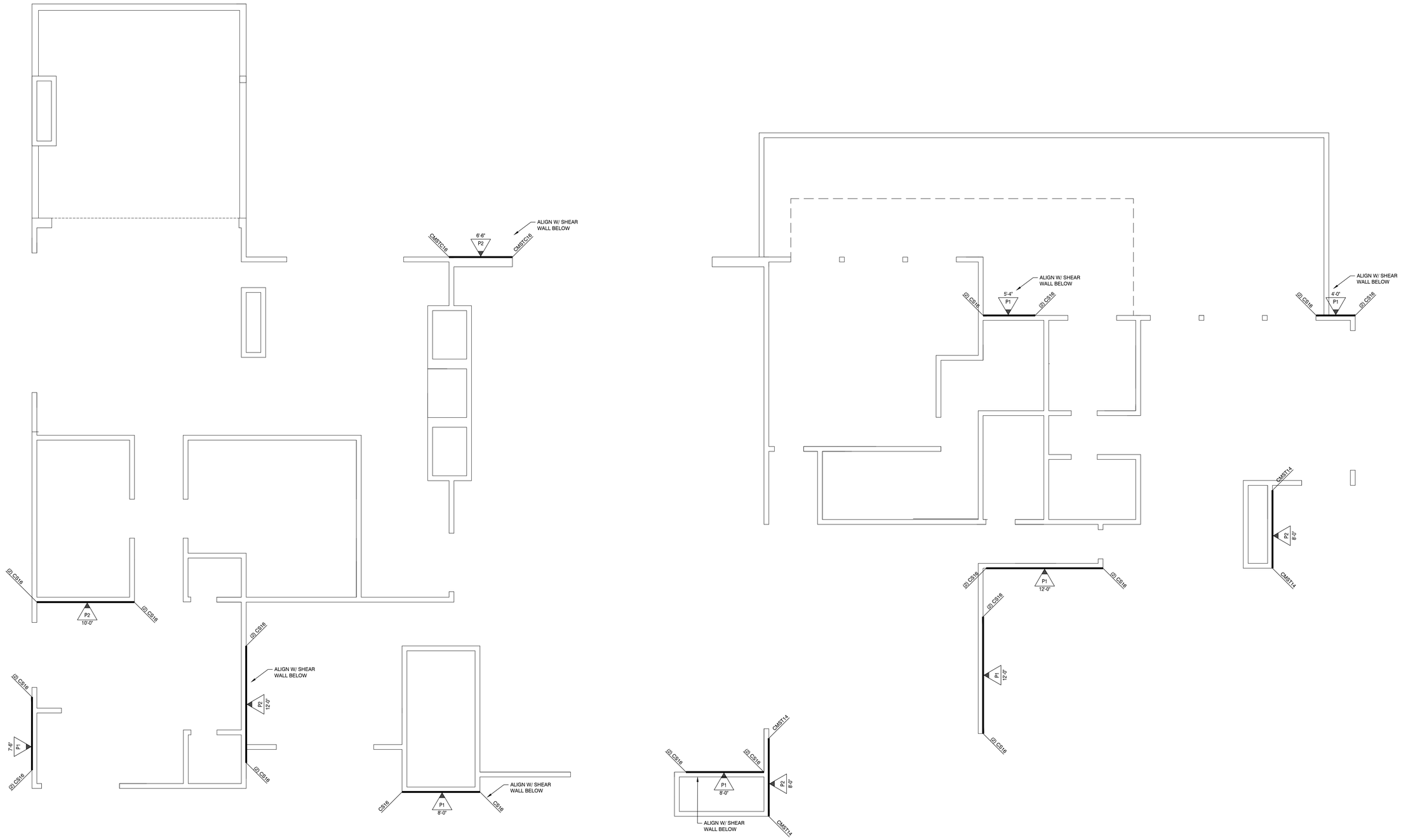
KENT A. BARBER
Exp. 12-31-20
CIVIL / STRUCTURAL
No. 017213
01/02/2020

S5



SECOND FLOOR SHEAR WALL PLAN

1/4"=1'-0"

SEE SHEET S1.1 FOR FOUNDATION NOTES AND HOLDOWN SCHEDULE

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ISSUED FOR:
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DATE: **12-10-19**

SHEET TITLE:
SHEAR WALL PLAN

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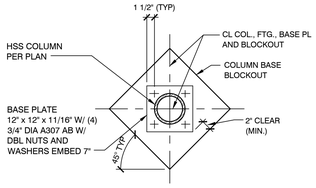
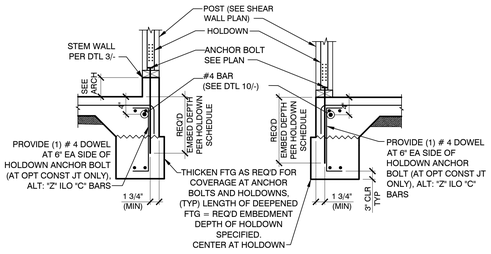
01/02/2020

S5.1



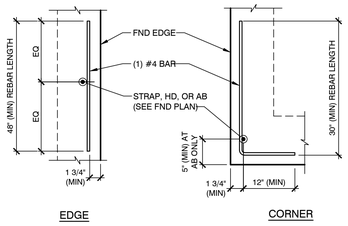
SECTION "A-A"



SEE DTL 1/- FOR INFORMATION NOT SHOWN

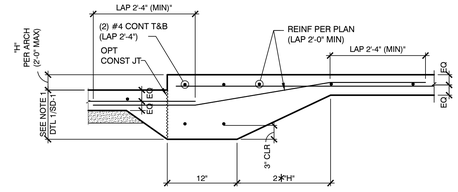
9 HOLDOWN W/ ANCHOR BOLT (TYPICAL)

NTS

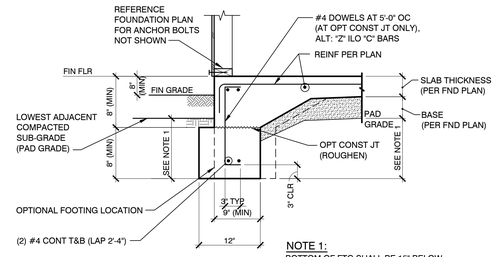


EDGE CORNER

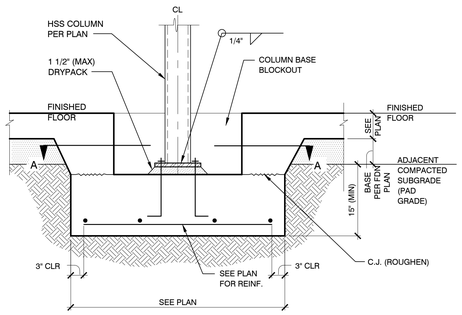
PLAN



NOTE 1:
GREATER DEPTH AS REQUIRED FOR HOLDOWNS AND HOLDOWN ANCHOR BOLTS.

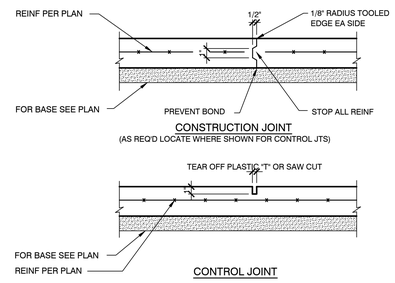


NOTE 1:
BOTTOM OF FTG SHALL BE 15" BELOW LOWEST ADJACENT COMPACTED SUB-GRADE (PAD GRADE). PROVIDE GREATER DEPTH AS REQUIRED FOR HOLDOWNS AND HOLDOWN ANCHOR BOLTS



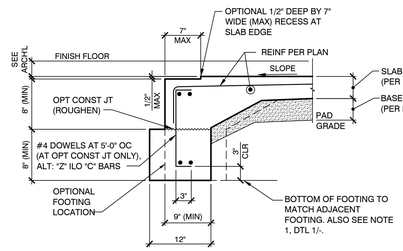
5 STEP IN SLAB

NTS



1 EDGE OF FOUNDATION

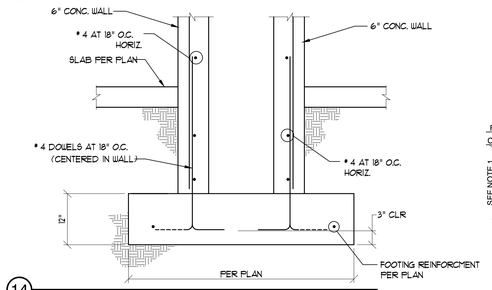
NTS



SEE DTL 1/- FOR INFO NOT SHOWN

13

NTS



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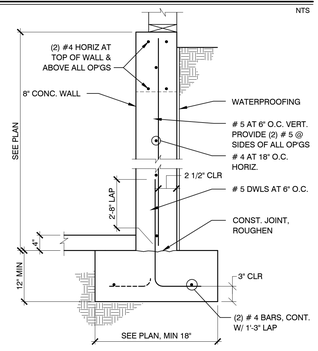
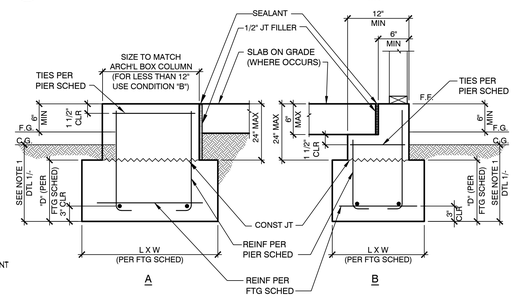


15

NTS

10 REINFORCING AT HOLDOWNS

NTS

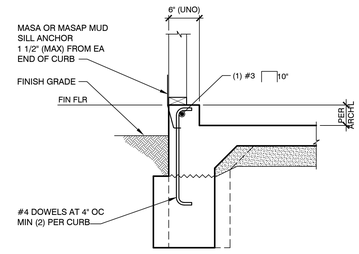


11 CONCRETE PIER AND FOOTING

NTS

6 TYP SLAB JOINTS

NTS



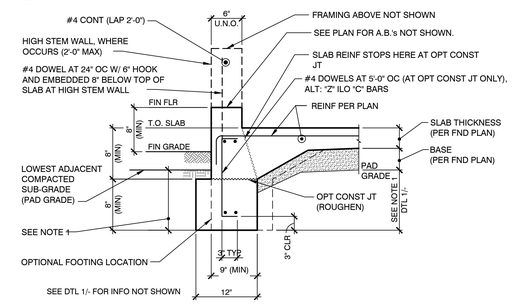
SEE DTL 3/- FOR INFORMATION NOT SHOWN

7 CURB BETWEEN GARAGE DOORS

NTS

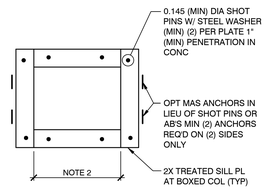
2 EDGE OF FOUNDATION (GARAGE)

N.T.S.



3 EDGE OF FOUNDATION (CURB)

NTS

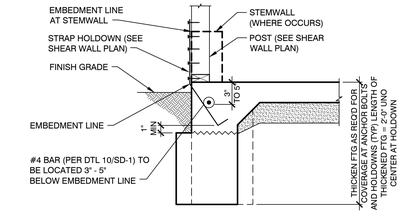


NOTES:
1. THIS DETAIL IS FOR BOX COL SILL PL (SEE FND PLAN FOR LOCATIONS)
2. PROVIDE MAS ANCHORS AT PL LENGTHS LESS THAN 8"

PLAN

12 BOX COLUMN SILL PLATE

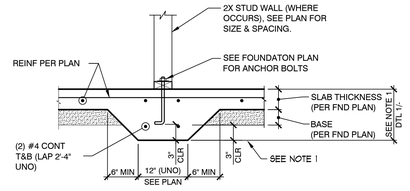
NTS



SEE DTL 1/- FOR INFORMATION NOT SHOWN

8 STRAP HOLDOWN

NTS



NOTE 1:
GREATER DEPTH AS REQUIRED FOR HOLDOWNS AND HOLDOWN ANCHOR BOLTS.

4 INTERIOR CONTINUOUS FTG

NTS

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www.lrnelng.com

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DRAWN BY: KAB

ISSUED FOR:
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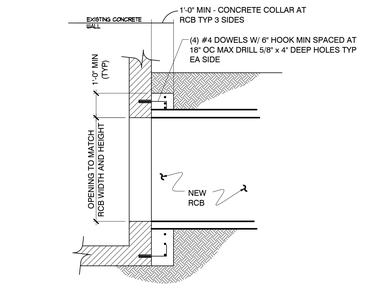
STRUCTURAL DETAILS

REVISIONS:

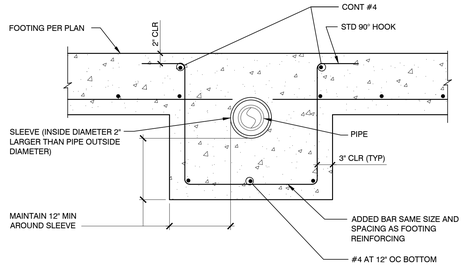
ASSURED DEVELOPMENT
ATHENS LOT 4

01/02/2020

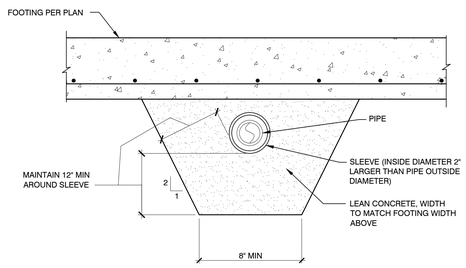
SD-1



11 NTS



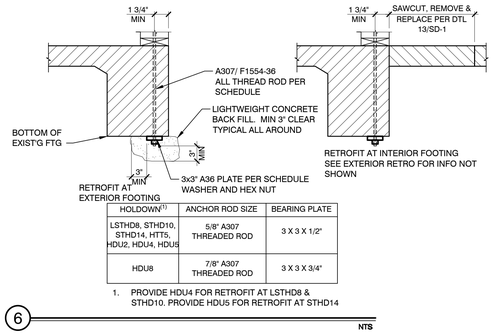
THROUGH FOOTING



BELOW FOOTING

9 PIPE SLEEVE DETAIL

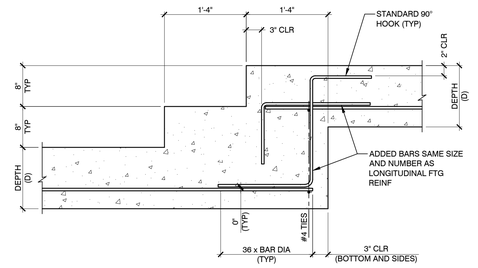
NTS



HOLDOWN[1]	ANCHOR ROD SIZE	BEARING PLATE
LSTHD8, STHD10, STHD14, HTT5, HDU2, HDU4, HDU5	5/8" A307 THREADED ROD	3 X 3 X 1/2"
HDU8	7/8" A307 THREADED ROD	3 X 3 X 3/4"

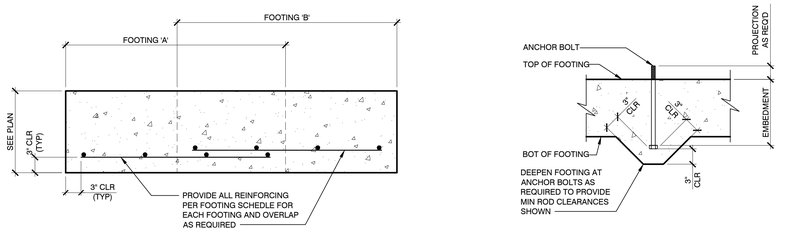
1. PROVIDE HDU4 FOR RETROFIT AT LSTHD8 & STHD10. PROVIDE HDU5 FOR RETROFIT AT STHD14

6 NTS

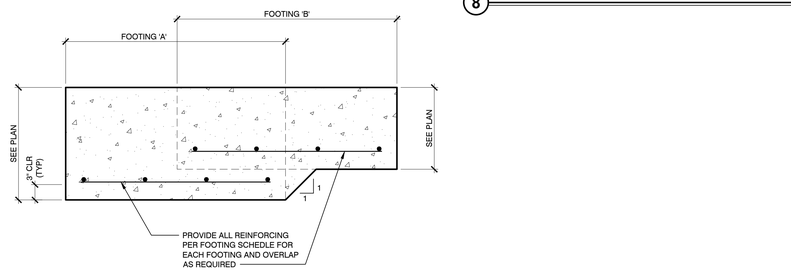


7 TYPICAL STEPPED FOOTING DETAIL

NTS



FOOTINGS OF EQUAL DEPTH



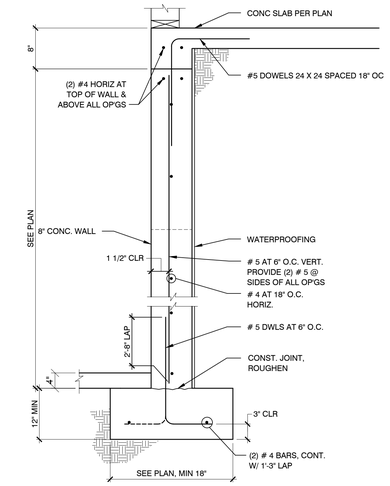
FOOTINGS OF DIFFERENT DEPTHS

10 OVERLAPPING SPREAD FOOTING DETAIL

NTS

8 ANCHOR BOLT EMBEDMENT DETAIL

NTS



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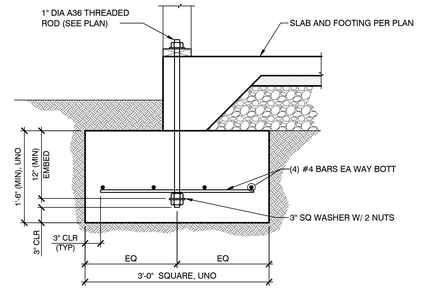
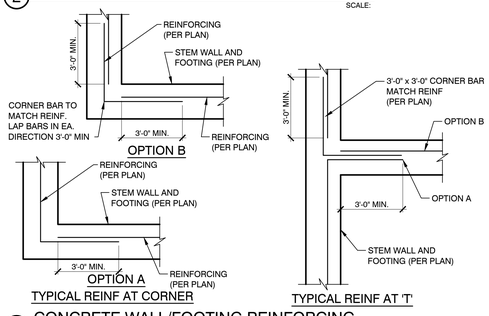
NTS



2 HOLDOWN FOOTING

SCALE:



5 CONCRETE WALL/FOOTING REINFORCING

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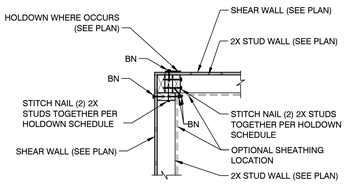
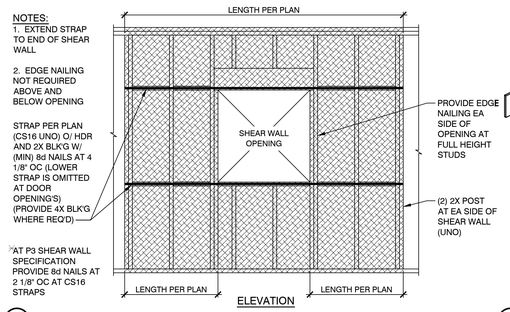
SD-1.1



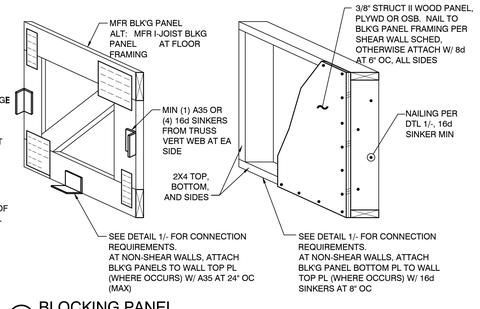
9 TWO SHEAR WALLS AT CORNER W/ SHARED HOLDOWN

NTS



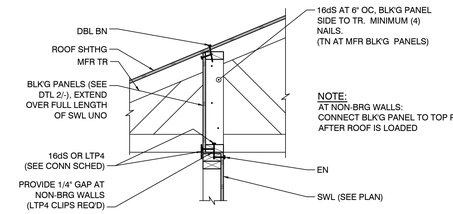
5 APPLICABLE ONLY WHERE SPECIFIED ON PLANS

NTS

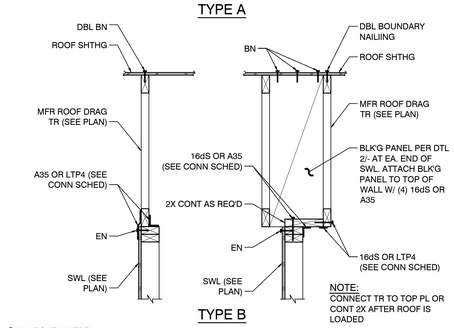


2 BLOCKING PANEL

NTS



TYPE A



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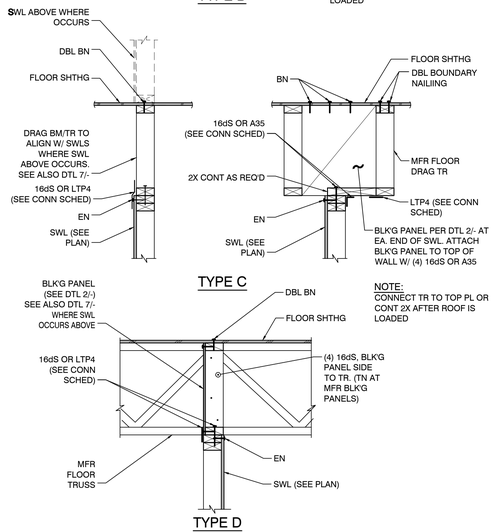
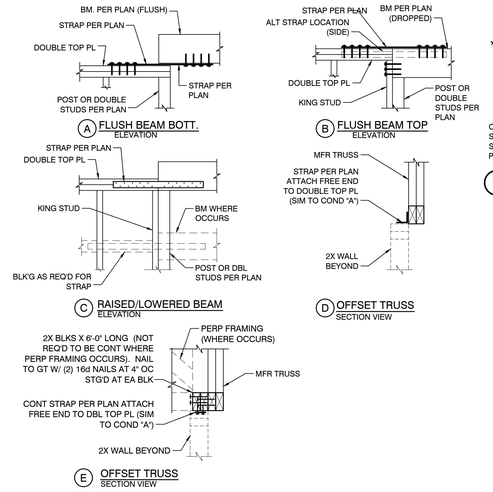


TYPE C

TYPE D



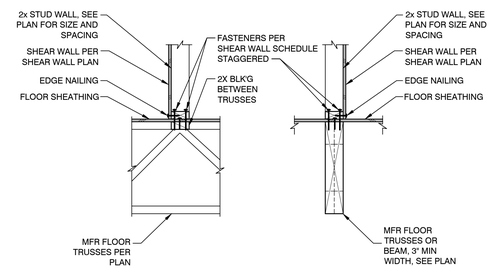
10 STRAP PLACEMENT

NTS

STRAP SCHEDULE	
DTL CALLED OUT ON PLANS	"L"
6	SEE SWL PLAN
6A	4'-0"
6B	5'-0"
6C	7'-0"

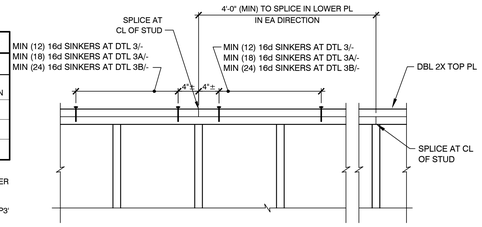
6 6A 6B 6C PLATE STEP

NTS



3 3A 3B TOP PLATE SPLICE

NTS



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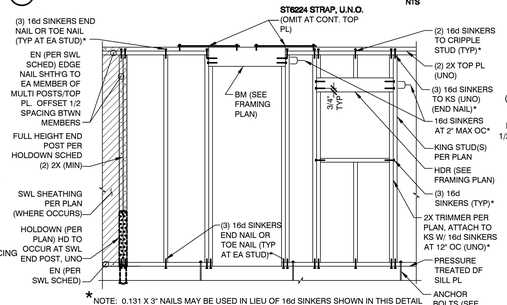
NTS



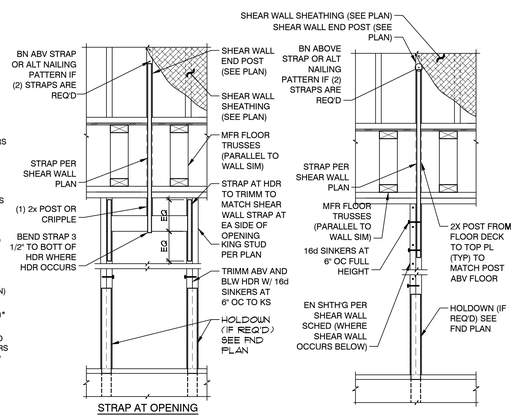
* NOTE: 0.131 X 3" NAILS MAY BE USED IN LIEU OF 16d SINKERS SHOWN IN THIS DETAIL

7 INT. S/W TO MFR FLOOR TRUSSES OR BEAM

NTS

8 TYPICAL WALL FRAMING

NTS



4 HOLD DOWN TRANSFER AT FLOOR

NTS

CONNECTION SCHEDULE		
	DFL / SPF	
SWL	A35 OR LTP4	16dS
NON-SWL	32" OC	8" OC
G1, G2, WSP ABW, PFG, PFH	32" OC	8" OC
P1	18" OC	5" OC
P2/P2a	12" OC	3" OC
P3	10" OC	2" OC (STG'D)
P4	8" OC	2" OC (STG'D)
P5	12" OC EA SIDE OF PL	SDS 1/4"X4 1/2" AT 4" OC
NOTE 1: DBL-SIDED SWLS, USE 1/2 OF THE SPACING SHOWN ABOVE		

1 SHEAR TRANSFER DTLS

NTS

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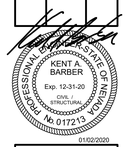
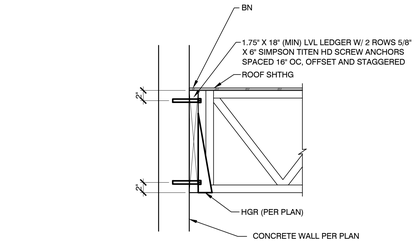
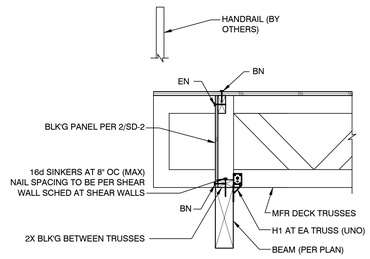


01/02/2020

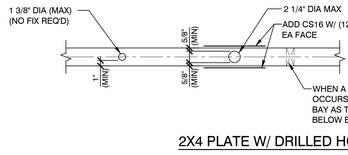
SD-2



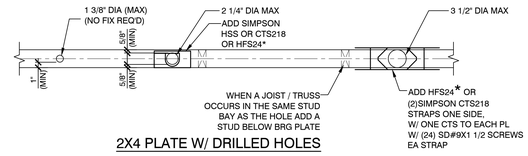
5 N.T.S.



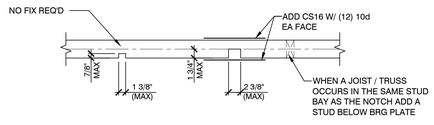
6 NTS



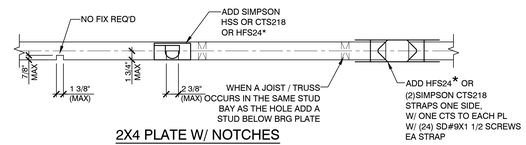
2X4 PLATE W/ DRILLED HOLES



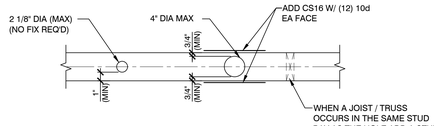
2X4 PLATE W/ DRILLED HOLES



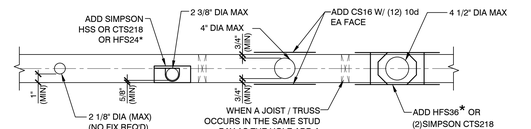
2X4 PLATE W/ NOTCHES



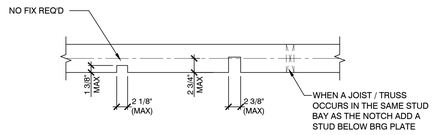
2X4 PLATE W/ NOTCHES



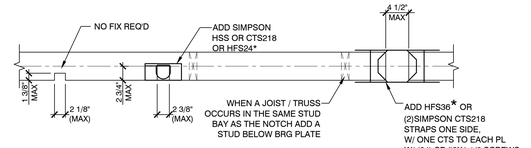
2X6 PLATE W/ DRILLED HOLES



2X6 PLATE W/ DRILLED HOLES



2X6 PLATE W/ NOTCHES



2X6 PLATE W/ NOTCHES

✱ HARDWARE MANUFACTURED BY HARDY FRAMES SEE MFR PRODUCT CATALOG

3 ALLOWABLE HOLES AND NOTCHES IN PLATES

INTERIOR WALLS (BEARING AND NON-BEARING) NTS

1 ALLOWABLE HOLES AND NOTCHES IN PLATES

EXTERIOR WALLS AND INTERIOR SHEAR WALLS NTS

2X4 STUD W/ NOTCHES

2X6 STUD W/ NOTCHES

2X4 STUD W/ NOTCHES

2X6 STUD W/ NOTCHES

2X4 STUD W/ DRILLED HOLES

2X6 STUD W/ DRILLED HOLES

2X4 STUD W/ DRILLED HOLES

2X6 STUD W/ DRILLED HOLES

4 ALLOWABLE HOLES AND NOTCHES IN STUDS

INTERIOR NON-BEARING WALLS NTS

2 ALLOWABLE HOLES AND NOTCHES IN STUDS

EXTERIOR WALLS, INTERIOR SHEAR WALLS AND INTERIOR BEARING WALLS NTS

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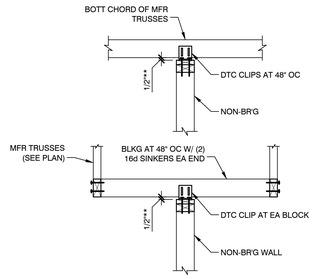
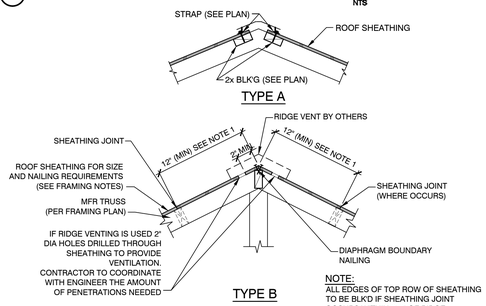
SD-2.1



** GAP TO BE 1/2" AFTER ROOF IS LOADED.

11 BRACING AT NON-BRG WALLS

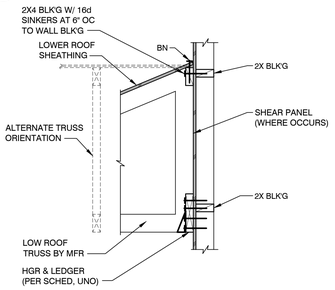
NTS



NOTE:
ALL EDGES OF TOP ROW OF SHEATHING TO BE BLK'D IF SHEATHING JOINT OCCURS WITHIN 2'-0" OF RIDGE

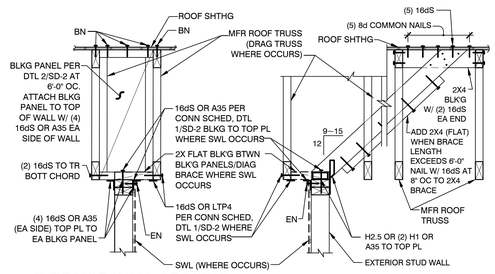
12 RIDGE BLOCKING

NTS



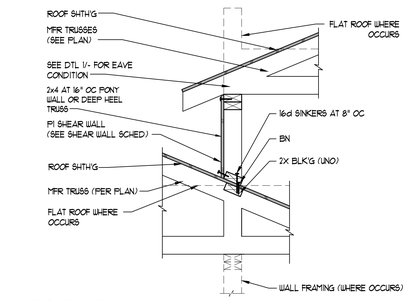
13 LOW ROOF TO WALL

NTS



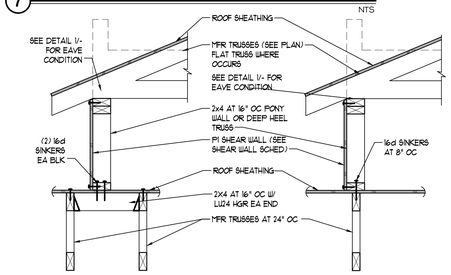
14 EXTERIOR WALL BRACING

NTS



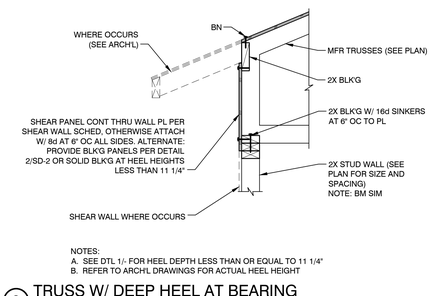
7 PONY WALL

NTS



8 PONY WALL

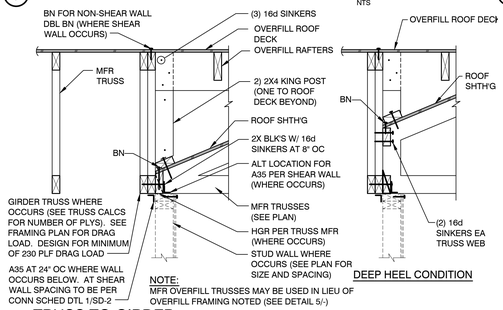
NTS



NOTES:
A. SEE DTL 1/- FOR HEEL DEPTH LESS THAN OR EQUAL TO 11 1/4"
B. REFER TO ARCH'L DRAWINGS FOR ACTUAL HEEL HEIGHT

9 TRUSS W/ DEEP HEEL AT BEARING

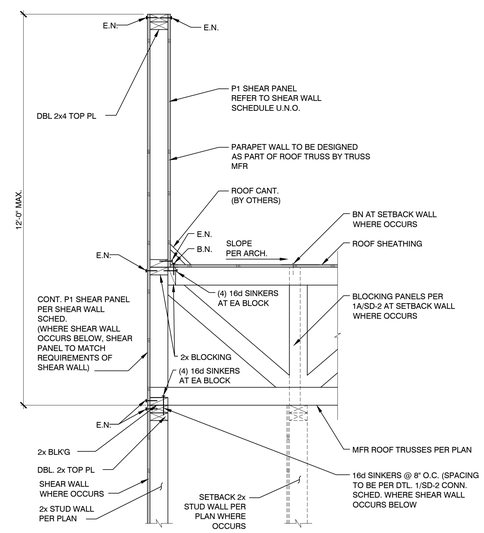
NTS



NOTE:
MFR OVERFILL TRUSSES MAY BE USED IN LIEU OF OVERFILL FRAMING NOTED (SEE DETAIL 5/-)

10 TRUSS TO GIRDER

NTS



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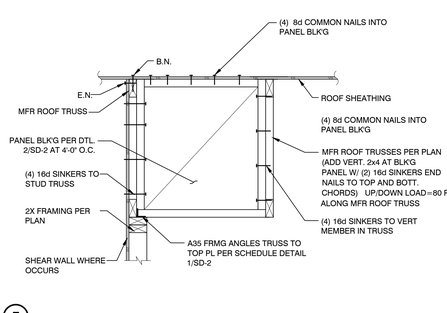
N.T.S.



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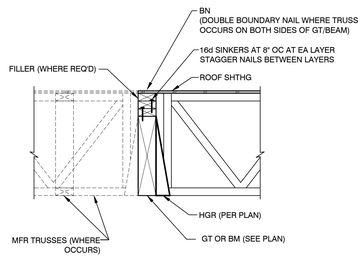
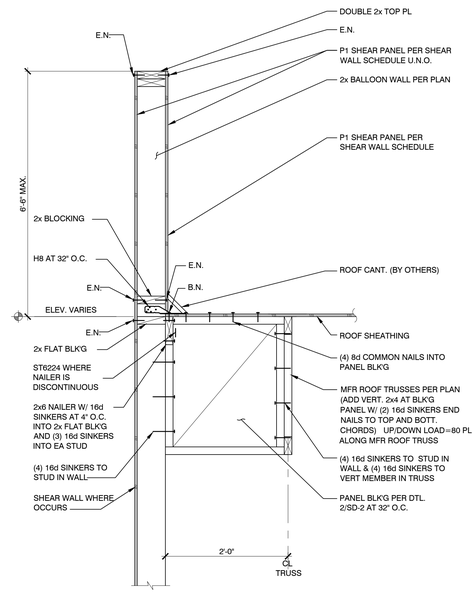
N.T.S.



6

N.T.S.



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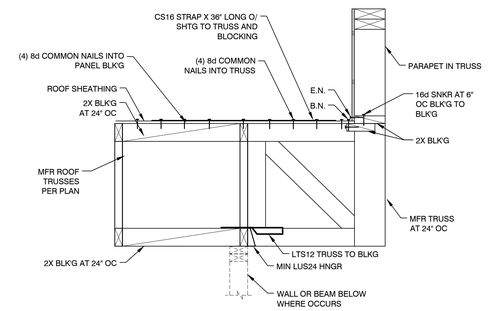

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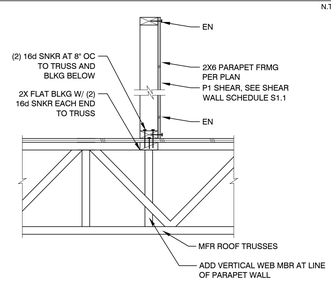

3

N.T.S.

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JOB NO: 1939-004-191
DESIGNED BY: KAB
DRAWN BY: KAB

ISSUED FOR:
CONSTRUCTION DOCUMENTS
DATE: 12-10-19

SHEET TITLE:
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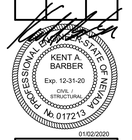
REVISIONS:

ASSURED DEVELOPMENT
ATHENS LOT 4



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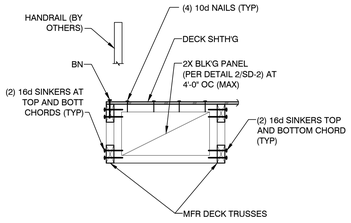
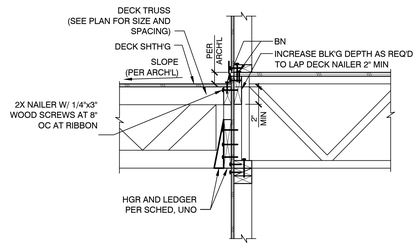
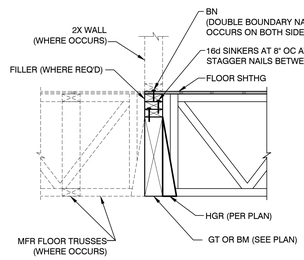
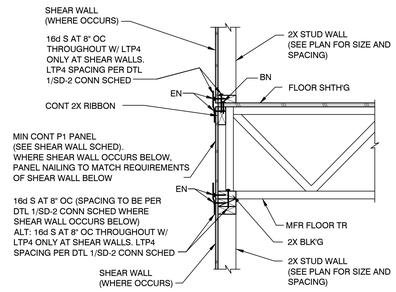




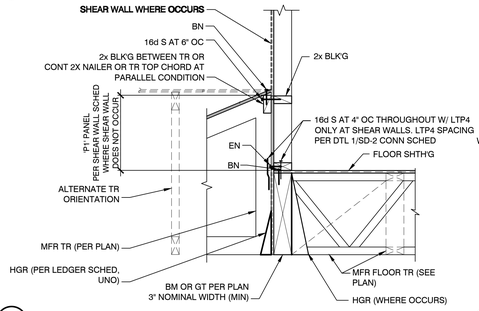
FOR INFORMATION NOT SHOWN SEE DTL 1/-





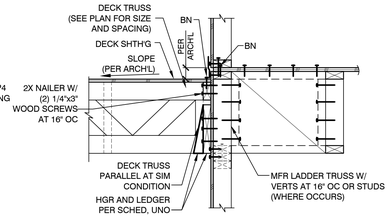
12 SHEAR WALL WHERE OCCURS

NTS



8 DECK LEDGER

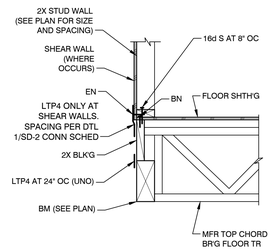
NTS



NOTE:
FOR INFO NOT SHOWN SEE DETAIL 2/-

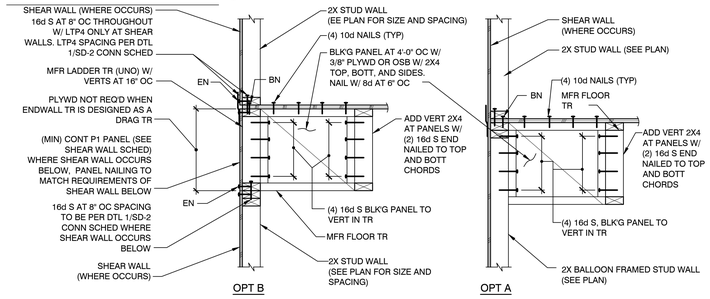
5 FLOOR TRUSS TO BEAM

NTS



1 FLR TR PERP. TO EXT WALL

NTS



13

REFER TO DTLS 1/- OR 2/- FOR INFORMATION NOT SHOWN

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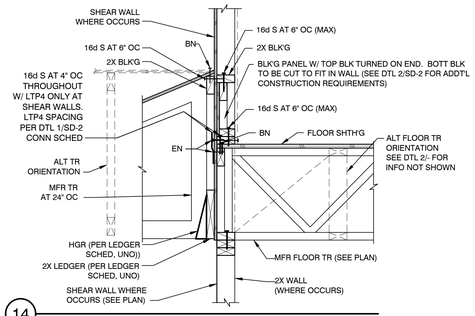

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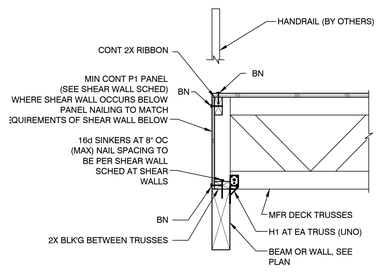
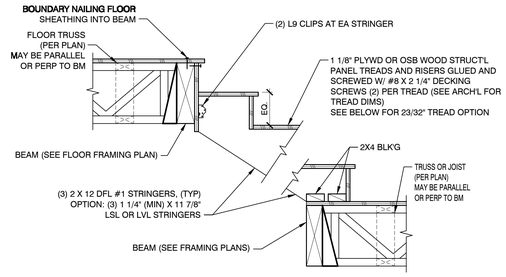


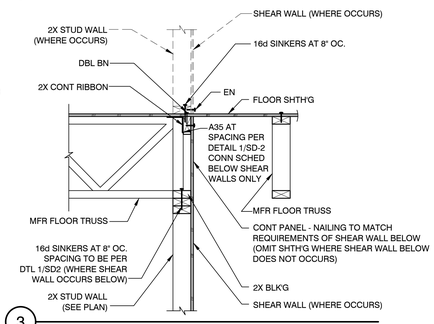
6 FLOOR TR TO BEAM

NTS



2 FLR TR PARALLEL TO EXT WALL

NTS



15

REFER TO DTLS 1/- OR 2/- FOR INFORMATION NOT SHOWN

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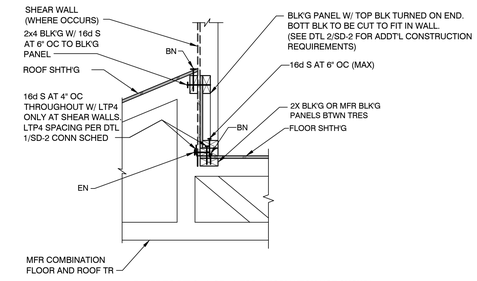

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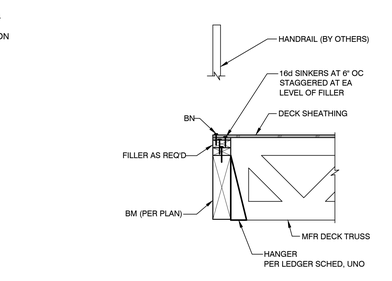
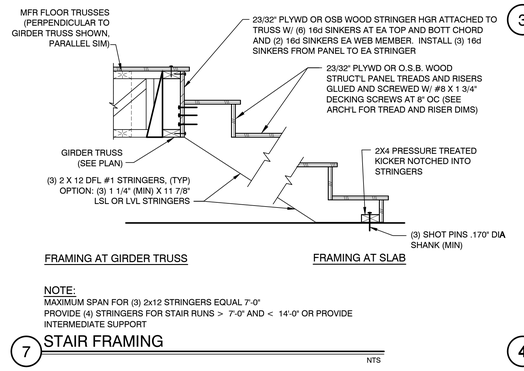


7 STAIR FRAMING

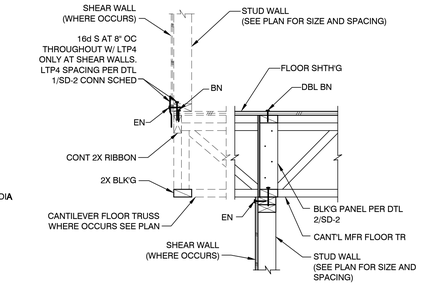
NTS



NOTE:
MAXIMUM SPAN FOR (3) 2x12 STRINGERS EQUAL 7'-0"
PROVIDE (4) STRINGERS FOR STAIR RUNS > 7'-0" AND < 14'-0" OR PROVIDE INTERMEDIATE SUPPORT

4

NTS



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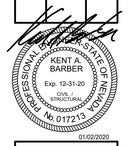

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