DRE STRUCTURAL DESIGN

1214 30th St Oakland, CA
Residential Renovation and Remodel Structural Calculations

Permit Set

June 3, 2020

Prepared For:

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STRUCTURAL NARRATIVE

The following calculations are for the new residential renovation located at 1214 30th St in Oakland, CA. Specifically, the calculations address the new gravity and lateral systems including the foundations.

Gravity System:
The gravity system is composed of wood framed roof and floors supported by a combination of wood joists and wood load-bearing walls. The walls are supported by shallow concrete foundations.

Lateral System:
The lateral system consists of wood flexible diaphragms spanning betweend wood shear walls.

All calculations are in accordance with the 2019 California Building Code.

Design Criteria

DETAILED DESIGN CRITERIA

BUILDING CODE

Governing Code:	2019 California Building Code
Authority Having Jurisdiction:	City of Oakland
Local Codes or Amendments:	2019 Building Code Amendments

SEOR STAMP

Dry Soil Density =	100 pcf
Wet Soil Density =	110 pcf
Passive Soil Pressure =	800 pcf
Active Soil Pressure =	45 pcf
At-Rest Soil Pressure =	60 pcf
Allowable Bearing Pressure, D+L =	1500 psf
Allowable Bearing Pressure, D+L+(E or W) =	2000 psf
Pier Skin Friction =	900 psf
Coefficient of Friction =	0.35
Soil Spring Modulus =	150 lbs/in

BUILDING SYSTEM DESCRIPTION

No. Stories:	2
Footprint:	990 ft^2
Floor Area:	1980 ft^2
Roof Area:	990 ft^2

Building Use:	Residential
Gravity System:	Wood load bearing walls and wood columns
Diaphragm System:	Plywood
Foundation System:	Shallow Foundations

DETAILED DESIGN CRITERIA

ASCE 7-16 Reference UNO:

SEISMIC DESIGN PARAMETERS

Analysis Procedure Used:	EQ (Equiv. Lat. Force, 12.8)			*Section 12.6*
Latitude:	37.8725 deg	Longitude:	-122.2831 deg	
Risk Category:	II	Use/Occupancy of Building Description		*Table 1.5-1*
I_E =	1.00	Importance Factor, Seismic		*Table 1.5-1*
I_P =	1.00	Importance Factor, Nonstructural Components		*13.1.3*
Soil Site Class =	D	Per Geotech Report, Site Class D otherwise		*Table 20.3-1*
S_S =	1.500 g	Mapped spectral response acceleration parameter		*USGS*
S_1 =	0.600 g	Mapped spectral response acceleration parameter		*USGS*
F_a =	1.2	Site coefficient		*Table 11.4-1*
F_v =	1.7	Site coefficient		*Table 11.4-2*
S_{DS} =	1.200 g	Design spectral response acceleration parameter		*11.4-3*
S_{D1} =	0.680 g	Design spectral response acceleration parameter		*11.4-4*
Seismic Design Category:	D			*Section 11.6*
Building System, N-S:	A. BEARING WALL SYSTEMS	15. Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance		*Table 12.2-1*
Building System, E-W:	A. BEARING WALL SYSTEMS	15. Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance		*Table 12.2-1*
Diaphragm=	Flexible Diaphragm	Plywood		
$\rho_{(N-S)}$ =	1.3	Redundancy factor, N-S		*12.3.4*
$\rho_{(E-W)}$ =	1.3	Redundancy factor, E-W		*12.3.4*
$R_{(N-S)}$ =	6.50	Response modification coefficient, N-S		*Table 12.2-1*
$R_{(E-W)}$ =	6.50	Response modification coefficient, E-W		*Table 12.2-1*
$\Omega_{o(N-S)}$ =	2.50	Overstrength factor, N-S		*Table 12.2-1*
$\Omega_{o(E-W)}$ =	2.50	Overstrength factor, E-W		*Table 12.2-1*
$C_{d(N-S)}$ =	4.00	Deflection amplification factor, N-S		*Table 12.2-1*
$C_{d(E-W)}$ =	4.00	Deflection amplification factor, E-W		*Table 12.2-1*
$T_{(N-S)}$ =	0.167 sec	Approximate Fundamental Period, N-S		*Section 12.8.2*
$T_{(E-W)}$ =	0.167 sec	Approximate Fundamental Period, E-W		*Section 12.8.2*
T_L =	8 sec	Long Period Transition Period		*USGS*
$V_{(N-S)}$ (ULT) =	0.185 *W	Base Shear, N-S, LRFD		*Section 12.8 or 12.14*
$V_{(N-S)}$ (ASD) =	0.129 *W	Base Shear, N-S, ASD		*Section 12.8 or 12.14*
$V_{(E-W)}$ (ULT) =	0.185 *W	Base Shear, E-W, LRFD		*Section 12.8 or 12.14*
$V_{(E-W)}$ (ASD) =	0.129 *W	Base Shear,E-W, LRFD		*Section 12.8 or 12.14*
Structural Irregularities	Horizontal:	none		*Table 12.3-1*
	Vertical:	none		*Table 12.3-2*

WIND DESIGN PARAMETERS

Wind Method Used:	Directional Procedure			*Chapter 27*
Basic Wind Speed =	110 MPH		Ultimate Design Wind Speed (3 second gust)	*Figure 26.5-1A,B or C*
Exposure Category:	C		Open Terrain	*26.7.3*
K_{zt} =	1.00		Topographic Factor	*26.8*
K_d =	0.85	Buildings	Directionality Factor	*Table 26.6-1*

DETAILED DESIGN CRITERIA

MATERIAL STRENGTH AND SPECIFICATIONS

CONCRETE:

Foundations, f'_c =	3000 psi	Designed for 2,500
Slab on grade, f'_c =	3000 psi	4,000 at 56 days at Interior
Structural walls, f'_c =	3000 psi	
Beams and Columns, f'_c =	3000 psi	
Fill over metal deck, f'_c =	3000 psi	
Elevated slabs, f'_c =	3000 psi	
Weight of normal weight concrete =	150 pcf	
Weight of lightweight concrete =	110 pcf	

CONCRETE REINFORCING:

Reinforcing Steel, f_y =	60 ksi	ASTM A615, Grade 60
Reinforcing Steel ties, f_y =	40 ksi	ASTM A615, Grade 40

DETAILED DESIGN CRITERIA

WOOD CONSTRUCTION:

6x Posts, F_b =	1200 psi	Douglas Fir #1
6x Beam, F_b =	1350 psi	Douglas Fir #1
4x Posts & Beams, F_b =	1000 psi	Douglas Fir #1
2x Joists & Rafters, F_b =	900 psi	Douglas Fir #2
2x Studs, F_b =	900 psi	Douglas Fir #2
Sheathing	PS1 / PS2	
Connections	Simpson Strong-Tie	
Glued-Laminated Beam (GLB), F_b =	2400 psi	24F-V4 (DF/DF) simple span, 24F-V8 (DF/DF) continuous span
Exterior GLB, F_b =	2000 psi	20F-V12 (AC/AC) simple span, 20F-V13 (AC/AC) continuous span
Parallel Strand Lumber (PSL), F_b =	2900 psi	Grade 2.0E
Laminated Veneer Lumber (LVL), F_b =	2600 psi	Grade 1.9E
Laminated Strand Lumber (LSL), F_b =	2600 psi	Grade 1.9E

DEFLECTION & VIBRATION DESIGN CRITERIA

		LIVE		DEAD + LIVE		0.6 WIND	
	Finish	Design	Code Min	Design	Code Min	Design	Code Min
Roof Framing	Ceiling	L/240	L/240	L/180	L/180	L/240	L/240
Floor Framing	-	L/360	L/360	L/240	L/240	-	-
Wall Framing	Flexible	-	-	-	-	L/120	L/120

GRAVITY / SEISMIC FLAT WEIGHT TAKEOFF (PSF)

Roof Load

CBC Live Load Category: 26. Roof: ordinary
Slope: 4.00:12

Material		Sloped	Deck	Joists	Girders	Seismic
Roofing		Y	3.0	3.0	3.0	3.0
1/2" PLY SHTG		Y	1.7	1.7	1.7	1.7
MEP		Y		1.0	1.0	1.0
Ceiling		Y		2.2	2.2	2.2
Attic Framing		Y				10.0
2x Joists		Y			2.2	2.2
Girders		Y				1.5
Misc. (includes .5 psf for insulation)		Y	2.0	2.0	2.0	2.0
Dead Load			6.7	9.9	12.1	23.6
Dead Load - Horiz Projection			7.1	10.4	12.8	24.9
Partitions			0.0	0.0	0.0	0.0
Live Load			20.0	20.0	20.0	20.0
Live Load - Reduced	**R_2 =** 1.00		20.0	20.0	20.0	20.0
Total Load			27.1	30.4	32.8	44.9

GRAVITY / SEISMIC FLAT WEIGHT TAKEOFF (PSF)

Floor Load

CBC Live Load Category: 25. Residential: Other

Material	Sloped	Deck	Joists	Girders	Seismic
Flooring	N	1.0	1.0	1.0	1.0
Sheathing / Decking	N	2.5	2.5	2.5	2.5
	N			0.0	0.0
M.E.P.	N		1.0	1.0	1.0
Ceiling	N		2.2	2.2	2.2
Joists	N			2.5	2.5
Girders	N				1.5
Columns	N				0.5
Misc.	N	1.5	1.5	1.5	1.5
Dead Load		5.0	8.2	10.7	12.7
Dead Load - Horiz Projection		5.0	8.2	10.7	12.7
Partitions		0.0	0.0	0.0	0.0
Live Load		40.0	40.0	40.0	0.0
Live Load - Reduced R_2 = 1.00		40.0	40.0	40.0	0.0
Total Load		45.0	48.2	50.7	12.7

GRAVITY / SEISMIC WALL WEIGHT TAKEOFF (PSF)

Exterior Wall - 2x4 Stud

Material	Weight
2x4 @ 16" OC	1.1
Insulation	1.0
1/2" plywood	1.7
Cement Plaster /Gyp	5.0
MEP	1.0
Misc	2.7
TOTAL	10.0

Interior Wall - 2x4 Stud

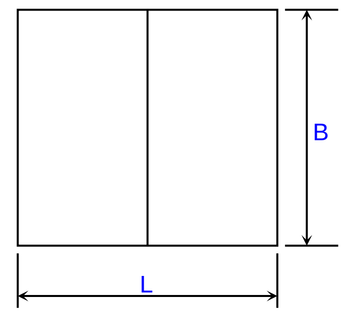
Material	Weight
2x4 @ 16" OC	1.1
Gyp board (two sides)	5.0
Misc	1.0
TOTAL	8

WIND

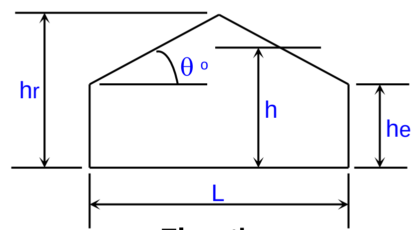
WIND LOADING ANALYSIS - Wall Components and Cladding
Per ASCE 7-16 Code for Buildings of Any Height
Using Part 1 & 3: Analytical Procedure (Section 30.4 & 30.6)

Input Data:

Wind Speed, V =	110	mph (Wind Map, Figure 26.5-1A-C)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	32.00	ft. (hr >= he)
Eave Height, he =	32.00	ft. (he <= hr)
Building Width =	45.00	ft. (Normal to Building Ridge)
Building Length =	22.00	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 28.6-1 & Figure 26.11-1)
Hurricane Region?	N	
Component Name =	Wall	(Girt, Siding, Wall, or Fastener)
Effective Area, Ae =	100	ft.^2 (Area Tributary to C&C)



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	0.00	deg.
Mean Roof Ht., h =	32.00	ft. (h = he, for roof angle <=10 deg.)

Wall External Pressure Coefficients, GCp:

GCp Zone 4 Pos. =	0.74	(Fig. 30.4-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 5 Pos. =	0.74	(Fig. 30.4-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 4 Neg. =	-0.83	(Fig. 30.4-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 5 Neg. =	-0.94	(Fig. 30.4-1, GCp is reduced by 10% for roof angle <=10 deg.)

Positive & Negative Internal Pressure Coefficients, GCpi (Figure 26.11-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If z <= 15 then: Kz = 2.01*(15/zg)^(2/α) , If z > 15 then: Kz = 2.01*(z/zg)^(2/α) (Table 30.3-1)

α =	9.50	(Table 26.9-1)
zg =	900	(Table 26.9-1)
Kh =	1.00	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 30.3.2, Eq. 30.3-1)

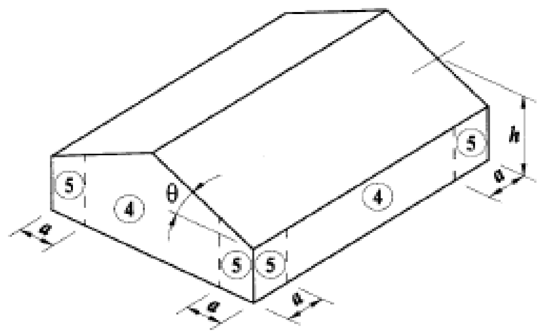
qh = 26.22 psf   qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)

Design Net External Wind Pressures (Sect. 30.4 & 30.6):
For h <= 60 ft.: p = qh*((GCp) - (+/-GCpi)) (psf)
For h > 60 ft.: p = q*(GCp) - qi*(+/-GCpi) (psf)
where: q = qz for windward walls, q = qh for leeward walls and side walls
qi = qh for all walls (conservatively assumed per Sect. 30.6)

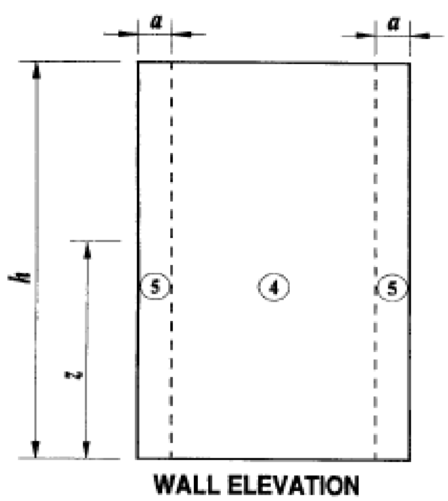
Wind Load Tabulation for Wall Components & Cladding							
Component	z	Kh	qh	p = Net Design Pressures (psf)			
	(ft.)		(psf)	Zone 4 (+)	Zone 4 (-)	Zone 5 (+)	Zone 5 (-)
Wall	0	1.00	26.22	24.15	-26.51	24.15	-29.42
	15.00	1.00	26.22	24.15	-26.51	24.15	-29.42
	20.00	1.00	26.22	24.15	-26.51	24.15	-29.42
	25.00	1.00	26.22	24.15	-26.51	24.15	-29.42
	30.00	1.00	26.22	24.15	-26.51	24.15	-29.42
For z = hr:	32.00	1.00	26.22	24.15	-26.51	24.15	-29.42
For z = he:	32.00	1.00	26.22	24.15	-26.51	24.15	-29.42
For z = h:	32.00	1.00	26.22	24.15	-26.51	24.15	-29.42

Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 5 (end zones), 'a' = 3.00 ft.
3. Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.
4. References : a. ASCE 7-10, "Minimum Design Loads for Buildings and Other Structures".
b. "Guide to the Use of the Wind Load Provisions of ASCE 7-02"
by: Kishor C. Mehta and James M. Delahay (2004).

Wall Components and Cladding:



Wall Zones for Buildings with h <= 60 ft.



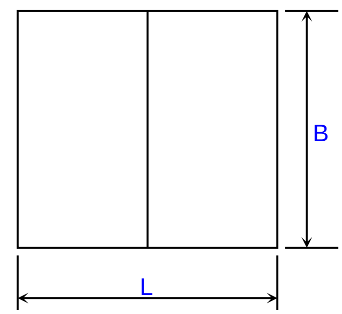
Wall Zones for Buildings with h > 60 ft.

WIND LOADING ANALYSIS - Roof Components and Cladding

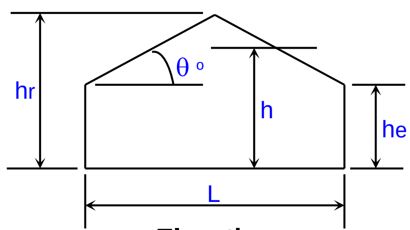
Per ASCE 7-16 Code for Bldgs. of Any Height with Gable Roof θ <= 45° or Monoslope Roof θ <= 3°
Using Part 1 & 3: Analytical Procedure (Section 30.4 & 30.6)

Input Data:

Wind Speed, V =	110	mph (Wind Map, Figure 26.5-1A-C)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	32.00	ft. (hr >= he)
Eave Height, he =	32.00	ft. (he <= hr)
Building Width =	22.00	ft. (Normal to Building Ridge)
Building Length =	45.00	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 28.6-1 & Figure 26.11-1)
Hurricane Region?	N	
Component Name =	Joist	(Purlin, Joist, Decking, or Fastener)
Effective Area, Ae =	100	ft.^2 (Area Tributary to C&C)
Overhangs? (Y/N)	Y	(if used, overhangs on all sides)



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	0.00	deg.
Mean Roof Ht., h =	32.00	ft. (h = he, for roof angle <=10 deg.)

Roof External Pressure Coefficients, GCp:

GCp Zone 1-3 Pos. =	0.20	(Fig. 30.4-2A, 30.4-2B, and 30.4-2C)
GCp Zone 1 Neg. =	-1.60	(Fig. 30.4-2A, 30.4-2B, and 30.4-2C)
GCp Zone 2 Neg. =	-1.60	(Fig. 30.4-2A, 30.4-2B, and 30.4-2C)
GCp Zone 3 Neg. =	-0.80	(Fig. 30.4-2A, 30.4-2B, and 30.4-2C)

Positive & Negative Internal Pressure Coefficients, GCpi (Figure 26.11-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If z <= 15 then: Kz = 2.01*(15/zg)^(2/α) , If z > 15 then: Kz = 2.01*(z/zg)^(2/α) (Table 30.3-1)

α =	9.50	(Table 26.9-1)
zg =	900	(Table 26.9-1)
Kh =	1.00	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 30.3.2, Eq. 30.3-1)

qh = 26.22 psf   qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)

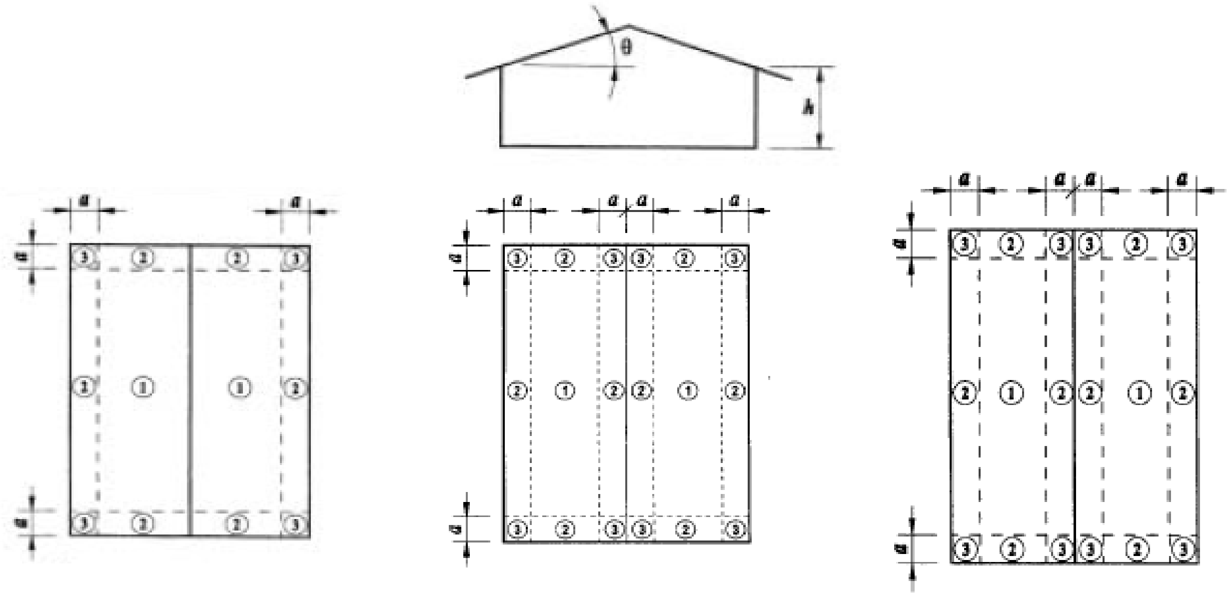
Design Net External Wind Pressures (Sect. 30.4 & 30.6):
For h <= 60 ft.: p = qh*((GCp) - (+/-GCpi)) (psf)
For h > 60 ft.: p = q*(GCp) - qi*(+/-GCpi) (psf)
where: q = qh for roof
qi = qh for roof (conservatively assumed per Sect. 30.6)

Wind Load Tabulation for Roof Components & Cladding

Component	z (ft.)	Kh	qh (psf)	p = Net Design Pressures (psf) Zone 1,2,3 (+)	Zone 1 (-)	Zone 2 (-)	Zone 3 (-)
Joist	0	1.00	26.22	9.96	-46.66	-46.66	-25.69
	15.00	1.00	26.22	9.96	-46.66	-46.66	-25.69
	20.00	1.00	26.22	9.96	-46.66	-46.66	-25.69
	25.00	1.00	26.22	9.96	-46.66	-46.66	-25.69
	30.00	1.00	26.22	9.96	-46.66	-46.66	-25.69
For z = hr:	32.00	1.00	26.22	9.96	-46.66	-46.66	-25.69
For z = he:	32.00	1.00	26.22	9.96	-46.66	-46.66	-25.69
For z = h:	32.00	1.00	26.22	9.96	-46.66	-46.66	-25.69

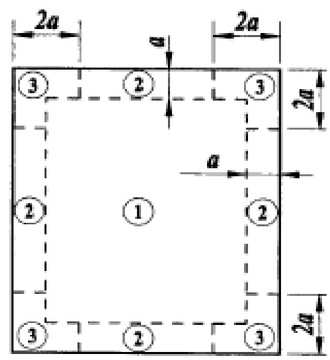
Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 2 (edge), 'a' = 3.00 ft.
3. Width of Zone 3 (corner), 'a' = 3.00 ft.
4. For monoslope roofs with θ <= 3 degrees, use Fig. 30.4-2A for 'GCp' values with 'qh'.
5. For buildings with h > 60' and θ > 10 degrees, use Fig. 30.6-1 for 'GCpi' values with 'qh'.
6. For all buildings with overhangs, use Fig. 30.4-2B for 'GCp' values per Sect. 30.10.
7. If a parapet >= 3' in height is provided around perimeter of roof with θ <= 10 degrees, Zone 3 shall be treated as Zone 2.
8. Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.
9. References : a. ASCE 7-02, "Minimum Design Loads for Buildings and Other Structures".
b. "Guide to the Use of the Wind Load Provisions of ASCE 7-02" by: Kishor C. Mehta and James M. Delahay (2004).

Roof Components and Cladding:



θ <= 7 deg.   7 deg. < θ <= 27 deg.   27 deg. < θ <= 45 deg.

Roof Zones for Buildings with h <= 60 ft.
(for Gable Roofs <= 45^o and Monoslope Roofs <= 3^o)



ROOF PLAN

Roof Zones for Buildings with h > 60 ft.
(for Gable Roofs <= 10^o and Monoslope Roofs <= 3^o)

SEISMIC

USGS DESIGN MAP SUMMARY REPORT

ASCE Seismic Base Shear

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St Oakland, CA

1214 30th St Oakland, CA

Risk Category Calculations per ASCE 7-16

Risk Category of Building or Other Structure :	"II" : All Buildings and other structures except those listed as Category I, III, and IV		SCE 7-16, Page 4, Table 1.5-1
Seismic Importance Factor	=	1	ASCE 7-16, Page 5, Table 1.5-2

ASCE 7-16 11.4.2

Max. Ground Motions, 5% Damping						
S_S	=	1.50 g, 0.2 sec response	Latitude	=	37.823	deg North
S_1	=	0.60 g, 1.0 sec response	Longitude	=	122.283	deg West

Conforms to ASCE 7 Section 12.8.1.3: Regular structure with period of 0.5 s or less, SDS limited to max of 0.7*SDS or 1.0 for calculatior

Site Class, Site Coeff. and Design Category

Site ClassificatioD" : Shear Wave Velocity 600 to 1,200 ft/sec		=	D	ASCE 7-16 Table 20.3-1
Site Coefficients Fa & Fv	Fa	=	1.20	ASCE 7-16 Table 11.4-1 & 11.4-2
(using straight-line interpolation from table val	Fv	=	1.50	
Maximum Considered Earthquake Accelerat	S_{MS} = Fa * Ss	=	1.800	ASCE 7-16 Eq. 11.4-1
	S_{M1} = Fv * S1	=	0.900	ASCE 7-16 Eq. 11.4-2
Design Spectral Acceleration	$S_{DS} = S_{MS}$ * 2/3	=	1.200	ASCE 7-16 Eq. 11.4-3
	$S_{D1} = S_{M1}$ * 2/3	=	0.600	ASCE 7-16 Eq. 11.4-4
Seismic Design Category		=	D	E 7-16 Table 11.6-1 & -2

IBC2018 (1613), ASCE 7-16 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

DSA Project?	**NO**		
Soil Site Class	D	Table 20.3-1, Default = D	
Response Spectral Acc. (0.2 sec) Ss =	**1.500g**	= 150.00%g	Figure 22-1, 22-3, 22-5, and 22-6
Response Spectral Acc.(1.0 sec) S1 =	**0.600g**	= 60.00%g	Figure 22-2, 22-4, 22-5, and 22-6
Site Coefficient F_a =	1.200		Table 11.4-1
Site Coefficient F_v =	1.700		Table 11.4-2
Max Considered Earthquake Acc. S_{MS} =	$F_a.S_s$	= 1.800	(11.4-1)
Max Considered Earthquake Acc. S_{M1} =	$F_v.S_1$	= 1.020	(11.4-2)
@ 5% Damped Design S_{DS} =	$2/3(S_{MS})$	= 1.200	(11.4-3)
S_{D1} =	$2/3(S_{M1})$	= 0.680	(11.4-4)
Building Risk Categories	II	Standard	Table 1.5-1
Redundancy Factor =	1.3		Section 12.3.4
Design Category Consideration:	Flexible Diaphragm		Section 12.3
Seismic Design Category for 0.1sec	D		Table 11.6-1
Seismic Design Category for 1.0sec	D		Table 11.6-2
S1 < .75g	NA		Section 11.6
Since Ta < .8Ts (see below), SDC =	D	Control (exception of Section 11.6 does not apply)	

IBC - Comply with Seismic Design Category D

IRC - Seismic Design Category = **E** T-R301.2.2.1.1

12.8 Equivalent lateral force procedure

Seismic Force Resisting System: A. BEARING WALL SYSTEMS T-12.2-1

15. Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance

C_t =	0.02		x = 0.75	T-12.8-2
Building ht. H_n =	**35 ft**		Limited Building Height (ft) = 65	
C_u =	1.4	for S_{D1} of	0.680g	Table 12.8-1
Approx Fundamental period, T_a =	$C_t(h_n)^x$	= 0.288	12.8-7	T_L = **8 Sec**
Calculated T shall not exceed ≤	Cu.Ta	= 0.403		Use T = 0.167 sec.
0.8Ts =	$0.8(S_{D1}/S_{DS})$	= 0.453	Control (exception of Section 11.6 does not apply)	
Is structure Regular & ≤ 5 stories ?	Yes			12.8.1.3
Response Spectral Acc.(0.2 sec) S_s =	1.500g			Max Ss ≤ 1.5g
F_a =	1.00			
@ 5% Damped Design S_{DS} =	$⅔(F_a.S_s)$	= 1.200g		11.4-3
Response Modification Coef. R =	**6.5**			Table-12.2-1
Over Strength Factor Ω_o =	**2.5**			foot note g
Importance factor I =	**1**			Table 1.5-1
Seismic Base Shear V =	$C_s W$			
C_s =	$\frac{S_{DS}}{R/I}$	=0.185		(12.8-2)
or need not to exceed, C_s =	$\frac{S_{D1}}{(R/I).T}$	= 0.625	For T≤ T_L	(12.8-3)
or C_s =	$\frac{S_{D1}T_L}{T^2(R/I)}$	N/A	For T > T_L	(12.8-4)
C_s shall not be less than $.044S_{DS}I$ =	0.053	0.01		(12.8-5)
Min C_s =	$0.5S_1I/R$	= 0.046	For S_1 ≥ 0.6g	(12.8-6)
Use C_s =	0.185			
Design base shear V (ULT)=	**0.185 W**			
Design base shear V (ASD)=	**0.129 W**			

***Control* Insert into appropriate load combinations**

Deflection Amplification factor C_d = 4 Use with ASCE 12.8.6, 12.8.7, and 12.9.2

North-South Diaphragm Weight Information:

Level	Area (sq ft)	Diaphragm Unit Weight (psf)	Diaphragm Weight (kips)	Wall Unit Weight (psf)	Wall Trib Width (ft)	Wall Length (ft)	Wall Weight (kips)	Level Weight (kips)
Roof	990	24.9	25	10	4.0	100.0	4	**29**
2nd	990	12.7	13	10	8.0	100.0	8	**21**
Σ			37				12	**49.2**

East-West Diaphragm Weight Information:

Level	Area (sq ft)	Diaphragm Unit Weight (psf)	Diaphragm Weight (kips)	Wall Unit Weight (psf)	Wall Trib Width (ft)	Wall Length (ft)	Wall Weight (kips)	Level Weight (kips)
Roof	990	24.9	25	10	4.0	88.0	4	**28**
2nd	990	12.7	13	10	8.0	88.0	7	**19.6**
Σ			37				11	**47.8**

ROOF PLAN/2ND FLOOR

1ST FLOOR PLAN

Seismic Story Force Distribution based on ASCE 7-16

S_{DS}= 1.200 Ta Period (12.8-7)= 0.167 k= 1.0 (12.8.3)
$I_{seismic}$= 1.00 rho (ρ) = 1.3

ASD OR ULT? ASD
V(ASD)= 0.129 Base V (ASD)= 6.4

Story Force Vertical Distribution (ASCE 7-16 12.8.3)

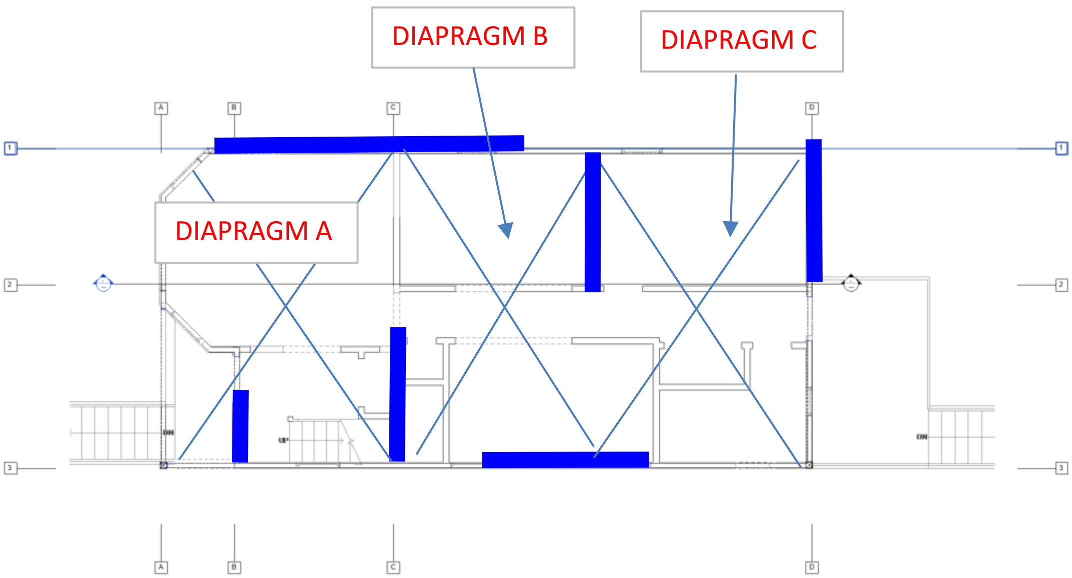
Level	w_x	h_x (ft.)	h_x^k	$w_x h_x^k$	Fx , ASD	Fx w/rho, ASD	Cv_x%
Roof	28.6	16.0	16.0	458	**4.678**	**6.1**	**73.6**
2nd Floor	20.6	8.0	8.0	165	**1.681**	**2.2**	**26.4**
Σ	49.2			623	6.4	8.3	

Vertical Diaphragm Distribution (ASCE 7-16 12.10.1.1)

Level	w_x	Σw_x	F_x	ΣF_x	Fpx , ASD	
Roof	29	29	4.7	4.68	**6.3**	<--Fpmin
2nd Floor	20.6	49	2.2	6.9	**4.5**	<--Fpmin
Σ	49.2		6.9			

Where Fpmin = rho * 0.2 SDS I Wx * 0.7 , ASD
Fpmax = rho * 0.4 SDS I Wx * 0.7 , ASD

Shearwall Layout - 2nd Floor



For Design:

1. *Since 3rd floor is sheathed with plywood, assume flexible diaphragm.*
2. *Distribute seismice forces by tributary area*
3. *See above for shear wall layout.*
4. *Assume roof diaphragm to be a series of simply supported beams*
5. Distribute Forces by diapragm :

Base Shear =	6.1	kips (floor base shear)
Fpx =	6.3	kips (diaphragm force)
Roof Area =	990	sqaure feet
v =	6.14	psf (story force)
v =	6.32	psf (diapragm force)

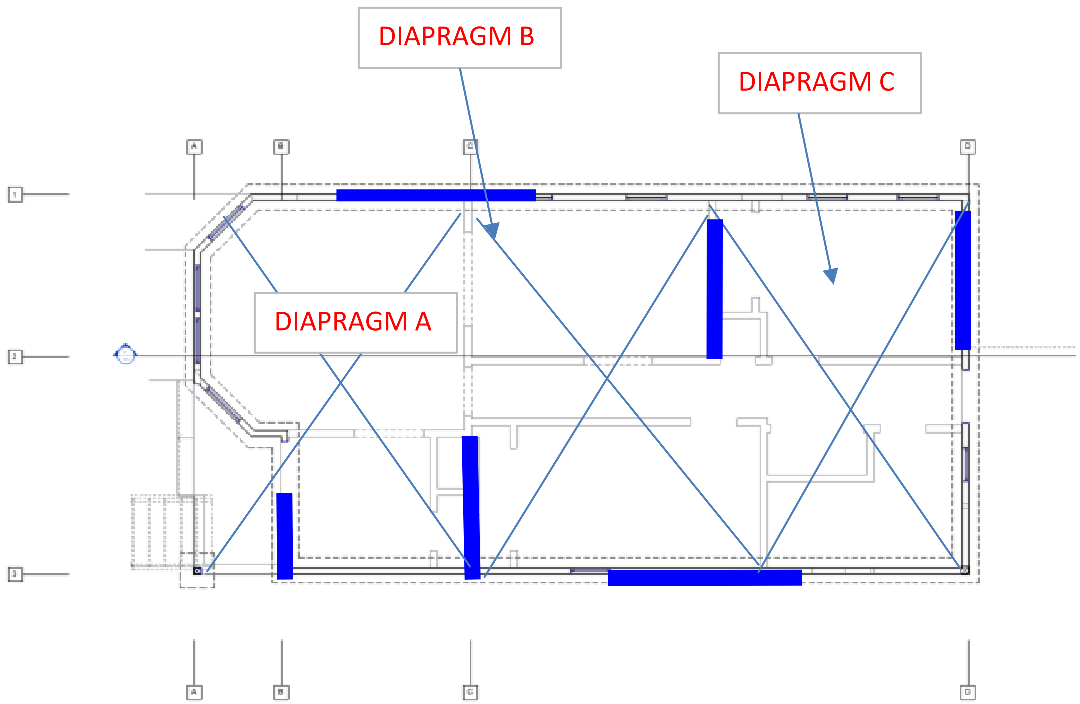
Story Force:

Diaphram A =	352	sqaure feet ---------- >	Vdiap(a) =	2162	lbs
Diaphram B =	319	sqaure feet ---------- >	Vdiap(b) =	1959	lbs
Diaphram C =	319	sqaure feet ---------- >	Vdiap(b) =	1959	lbs

Diaprhagm Force:

Diaphram A =	352	sqaure feet ---------- >	Vdiap(a) =	2223	lbs
Diaphram B =	319	sqaure feet ---------- >	Vdiap(b) =	2015	lbs
Diaphram C =	319	sqaure feet ---------- >	Vdiap(b) =	2015	lbs

Shearwall Layout - 1st Floor



For Design:

1. *Since 2nd floor is sheathed with plywood, assume flexible diaphragm.*
2. *Distribute seismice forces by tributary area*
3. *See above for shear wall layout.*
4. *Assume roof diaphragm to be a series of simply supported beams*
5. Distribute Forces by diapragm :

Base Shear =	2.2	kips (roof base shear)
Fpx =	4.5	kips (diaphragm force)
Roof Area =	990	sqaure feet
v =	2.21	psf (story force)
v =	4.54	psf (diapragm force)

STORY FORCE

Diaphram A =	352	sqaure feet ---------- >	Vdiap(a) =	777	lbs
Diaphram B =	319	sqaure feet ---------- >	Vdiap(b) =	704	lbs
Diaphram C =	319	sqaure feet ---------- >	Vdiap(b) =	704	lbs

DIAPHRAGM FORCE

Diaphram A =	352	sqaure feet ---------- >	Vdiap(a) =	1598	lbs
Diaphram B =	319	sqaure feet ---------- >	Vdiap(b) =	1448	lbs
Diaphram C =	319	sqaure feet ---------- >	Vdiap(b) =	1448	lbs

IBC2018, ASCE 7-16 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

Level	North/South		East/West		Area (sf)
	F_X (ASD)	F_{PX} (ASD)	F_X (ASD)	F_{PX} (ASD)	
Roof	2.2 kips	2.2 kips	2.2 kips	2.2 kips	352
2nd floor	x	x	x	x	x
2nd Floor	x	x	x	x	x
	x	x	x	x	x

Seismic Loading Level: Roof
Loading Direction: North/South

F_X = **2.2** kips (ASD)
F_{PX} = **2.2** kips (ASD)
Total Level Area = 352 ft^2
% of Total F_x = 50

Gridline: B & C

Span Type	Diaphragm Span		Length	Width	Area	Story Force	Diaphragm Force	Distributed Load	Diaphragm Shear	TC Couple
			(ft)	(ft)	(ft^2)	(kips)	(kips)	(plf)	(plf)	(lbs)
Simple	B	C	16.0	22.0	352	1.08	1.11	69	**51**	**101**
						1.08	**1.11**			

Seismic Loading Level: Roof
Loading Direction: East/West

F_X = **2.2** kips (ASD)
F_{PX} = **2.2** kips (ASD)
Total Level Area = 352 ft^2
% of Total F_x = 50

Gridline: 1 & 3

Span Type	Diaphragm Span		Length	Width	Area	Story Force	Diaphragm Force	Distributed Load	Diaphragm Shear	TC Couple
			(ft)	(ft)	(ft^2)	(kips)	(kips)	(plf)	(plf)	(lbs)
Simple	1	3	22.0	16.0	352	1.08	1.08	49	**68**	**186**
						1.08	**1.08**			

IBC2018, ASCE 7-16 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

Level	North/South		East/West		Area (sf)
	F_X (ASD)	F_{PX} (ASD)	F_X (ASD)	F_{PX} (ASD)	
Roof	2.0 kips	2.0 kips	2.0 kips	2.0 kips	319
3rd Floor - Diaph B	x	x	x	x	x
2nd Floor	x	x	x	x	x
	x	x	x	x	x

Seismic Loading Level: Roof — F_X = **2.0** **kips (ASD)**
Loading Direction: North/South — F_{PX} = **2.0** **kips (ASD)**
Total Level Area = 319 **ft²**
% of Total F_x = 50

Gridline: C & C.5

Span Type	Diaphragm Span		Length (ft)	Width (ft)	Area (ft²)	Story Force (kips)	Diaphragm Force (kips)	Distributed Load (plf)	Diaphragm Shear (plf)	TC Couple (lbs)
Simple	C	C.5	10.0	22.0	319	0.98	1.01	101	**46**	**57**
						0.98	**1.01**			

Seismic Loading Level: Roof — F_X = **2.0** **kips (ASD)**
Loading Direction: East/West — F_{PX} = **2.0** **kips (ASD)**
Total Level Area = 319 **ft²**
% of Total F_x = 50

Gridline: 1 & 3

Span Type	Diaphragm Span		Length (ft)	Width (ft)	Area (ft²)	Story Force (kips)	Diaphragm Force (kips)	Distributed Load (plf)	Diaphragm Shear (plf)	TC Couple (lbs)
Simple	1	5	22.0	10.0	319	0.98	0.98	45	**98**	**269**
						0.98	**0.98**			

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Level	North/South		East/West		Area (sf)
	F_X (ASD)	F_{PX} (ASD)	F_X (ASD)	F_{PX} (ASD)	
Roof	2.0 kips	2.0 kips	2.0 kips	2.0 kips	319
3rd Floor	x	x	x	x	x
2nd Floor	x	x	x	x	x
	x	x	x	x	x

Seismic Loading Level: Roof | F_X = **2.0** **kips (ASD)**
Loading Direction: North/South | F_{PX} = **2.0** **kips (ASD)**
Total Level Area = 319 **ft²**
% of Total F_x = 50

Gridline: C.5 & D

Span Type	Diaphragm Span		Length (ft)	Width (ft)	Area (ft²)	Story Force (kips)	Diaphragm Force (kips)	Distributed Load (plf)	Diaphragm Shear (plf)	TC Couple (lbs)
Simple	C.5	D	10.0	22.0	319	0.98	1.01	201	**46**	**114**
						0.98	**1.01**			

Seismic Loading Level: Roof | F_X = **2.0** **kips (ASD)**
Loading Direction: East/West | F_{PX} = **2.0** **kips (ASD)**
Total Level Area = 319 **ft²**
% of Total F_x = 50

Gridline: 1 & 3

Span Type	Diaphragm Span		Length (ft)	Width (ft)	Area (ft²)	Story Force (kips)	Diaphragm Force (kips)	Distributed Load (plf)	Diaphragm Shear (plf)	TC Couple (lbs)
Simple	1	3	22.0	10.0	319	0.98	1.01	46	**101**	**277**
						0.98	**1.01**			

IBC2018, ASCE 7-16 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

Level	North/South		East/West		Area (sf)
	F_X (ASD)	F_{PX} (ASD)	F_X (ASD)	F_{PX} (ASD)	
Roof	x	x	x	x	x
3rd Floor	x	x	x	x	x
2nd Floor - Diaph A	0.8 kips	1.6 kips	0.8 kips	1.6 kips	352
	x	x	x	x	x

Seismic Loading Level: 2nd Floor - Diaph A — F_X = **0.8** **kips (ASD)**
Loading Direction: North/South — F_{PX} = **1.6** **kips (ASD)**
Total Level Area = 352 **ft²**
% of Total F_x = 50

Gridline: B & C.5

Span Type	Diaphragm Span		Length	Width	Area	Story Force	Diaphragm Force	Distributed Load	Diaphragm Shear	TC Couple
			(ft)	(ft)	(ft²)	(kips)	(kips)	(plf)	(plf)	(lbs)
Simple	B	C	16.0	22.0	352	0.39	0.80	50	**36**	**73**
						0.39	**0.80**			

Seismic Loading Level: 2nd Floor - Diaph A — F_X = **0.8** **kips (ASD)**
Loading Direction: East/West — F_{PX} = **1.6** **kips (ASD)**
Total Level Area = 352 **ft²**
% of Total F_x = 50

Gridline: 1 & 3

Span Type	Diaphragm Span		Length	Width	Area	Story Force	Diaphragm Force	Distributed Load	Diaphragm Shear	TC Couple
			(ft)	(ft)	(ft²)	(kips)	(kips)	(plf)	(plf)	(lbs)
Simple	1	3	22.0	16.0	352	0.39	0.80	36	**50**	**137**
						0.39	**0.80**			

IBC2018, ASCE 7-16 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

Level	North/South		East/West		Area (sf)
	F_X (ASD)	F_{PX} (ASD)	F_X (ASD)	F_{PX} (ASD)	
Roof	x	x	x	x	x
3rd Floor	x	x	x	x	x
2nd Floor - Diaph B	0.7 kips	1.4 kips	0.7 kips	1.4 kips	319
	x	x	x	x	x

Seismic Loading Level: 2nd Floor - Diaph B
Loading Direction: North/South

F_X = **0.7** **kips (ASD)**
F_{PX} = **1.4** **kips (ASD)**
Total Level Area = 319 **ft²**
% of Total F_x = 50

Gridline: C & C.5

Span Type	Diaphragm Span		Length	Width	Area	Story Force	Diaphragm Force	Distributed Load	Diaphragm Shear	TC Couple
			(ft)	(ft)	(ft²)	(kips)	(kips)	(plf)	(plf)	(lbs)
Simple	C	C.5	10.0	22.0	319	0.35	0.72	72	**33**	**41**
						0.35	**0.72**			

Seismic Loading Level: 2nd Floor - Diaph B
Loading Direction: East/West

F_X = **0.7** **kips (ASD)**
F_{PX} = **1.4** **kips (ASD)**
Total Level Area = 319 **ft²**
% of Total F_x = 50

Gridline: 1 & 3

Span Type	Diaphragm Span		Length	Width	Area	Story Force	Diaphragm Force	Distributed Load	Diaphragm Shear	TC Couple
			(ft)	(ft)	(ft²)	(kips)	(kips)	(plf)	(plf)	(lbs)
Simple	1	3	22.0	10.0	319	0.35	0.72	33	**72**	**199**
						0.35	**0.72**			

IBC2018, ASCE 7-16 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

Level	North/South		East/West		Area (sf)
	F_X (ASD)	F_{PX} (ASD)	F_X (ASD)	F_{PX} (ASD)	
Roof	X	x	X	x	x
3rd Floor	x	x	x	x	x
2nd Floor	0.7 kips	1.4 kips	0.7 kips	1.4 kips	319
	x	x	x	x	x

Seismic Loading Level: 2nd Floor
Loading Direction: North/South

F_X = **0.7** **kips (ASD)**
F_{PX} = **1.4** **kips (ASD)**
Total Level Area = 319 **ft²**
% of Total F_x = 50

Gridline: C.5 & D

Span Type	Diaphragm Span		Length (ft)	Width (ft)	Area (ft²)	Story Force (kips)	Diaphragm Force (kips)	Distributed Load (plf)	Diaphragm Shear (plf)	TC Couple (lbs)
Simple	C.5	D	10.0	22.0	319	0.35	0.72	145	**33**	**82**
						0.35	**0.72**			

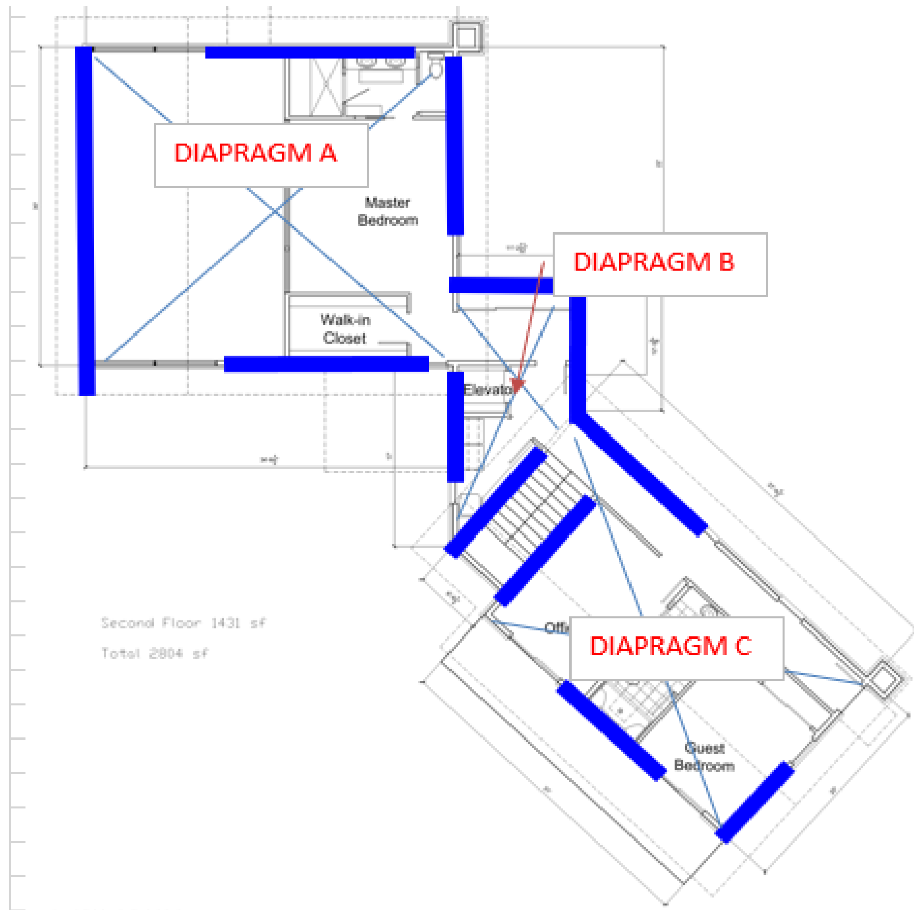
Seismic Loading Level: 2nd Floor
Loading Direction: East/West

F_X = **0.7** **kips (ASD)**
F_{PX} = **1.4** **kips (ASD)**
Total Level Area = 319 **ft²**
% of Total F_x = 50

Gridline: 1 & 3

Span Type	Diaphragm Span		Length (ft)	Width (ft)	Area (ft²)	Story Force (kips)	Diaphragm Force (kips)	Distributed Load (plf)	Diaphragm Shear (plf)	TC Couple (lbs)
Simple	1	3	22.0	10.0	319	0.35	0.72	33	**72**	**199**
						0.35	**0.72**			

ROOF DIAPHRAGM A DESIGN



Diaphragm Design:

N-S Direction

Roof

	Trib Area (ft2)	Force (lbs)	Diap Length	Diaph Shears
Line A	160	1.0	22.0	45.8
Line C	160	1.0	22.0	45.8
	319	2.0		

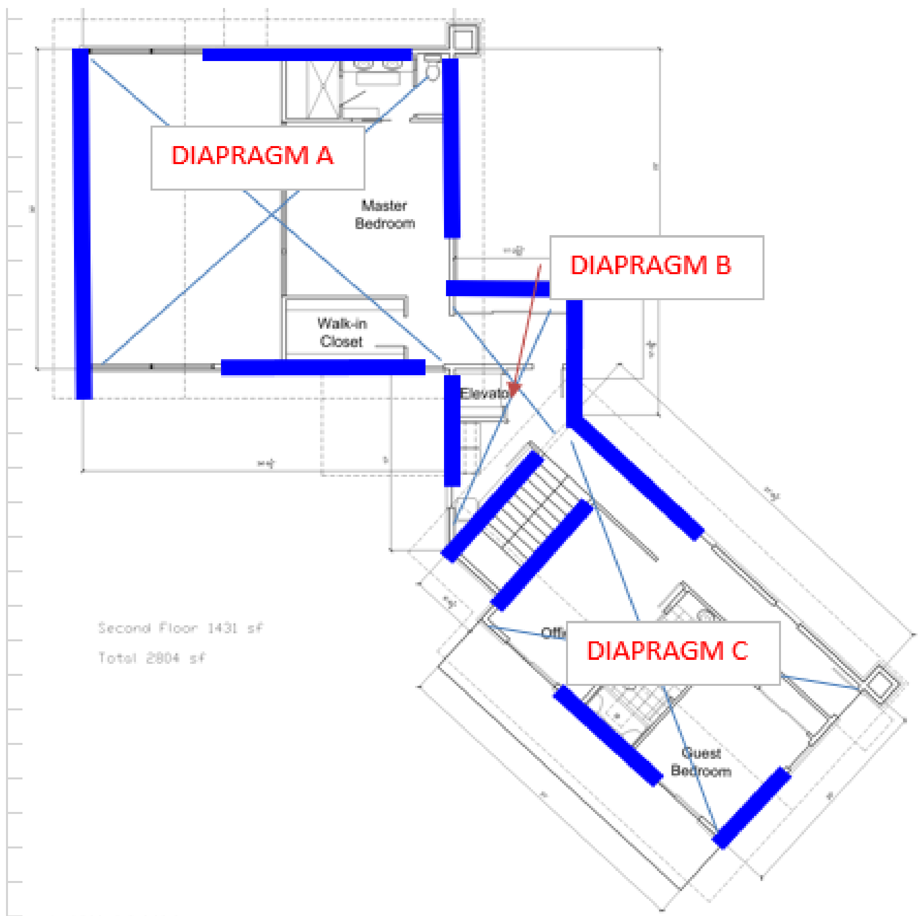
E-W Direction

Roof

	Trib Area (ft2)	Force (lbs)	Diaph Length	Diaph Shears
Line 1	159.5	1.007	10.0	100.732
Line 3	159.5	1.01	10.0	100.732
Σ	319	2.0		

Since diaphragm shears are small, provide 15/32" CDX plywood with 8d at 6" OC (BN) and 8d at 12" OC (field)

2ND FLOOR DIAPHRAGM A DESIGN



Diaphragm Design:

N-S Direction

2nd Floor

	Trib Area (ft2)	Force (lbs)	Diap Length	Diaph Shears
Line A	225	#N/A	30	#N/A
Line C	225	#N/A	30	#N/A
	450	#N/A		

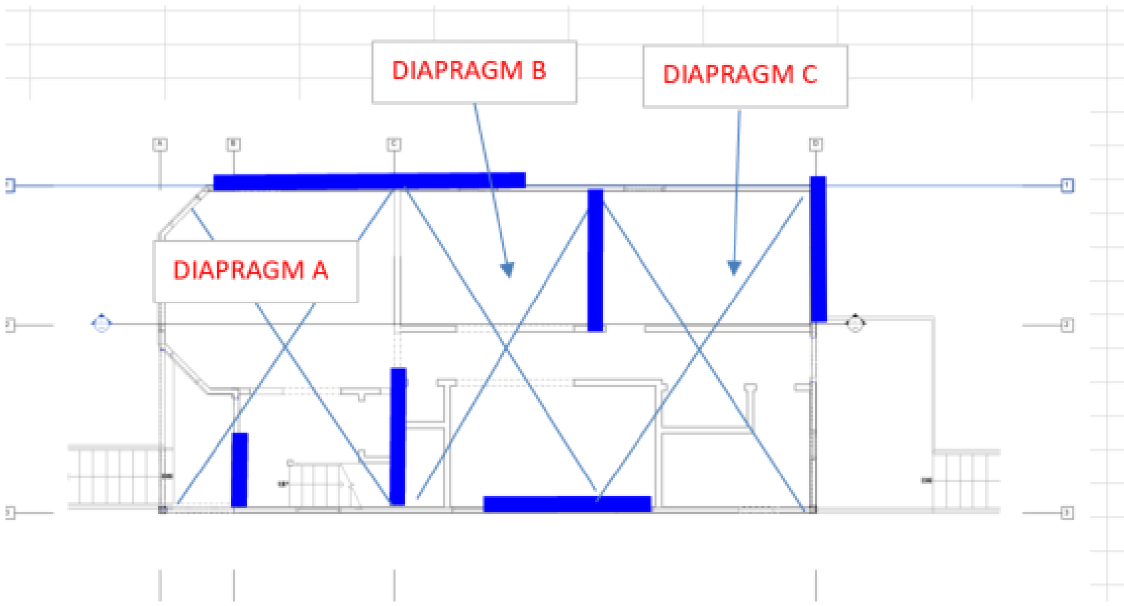
E-W Direction

2nd Floor

	Trib Area (ft2)	Force (lbs)	Diaph Length	Diaph Shears
Line 2	176	#N/A	16	#N/A
Line 3	176	#N/A	16	#N/A
Σ	352	#N/A		

Since diaphragm shears are small, provide 15/32" CDX plywood with 8d at 6" OC (BN) and 8d at 12" OC (field)

2nd FLOOR SHEARWALL LAYOUT:



Shearwall Design:

N-S Direction

2nd Floor

	Trib Area (ft2)	Force (lbs)
Line B		1081
Line C		2061
Line C.5		1959
Line D		980
	0	6080.8

E-W Direction

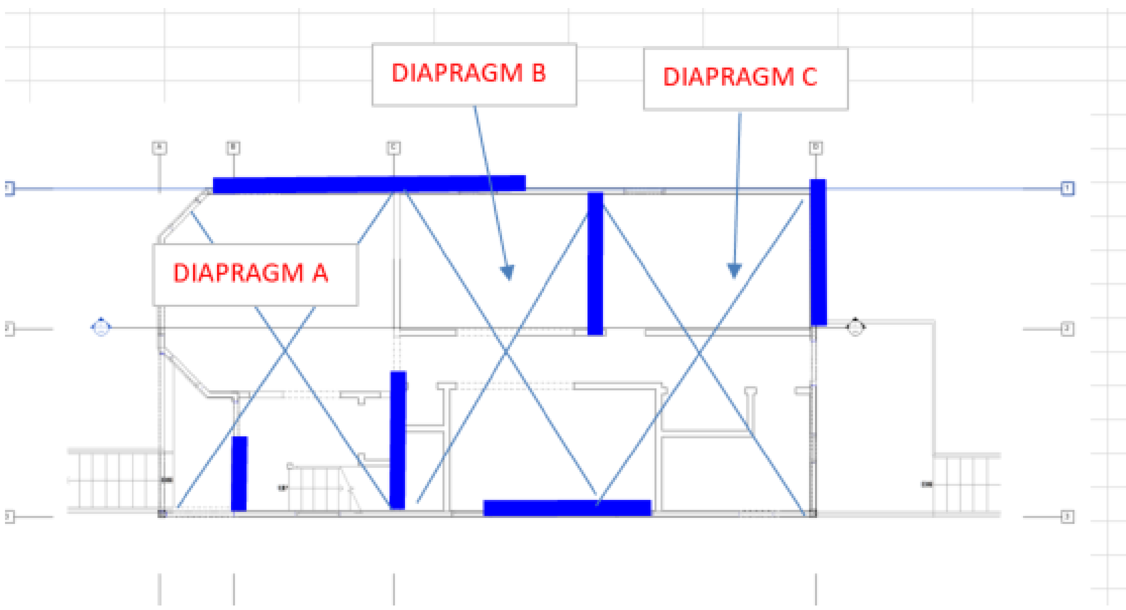
2nd Floor

	Trib Area (ft2)	Force (lbs)
Line 1		3040
Line 3		3040
Σ	0	6080.8

Notes:

The forces above are allowable stress design.
The forces above assumes rho = 1.3

1st FLOOR SHEARWALL LAYOUT:



Shearwall Design:

N-S Direction

2nd Floor

	Trib Area (ft2)	Force (lbs)
Line B		388.43
Line C		740.45
Line C.5		704.04
Line D		352.02
	0	2184.9

E-W Direction

2nd Floor

	Trib Area (ft2)	Force (lbs)
Line 1		1092.47
Line 3		1092.47
Σ	0	2184.9

Notes:

The forces above are allowable stress design.
The forces above assumes rho = 1.3

SHEARWALL DESIGN (GRIDLINE B)

DSA? **NO**

UPPER FLOOR:

DIRECTION: **North-South**

WALL LINE: **Line B** **SEISMIC** Sds= **1.2**

Total Wall Line Shear (ASD)= V (lb) = **1081**

Wall Lengths = L (ft) =	**5**								

Total Wall Length = L total (ft.) = **5**
Minimum Wall Length = L min. (ft.) = **5**
Wall Height = h (ft.) = **8**

Tributary Dead Load = DL (psf) = **12.8**
Tributary Width = TW (ft.) = **10**
Wall Dead Load = WDL (psf) = **8.0**
Total Dead Load at Wall = w_{DL} (plf) = 191.5

Uplift Force at Tie Down for "L" noted above =	1903								
Minimum Holdown for each "L" noted above =	HDU2								

Wall shear per lineal foot = v (plf) = V / L = **216**
Nail Size = **10d** Ply Grade = **CD / OSB**

Use SW	**CD / OSB w/ 10d@ 6"o.c.**	**(Capacity = 310 plf)**	**DCR = 0.70**

Max. Uplift Force at Tie Down = U (lb) ={(v*h*Lmin) - [(0.6-.14Sds)*wDL*Lmin2/2]} / (Lmin-1')= **1903**
Holdown Type = **HDU**

Use:	**HDU2**	**(Capacity = 3075 lb)**	**DCR = 0.62**

Max. Comp Force at End Post = C (lb) ={(v*h*Lmin) + [(1.0+.14Sds)*wDL*Lmin2/2]} / (Lmin)= **2289**

LOWER FLOOR:

SEISMIC Sds= **1.2**

Total Wall Line Shear (ASD)= V (lb) = **388**

Wall Lengths = L (ft) =	**5**								

Total Wall Length = L total (ft.) = **5**
Minimum Wall Length = L min. (ft.) = **5**
Wall Height = h (ft.) = **8.0**

Tributary Dead Load = DL (psf) = **12.8**
Tributary Width = TW (ft.) = **10**
Wall Dead Load = WDL (psf) = **8.0**
Total Dead Load at Wall = w_{DL} (plf) = 191.5

Uplift Force at Tie Down for "L" noted above =	4584								
Minimum Holdown for each "L" noted above =	HDU5								

Wall shear per lineal foot = v (plf) = V / L = **294** <-including shear from wall above
Nail Size = **10d** Ply Grade = **CD / OSB**

Use SW	**CD / OSB w/ 10d@ 6"o.c.**	**(Capacity = 310 plf)**	**DCR = 0.95**

Max. Uplift Force at Tie Down = U (lb) ={(v*h*Lmin) - [(0.6-.14Sds)*wDL*Lmin2/2]} / (Lmin-1')= **4584**
Holdown Type = **HDU**

Use:	**HDU5**	**(Capacity = 5645 lb)**	**DCR = 0.81**

Max. Comp Force at End Post = C (lb) ={(v*h*Lmin) + [(1.0+.14Sds)*wDL*Lmin2/2]} / (Lmin)= **5199**

SHEARWALL DESIGN (GRIDLINE C)

DSA? **NO**

UPPER FLOOR:

DIRECTION: **North-South**

WALL LINE: **Grid C** **SEISMIC** Sds= **1.2**

Total Wall Line Shear (ASD)= V (lb) = **2061**

Wall Lengths =	L (ft) =	9							

Total Wall Length = L total (ft.) = **9**

Minimum Wall Length = L min. (ft.) = **9**

Wall Height = h (ft.) = **8**

Tributary Dead Load = DL (psf) = **10.0**

Tributary Width = TW (ft.) = **10**

Wall Dead Load = WDL (psf) = **8.0**

Total Dead Load at Wall = w_{DL} (plf) = 164

Uplift Force at Tie Down for "L" noted above =	1702							
Minimum Holdown for each "L" noted above =	HDU2							

Wall shear per lineal foot = v (plf) = V / L = **229**

Nail Size = **10d** Ply Grade = **CD / OSB**

Use SW	**CD / OSB w/ 10d@ 6"o.c.**	**(Capacity = 310 plf)**	**DCR = 0.74**

Max. Uplift Force at Tie Down = U (lb) ={(v*h*Lmin) - [(0.6-.14Sds)*wDL*Lmin2/2]} / (Lmin-1')= **1702**

Holdown Type = **HDU**

Use:	**HDU2**	**(Capacity = 3075 lb)**	**DCR = 0.55**

Max. Comp Force at End Post = C (lb) ={(v*h*Lmin) + [(1.0+.14Sds)*wDL*Lmin2/2]} / (Lmin)= **2694**

LOWER FLOOR:

SEISMIC Sds= **1.2**

Total Wall Line Shear (ASD)= V (lb) = **740**

Wall Lengths =	L (ft) =	9							

Total Wall Length = L total (ft.) = **9**

Minimum Wall Length = L min. (ft.) = **9**

Wall Height = h (ft.) = **8.0**

Tributary Dead Load = DL (psf) = **10.0**

Tributary Width = TW (ft.) = **10**

Wall Dead Load = WDL (psf) = **8.0**

Total Dead Load at Wall = w_{DL} (plf) = 164

Uplift Force at Tie Down for "L" noted above =	4145							
Minimum Holdown for each "L" noted above =	HDU4							

Wall shear per lineal foot = v (plf) = V / L = **311** <-including shear from wall above

Nail Size = **10d** Ply Grade = **CD / OSB**

Use SW	**CD / OSB w/ 10d@ 4"o.c.**	**(Capacity = 460 plf)**	**DCR = 0.68**

Max. Uplift Force at Tie Down = U (lb) ={(v*h*Lmin) - [(0.6-.14Sds)*wDL*Lmin2/2]} / (Lmin-1')= **4145**

Holdown Type = **HDU**

Use:	**HDU4**	**(Capacity = 4565 lb)**	**DCR = 0.91**

Max. Comp Force at End Post = C (lb) ={(v*h*Lmin) + [(1.0+.14Sds)*wDL*Lmin2/2]} / (Lmin)= **6046**

SHEARWALL DESIGN (GRIDLINE C.5)

DSA? **NO**

UPPER FLOOR:

DIRECTION: **North-South**

WALL LINE: **Grid C.5** **SEISMIC** Sds= **1.2**

Total Wall Line Shear (ASD)= V (lb) = **2061**

Wall Lengths =	L (ft) =	9							

Total Wall Length = L total (ft.) = **9**
Minimum Wall Length = L min. (ft.) = **9**
Wall Height = h (ft.) = **8**

Tributary Dead Load = DL (psf) = **10.0**
Tributary Width = TW (ft.) = **10**
Wall Dead Load = WDL (psf) = **8.0**
Total Dead Load at Wall = w_{DL} (plf) = 164

Uplift Force at Tie Down for "L" noted above =	1702							
Minimum Holdown for each "L" noted above =	HDU2							

Wall shear per lineal foot = v (plf) = V / L = **229**
Nail Size = **10d** Ply Grade = **CD / OSB**

Use SW	**CD / OSB w/ 10d@ 6"o.c.**	**(Capacity = 310 plf)**	**DCR = 0.74**

Max. Uplift Force at Tie Down = U (lb) ={(v*h*Lmin) - [(0.6-.14Sds)*wDL*Lmin2/2]} / (Lmin-1')= **1702**
Holdown Type = **HDU**

Use:	**HDU2**	**(Capacity = 3075 lb)**	**DCR = 0.55**

Max. Comp Force at End Post = C (lb) ={(v*h*Lmin) + [(1.0+.14Sds)*wDL*Lmin2/2]} / (Lmin)= **2694**

LOWER FLOOR:

SEISMIC Sds= **1.2**

Total Wall Line Shear (ASD)= V (lb) = **704**

Wall Lengths =	L (ft) =	9							

Total Wall Length = L total (ft.) = **9**
Minimum Wall Length = L min. (ft.) = **9**
Wall Height = h (ft.) = **8.0**

Tributary Dead Load = DL (psf) = **10.0**
Tributary Width = TW (ft.) = **10**
Wall Dead Load = WDL (psf) = **8.0**
Total Dead Load at Wall = w_{DL} (plf) = 164

Uplift Force at Tie Down for "L" noted above =	4108							
Minimum Holdown for each "L" noted above =	HDU4							

Wall shear per lineal foot = v (plf) = V / L = **307** <-including shear from wall above
Nail Size = **10d** Ply Grade = **CD / OSB**

Use SW	**CD / OSB w/ 10d@ 6"o.c.**	**(Capacity = 310 plf)**	**DCR = 0.99**

Max. Uplift Force at Tie Down = U (lb) ={(v*h*Lmin) - [(0.6-.14Sds)*wDL*Lmin2/2]} / (Lmin-1')= **4108**
Holdown Type = **HDU**

Use:	**HDU4**	**(Capacity = 4565 lb)**	**DCR = 0.90**

Max. Comp Force at End Post = C (lb) ={(v*h*Lmin) + [(1.0+.14Sds)*wDL*Lmin2/2]} / (Lmin)= **6013**

Wood Shear Wall with an Opening Based on "2009 IBC Structural/Seismic Design Manual, Vol 2" pg.32 (Force Transfer Around Opening)

INPUT DATA:

DSA? NO

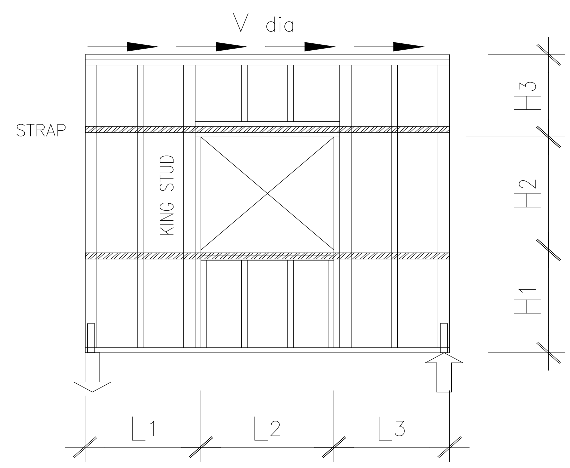
DIRECTION: E/W

WALL LINE: Line 5 (EQ) SEISMIC Sds= 1.20

Total Wall Shear (ASD)= V (lb) = 980

DIMENSIONS: L_1 = 6 ft , L_2 = 3 ft , L_3 = 2 ft

H_1 = 3 ft , H_2 = 3 ft , H_3 = 2 ft



Tributary Dead Load = DL (psf) = 12.755

Tributary Width = TW (ft.) = 10

Wall Dead Load = WDL (psf) = 10

Total Dead Load at Wall = wDL (plf) = 207.55

Nail Size = 10d

Ply Grade = CD / OSB

Holdown Type = HDU

DESIGN SUMMARY:

CALC MARK: 7

Use SW CD / OSB w/ 10d@ 6"o.c. **(Capacity = 310 plf) > 156 plf, demand**

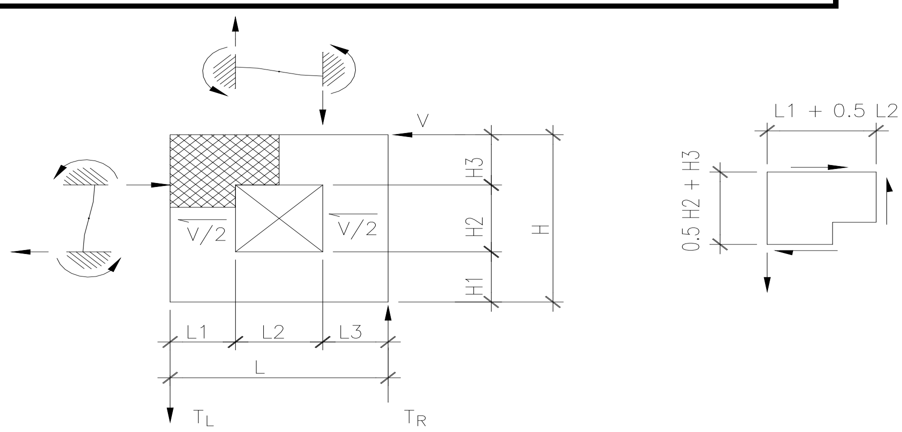
Max. Uplift Force at Tie Down = U (lb) ={(V*h) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1')= **241**

Use: Neglect (Capacity = 0 lb)

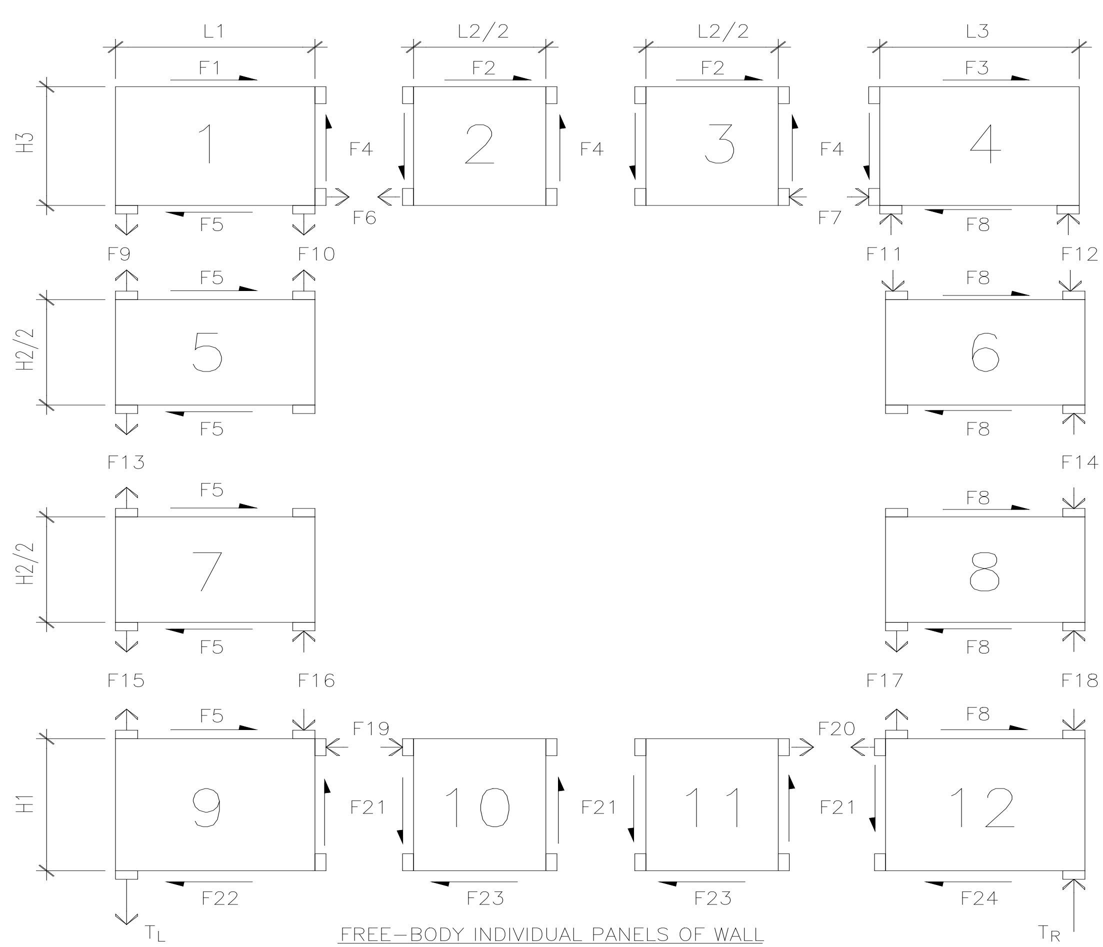
Max. Comp Force at End Post = C (lb) ={(V*h) + [(1.0+.14Sds)*wDL*L2/2]} / (L)= **2046**

MAX STRAP FORCE: F = **0.23** k

Use: A - CS16 (Capacity = 1705 lb)



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE-BODY INDIVIDUAL PANELS OF WALL

ANALYSIS:

DETERMINE FORCES & SHEAR STRESS OF FREE-BODY INDIVIDUAL PANELS OF WALL (See diagram)

V_{dia} = 89 plf

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)
1	6.00	2.00	72
2	1.50	2.00	156
3	1.50	2.00	156
4	2.00	2.00	39
5	6.00	1.50	111
6	2.00	1.50	156
7	6.00	1.50	111
8	2.00	1.50	156
9	6.00	3.00	78
10	1.50	3.00	134
11	1.50	3.00	134
12	2.00	3.00	56

NO.	FORCE (lb)	NO.	FORCE (lb)
F1	434	F13	312
F2	234	F14	312
F3	78	F15	479
F4	312	F16	167
F5	668	F17	234
F6	234	F18	546
F7	234	F19	200
F8	312	F20	200
F9	145	F21	401
F10	167	F22	468
F11	234	F23	200
F12	78	F24	111

DETERMINE REQUIRED CAPACITY

v_b = **156** plf

CHECK MAX SHEAR WALL DIMENSION RATIO

H / W = **1.5**

Wood Shear Wall with an Opening Based on "2009 IBC Structural/Seismic Design Manual, Vol 2" pg.32 (Force Transfer Around Opening)

INPUT DATA:

DSA? NO

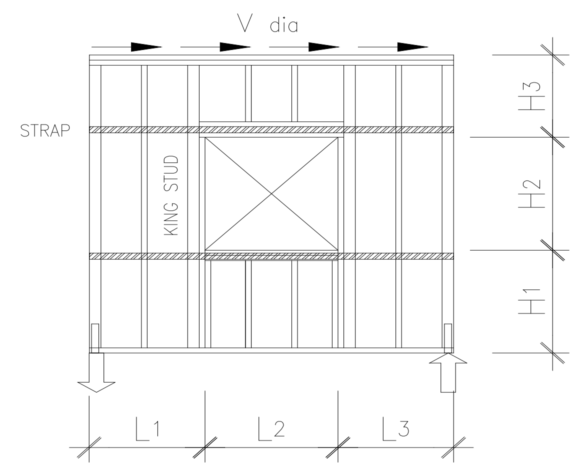
DIRECTION: E/W

WALL LINE: Line D (EQ) SEISMIC Sds= 1.20

Total Wall Shear (ASD)= V (lb) = 352

DIMENSIONS: L_1 = 6 ft , L_2 = 3 ft , L_3 = 2 ft

H_1 = 3 ft , H_2 = 3 ft , H_3 = 2 ft



Tributary Dead Load = DL (psf) = 12.755

Tributary Width = TW (ft.) = 10

Wall Dead Load = WDL (psf) = 10

Total Dead Load at Wall = wDL (plf) = 207.55

Nail Size = 10d

Ply Grade = CD / OSB

Holdown Type = HDU

DESIGN SUMMARY:

CALC MARK: 7

Use SW CD / OSB w/ 10d@ 6"o.c. (Capacity = 310 plf) > 56 plf, demand

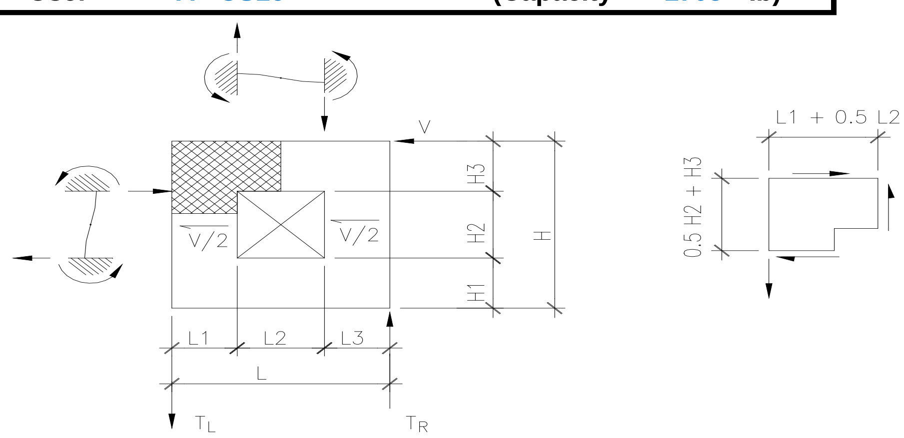
Max. Uplift Force at Tie Down = U (lb) =\{(V*h) - [(0.6-.14Sds)*wDL*L2/2]\} / (L-1')= 0

Use: No Uplift (Capacity = 0 lb)

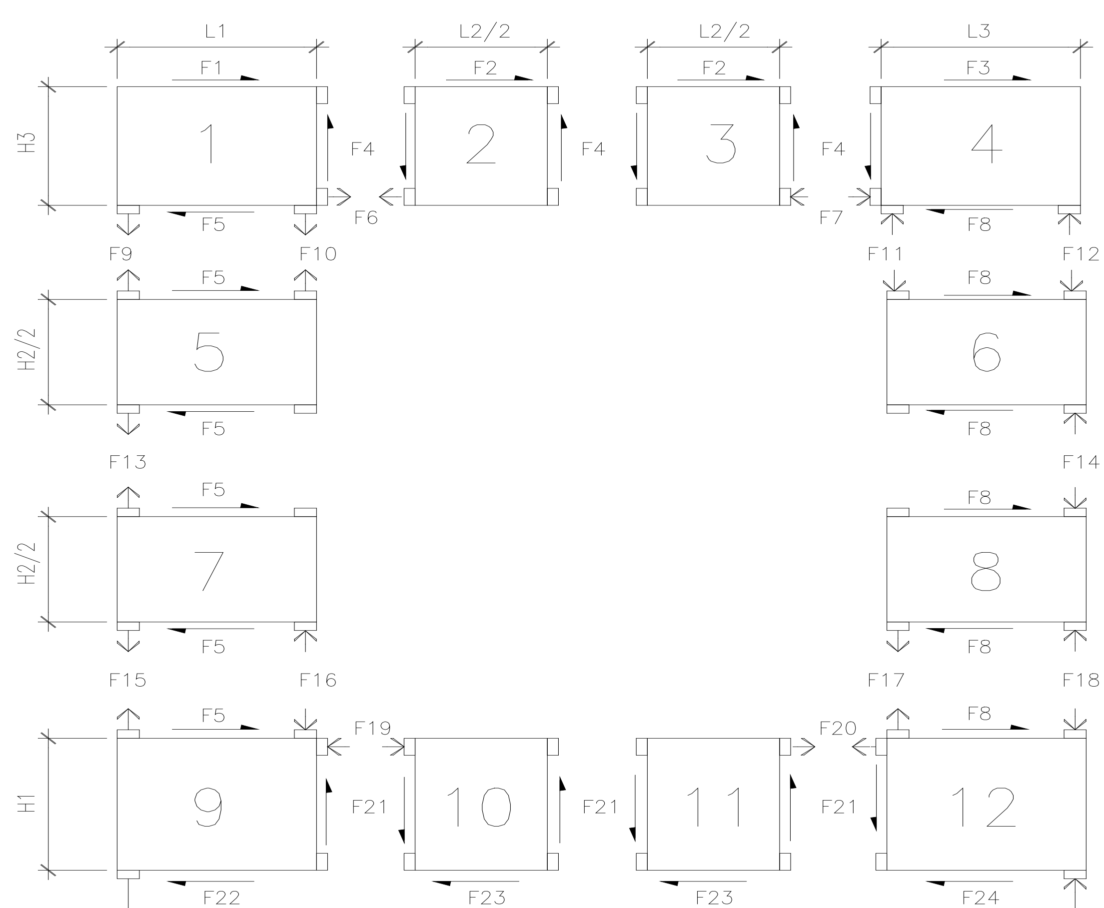
Max. Comp Force at End Post = C (lb) =\{(V*h) + [(1.0+.14Sds)*wDL*L2/2]\} / (L)= 1589

MAX STRAP FORCE: F = 0.08 k

Use: A - CS16 (Capacity = 1705 lb)



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE-BODY INDIVIDUAL PANELS OF WALL

ANALYSIS:

DETERMINE FORCES & SHEAR STRESS OF FREE-BODY INDIVIDUAL PANELS OF WALL (See diagram)

V_{dia} = 32 plf

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)
1	6.00	2.00	26
2	1.50	2.00	56
3	1.50	2.00	56
4	2.00	2.00	14
5	6.00	1.50	40
6	2.00	1.50	56
7	6.00	1.50	40
8	2.00	1.50	56
9	6.00	3.00	28
10	1.50	3.00	48
11	1.50	3.00	48
12	2.00	3.00	20

NO.	FORCE (lb)	NO.	FORCE (lb)
F1	156	F13	112
F2	84	F14	112
F3	28	F15	172
F4	112	F16	60
F5	240	F17	84
F6	84	F18	196
F7	84	F19	72
F8	112	F20	72
F9	52	F21	144
F10	60	F22	168
F11	84	F23	72
F12	28	F24	40

DETERMINE REQUIRED CAPACITY

v_b = 56 plf

CHECK MAX SHEAR WALL DIMENSION RATIO

H / W = 1.5

SHEARWALL DESIGN (GRIDLINE 1)

DSA? NO

UPPER FLOOR:

DIRECTION: N/S

WALL LINE: Line 1 SEISMIC Sds= 1.2

Total Wall Line Shear (ASD)= V (lb) = 3040

Wall Lengths = L (ft) =	10	4							

Total Wall Length = L total (ft.) = 14
Minimum Wall Length = L min. (ft.) = 4
Wall Height = h (ft.) = 8

Tributary Dead Load = DL (psf) = 12.8
Tributary Width = TW (ft.) = 10
Wall Dead Load = WDL (psf) = 8.0
Total Dead Load at Wall = w_{DL} (plf) = 191.5

Uplift Force at Tie Down for "L" noted above =	1471	2096							
Minimum Holdown for each "L" noted above =	HD2A	HD2A							

Wall shear per lineal foot = v (plf) = V / L = 217
Nail Size = 10d Ply Grade = CD / OSB

Use SW	**CD / OSB w/ 10d@ 6"o.c.**	**(Capacity = 310 plf)**	**DCR = 0.70**

Max. Uplift Force at Tie Down = U (lb) ={(v*h*Lmin) - [(0.6-.14Sds)*wDL*Lmin2/2]} / (Lmin-1')= 2096
Holdown Type = HD

Use:	**HD2A**	**(Capacity = 2775 lb)**	**DCR = 0.76**

Max. Comp Force at End Post = C (lb) ={(v*h*Lmin) + [(1.0+.14Sds)*wDL*Lmin2/2]} / (Lmin)= 2185

LOWER FLOOR:

SEISMIC Sds= 1.2

Total Wall Line Shear (ASD)= V (lb) = 1092

Wall Lengths = L (ft) =	10	4							

Total Wall Length = L total (ft.) = 14
Minimum Wall Length = L min. (ft.) = 4
Wall Height = h (ft.) = 8.0

Tributary Dead Load = DL (psf) = 12.8
Tributary Width = TW (ft.) = 10
Wall Dead Load = WDL (psf) = 8.0
Total Dead Load at Wall = w_{DL} (plf) = 191.5

Uplift Force at Tie Down for "L" noted above =	3635	5024							
Minimum Holdown for each "L" noted above =	HDU4	HDU5							

Wall shear per lineal foot = v (plf) = V / L = 295 <-including shear from wall above
Nail Size = 10d Ply Grade = CD / OSB

Use SW	**CD / OSB w/ 10d@ 6"o.c.**	**(Capacity = 310 plf)**	**DCR = 0.95**

Max. Uplift Force at Tie Down = U (lb) ={(v*h*Lmin) - [(0.6-.14Sds)*wDL*Lmin2/2]} / (Lmin-1')= 5024
Holdown Type = HDU

Use:	**HDU5**	**(Capacity = 5645 lb)**	**DCR = 0.89**

Max. Comp Force at End Post = C (lb) ={(v*h*Lmin) + [(1.0+.14Sds)*wDL*Lmin2/2]} / (Lmin)= 4994

SHEARWALL DESIGN (GRIDLINE 3)

DSA? **NO**

UPPER FLOOR:

DIRECTION: **E/W**

WALL LINE: **Grid 3** **SEISMIC** Sds= **1.2**

Total Wall Line Shear (ASD)= V (lb) = **3040**

Wall Lengths =	L (ft) =	7.5	6							

Total Wall Length = L total (ft.) = **13.5**

Minimum Wall Length = L min. (ft.) = **6**

Wall Height = h (ft.) = **8**

Tributary Dead Load = DL (psf) = **10.0**

Tributary Width = TW (ft.) = **10**

Wall Dead Load = WDL (psf) = **8.0**

Total Dead Load at Wall = w_{DL} (plf) = 164

Uplift Force at Tie Down for "L" noted above =	1772	1907							
Minimum Holdown for each "L" noted above =	HD2A	HD2A							

Wall shear per lineal foot = v (plf) = V / L = **225**

Nail Size = **10d** Ply Grade = **CD / OSB**

Use SW	**CD / OSB w/ 10d@ 6"o.c.**	**(Capacity = 310 plf)**	**DCR = 0.73**

Max. Uplift Force at Tie Down = U (lb) ={(v*h*Lmin) - [(0.6-.14Sds)*wDL*Lmin2/2]} / (Lmin-1')= **1907**

Holdown Type = **HD**

Use:	**HD2A**	**(Capacity = 2775 lb)**	**DCR = 0.69**

Max. Comp Force at End Post = C (lb) ={(v*h*Lmin) + [(1.0+.14Sds)*wDL*Lmin2/2]} / (Lmin)= **2376**

LOWER FLOOR:

SEISMIC Sds= **1.2**

Total Wall Line Shear (ASD)= V (lb) = **1092**

Wall Lengths =	L (ft) =	7.5	6							

Total Wall Length = L total (ft.) = **13.5**

Minimum Wall Length = L min. (ft.) = **6**

Wall Height = h (ft.) = **8.0**

Tributary Dead Load = DL (psf) = **10.0**

Tributary Width = TW (ft.) = **10**

Wall Dead Load = WDL (psf) = **8.0**

Total Dead Load at Wall = w_{DL} (plf) = 164

Uplift Force at Tie Down for "L" noted above =	4292	4591							
Minimum Holdown for each "L" noted above =	HDU4	HDU5							

Wall shear per lineal foot = v (plf) = V / L = **306** <-including shear from wall above

Nail Size = **10d** Ply Grade = **CD / OSB**

Use SW	**CD / OSB w/ 10d@ 6"o.c.**	**(Capacity = 310 plf)**	**DCR = 0.99**

Max. Uplift Force at Tie Down = U (lb) ={(v*h*Lmin) - [(0.6-.14Sds)*wDL*Lmin2/2]} / (Lmin-1')= **4591**

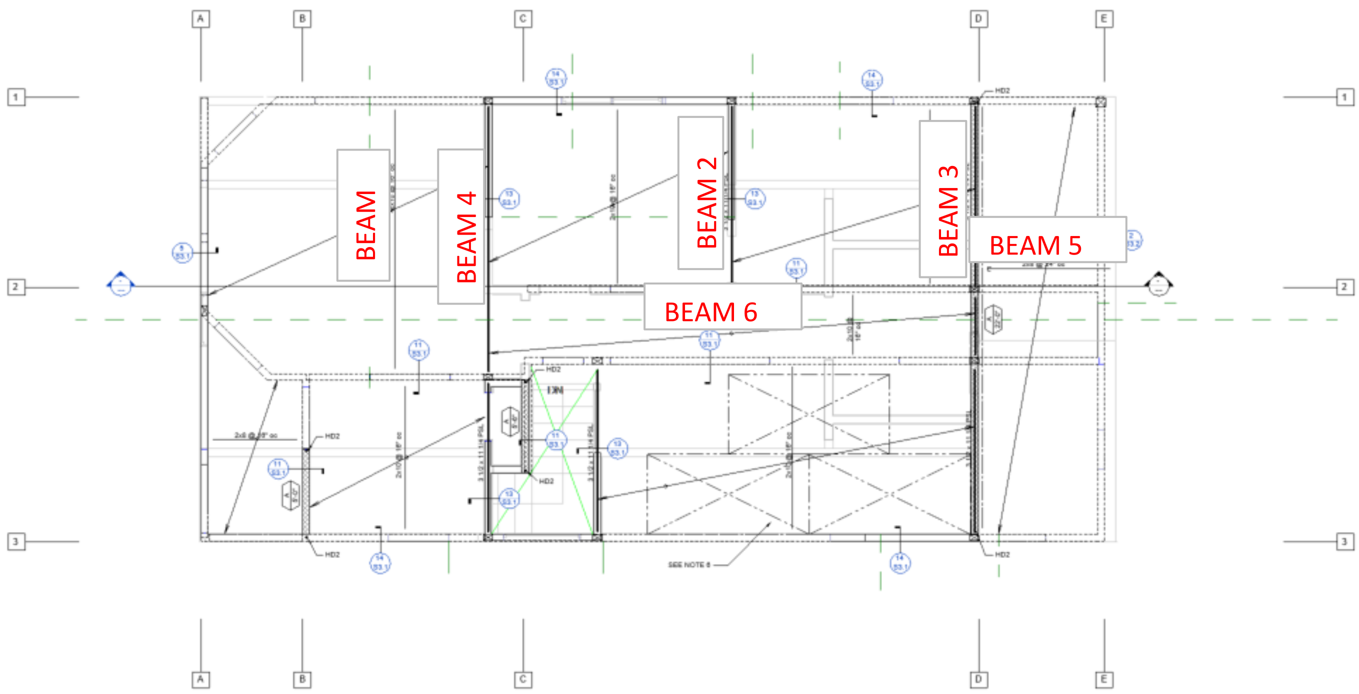
Holdown Type = **HDU**

Use:	**HDU5**	**(Capacity = 5645 lb)**	**DCR = 0.81**

Max. Comp Force at End Post = C (lb) ={(v*h*Lmin) + [(1.0+.14Sds)*wDL*Lmin2/2]} / (Lmin)= **5400**

3rd FLOOR FRAMING DESIGN

3rd FLOOR FRAMING LAYOUT:

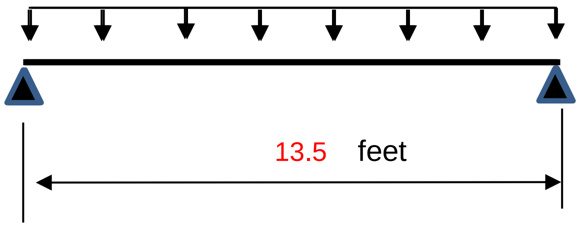


For Design:

1. Assume Loading

DL =	10.7	psf (girder)
DL =	8	psf (joist)
LL =	40	psf

Beam 1:

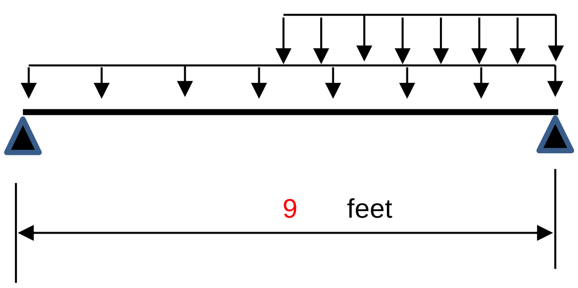


Floor Loading:

Dead Load =	8	psf
Live Load =	40	psf
Trib Area =	1.33	feet
Wdl =	10.906	plf
Wll =	53.2	plf

Therefore, provide 2x10@16" oc
See Enercalc next page

Beam 2:



Floor Loading:

Dead Load =	11	psf
Live Load =	40	psf
Trib Area =	2	feet
Wdl =	21.4	plf
Wll =	80	plf

Therefore, provide 3 1/2 x 14 PSL

See Enercal next page

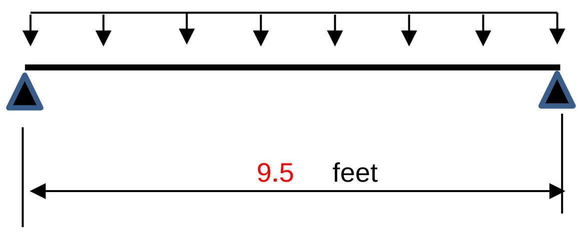
Roof Loading:

Dead Load =	13	psf
Live Load =	20	psf
Trib Area =	6	feet
Wdl =	76.527	plf
Wll =	120	plf

Wall Weight:

Dead Load =	10	psf
Height =	8	feet
wwall =	80	plf

Beam 3:



Therefore, provide 3 1/2 x 9 1/4 PSL

See Enercalc next page

Floor Loading:

Dead Load =	8	psf
Live Load =	40	psf
Trib Area =	2	feet
Wdl =	16	plf
Wll =	80	plf

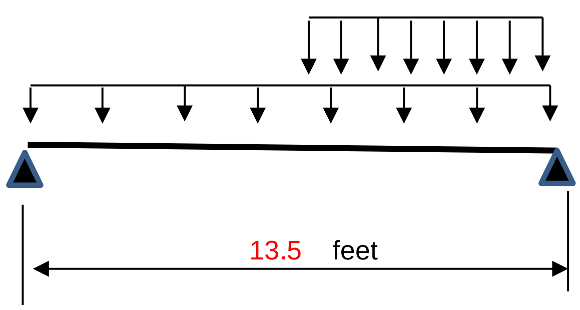
Roof Loading:

Dead Load =	13	psf
Live Load =	20	psf
Trib Area =	5	feet
Wdl =	63.773	plf
Wll =	100	plf

Wall Weight:

Dead Load =	10	psf
Height =	8	feet
wwall =	80	plf

Beam 4:



Therefore, provide 3 1/2 x 11 7/8 PSL
See Enercalc next page

Floor Loading:

Dead Load =	11	psf
Live Load =	40	psf
Trib Area =	2	feet
Wdl =	21	plf
Wll =	80	plf

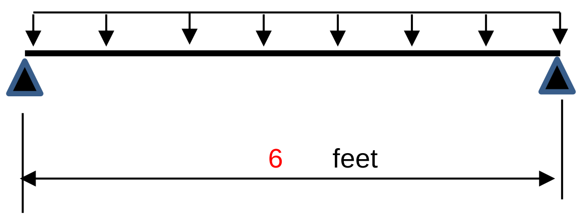
Roof Loading:

Dead Load =	13	psf
Live Load =	20	psf
Trib Area =	5	feet
Wdl =	64	plf
Wll =	100	plf

Wall Weight:

Dead Load =	10	psf
Height =	8	feet
wwall =	80	plf

Beam 5:



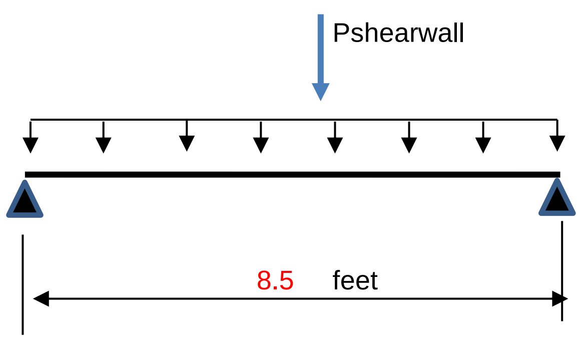
Roof Loading:

Dead Load =	13	psf
Live Load =	20	psf
Trib Area =	2	feet
Wdl =	26	plf
Wll =	40	plf

Therefore, provide 2x8 @ 24" oc

See Enercalc next page

Beam 6:



*For design

1. Special seismic load combination per ASCE07-16 section 12.4.3.2 applies
   Omega = 2.5

Therefore, provide 3 1/2 x 11 1/4 PSL

See Enercalc next page

Floor Loading:

Dead Load =	11	psf
Live Load =	40	psf
Trib Area =	8	feet
Wdl =	86	plf
Wll =	320	plf

Seismic Point Load:

Psesimic = 1588 lbs
*per Enercalc beam design

Wall Weight:

Dead Load =	10	psf
Height =	8	feet
Trib Length =	5	feet
Pwall =	85	lbs

Roof Loading:

Dead Load =	11	psf
Live Load =	20	psf
Trib Area =	6	feet
Trib Length =	10	feet
Pdl =	642	lbs

3RD FLOOR BEAM - B1

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

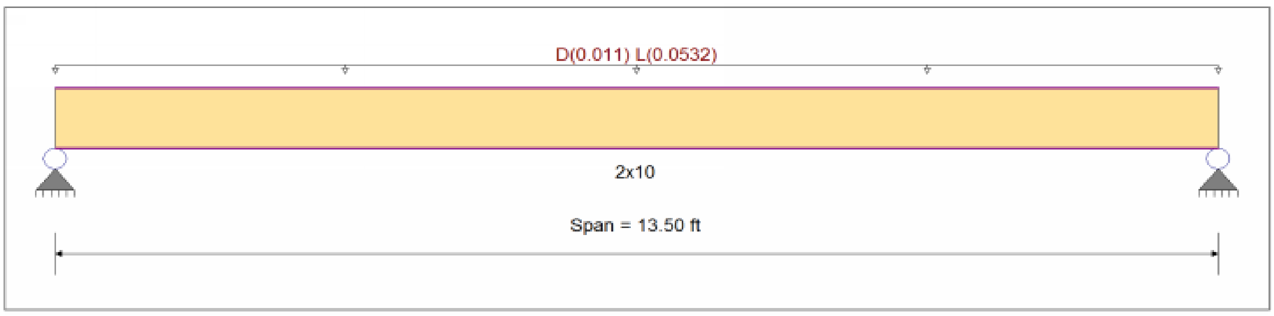
DESCRIPTIO 1214 30th St - 3rd Floor Beam 1

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Metho	Allowable Stress Design	Fb +	1,000.0 psi	*E : Modulus of Elasti*	
Load Combinati	IBC 2018	Fb -	1,000.0 psi	Ebend- xx	1,700.0 ksi
		Fc - Prll	1,500.0 psi	Eminbend - x	620.0 ksi
Wood Species	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade	No.1	Fv	180.0 psi		
		Ft	675.0 psi	Density	31.210 pcf
Beam Bracing	Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increa:	



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0110, L = 0.05320 , Tributary Width = 1.0 ft, (Floor Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.590** : 1	Maximum Shear Stress Ratio	=	**0.243** : 1
Section used for this span		**2x10**	Section used for this span		**2x10**
	=	858.92 psi		=	43.67 psi
	=	1,454.75 psi		=	180.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	6.750 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.238 in	Ratio =	681 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.300 in	Ratio =	539 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.137	0.056	0.90	1.100	1.00	1.15	1.00	1.00	1.00	0.32	179.01	1309.28	0.08	9.10	162.00
Length = 0.04927 ft	1	0.002	0.056	0.90	1.100	1.00	1.15	1.00	1.00	1.00	0.00	2.60	1309.28	0.08	9.10	162.00
+D+L+H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.590	0.243	1.00	1.100	1.00	1.15	1.00	1.00	1.00	1.53	858.92	1454.75	0.40	43.67	180.00
Length = 0.04927 ft	1	0.009	0.243	1.00	1.100	1.00	1.15	1.00	1.00	1.00	0.02	12.49	1454.75	0.40	43.67	180.00
+D+Lr+H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.098	0.040	1.25	1.100	1.00	1.15	1.00	1.00	1.00	0.32	179.01	1818.44	0.08	9.10	225.00
Length = 0.04927 ft	1	0.001	0.040	1.25	1.100	1.00	1.15	1.00	1.00	1.00	0.00	2.60	1818.44	0.08	9.10	225.00
+D+S+H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.107	0.044	1.15	1.100	1.00	1.15	1.00	1.00	1.00	0.32	179.01	1672.96	0.08	9.10	207.00
Length = 0.04927 ft	1	0.002	0.044	1.15	1.100	1.00	1.15	1.00	1.00	1.00	0.00	2.60	1672.96	0.08	9.10	207.00
+D+0.750Lr+0.750L+H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00

3RD FLOOR BEAM - B1

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St - 3rd Floor Beam 1

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 13.451 ft	1	0.379	0.156	1.25	1.100	1.00	1.15	1.00	1.00	1.00	1.23	688.94	1818.44	0.32	35.03	225.00
Length = 0.04927 ft	1	0.006	0.156	1.25	1.100	1.00	1.15	1.00	1.00	1.00	0.02	10.02	1818.44	0.32	35.03	225.00
+D+0.750L+0.750S+H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.412	0.169	1.15	1.100	1.00	1.15	1.00	1.00	1.00	1.23	688.94	1672.96	0.32	35.03	207.00
Length = 0.04927 ft	1	0.006	0.169	1.15	1.100	1.00	1.15	1.00	1.00	1.00	0.02	10.02	1672.96	0.32	35.03	207.00
+D+0.60W+H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.077	0.032	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.32	179.01	2327.60	0.08	9.10	288.00
Length = 0.04927 ft	1	0.001	0.032	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.00	2.60	2327.60	0.08	9.10	288.00
+D+0.70E+H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.077	0.032	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.32	179.01	2327.60	0.08	9.10	288.00
Length = 0.04927 ft	1	0.001	0.032	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.00	2.60	2327.60	0.08	9.10	288.00
+D+0.750Lr+0.750L+0.450W-					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.296	0.122	1.60	1.100	1.00	1.15	1.00	1.00	1.00	1.23	688.94	2327.60	0.32	35.03	288.00
Length = 0.04927 ft	1	0.004	0.122	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.02	10.02	2327.60	0.32	35.03	288.00
+D+0.750L+0.750S+0.450W+					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.296	0.122	1.60	1.100	1.00	1.15	1.00	1.00	1.00	1.23	688.94	2327.60	0.32	35.03	288.00
Length = 0.04927 ft	1	0.004	0.122	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.02	10.02	2327.60	0.32	35.03	288.00
+D+0.750L+0.750S+0.5250E-					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.296	0.122	1.60	1.100	1.00	1.15	1.00	1.00	1.00	1.23	688.94	2327.60	0.32	35.03	288.00
Length = 0.04927 ft	1	0.004	0.122	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.02	10.02	2327.60	0.32	35.03	288.00
+0.60D+0.60W+0.60H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.046	0.019	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.19	107.41	2327.60	0.05	5.46	288.00
Length = 0.04927 ft	1	0.001	0.019	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.00	1.56	2327.60	0.05	5.46	288.00
+0.60D+0.70E+0.60H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.046	0.019	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.19	107.41	2327.60	0.05	5.46	288.00
Length = 0.04927 ft	1	0.001	0.019	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.00	1.56	2327.60	0.05	5.46	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.3004	6.799		0.0000	0.000

Vertical Reactions

Support notation : Far left is # Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.454	0.454
Overall MINimum	0.359	0.359
+D+H	0.095	0.095
+D+L+H	0.454	0.454
+D+Lr+H	0.095	0.095
+D+S+H	0.095	0.095
+D+0.750Lr+0.750L+H	0.364	0.364
+D+0.750L+0.750S+H	0.364	0.364
+D+0.60W+H	0.095	0.095
+D+0.70E+H	0.095	0.095
+D+0.750Lr+0.750L+0.450W+H	0.364	0.364
+D+0.750L+0.750S+0.450W+H	0.364	0.364
+D+0.750L+0.750S+0.5250E+H	0.364	0.364
+0.60D+0.60W+0.60H	0.057	0.057
+0.60D+0.70E+0.60H	0.057	0.057
D Only	0.095	0.095
Lr Only		
L Only	0.359	0.359
S Only		
W Only		
E Only		
H Only		

3RD FLOOR BEAM - B2

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St - 3rd Floor Beam 2

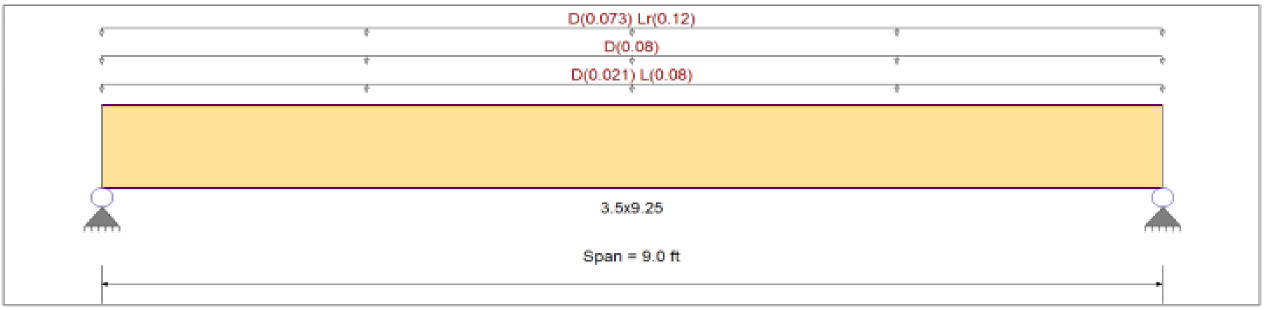
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Metho	Allowable Stress Design	Fb +	2900 psi	*E : Modulus of Elasti*	
Load Combinati	IBC 2018	Fb -	2900 psi	Ebend- xx	2000 ksi
		Fc - Prll	2900 psi	Eminbend - x	1016.535 ksi
Wood Species	iLevel Truss Joist	Fc - Perp	750 psi		
Wood Grade	Parallam PSL 2.0E	Fv	290 psi		
		Ft	2025 psi	Density	45.07 pcf

Beam Bracing Beam is Fully Braced against lateral-torsional buckling



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0210, L = 0.080 , Tributary Width = 1.0 ft, (Floor Load)
Uniform Load : D = 0.080 , Tributary Width = 1.0 ft, (Wall Loading)
Uniform Load : D = 0.0730, Lr = 0.120 , Tributary Width = 1.0 ft, (Roof Loading)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.224**: 1	Maximum Shear Stress Ratio	=	**0.160** : 1
Section used for this span		**3.5x9.25**	Section used for this span		**3.5x9.25**
	=	813.38 psi		=	57.97 psi
	=	3,625.00 psi		=	362.50 psi
Load Combination		+D+0.750Lr+0.750L+H	Load Combination		+D+0.750Lr+0.750L+H
Location of maximum on span	=	4.500 ft	Location of maximum on span	=	8.245 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.039 in	Ratio =	2798 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.107 in	Ratio =	1004 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.172	0.122	0.90	1.000	1.00	1.00	1.00	1.00	1.00	1.86	448.24	2610.00	0.69	31.95	261.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.222	0.158	1.00	1.000	1.00	1.00	1.00	1.00	1.00	2.67	642.98	2900.00	0.99	45.82	290.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.204	0.146	1.25	1.000	1.00	1.00	1.00	1.00	1.00	3.08	740.35	3625.00	1.14	52.76	362.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.134	0.096	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.86	448.24	3335.00	0.69	31.95	333.50
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.224	0.160	1.25	1.000	1.00	1.00	1.00	1.00	1.00	3.38	813.38	3625.00	1.25	57.97	362.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

3RD FLOOR BEAM - B2

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St - 3rd Floor Beam 2

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 9.0 ft	1	0.178	0.127	1.15	1.000	1.00	1.00	1.00	1.00	1.00	2.47	594.30	3335.00	0.91	42.35	333.50
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.097	0.069	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.86	448.24	4640.00	0.69	31.95	464.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.097	0.069	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.86	448.24	4640.00	0.69	31.95	464.00
+D+0.750Lr+0.750L+0.450W·					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.175	0.125	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.38	813.38	4640.00	1.25	57.97	464.00
+D+0.750L+0.750S+0.450W+					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.128	0.091	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.47	594.30	4640.00	0.91	42.35	464.00
+D+0.750L+0.750S+0.5250E·					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.128	0.091	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.47	594.30	4640.00	0.91	42.35	464.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.058	0.041	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.12	268.94	4640.00	0.41	19.17	464.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.058	0.041	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.12	268.94	4640.00	0.41	19.17	464.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750Lr+0.750L+0.450W+H	1	0.1075	4.533		0.0000	0.000

Vertical Reactions

Support notation : Far left is # Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.504	1.504
Overall MINimum	0.360	0.360
+D+H	0.829	0.829
+D+L+H	1.189	1.189
+D+Lr+H	1.369	1.369
+D+S+H	0.829	0.829
+D+0.750Lr+0.750L+H	1.504	1.504
+D+0.750L+0.750S+H	1.099	1.099
+D+0.60W+H	0.829	0.829
+D+0.70E+H	0.829	0.829
+D+0.750Lr+0.750L+0.450W+H	1.504	1.504
+D+0.750L+0.750S+0.450W+H	1.099	1.099
+D+0.750L+0.750S+0.5250E+H	1.099	1.099
+0.60D+0.60W+0.60H	0.497	0.497
+0.60D+0.70E+0.60H	0.497	0.497
D Only	0.829	0.829
Lr Only	0.540	0.540
L Only	0.360	0.360
S Only		
W Only		
E Only		
H Only		

3RD FLOOR BEAM - B3

Wood Beam

Lic. # : KW-06012032 | DRE Structural Design

DESCRIPTIO 1214 30th St - 3rd Floor Beam 3

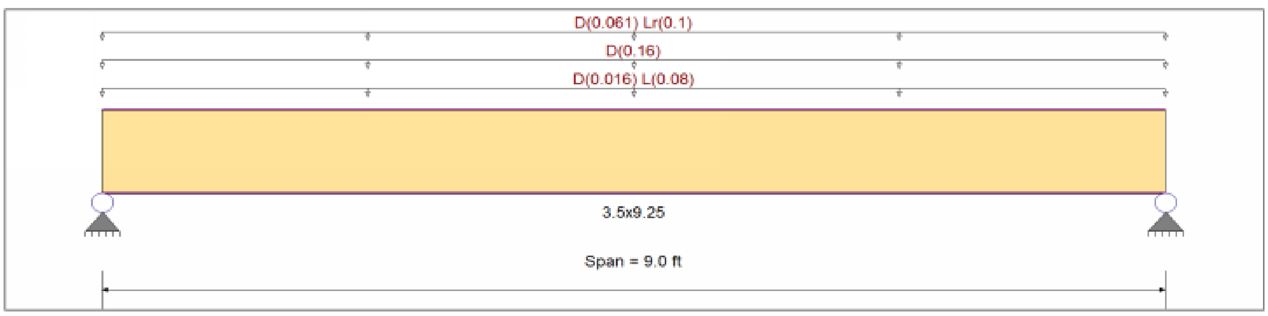
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Metho	Allowable Stress Design	Fb +	2,900.0 psi	*E : Modulus of Elasti*	
Load Combinati	IBC 2018	Fb -	2,900.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,900.0 psi	Eminbend - x	1,016.54 ksi
Wood Species	iLevel Truss Joist	Fc - Perp	750.0 psi		
Wood Grade	Parallam PSL 2.0E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.070 pcf

Beam Bracing Beam is Fully Braced against lateral-torsional buckling



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0160, L = 0.080 , Tributary Width = 1.0 ft, (Floor Load)
Uniform Load : D = 0.160 , Tributary Width = 1.0 ft, (Wall Loading)
Uniform Load : D = 0.0610, Lr = 0.10 , Tributary Width = 1.0 ft, (Roof Loading)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.275**: 1	Maximum Shear Stress Ratio	=	**0.196** : 1
Section used for this span		**3.5x9.25**	Section used for this span		**3.5x9.25**
	=	796.34 psi		=	56.75 psi
	=	2,900.00 psi		=	290.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	4.500 ft	Location of maximum on span	=	8.245 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.032 in	Ratio =	3358 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.123 in	Ratio =	878 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.230	0.164	0.90	1.000	1.00	1.00	1.00	1.00	1.00	2.50	601.60	2610.00	0.93	42.88	261.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.275	0.196	1.00	1.000	1.00	1.00	1.00	1.00	1.00	3.31	796.34	2900.00	1.22	56.75	290.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.233	0.166	1.25	1.000	1.00	1.00	1.00	1.00	1.00	3.51	845.03	3625.00	1.30	60.22	362.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.180	0.129	1.15	1.000	1.00	1.00	1.00	1.00	1.00	2.50	601.60	3335.00	0.93	42.88	333.50
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.257	0.183	1.25	1.000	1.00	1.00	1.00	1.00	1.00	3.87	930.23	3625.00	1.43	66.30	362.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

3RD FLOOR BEAM - B3

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St - 3rd Floor Beam 3

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 9.0 ft	1	0.224	0.160	1.15	1.000	1.00	1.00	1.00	1.00	1.00	3.11	747.66	3335.00	1.15	53.28	333.50
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.130	0.092	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.50	601.60	4640.00	0.93	42.88	464.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.130	0.092	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.50	601.60	4640.00	0.93	42.88	464.00
+D+0.750Lr+0.750L+0.450W-					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.200	0.143	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.87	930.23	4640.00	1.43	66.30	464.00
+D+0.750L+0.750S+0.450W+					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.161	0.115	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.11	747.66	4640.00	1.15	53.28	464.00
+D+0.750L+0.750S+0.5250E-					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.161	0.115	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.11	747.66	4640.00	1.15	53.28	464.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.078	0.055	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.50	360.96	4640.00	0.56	25.73	464.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.078	0.055	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.50	360.96	4640.00	0.56	25.73	464.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750Lr+0.750L+0.450W+H	1	0.1229	4.533		0.0000	0.000

Vertical Reactions

Support notation : Far left is #' Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.720	1.720
Overall MINimum	0.360	0.360
+D+H	1.112	1.112
+D+L+H	1.472	1.472
+D+Lr+H	1.562	1.562
+D+S+H	1.112	1.112
+D+0.750Lr+0.750L+H	1.720	1.720
+D+0.750L+0.750S+H	1.382	1.382
+D+0.60W+H	1.112	1.112
+D+0.70E+H	1.112	1.112
+D+0.750Lr+0.750L+0.450W+H	1.720	1.720
+D+0.750L+0.750S+0.450W+H	1.382	1.382
+D+0.750L+0.750S+0.5250E+H	1.382	1.382
+0.60D+0.60W+0.60H	0.667	0.667
+0.60D+0.70E+0.60H	0.667	0.667
D Only	1.112	1.112
Lr Only	0.450	0.450
L Only	0.360	0.360
S Only		
W Only		
E Only		
H Only		

3RD FLOOR BEAM - B4

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

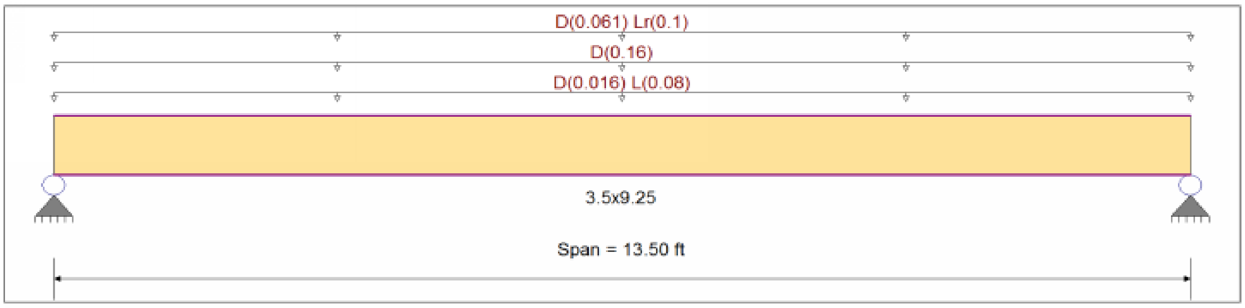
DESCRIPTIO 1214 30th St - 3rd Floor Beam 4

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Metho	Allowable Stress Design	Fb +	2,900.0 psi	*E : Modulus of Elasti*	
Load Combinati	IBC 2018	Fb -	2,900.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,900.0 psi	Eminbend - x	1,016.54 ksi
Wood Species	iLevel Truss Joist	Fc - Perp	750.0 psi		
Wood Grade	Parallam PSL 2.0E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.070 pcf
Beam Bracing	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0160, L = 0.080 , Tributary Width = 1.0 ft, (Floor Load)
Uniform Load : D = 0.160 , Tributary Width = 1.0 ft, (Wall Loading)
Uniform Load : D = 0.0610, Lr = 0.10 , Tributary Width = 1.0 ft, (Roof Loading)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.618** : 1	Maximum Shear Stress Ratio	=	**0.314** : 1
Section used for this span		**3.5x9.25**	Section used for this span		**3.5x9.25**
	=	1,791.77 psi		=	91.11 psi
	=	2,900.00 psi		=	290.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	6.750 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.163 in	Ratio =	994 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.622 in	Ratio =	260 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.519	0.264	0.90	1.000	1.00	1.00	1.00	1.00	1.00	5.63	1,353.60	2610.00	1.49	68.83	261.00
Length = 0.04927 ft	1	0.008	0.264	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.08	19.69	2610.00	1.49	68.83	261.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.618	0.314	1.00	1.000	1.00	1.00	1.00	1.00	1.00	7.45	1,791.77	2900.00	1.97	91.11	290.00
Length = 0.04927 ft	1	0.009	0.314	1.00	1.000	1.00	1.00	1.00	1.00	1.00	0.11	26.06	2900.00	1.97	91.11	290.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.525	0.267	1.25	1.000	1.00	1.00	1.00	1.00	1.00	7.91	1,901.32	3625.00	2.09	96.68	362.50
Length = 0.04927 ft	1	0.008	0.267	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.12	27.66	3625.00	2.09	96.68	362.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.406	0.206	1.15	1.000	1.00	1.00	1.00	1.00	1.00	5.63	1,353.60	3335.00	1.49	68.83	333.50

3RD FLOOR BEAM - B4

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St - 3rd Floor Beam 4

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 0.04927 ft	1	0.006	0.206	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.08	19.69	3335.00	1.49	68.83	333.50
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.577	0.294	1.25	1.000	1.00	1.00	1.00	1.00	1.00	8.71	2,093.02	3625.00	2.30	106.42	362.50
Length = 0.04927 ft	1	0.008	0.294	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.13	30.44	3625.00	2.30	106.42	362.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.504	0.256	1.15	1.000	1.00	1.00	1.00	1.00	1.00	7.00	1,682.23	3335.00	1.85	85.54	333.50
Length = 0.04927 ft	1	0.007	0.256	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.10	24.47	3335.00	1.85	85.54	333.50
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.292	0.148	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.63	1,353.60	4640.00	1.49	68.83	464.00
Length = 0.04927 ft	1	0.004	0.148	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.08	19.69	4640.00	1.49	68.83	464.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.292	0.148	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.63	1,353.60	4640.00	1.49	68.83	464.00
Length = 0.04927 ft	1	0.004	0.148	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.08	19.69	4640.00	1.49	68.83	464.00
+D+0.750Lr+0.750L+0.450W·					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.451	0.229	1.60	1.000	1.00	1.00	1.00	1.00	1.00	8.71	2,093.02	4640.00	2.30	106.42	464.00
Length = 0.04927 ft	1	0.007	0.229	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.13	30.44	4640.00	2.30	106.42	464.00
+D+0.750L+0.750S+0.450W+					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.363	0.184	1.60	1.000	1.00	1.00	1.00	1.00	1.00	7.00	1,682.23	4640.00	1.85	85.54	464.00
Length = 0.04927 ft	1	0.005	0.184	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.10	24.47	4640.00	1.85	85.54	464.00
+D+0.750L+0.750S+0.5250E·					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.363	0.184	1.60	1.000	1.00	1.00	1.00	1.00	1.00	7.00	1,682.23	4640.00	1.85	85.54	464.00
Length = 0.04927 ft	1	0.005	0.184	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.10	24.47	4640.00	1.85	85.54	464.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.175	0.089	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.38	812.16	4640.00	0.89	41.30	464.00
Length = 0.04927 ft	1	0.003	0.089	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.05	11.81	4640.00	0.89	41.30	464.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.175	0.089	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.38	812.16	4640.00	0.89	41.30	464.00
Length = 0.04927 ft	1	0.003	0.089	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.05	11.81	4640.00	0.89	41.30	464.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750Lr+0.750L+0.450W+H	1	0.6222	6.799		0.0000	0.000

Vertical Reactions

Support notation : Far left is #· Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.579	2.579
Overall MINimum	0.540	0.540
+D+H	1.668	1.668
+D+L+H	2.208	2.208
+D+Lr+H	2.343	2.343
+D+S+H	1.668	1.668
+D+0.750Lr+0.750L+H	2.579	2.579
+D+0.750L+0.750S+H	2.073	2.073
+D+0.60W+H	1.668	1.668
+D+0.70E+H	1.668	1.668
+D+0.750Lr+0.750L+0.450W+H	2.579	2.579
+D+0.750L+0.750S+0.450W+H	2.073	2.073
+D+0.750L+0.750S+0.5250E+H	2.073	2.073
+0.60D+0.60W+0.60H	1.001	1.001
+0.60D+0.70E+0.60H	1.001	1.001
D Only	1.668	1.668
Lr Only	0.675	0.675
L Only	0.540	0.540
S Only		
W Only		
E Only		
H Only		

3RD FLOOR BEAM - B5

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

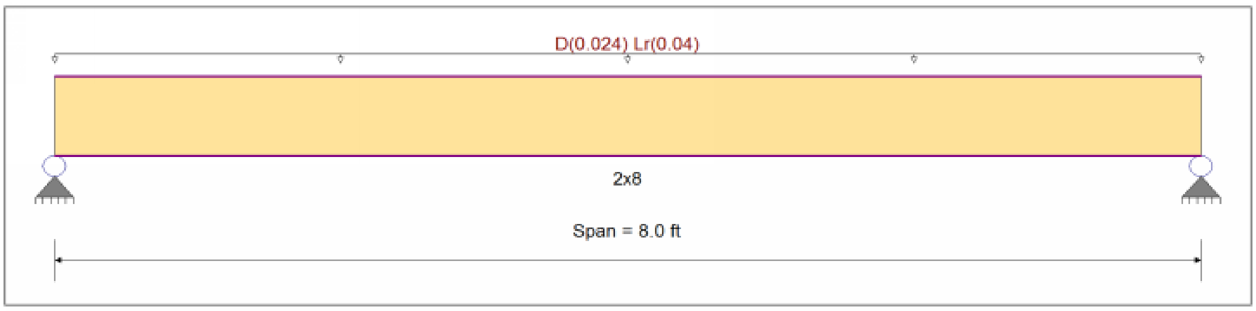
DESCRIPTIO 1214 30th St - 3rd Floor Beam 5

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Metho	Allowable Stress Design	Fb +	1000 psi	*E : Modulus of Elasti*	
Load Combinati	IBC 2018	Fb -	1000 psi	Ebend- xx	1700 ksi
		Fc - Prll	1500 psi	Eminbend - x	620 ksi
Wood Species	Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade	No.1	Fv	180 psi		
		Ft	675 psi	Density	31.21 pcf
Beam Bracing	Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increa	



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0240, Lr = 0.040 , Tributary Width = 1.0 ft, (Floor Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.244: 1	Maximum Shear Stress Ratio	=	0.139 : 1
Section used for this span		2x8	Section used for this span		2x8
	=	484.78psi		=	31.27 psi
	=	1,983.75psi		=	225.00 psi
Load Combination		+D+Lr+H	Load Combination		+D+Lr+H
Location of maximum on span	=	4.000ft	Location of maximum on span	=	7.416 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.046 in	Ratio =	2096 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.076 in	Ratio =	1263 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.135	0.077	0.90	1.200	1.00	1.15	1.00	1.00	1.00	0.21	192.55	1428.30	0.09	12.42	162.00
+D+L+H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.121	0.069	1.00	1.200	1.00	1.15	1.00	1.00	1.00	0.21	192.55	1587.00	0.09	12.42	180.00
+D+Lr+H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.244	0.139	1.25	1.200	1.00	1.15	1.00	1.00	1.00	0.53	484.78	1983.75	0.23	31.27	225.00
+D+S+H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.106	0.060	1.15	1.200	1.00	1.15	1.00	1.00	1.00	0.21	192.55	1825.05	0.09	12.42	207.00
+D+0.750Lr+0.750L+H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.208	0.118	1.25	1.200	1.00	1.15	1.00	1.00	1.00	0.45	411.72	1983.75	0.19	26.55	225.00
+D+0.750L+0.750S+H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Length = 8.0 ft	1	0.106	0.060	1.15	1.200	1.00	1.15	1.00	1.00	1.00	0.21	192.55	1825.05	0.09	12.42	207.00
+D+0.60W+H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00

3RD FLOOR BEAM - B5

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St - 3rd Floor Beam 5

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 8.0 ft	1	0.076	0.043	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.21	192.55	2539.20	0.09	12.42	288.00
+D+0.70E+H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.076	0.043	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.21	192.55	2539.20	0.09	12.42	288.00
+D+0.750Lr+0.750L+0.450W·					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.162	0.092	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.45	411.72	2539.20	0.19	26.55	288.00
+D+0.750L+0.750S+0.450W+					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.076	0.043	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.21	192.55	2539.20	0.09	12.42	288.00
+D+0.750L+0.750S+0.5250E·					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.076	0.043	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.21	192.55	2539.20	0.09	12.42	288.00
+0.60D+0.60W+0.60H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.045	0.026	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.13	115.53	2539.20	0.05	7.45	288.00
+0.60D+0.70E+0.60H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.045	0.026	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.13	115.53	2539.20	0.05	7.45	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr+H	1	0.0760	4.029		0.0000	0.000

Vertical Reactions

Support notation : Far left is # Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.265	0.265
Overall MINimum	0.160	0.160
+D+H	0.105	0.105
+D+L+H	0.105	0.105
+D+Lr+H	0.265	0.265
+D+S+H	0.105	0.105
+D+0.750Lr+0.750L+H	0.225	0.225
+D+0.750L+0.750S+H	0.105	0.105
+D+0.60W+H	0.105	0.105
+D+0.70E+H	0.105	0.105
+D+0.750Lr+0.750L+0.450W+H	0.225	0.225
+D+0.750L+0.750S+0.450W+H	0.105	0.105
+D+0.750L+0.750S+0.5250E+H	0.105	0.105
+0.60D+0.60W+0.60H	0.063	0.063
+0.60D+0.70E+0.60H	0.063	0.063
D Only	0.105	0.105
Lr Only	0.160	0.160
L Only		
S Only		
W Only		
E Only		
H Only		

3RD FLOOR BEAM - B6

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

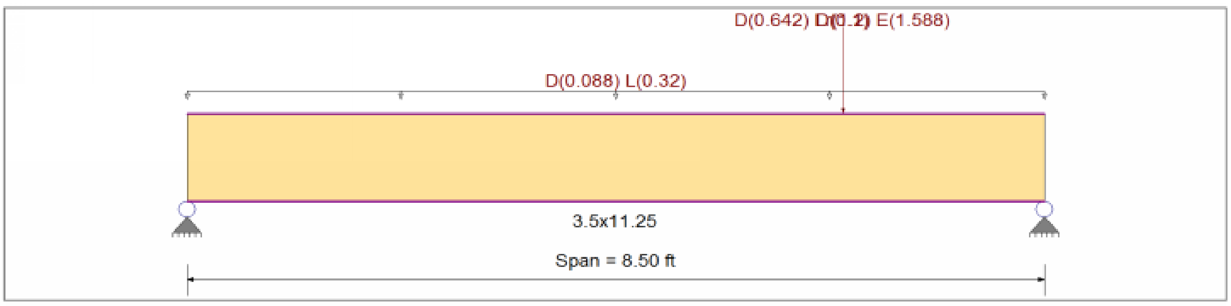
DESCRIPTIO 1214 30th St - 3rd Floor Beam 6

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Metho	Allowable Stress Design	Fb +	2,900.0 psi	*E : Modulus of Elasti*	
Load Combinati	IBC 2018	Fb -	2,900.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,900.0 psi	Eminbend - x	1,016.54 ksi
Wood Species	iLevel Truss Joist	Fc - Perp	750.0 psi		
Wood Grade	Parallam PSL 2.0E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.070 pcf
Beam Bracing	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0880, L = 0.320 , Tributary Width = 1.0 ft, (Floor Loading)
Point Load : D = 0.6420, Lr = 1.20, E = 1.588 k @ 6.50 ft, (Roof Point Loads)
Point Load : D = 0.10 k @ 6.50 ft, (Wall Point Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.256** : 1	Maximum Shear Stress Ratio	=	**0.280** : 1
Section used for this span		**3.5x11.25**	Section used for this span		**3.5x11.25**
	=	743.50 psi		=	129.69 psi
	=	2,900.00 psi		=	464.00 psi
Load Combination		+D+L	Load Combination		+1.126D+0.750L+1.313E
Location of maximum on span	=	4.653 ft	Location of maximum on span	=	7.569 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.046 in	Ratio =	2240 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.077 in	Ratio =	1319 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 8.50 ft	1	0.112	0.131	0.90	1.000	1.00	1.00	1.00	1.00	1.00	1.80	292.57	2610.00	0.90	34.30	261.00
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.50 ft	1	0.256	0.258	1.00	1.000	1.00	1.00	1.00	1.00	1.00	4.57	743.50	2900.00	1.96	74.77	290.00
+D+Lr					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

+D+Lr					[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]			[illegible]	[illegible]	[illegible]	[illegible]
Length = 8.50 ft	1	0.162	0.191	1.25	1.000	1.00	1.00	1.00	1.00	1.00	3.62	588.14	3625.00	1.82	69.26	362.50
+D+0.750Lr+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.50 ft	1	0.221	0.251	1.25	1.000	1.00	1.00	1.00	1.00	1.00	4.93	802.11	3625.00	2.39	90.87	362.50
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.50 ft	1	0.188	0.194	1.15	1.000	1.00	1.00	1.00	1.00	1.00	3.86	627.45	3335.00	1.70	64.65	333.50
+1.168D+1.750E					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

3RD FLOOR BEAM - B6

Wood Beam

Lic. # : KW-06012032 — DRE Structural Design

DESCRIPTIO 1214 30th St - 3rd Floor Beam 6

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 8.50 ft	1	0.222	0.261	1.60	1.000	1.00	1.00	1.00	1.00	1.00	6.33	1,028.49	4640.00	3.18	121.02	464.00
+1.126D+0.750L+1.313E					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.50 ft	1	0.237	0.280	1.60	1.000	1.00	1.00	1.00	1.00	1.00	6.78	1,101.37	4640.00	3.40	129.69	464.00
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.50 ft	1	0.038	0.044	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.08	175.54	4640.00	0.54	20.58	464.00
+0.4320D+1.750E					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.50 ft	1	0.176	0.206	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.01	814.62	4640.00	2.51	95.78	464.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750Lr+0.750L	1	0.0773	4.467		0.0000	0.000

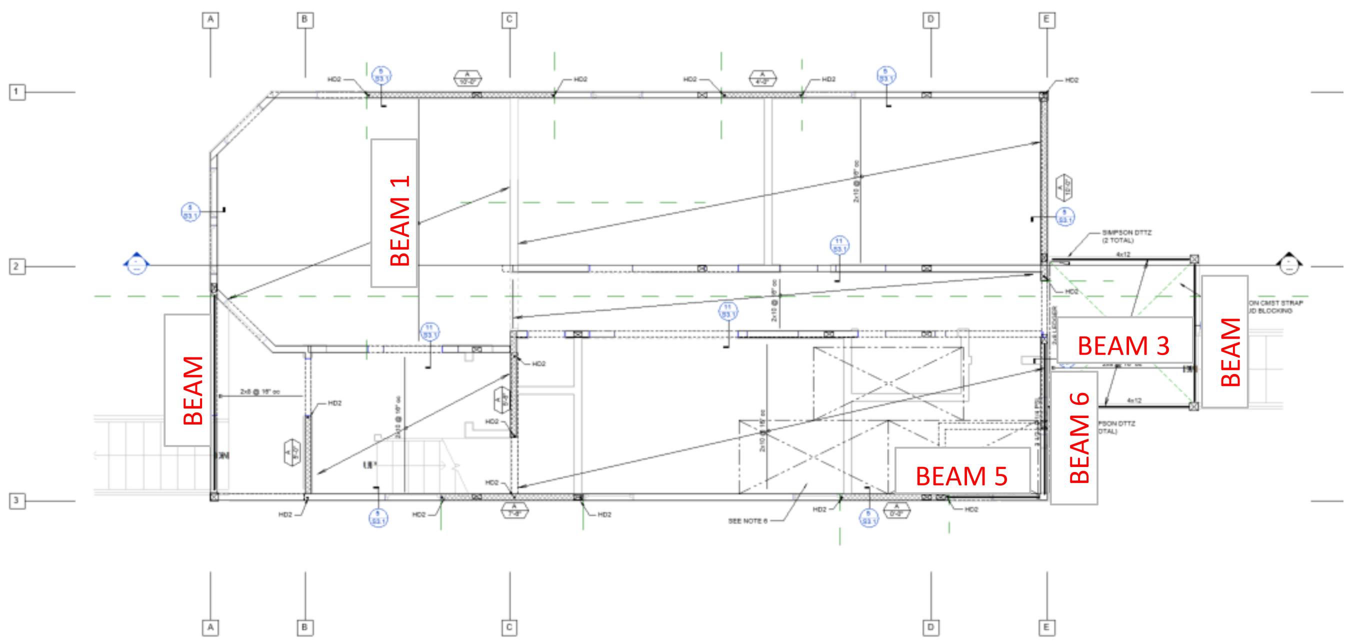
Vertical Reactions

Support notation : Far left is # — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.961	2.702
Overall MINimum	0.374	1.214
D Only	0.601	0.994
+D+L	1.961	2.354
+D+Lr	0.883	1.911
+D+0.750Lr+0.750L	1.833	2.702
+D+0.750L	1.621	2.014
+D+0.70E	0.863	1.844
+D+0.750L+0.5250E	1.817	2.651
+0.60D	0.361	0.596
+0.60D+0.70E	0.622	1.446
Lr Only	0.282	0.918
L Only	1.360	1.360
E Only	0.374	1.214

2nd FLOOR FRAMING DESIGN

2nd FLOOR FRAMING LAYOUT:



For Design:

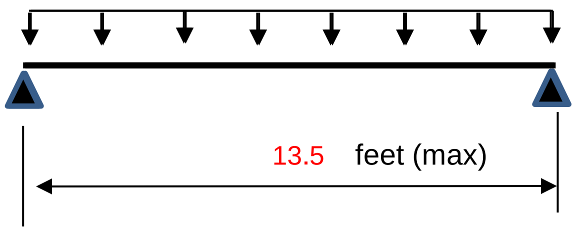
1. Assume Loading

DL =	10.7	psf (girder)
DL =	8.2	psf (joist)
LL =	40	psf

Beam 1:

Floor Loading:

Dead Load =	8	psf
Live Load =	40	psf
Trib Area =	1.33	feet
Wdl =	10.906	plf
Wll =	53.2	plf

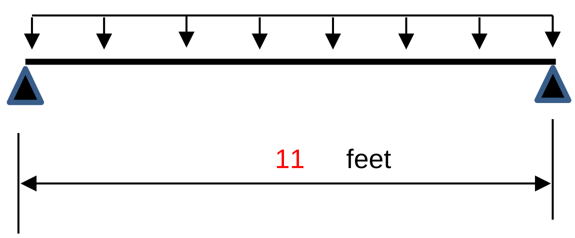


Therefore, provide 2x10 @ 16" OC
See Enercalc next page

Beam 2:

Floor Loading:

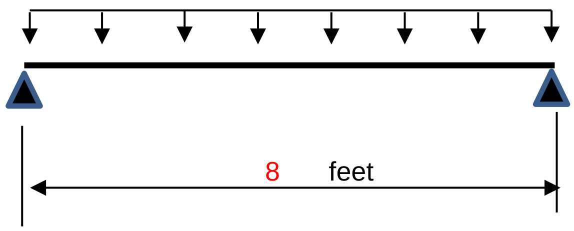
Dead Load =	11	psf
Live Load =	40	psf
Trib Area =	4	feet
Wdl =	42.8	plf
Wll =	160	plf



Therefore, provide 3 1/2 x 14 PSL

See Enercal next page

Beam 3:



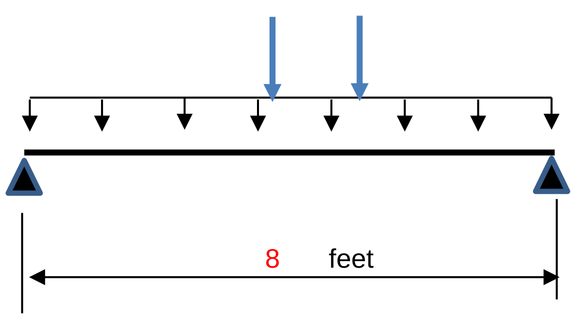
Floor Loading:

Dead Load =	8	psf
Live Load =	60	psf
Trib Area =	1.33	feet
Wdl =	11	plf
Wll =	79.8	plf

Therefore, provide 2x8 @ 16" oc

See next page for Enercalc

Beam 4:



Floor Loading:

Dead Load =	11	psf
Live Load =	40	psf
Trib Area =	4	feet
Wdl =	43	plf
Wll =	160	plf

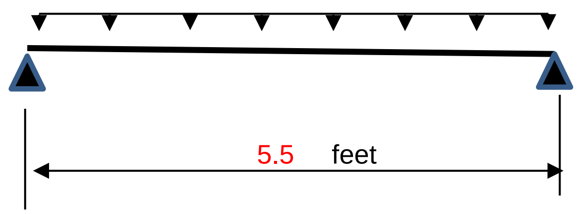
Stair Stringer Reaction:

Dead Load =	11	psf
Live Load =	40	psf
Trib Area =	2	feet
Trib Length =	5	feet
Pdl =	107	lbs
Pll =	400	lbs

Therefore, provide 4x12

See Enercalc next page

Beam 5:



Therefore, provide 3 1/2 x 9 1/4 PSL

See next page for Enercalc

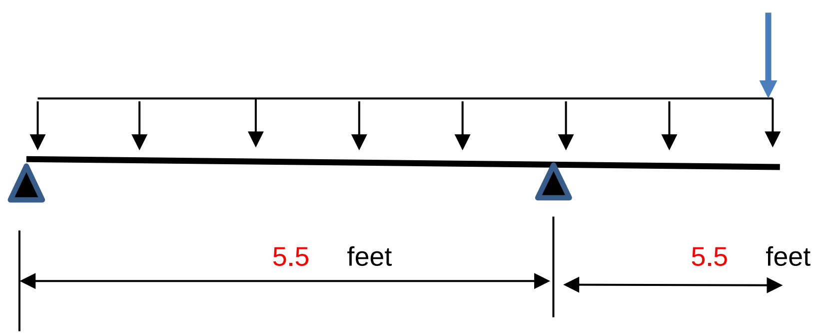
Wall Loading:

Dead Load =	10	psf
Trib Area =	16	feet
Wdl =	160	plf

Floor Loading:

Dead Load =	11	psf
Live Load =	40	psf
Trib Area =	4	feet
Wdl =	42.8	plf
Wll =	160	plf

Beam 6:



Therefore, provide 3 1/2 x 9 1/4 PSL

See Enercal next page

Floor Loading:

Dead Load =	8	psf
Live Load =	60	psf
Trib Area =	4	feet
Wdl =	32.8	plf
Wll =	240	plf

Wall Loading:

Dead Load =	10	psf
Trib Area =	16	feet
Wdl =	160	plf

Beam 5 Reaction:

Pdl =	586	lbs
Pll =	1026	lbs

2nd FLOOR BEAM - B1

Wood Beam

Lic. # : KW-06012032 | DRE Structural Design

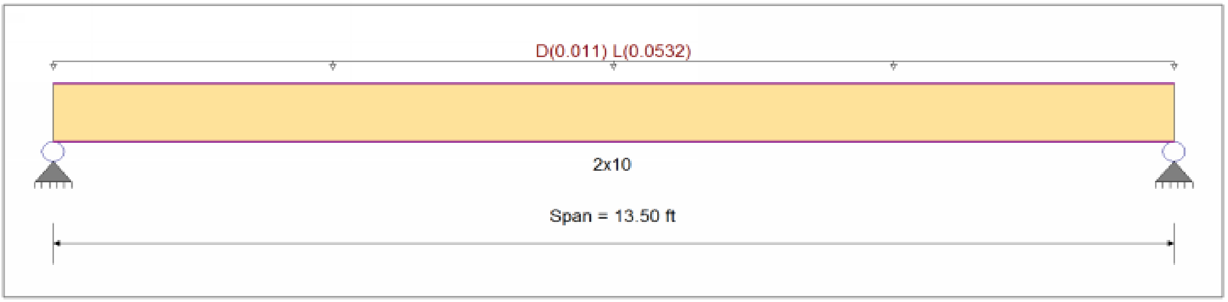
DESCRIPTIO 1214 30th St - 2nd Floor Beam 1

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Metho	Allowable Stress Design	Fb +	1000 psi	*E : Modulus of Elasti*		
Load Combinati	IBC 2018	Fb -	1000 psi	Ebend- xx	1700 ksi	
		Fc - Prll	1500 psi	Eminbend - x	620 ksi	
Wood Species	Douglas Fir-Larch	Fc - Perp	625 psi			
Wood Grade	No.1	Fv	180 psi			
		Ft	675 psi	Density	31.21 pcf	
Beam Bracing	Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increa:		



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0110, L = 0.05320 , Tributary Width = 1.0 ft, (Floor Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.590**: 1	Maximum Shear Stress Ratio	=	**0.243** : 1
Section used for this span		**2x10**	Section used for this span		**2x10**
	=	858.92 psi		=	43.67 psi
	=	1,454.75 psi		=	180.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	6.750 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.238 in	Ratio =	681 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.300 in	Ratio =	539 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.137	0.056	0.90	1.100	1.00	1.15	1.00	1.00	1.00	0.32	179.01	1309.28	0.08	9.10	162.00
Length = 0.04927 ft	1	0.002	0.056	0.90	1.100	1.00	1.15	1.00	1.00	1.00	0.00	2.60	1309.28	0.08	9.10	162.00
+D+L+H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.590	0.243	1.00	1.100	1.00	1.15	1.00	1.00	1.00	1.53	858.92	1454.75	0.40	43.67	180.00
Length = 0.04927 ft	1	0.009	0.243	1.00	1.100	1.00	1.15	1.00	1.00	1.00	0.02	12.49	1454.75	0.40	43.67	180.00
+D+Lr+H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.098	0.040	1.25	1.100	1.00	1.15	1.00	1.00	1.00	0.32	179.01	1818.44	0.08	9.10	225.00
Length = 0.04927 ft	1	0.001	0.040	1.25	1.100	1.00	1.15	1.00	1.00	1.00	0.00	2.60	1818.44	0.08	9.10	225.00
+D+S+H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.107	0.044	1.15	1.100	1.00	1.15	1.00	1.00	1.00	0.32	179.01	1672.96	0.08	9.10	207.00
Length = 0.04927 ft	1	0.002	0.044	1.15	1.100	1.00	1.15	1.00	1.00	1.00	0.00	2.60	1672.96	0.08	9.10	207.00
+D+0.750Lr+0.750L+H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00

2nd FLOOR BEAM - B1

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St - 2nd Floor Beam 1

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 13.451 ft	1	0.379	0.156	1.25	1.100	1.00	1.15	1.00	1.00	1.00	1.23	688.94	1818.44	0.32	35.03	225.00
Length = 0.04927 ft	1	0.006	0.156	1.25	1.100	1.00	1.15	1.00	1.00	1.00	0.02	10.02	1818.44	0.32	35.03	225.00
+D+0.750L+0.750S+H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.412	0.169	1.15	1.100	1.00	1.15	1.00	1.00	1.00	1.23	688.94	1672.96	0.32	35.03	207.00
Length = 0.04927 ft	1	0.006	0.169	1.15	1.100	1.00	1.15	1.00	1.00	1.00	0.02	10.02	1672.96	0.32	35.03	207.00
+D+0.60W+H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.077	0.032	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.32	179.01	2327.60	0.08	9.10	288.00
Length = 0.04927 ft	1	0.001	0.032	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.00	2.60	2327.60	0.08	9.10	288.00
+D+0.70E+H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.077	0.032	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.32	179.01	2327.60	0.08	9.10	288.00
Length = 0.04927 ft	1	0.001	0.032	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.00	2.60	2327.60	0.08	9.10	288.00
+D+0.750Lr+0.750L+0.450W-					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.296	0.122	1.60	1.100	1.00	1.15	1.00	1.00	1.00	1.23	688.94	2327.60	0.32	35.03	288.00
Length = 0.04927 ft	1	0.004	0.122	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.02	10.02	2327.60	0.32	35.03	288.00
+D+0.750L+0.750S+0.450W+					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.296	0.122	1.60	1.100	1.00	1.15	1.00	1.00	1.00	1.23	688.94	2327.60	0.32	35.03	288.00
Length = 0.04927 ft	1	0.004	0.122	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.02	10.02	2327.60	0.32	35.03	288.00
+D+0.750L+0.750S+0.5250E-					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.296	0.122	1.60	1.100	1.00	1.15	1.00	1.00	1.00	1.23	688.94	2327.60	0.32	35.03	288.00
Length = 0.04927 ft	1	0.004	0.122	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.02	10.02	2327.60	0.32	35.03	288.00
+0.60D+0.60W+0.60H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.046	0.019	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.19	107.41	2327.60	0.05	5.46	288.00
Length = 0.04927 ft	1	0.001	0.019	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.00	1.56	2327.60	0.05	5.46	288.00
+0.60D+0.70E+0.60H					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.451 ft	1	0.046	0.019	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.19	107.41	2327.60	0.05	5.46	288.00
Length = 0.04927 ft	1	0.001	0.019	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.00	1.56	2327.60	0.05	5.46	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.3004	6.799		0.0000	0.000

Vertical Reactions

Support notation : Far left is #' Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.454	0.454
Overall MINimum	0.359	0.359
+D+H	0.095	0.095
+D+L+H	0.454	0.454
+D+Lr+H	0.095	0.095
+D+S+H	0.095	0.095
+D+0.750Lr+0.750L+H	0.364	0.364
+D+0.750L+0.750S+H	0.364	0.364
+D+0.60W+H	0.095	0.095
+D+0.70E+H	0.095	0.095
+D+0.750Lr+0.750L+0.450W+H	0.364	0.364
+D+0.750L+0.750S+0.450W+H	0.364	0.364
+D+0.750L+0.750S+0.5250E+H	0.364	0.364
+0.60D+0.60W+0.60H	0.057	0.057
+0.60D+0.70E+0.60H	0.057	0.057
D Only	0.095	0.095
Lr Only		
L Only	0.359	0.359
S Only		
W Only		
E Only		
H Only		

2nd FLOOR BEAM - B2

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St - 2nd Floor Beam 2

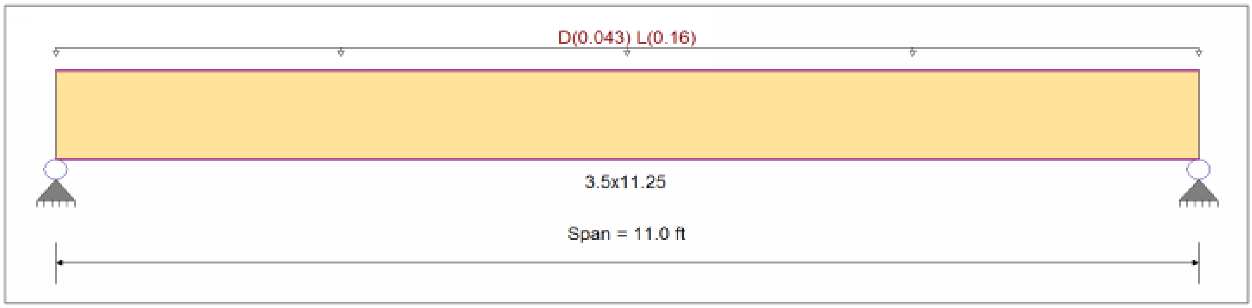
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Metho	Allowable Stress Design	Fb +	1,000.0 psi	*E : Modulus of Elasti*	
Load Combinati	IBC 2018	Fb -	1,000.0 psi	Ebend- xx	1,700.0 ksi
		Fc - Prll	1,500.0 psi	Eminbend - x	620.0 ksi
Wood Species	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade	No.1	Fv	180.0 psi		
		Ft	675.0 psi	Density	31.210 pcf

Beam Bracing Beam is Fully Braced against lateral-torsional buckling



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0430, L = 0.160 , Tributary Width = 1.0 ft, (Floor Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.520** : 1	Maximum Shear Stress Ratio	=	**0.205** : 1
Section used for this span		**3.5x11.25**	Section used for this span		**3.5x11.25**
	=	520.04 psi		=	36.88 psi
	=	1,000.00 psi		=	180.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	5.500 ft	Location of maximum on span	=	10.077 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.075 in	Ratio =	1757 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.099 in	Ratio =	1329 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.141	0.055	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.78	126.69	900.00	0.24	8.98	162.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.520	0.205	1.00	1.000	1.00	1.00	1.00	1.00	1.00	3.20	520.04	1000.00	0.97	36.88	180.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.101	0.040	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.78	126.69	1250.00	0.24	8.98	225.00
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.110	0.043	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.78	126.69	1150.00	0.24	8.98	207.00
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.337	0.133	1.25	1.000	1.00	1.00	1.00	1.00	1.00	2.59	421.70	1250.00	0.79	29.91	225.00
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.367	0.144	1.15	1.000	1.00	1.00	1.00	1.00	1.00	2.59	421.70	1150.00	0.79	29.91	207.00
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

2nd FLOOR BEAM - B2

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St - 2nd Floor Beam 2

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L		M	fb	F'b	V	fv	F'v
Length = 11.0 ft	1	0.079	0.031	1.60	1.000	1.00	1.00	1.00	1.00	1.00		0.78	126.69	1600.00	0.24	8.98	288.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00				0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.079	0.031	1.60	1.000	1.00	1.00	1.00	1.00	1.00		0.78	126.69	1600.00	0.24	8.98	288.00
+D+0.750Lr+0.750L+0.450W·					1.000	1.00	1.00	1.00	1.00	1.00				0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.264	0.104	1.60	1.000	1.00	1.00	1.00	1.00	1.00		2.59	421.70	1600.00	0.79	29.91	288.00
+D+0.750L+0.750S+0.450W+					1.000	1.00	1.00	1.00	1.00	1.00				0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.264	0.104	1.60	1.000	1.00	1.00	1.00	1.00	1.00		2.59	421.70	1600.00	0.79	29.91	288.00
+D+0.750L+0.750S+0.5250E·					1.000	1.00	1.00	1.00	1.00	1.00				0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.264	0.104	1.60	1.000	1.00	1.00	1.00	1.00	1.00		2.59	421.70	1600.00	0.79	29.91	288.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00				0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.048	0.019	1.60	1.000	1.00	1.00	1.00	1.00	1.00		0.47	76.02	1600.00	0.14	5.39	288.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00				0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.048	0.019	1.60	1.000	1.00	1.00	1.00	1.00	1.00		0.47	76.02	1600.00	0.14	5.39	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.0993	5.540		0.0000	0.000

Vertical Reactions

Support notation : Far left is # Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.163	1.163
Overall MINimum	0.880	0.880
+D+H	0.283	0.283
+D+L+H	1.163	1.163
+D+Lr+H	0.283	0.283
+D+S+H	0.283	0.283
+D+0.750Lr+0.750L+H	0.943	0.943
+D+0.750L+0.750S+H	0.943	0.943
+D+0.60W+H	0.283	0.283
+D+0.70E+H	0.283	0.283
+D+0.750Lr+0.750L+0.450W+H	0.943	0.943
+D+0.750L+0.750S+0.450W+H	0.943	0.943
+D+0.750L+0.750S+0.5250E+H	0.943	0.943
+0.60D+0.60W+0.60H	0.170	0.170
+0.60D+0.70E+0.60H	0.170	0.170
D Only	0.283	0.283
Lr Only		
L Only	0.880	0.880
S Only		
W Only		
E Only		
H Only		

2nd FLOOR BEAM - B3

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

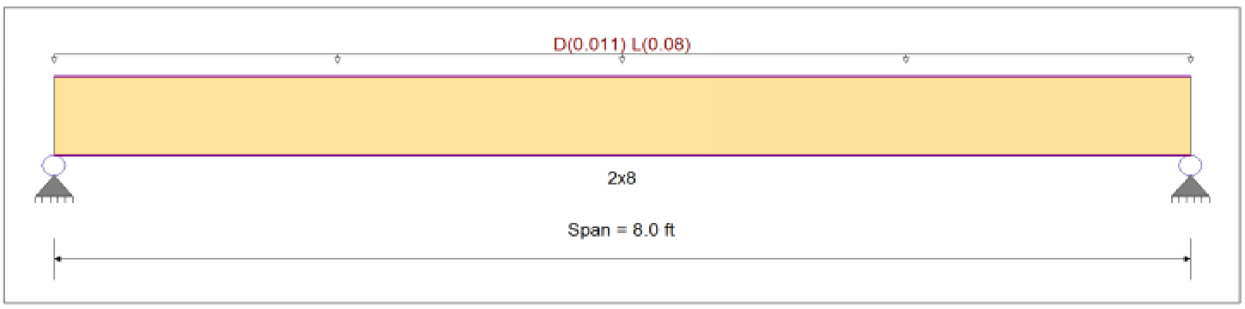
DESCRIPTIO 1214 30th St - 2nd Floor Beam 3

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Metho	Allowable Stress Design	Fb +	1,000.0 psi	*E : Modulus of Elast*	
Load Combinati	IBC 2018	Fb -	1,000.0 psi	Ebend- xx	1,700.0 ksi
		Fc - Prll	1,500.0 psi	Eminbend - x	620.0 ksi
Wood Species	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade	No.1	Fv	180.0 psi		
		Ft	675.0 psi	Density	31.210 pcf
Beam Bracing	Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increa	



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0110, L = 0.080 , Tributary Width = 1.0 ft, (Floor Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.430** : 1	Maximum Shear Stress Ratio	=	**0.244** : 1
Section used for this span		**2x8**	Section used for this span		**2x8**
	=	682.03 psi		=	43.99 psi
	=	1,587.00 psi		=	180.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	4.000 ft	Location of maximum on span	=	7.416 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.092 in	Ratio =	1048 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.107 in	Ratio =	898 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.068	0.039	0.90	1.200	1.00	1.15	1.00	1.00	1.00	0.11	97.58	1428.30	0.05	6.29	162.00
+D+L+H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.430	0.244	1.00	1.200	1.00	1.15	1.00	1.00	1.00	0.75	682.03	1587.00	0.32	43.99	180.00
+D+Lr+H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.049	0.028	1.25	1.200	1.00	1.15	1.00	1.00	1.00	0.11	97.58	1983.75	0.05	6.29	225.00
+D+S+H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.053	0.030	1.15	1.200	1.00	1.15	1.00	1.00	1.00	0.11	97.58	1825.05	0.05	6.29	207.00
+D+0.750Lr+0.750L+H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.270	0.154	1.25	1.200	1.00	1.15	1.00	1.00	1.00	0.59	535.92	1983.75	0.25	34.56	225.00
+D+0.750L+0.750S+H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.294	0.167	1.15	1.200	1.00	1.15	1.00	1.00	1.00	0.59	535.92	1825.05	0.25	34.56	207.00
+D+0.60W+H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00

2nd FLOOR BEAM - B3

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St - 2nd Floor Beam 3

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 8.0 ft	1	0.038	0.022	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.11	97.58	2539.20	0.05	6.29	288.0
+D+0.70E+H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.0
Length = 8.0 ft	1	0.038	0.022	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.11	97.58	2539.20	0.05	6.29	288.0
+D+0.750Lr+0.750L+0.450W-					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.0
Length = 8.0 ft	1	0.211	0.120	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.59	535.92	2539.20	0.25	34.56	288.0
+D+0.750L+0.750S+0.450W+					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.0
Length = 8.0 ft	1	0.211	0.120	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.59	535.92	2539.20	0.25	34.56	288.0
+D+0.750L+0.750S+0.5250E-					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.0
Length = 8.0 ft	1	0.211	0.120	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.59	535.92	2539.20	0.25	34.56	288.0
+0.60D+0.60W+0.60H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.0
Length = 8.0 ft	1	0.023	0.013	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.06	58.55	2539.20	0.03	3.78	288.0
+0.60D+0.70E+0.60H					1.200	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.0
Length = 8.0 ft	1	0.023	0.013	1.60	1.200	1.00	1.15	1.00	1.00	1.00	0.06	58.55	2539.20	0.03	3.78	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.1069	4.029		0.0000	0.000

Vertical Reactions

Support notation : Far left is #' Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.373	0.373
Overall MINimum	0.320	0.320
+D+H	0.053	0.053
+D+L+H	0.373	0.373
+D+Lr+H	0.053	0.053
+D+S+H	0.053	0.053
+D+0.750Lr+0.750L+H	0.293	0.293
+D+0.750L+0.750S+H	0.293	0.293
+D+0.60W+H	0.053	0.053
+D+0.70E+H	0.053	0.053
+D+0.750Lr+0.750L+0.450W+H	0.293	0.293
+D+0.750L+0.750S+0.450W+H	0.293	0.293
+D+0.750L+0.750S+0.5250E+H	0.293	0.293
+0.60D+0.60W+0.60H	0.032	0.032
+0.60D+0.70E+0.60H	0.032	0.032
D Only	0.053	0.053
Lr Only		
L Only	0.320	0.320
S Only		
W Only		
E Only		
H Only		

2nd FLOOR BEAM - B4

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St - 2nd Floor Beam 4

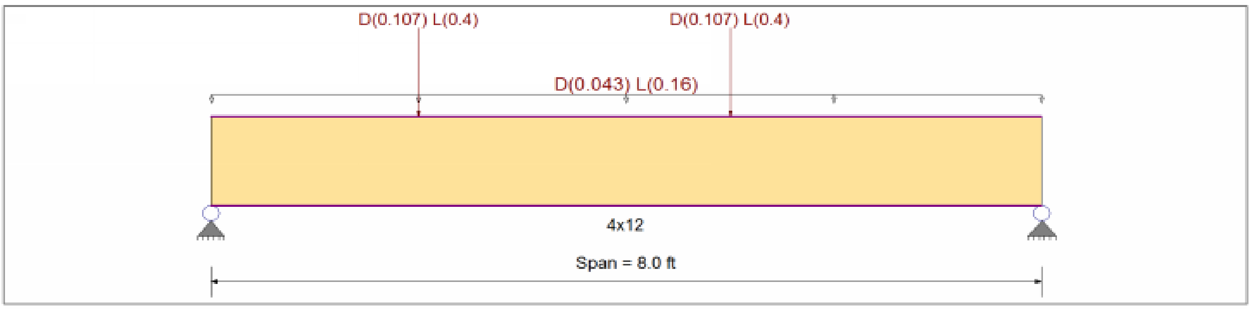
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Metho	Allowable Stress Design	Fb +	1,000.0 psi	*E : Modulus of Elasti*	
Load Combinati	IBC 2018	Fb -	1,000.0 psi	Ebend- xx	1,700.0 ksi
		Fc - Prll	1,500.0 psi	Eminbend - x	620.0 ksi
Wood Species	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade	No.1	Fv	180.0 psi		
		Ft	675.0 psi	Density	31.210 pcf

Beam Bracing Beam is Fully Braced against lateral-torsional buckling



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0430, L = 0.160 , Tributary Width = 1.0 ft, (Floor Load)
Point Load : D = 0.1070, L = 0.40 k @ 5.0 ft
Point Load : D = 0.1070, L = 0.40 k @ 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.439**: 1	Maximum Shear Stress Ratio	=	**0.258** : 1
Section used for this span		**4x12**	Section used for this span		**4x12**
	=	482.62psi		=	46.43 psi
	=	1,100.00psi		=	180.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	4.292ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.038 in	Ratio =	2537 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.049 in	Ratio =	1955 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.112	0.065	0.90	1.100	1.00	1.00	1.00	1.00	1.00	0.68	110.77	990.00	0.28	10.60	162.00
+D+L+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.439	0.258	1.00	1.100	1.00	1.00	1.00	1.00	1.00	2.97	482.62	1100.00	1.22	46.43	180.00
+D+Lr+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.081	0.047	1.25	1.100	1.00	1.00	1.00	1.00	1.00	0.68	110.77	1375.00	0.28	10.60	225.00
+D+S+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.088	0.051	1.15	1.100	1.00	1.00	1.00	1.00	1.00	0.68	110.77	1265.00	0.28	10.60	207.00
+D+0.750Lr+0.750L+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.283	0.167	1.25	1.100	1.00	1.00	1.00	1.00	1.00	2.40	389.66	1375.00	0.98	37.48	225.00
+D+0.750L+0.750S+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

2nd FLOOR BEAM - B4

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St - 2nd Floor Beam 4

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 8.0 ft	1	0.308	0.181	1.15	1.100	1.00	1.00	1.00	1.00	1.00	2.40	389.66	1265.00	0.98	37.48	207.00
+D+0.60W+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.063	0.037	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.68	110.77	1760.00	0.28	10.60	288.00
+D+0.70E+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.063	0.037	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.68	110.77	1760.00	0.28	10.60	288.00
+D+0.750Lr+0.750L+0.450W·					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.221	0.130	1.60	1.100	1.00	1.00	1.00	1.00	1.00	2.40	389.66	1760.00	0.98	37.48	288.00
+D+0.750L+0.750S+0.450W+					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.221	0.130	1.60	1.100	1.00	1.00	1.00	1.00	1.00	2.40	389.66	1760.00	0.98	37.48	288.00
+D+0.750L+0.750S+0.5250E·					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.221	0.130	1.60	1.100	1.00	1.00	1.00	1.00	1.00	2.40	389.66	1760.00	0.98	37.48	288.00
+0.60D+0.60W+0.60H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.038	0.022	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.41	66.46	1760.00	0.17	6.36	288.00
+0.60D+0.70E+0.60H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.038	0.022	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.41	66.46	1760.00	0.17	6.36	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.0491	4.000		0.0000	0.000

Vertical Reactions

Support notation : Far left is #· Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.417	1.290
Overall MINimum	1.090	0.990
+D+H	0.327	0.300
+D+L+H	1.417	1.290
+D+Lr+H	0.327	0.300
+D+S+H	0.327	0.300
+D+0.750Lr+0.750L+H	1.144	1.042
+D+0.750L+0.750S+H	1.144	1.042
+D+0.60W+H	0.327	0.300
+D+0.70E+H	0.327	0.300
+D+0.750Lr+0.750L+0.450W+H	1.144	1.042
+D+0.750L+0.750S+0.450W+H	1.144	1.042
+D+0.750L+0.750S+0.5250E+H	1.144	1.042
+0.60D+0.60W+0.60H	0.196	0.180
+0.60D+0.70E+0.60H	0.196	0.180
D Only	0.327	0.300
Lr Only		
L Only	1.090	0.990
S Only		
W Only		
E Only		
H Only		

2nd FLOOR BEAM - B5

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St - 2nd Floor Beam 5

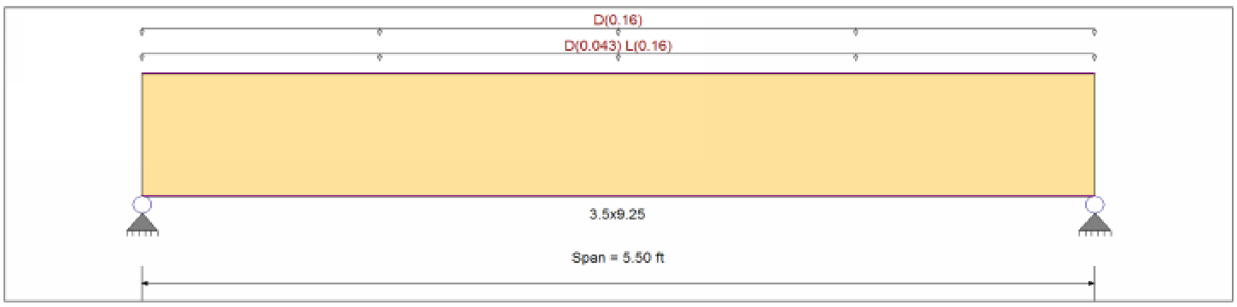
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Metho	Allowable Stress Design	Fb +	2900 psi	*E : Modulus of Elasti*	
Load Combinati	IBC 2018	Fb -	2900 psi	Ebend- xx	2000 ksi
		Fc - Prll	2900 psi	Eminbend - x	1016.535 ksi
Wood Species	iLevel Truss Joist	Fc - Perp	750 psi		
Wood Grade	Parallam PSL 2.0E	Fv	290 psi		
		Ft	2025 psi	Density	45.07 pcf

Beam Bracing: Beam is Fully Braced against lateral-torsional buckling



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0430, L = 0.160 , Tributary Width = 1.0 ft, (Floor Load)
Uniform Load : D = 0.160 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.117**: 1	Maximum Shear Stress Ratio	=	**0.118** : 1
Section used for this span		**3.5x9.25**	Section used for this span		**3.5x9.25**
	=	339.22 psi		=	34.36 psi
	=	2,900.00 psi		=	290.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	2.750 ft	Location of maximum on span	=	4.737 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.007 in	Ratio =	9196 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.017 in	Ratio =	3943 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 5.50 ft	1	0.074	0.075	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.81	193.76	2610.00	0.42	19.62	261.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 5.50 ft	1	0.117	0.118	1.00	1.000	1.00	1.00	1.00	1.00	1.00	1.41	339.22	2900.00	0.74	34.36	290.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 5.50 ft	1	0.053	0.054	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.81	193.76	3625.00	0.42	19.62	362.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 5.50 ft	1	0.058	0.059	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.81	193.76	3335.00	0.42	19.62	333.50
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 5.50 ft	1	0.084	0.085	1.25	1.000	1.00	1.00	1.00	1.00	1.00	1.26	302.85	3625.00	0.66	30.67	362.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Length = 5.50 ft	1	0.091	0.092	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.26	302.85	3335.00	0.66	30.67	333.50

2nd FLOOR BEAM - B5

Wood Beam

Lic. # : KW-06012032 DRE Structural Design

DESCRIPTIO 1214 30th St - 2nd Floor Beam 5

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.
Length = 5.50 ft	1	0.042	0.042	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.81	193.76	4640.00	0.42	19.62	464.
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.
Length = 5.50 ft	1	0.042	0.042	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.81	193.76	4640.00	0.42	19.62	464.
+D+0.750Lr+0.750L+0.450W-					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.
Length = 5.50 ft	1	0.065	0.066	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.26	302.85	4640.00	0.66	30.67	464.
+D+0.750L+0.750S+0.450W+					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.
Length = 5.50 ft	1	0.065	0.066	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.26	302.85	4640.00	0.66	30.67	464.
+D+0.750L+0.750S+0.5250E-					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.
Length = 5.50 ft	1	0.065	0.066	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.26	302.85	4640.00	0.66	30.67	464.
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.
Length = 5.50 ft	1	0.025	0.025	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.48	116.26	4640.00	0.25	11.77	464.
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.
Length = 5.50 ft	1	0.025	0.025	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.48	116.26	4640.00	0.25	11.77	464.

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.0167	2.770		0.0000	0.000

Vertical Reactions

Support notation : Far left is #' Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.026	1.026
Overall MINimum	0.440	0.440
+D+H	0.586	0.586
+D+L+H	1.026	1.026
+D+Lr+H	0.586	0.586
+D+S+H	0.586	0.586
+D+0.750Lr+0.750L+H	0.916	0.916
+D+0.750L+0.750S+H	0.916	0.916
+D+0.60W+H	0.586	0.586
+D+0.70E+H	0.586	0.586
+D+0.750Lr+0.750L+0.450W+H	0.916	0.916
+D+0.750L+0.750S+0.450W+H	0.916	0.916
+D+0.750L+0.750S+0.5250E+H	0.916	0.916
+0.60D+0.60W+0.60H	0.352	0.352
+0.60D+0.70E+0.60H	0.352	0.352
D Only	0.586	0.586
Lr Only		
L Only	0.440	0.440
S Only		
W Only		
E Only		
H Only		

2nd FLOOR BEAM - B6

Wood Beam

Lic. # : KW-06012032 | DRE Structural Design

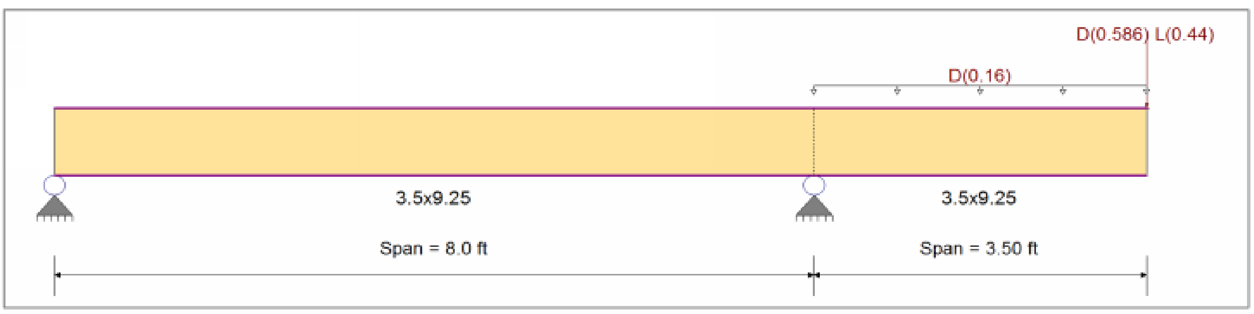
DESCRIPTIO 1214 30th St - 2nd Floor Beam 6

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Metho	Allowable Stress Design	Fb +	2,900.0 psi	*E : Modulus of Elasti*	
Load Combinati	IBC 2018	Fb -	2,900.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,900.0 psi	Eminbend - x	1,016.54 ksi
Wood Species	iLevel Truss Joist	Fc - Perp	750.0 psi		
Wood Grade	Parallam PSL 2.0E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.070 pcf
Beam Bracing	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 2
Point Load : D = 0.5860, L = 0.440 k @ 3.50 ft, (Floor Load)
Uniform Load : D = 0.160 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.384**: 1	Maximum Shear Stress Ratio	=	**0.238** : 1
Section used for this span		**3.5x9.25**	Section used for this span		**3.5x9.25**
	=	1,113.91 psi		=	69.11 psi
	=	2,900.00 psi		=	290.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	8.000 ft	Location of maximum on span	=	8.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.077 in	Ratio =	1086 >=360
Max Upward Transient Deflection	-0.024 in	Ratio =	4027 >=360
Max Downward Total Deflection	0.226 in	Ratio =	372 >=240
Max Upward Total Deflection	-0.070 in	Ratio =	1376 >=240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.285	0.187	0.90	1.000	1.00	1.00	1.00	1.00	1.00	3.09	743.65	2610.00	1.05	48.73	261.00
Length = 3.50 ft	2	0.285	0.187	0.90	1.000	1.00	1.00	1.00	1.00	1.00	3.09	743.65	2610.00	1.05	48.73	261.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.384	0.238	1.00	1.000	1.00	1.00	1.00	1.00	1.00	4.63	1,113.91	2900.00	1.49	69.11	290.00
Length = 3.50 ft	2	0.384	0.238	1.00	1.000	1.00	1.00	1.00	1.00	1.00	4.63	1,113.91	2900.00	1.49	69.11	290.00

Length = 3.50 ft	2	0.384	0.236	1.00	1.000	1.00	1.00	1.00	1.00	1.00	4.63	1,113.91	2900.00	1.49	69.11	290.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.205	0.134	1.25	1.000	1.00	1.00	1.00	1.00	1.00	3.09	743.65	3625.00	1.05	48.73	362.50
Length = 3.50 ft	2	0.205	0.134	1.25	1.000	1.00	1.00	1.00	1.00	1.00	3.09	743.65	3625.00	1.05	48.73	362.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.223	0.146	1.15	1.000	1.00	1.00	1.00	1.00	1.00	3.09	743.65	3335.00	1.05	48.73	333.50

2nd FLOOR BEAM - B6

Wood Beam

Lic. # : KW-06012032 | DRE Structural Design

DESCRIPTIO 1214 30th St - 2nd Floor Beam 6

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 3.50 ft	2	0.223	0.146	1.15	1.000	1.00	1.00	1.00	1.00	1.00	3.09	743.65	3335.00	1.05	48.73	333.50
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.282	0.177	1.25	1.000	1.00	1.00	1.00	1.00	1.00	4.25	1,021.34	3625.00	1.38	64.02	362.50
Length = 3.50 ft	2	0.282	0.177	1.25	1.000	1.00	1.00	1.00	1.00	1.00	4.25	1,021.34	3625.00	1.38	64.02	362.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.306	0.192	1.15	1.000	1.00	1.00	1.00	1.00	1.00	4.25	1,021.34	3335.00	1.38	64.02	333.50
Length = 3.50 ft	2	0.306	0.192	1.15	1.000	1.00	1.00	1.00	1.00	1.00	4.25	1,021.34	3335.00	1.38	64.02	333.50
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.160	0.105	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.09	743.65	4640.00	1.05	48.73	464.00
Length = 3.50 ft	2	0.160	0.105	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.09	743.65	4640.00	1.05	48.73	464.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.160	0.105	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.09	743.65	4640.00	1.05	48.73	464.00
Length = 3.50 ft	2	0.160	0.105	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.09	743.65	4640.00	1.05	48.73	464.00
+D+0.750Lr+0.750L+0.450W·					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.220	0.138	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.25	1,021.34	4640.00	1.38	64.02	464.00
Length = 3.50 ft	2	0.220	0.138	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.25	1,021.34	4640.00	1.38	64.02	464.00
+D+0.750L+0.750S+0.450W+					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.220	0.138	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.25	1,021.34	4640.00	1.38	64.02	464.00
Length = 3.50 ft	2	0.220	0.138	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.25	1,021.34	4640.00	1.38	64.02	464.00
+D+0.750L+0.750S+0.5250E·					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.220	0.138	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.25	1,021.34	4640.00	1.38	64.02	464.00
Length = 3.50 ft	2	0.220	0.138	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.25	1,021.34	4640.00	1.38	64.02	464.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.096	0.063	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.86	446.19	4640.00	0.63	29.24	464.00
Length = 3.50 ft	2	0.096	0.063	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.86	446.19	4640.00	0.63	29.24	464.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.096	0.063	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.86	446.19	4640.00	0.63	29.24	464.00
Length = 3.50 ft	2	0.096	0.063	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.86	446.19	4640.00	0.63	29.24	464.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+L+H	-0.0697	4.648
+D+L+H	2	0.2256	3.500		0.0000	4.648

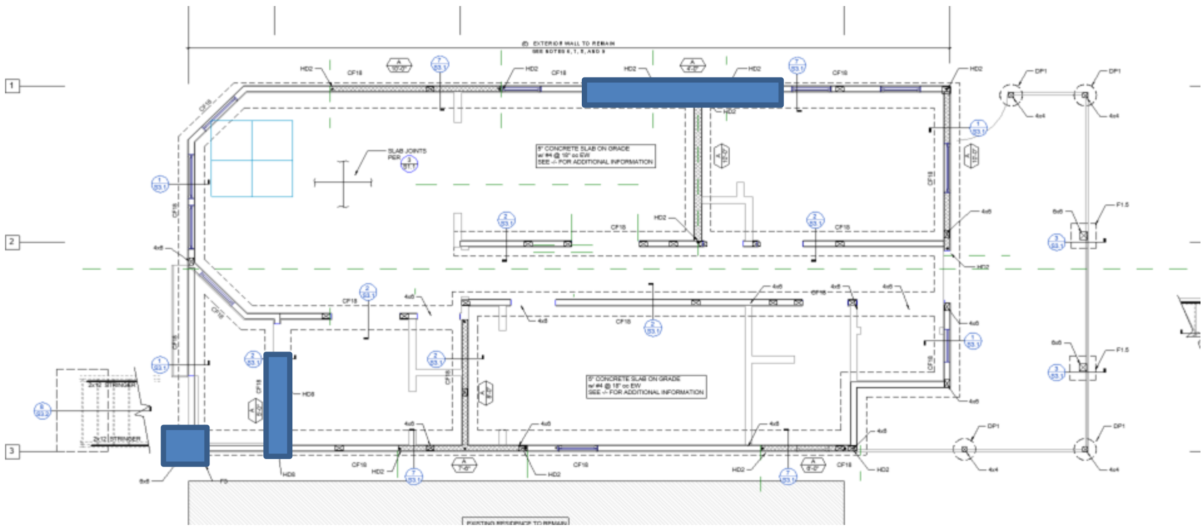
Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	-0.539	2.241	
Overall MINimum	-0.346	0.633	
+D+H	-0.346	1.609	
+D+L+H	-0.539	2.241	
+D+Lr+H	-0.346	1.609	
+D+S+H	-0.346	1.609	
+D+0.750Lr+0.750L+H	-0.490	2.083	
+D+0.750L+0.750S+H	-0.490	2.083	
+D+0.60W+H	-0.346	1.609	
+D+0.70E+H	-0.346	1.609	
+D+0.750Lr+0.750L+0.450W+H	-0.490	2.083	
+D+0.750L+0.750S+0.450W+H	-0.490	2.083	
+D+0.750L+0.750S+0.5250E+H	-0.490	2.083	
+0.60D+0.60W+0.60H	-0.208	0.965	
+0.60D+0.70E+0.60H	-0.208	0.965	
D Only	-0.346	1.609	
Lr Only			

FOUNDAITON DESIGN

New Foundation Layout



For Design:

1. See above for new footing layout.
2. Assume allowable soil pressure = 1500 psf (code minimum)
3. Assume 1/3 increase for short term loading
4. Check 2 locations
    1. Check new shearwall along Grid 1

Pdl =	2.86	kips	includes roof dead and live load
Plr =	7.8	kips	
Pseismic =	1.1	kips	

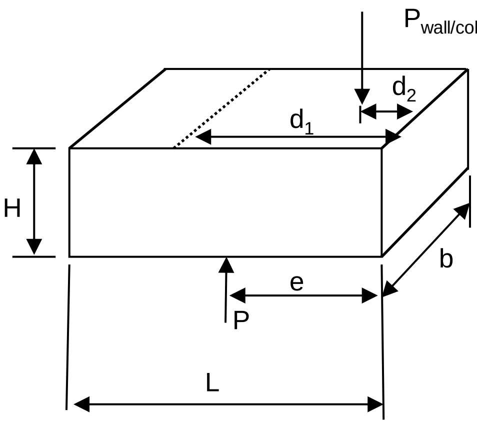
    2. Check new shearwall along Grid B

Pdl =	0.8	kips	includes roof dead and live load
Plr =	1.2	kips	
Pseismic =	0.397	kips	

    3. Check new post on existing cont ftg (Grids A/3)

Pdl =	0.44	kips	includes roof dead and live load
Plr =	0.8	kips	
Pseismic =		kips	

Eccentrically Loaded Footing Design



$P_{wall/col}$ = 1,066 lbs
d_1 = 2.0 ft d_2 = 0.5 ft

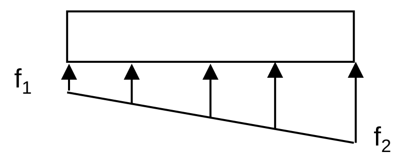
Allowable Soil Pressure= 1500 psf

☐ Short Term Loads(4/3 increase)

H = 1.5 ft
L = 1.5 ft
b = 1.0 ft

Weight of conc. = 150 pcf

Loading Diagram



Stem Thickness = 0 in
Stem Width = 0.0 ft

Slab Thickness = 5 in
Eff. Slab Width = 6.0 ft

P_{ftg} = 338 lbs
P_{stem} = 0 lbs I_{ftg} = 0.3 ft4
P_{slab} = 375 lbs

P_{total} = 1,779 lbs

Moment about center of footing = -15 lbs-ft

e = M / P = -0.01 ft **OK**

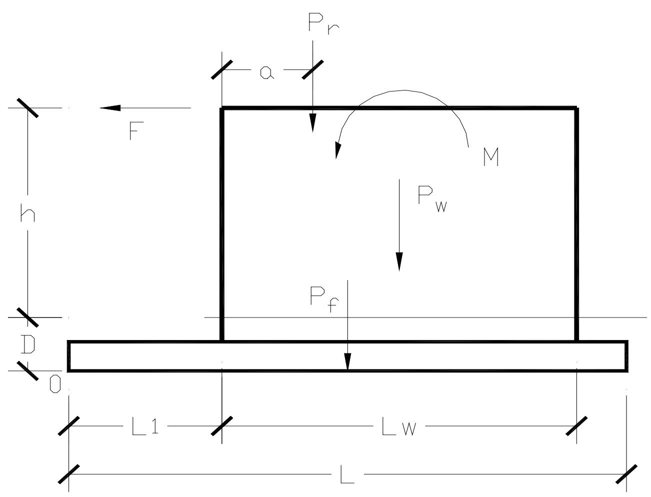
f_1 = 1225 psf OK < 1500 psf

f_2 = 1146 psf OK < 1500 psf

Uniform bearing pressure form footing, stem and slab = 475 psf

f_{max} withouth Slab, Stem or Footing = 671 psf OK < 1500 psf

Footing Design of Shear Wall Based on ACI 318-14



INPUT DATA: **Line 1 (10'-0")**

WALL LENGTH	L_w =	10	ft			
WALL HEIGHT	h =	8	ft			
WALL THICKNESS	t =	4	in			
FOOTING LENGTH	L =	15	ft			
	L_1 =	5	ft			
FOOTING WIDTH	B =	1.5	ft			
FOOTING THICKNESS	T =	18	in			
FOOTING EMBEDMENT DEPTH	D =	2.5	ft			
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	2000	psf			
UNFACTORED DEAD LOAD AT TOP WAI	$P_{r,DL}$ =	2.86	kips			
UNFACTORED LIVE LOAD AT TOP WALI	$P_{r,LL}$ =	7.8	kips			
TOP LOAD LOCATION	a =	5	ft			
WALL SELF WEIGHT	P_w =	1.6	kips			
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic			
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	1.1	kips			**THE FOOTING DESIGN IS ADEQUATE.**
	M =	46.8	ft-kips			
CONCRETE STRENGTH	f_c' =	2500	psi			
REBAR YIELD STRESS	f_y =	60	ksi			
TOP BARS, LONGITUDINAL		3	#	5		< == Not Required
BOTTOM BARS, LONGITUDINAL		3	#	5		
BOTTOM BARS, TRANSVERSE	#	4	@	24	in o.c.	< == Not Required

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1, & ASCE 7-16 12.13.4)

$F = M_R / M_O =$ 1.39 > 1.4 x 0.75 / 0.9 for seismic **[Satisfactory]**

Where P_f = 4.89375 kips (footing self weight)

$M_O = F\,(h + D) + M =$ 58 ft-kips (overturning moment)

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 81 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 17.15 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 159 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 1.61 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases} \quad = \quad 1.25 \text{ ksf} \quad < \quad 4/3\, q_a$$

[Satisfactory]

Where e = 1.61 ft, < (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

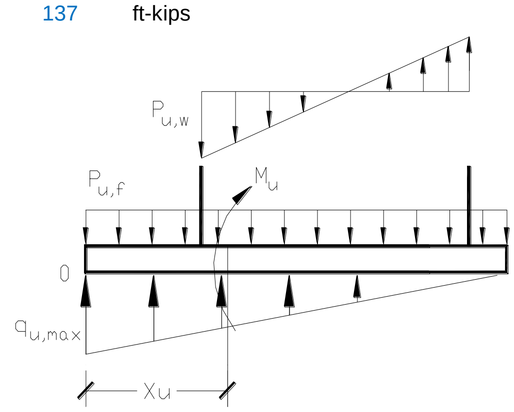
$M_{u,R} =$ $1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,LL}(L_1 + a) =$ 137 ft-kips

$M_{u,o} =$ $1.4\,[F(h + D) + M] =$ 82 ft-kips

$P_u =$ $1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,LL} =$ 15 kips

$e_u = 0.5L - (M_{u,R} - M_{u,o}) / P_u =$ 3.87 ft

$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases} \quad 1.85 \text{ ksf}$$



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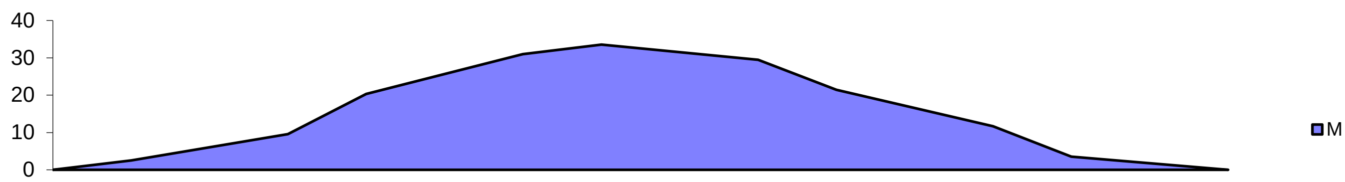
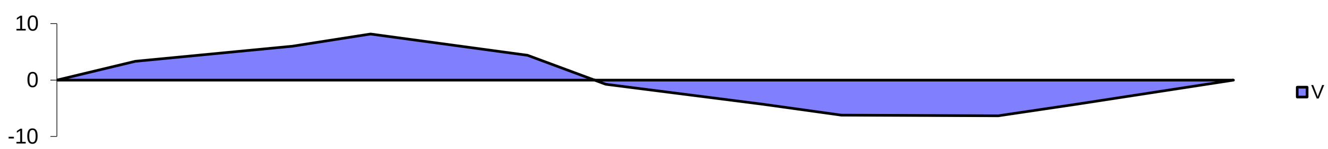
BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	1.50	3.00	4.50	6.00	7.50	9.00	10.50	12.00	13.50	15.00
$P_{u,w}$ (klf)	0.0	0.0	0.0	0.0	4.8	3.4	1.9	0.4	-1.0	-2.5	-4.0
$M_{u,w}$ (ft-k)	0	0	0	0	-3	-16	-36	-61	-87	-110	-128
$V_{u,w}$ (kips)	0	0	0	0	-5	-12	-15	-17	-17	-14	-9
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	0	-2	-4	-7	-11	-16	-22	-28	-36	-44
$V_{u,f}$ (kips)	0	-1	-1	-2	-2	-3	-4	-4	-5	-5	-6
q_u (ksf)	-1.9	-1.6	-1.3	-1.1	-0.8	-0.6	-0.3	-0.1	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	3	11	24	41	60	82	104	127	149	172
$V_{u,q}$ (kips)	0	4	7	10	12	14	15	15	15	15	15
ΣM_u (ft-k)	**0**	**3**	**10**	**20**	**31**	**34**	**30**	**21**	**12**	**3**	**0**
ΣV_u (kips)	**0**	**3**	**6**	**8**	**4**	**-1**	**-4**	**-6**	**-6**	**-4**	**0**





Location	$M_{u,max}$		d (in)	ρ_{reqD}	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	0	ft-k	14.69	0.0000	0.0000	8	kips	22	kips
Bottom Longitudinal	34	ft-k	14.69	0.0020	0.0036	8	kips	22	kips
Bottom Transverse	0	ft-k / ft	14.13	0.0000	0.0000	0	kips / ft	14	kips / ft

Where
$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

ρ_{min} = 0.0018

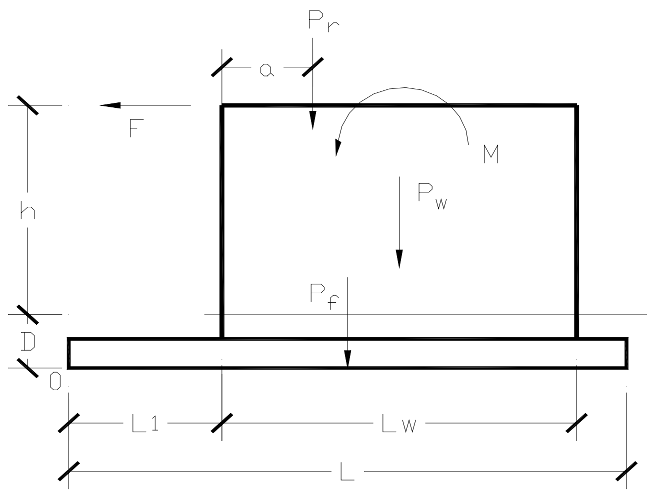
$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0129

[Satisfactory]

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: **Line B (5'-0")**

WALL LENGTH	L_w =	5	ft				
WALL HEIGHT	h =	8	ft				
WALL THICKNESS	t =	4	in				
FOOTING LENGTH	L =	9	ft				
	L_1 =	4	ft				
FOOTING WIDTH	B =	1.5	ft				
FOOTING THICKNESS	T =	18	in				
FOOTING EMBEDMENT DEPTH	D =	3.5	ft				
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	2000	psf				
UNFACTORED DEAD LOAD AT TOP WAI	$P_{r,DL}$ =	1	kips				
UNFACTORED LIVE LOAD AT TOP WALI	$P_{r,LL}$ =	1.2	kips				
TOP LOAD LOCATION	a =	2.5	ft				
WALL SELF WEIGHT	P_w =	1	kips				
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic				
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	0.391	kips				**THE FOOTING DESIGN IS ADEQUATE.**
	M =	16.5	ft-kips				
CONCRETE STRENGTH	f_c' =	2500	psi				
REBAR YIELD STRESS	f_y =	60	ksi				
TOP BARS, LONGITUDINAL		3		#	5		**< == Not Required**
BOTTOM BARS, LONGITUDINAL		3		#	5		
BOTTOM BARS, TRANSVERSE	#	4		@	24	in o.c.	**< == Not Required**



ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1, & ASCE 7-16 12.13.4)

$F = M_R / M_O$ = 1.25 > 1.4 x 0.75 / 0.9 for seismic **[Satisfactory]**

Where P_f = 2.93625 kips (footing self weight)

$M_O = F (h + D) + M$ = 21 ft-kips (overturning moment)

$M_R = (P_{r,DL}) (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w)$ = 26 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f$ = 6.14 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL}) (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w)$ = 34 ft-kips (resisting moment with live load)

$e = 0.5 L - (M_R - M_O) / P$ = 2.38 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$ = 1.29 ksf < 4 / 3 q_a

[Satisfactory]

Where e = 2.38 ft, > (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

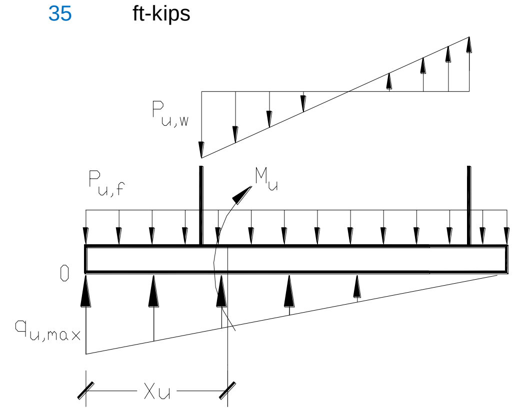
$M_{u,R}$ = $1.2 [P_{r,DL} (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w)] + 0.5 P_{r,LL}(L_1 + a)$ = 35 ft-kips

$M_{u,o}$ = $1.4 [F(h + D) + M]$ = 29 ft-kips

P_u = $1.2 (P_{r,DL} + P_f + P_w) + 0.5 P_{r,LL}$ = 7 kips

$e_u = 0.5L - (M_{u,R} - M_{u,o}) / P_u$ = 3.59 ft

$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$ 3.17 ksf



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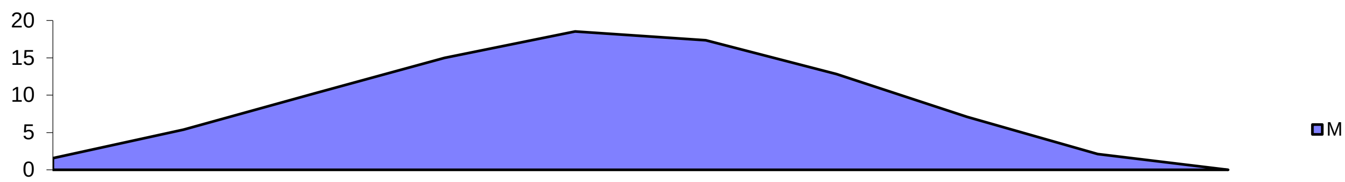
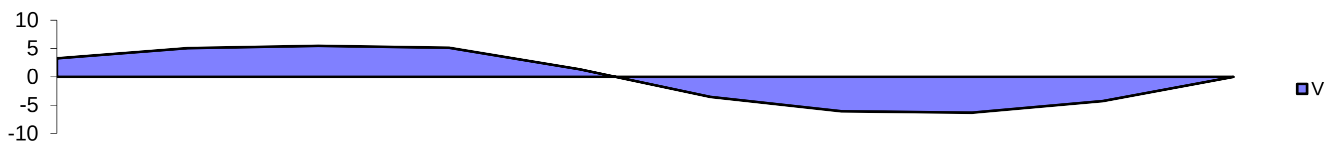
BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	0.90	1.80	2.70	3.60	4.50	5.40	6.30	7.20	8.10	9.00
$P_{u,w}$ (klf)	0.0	0.0	0.0	0.0	0.0	6.2	3.7	1.2	-1.4	-3.9	-6.5
$M_{u,w}$ (ft-k)	0	0	0	0	0	-1	-6	-15	-24	-32	-37
$V_{u,w}$ (kips)	0	0	0	0	0	-3	-8	-10	-10	-8	-3
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	0	-1	-1	-3	-4	-6	-8	-10	-13	-16
$V_{u,f}$ (kips)	0	0	-1	-1	-1	-2	-2	-2	-3	-3	-4
q_u (ksf)	-3.2	-2.1	-1.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	2	6	12	18	23	29	35	41	47	53
$V_{u,q}$ (kips)	0	4	6	7	7	7	7	7	7	7	7
ΣM_u (ft-k)	**0**	**2**	**5**	**10**	**15**	**19**	**17**	**13**	**7**	**2**	**0**
ΣV_u (kips)	**0**	**3**	**5**	**5**	**5**	**1**	**-4**	**-6**	**-6**	**-4**	**0**





Location	$M_{u,max}$		d (in)	ρ_{reqD}	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\ \phi\ b\ d\ (f_c')^{0.5}$	
Top Longitudinal	0	ft-k	14.69	0.0000	0.0000	6	kips	22	kips
Bottom Longitudinal	19	ft-k	14.69	0.0018	0.0036	6	kips	22	kips
Bottom Transverse	0	ft-k / ft	14.13	0.0000	0.0000	0	kips / ft	14	kips / ft

Where $$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

ρ_{min} = 0.0018

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0129

[Satisfactory]

COLUMNS AND FOOTINGS: Grid A.5/1.54

Allowable DL + LL Soil Pressure = 1500 psf
Minimum Footing Depth = 18 in

Roof DL = 0 psf   Roof LL = 0 psf
Floor DL = 10.7 psf   Floor LL = 40 psf
Wall DL = 10 psf

Column at A.5/3 (Interior Column)

Roof	L_{1A} = 0.0 ft	L_{2A} = 0.0 ft	$A = L_{1A} * L_{2A}$ = 0.0 ft^2	LL = 0 psf
Floor	L_{1B} = 10.0 ft	L_{2B} = 12.0 ft	$A = L_{1B} * L_{2B}$ = 120.0 ft^2	LL = 40 psf
Wall	h = 0.0 ft	L = 0.0 ft	A = h x L = 0.0 ft^2	

Item	Area	Unit DL	+	Unit LL	Dead Load	Live Load	Earthquake Load	Total Load
Roof	0	0		0	0	0		0
Floor	120	10.7		40	440	800		1,240
Wall	0	10			0			0
Earthquake								1,146
					440	800		2,386

$A_{FTNG.} = \frac{2386}{1500}$ = 1.591 ft^2

2.250 ft^2

Use
1.500 x 1.500 x 18 in **Deep Footing**

Therefore, provide 2'x2'x18" deep footing
is adequated to support the new column load.