Harriott Valentine Engineers Inc.

STRUCTURAL CALCULATIONS

Project:
Tangled Ride Resdence
6025 77th Ave SE
Mecer Island WA 98040

Architect:
Stuart Silk Architects
2400 N 45th St
Seattle WA 98103

Structural Engineer:
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2/9/18

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SECTION 1: GENERAL

CRITERIA

Tangled Ride
6025 77th Ave SE
Mercer Island WA 98040

Gravity

Roof (Vaulted)	dead	Tesla 3/8" tiles	7.3	Snow	25 Psf
		1/2" plywood	1.5		
		2x12 @ 24"oc	2.2		
		R38 insulation	1.4		
		5/8" gyp. wallboard	2.8		
		Slope Factor	4.6		
		miscellaneous	3.2 14%		
			23.0 psf		
Roof	dead	Tesla 3/8" tiles	7.3	Snow	25 Psf
		1/2" plywood	1.5		
		2x12 @ 24"oc	2.2		
		Slope Factor	3.3		
		miscellaneous	2.7 16%		
			17.0 psf		
Ceiling	dead	2x6 @ 24"oc	1.1		
		R38 insulation	1.4		
		5/8" gyp. wallboard	2.8		
		miscellaneous	0.7 12%		
			6.0 psf		

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Second Floor	dead	3/4" hardwood	3.0	Residential	40 Psf
		3/4" plywood	2.3		
		14" TJI 560 @ 16" oc	3.2		
		Batton	1.0		
		5/8" gyp. wallboard	2.8		
		miscellaneous	2.7 18%		
			15.0 psf		
Main Floor	dead	3/4" hardwood	3.0	Residential	40 Psf
		3/4" plywood	2.3		
		14" TJI 560 @ 16" oc	3.2		
		Batton	1.0		
		5/8" gyp. wallboard	2.8		
		miscellaneous	2.7 18%		
			15.0 psf		
Entry Area	dead	1" Tile	14.6	Residential	40 Psf
		3/4" plywood	2.3		
		LVL's @ 16" oc	5.3		
		Batton	1.0		
		5/8" gyp. wallboard	2.8		
		miscellaneous	3.0 10%		
			29.0 psf		
Walls	dead	3/8" Cedar Shakes	0.6		
		1/2" plywood	1.5		
		2x6 @ 16"oc	1.7		
		R21 insulation	0.8		
		5/8" gyp. wallboard	2.8		
		miscellaneous	1.6 18%		
			9.0 psf		

Lateral

wind	wind importance factor	1.0		
	basic wind speed	110	mph	
	wind exposure	D		
	topographical factor (Kzt)	1.00		
seismic	seismic importance factor	1.0		
	latitude	47.548	°	
	longitude	-122.235	°	
	mapped spectral response accel. at short periods (Ss)	1.463	g	(from USGS)
	seismic design category	D		
	response modification factoı	6.5		(plywood shearwalls)
	response modification factoı	3.5		(ordinary steel moment frame)

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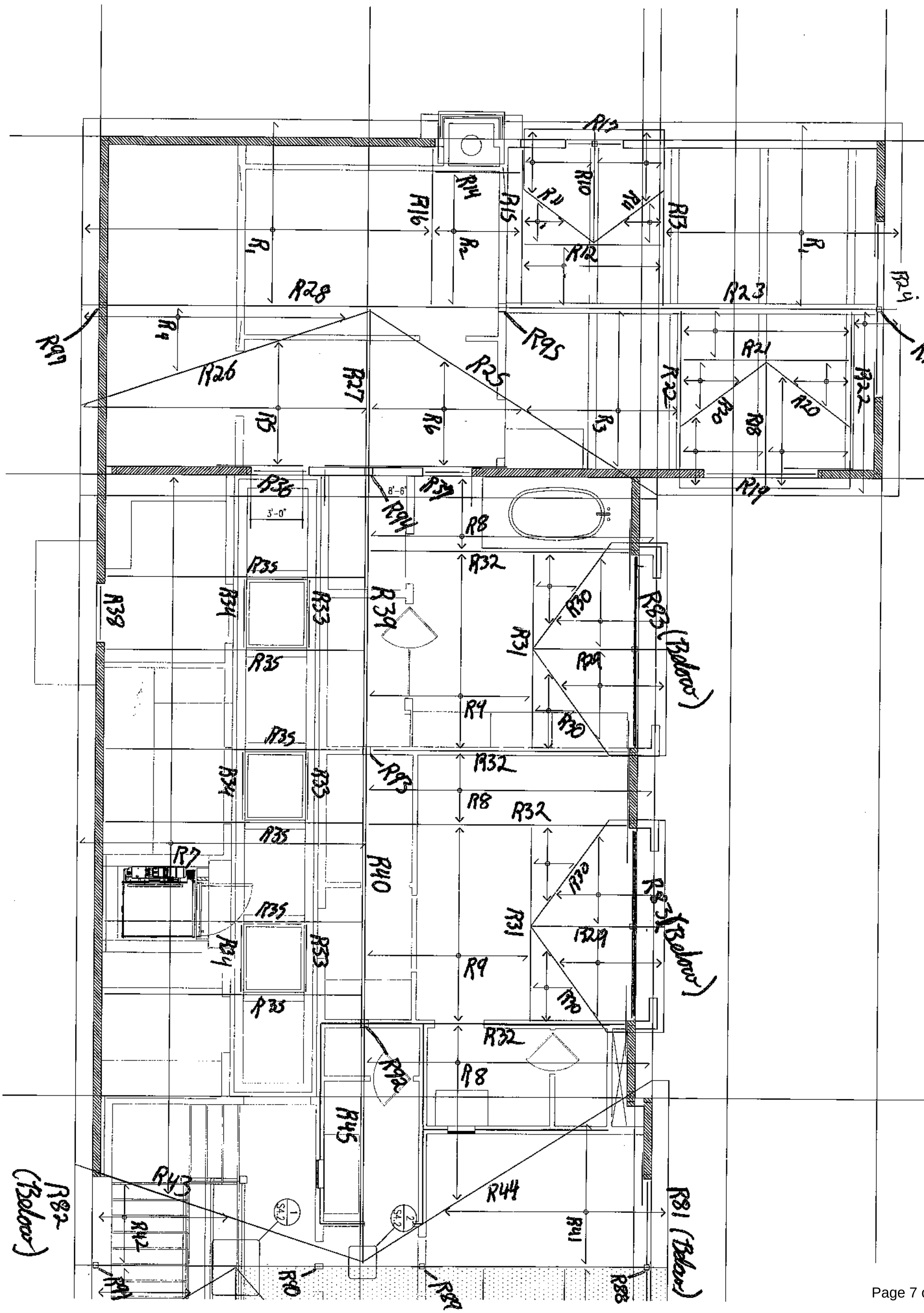
SECTION 2: FRAMING

Roof Framing Plan (South)



Roof plywood | Grade = $\frac{1}{2}''$ - span rating = 24/0

Uniform 3-Span loads

Bending

$$wb = \frac{(120)(F_b S)}{L_1^2}$$

$$wb = \frac{(120)(250)}{24^2}$$

$wb = 52$ psf

Shear

$$wb = \frac{(20)(F_s Ib/Q)}{L_2}$$

$$wb = \frac{(20)(165)}{20.5}$$

$wb = 160$ psf

Deflection

$$wb = \frac{(1743)(EI)(\Delta_{max})}{L_3^4}$$

$$\Delta_{max} = \frac{(24)}{240} = .1''$$

$$wb = \frac{(1743)(60,000)(.1)}{20.75^4}$$

$wb = 56.$ psf

Max psf for 3 span spaced @ 24" oc = 52 psf

Loads = 23D + 25S = 48 psf

Plywood pass

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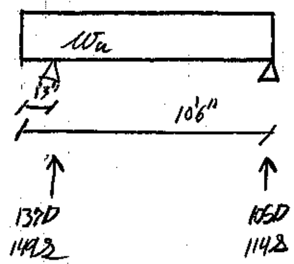
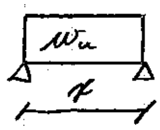
R1 $w_u = \frac{23D}{25S}$



D+S @ 24" oc
V = .45k
M = .99 k-ft
Δ_L = 7.87/EI
Δ_T = 15.12/EI

2X12 @ 24" oc
V_r = 1.69k
M_r = 2.24 k-ft
Δ_L = .034" = L/3263
Δ_T = .065" = L/1698

R2 $w_u = \frac{23D}{25S}$ Max L = 15'6"



D+S @ 24" oc
V = .74k
M = 2.884 k-ft
Δ_L = 64.9/EI
Δ_T = 124.6/EI

2X12 @ 24" oc · Cd = 1.15 · Cr
V_r = 1.94k
M_r = 2.96 k-ft
Δ_L = .28" = L/662
Δ_T = .538" = L/345

true span = 7'10½" ≤ 15'6"

Ra = Rb = 91D, 99S

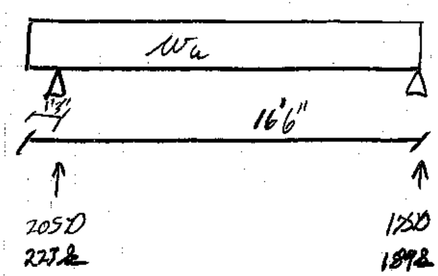
R3 $w_u = \frac{23D}{25S}$ span = 9'7" ≤ 15'6" — 2X12 @ 24" oc pass by R2 calculations.

Ra = Rb = 111D, 120S

R4 $w_u = \frac{23D}{25S}$ span = 5'8" ≤ 15'6" — 2X12 @ 24" oc pass by R2 calculation

Ra = Rb = 66D, 71S

R5 $w_u = \frac{23D}{25S}$ span = 15'2" ≤ 15'6" — 2X12 @ 24" oc pass by R2 calculation Ra = Rb = 175D, 190S



D+S @ 24" oc
V = .74k
M = 2.75 k-ft
Δ_L = 59.86/EI
Δ_T = 114.9/EI

2X12 @ 24" oc · Cr · Cd = 1.15
V_r = 1.94k
M_r = 2.96 k-ft
Δ_L = .258" = L/707
Δ_T = .496" = L/368

R6 $w_u = \frac{23D}{25S}$ span = 14' ≤ 15'6" — 2X12 @ 24" oc pass by R2 calculation Ra = Rb = 161D, 175S

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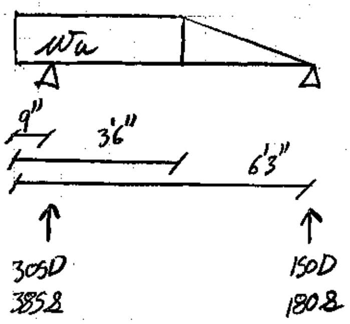
R7 Same conditions as R5 – 2X12 @ 24" oc

R8 Same conditions as R7 & R5 – 2X12 @ 24" oc

R9 $w_a = \frac{23D}{25S}$ span = 9'6" ≤ 15'6" – 2X12 @ 24" oc pass by R2 calculations

$R_a = R_b = \frac{110D}{119S}$

R10 $w_a = \frac{92D}{116S}$



D+S

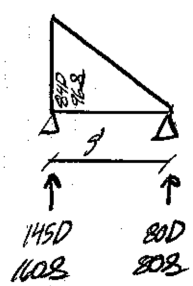
$V = .54k$

$M = .63 k \cdot ft$

$\Delta_T = 3.34/EI$

(2)2X12 pass by inspection

R11



D+S

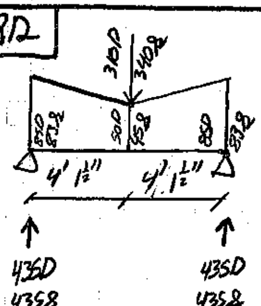
$V = .31 k$

$M = .3 k \cdot ft$

$\Delta_T = 1.31/EI$

(2)2X12 pass by inspection

R12



D+S

$V = .87k$

$M = 2.36 k \cdot ft$

$\Delta_\Delta = 12.98/EI$

$\Delta_T = 25.78/EI$

(2)2X12

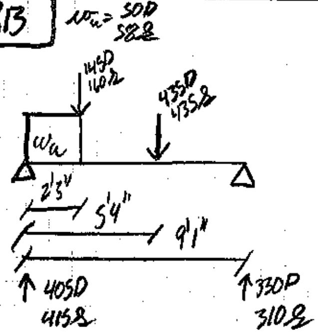
$V_r = 3.38k$

$M_r = 4.48 k \cdot ft$

$\Delta_S = .028" = L/3529$

$\Delta_T = .055" = L/1777$

R13 $w_a = \frac{50D}{58S}$



D+S

$V = .82k$

$M = 2.37 k \cdot ft$

$\Delta_T = 31.21/EI$

(2)2X12

$V_r = 3.38k$

$M_r = 4.48 k \cdot ft$

$\Delta_T = .067" = L/1616$

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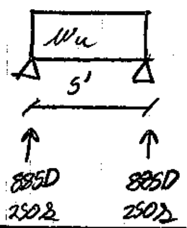
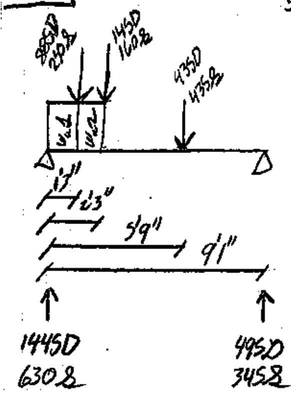
R14 $w_u = \frac{353D}{99S}$ ← chimney Brick



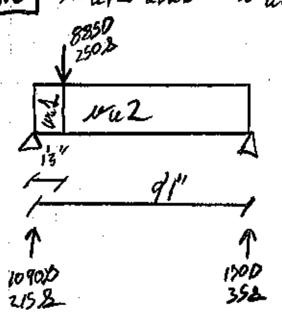
D+S	(2)2X12
$V = 1.13k$	$V_a = 3.38k$
$M = 1.41$ k.ft	$M_a = 4.48$ k.ft
$\Delta_T = 6.35/EI$	$\Delta_T = .014'' = L/4368$

R15 $w_u1 = \frac{306D}{58S}$ $w_u2 = \frac{50D}{58S}$



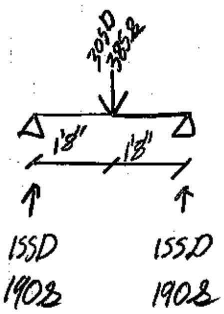
D+S	(2)2X12
$V = 2.05k$	$V_a = 3.38k$
$M = 2.83$ k.ft	$M_a = 4.48$ k.ft
$\Delta_T = 44.06/EI$	$\Delta_T = .095'' = L/1144$

R16 $w_{u1} = 262D$ $w_{u2} = 6D$



(2)2X12 pass by R15 calculations (Less Load)

R17



(2)2x8 pass by inspection

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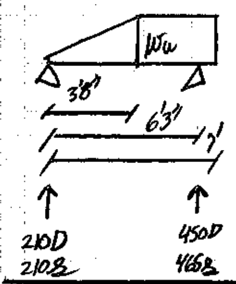
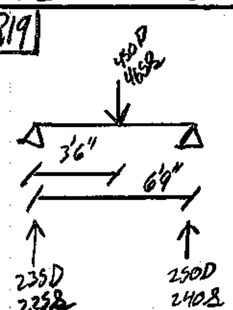
R18 $w_u = $ 126D / 130S



D+S
$V = .72\,k$
$M = .95\,k\cdot ft$
$\Delta_T = 6.45/EI$

(2)2x12
$V_r = 3.38\,k$
$M_r = 4.48\,k\cdot ft$
$\Delta_T = .0139'' = L/5381$

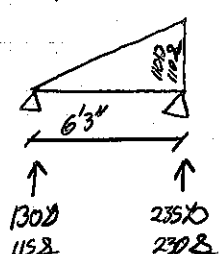
R19



D+S
$V = .49\,k\cdot ft$
$M = 1.57\,k\cdot ft$
$\Delta_T = 103/EI$

(2)2x8
$V_r = 2.18\,k$
$M_r = 2.24\,k\cdot ft$
$\Delta_T = .083'' = L/953$

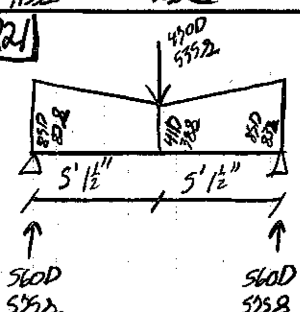
R20



D+S
$V = .47\,k$
$M = .56\,k\cdot ft$
$\Delta_T = 3.88/EI$

(2)2x12 pass by inspection

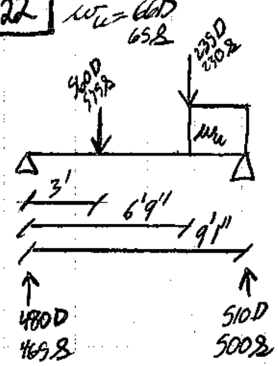
R21



D+S
$V = 1.14\,k$
$M = 4.0\,k\cdot ft$
$\Delta_\Delta = 34.2/EI$
$\Delta_T = 66.5/EI$

(2)2x12
$V_r = 3.38\,k$
$M_r = 4.48\,k\cdot ft$
$\Delta_\Delta = .073'' = L/1664$
$\Delta_T = .143'' = L/856$

R22 $w_u = $ 66D / 65S



D+S
$V = .93\,k$
$M = 2.75\,k\cdot ft$
$\Delta_\Delta = 19.09/EI$
$\Delta_T = 37.1/EI$

(2)2x12
$V_r = 3.38\,k$
$M_r = 4.48\,k\cdot ft$
$\Delta_T = .08'' = L/1359$

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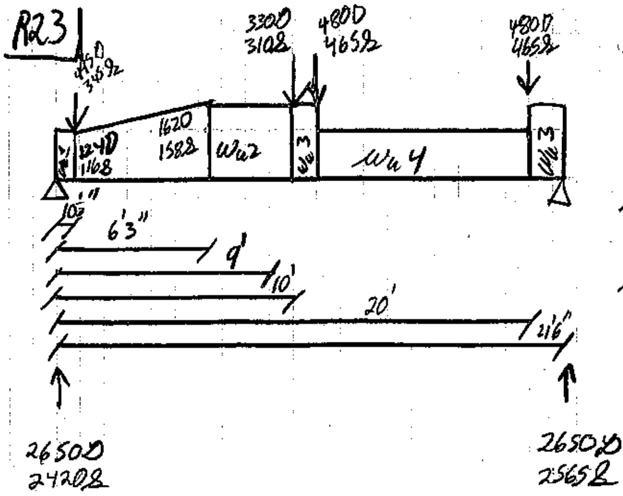
R23



w_u1 = 172D 169L

w_u2 = 162D 158L

w_u3 = 227D 228L

w_u4 = 157D 152L

D+L

V = 5.22 K

M = 27.71 K-ft

Δ_L = 1077/EI

Δ_T = 2240/EI

Glulam 5 1/8 X 15 • (C_v = .98)

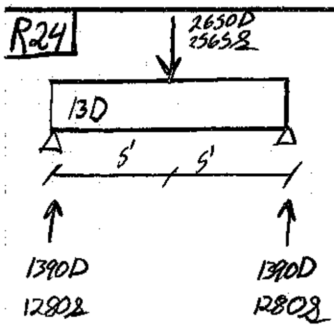
V_r = 13.58K

M_r = 37.5 K-ft

Δ_L = .415" = L/622

Δ_T = .863" = L/299

R24



D+L

V = 2.67K

M = 13.2 K-ft

Δ_T = 1906 /EI

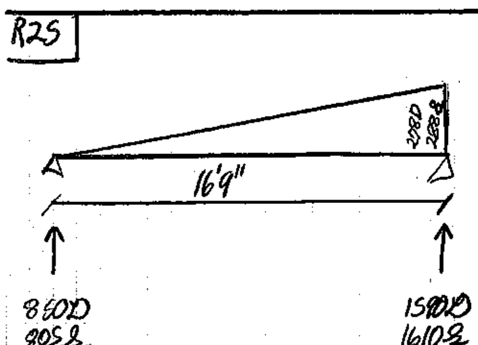
3 1/2 X 11 1/4 PSL

V_r = 7.6K

M_r = 17.97 K-ft

Δ_T = .229" = L/522

R25



D+L

V = 3.2K

M = 10.4 K-ft

Δ_L = 255.4/EI

Δ_T = 513.5/EI

3 1/2 X 11 1/4 LSL

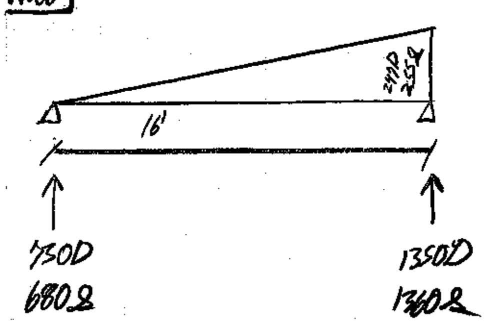
V_r = 8.14K

M_r = 14.39 K-ft

Δ_L = .397" = L/506

Δ_T = .798" = L/251

R26



D+L

V = 2.71K

M = 8.44 K-ft

Δ_L = 188.3/EI

Δ_T = 380.3/EI

3 1/2 X 11 1/4 LSL

V_r = 8.14K

M_r = 14.39 K-ft

Δ_L = .292" = L/655

Δ_T = .591" = L/324

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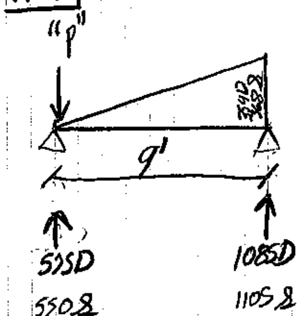
R27



D+S

$V = 2.2$ or (4.23K) "P"

$M = 3.82$ K-ft

$\Delta_D = 27.2/EI$

$\Delta_T = 54.5/EI$

3½x11 7/8 PSL

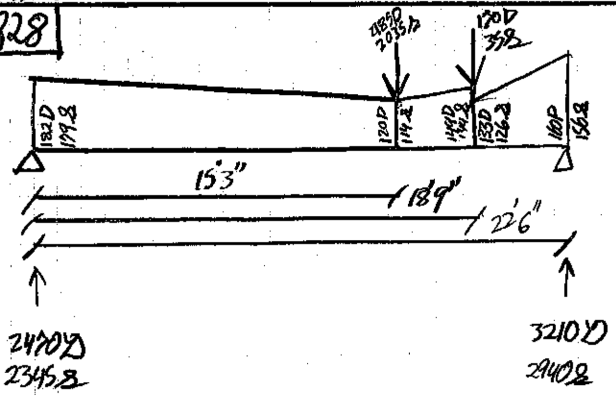
$V_r = 8.03$ K

$M_r = 19.9$ K-ft

$\Delta_T = .0558'' = L/1934$

* "P" is there as a connection calculation

R28



D+S

$V = 6.15$ K

$M = 36.33$ K-ft

$\Delta_D = 1496/EI$

$\Delta_T = 3099/EI$

5⅛x16½ GLB (Cv=.96)

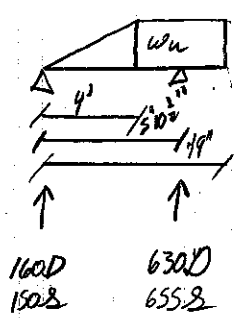
$V_r = 14.94$ K

$M_r = 44.94$ K-ft

$\Delta_D = .433'' = L/623$

$\Delta_T = .897'' = L/301$

R29 w_u = 135D 140S



D+S

$V = .77$ K

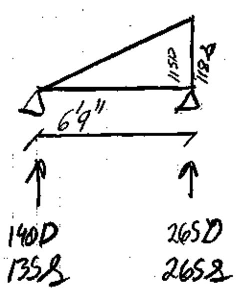
$M = .6$ K-ft

$\Delta_D = 1.7/EI$

$\Delta_T = 3.4/EI$

(2) 2x12 - by inspection

R30



D+S

$V = .53$ K

$M = .7$ K-ft

$\Delta_T = 5.6/EI$

(2) 2x12 by inspection

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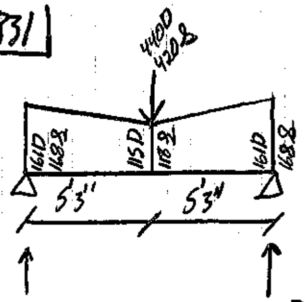
R31



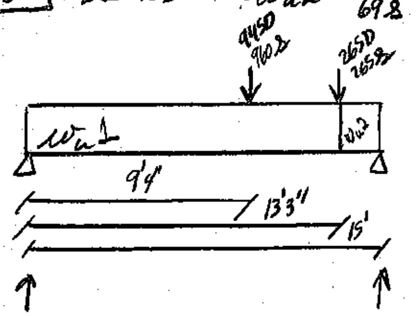
945D 960L — 945D 960L

D+L
$V = 1.91k$
$M = 5.91$ k·ft
$\Delta_L = 54.7/EI$
$\Delta_T = 109/EI$

(3)2x12
$V_r = 5.07$ k
$M_r = 6.12$ k·ft
$\Delta_L = .078" = L/1599$
$\Delta_T = .157" = L/802$

R32

$w_u1 = 10D$ $w_u2 = 73D$ 69L



420D 400L — 1000D 945L

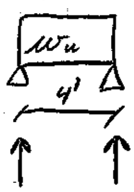
D+L
$V = 1.95k$
$M = 7.68$ k·ft
$\Delta_L = 121.5/EI$
$\Delta_T = 252.5/EI$

(3)2x12 · $C_d = 1.15$
$V_r = 5.83$ k $= 3.52$k (notched for hanger)
$M_r = 7.72$ k·ft
$\Delta_L = .135" = L/1028$
$\Delta_T = .363" = L/494$

$3\frac{1}{2}$x$11\frac{1}{4}$ LSL
$V_r = 8.14$k
$M_r = 14.39$k·ft
$\Delta_T = .392" = L/458$

R33

$w_u = 72D$ 88L

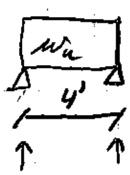


145D 175L — 145D 175L

(2)2x12 by inspection

R34

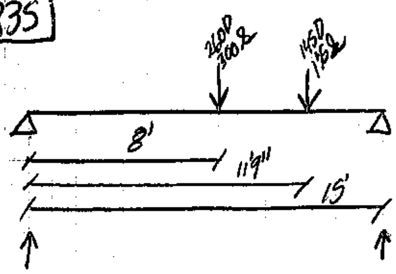
$w_u = 129D$ 150L



260D 300L — 260D 300L

(2)2x12 by inspection

R35



205D 175L — 300D 300L

D+L
$V = .6k$
$M = 2.84$ k·ft
$\Delta_L = 49.2/EI$
$\Delta_T = 99.4/EI$

(2)2x12
$V_r = 3.38$k
$M_r = 4.48$ k·ft
$\Delta_T = .214" = L/838$

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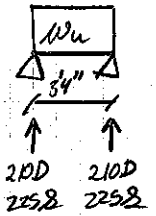
R36 & R37 w_u = 50D

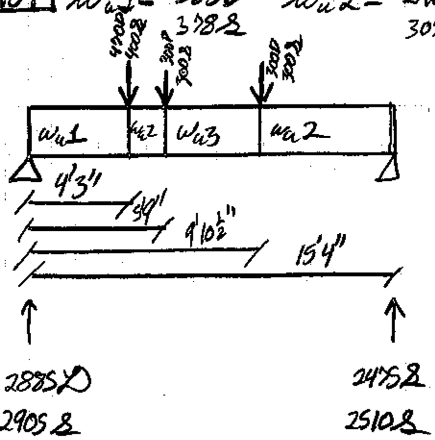
Spans = 2'10" / 3'4" → (2) 2X8 by inspection Ra = Rb = < 70D / 85D

R38 w_u = 125D
136S



(2) 2X8 by inspection

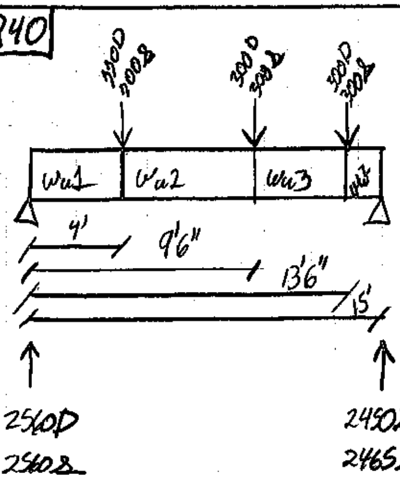
R39 w_u1 = 363D 378S w_u2 = 298D 307S w_u3 = 163D 163S



D+S
V = 5.8K
M = 19.99 k-ft
Δ_S = 431/EI
Δ_T = 862.6/EI

5¼ X 11⅞ PSL
V_r = 13.86 K
M_r = 34.33 k-ft
Δ_S = .293" = L/625
Δ_T = .588" = L/312

R40



w_u1 = 225D 231S
w_u2 = 298D 307S
w_u3 = 163D 163S

D+S
V = 5.12K
M = 19.58 k-ft
Δ_S = 396.7/EI
Δ_T = 792.7/EI

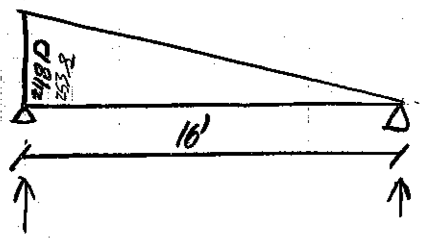
5¼ X 11⅞ PSL
V_r = 13.86K
M_r = 34.33 k-ft
Δ_S = .27" = L/665
Δ_T = .54" = L/332

R41 | $w_u = \frac{23D}{25S}$ span = 9'9" ≤ 15'6" — 2X12 @24" oc pass by R2 calculation

$R_a = R_b = \frac{112D}{122S}$

R42 | $w_u = \frac{23D}{25S}$ span = 5'3" ≤ 15'6" — 2X12@24" oc pass by R2 calculation $R_a = R_b = \frac{60D}{65S}$

R43



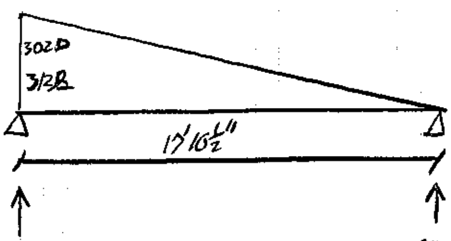
1365D
1350S

740D
695S

D+S
$V = 2.71k$
$M = 8.45k\text{-ft}$
$\Delta_L = 186.8/EI$
$\Delta_T = 381/EI$

3½X11⅞ LSL
$V_r = 8.59k$
$M_r = 19.95\ k\text{-ft}$
$\Delta_L = .246" = L/777$
$\Delta_T = .503" = L/381$

R44



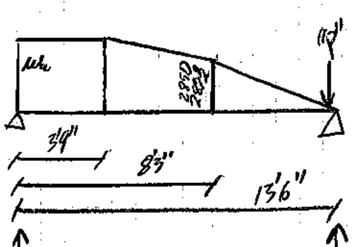
1845D
1860S

990D
930S

D+S
$V = 3.7k$
$M = 12.86k\text{-ft}$
$\Delta_L = 358.9/EI$
$\Delta_T = 723.6/EI$

3½X11⅞ PSL
$V_r = 8.03k$
$M_r = 19.9k\text{-ft}$
$\Delta_L = .367" = L/583$
$\Delta_T = .741" = L/289$

R45 | $w_u = \frac{370D}{378S}$



2225D
2260S

1435D
1410S

D+S "P"
$V = 4.48k$ or $6.18k$
$M = 13.52k\text{-ft}$
$\Delta_L = 218.5/EI$
$\Delta_T = 435.6/EI$

5¼X11⅞ PSL
$V_r = 12.05k$
$M_r = 29.85k\text{-ft}$
$\Delta_L = .149" = L/1086$
$\Delta_T = .297" = L/545$

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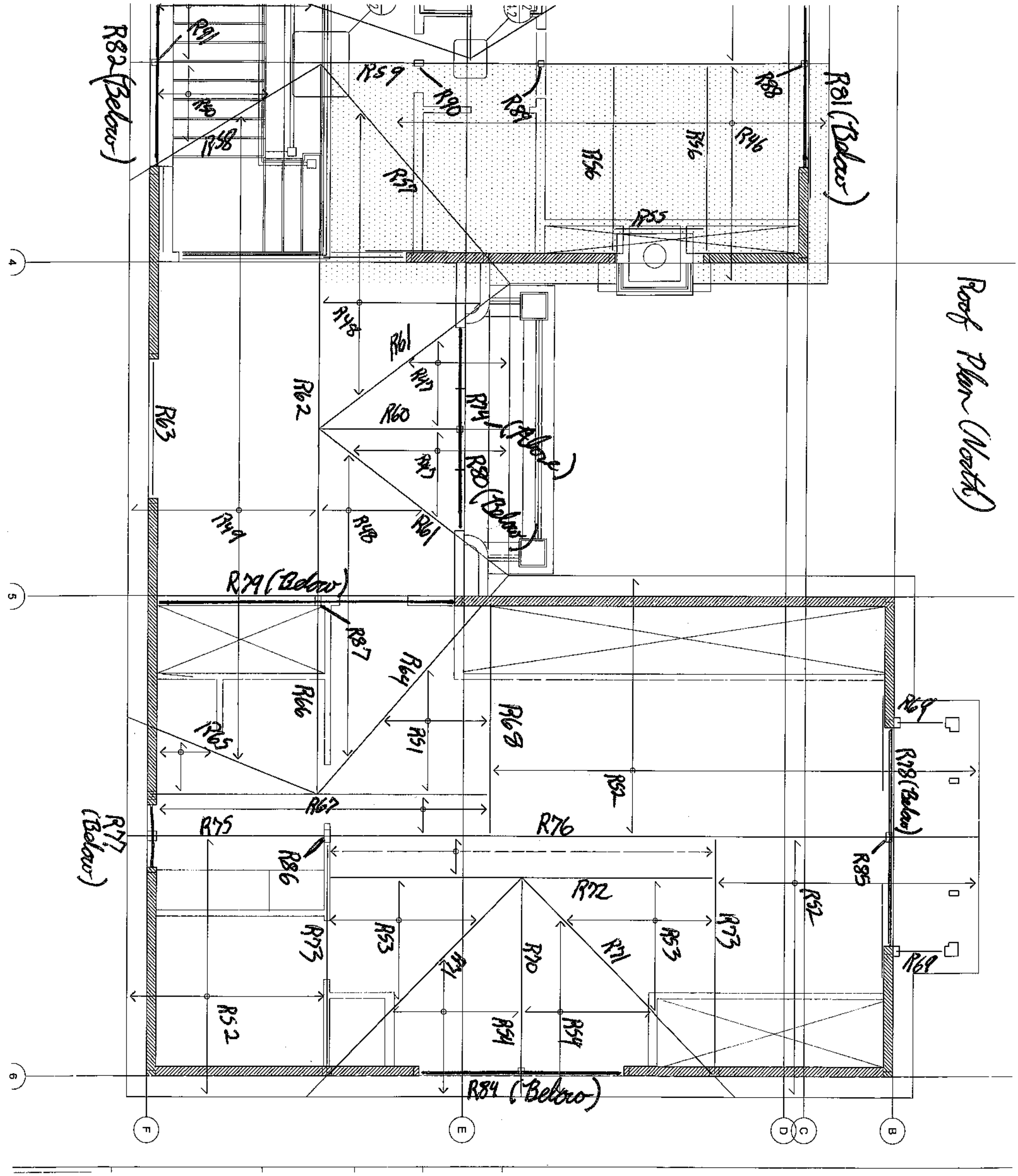
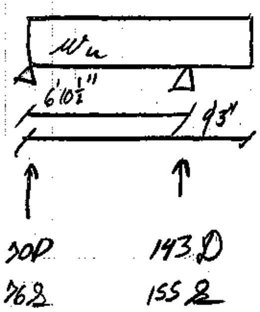

Roof Plan (North)
R82 (Below)
R81 (Below)
R91
R90
R89
R88
R59
R58
R57
R56
R46
R55
R48
R61
R62
R63
R60
R74 (Above)
R80 (Below)
R49
R79 (Below)
R87
R64
R66
R65
R68
R69
R67
R75
R76
R77 (Below)
R78 (Below)
R86
R72
R85
R73
R70
R71
R53
R52
R54
R84 (Below)
4
5
6
F
E
D
C
B

R46 Same conditions as R1 2X12 @ 24" oc

R47 $w_u = {}^{23D}_{25S}$ span = 7'3" ≤ 16'6" 2X12 @ 24" oc pass by R2 calculation $R_a = R_b = {}^{84D}_{91S}$

R48

Case 1
$w_u = {}^{23D}_{25S}$

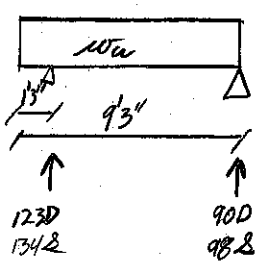


D+S @ 24" oc
$V = .37k$
$M = .44$ k-ft
$\Delta_T = 3.45/EI$

2X12 @ 24" oc
$V_r = 1.69k$
$M_r = 2.24$ k-ft
$\Delta_T = .0149" = L/5533$

Case 2 $w_u = {}^{23D}_{25S}$ span = 8' ≤ 15'6"
2X12 @ 24" oc pass by R2 calculations, $R_a = R_b = {}^{92D}_{100S}$

R49 $w_u = {}^{23D}_{25S}$



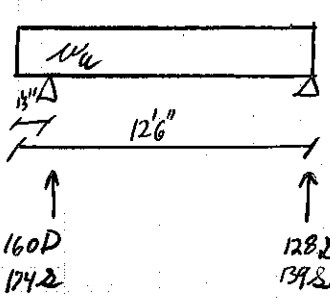
D+S @ 24" oc
$V = .4k$
$M = .73$ k-ft
$\Delta_T = 8.33/EI$

2X12 @ 24" oc
$V_r = 1.69k$
$M_r = 2.24$ k-ft
$\Delta_T = .036" = L/2666$

R50 $w_u = {}^{23D}_{25S}$ span = 5' ≤ 15'6" – 2X12 @ 24" oc pass by R2 calculations
$R_a = R_b = {}^{58D}_{63S}$

R51 $w_u = {}^{23D}_{25S}$ span = 9' ≤ 15'6" – 2X12 @ 24" oc pass by R2 calculations
$R_a = R_b = {}^{104D}_{112S}$

R52 $w_u = {}^{23D}_{25S}$



D+S @ 24" oc
$V = .55k$
$M = 1.48$ k-ft
$\Delta_S = 17.48/EI$
$\Delta_T = 37.6/EI$

2X12 @ 24" oc
$V_r = 1.69k$
$M_r = 2.24$ k-ft
$\Delta_S = .0755" = L/1987$
$\Delta_T = .145" = L/929$

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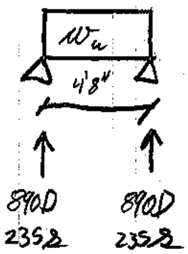
RS3 $w_u = \frac{23D}{25S}$ span = 9' ≤ 15'6" 2X12 @ 24" oc pass by R2 calculation $R_a = R_b = \frac{104D}{112S}$

RS4 $w_u = \frac{23D}{25S}$ span = 9'6" ≤ 15'6" 2X12 @ 24" oc pass by R2 calculation $R_a = R_b = \frac{109D}{119S}$

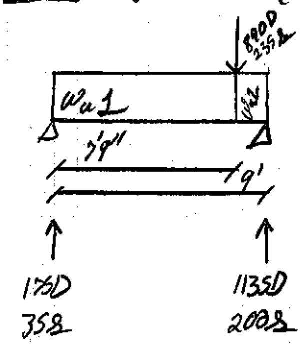
RS5 $w_u = \frac{382D}{100S}$ ← including chimney



D+S
$V = 1.12K$
$M = 1.32$ K·ft
$\Delta_T = 5.14/EI$

(2) 2X12
$V_r = 3.38K$
$M_r = 4.48$ K·ft
$\Delta_T = .011" = L/5038$

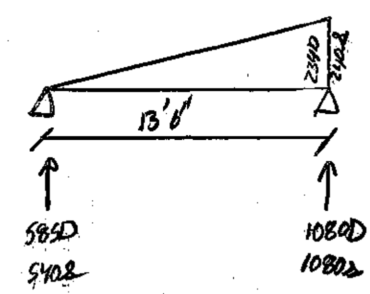
RS6 $w_u 1 = 6D$ $w_u 2 = 300D$



D+S
$V = 1.34K$
$M = 1.44$ K·ft
$\Delta_T = 15.1/EI$

(2) 2X12
$V_r = 3.38K$
$M_r = 4.48$ K·ft
$\Delta_T = .032" = L/3310$

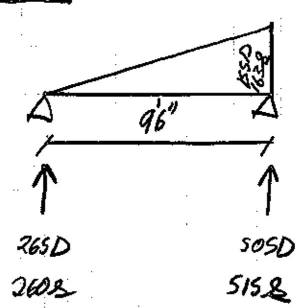
RS7



D+S
$V = 2.16$ K
$M = 5.68$ K·ft
$\Delta_D = 89.8/EI$
$\Delta_T = 182.2/EI$

$3\frac{1}{2}$X $11\frac{1}{4}$ LSL
$V_r = 8.14$ K
$M_r = 14.39$ K·ft
$\Delta_D = .139" = L/1160$
$\Delta_T = .283" = L/571$

RS8



D+S
$V = 1.1K$
$M = 1.88$ K·ft
$\Delta_T = 29.8/EI$

(2) 2X12
$V_r = 3.38K$
$M_r = 4.48$ K·ft
$\Delta_T = .064" = L/1770$

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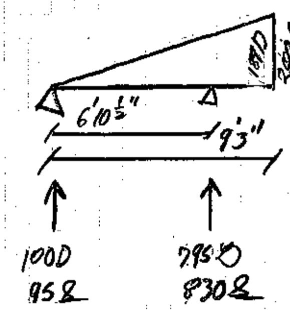
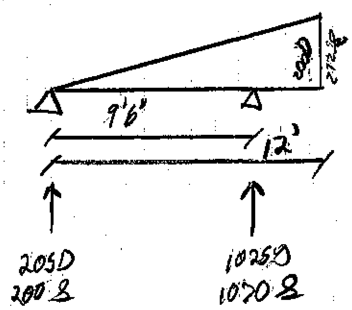
R60



D+S
V = .82 k
M = 1.0 k-ft
Δ_T = 3.2/EI

(2)2X12 pass by inspection

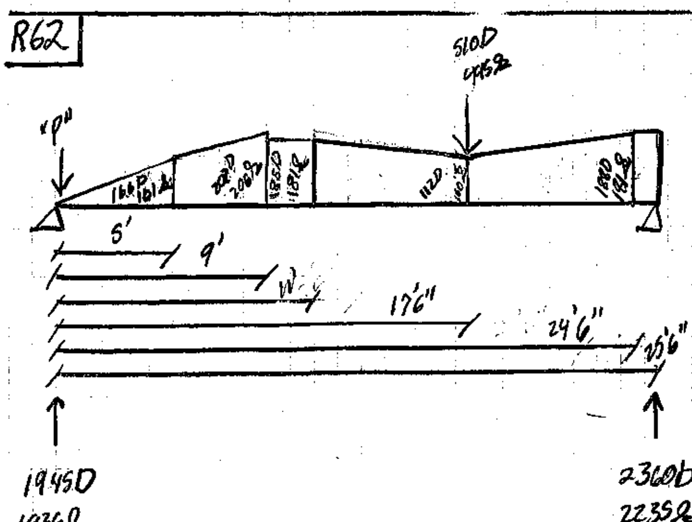
R61



D+S
V = 1.17 k
M = 1.27 k-ft
Δ_T = 14.7 /EI

(2)2X12
V_r = 3.38 k
M_r = 4.18 k-ft
Δ_T = .0404 = L/2821

R62

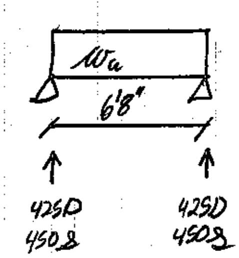


D+S "P"
V = 4.6 k or 5.16 k
M = 29.1 k-ft
Δ_s = 1625/EI
Δ_T = 3400/EI

5⅛X16½ GL 73 (C_v = .95)
V_r = 14.94 k
M_r = 44.19 k-ft
Δ_s = .471" = L/650
Δ_T = .985" = L/311

* "P" is for shear-connection design only

R63 w_u = 127D / 134S



D+S
V = .87 k
M = 1.45
Δ_T = 11.6/EI

(2)2X8
V_r = 2.18 k
M_r = 2.24 k-ft
Δ_T = .094" = L/853

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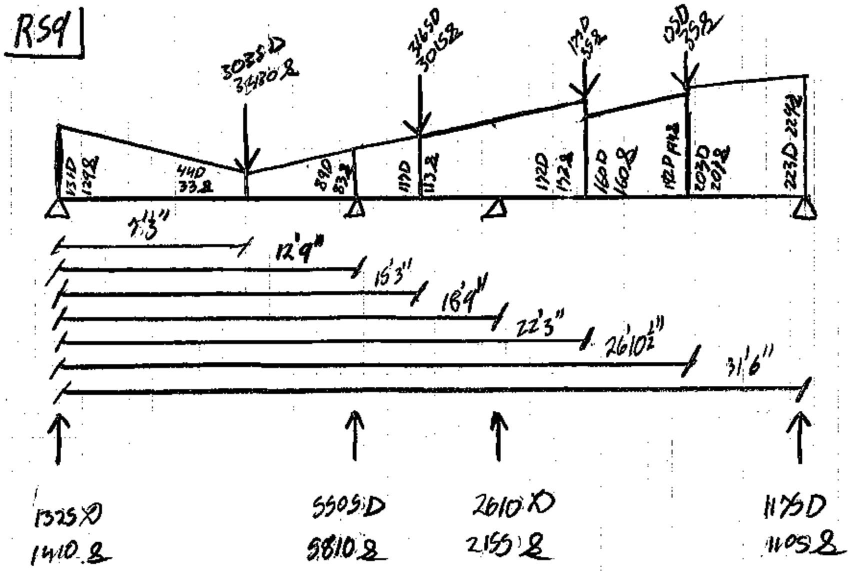
RS9



D+L

$V = 5.7^k$

$M = 14.6\ k\cdot ft$

$\Delta_L =$ [illegible] $/EI$

$\Delta_T = 297.8/EI$

$3\frac{1}{2} \times 11\frac{7}{8}$ LSL

$V_r = 8.59k$

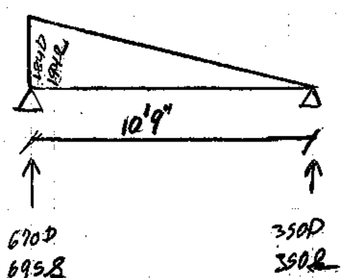
$M_r = 15.95\ k\cdot ft$

$\Delta_T = .393'' = L/388$

~~Bearing @ 900 - 4x4~~

Bearing @ 625 - 4x6

R64



D+L

$V = 1.36k$

$M = 2.84\ k\cdot ft$

$\Delta_\Delta =$ [illegible] $/EI$

$\Delta_T = 57.6/EI$

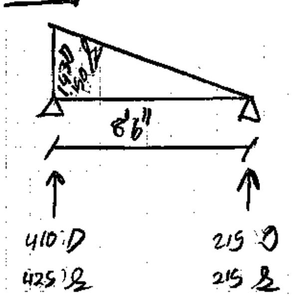
(2) 2x12

$V_r = 3.38k$

$M_r = 4.48\ k\cdot ft$

$\Delta_T = .124'' = L/1036$

R65



D+L

$V = .84\ k$

$M = 1.38\ k\cdot ft$

$\Delta_T = 17.5/EI$

(2) 2x12

$V_2 = 3.38k$

$M_2 = 4.48\ k\cdot ft$

$\Delta_T = .037'' = L/2697$

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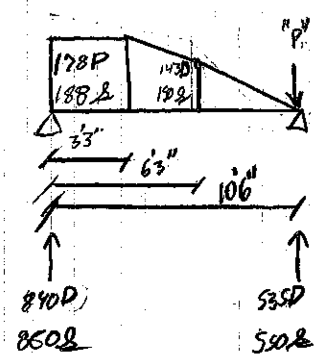
R66



D+L "P"

$V = 1.72K$ or $2.22K$

$M = 4.06$ K-ft

$\Delta_L = 40.46/EI$

$\Delta_T = 79.06/EI$

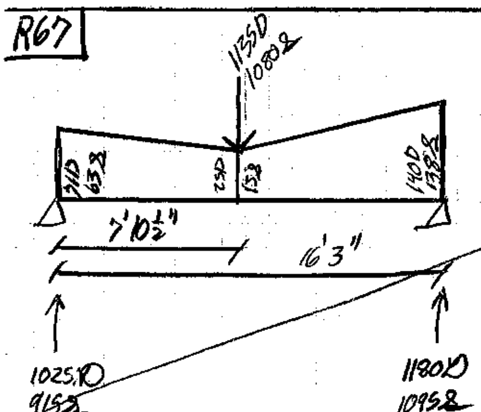
(2)2X12

$V_2 = 3.38$ K

$M_2 = 4.48$ K-ft

$\Delta_T = .071" = L/737$

R67



D+L

$V = 2.3K$

$M = 12.11$ K-ft

$\Delta_L = 237.2/EI$

$\Delta_T = 497.71/EI$

3½X 11⅞ LSL

$V_2 = 8.59K$

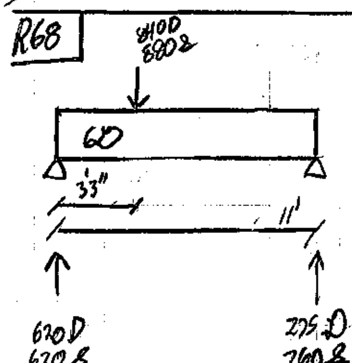
$M_2 = 15.95$ K-ft

$\Delta_L = .313" = L/621$

$\Delta_T = .657" = L/296$

$.773" = L/252$ (11¼ LSL)

R68



D+L

$V = 1.24$ K

$M = 4.0$ K-ft

$\Delta_T = 67.0/EI$

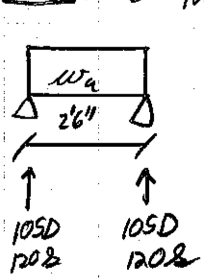
(2)2X12

$V_2 = 3.38$ K

$M_2 = 4.48$ K-ft

$\Delta_T = .144" = L/911$

R69 $w_u = \frac{84D}{96L}$



(2)2X6 pass by inspection

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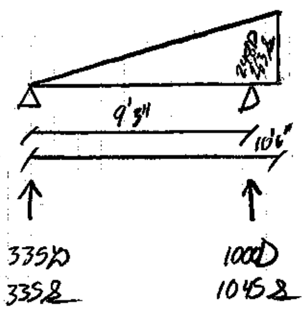
R70

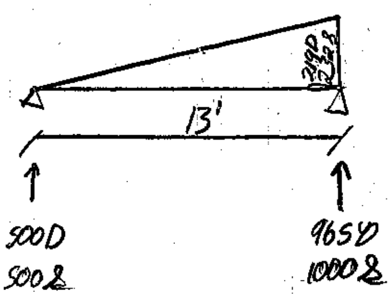


D+S
V = 1.44k
M = 2.37k·ft
Δ_S = 17.3/EI
Δ_T = 34.1/EI

(2) 2x12
V_r = 3.38 k
M_r = 4.48 k·ft
Δ_T = .073" = L/1506

→ 4x12 fit
M.8C4

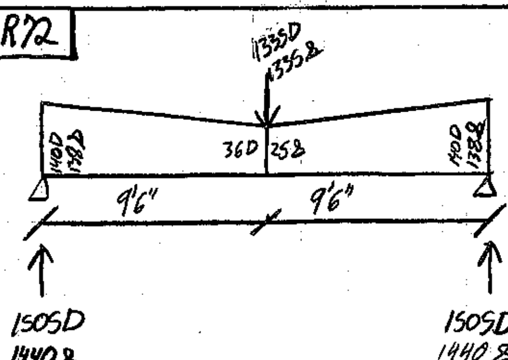
R71



D+S
V = 1.97 k
M = 4.95 k·ft
Δ_S = 74.6/EI
Δ_T = 147.1/EI

(2) 2x12 · Cd = 1.15
V_r = 3.88 k
M_r = 5.15 k·ft
Δ_S = .161" = L/967
Δ_T = .317" = L/490

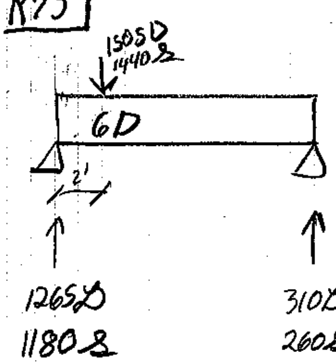
R72



D+S
V = 2.94 k
M = 18.7 k·ft
Δ_S = 522/EI
Δ_T = 1067/EI

5 1/4 x 11 1/4 PSL
V_r = 11.42 k
M_r = 26.95 k·ft
Δ_S = .418" = L/544
Δ_T = .856" = L/266

R73



D+S
V = 2.44k
M = 4.87 k·ft
Δ_T = 77/EI

(2) 2x12 · Cd = 1.15
V_r = 3.88k
M_r = 5.15 k·ft
Δ_T = .166" = L/793

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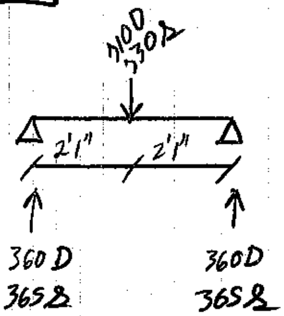
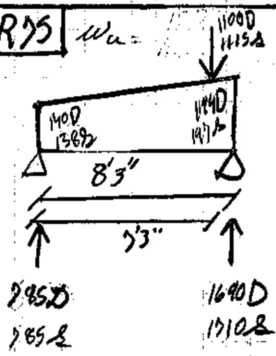
R74



D+S
V = .725K
M = 1.51 K-ft

(2) 2X8
V_2 = 2.18K
M_2 = 2.24 K-ft

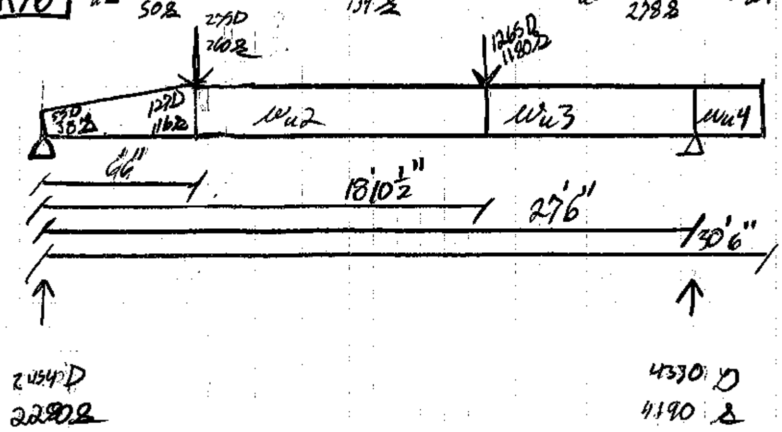
R75 w_u =



D+S
V = 3.41 K
M = 4.04 K-ft
Δ_S = 25.5 /EI
Δ_T = 50.9 /EI

(2) 2X12
V_2 = 3.38K
M_2 = 4.48 K-ft
Δ_T = .109" = L/900

R76 w_u1 = 66D, 50S w_u2 = 148D, 139S w_u3 = 276D, 278S w_u4 = 147D, 138S



Min 4X6 @ 625 psi

D+S
V = 9.7 K
M = 44.58 K-ft
Δ_S = 2905/EI
Δ_T = 5967/EI
Δ_D = 3052/EI

5 1/8 X18 GLB (C_v = .93) w/ 1/2" Camber

V_2 = 16.3K
M_2 = 53.74 K-ft
Δ_D = (.681").8 = .544"
Δ_S = .648" = L/504
Δ_T = (1.331 − .5) = .831" = L/393

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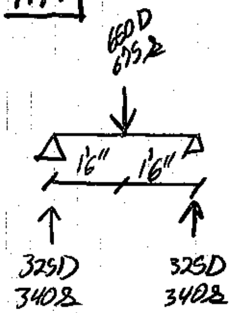
Project

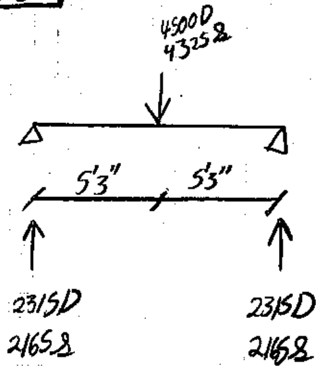
R77



D+S
V = .665 K
M = 1.0 K·ft

(2)2X8
V_a = 2.18 K
M_a = 2.24 K·ft

R78

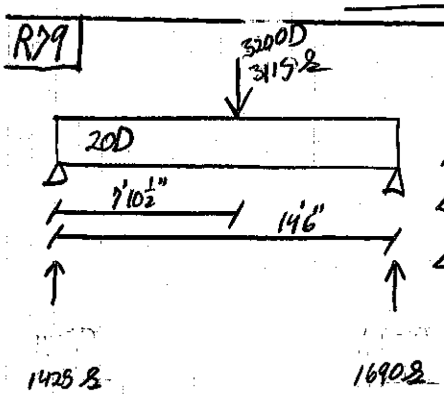


D+S
V = 4.48 K
M = 23.34 K·ft
Δ_L = 180.2/EI
Δ_T = 371.3/EI

$3\frac{1}{2}$ X 14 PSL (CL = .947)
V_a = 9.47 K
M_a = 25.72 K·ft
Δ_L = .112" = L/1118
Δ_T = .232" = L/542

Bear $3\frac{1}{4}$" min

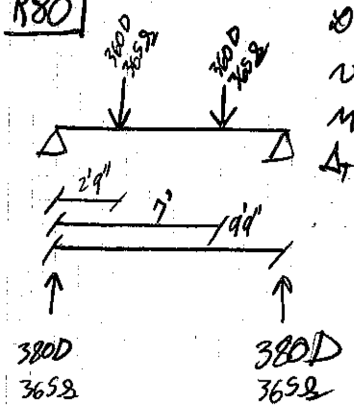
R79



D+S
V = 3.52 K
M = 22.851 K·ft
Δ_L = /EI
Δ_T = 695.2/EI

$5\frac{1}{4}$ X $11\frac{1}{4}$ PSL (CL = .983)
V_a = 11.42 K
M_a = 26.5 K·ft
Δ_T = .550" = L/311

R80



D+S
V = .75 K
M = 2.04 K·ft
Δ_T = 37.4/EI

(2)2X8
V_a = 2.18 K
M_a = 2.24 K·ft
Δ_T = .302" = L/387

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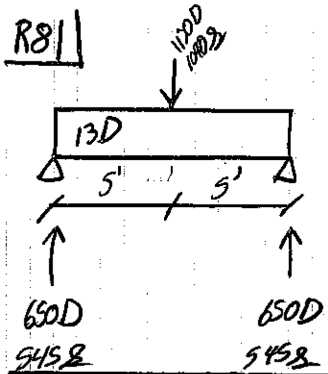
R81



D+S

$V = 1.2k$

$M = 5.81\ k\cdot ft$

$\Delta_T = 84.2/EI$

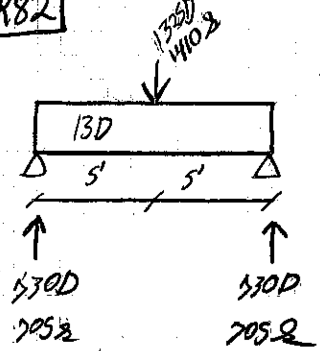
<u>3½x9¼ LSL</u>

$V_r = 6.69k$

$M_r = 9.909\ k\cdot ft$

$\Delta_T = .235'' = L/510$

R82



D+S

$V = 1.43k$

$M = 7k\cdot ft$

$\Delta_T = 101.3/EI$

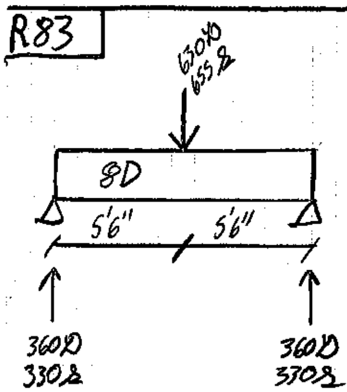
<u>3½x9¼ LSL</u>

$V_r = 6.69k$

$M_r = 9.905k\cdot ft$

$\Delta_T = .133''$

R83



D+S

$V = .69k$

$M = 3.65k\cdot ft$

$\Delta_T = 64.2/EI$

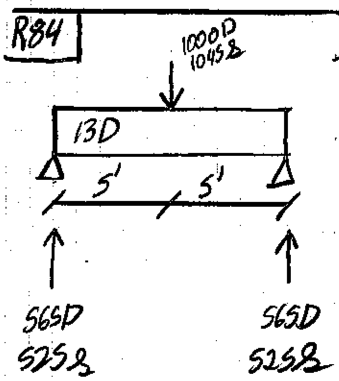
<u>(2) 1¾x7¼ LVL</u>

$V_r = 4.8k$

$M_r = 7.11k\cdot ft$

$\Delta_T = .286'' = L/460$

R84



Same conditions as R81 with Less Loads

3½x9¼ LSL pass by inspection

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R85 | P = 4490 D / 4325 S → 8.815 K 4X6 @ 625 psi ℓ ≈ 7'

Axial 4X6 = 11.33 K ✓

R86 | P = 1265 D / 1190 S + 1690 D / 1710 S + 2425 D / 2280 S | 5380 D / 5170 S → 10.55 K 6X6 @ 405 psi

ℓ = 14' Axial 6X6 DF = 12.25 (Hf sill)
13.35 (DF sill or greater)

R87 | P = 2400 D / 2260 S + 730 D / 730 S | 3130 D / 2990 S → 6.12 K ℓ = 7'

4X6 @ 625 psi pass by R85 calculation

R88 | 1170 D / 1090 S | 2.26 K 4X4 HF ℓ = 7' axial = 7.26 K @ 592 psi
4.96 K @ 405 psi

R89 | 2750 D / 2500 S → 5.25 K 625 psi T & B 4X4 pass by R88 calc

R90 | 5255 D / 4985 S → 10.24 K 4X6 @ 625 psi ✓
pass by R85 calculation

R91 | 1075 D / 1005 S → 2.08 K 4X4 pass by R88 calculation

R92 | 2225 D / 2260 S + 470 D / 400 S + 2450 D / 2465 S | 5145 D / 5125 S → 10.27 K use 6X6 DF pass by R86 calc

R93 | 2475 D / 2510 S + 2560 D / 2560 S + 300 D / 300 S + 470 D / 400 S | 5805 D / 5770 S → 11.575 K 6X6 DF pass by R86 calculation

R94 | 2885 D / 2905 S + 1085 D / 1105 S | 3970 D / 4010 S → 7.98 K 6X6 DF pass by R86 calculation

R95 | 3210 D / 2940 S + 2650 D / 2420 S | 5860 D / 5360 S → 11.22 K 6X6 DF pass by R86 calculation

R96 | 2650 D / 2565 S → 5.215 K 4X4 pass by R88

R97 | 2470 D / 2345 S → 4.815 K 4X6 Braced by SW Axial = 7.45 K (ℓ = 14')

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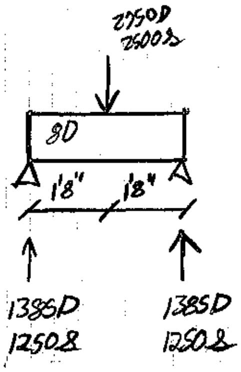
F

Ceiling Header under R89



D+S

$V = 2.63K$

$M = 4.39 K \cdot ft$

$\Delta_T = 7.02/EI$

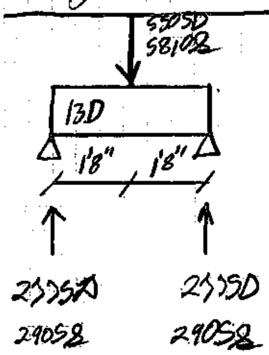
(2) $1\frac{3}{4} \times 5\frac{1}{2}$ LVL, $C_d = 1.15$, ($C_L = .978$)

$V_r = 4.2 K$

$M_r = 4.78 K \cdot ft$

$\Delta_T = .073'' = L/547$

Ceiling header under R90



D+S

$V = 5.7 K$

$M = 9.45 K \cdot ft$

$\Delta_T = 15.1/EI$

$3\frac{1}{2} \times 9\frac{1}{4}$ LSL

$V_r = 6.69K$

$M_r = 9.9 K \cdot ft$

$\Delta_T = .023'' = L/1203$

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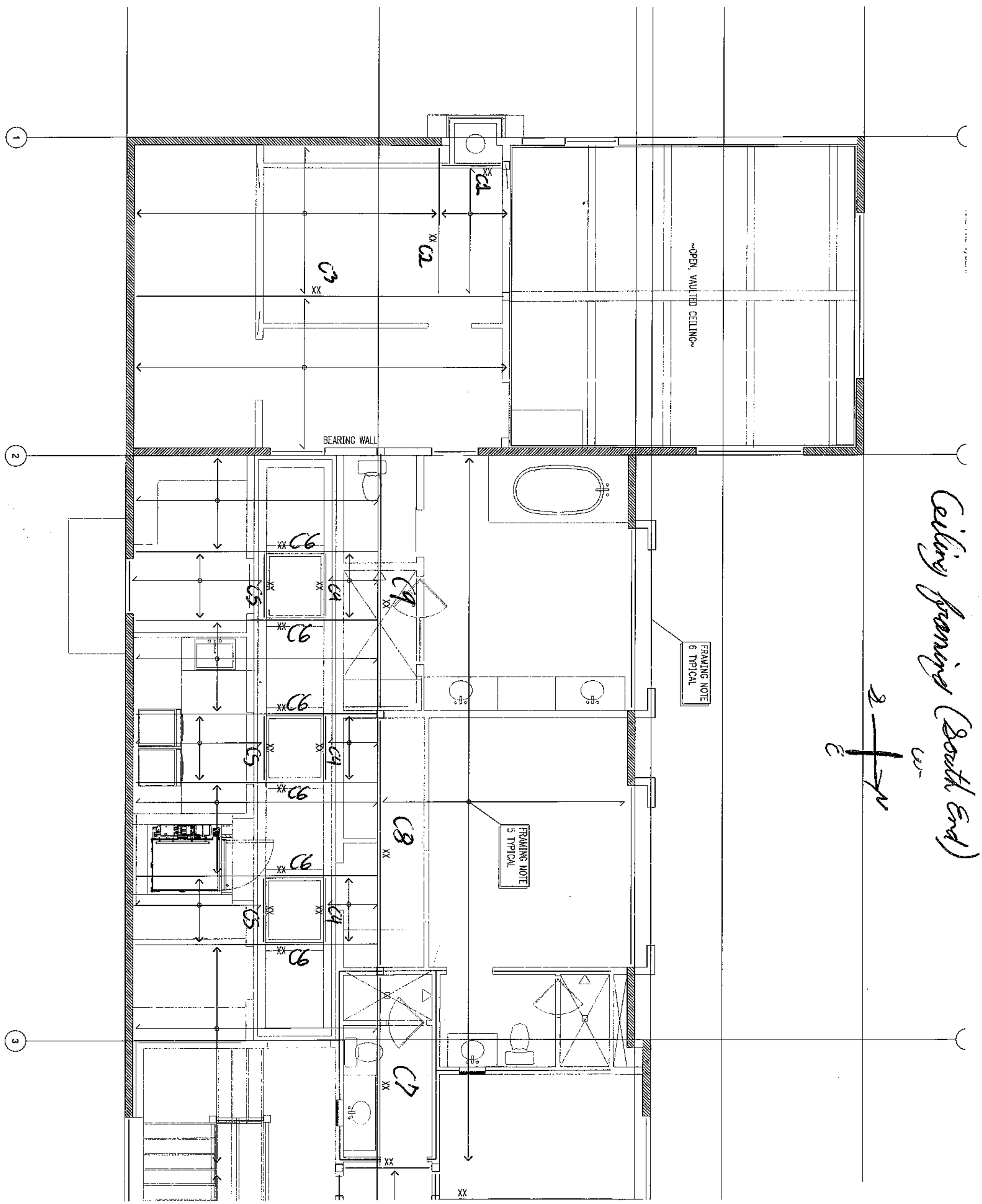
Ceiling framing (South End)
~OPEN, VAULTED CEILING~
BEARING WALL
FRAMING NOTE 6 TYPICAL
FRAMING NOTE 5 TYPICAL
C1
C2
C3
C4
C5
C6
C7
C8
C9
1
2
3

Ceiling attachment

Max Load = (16)'(2)'(½)(7) = 112 #S dead

10d nails good for 73 #S / nail (cd=.9) use 3 nails

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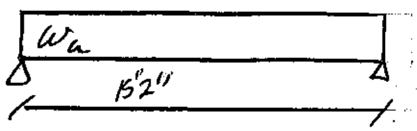
Ceiling Framing

w_u = 2X6 @16" oc 1.1
R38 insulation 1.4
5/8" GWB 2.8
WTF .7 11.6%
6.0 psf

Max span = 15'2" ≤ 16' — Max Length you can buy



D @16" oc
V = .06 k
M = .231 k-ft
Δ_T = 9.52/EI

2x6 @16" oc · Cd=.9
V_r = .747 k
M_r = .63 k-ft
Δ_T = .352" = L/515

D @ 24" oc
V = .09 k
M = .346 k-ft
Δ_T = 14.28/EI

2x6 @24" oc, Cd=.9
V_r = .747 k
M_r = .63 k-ft
Δ_T = .529" = L/343

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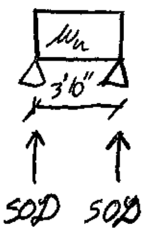
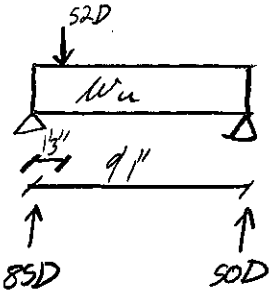
C1 | $w_u = 27D$



(2) 2X6 pass by inspection

C2 | $w_u = 9D$



D

$V = .085k$

$M = .13$ k-ft

$\Delta_T = 1.94/EI$

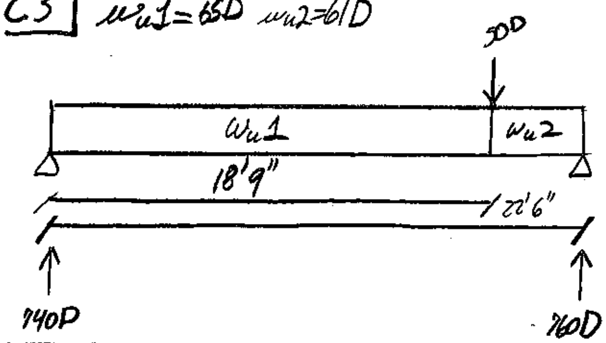
(2) 2X6 · Cd=.9

$V_r = 1.49k$

$M_r = 1.26$ k-ft

$\Delta_T = .035" = L/3034$

C3 | $w_{u1} = 65D$ $w_{u2} = 61D$



D

$V = .76k$

$M = 4.19$ k-ft

$\Delta_T = 383.1/EI$

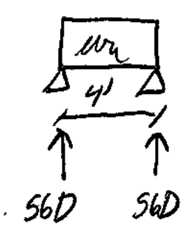
3½X9¼ LSL · Cd=.9, CL=.847

$V_r = 6.02k$

$M_r = 7.55$ k-ft

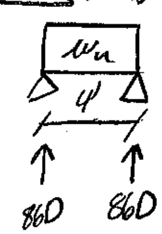
$\Delta_T = 1.069" = L/252$

C4 | $w_u = 28D$



(2) 2X6 pass by inspection

C5 | $w_u = 43D$



(2) 2X6 pass by inspection

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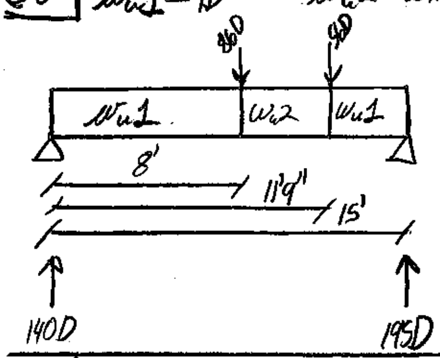
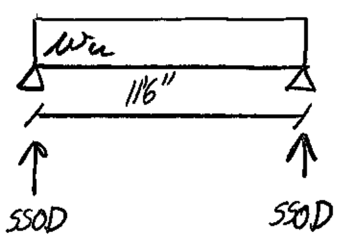
C6 | $w_u1 = 9D$ $w_u2 = 24D$



D
$V = .2k$
$M = .82 k\cdot ft$
$\Delta_T = 30.56/EI$

(2)2x6. Cd=.9
$V_r = 1.49k$
$M_r = 1.26 k\cdot ft$
$\Delta_T = .565" = L/318$

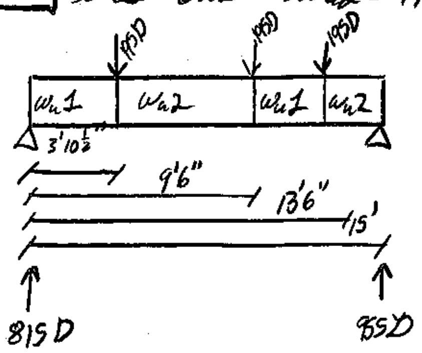
C7 | $w_u = 100D$



D
$V = .55 k$
$M = 1.51 k\cdot ft$
$\Delta_T = 32.94/EI$

(3)2x6 · Cd=.9
$V_r = 2.24 k$
$M_r = 1.89 k\cdot ft$
$\Delta_T = .407" = L/324$

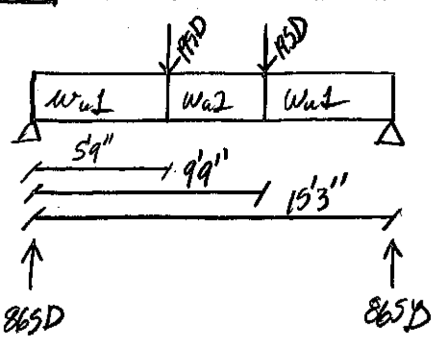
C8 | $w_u1 = 62D$ $w_u2 = 97D$



D
$V = .95 k$
$M = 3.42 k\cdot ft$
$\Delta_T = 138.7/EI$

(2) 1¾ x 7¼ LVL · Cd=.9
$V_r = 4.33 k$
$M_r = 6.39 k\cdot ft$
$\Delta_T = .619" = L/290$

C9 | $w_u1 = 97D$ $w_u2 = 62D$



D
$V = .87 k$
$M = 3.45 k\cdot ft$
$\Delta_T = 145.8/EI$

(2) 1¾ x 7¼ LVL · Cd=.9
$V_r = 4.33 k$
$M_r = 6.39 k\cdot ft$
$\Delta_T = .65" = L/281$

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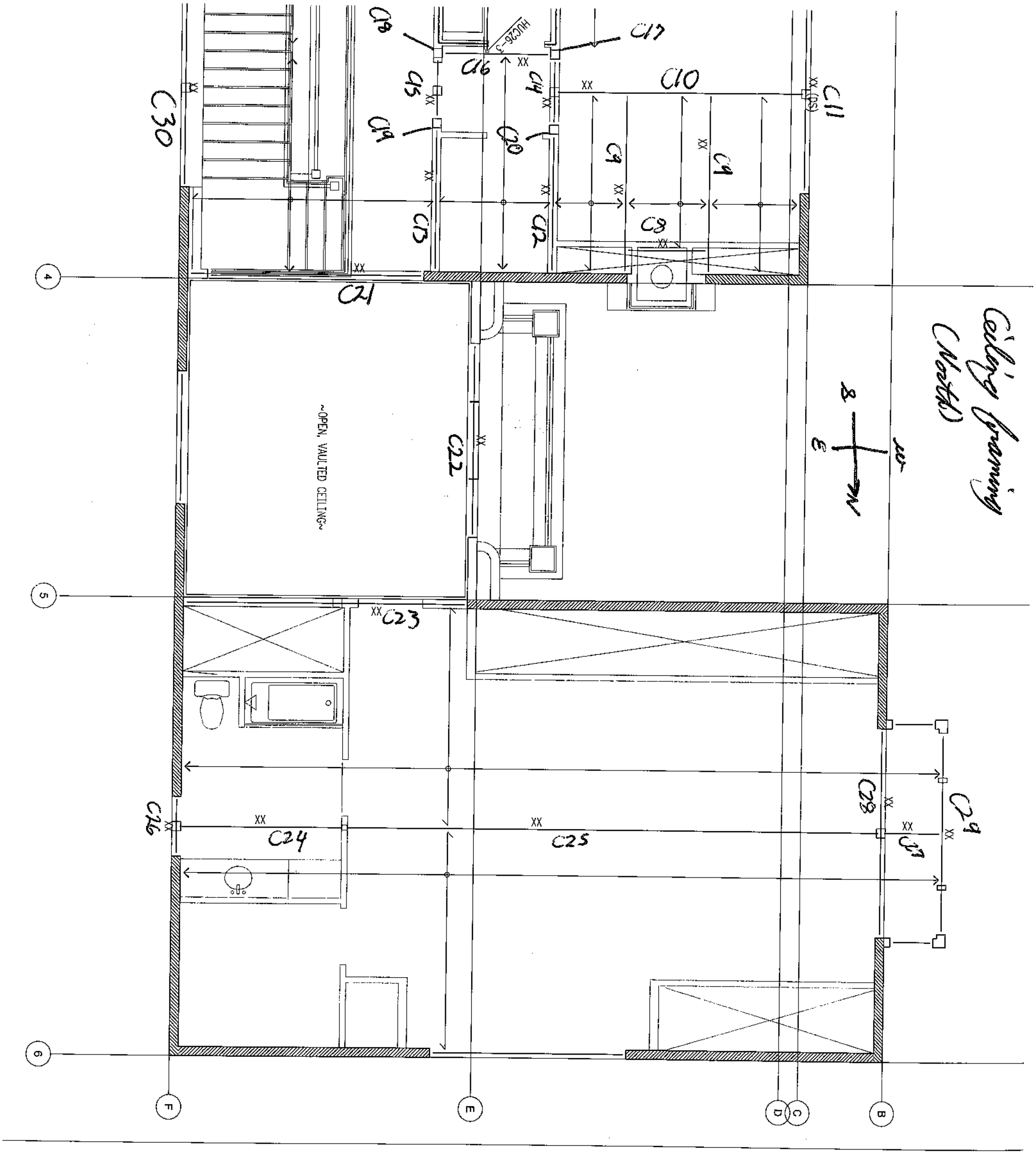
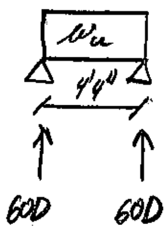
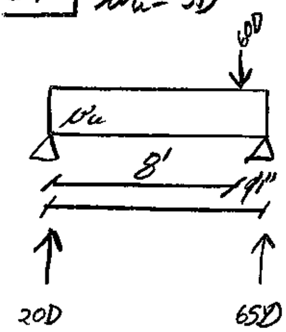

Ceiling framing (North)
~OPEN, VAULTED CEILING~
HUC26-3
C8
C9
C10
C11
C12
C13
C14
C15
C16
C17
C18
C19
C20
C21
C22
C23
C24
C25
C26
C27
C28
C29
C30
4
5
6
F
E
D
C
B

C8 | $w_u = 22D$



(2) 2x6 pass by inspection

C9 | $w_u = 3D$



D

$V = .07k$

$M = .07 \text{ k-ft}$

$\Delta_T = 1.035/EI$

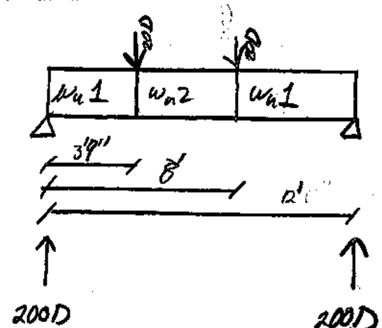
(2) 2x6, Cd = .9

$V_r = 1.49k$

$M_r = 1.26 \text{ k-ft}$

$\Delta_T = .019" = L/5686$

C10 | $w_u1 = 32D$ $w_u2 = 29D$



D

$V = .2k$

$M = .6 \text{ k-ft}$

$\Delta_T = 15.75/EI$

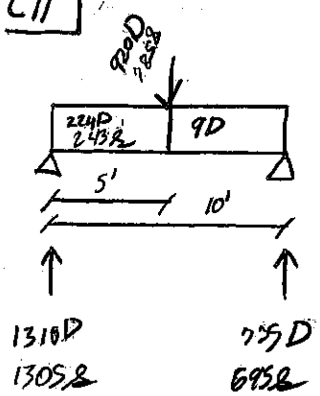
(2) 2x6 (Cd = .9)

$V_r = 1.49k$

$M_r = 1.26 \text{ k-ft}$

$\Delta_T = .291" = L/493$

C11 |



D+S

$V = 2.61 \text{ k}$

$M = 7.24 \text{ k-ft}$

$\Delta_L = 55.5 /EI$

$\Delta_T = 114.9 /EI$

$3\frac{1}{2} \times 9\frac{1}{4}$ LSL

$V_r = 6.69k$

$M_r = 9.905 \text{ k-ft}$

$\Delta_L = .156" = L/224$

$\Delta_T = .32" = L/373$

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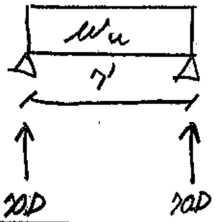
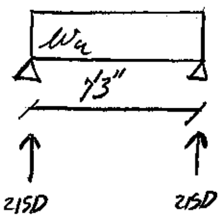
C12 $w_u = 21D$

(2)2x6 pass by inspection



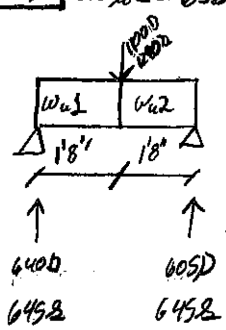
C13 $w_u = 59D$

D	(3)2x6 · (C_d = .9)
V = .22 k	V_r = 1.49 k
M = .39 k-ft	M_r = 1.26 k-ft
Δ_T = 3.67/EI	Δ_T = .068" = L/1280



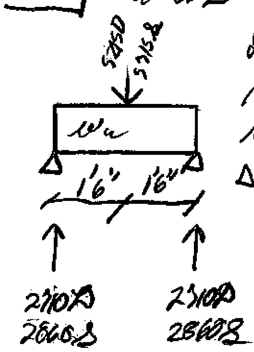
C14 $w_u1 = 63D$ $w_u2 = 24D$

D+S	(2) 1 3/4 x 5 1/2 LVL
V = 1.3 k	V_r = 3.66 k
M = 2.05 k-ft	M_r = 4.25 k-ft
Δ_T = 3.30/EI	Δ_T = .041" = L/972



C15 $w_u = 62D$

D+S	3 1/2 x 9 1/4 LSL
V = 5.6 k	V_r = 6.69 k
M = 9.19 k-ft	M_r = 9.9 k-ft
Δ_T = 14.75/EI	Δ_T = .041" = L/873.5



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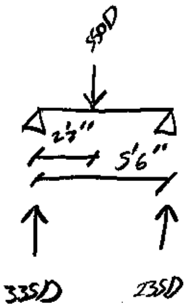
C16



D
V = .34k
M = .74k-ft
Δ_T = 3.21/EI

(2) 2x6 (Cd=.9)
V_r = 1.49k
M_r = 1.26k-ft
Δ_T = .059" = L/1110

C17 | P = 235D + 640 D, 645S | 835D, 645S ≻ 1.52k ~~(2) 2x6~~ 4x6 − Hf #2 = 2.31k
2x6 fail ratio @ L=9'

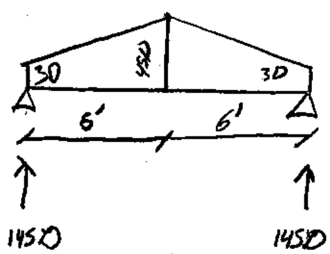
C18 | P = 335D + 2710D, 2860S | 3045, 2860S ≻ 5.905k ~~(2) 2x6~~ 4x6 Hf

C19 | 2710D, 2860S + 215D | 2925D, 2860S ≻ 5.285k ~~(2) 2x6~~ 4x6 Hf

C20 | 605D, 645S + 70D | 655D, 645S ≻ 1.32k ~~(2) 2x6~~ 4x6 Hf

---

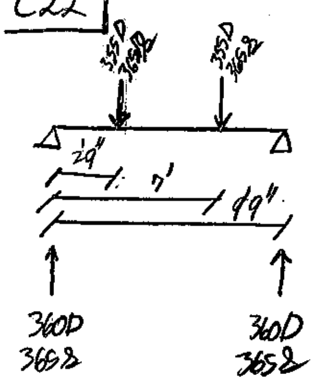
C21



D
V = .15k
M = .56k-ft
Δ_T = 13.94/EI

(2) 2x6, (Cd=.9)(CL=.826)
V_r = 1.49k
M_r = 1.04k-ft
Δ_T = .258" = L/557

C22



D+S
V = .74k
M = 2.03k-ft
Δ_T = 321/EI

(2) 2x8
V_r = 2.18k
M_r = 2.24k-ft
Δ_T = .299" = L/390

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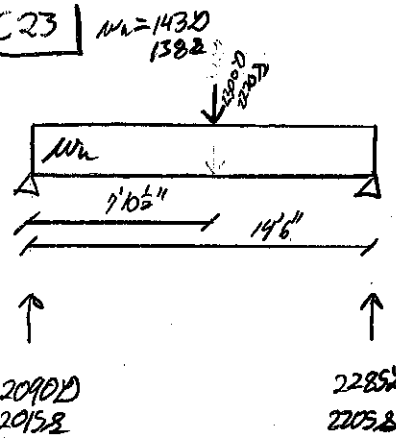
C23



D+ℒ

$V = 4.49k$

$M = 23.59$ k-ft

$\Delta_L = 398.4/EI$

$\Delta_T = 770.5/EI$

$5\frac{1}{4} \times 11\frac{1}{4}$ PSL

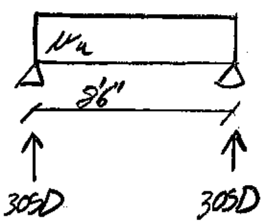
$V_R = 11.42k$

$M_R = 26.95$ k-ft

$\Delta_L = .303'' = L/572$

$\Delta_T = .618'' = L/281$

C24



D

$V = .31k$

$M = .651$ k-ft

$\Delta_T = 8.45/EI$

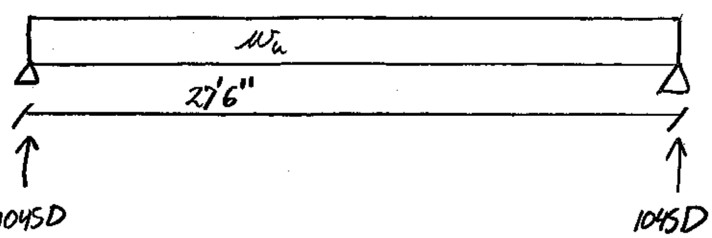
(2) 2X6. (Cd=.9)

$V_R = 1.49k$

$M_R = 1.26$ k-ft

$\Delta_T = .157'' = L/651$

C25



D

$V = 1.1k$

$M = 7.12$ k-ft

$\Delta_T = 933.9/EI$

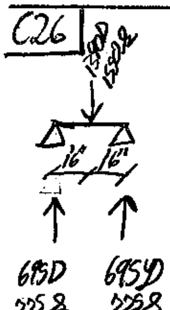
$3\frac{1}{2} \times 11\frac{7}{8}$ LSL (Cd=.9)(CL=.635)

$V_R = 2.23k$

$M_R = 9.11$ k-ft

$\Delta_T = 1.293'' = L/255$

C26



D+ℒ

$V = 1.47k$

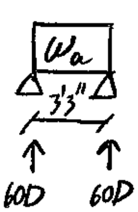
$M = 2.21$ k-ft

(2) 2x8

$V_R = 2.18k$

$M_R = 2.29$ k-ft

C27



(2) 2x6 pass by inspection

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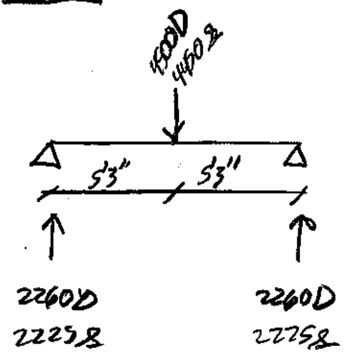
Seattle, Washington 98101-2447
tel. 206 624 4760 | fax 447 6971

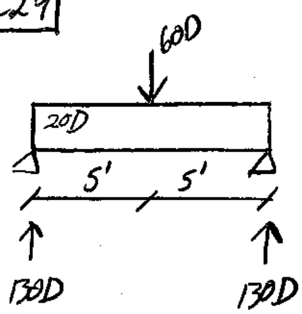
C28



D+S
$v = 4.5k$
$M = 23.5 k\cdot ft$
$\Delta_T = 372.9/EI$

5¼ X 11¼ 48L (?)
$v_r = 11.42 k$
$M_r = 26.95 k\cdot ft$
$\Delta_T = .299'' = L/421$

C29

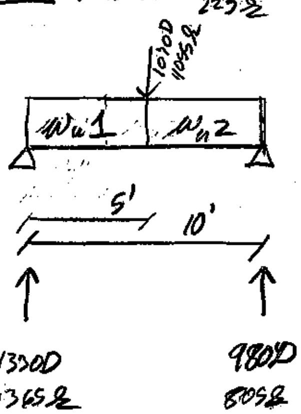


D
$v = .13k$
$M = .41 k\cdot ft$
$\Delta_T = 6.66/EI$

(2) 2X6 by inspection

C30

$w_u 1 = 209D$, $223S$ $w_u 2 = 49D$



D+S
$v = 2.74k$
$M = 8.31 k\cdot ft$
$\Delta_S = 63.06/EI$
$\Delta_T = 130.3/EI$

3½ X 9¼ LSL
$v_r = 6.69 k$
$M_r = 9.9 k\cdot ft$
$\Delta_S = .176'' = L/681$
$\Delta_T = .363'' = L/329$

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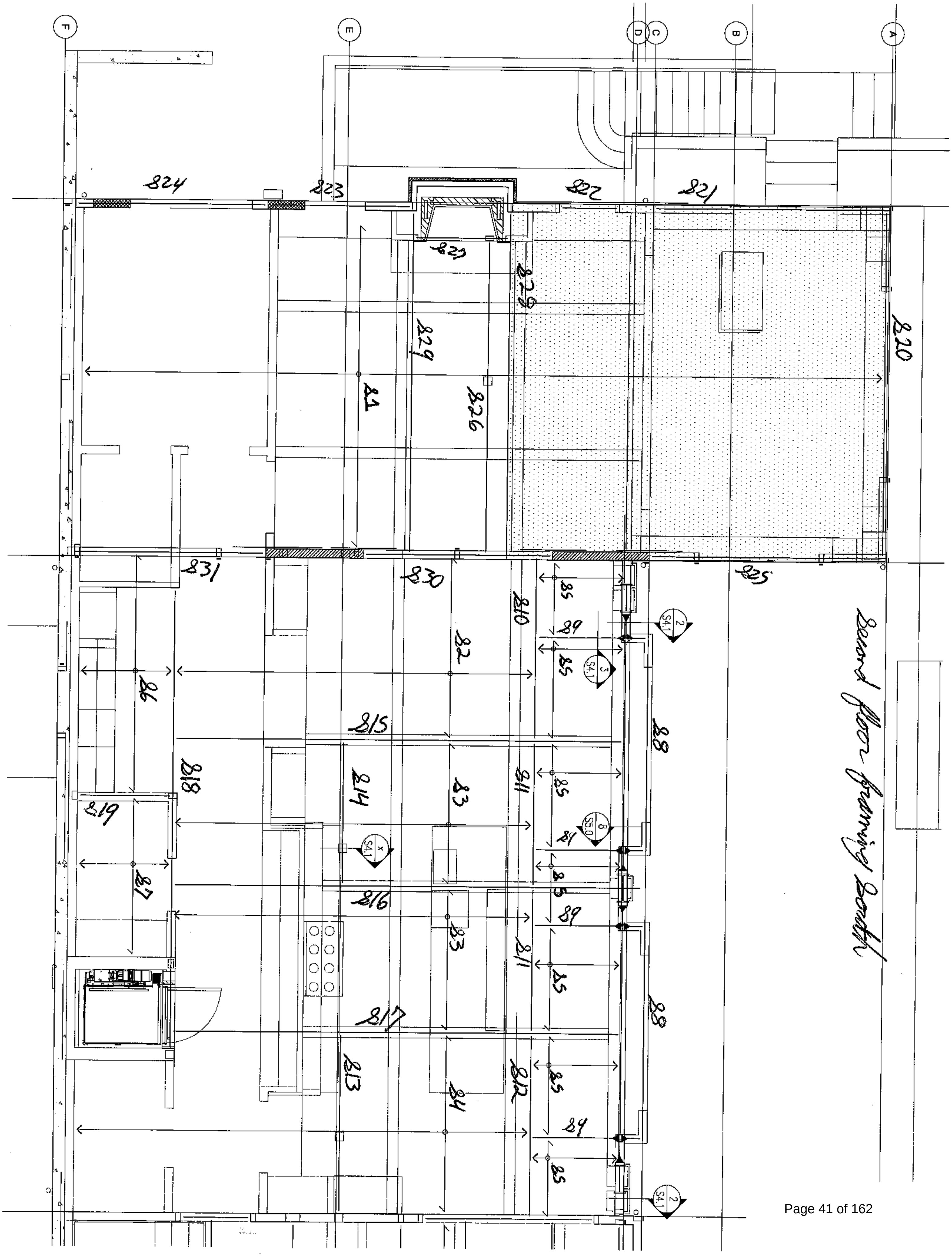

F
E
D
C
B
A
Second floor framing South
S4.1
S5.0

81 | See forte output – 14" TJI 560 @ 12" oc

82 | See forte output – 14" TJI 230 @ 16" oc

83 | See forte output – 14" TJI 230 @ 16" oc

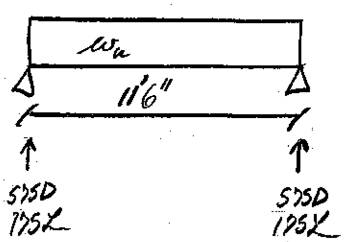
84 | See forte output – 14" TJI 230 @ 16" oc

85 | Same Loads, shorter spans as 84. 14" TJI 230 @ 16" OC

86 | See forte output – 14" TJI 230 @ 16" oc

87 | See forte output – 14" TJI 230 @ 16" oc

88 | w_u = 100D 30L



D+L

$V = .75k$

$M = 2.15 k \cdot ft$

$\Delta_L = 11.8/EI$

$\Delta_T = 51.5/EI$

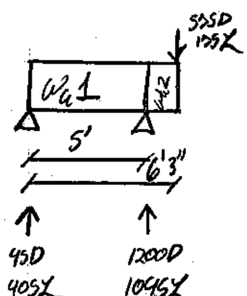
1¾X14 LSL • (C_L = .325)

$V_r = 5.05k$

$M_r = 3.55 k \cdot ft$

$\Delta_T = .083" = L/1672$

89 | w_u1 = 88D 195L w_u2 = 186D 240L



D+L

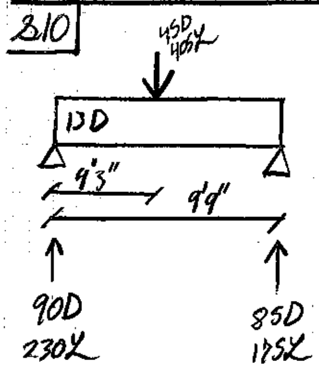
$V = 1.28k$

$M = 1.27 k \cdot ft$

$\Delta_T = 2.45/EI$

W8X15 pass by inspection

810 |



3½X14 LSL pass by inspection

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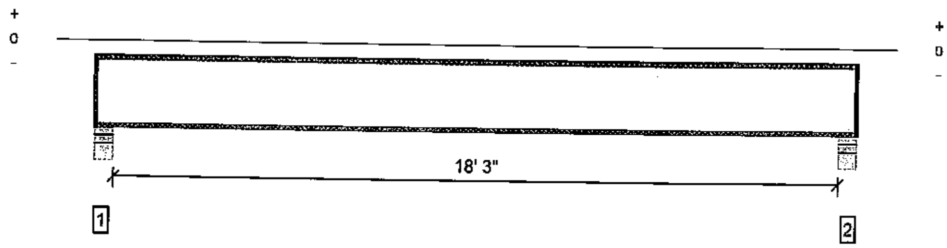
FORTE® **MEMBER REPORT** *Second Floor, S1*

1 piece(s) 14" TJI® 560 @ 12" OC

PASSED

Overall Length: 19' 2"



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

Member Reaction (lbs)	521 @ 4 1/2"	1725 (3.50")	Passed (30%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	502 @ 5 1/2"	2390	Passed (21%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2332 @ 9' 7"	11275	Passed (21%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.128 @ 9' 7"	0.276	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.176 @ 9' 7"	0.368	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	58	55	Passed	--	--

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 12' 11" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 19' o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

1 - Stud wall - HF	5.50"	4.25"	1.75"	144	383	527	1 1/4" Rim Board
2 - Stud wall - HF	5.50"	4.25"	1.75"	144	383	527	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

1 - Uniform (PSF)	0 to 19' 2"	12"	15.0	40.0	Residential - Living Areas

Forte Software Operator	Job Notes
Jonathan Carlson Harriott Valentine Engineers (206) 624-4760 jcarlson@harriottvalentine.com	

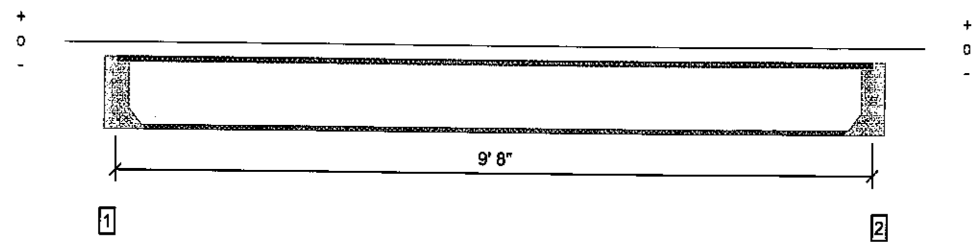
FORTE®

MEMBER REPORT *Second Floor, S2*

PASSED

1 piece(s) 14" TJI® 230 @ 16" OC

Overall Length: 10' 3"



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

Member Reaction (lbs)	354 @ 3 1/2"	1060 (1.75")	Passed (33%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	354 @ 3 1/2"	1945	Passed (18%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	857 @ 5' 1 1/2"	4990	Passed (17%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.028 @ 5' 1 1/2"	0.145	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.039 @ 5' 1 1/2"	0.193	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	64	55	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 9' 7" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 9' 8" o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

1 - Hanger on 14" HF beam	3.50"	Hanger[1]	1.75" / - [2]	103	273	376	See note [1]
2 - Hanger on 14" HF beam	3.50"	Hanger[1]	1.75" / - [2]	103	273	376	See note [1]

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

1 - Face Mount Hanger	THF23140	2.50"	N/A	18-10d x 3	2-10d x 1-1/2	
2 - Face Mount Hanger	THF23140	2.50"	N/A	18-10d x 3	2-10d x 1-1/2	

1 - Uniform (PSF)	0 to 10' 3"	16"	15.0	40.0	Residential - Living Areas

Forte Software Operator	Job Notes
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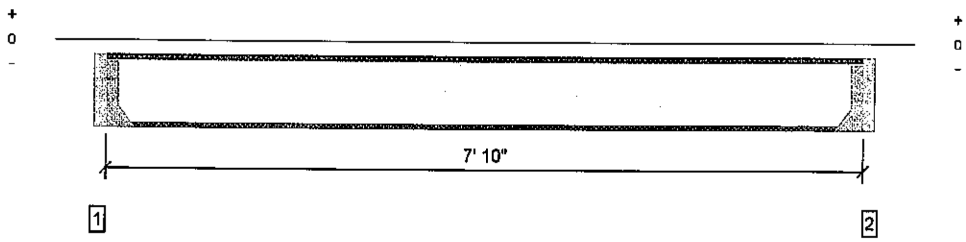
FORTE® **MEMBER REPORT** *Second Floor, S3* **PASSED**

1 piece(s) 14" TJI® 230 @ 16" OC

Overall Length: 8' 5"



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

Member Reaction (lbs)	287 @ 3 1/2"	1060 (1.75")	Passed (27%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	287 @ 3 1/2"	1945	Passed (15%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	562 @ 4' 2 1/2"	4990	Passed (11%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.014 @ 4' 2 1/2"	0.117	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.020 @ 4' 2 1/2"	0.157	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	66	55	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 7' 10" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 7' 10" o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

1 - Hanger on 14" HF beam	3.50"	Hanger[1]	1.75" / - [2]	84	224	308	See note [1]
2 - Hanger on 14" HF beam	3.50"	Hanger[1]	1.75" / - [2]	84	224	308	See note [1]

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

1 - Face Mount Hanger	THF23140	2.50"	N/A	18-10d x 3	2-10d x 1-1/2	
2 - Face Mount Hanger	THF23140	2.50"	N/A	18-10d x 3	2-10d x 1-1/2	

1 - Uniform (PSF)	0 to 8' 5"	16"	15.0	40.0	Residential - Living Areas

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

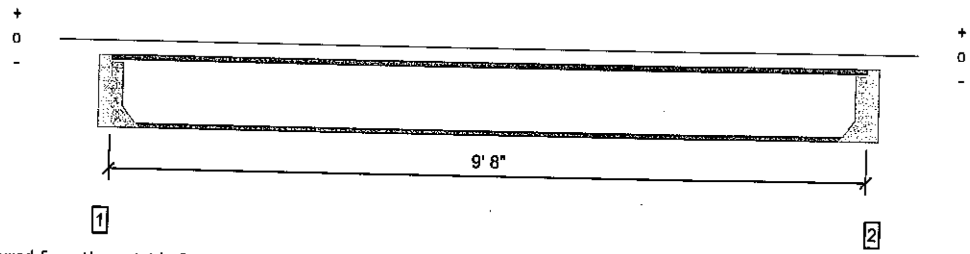
SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
Jonathan Carlson Harriott Valentine Engineers (206) 624-4760 jcarlson@harriottvalentine.com	

1 piece(s) 14" TJI® 230 @ 16" OC

PASSED

Overall Length: 10' 3"



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

Member Reaction (lbs)	354 @ 3 1/2"	1060 (1.75")	Passed (33%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	354 @ 3 1/2"	1945	Passed (18%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	857 @ 5' 1 1/2"	4990	Passed (17%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.028 @ 5' 1 1/2"	0.145	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.039 @ 5' 1 1/2"	0.193	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	64	55	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 9' 7" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 9' 8" o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

1 - Hanger on 14" HF beam	3.50"	Hanger[1]	1.75" / - [2]	103	273	376	See note [1]
2 - Hanger on 14" HF beam	3.50"	Hanger[1]	1.75" / - [2]	103	273	376	See note [1]

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

1 - Face Mount Hanger	THF23140	2.50"	N/A	18-10d x 3	2-10d x 1-1/2	
2 - Face Mount Hanger	THF23140	2.50"	N/A	18-10d x 3	2-10d x 1-1/2	

1 - Uniform (PSF)	0 to 10' 3"	16"	15.0	40.0	Residential - Living Areas

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

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Forte Software Operator	Job Notes
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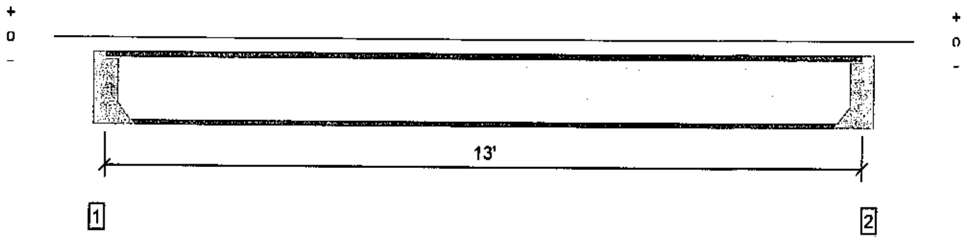
1 piece(s) 14" TJI® 230 @ 16" OC

Overall Length: 13' 7"



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

Member Reaction (lbs)	477 @ 3 1/2"	1060 (1.75")	Passed (45%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	477 @ 3 1/2"	1945	Passed (25%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1549 @ 6' 9 1/2"	4990	Passed (31%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.078 @ 6' 9 1/2"	0.195	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.107 @ 6' 9 1/2"	0.260	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	57	55	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 7' 7" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 13' o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

1 - Hanger on 14" HF beam	3.50"	Hanger¹	1.75" / - ²	136	362	498	See note ¹
2 - Hanger on 14" HF beam	3.50"	Hanger¹	1.75" / - ²	136	362	498	See note ¹

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- ¹ See Connector grid below for additional information and/or requirements.
- ² Required Bearing Length / Required Bearing Length with Web Stiffeners

1 - Face Mount Hanger	THF23140	2.50"	N/A	18-10d x 3	2-10d x 1-1/2	
2 - Face Mount Hanger	THF23140	2.50"	N/A	18-10d x 3	2-10d x 1-1/2	

1 - Uniform (PSF)	0 to 13' 7"	16"	15.0	40.0	Residential - Living Areas

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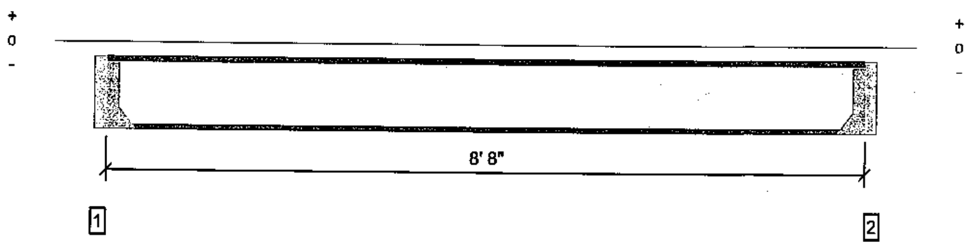
1 piece(s) 14" TJI® 230 @ 16" OC

Overall Length: 9' 3"



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

Member Reaction (lbs)	318 @ 3 1/2"	1060 (1.75")	Passed (30%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	318 @ 3 1/2"	1945	Passed (16%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	689 @ 4' 7 1/2"	4990	Passed (14%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.020 @ 4' 7 1/2"	0.130	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.027 @ 4' 7 1/2"	0.173	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	66	55	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 8' 8" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 8' 8" o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

1 - Hanger on 14" HF beam	3.50"	Hanger¹	1.75" / - ²	93	247	340	See note ¹
2 - Hanger on 14" HF beam	3.50"	Hanger¹	1.75" / - ²	93	247	340	See note ¹

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- ¹ See Connector grid below for additional information and/or requirements.
- ² Required Bearing Length / Required Bearing Length with Web Stiffeners

1 - Face Mount Hanger	THF23140	2.50"	N/A	18-10d x 3	2-10d x 1-1/2	
2 - Face Mount Hanger	THF23140	2.50"	N/A	18-10d x 3	2-10d x 1-1/2	

1 - Uniform (PSF)	0 to 9' 3"	16"	15.0	40.0	Residential - Living Areas

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

SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
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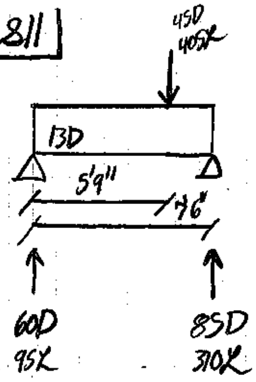
811



3½X14 LSL pass by inspection

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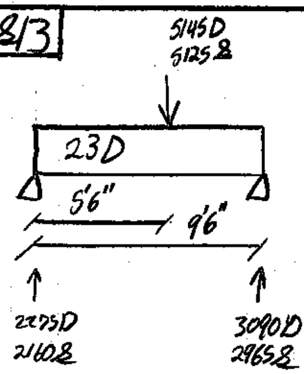
812 Mirror of 810, same conditions 3½X14 LSL

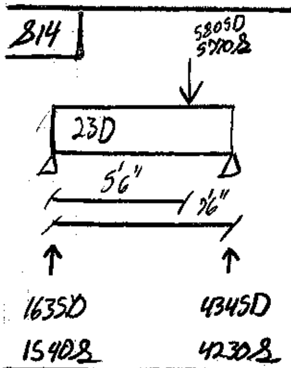
813



D+S
$v = 6.06K$
$M = 24.04 K\cdot ft$
$\Delta_\Delta = 153/EI$
$\Delta_T = 310.8/EI$

5¼X14 PSL
$v_r = 14.21K$
$M_r = 40.74 K\cdot ft$
$\Delta_\Delta = .063" = L/1789$
$\Delta_T = .129" = L/881$

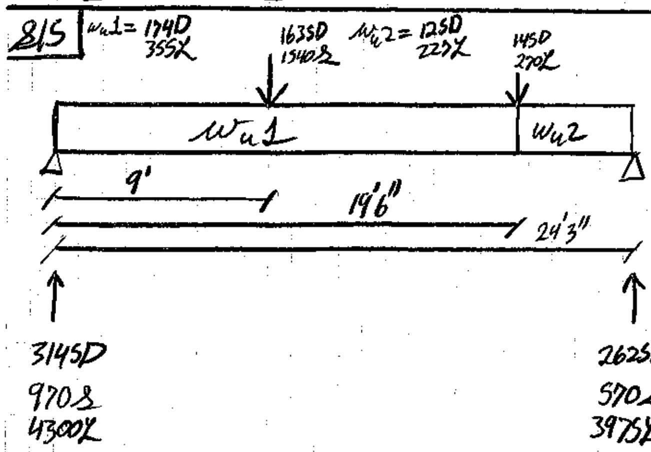
814



D+S
$v = 8.58K$
$M = 17.10 K\cdot ft$
$\Delta_\Delta = 64.4/EI$
$\Delta_T = 130.8/EI$

5¼X14 PSL
$v_r = 14.21K$
$M_r = 40.74 K\cdot ft$
$\Delta_T = .054" = L/1652$

815



D+L
$v = 7.44K$
$M = 46.58 K\cdot ft$
$\Delta_L = 2750/EI$
$\Delta_T = 4875/EI$

W8X35
$v_r = 83.5K$
$M_r = 165.9 K\cdot ft$
$\Delta_L = .185" = L/1565$
$\Delta_T = .329" = L/882$

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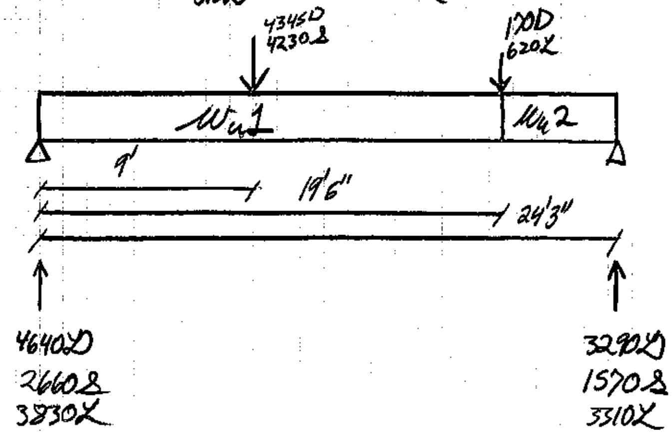
816 $w_u 1 = \frac{158D}{315L}$ $w_u 2 = \frac{70D}{80L}$

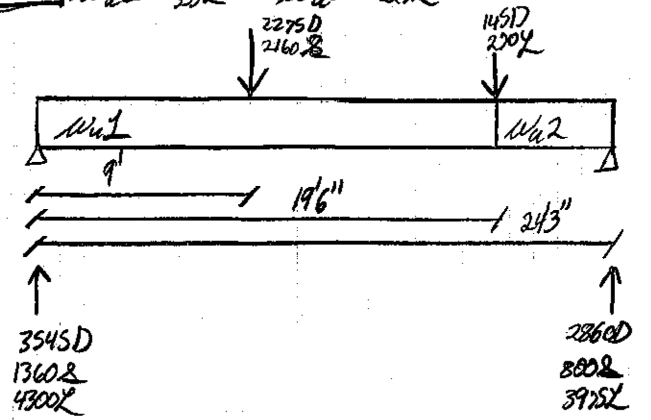


D+.75S+.75L
V = 9.51 k
M = 69.62 k·ft
Δ_L = 3333/EI
Δ_T = 6585/EI

<u>W18X46</u>
V_2 = 100 k
M_2 = 226.2 k·ft
Δ_L = .161" = L/1802
Δ_T = .318" = L/912

---

817 $w_u 1 = \frac{174D}{355L}$ $w_u 2 = \frac{125D}{225L}$

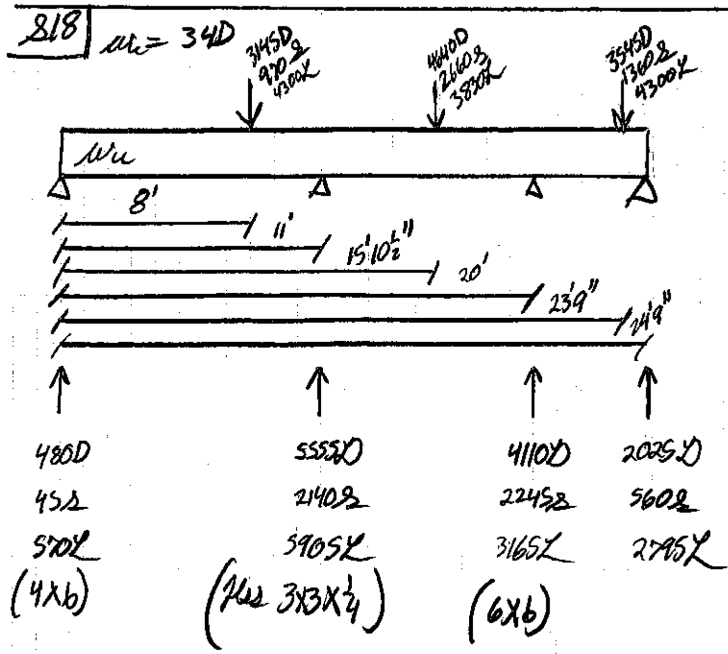


D+.75S+.75L
V = 7.77 k
M = 52.1 k·ft
Δ_L = 2820/EI
Δ_T = 5232/EI

<u>W18X35</u>
V_2 = 83.5 k
M_2 = 165.9 k·ft
Δ_L = .19" = L/1526
Δ_T = .353" = L/822

---

818 w_u = 34D



D+L
V = 6.75 k
M = 12.96 k·ft
Δ_T = 120.1/EI

<u>W14X34</u> pass by <u>inspection</u>

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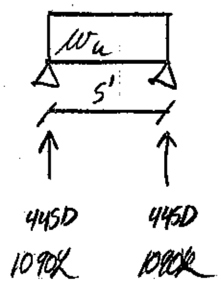
819 | $w_u = $ 132D, 435L



D+L

$V = 1.53$ K

$M = 1.92$ K-ft

$\Delta_T = 8.6\ /EI$

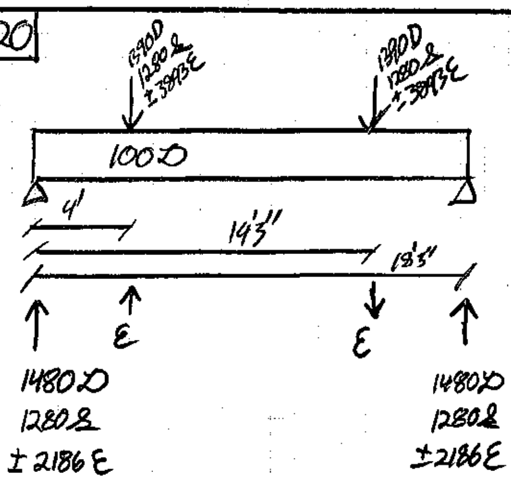
3½x14 LSL

$V_r = 10.1$ K

$M_r = 21.8$ K-ft

$\Delta_T = .007'' = L/8644$

820



Combo (Cd=1.0)

6B $V = 4.18$K

6B $M = 16.18$ K-ft

$\Delta_L = 344.7/EI$

$\Delta_T = 968/EI$

7x14 PSL

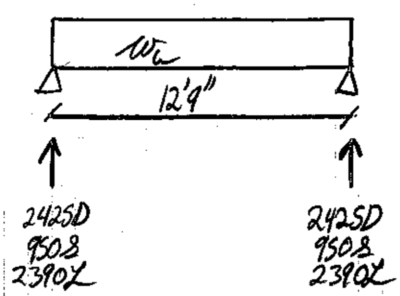
$V_r = 18.94$K

$M_r = 54.3$ K-ft

$\Delta_L = .107'' = L/2034$

$\Delta_T = .302'' = L/724$

821 | $w_u = $ 380D, 149S, 390L



D+L

$V = 4.81$ K

$M = 15.34$ K-ft

$\Delta_L = 222.9/EI$

$\Delta_T = 448.9/EI$

5¼x14 PSL

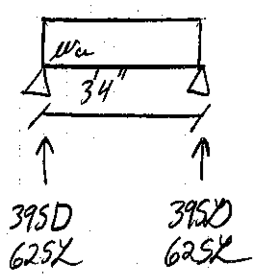
$V_r = 14.21$ K

$M_r = 40.74$ K-ft

$\Delta_T = .186'' = L/818$

822 | $w_u = $ 236D, 375L

3½x14 LSL pass by inspection



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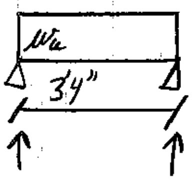
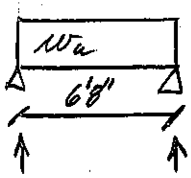
823 | w_u = 380D, 149S, 370L

$3\frac{1}{2}$X14 LSL pass by inspection



635D	635D
215S	215S
615L	615L

824 | w_u = 380D, 149S, 370L

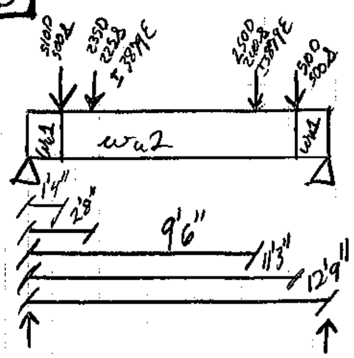


D+S+L
$V = 3k$
$M = 5$ k-ft
$\Delta_T = 39.95/EI$

$3\frac{1}{2}$X14 LSL
$V_r = 10.12K$
$M_r = 21.89$ k-ft
$\Delta_T = .032" = L/2482$

1265D	1265D
495S	495S
1235L	1235L

825 | w_{u1} = 380D, 149S, 375L ; w_{u2} = 245D, 375L

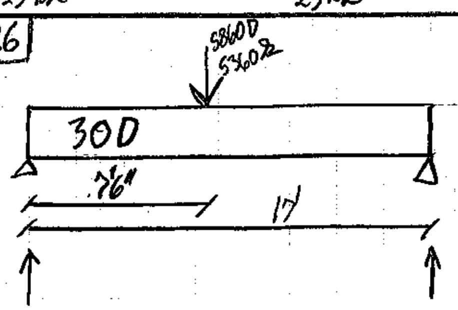


Gov. (C_d=1.0)
6B $V = 5k$
3 $M = 14.2$ k-ft
3 $\Delta_L = 222.9/EI$
3 $\Delta_T = 421.7/EI$

$5\frac{1}{4}$X14 PSL
$V_r = 14.21k$
$M_r = 40.74$ k-ft
$\Delta_T = .175" = L/871$

2510D	2500D
945S	940S
2390L	2390L

826



D+S
$V = 6.86k$
$M = 47.25$ k-ft
$\Delta_D = 909.6/EI$
$\Delta_T = 1960/EI$

W 14X34
$V_r = 59.5k$
$M_r = 56.48$ k-ft
$\Delta_S = .092" = L/2211$
$\Delta_T = .198" = L/1026$

3700D	2670D
2995S	2365S

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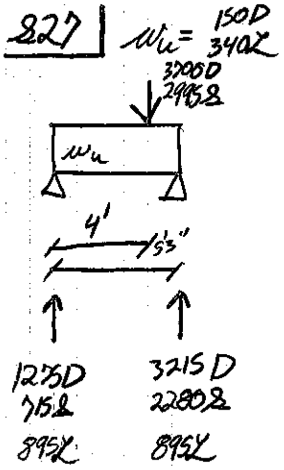
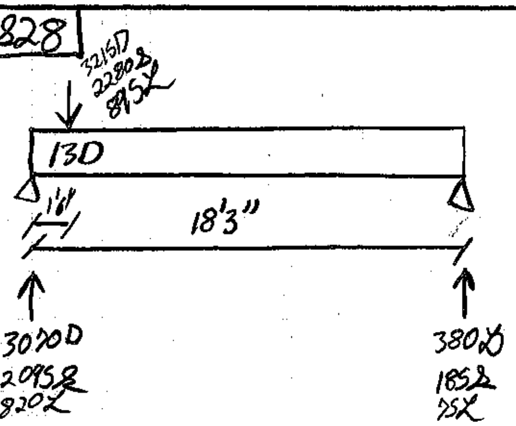
827



D+S+L
V = 6.39K
M = 7.6 k-ft
Δ_T = 31.69/EI

$3\frac{1}{2}$X14 LsL
V_r = 10.12K
M_r = 21.84 k-ft
Δ_T = .0255" = L/2465

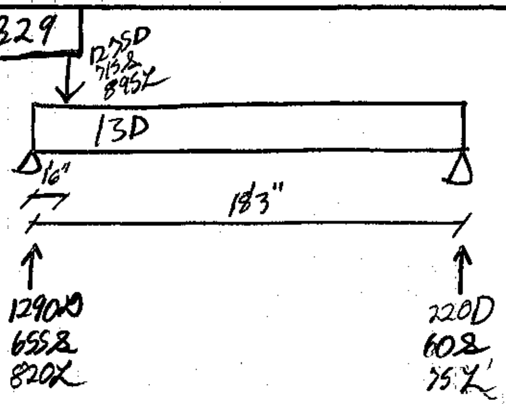
828



D+S+L
V = 6K
M = 8.96k-ft
Δ_T = 381.9/EI

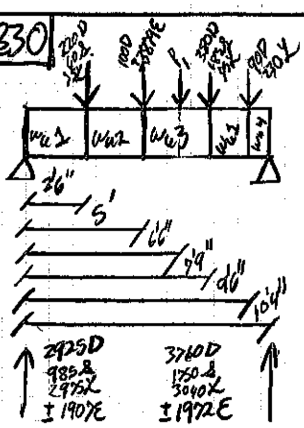
$3\frac{1}{2}$X14 LsL
V_r = 10.12K
M_r = 21.84k-ft
Δ_T = .307" = L/711

829



$3\frac{1}{2}$X14 LsL pass by 828 calculation

830



P_1 = 2670D 2365S

w_u1 = 345D 575L

w_u2 = 233D 540L

w_u3 = 332D 540L

w_u4 = 302D 460L

*See forte output

7X14 pass strength & deflection

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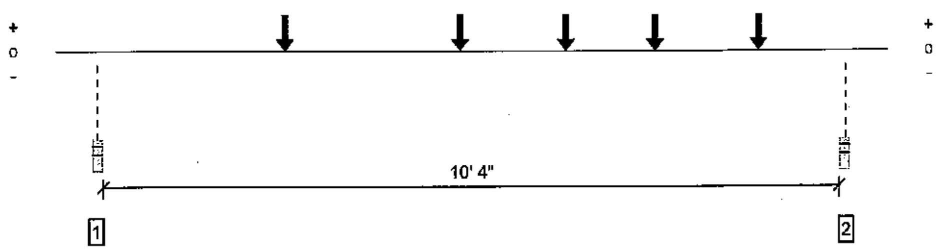
FORTE® MEMBER REPORT *Second Floor, S21 230*

FAILED

1 piece(s) 7" x 14" 2.0E Parallam® PSL

Support 2 failed the reaction check due to insufficient bearing capacity.
This product failed due to an excessive uplift of -1654 lbs at support located at 1 1/2".

Overall Length: 10' 10"



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

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Member Reaction (lbs)	9975 @ 10' 8 1/2"	8505 (3.00")	Failed (117%)	--	1.0 D + 0.525 E + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	5990 @ 9' 5"	18947	Passed (32%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	33942 @ 5' 3"	86918	Passed (39%)	1.60	1.0 D + 0.525 E + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.156 @ 5' 3"	0.159	Passed (L/813)	--	1.0 D + 0.525 E + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.238 @ 5' 6 3/16"	0.212	Failed (L/534)	--	1.0 D + 0.525 E + 0.75 L + 0.75 S (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 10' 10" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 10' 10" o/c unless detailed otherwise.
- -937 lbs uplift at support 10' 8 1/2". Strapping or other restraint may be required.
- Member should be side-loaded from both sides of the member to prevent rotation.

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1 - Stud wall - HF	3.00"	3.00"	3.06"	3078	2972	985	5001/-5001	12036/-5001	Blocking
2 - Stud wall - HF	3.00"	3.00"	3.52"	3917	3041	1749	4696/-4696	13403/-4696	Blocking

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

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0 - Self Weight (PLF)	0 to 10' 10"	N/A	30.7				
1 - Uniform (PLF)	3" to 2' 9" (Front)	N/A	345.0	575.0	-	-	Residential - Living Areas
2 - Uniform (PLF)	8' to 9' 6" (Front)	N/A	345.0	575.0	-	-	Residential - Living Areas
3 - Uniform (PLF)	2' 9" to 5' 3" (Top)	N/A	233.0	540.0	-	-	Residential - Living Areas
4 - Uniform (PLF)	5' 3" to 8' (Top)	N/A	332.0	540.0	-	-	Residential - Living Areas
5 - Uniform (PLF)	9' 6" to 10' 7" (Top)	N/A	302.0	460.0	114.0	-	Residential - Living Areas
6 - Point (lb)	2' 9" (Top)	N/A	220	75	60	-	
7 - Point (lb)	5' 3" (Top)	N/A	100	-	-	9697	E=2.5 for Omega
8 - Point (lb)	6' 9" (Top)	N/A	2670	-	2365	-	
9 - Point (lb)	8' (Top)	N/A	380	75	185	-	
10 - Point (lb)	9' 6" (Top)	N/A	90	230	-	-	

Weyerhaeuser Notes

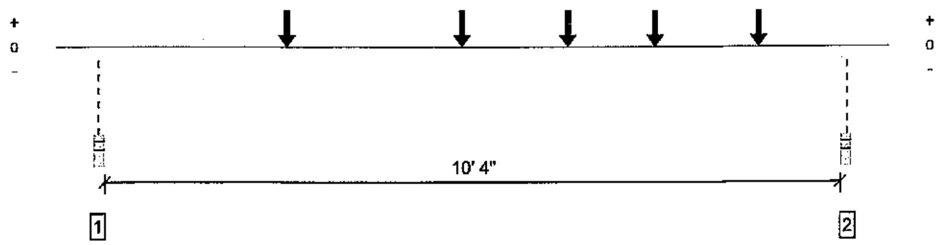
SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
Jonathan Carlson Harriott Valentine Engineers (206) 624-4760 jcarlson@harriottvalentine.com	

FORTE® **MEMBER REPORT** *Second Floor, S21 (830) Deflection*

1 piece(s) 7" x 14" 2.0E Parallam® PSL

Overall Length: 10' 10"



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	7510 @ 10' 8 1/2"	8505 (3.00")	Passed (88%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	5990 @ 9' 5"	18947	Passed (32%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	19686 @ 6' 7 3/4"	54324	Passed (36%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.073 @ 5' 6 13/16"	0.159	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.156 @ 5' 7 3/16"	0.212	Passed (L/817)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 10' 10" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 10' 10" o/c unless detailed otherwise.
- Member should be side-loaded from both sides of the member to prevent rotation.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Stud wall - HF	3.00"	3.00"	2.13"	3078	2972	985	7035	Blocking
2 - Stud wall - HF	3.00"	3.00"	2.65"	3917	3041	1749	8707	Blocking

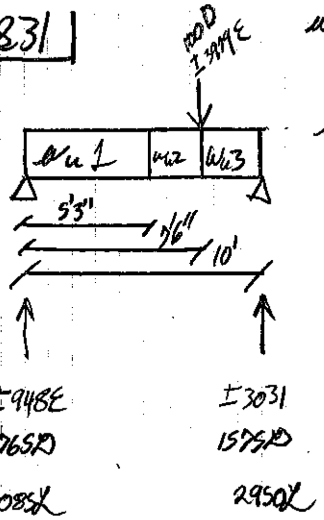
- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 10' 10"	N/A	30.7			
1 - Uniform (PLF)	3" to 2' 9" (Front)	N/A	345.0	575.0	-	Residential - Living Areas
2 - Uniform (PLF)	8' to 9' 6" (Front)	N/A	345.0	575.0	-	Residential - Living Areas
3 - Uniform (PLF)	2' 9" to 5' 3" (Top)	N/A	233.0	540.0	-	Residential - Living Areas
4 - Uniform (PLF)	5' 3" to 8' (Top)	N/A	332.0	540.0	-	Residential - Living Areas
5 - Uniform (PLF)	9' 6" to 10' 7" (Top)	N/A	302.0	460.0	114.0	Residential - Living Areas
6 - Point (lb)	2' 9" (Top)	N/A	220	75	60	
7 - Point (lb)	5' 3" (Top)	N/A	100	-	-	E=2.5 for Omega
8 - Point (lb)	6' 9" (Top)	N/A	2670	-	2365	
9 - Point (lb)	8' (Top)	N/A	380	75	185	
10 - Point (lb)	9' 6" (Top)	N/A	90	230	-	

Weyerhaeuser Notes

Forte Software Operator	Job Notes
Jonathan Carlson Harriott Valentine Engineers (206) 624-4760 jcarlson@harriottvalentine.com	

831



$w_u1 = 559D$
$630L$

$w_u2 = 338D$
$575L$

$w_u3 = 234D$
$575L$

$D+L+1.75E$

$V = 9.75k$

$M = 22.07k\text{-}ft$

$\Delta_L = 136/EI$

$\Delta_T = 387.3/EI$

$5\frac{1}{4} \times 14$ PSL

$V_r = 14.21k$

$M_r = 40.74\ k\text{-}ft$

$\Delta_L = .056'' = L/2119$

$\Delta_T = .16'' = L/744$

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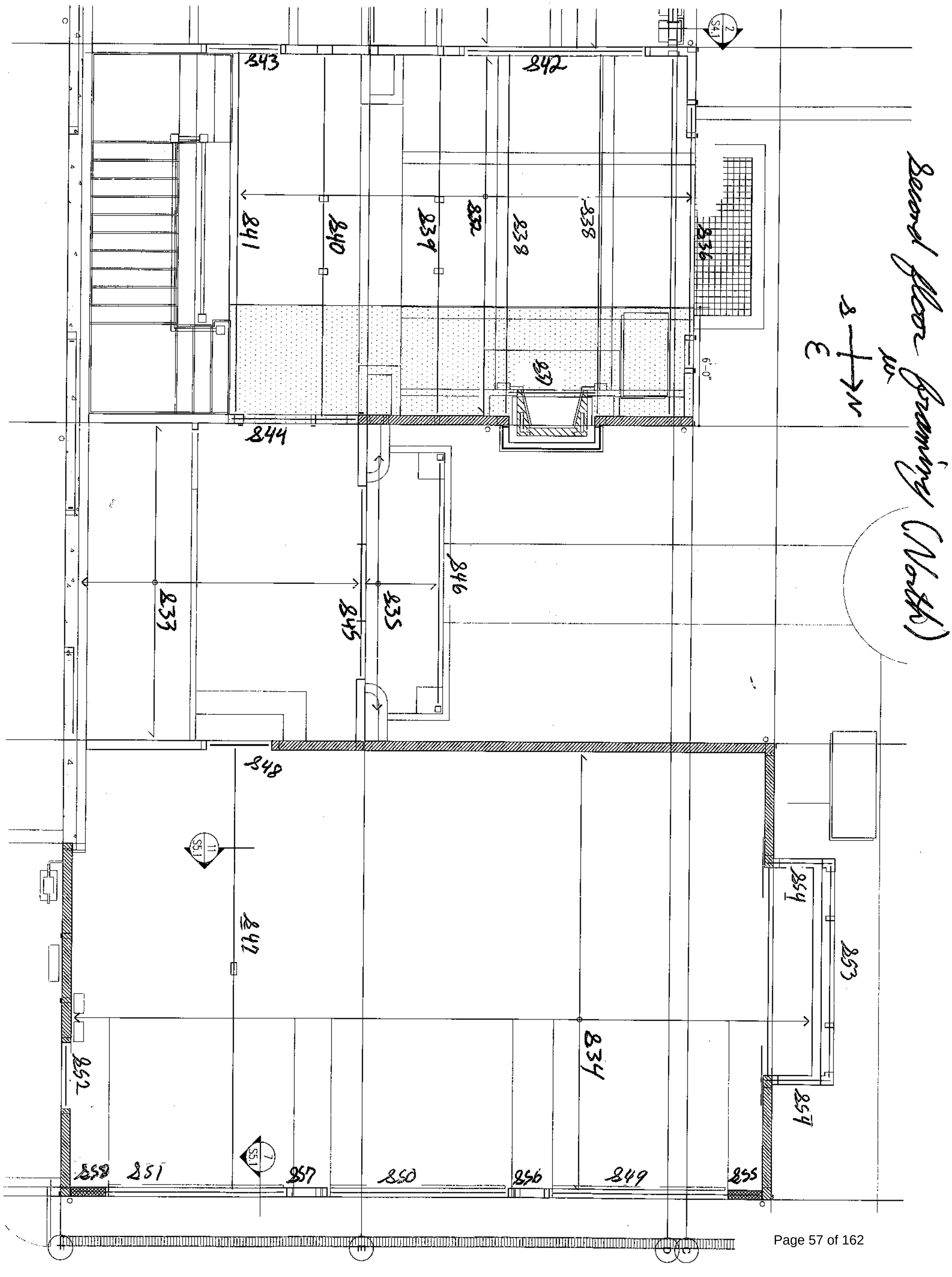
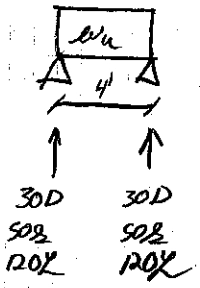

Second floor framing (North)
843
842
841
840
839
832
838
838
836
837
6'-0"
844
833
845
835
846
848
847
834
853
854
854
852
858
851
857
850
856
849
855
S4.1
S5.1
S5.1

832 | See forte output – 14" TJI 560 @ 12" oc

833 | See forte output – 14" TJI 230 @ 12" oc

834 | See forte output – (2) 1¾ X 14 LSL @ 12" oc

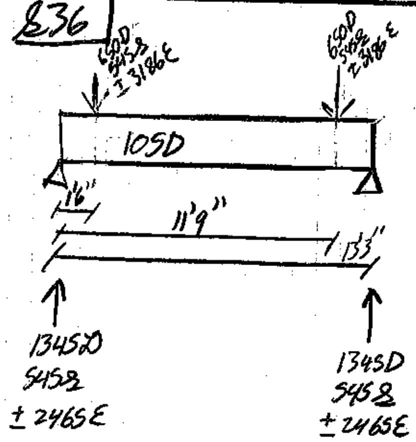
835 | w_u = 15D, 25S, 60L



D+S+L @ 24"oc
V = .32K
M = .32K-ft
Δ_T = .921/EI

2X6 @ 24" oc
V_r = .83K
M_r = .7K-ft
Δ_T = .034" = L/1406

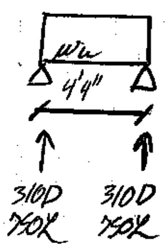
836



D+S+1.75E
V = 6.2K
M = 9.19K-ft
Δ_T = 153.4/EI

5¼X14 PSL
V_r = 14.21K
M_r = 40.74 K-ft
Δ_T = .063" = L/2489

837 | w_u = 142D, 345L



D+L
V = 1.06K
M = 1.14K-ft
Δ_T = 3.86/EI

3½X14 LSL by inspection

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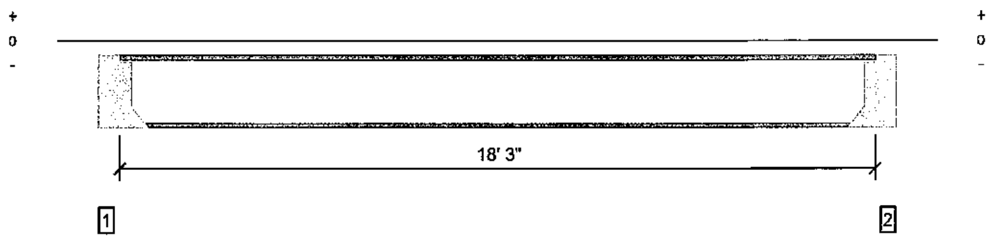
FORTE® MEMBER REPORT *Second Floor, S32*

1 piece(s) 14" TJI® 560 @ 12" OC

PASSED

Overall Length: 19' 3"



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	502 @ 6"	1265 (1.75")	Passed (40%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	502 @ 6"	2390	Passed (21%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2290 @ 9' 7 1/2"	11275	Passed (20%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.124 @ 9' 7 1/2"	0.274	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.170 @ 9' 7 1/2"	0.365	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	58	55	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 13' o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 18' 3" o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

Supports	Bearing Length			Loads to Supports (lbs)			
	Total	Available	Required	[illegible]	[illegible]	[illegible]	Accessories
1 - Hanger on 14" HF beam	6.00"	Hanger[1]	1.75" / - [2]	137	365	502	See note [1]
2 - Hanger on 14" HF beam	6.00"	Hanger[1]	1.75" / - [2]	137	365	502	See note [1]

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Connector: [illegible]						
Support	Model	Seat Length	[illegible]	Face Nails	Member Nails	Accessories
1 - Face Mount Hanger	THF35140	2.50"	N/A	20-10d x 3	2-10d x 1-1/2	
2 - Face Mount Hanger	THF35140	2.50"	N/A	20-10d x 3	2-10d x 1-1/2	

Loads	Location (Side)	Spacing	[illegible]	[illegible]	[illegible]
1 - Uniform (PSF)	6" to 18' 9"	12"	15.0	40.0	Residential - Living Areas

Weyerhaeuser Notes

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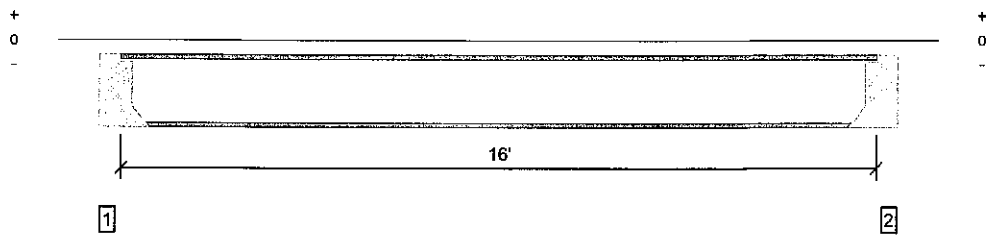
The product application, input design loads, dimensions and support information have been provided by Forte Software Operator

SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
Jonathan Carlson Harriott Valentine Engineers (206) 624-4760 jcarlson@harriottvalentine.com	

1 piece(s) 14" TJI® 230 @ 12" OC

Overall Length: 17'



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	440 @ 6"	1060 (1.75")	Passed (42%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	440 @ 6"	1945	Passed (23%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1760 @ 8' 6"	4990	Passed (35%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.126 @ 8' 6"	0.240	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.173 @ 8' 6"	0.320	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	55	55	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 7' 2" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 16' o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

Supports	Bearing Length			Loads to Supports (lbs)			
	Total	Available	Required	Dead	Floor Live	Total	Accessories
1 - Hanger on 14" HF beam	6.00"	Hanger[1]	1.75" / - [2]	120	320	440	See note [1]
2 - Hanger on 14" HF beam	6.00"	Hanger[1]	1.75" / - [2]	120	320	440	See note [1]

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Connector: USP Connectors

Support	Model	Seat Length	Top Nails	Face Nails	Member Nails	Accessories
1 - Face Mount Hanger	THF23140	2.50"	N/A	18-10d x 3	2-10d x 1-1/2	
2 - Face Mount Hanger	THF23140	2.50"	N/A	18-10d x 3	2-10d x 1-1/2	

Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	6" to 16' 6"	12"	15.0	40.0	Residential - Living Areas

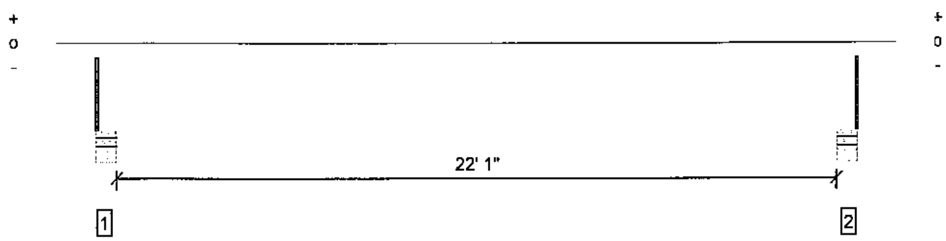
Weyerhaeuser Notes

Forte Software Operator	Job Notes
Jonathan Carlson Harriott Valentine Engineers (206) 624-4760 jcarlson@harriottvalentine.com	

FORTE® MEMBER REPORT *Second Floor, S34* **PASSED**

2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL @ 12" OC

Overall Length: 23' 1"



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	607 @ 5"	6733 (4.75")	Passed (9%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	543 @ 1' 8"	9310	Passed (6%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3403 @ 11' 6 1/2"	25229	Passed (13%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.139 @ 11' 6 1/2"	0.334	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.191 @ 11' 6 1/2"	0.445	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	56	55	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 22' 11" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 22' 11" o/c unless detailed otherwise.
- A 4% increase in the moment capacity has been added to account for repetitive member usage.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - HF	6.00"	4.75"	1.50"	166	442	608	1 1/4" Rim Board
2 - Stud wall - HF	6.00"	4.75"	1.50"	166	442	608	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	6" to 22' 7"	12"	15.0	40.0	Residential - Living Areas

Weyerhaeuser Notes

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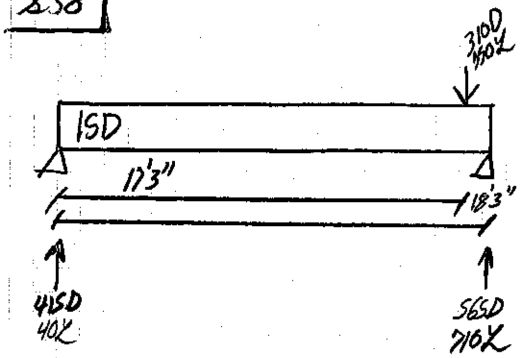
The product application, input design loads, dimensions and support information have been provided by Forte Software Operator

SUSTAINABLE FORESTRY INITIATIVE

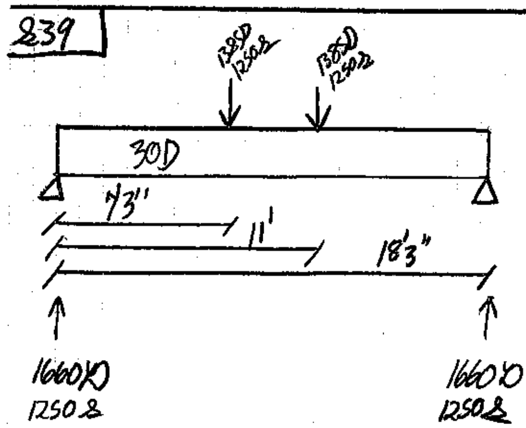
Forte Software Operator	Job Notes
Jonathan Carlson Harriott Valentine Engineers (206) 624-4760 jcarlson@harriottvalentine.com	

838



$3\frac{1}{2}$X14 LSL pass by inspection

839



D+S

$V = 2.9k$

$M = 20.35$ k-ft

$\Delta_L = 514.2/EI$

$\Delta_T = 1160/EI$

3X14 PSL

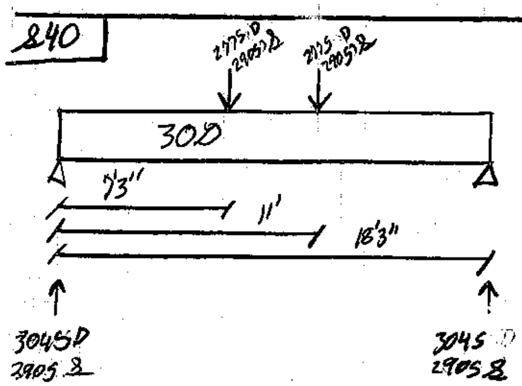
$V_r = 18.9k$

$M_r = 54.3$ k-ft

$\Delta_L = .16'' = L/1362$

$\Delta_T = .362'' = L/604$

840



D+S

$V = 6.01k$

$M = 42.18$ k-ft

$\Delta_S =$ [illegible] $/EI$

$\Delta_T = 2414/EI$

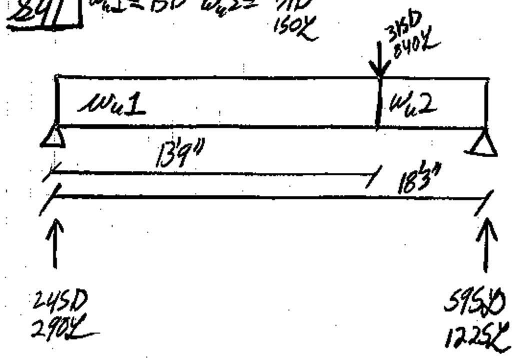
W14X34

$V_r = 59.5k$

$M_r = 49.31$ k-ft

$\Delta_T = .244'' = L/894$

841 $w_u1 = 15D$ $w_u2 = 71D$ 150L



D+L

$V = 1.82k$

$M = 5.95$ k-ft

$\Delta_L = 180.5/EI$

$\Delta_T = 285/EI$

$3\frac{1}{2}$X14 LSL

$V_r = 10.1k$

$M_r = 21.84$ k-ft

$\Delta_T = .229'' = L/952$

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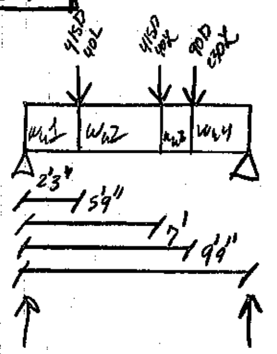
842



1505D 2625L — 1340D 2480L

w_u1 = 227D 570L

w_u2 = 218D 545L

w_u3 = 177D 435L

w_u4 = 185D 455L

D+L

V = 4.13k

M = 9.61 k·ft

ΔL = 94.6/EI

ΔT = 151.2/EI

3½x14 LsL

V_2 = 10.1k

M_2 = 21.8 k·ft

ΔT = .121" = L/918

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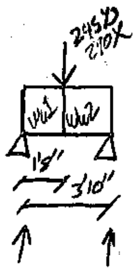
843 w_u1 = 90D 200L w_u2 = 230D 570L



395D 775L — 495D 1085L

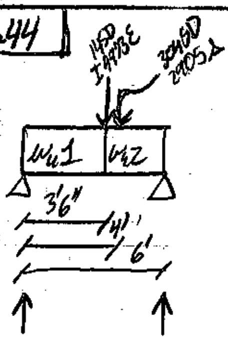
D+L

V = 1.6k

M = 1.56 k·ft

3½x14 LsL by inspection

844



1980D 2085L 970S ±1969E — 3165D 2085L 1935S ±3069E

w_u1 = 289D 695L

w_u2 = 383D 695L

D+S+L+1.75E

V = 11.9k

M = 23.68 k·ft

ΔT = 132/EI

5¼x14 P&L

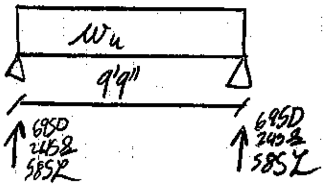
V_2 = 14.12k

M_2 = 40.74 k·ft

ΔT = .054" = L/1310

---

845 w_u = 142D 50S 120L



D+S+L

V = 1.52k

M = 3.71 k·ft

ΔT = 63.4/EI

3½x14 LsL

V_2 = 10.1k

M_2 = 21.8 k·ft

ΔT = .051" = L/2288

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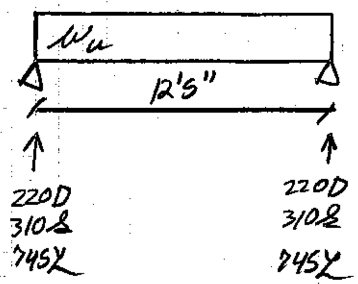
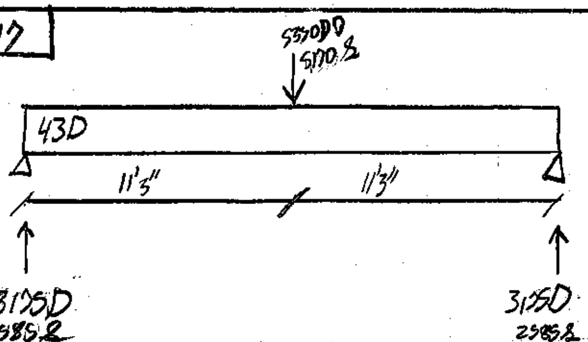
846 w_u = 35D, 50S, 120L



D+L
V = .96K
M = 2.99 K·ft
Δ_L = 64.1/EI
Δ_T = 82.9/EI

(3) 2X10 · Gr 'A'
V_r = 3.23K
M_r = 4.0K·ft
Δ_L = .175" = L/850
Δ_T = .226" = L/658

847

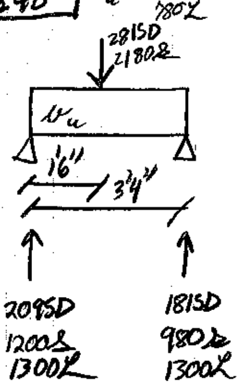


D+S
V = 5.8'K
M = 62.0 K·ft
Δ_S = 2120/EI
Δ_T = 4574/EI

W14X43
V_r = 59.5K
M_r = 62.4 K·ft
Δ_S = [illegible]
Δ_T = .368" = L/532

---

848 w_u = 327D, 780L



W14X39 pass by inspection

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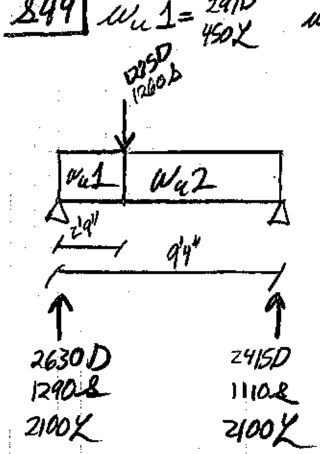
849 w_u1 = 291D, 450L w_u2 = 451D, 174S, 450L



D+S+L
V = 6.02K
M = 14.73K·ft
Δ_L = 129.6/EI
Δ_T = 230.4/EI

5 1/4 X 14 PSL
V_r = 14.2K
M_r = 40.7 K·ft
Δ_T = .095" = L/1167

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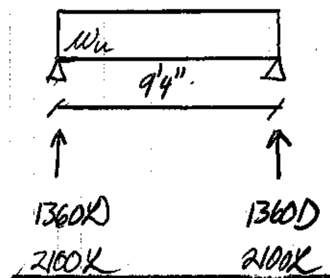
850 | w_u = 291D 450L

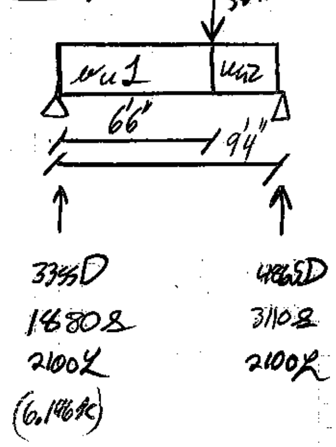


D+L
V = 3.46k
M = 8.07k-ft
Δ_T = 126.5/EI

3½x14 LSL
V_r = 10.1k
M_r = 21.8 k-ft
Δ_T = .102" = L/1097

851 | 4490D 3845S

w_u1 = 451D 174S 450L w_u2 = 291D 450L



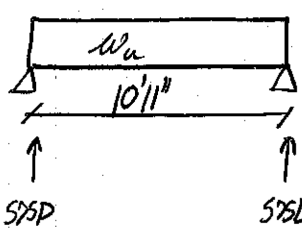
D+.75S+.75L
V = 8.68 k
M = 20.9 k-ft
Δ_L = 134.8/EI
Δ_T = 305.8/EI

5¼x14 PSL
V_r = 14.2k
M_r = 40.7k-ft
Δ_L =
Δ_T = .127" = L/879

852 | w_u = 105D

1¾ LSL rim pass by inspection

853 | w_u = 105D

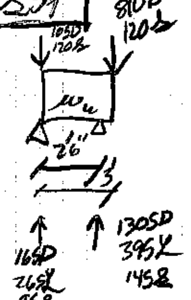


D
V = .58k
M = 1.57k-ft
Δ_T = 33.55/EI

1¾x14 LSL rim • Cd=.9 (CL=.341)
V_r = 4.55k
M_r = 3.35 k-ft
Δ_T = .054" = L/2420

854 | 810D 120S

w_u = 185D 220L



D+S+L
V = 1.12k
M = .52k-ft
Δ_T = .21/EI

3½x14 LSL by inspection

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SS5

P = 2415D / 1110S / 2100X | max ↓ = 4.82K

fit W21X11 good for 2305 #S

SS6

P = 2630D / 1290S / 2100X + 1360D / 2100X + 565D / 525S | 4555D / 1815S / 4200X   max ↓ = 9.06K — cant use any Strong Wall

SS7

P = 1360D / 2100X + 565D / 525S + 4615D / 2830S / 2100X   max ↓ ≥ 7.5K cant use any strong wall

SS8

P = 3245D / 1755S / 2100X | max ↓ = 6.13K fit W21X11 good for 2305 #S

Max shear = 4610 #S down garage line

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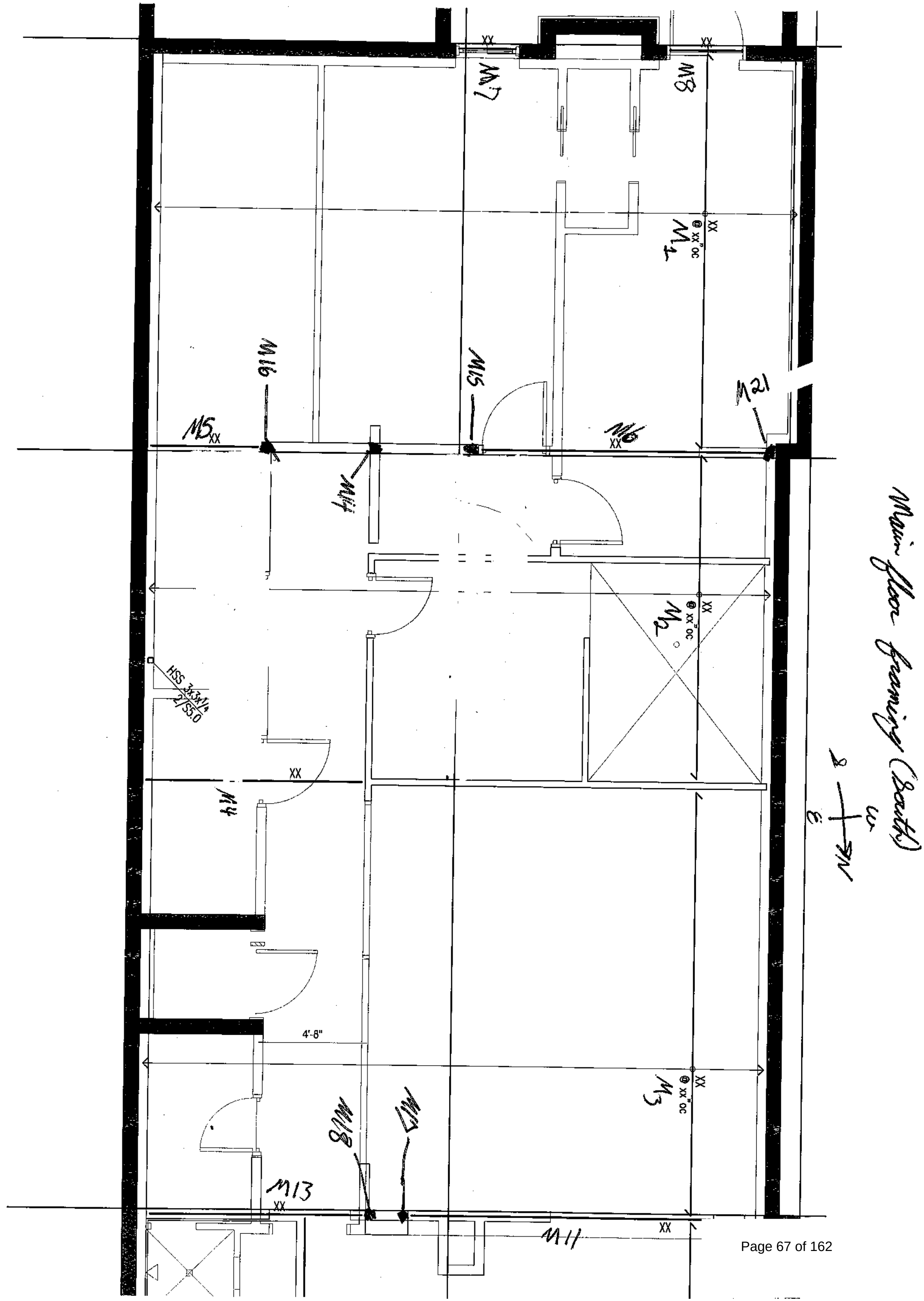
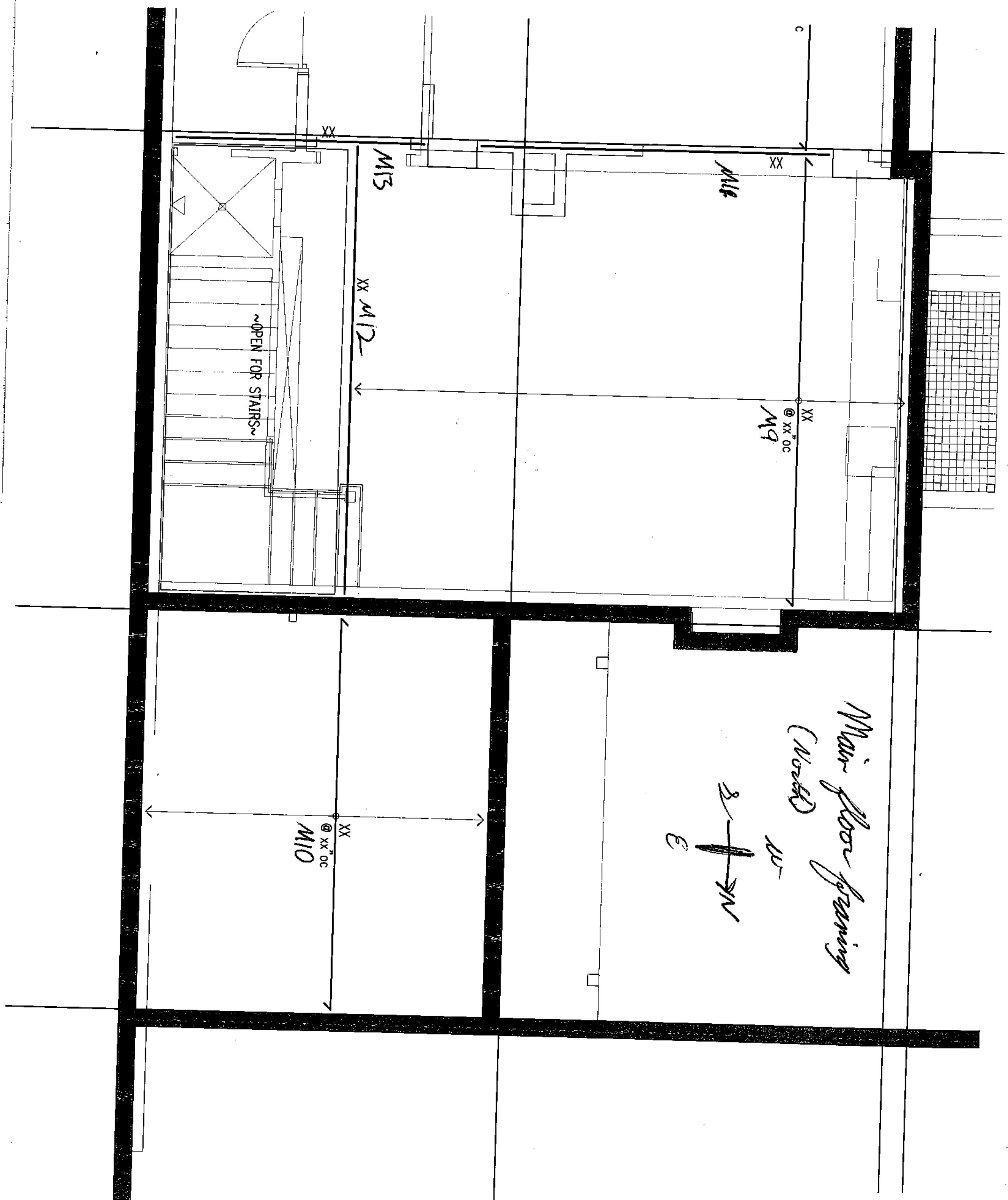

Main floor framing (South)
M1
M2
M3
M5 XX
M6 XX
M7
M8
M11
M13 XX
M15
M16
M17
M18
M21
HSS 3x3x1/4
2/S5.0
4'-8"

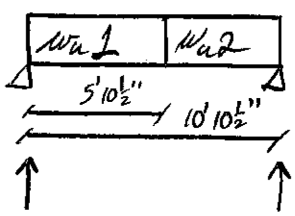

~OPEN FOR STAIRS~
XX M12
M13
M11
XX
@ XX" OC
M9
M10
Main floor framing
(North)

M1 See forte output 14" TJI 560 @12" oc

M2 See forte output 14" TJI 230 @12" oc

M3 See forte output (2) 1¾X14 LVL @16" oc

M4 w_u1 = 178D, 433L w_u2 = 280D, 709L

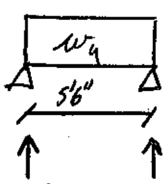


1085D, 2670L   1360D, 3410L

D+L
V = 4.77K
M = 11.51 K-ft
Δ_L = 173.9/EI
Δ_T = 243.9/EI

3½X14 LSL
V_r = 10.1K
M_r = 21.84 K-ft
Δ_L = .14" = L/930
Δ_T = .196" = L/663

M5 w_u = 268D, 675L

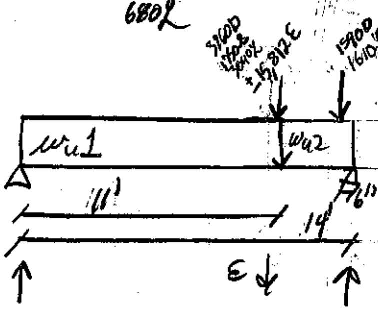


740D, 1855L   740D, 1855L

D+L
V = 2.6K
M = 3.57 K-ft
Δ_T = 19.41/EI

3½X14 LSL
V_r = 10.1K
M_r = 21.8 K-ft
Δ_T = .016" = L/4215

M6 w_u1 = 289D, 680L w_u2 = 621D, 1140L



2995D, 4355S, 5560L, ±3389 E(N/S)   7400 D, 2930 S, 8380 L, ±12,423E 3 E(N/S)

combo
6B V = 32.95 K
6B M = 85.8 K-ft
2 Δ_L = 812.6/EI
2 Δ_T = 1355 /EI

W14X34
V_r = 59.5K
M_r = 136.2 K-ft
Δ_T = .137" = L/1222

Max P before Web crippling = 65.1K over 3" — No web stiffener required

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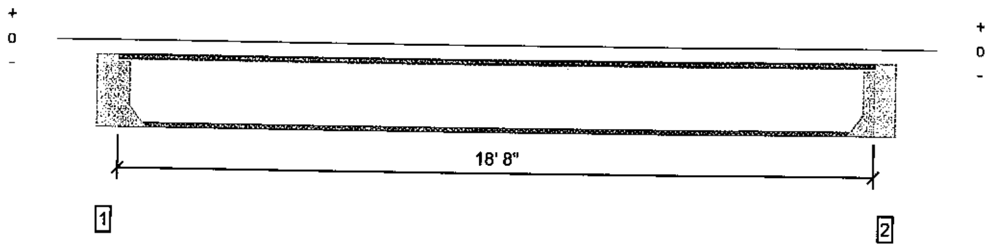
FORTE® **MEMBER REPORT** *Main Floor, M1*

PASSED

1 piece(s) 14" TJI® 560 @ 12" OC

Overall Length: 19' 8"



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

Member Reaction (lbs)	513 @ 6"	1265 (1.75")	Passed (41%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	513 @ 6"	2390	Passed (21%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2396 @ 9' 10"	11275	Passed (21%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.135 @ 9' 10"	0.280	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.185 @ 9' 10"	0.373	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	57	55	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 12' 9" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 18' 8" o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

1 - Hanger on 14" HF beam	6.00"	Hanger[1]	1.75" / - [2]	140	373	513	See note [1]
2 - Hanger on 14" HF beam	6.00"	Hanger[1]	1.75" / - [2]	140	373	513	See note [1]

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

1 - Face Mount Hanger	THF35140	2.50"	N/A	20-10d x 3	2-10d x 1-1/2	
2 - Face Mount Hanger	THF35140	2.50"	N/A	20-10d x 3	2-10d x 1-1/2	

1 - Uniform (PSF)	6" to 19' 2"	12"	15.0	40.0	Residential - Living Areas

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

SUSTAINABLE FORESTRY INITIATIVE

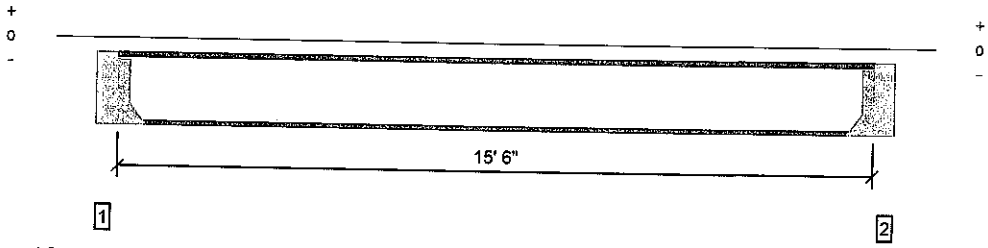
Forte Software Operator	Job Notes
Jonathan Carlson Harriott Valentine Engineers (206) 624-4760 jcarlson@harriottvalentine.com	

FORTE® **MEMBER REPORT** *Main Floor, M2*

PASSED

1 piece(s) 14" TJI® 230 @ 12" OC

Overall Length: 16' 6"



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	426 @ 6"	1060 (1.75")	Passed (40%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	426 @ 6"	1945	Passed (22%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1652 @ 8' 3"	4990	Passed (33%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.112 @ 8' 3"	0.233	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.154 @ 8' 3"	0.310	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	56	55	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 7' 4" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 15' 6" o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on 14" HF beam	6.00"	Hanger[1]	1.75" / - [2]	116	310	426	See note [1]
2 - Hanger on 14" HF beam	6.00"	Hanger[1]	1.75" / - [2]	116	310	426	See note [1]

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Nails	Face Nails	Member Nails	Accessories
1 - Face Mount Hanger	THF23140	2.50"	N/A	18-10d x 3	2-10d x 1-1/2	
2 - Face Mount Hanger	THF23140	2.50"	N/A	18-10d x 3	2-10d x 1-1/2	

Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	6" to 16'	12"	15.0	40.0	Residential - Living Areas

Weyerhaeuser Notes

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

SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
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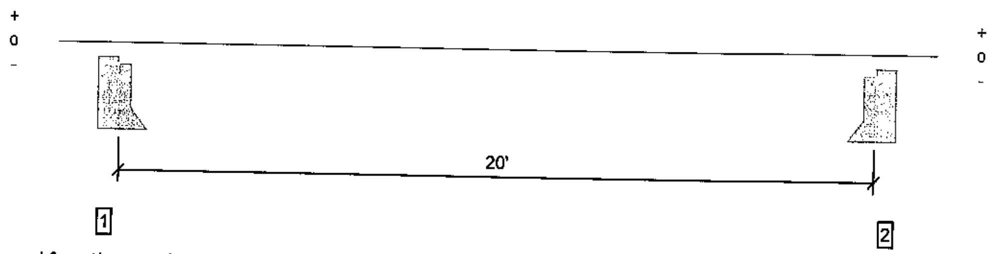
FORTE® MEMBER REPORT *Main Floor, M3* **PASSED**

2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL @ 16" OC

Overall Length: 21'



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	693 @ 6"	3938 (1.50")	Passed (18%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	612 @ 1' 8"	9310	Passed (7%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3467 @ 10' 6"	25229	Passed (14%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.121 @ 10' 6"	0.300	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.157 @ 10' 6"	0.400	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	58	55	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 20' o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 20' o/c unless detailed otherwise.
- A 4% increase in the moment capacity has been added to account for repetitive member usage.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on 14" HF beam	6.00"	Hanger[1]	1.50"	168	560	728	See note [1]
2 - Hanger on 14" HF beam	6.00"	Hanger[1]	1.50"	168	560	728	See note [1]

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Nails	Face Nails	Member Nails	Accessories
1 - Face Mount Hanger	JUS410	2.00"	N/A	8-16d x 3-1/2	6-16d x 3-1/2	
2 - Face Mount Hanger	JUS410	2.00"	N/A	8-16d x 3-1/2	6-16d x 3-1/2	

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 21'	16"	12.0	40.0	Residential - Living Areas

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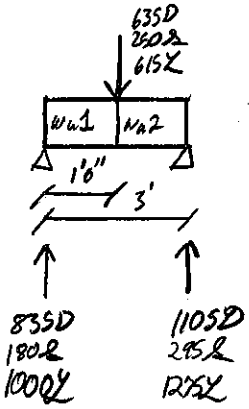
M7 $w_u1 = 253D,\ 370L$ $w_u2 = 618D,\ 149S,\ 740L$



D+S+L

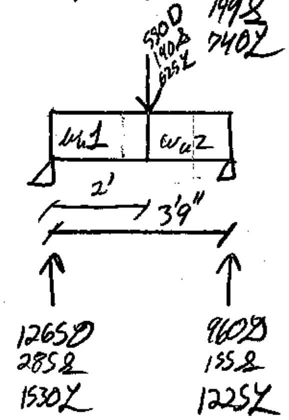
$V = 2.68k$

$M = 2.32k\cdot ft$

$\Delta_T = 3.4/EI$

$3\frac{1}{2} \times 14$ LSL pass by inspection

M8 $w_u1 = 618D,\ 149S,\ 740L$ $w_u2 = 253D,\ 370L$



D+S+L

$V = 3.1k$

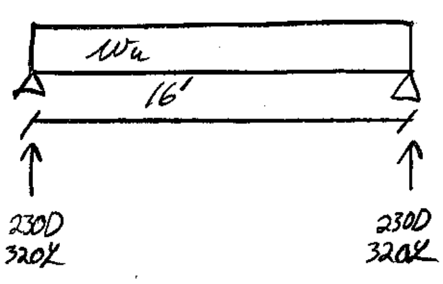
$M = 3.2k\cdot ft$

$\Delta_T = 7.52/EI$

$3\frac{1}{2} \times 14$ LSL pass by inspection

M9 See forte output – 14" TJI 230 @16" oc

M10 $w_u = 29D,\ 40L$



D+L @16" oc

$V = .74k$

$M = 2.95k\cdot ft$

$\Delta_L = 78.64/EI$

$\Delta_T = 135.6/EI$

$1\frac{3}{4} \times 11\frac{1}{4}$ @16" oc

$V_r = 3.74k$

$M_r = 8.07k\cdot ft$

$\Delta_T = .326" = L/588$ – fail

$1\frac{3}{4} \times 11\frac{1}{4}$ @12" oc

$\Delta_T = .245" = 785$

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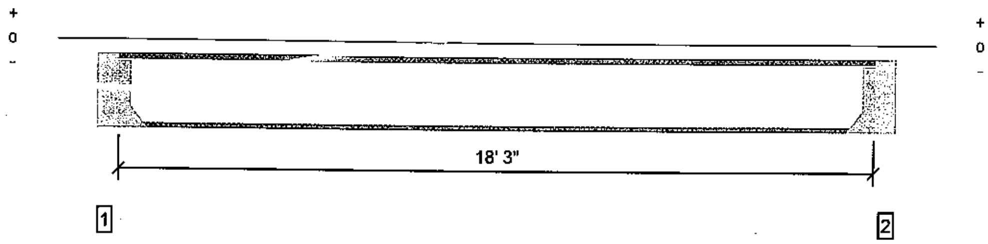
FORTE® **MEMBER REPORT** *Main Floor, M9* **PASSED**

1 piece(s) 14" TJI® 560 @ 12" OC

Overall Length: 19' 3"



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	475 @ 6"	1265 (1.75")	Passed (38%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	475 @ 6"	2390	Passed (20%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2165 @ 9' 7 1/2"	11275	Passed (19%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.124 @ 9' 7 1/2"	0.274	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.161 @ 9' 7 1/2"	0.365	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	58	55	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/800) and TL (L/600).
- Top Edge Bracing (Lu): Top compression edge must be braced at 13' 5" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 18' 3" o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

Supports	Bearing Length			Loads to Supports (lbs)			
	Total	Available	Required	Dead	Floor Live	Total	Accessories
1 - Hanger on 14" HF beam	6.00"	Hanger[1]	1.75" / - [2]	116	385	501	See note [1]
2 - Hanger on 14" HF beam	6.00"	Hanger[1]	1.75" / - [2]	116	385	501	See note [1]

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Nails	Face Nails	Member Nails	Accessories
1 - Face Mount Hanger	THF35140	2.50"	N/A	20-10d x 3	2-10d x 1-1/2	
2 - Face Mount Hanger	THF35140	2.50"	N/A	20-10d x 3	2-10d x 1-1/2	

Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 19' 3"	12"	12.0	40.0	Residential - Living Areas

Weyerhaeuser Notes

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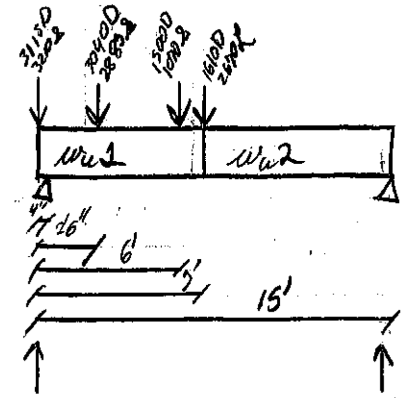
MB1 $w_u 1 = \frac{624D}{1335L}$ $w_u 2 = \frac{410D}{765L}$



D+L
$V = 18.6K$
$M = 64K \cdot ft$
$\Delta_L = 1349/EI$
$\Delta_T = 2335.5/EI$

W 14X34 (d=14")
$V_r = 59.5K$
$M_r = 136.2K \cdot ft$
$\Delta_L = .136" = L/1286$
$\Delta_T = .236" = L/743$

11,895D
10,450L
6400S

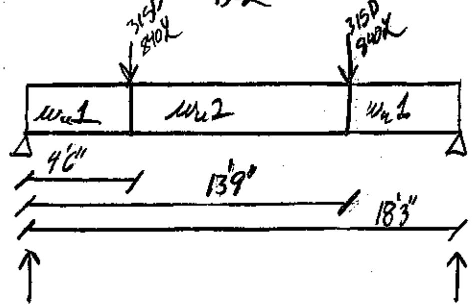
5020D
7685L
765S

Max P↓ = 24.54K
Max P↓ 12205K — Min 6X8 bearing

H = 9' HSS 3X3X¼ good for 35.6K

6X8 Hf#1 H=11'
Axial Capacity = 20.49K ✓

MB2 $w_u 1 = \frac{71D}{150L}$ $w_u 2 = 15D$



D+L
$V = 2.22K$
$M = 39.1K \cdot ft$
$\Delta_L = 354.7/EI$
$\Delta_T = 525/EI$

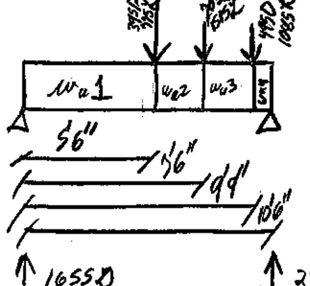
3½X14 LSL
$V_r = 10.1K$
$M_r = 21.8K \cdot ft$
$\Delta_T = .423" = L/517$ — fail

5¼X14 PSL
$V_r = 14.21K$
$M_r = 13.25K \cdot ft$
$\Delta_T = .218" = L/1001$

705D
1515L

705D
1515L

MB3 $w_u 1 = \frac{238D}{535L}$ $w_u 2 = \frac{165D}{400L}$ $w_u 3 = \frac{306D}{770L}$ $w_u 4 = \frac{602D}{1343L}$



D+L
$V = 8.58K$
$M = 16.38K \cdot ft$
$\Delta_L = 211.1/EI$
$\Delta_T = 318.4/EI$

3½X14 LSL
$V_r = 10.1K$
$M_r = 21.8K \cdot ft$
$\Delta_L = .17" = L/740$
$\Delta_T = .256" = L/490$

↑ 1655D
2890L

↑ 2715D
5870L

} — 6X6 Hf #1

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M14 P = 2185D, 490S, 2990L, ±10,999E N/S

Max ↓ = 14.8K Min 6X8 for Bearing

Axial 6X8 Hf #1 = 25.24K H=9'

M15 P = 2925D, 985S, 2975L, -1107E + 3990D, 4010S + at 29 of M1 2995D, 1135S, 5560L + 13904E + 11935E, -3309E | 9890D, 5430S, 8535L, ±20,593E (M/S)

E W; E ↑ → ↓

Max ↓ = 32.16 K (830) Max ↑ 9.89 K (894) (Asd) Lrfd = 13.57 K ↑, 43.96 K ↓ (M6)

H=9' Hss 3X3X¼ 35.6 K ✓

M16 P = 240D, 1855L, ±11,935E (M15)

Max ↓ 9.19K, Max ↑ 8.01K

Lrfd max ↓ 14.87K, Max ↑ 11.413K

6X6 Hf #1 for bearing Axial = 19.15K ✓

M17 P = 11,895D, 10,450L, 6400S

Max ↓ = 24.25K H=9' Axial Capacity Hss 3X3X¼ = 35.6K

M18 P = 2715D, 5870L Max ↓ = 8.59K (3)2X6

M21

Max ↓ (Asd) = 21.49 K

Hss 3X3X¼ H=9' Axial = 35.6 K ✓

Max ↑ (Asd) = 5.67 K

Max ↓ (Lrfd) = 30.28 K

Max ↑ (Lrfd) = 7.82 K

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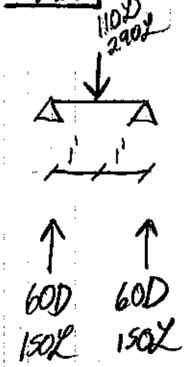
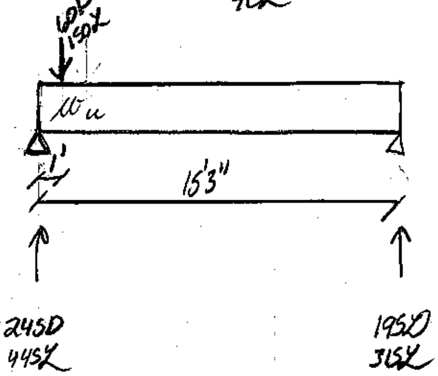
M19



$3\frac{1}{2}$X14 LSL pass by inspection

M20

w_u = 25D
40L



D+L

$V = .7^k$

$M = 2$ k, ft

$\Delta_L = 52.4/EI$

$\Delta_T = 84.3/EI$

$3\frac{1}{2}$X14 LSL

$V_r = 10.1^k$

$M_r = 21.8$ k, ft

$\Delta_L =$

$\Delta_T = .067" = L/2691$

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WF to Wood connection

Bolt size = 5/8" ϕ Min End distance = 4D ($2\frac{1}{2}$")

Min spacing in row = 3d ($1\frac{7}{8}$") @3" ✓

Min spacing between rows = 5d ($3\frac{1}{8}$")

Min edge distance = 4d ($2\frac{1}{2}$")

Capacity 5/8" ϕ into $3\frac{1}{2}$" = 1207 #S / Bolt } w/ (6) Bolts = 7242#S
" " " $5\frac{1}{4}$" = 1207 #S / Bolt

Greatest Load = 5415 #S ✓

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Hss → WF connection

Hss → $\frac{1}{2}$" steel PL

3/16 fillet all around

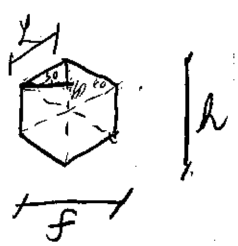
$L = (2.5)(4) +$ corners

$L = 10$" Shear capacity = 27.9 K

Tensile capacity of Bolt = 13.8 K / Bolt (Table 7-2)

F = 1 1/16 for 5/8"

$L = \sqrt{((.5)(1\tfrac{1}{16}))^2 + ((.75)(1\tfrac{1}{16}))^2} = .593$ in



$\text{Area} = (L)(f) + (f)((.25)(f)) =$

$(.593)(1.0625) \quad (1.0625)^2(.25) = .9133 \text{ in}^2$

$\text{Area hole} = \left(\frac{11}{32}\right)^2 \pi = .3712 \text{ in}^2$

$\text{Area Bearing} = .5421 \text{ in}^2 \quad \text{Bearing} = \frac{\text{area} \cdot f_y}{\Omega} = \frac{(.5421)(36)}{2}$

9.75 K/Bolt

$\text{shear capacity PL} = \frac{\pi(f)(t)(f_y)}{\Omega} = \frac{60}{\Omega} = 30$ K/Bolt

shear yielding

Max T/Bolt = 9.75 K

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F

<u>Kss → WF Connection Continued</u>

$M = \frac{(9.75)(1.5)(2)}{12} = 2.43$ k·ft   Mr R = .76 k·ft

actual demand = 8.5k/4 = 2.125 k/Bolt

$M = \frac{(2.125)(2)(1.5)}{12} = .531$ k·ft ≥ .76 k·ft

Max T total for connection = <u>12.16 k</u> governed by moment

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F

<u>Deflection Calcs - Camber</u>

$L = 29'9''$

Min Camber $= \frac{1}{8}'' + \frac{\left(\frac{L}{10}\right)}{8}$ ← 8ths of an in
(L in feet)

Min C = .4968''

Max $\Delta = d$

$d = 3/8''$

$\Delta_D = (\text{min Camber}/(.8))$ ← total

$\Delta_D = \frac{\text{min Camber}}{(.8)}(.2)$ ← Effective $\frac{.2}{.8} = (.25)$

$\Delta_D = .621''$ ← total

$\Delta_D = .124''$ ← Effective

$\Delta D = (.25)(\text{min Camber})$

$\Delta_L + \Delta_D \le d$

$\Delta_L \le d - (.25)\text{min Camber}$

$\Delta_L + \Delta_D = d$

$\Delta_L = .375 - .124 = .2505''$

Ratio (min) $= \frac{\Delta_D}{\Delta_L} = \frac{\text{min Camber}/.8}{d - .2\text{ min Camber}}$

Ratio $= \frac{.621}{.2505} = \underline{2.47}$

Δ_D govern all so Δ_L must be less than or equal to min ratio because as Δ_D increases Δ_L must decrease, min Δ_D though to achieve = min Camber/.8

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Estimated shear capacity of 5/8" welded stud.
Assume 3/16 fillet with $L = (2)(5/8) = \underline{1.25 \text{ in}}$
$R_n = 3.4$ K / stud

M6 – Calcs

Need to transfer 4.72 K

tolerance factor

f'c of HF = 1.3 ksi projected area = $(5/8)(1\frac{1}{2})(.9)$

fc / stud = $(1.3)(\frac{5}{8})(1\frac{1}{2})(.9) = 1.096$ K/stud

Max L = 29" – 6" = 27"

need 5 studs

space @ 5" oc
we get 5.4 studs ✓

M15 – Calcs

$5\frac{1}{2} - 3 = 2\frac{1}{2}$
$2\frac{1}{2}/2 = 1\frac{1}{4}$"

DF sides
Need to transfer 10,860 #S & 12,655 #S

f'c = 1.35 ksi Projected Bearing = $(1.35)(\frac{5}{8})(3-1) = 1.68$ K

get 1685 #S / stud

min # studs = 7 studs over (8')(2) space max 27" oc
min # studs = 8 studs over (11')(2) space max 33" oc

space @ 24" oc

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Wall Ledgers

Ledger need to be $3\frac{1}{2}$X14 for 16d nails (Hangers) X Joist depth

$3\frac{1}{2}$x14 NL = 13D

Wu=13D+137D | 150D > 1.2D+1.6L
365L | 365L   764 plf   (764) $\frac{32}{12}$ = 2038

5/8" J bolts @ 32" oc provides 1893 plf ✓

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Design Method	Allowable Stress Design (ASD)
Connection Type	Lateral loading
Fastener Type	Bolt
Loading Scenario	Single Shear - Wood Main Member

Main Member Type	Douglas Fir-Larch
Main Member Thickness	-- Other (in inches) -- 3.5
Main Member: Angle of Load to Grain	0
Side Member Type	Steel
Side Member Thickness	1/4 in.
Side Member: Angle of Load to Grain	0
Fastener Diameter	5/8 in.
Load Duration Factor	C_D = 1.0
Wet Service Factor	C_M = 1.0
Temperature Factor	C_t = 1.0

Connection Yield Modes

Im	3062 lbs.
Is	3398 lbs.
II	1420 lbs.
IIIm	1776 lbs.
IIIs	1207 lbs.
IV	1534 lbs.

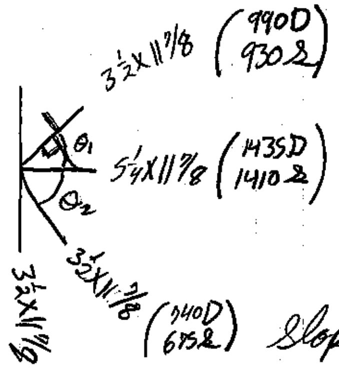
Adjusted ASD Capacity	**1207 lbs.**

- Bolt bending yield strength of 45,000 psi is assumed.
- The Adjusted ASD Capacity is only applicable for bolts with adequate end distance, edge distance and spacing per NDS chapter 11.
- ASTM A36 Steel is assumed for steel side members 1/4 in. thick, and ASTM A653 Grade 33 Steel is assumed for steel side members less than 1/4 in. thick.

The Connection Calculator was designed and created by Cameron Knudson, Michael Dodson and David Pollock at Washington State University. Support for development of the Connection Calculator was provided by American Wood Council.

Custom hanger

Slope = $\frac{7'9\frac{1}{2}''}{18'3''} = \frac{93.5}{219} = \frac{4.986}{12}$ = 24.55° slope



$\theta_1 = 57.12°$

$\theta_2 = 71.89°$

Slope = $\frac{5'1\frac{3}{4}''}{16'} = \frac{3.859}{12}$ = 18.75° slope

Min Bearing psi = 625

total = 3165D / 3015S ≻ 6180 #S Needs 9.88 in²

≈ 6x2 = (12)(625) = 7500 #S ✓ (L 4x3 x ¼)

(3½)(2)(625) = 4375 #S ≥ (990+930) ✓

(5¼)(2)(625) = 6562 #S ≥ (1435+1410) ✓

5% Lateral = (7500)(.05) = 375 #S

10D nail = 162 #S @ Cd=1.0 Min of 2 nails @ Cd=1.6 ✓ per member.

Min Weld = 4" = 11.1 K ✓

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Design Method	Allowable Stress Design (ASD)
Connection Type	Lateral loading
Fastener Type	Nail
Loading Scenario	Single Shear

Main Member Type	Douglas Fir-Larch
Main Member Thickness	3.5 in.
Side Member Type	Steel
Side Member Thickness	1/4 in.
Nail Type	Common Wire
Nail Size	10d (D = 0.148 in.; L = 3 in.)
Load Duration Factor	C_D = 1.0
Wet Service Factor	C_M = 1.0
End Grain Factor	C_eg = 1.0
Temperature Factor	C_t = 1.0
Diaphragm Factor	C_di = 1.0

Connection Yield Modes

Im	860 lbs.
Is	1463 lbs.
II	376 lbs.
IIIm	362 lbs.
IIIs	228 lbs.
IV	162 lbs.

Adjusted ASD Capacity	**162 lbs.**

- Nail bending yield strength of 90000 psi is assumed.
- The Adjusted ASD Capacity does not apply for toe-nails installed in wood members.
- Length of tapered tip is assumed to be two times the nail diameter for calculating dowel bearing length in the main member.
- The Adjusted ASD Capacity only applies for nails that have been driven flush with the side member surface. It does not apply for nails that have been overdriven into the side member.
- ASTM A36 Steel is assumed for steel side members 1/4 in. thick, and ASTM A653 Grade 33 Steel is assumed for steel side members less than 1/4 in. thick.

While every effort has been made to insure the accuracy of the information presented, and special effort has been made to assure that the information reflects the state-of-the-art, neither the American Wood Council nor its members assume any responsibility for any particular design prepared from this on-line Connection Calculator. Those using this on-line Connection Calculator assume all liability from its use.

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Design Method	Allowable Stress Design (ASD) ▼
Connection Type	Lateral loading ▼
Fastener Type	Bolt ▼
Loading Scenario	Double Shear - Steel Main Member ▼

Main Member Type	Steel ▼
Main Member Thickness	1/4 in. ▼
Main Member: Angle of Load to Grain	0
Side Member Type	Douglas Fir-Larch ▼
Side Member Thickness	1.5 in. ▼
Side Member: Angle of Load to Grain	0
Fastener Diameter	1/2 in. ▼
Load Duration Factor	C_D = 1.0 ▼
Wet Service Factor	C_M = 1.0 ▼
Temperature Factor	C_t = 1.0 ▼

Connection Yield Modes

Im	2719 lbs.
Is	2100 lbs.
IIIs	1566 lbs.
IV	1963 lbs.

Adjusted ASD Capacity	**1566 lbs.**

- Bolt bending yield strength of 45,000 psi is assumed.
- The Adjusted ASD Capacity is only applicable for bolts with adequate end distance, edge distance and spacing per NDS chapter 11.
- ASTM A36 Steel is assumed for 1/4 in. and thicker steel main members , and ASTM A653 Grade 33 Steel is assumed for steel main members less than 1/4 in. thick.

While every effort has been made to insure the accuracy of the information presented, and special effort has been made to assure that the information reflects the state-of-the-art, neither the American Wood Council nor its members assume any responsibility for any particular design prepared from this on-line Connection Calculator. Those using this on-line Connection Calculator assume all liability from its use.

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SECTION 3: LATERAL

USGS Design Maps Summary Report

User–Specified Input

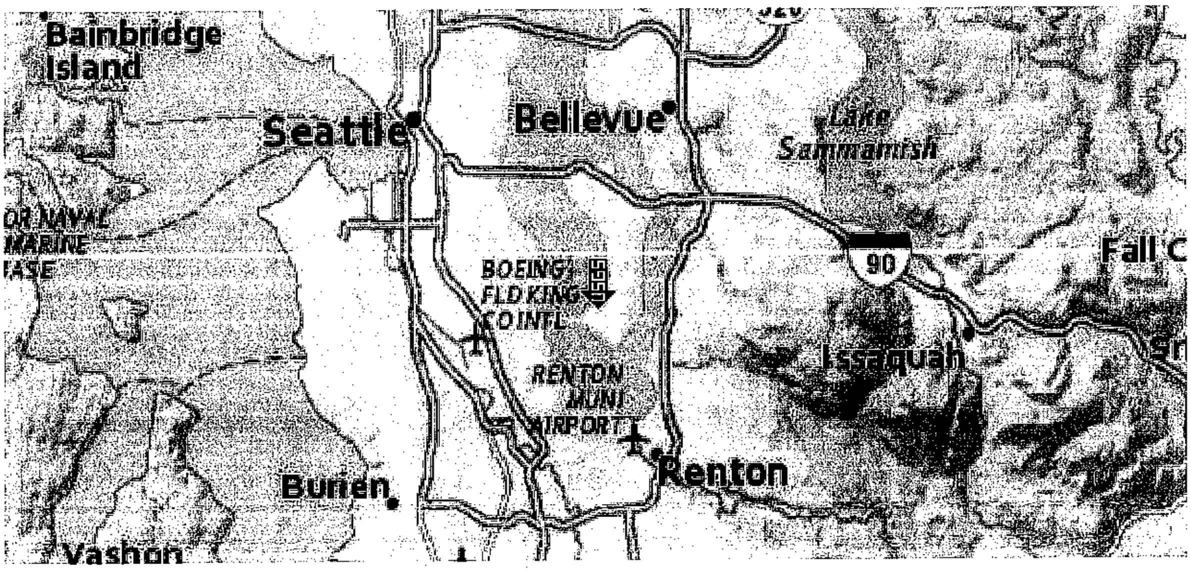
Report Title Hart Residence
Wed January 3, 2018 18:16:46 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 47.54873°N, 122.23501°W

Site Soil Classification Site Class D – "Stiff Soil"

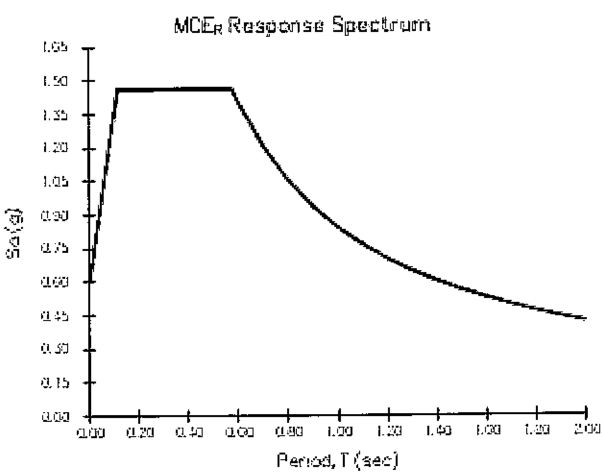
Risk Category I/II/III

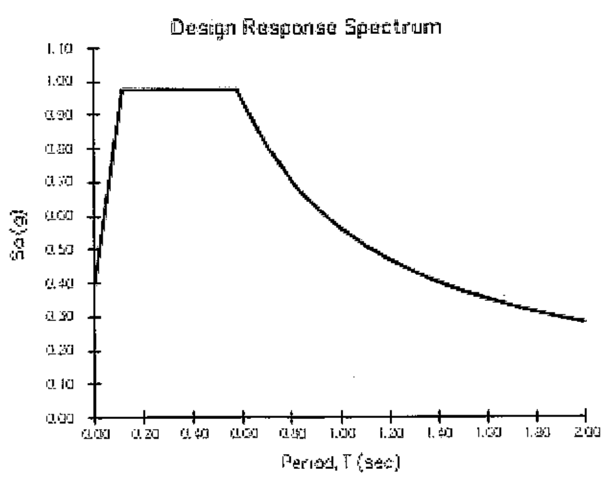


USGS–Provided Output

S_S =	1.463 g	S_{MS} =	1.463 g	S_{DS} =	0.975 g
S_1 =	0.562 g	S_{M1} =	0.843 g	S_{D1} =	0.562 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

SEISMIC DESIGN

Tangled Ride Residence (North South)
ASCE 7-10
Equivalent Lateral Force Procedure

Occupancy Category	II	Table 1-1
Seismic Design Category	D	Table 11.6-1
Importance Factor	1.00	Table 11.5-1
Site Class	D	Table 20.3-1
S_s	146.30 %g	(from USGS Seismic Hazard Curves, 2008 data)
S_1	56.20 %g	(from USGS Seismic Hazard Curves, 2008 data)
F_a	1.00	Table 11.4-1
F_v	1.50	Table 11.4-2
C_t	0.02	Table 12.8-2
x	0.75	Table 12.8-2
h_n	25.00 feet	(height to highest level)

$S_{MS} = F_a*S_s$	1.4630	Eq. 11.4-1
$S_{M1} = F_v*S_1$	0.8430	Eq. 11.4-2
$S_{DS} = (2/3)*S_{MS}$	0.9753 g	Eq. 11.4-3
$S_{D1} = (2/3)*S_{M1}$	0.5620 g	Eq. 11.4-4
Period $T_a = C_t*h_n$^x	0.2236 s	Eq. 12.8-7
T_o	0.1152 s	per section 11.4.5
T_S	0.5762 s	per section 11.4.5
S_a	0.9753 g	per section 11.4.5

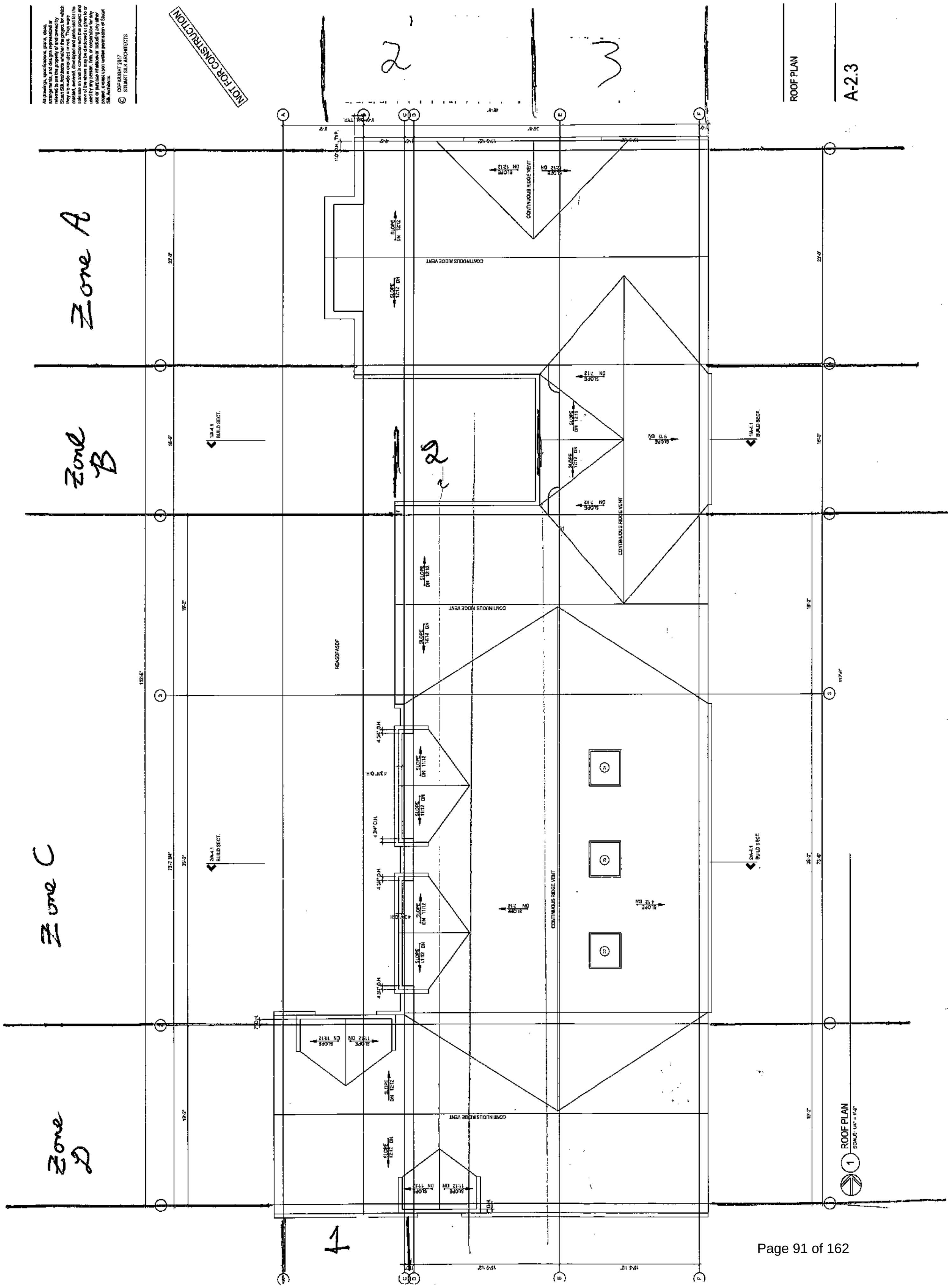
R	6.5	Table 12.2-1
Ωo	2.5	Table 12.2-1
C_d	4	Table 12.2-1
Section 9.5.5 ok?	Yes	Table 12.6-1

Equivalent Lateral Force Procedure (section 12.8)

Cs	0.1501	Eq. 12.8-2
W, weight	282,595 lb	per table below
Q_E	42,404 lb	Eq. 12.8-1

Vertical Force Distribution (section 12.8.3)
k = 1.00

Level	Hx	Floor Area	Seismic Dead Ld	Floor Wt.	Wall Length	Wall Wt.	Total Wt.	WxHx	Cvx	(LRFD) Q_E	(ASD) $0.7Q_E$
	(ft)	(ft2)	(psf)	(k)	(ft)	(k)	(k)	(k-ft)	(%)	(k)	(k)
Roof	25.00	4190	23	96.4	680	38.8	135.1	3378.3	75.0	31.81	22.27
Second Floor	12.33	3750	15	56.3	630	35.0	91.2	1125.0	25.0	10.59	7.42
Main	0.00	3750	15	56.3	630	0.0	56.3	0.0	0.0	0.00	0.00
							282.60	4503.24	100.00	42.40	**29.68**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ROOF PLAN
A-2.3
Zone A
Zone B
Zone C
Zone D
CONTINUOUS RIDGE VENT
ROOF PLAN
SCALE: 1/4" = 1'-0"

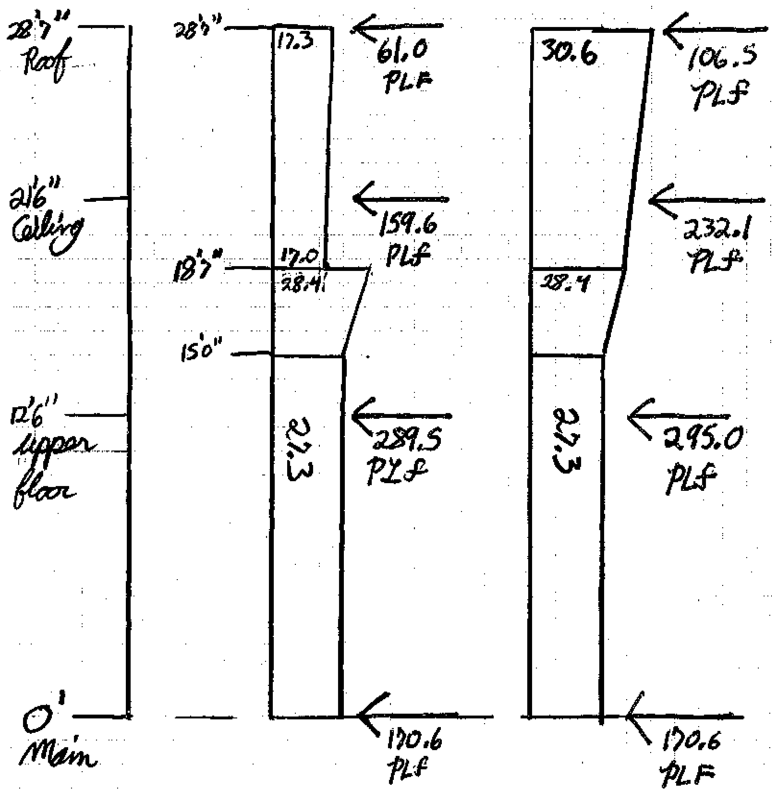
Winds – Directional Procedure

Exposure D

$K_{zt} = 1.0$ (Below halfway point of hill)

Wind Pressures Zone A (Pressures Contain Leeward & Windward pressures @ 1.0 w)

East/West $L = 36'8''$
$B = 23'$



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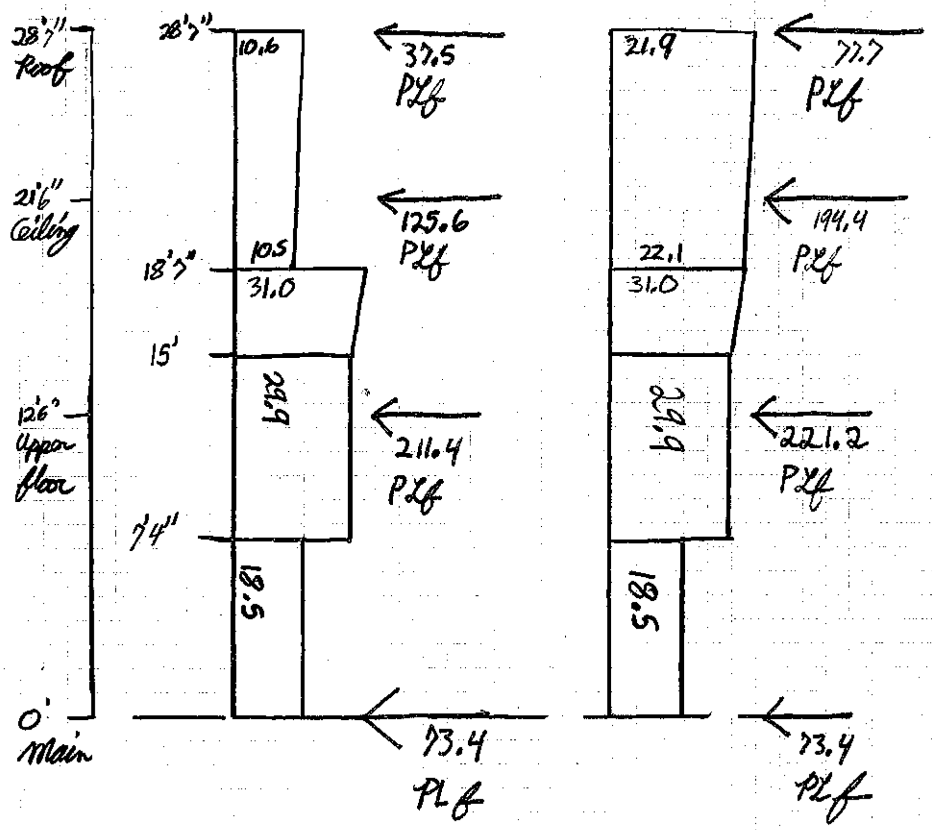
Wind Pressures Zone B (Pressures contain any applicable Leeward & Windward pressures @1.0W)

East/West

L= 16'8"

B= 16'



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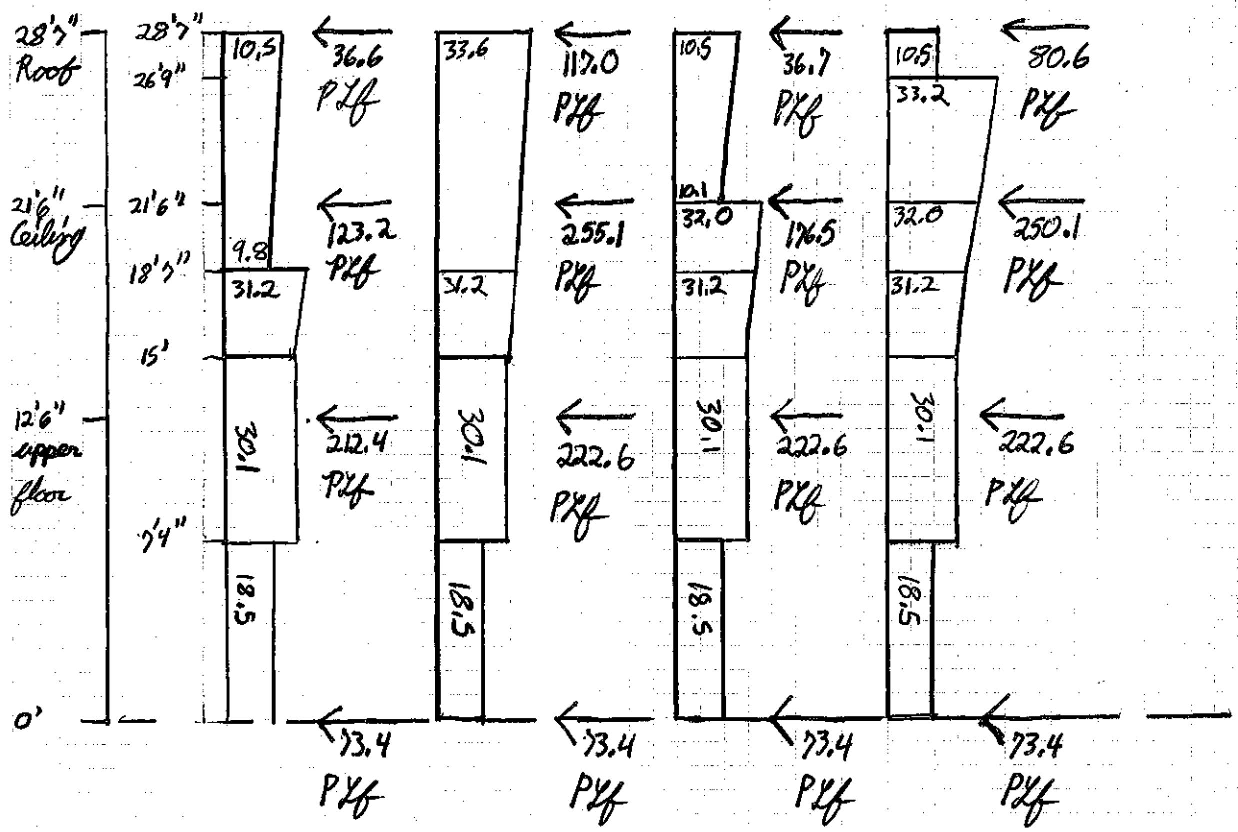
Wind Pressures Zone C (Pressures Contain any apliable Leeward & windward pressures @ 1.0 W)

East/West

L= 32'3"

B= 54'4"



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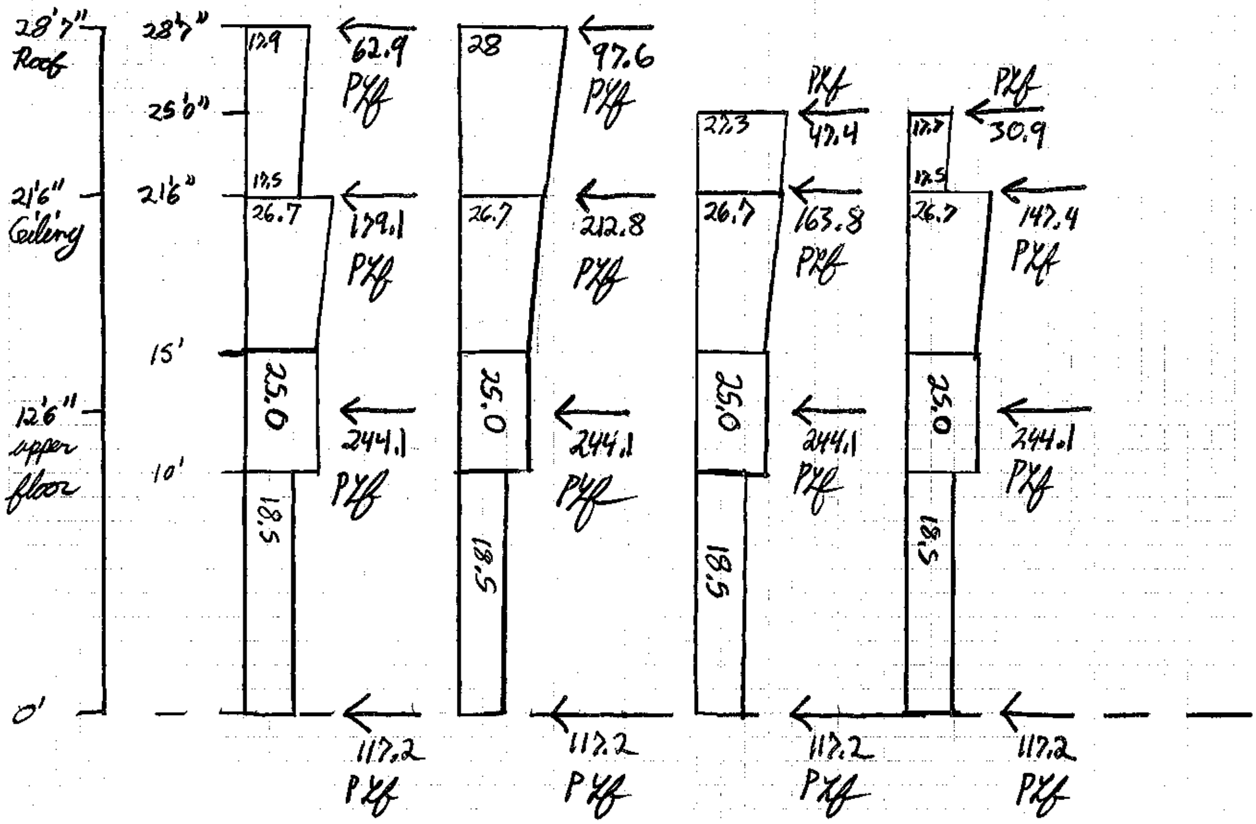
Wind Pressures Zone D (Pressures contain any aplicable Leeward & windward pressures @ 1.0 W)

East/West

L = 45'5"

B = 19'2"



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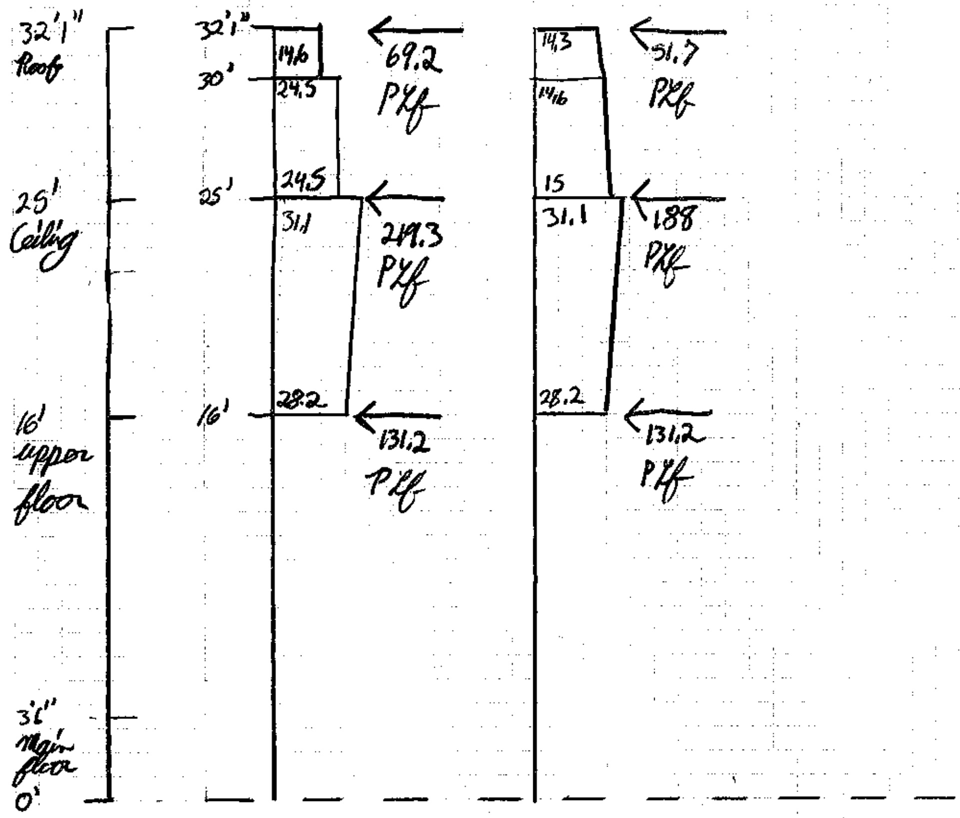
Wind Pressures Zone 1 (Pressures contain any aplicable leeward & windward pressures)

North / South

$\alpha = 19'2"$

$\beta = 14'2"$



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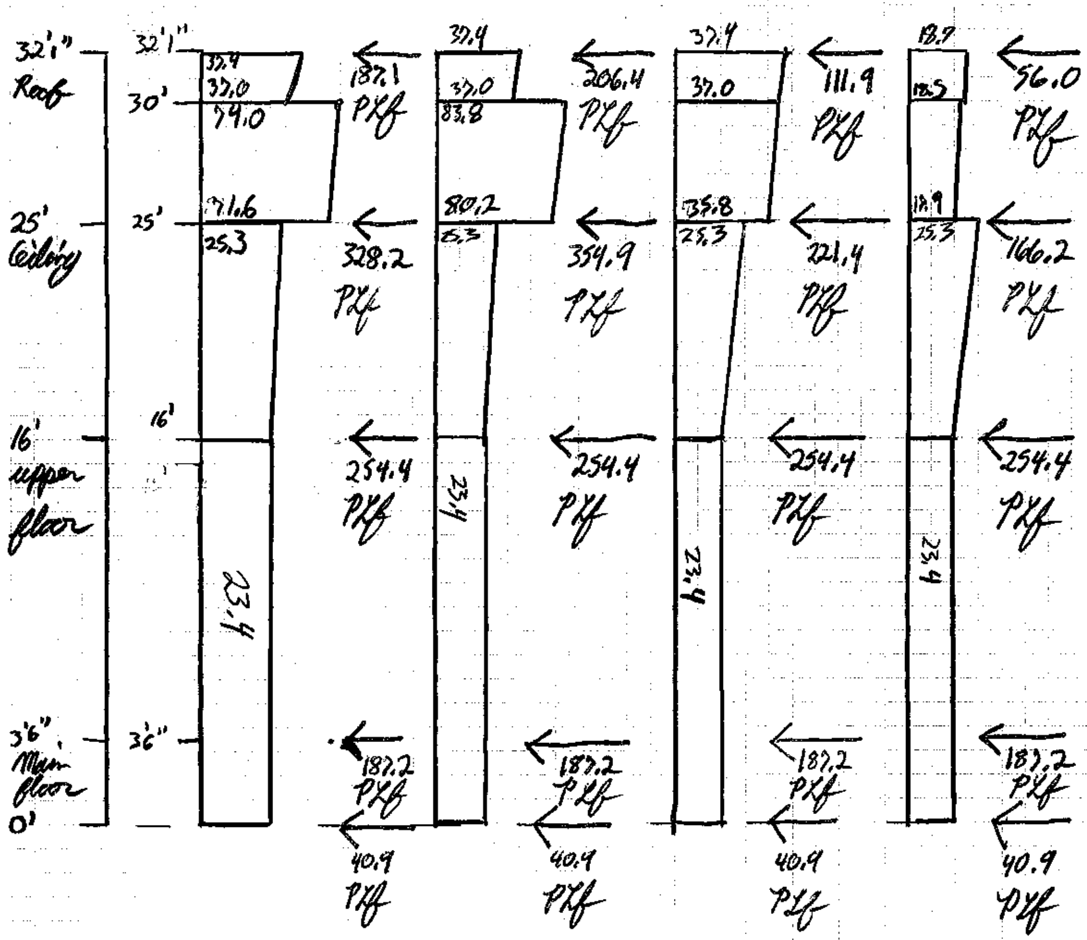
Wind Pressures Zone 2 (Pressures contain any applicable leeward & windward pressures @ 1.0 W)

North / South

L = 73'6"

B = 15'7"



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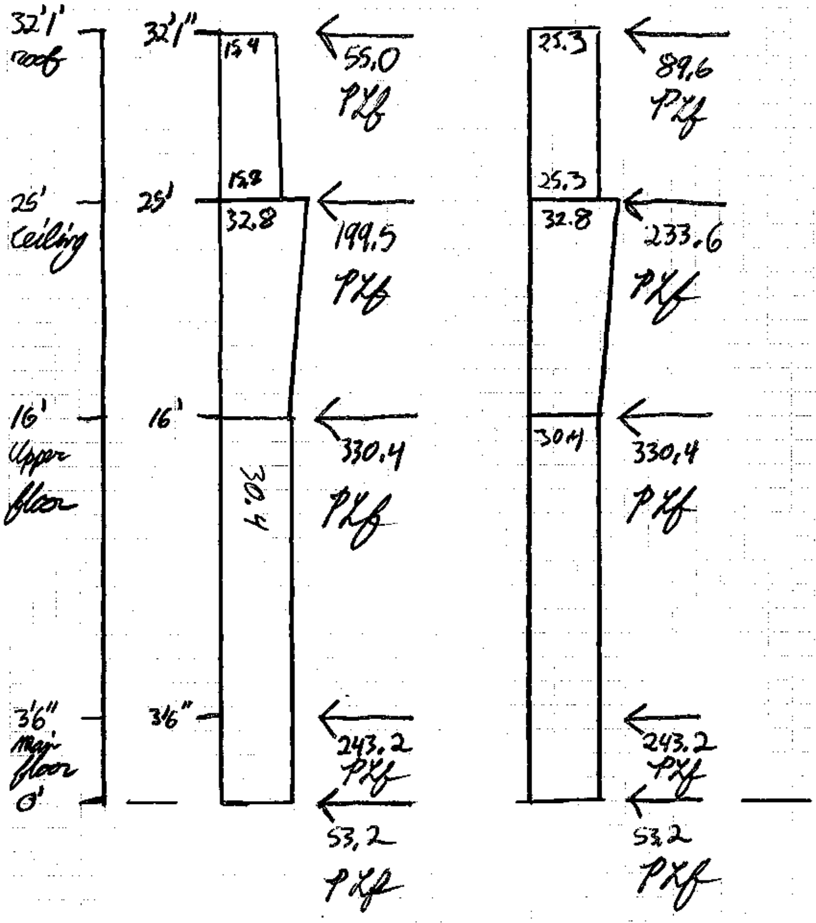
Wind Pressures Zone 2' (Pressures contain any aplicable leeward & windward pressures @ 1.0 W)

North/South

L = 23'

B = 23'3"



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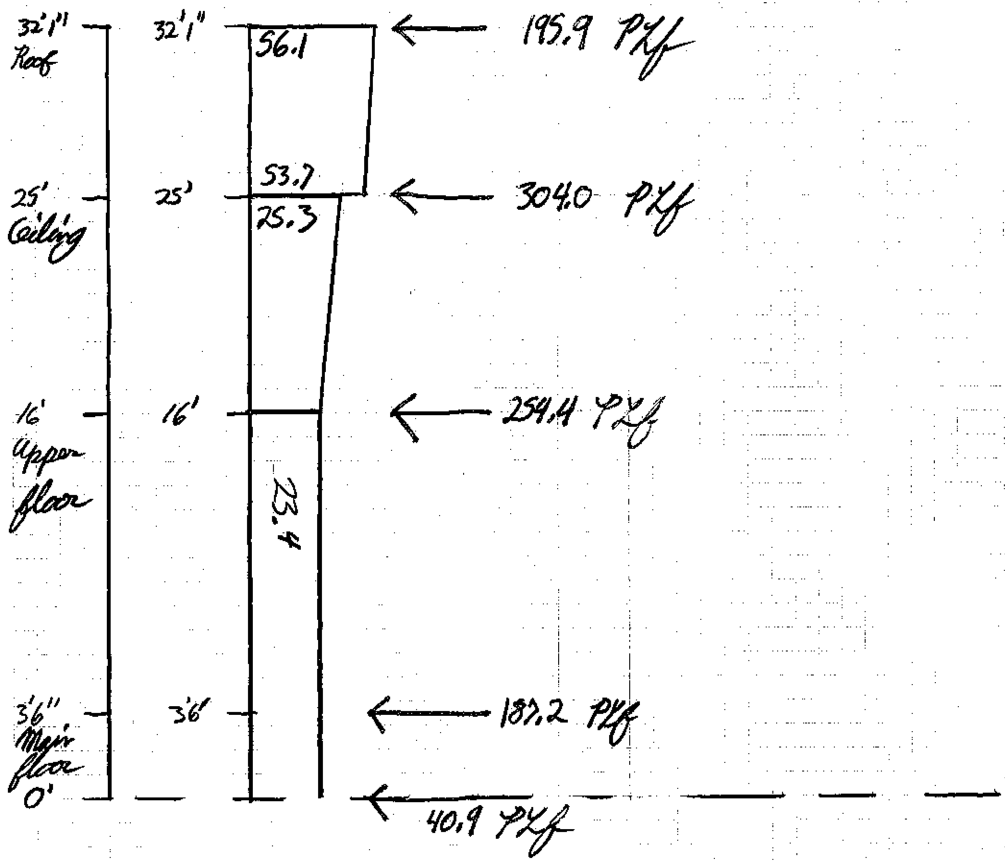
Wind Pressures Zone 3 | (Pressures contain all aplicable leeward & windward pressures @ 1.0 W)

North / South

L = 112'6"

B = 16'7"



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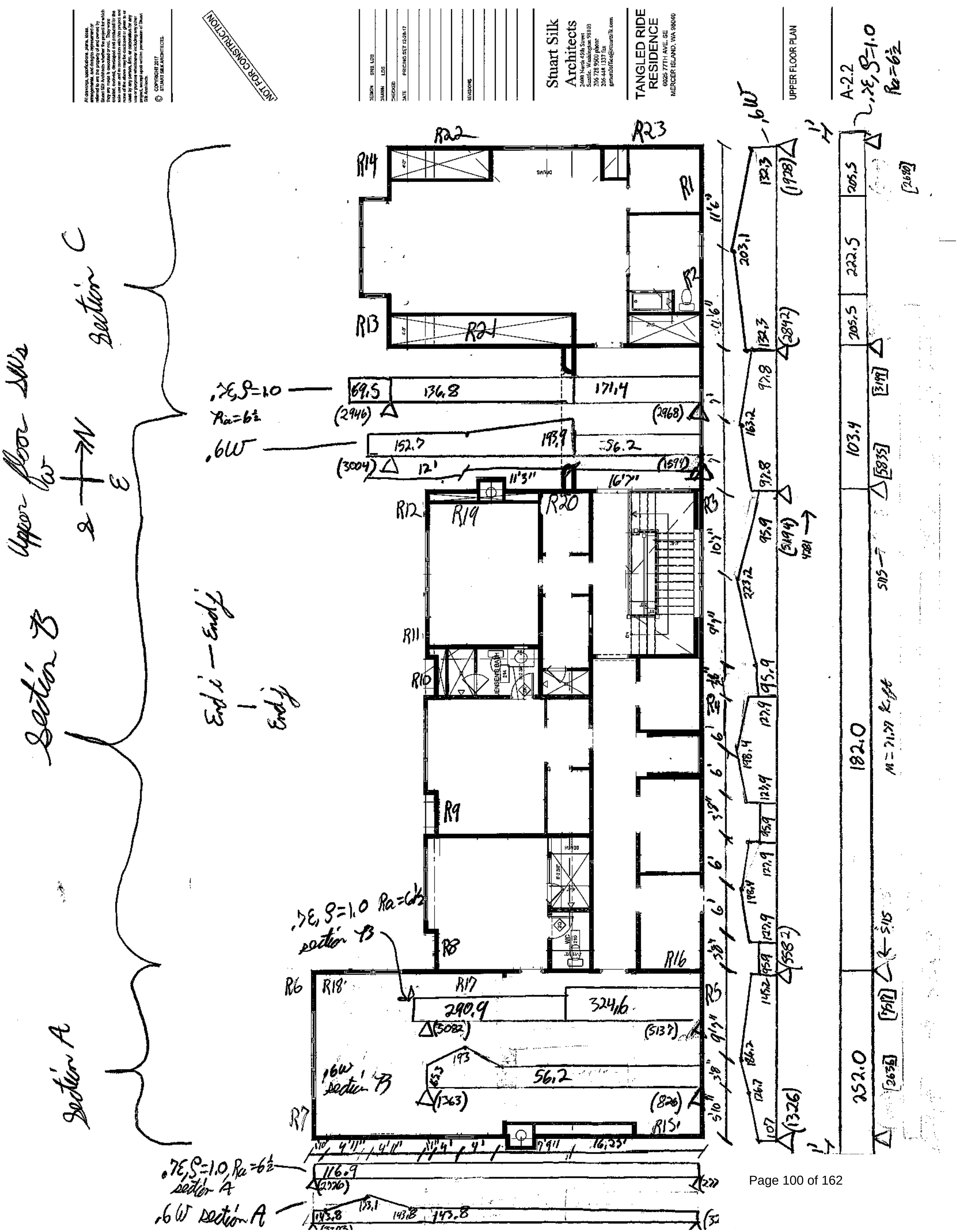
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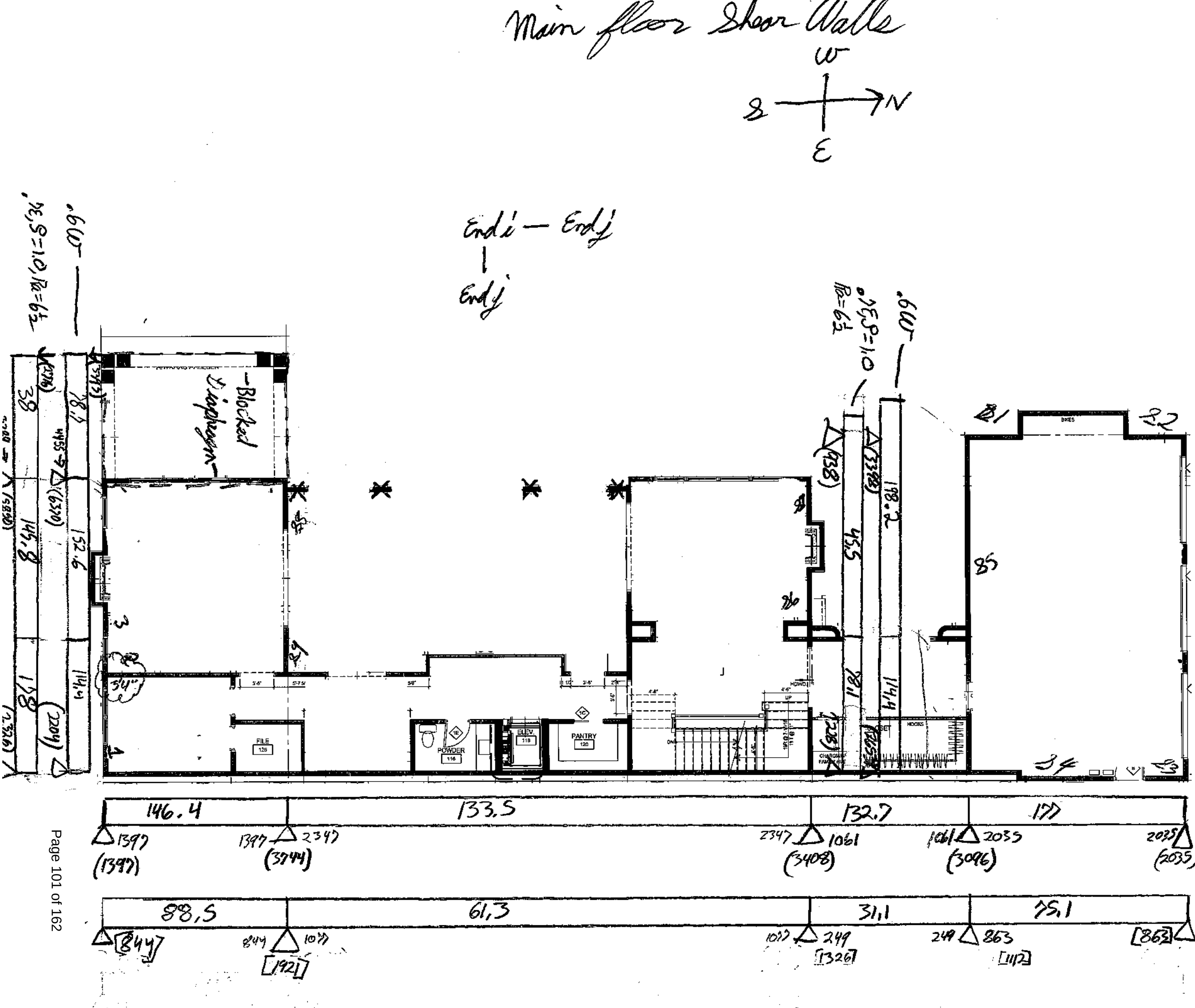
TANGLED RIDE RESIDENCE
6025 77TH AVE. SE
MERCER ISLAND, WA 98040
UPPER FLOOR PLAN
A-2.2
JENSENS BATH



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SNS, LDS

LDS

PRICING SET 12-20-17

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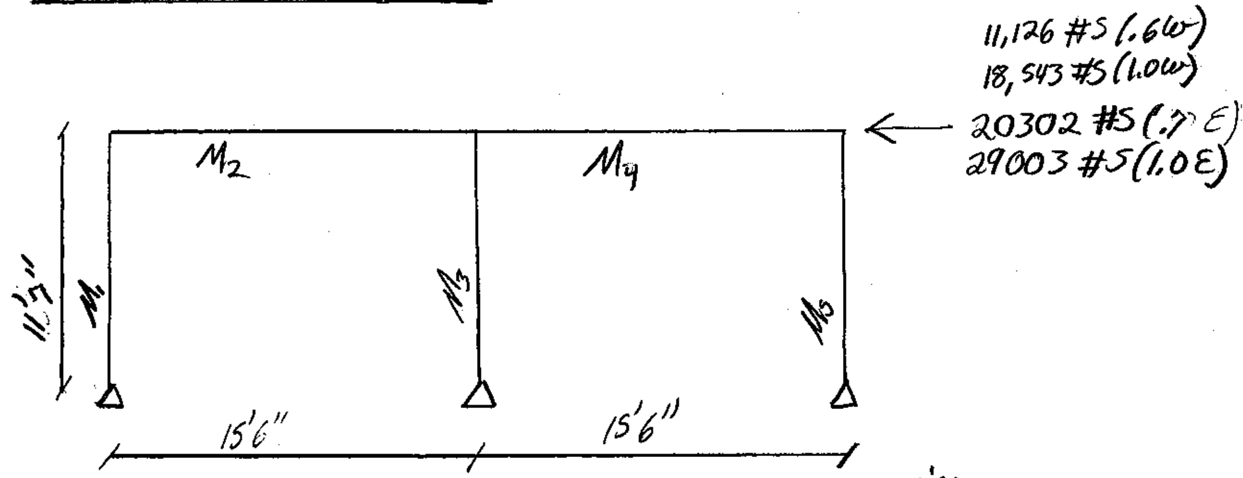
206 448 1337 fax
generaloffice@stuartsilk.com

ANGLED RIDE RESIDENCE
6025 77TH AVE. SE
ERCER ISLAND, WA 98040

MAIN FLOOR PLAN

A-2.1

Steel Moment frame



Max $\delta = \frac{(139")(.02)}{3} = .926"$

Cd = 3

M1, M2, M4, M5 as W18X50
& M3 as W18X65
pass strength
Δ = .395"

.7E drift

~~W10X30's Δ = 1.456" – fail~~

~~W14X26's Δ = 1.019" – fail~~

~~W14X30's Δ = .863" – fail~~

~~W14X34's Δ = .743" – pass~~

W14X34's Δ = 1.001" – fail

W14X38's Δ =

W16X31' Δ = .908" – pass

W16X31 – fail strength

See attached Risa outputs

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Member ASD Steel Code Checks (By Combination)

	LC	Member	Shape	UC Max	Loc[ft]	Shear UC	Loc[ft]	Fa[ksi]	Ft[ksi]	Fb[ksi]	Cb	Cm	Eqn
1	1	M1	W18X50	.307	11.583	.043	0	18.168	30	30	1.75	.6	H1-2
2	1	M2	W18X50	.317	0	.072	0	11.775	30	30	2.3	.85	H1-3
3	1	M3	W18X65	.371	0	.057	0	18.597	30	30	1.75	.6	H1-2
4	1	M4	W18X50	.298	15.5	.072	0	11.775	30	30	2.3	.85	H2-1
5	1	M5	W18X50	.306	0	.043	0	18.168	30	30	1.75	.6	H2-1
6	2	M1	W18X50	.723	11.583	.103	0	18.168	30	30	1.75	.6	H2-1
7	2	M2	W18X50	.748	15.5	.229	15.5	11.775	30	30	2.3	.85	H1-3
8	2	M3	W18X65	.972	0	.141	0	18.597	30	30	1.75	.6	H1-3
9	2	M4	W18X50	.829	15.5	.221	15.5	11.775	30	30	2.3	.85	H1-3
10	2	M5	W18X50	.811	0	.112	0	18.168	30	30	1.75	.6	H1-2
11	3	M1	W18X50	.810	11.583	.112	0	18.168	30	30	1.75	.6	H1-2
12	3	M2	W18X50	.829	0	.220	0	11.775	30	30	2.3	.85	H1-3
13	3	M3	W18X65	.974	0	.141	0	18.597	30	30	1.75	.6	H1-3
14	3	M4	W18X50	.752	0	.230	0	11.775	30	30	2.3	.85	H1-3
15	3	M5	W18X50	.721	0	.102	0	18.168	30	30	1.75	.6	H2-1
16	4	M1	W18X50	.501	11.583	.072	0	18.168	30	30	1.75	.6	H2-1
17	4	M2	W18X50	.633	15.5	.215	15.5	11.775	30	30	2.3	.85	H1-3
18	4	M3	W18X65	.742	0	.106	0	18.597	30	30	1.75	.6	H1-2
19	4	M4	W18X50	.658	15.5	.202	15.5	11.775	30	30	2.3	.85	H1-3
20	4	M5	W18X50	.651	0	.089	0	18.168	30	30	1.75	.6	H1-2
21	5	M1	W18X50	.649	11.583	.089	0	18.168	30	30	1.75	.6	H1-2
22	5	M2	W18X50	.656	0	.200	0	11.775	30	30	2.3	.85	H1-3
23	5	M3	W18X65	.746	0	.106	0	18.597	30	30	1.75	.6	H1-2
24	5	M4	W18X50	.637	0	.216	0	11.775	30	30	2.3	.85	H1-3
25	5	M5	W18X50	.498	0	.072	0	18.168	30	30	1.75	.6	H2-1
26	6	M1	W18X50	.784	11.583	.109	0	18.168	30	30	1.75	.6	H1-2
27	6	M2	W18X50	.808	0	.196	0	11.775	30	30	2.3	.85	H1-3
28	6	M3	W18X65	.946	0	.141	0	18.597	30	30	1.75	.6	H1-3
29	6	M4	W18X50	.731	15.5	.200	0	11.775	30	30	2.3	.85	H2-1
30	6	M5	W18X50	.747	0	.106	0	18.168	30	30	1.75	.6	H2-1
31	7	M1	W18X50	.749	11.583	.105	0	18.168	30	30	1.75	.6	H2-1
32	7	M2	W18X50	.732	0	.200	15.5	11.775	30	30	2.3	.85	H2-1
33	7	M3	W18X65	.946	0	.141	0	18.597	30	30	1.75	.6	H1-3
34	7	M4	W18X50	.808	15.5	.197	15.5	11.775	30	30	2.3	.85	H1-3
35	7	M5	W18X50	.784	0	.109	0	18.168	30	30	1.75	.6	H1-2

Load Combinations

	Description	Solve	PDelta	SRSS	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...
1	.7E	Yes			4	.7														
2	1.1365D-1.75E	Yes			1	1.136	4	-1.75												
3	1.1365D+1.75E	Yes			1	1.136	4	1.75												
4	1.10238D+.75L+...	Yes			1	1.102	2	.75	3	.75	4	-1.3								
5	1.10238D+.75L+...	Yes			1	1.102	2	.75	3	.75	4	1.312								
6	.4635D+1.75E	Yes			1	.464	4	1.75												
7	.4635D-1.75E	Yes			1	.464	4	-1.75												
8	1.395D+2.5E+L+...				1	1.395	2	.2	3	1	4	2.5								
9	1.395D-2.5E+L+...				1	1.395	2	.2	3	1	4	-2.5								
10	.705D+2.5E				1	.705	4	2.5												
11	.705D-2.5E				1	.705	4	-2.5												

LATERAL FORCE DISTRIBUTION (SEISMIC)

Roof SW's

va' = allowable shear values multiplied by 2l/h
for wall aspect ratios greater than 2:1, and only for seismic

Longitudinal (N/S)

WALL	F (lb)	V (abv)	V (total)	L (ft)	v (plf)		h (ft)	h/l	va'	M ot (lbft)	OT (lb)	DL max (lb)	T (lb)	T (abv)	T Total (lbs)	HD	TL (lb)	C (lb)	C (abv)	C Total (lbs)	Studs 2x6
F	10881		10881																		
R1	1217	0	1217	10.00	158	SW1	9.00	0.90	N/A	14238	1424	438	986	0	986	CS20	665	2089	0	2088.8	2
												243	1181	0	1181	CS18	400	1824	0	1823.8	2
R2	1785	0	1785	14.67	158	SW1	9.00	0.61	N/A	20882	1424	611	812	0	812	CS20	875	2299	0	2298.8	2
												551	872	0	872	CS20	665	2089	0	2088.8	2
R3	1126	0	1126	9.25	158	SW1	9.00	0.97	N/A	13170	1424	1232	192	0	192	HDU2	2130	3554	0	3553.8	3
												821	603	0	603	CS22	875	2299	0	2298.8	2
R4	3671	0	3671	30.17	158	SW1	9.00	0.30	N/A	42950	1424	2837	-1414	0	-1414	None	815	2239	0	2238.8	2
												3770	-2347	0	-2347	None	3825	5249	0	5248.8	3
R5	3083	0	3083	25.33	158	SW1	9.00	0.36	N/A	36069	1424	616	808	0	808	HDU2	400	1824	0	1823.8	2
Lt				89.42								865	559	0	559	HDU2	815	2239	0	2238.8	2
F	2776	0	2776																		
R6	1388	0	1388	4.58	394	SW3	9.00	1.96	N/A	16240	3543	945	2598	0	2598	(2)CS18	2670	6213	0	6213.2	2
												111	3432	0	3432	(2)CS14	400	3943	0	3943.2	2
R7	1388	0	1388	4.58	394	SW3	9.00	1.96	N/A	16240	3543	111	3432	0	3432	(2)CS14	400	3943	0	3943.2	2
Lt				9.17								945	2598	0	2598	(2)CS18	2670	6213	0	6213.2	2
F	5082		5082																		
R8	971	0	971	3.92	322	SW3	9.00	2.30	396	11360	2900	336	2564	0	2564	(2)CS18	400	3300	0	3300.5	2
												1152	1748	0	1748	HDU2	2635	5535	0	5535.5	2
R9	888	0	888	3.58	322	SW3	9.00	2.51	362	10393	2900	1123	1777	0	1777	HDU2	2635	5535	0	5535.5	2
												1123	1777	0	1777	HDU2	2635	5535	0	5535.5	2
R10	950	0	950	3.83	322	SW3	9.00	2.35	388	11118	2900	1097	1804	0	1804	HDU2	2635	5535	0	5535.5	2
												1436	1465	0	1465	HDU2	3705	6605	0	6605.5	2
R11	1136	0	1136	4.58	322	SW3	9.00	1.96	N/A	13294	2900	111	2789	0	2789	(2)CS16	400	3300	0	3300.5	2
												501	2399	0	2399	(2)CS18	1195	4095	0	4095.5	2
R12	1136	0	1136	4.58	322	SW3	9.00	1.96	N/A	13294	2900	501	2399	0	2399	(2)CS18	1195	4095	0	4095.5	2
Lt				20.50								111	2789	0	2789	(2)CS16	400	3300	0	3300.5	2
	2946	0	2946																		
13	1473	0	1473	5.75	333	SW2	9.00	1.57	N/A	17234	2997	140	2858	0	2858	(2)CS16	400	3397	0	3397.2	2
												1529	1469	0	1469	CS16	4480	7477	0	7477.2	3
14	1473	0	1473	5.75	333	SW2	9.00	1.57	N/A	17234	2997	1529	1469	0	1469	CS16	4480	7477	0	7477.2	3
Lt				11.50								140	2858	0	2858	(2)CS16	400	3397	0	3397.2	2
10	1.3																				

Roof SW's

Transverse (E/W)

WALL	F	V	V	L	v		h	h/l	va'	M ot	OT	DL max	T	T	T Total	HD	TL	C	C	C Total	Studs
	(lb)	(abv)	(total)	(ft)	(plf)		(ft)			(lbft)	(lb)	(lb)	(lb)	(abv)	(lbs)		(lb)	(lb)	(abv)	(lbs)	2x6
R15	2656	0	2656	19.75	175	SW1	9.00	0.46	N/A	31075	1573	1946	-372	0	-372	none	1305	2878	0	2878.4	2
												1292	282	0	282	HDU2	400	1973	0	1973.4	2
F	7517		7517																		
R16	2514	0	2514	8.33	392	SW3	9.00	1.08	N/A	29414	3530	203	3327	0	3327	(2)CS16	400	3930	0	3929.7	2
												203	3327	0	3327	HDU5	400	3930	0	3929.7	2
R17	4022	0	4022	13.33	392	SW3	9.00	0.68	N/A	47063	3530	465	3065	0	3065	(2)CS18	460	3990	0	3989.7	2
												324	3206	0	3206	(2)CS18	400	3930	0	3929.7	2
R18	980	0	980	3.25	392	SW4	9.00	2.77	430	11472	3530	79	3451	0	3451	(2)CS14	400	3930	0	3929.7	2
Lt				24.92								229	3301	0	3301	(2)CS14	490	4020	0	4019.7	2
F	5835	0	5835																		
R19	1838	0	1838	4.75	503	SW4	9.00	1.89	N/A	21499	4526	311	4216	0	4216	(2)CS14	400	4926	0	4926.2	2
												992	3535	0	3535	HDU5	1335	5861	0	5861.2	2
R20	3997	0	3997	10.33	503	SW4	9.00	0.87	N/A	46770	4526	932	3594	0	3594	HDU5	1335	5861	0	5861.2	2
Lt				15.08								338	4188	0	4188	(2)CS14	360	4886	0	4886.2	2
R21	3199	0	3199	20.75	200	SW1	9.00	0.43	N/A	37428	1804	1500	304	0	304	CS22	400	2204	0	2203.8	2
												2490	-686	0	-686	None	3085	4889	0	4888.8	3
F	2680	0	2680																		
R22	1340	0	1340	13.33	131	SW2	9.00	0.68	N/A	15678	1176	324	852	0	852	CS20	400	1576	0	1575.9	2
												663	513	0	513	None	1090	2266	0	2265.9	2
R23	1340	0	1340	13.33	131	SW2	9.00	0.68	N/A	15678	1176	663	513	0	513	None	1090	2266	0	2265.9	2
Lt				26.67								324	852	0	852	CS20	400	1576	0	1575.9	2

Rho 1.3

Second Floor SW's

Longitudinal (N/S)

WALL	F (lb)	V (abv)	V (total)	L (ft)	v (plf)		h (ft)	h/l	va'	M ot (lbft)	OT (lb)	DL max (lb)	T (lb)	T (abv)	T Total (lbs)	HD	TL (lb)	C (lb)	C (abv)	C Total (lbs)	Studs 2x6
F	938	2946	3884																		
S1	469	1473	1942	5.75	439	SW4	11.00	1.91	N/A	27771	4830	171	4659	2858	7516	HDU14	600	5430	2997	8426.9	6x8 DF
												270	4560	1469	6028	HDU11	4480	9310	2997	12307	6x8 DF
S2	469	1473	1942	5.75	439	SW4	11.00	1.91	N/A	27771	4830	270	4560	1469	6028	HDU11	4480	9310	2997	12307	6x8 DF
Lt				11.50								171	4659	2858	7516	HDU14	600	5430	2997	8426.9	6x8 DF
F	1228	1764	2992																		
S3	346	497	842	3.92	280	SW3	11.00	2.81	324	12043	3075	116	2959	0	2959	MSTCM60	600	3675	0	3674.9	2
												116	2959	1181	4139	MSTCM60	600	3675	1424	5098.7	3
S4	882	1268	2150	10.00	280	SW3	11.00	1.10	N/A	30749	3075	297	2778	0	2778	MSTCM60	600	3675	0	3674.9	2
Lt				13.92								297	2778	0	2778	MSTCM60	600	3675	0	3674.9	2
Transverse (E/W)																					
S5	1112	3199	4311	25.50	220	SW1	11.00	0.43	N/A	61647	2418	2050	367	304	671	HDU14	600	3018	1804	4821.3	3
												3139	-722	0	-722	None	3525	5943	0	5942.5	3
F	1326	5835	7161																		
S6	815	3587	4403	6.92	827	SW6	11.00	1.59	N/A	62958	9102	829	8274	3594	11868	HDU14	2610	11712	4526	16239	6x8 DF
												2341	6762	2741	9503	HDU11	5894	14996	2741	17737	6x8 DF
S7	511	2248	2758	4.33	827	SW6	11.00	2.54	938	39444	9102	307	8796	4216	13011	HDU14	600	9702	4526	14229	6x8 DF
Lt				11.25								646	8457	3535	11991	HDU14	2610	11712	4526	16239	6x8 DF
F	1921	7517	9438																		
S8	961	3759	4719	5.33	1150	SW6	11.00	2.06	1154	67482	12653	1884	10769	1892	12661	HDU14	5011	17664	1892	19556	6x8 DF
												2634	10019	1793	11813	HDU14	7353	20005	1793	21799	6x8 DF
S9	961	3759	4719	5.33	1150	SW6	11.00	2.06	1154	67482	12653	2256	10397	-1736	8661	CMST12	5895	18548	-1736	16811	6
Lt				10.67								1446	11207	-2644	8563	CMST12	4525	17178	-2644	14534	6

Basement SW's

Transverse (E/W)

WALL	F (lb)	V (abv)	V (total)	L (ft)	v (plf)		h (ft)	h/l	va'	M ot (lbft)	OT (lb)	DL max (lb)	T (lb)	T (abv)	T Total (lbs)	HD	TL (lb)	C (lb)	C (abv)	C Total (lbs)	Studs
M1	9438	0	9438	10.17	1207	SW1	9.00	0.89	N/A	110425	10861	6683	4179	7821	12000		19196	30058	7821	37879	ISS 3x3
												1466	9396	0	9396		2595	13456	0	13456	6x8 DF

LATERAL FORCE DISTRIBUTION (WIND)

Roof SW's

va' = allowable shear values multiplied by 2l/h
for wall aspect ratios greater than 2:1, and only for seismic

Longitudinal (N/S)

WALL	F (lb)	V (abv)	V (total)	L (ft)	v (plf)		h (ft)	h/l	va'	M ot (lbft)	OT (lb)	DL max (lb)	T (lb)	T (abv)	T Total (lbs)	HD	TL (lb)	C (lb)	C (abv)	C Total (lbs)	Studs 2x6
F	5712		5712																		
R1	639	0	639	10.00	64	SW1	9.00	0.90	N/A	5749	575	438	137	0	137	CS20	665	1240	0	1239.9	2
												243	332	0	332	CS18	400	975	0	974.93	2
R2	937	0	937	14.67	64	SW1	9.00	0.61	N/A	8432	575	611	-36	0	-36	CS20	875	1450	0	1449.9	2
												551	24	0	24	CS20	665	1240	0	1239.9	2
R3	591	0	591	9.25	64	SW1	9.00	0.97	N/A	5318	575	1232	-657	0	-657	HDU2	2130	2705	0	2704.9	3
												821	-246	0	-246	CS22	875	1450	0	1449.9	2
R4	1927	0	1927	30.17	64	SW1	9.00	0.30	N/A	17344	575	2837	-2262	0	-2262	None	815	1390	0	1389.9	2
												3770	-3195	0	-3195	None	3825	4400	0	4399.9	3
R5	1618	0	1618	25.33	64	SW1	9.00	0.36	N/A	14565	575	616	-41	0	-41	HDU2	400	975	0	974.93	2
Lt				89.42								865	-290	0	-290	HDU2	815	1390	0	1389.9	2
F	3393	0	3393																		
R6	1697	0	1697	4.58	370	SW3	9.00	1.96	N/A	15269	3331	945	2386	0	2386	(2)CS18	2670	6001	0	6001.3	2
												111	3220	0	3220	(2)CS14	400	3731	0	3731.3	2
R7	1697	0	1697	4.58	370	SW3	9.00	1.96	N/A	15269	3331	111	3220	0	3220	(2)CS14	400	3731	0	3731.3	2
Lt				9.17								945	2386	0	2386	(2)CS18	2670	6001	0	6001.3	2
F	1363		1363																		
R8	260	0	260	3.92	66	SW3	9.00	2.30	N/A	2344	598	336	262	0	262	(2)CS18	400	998	0	998.39	2
												1152	-554	0	-554	HDU2	2635	3233	0	3233.4	2
R9	238	0	238	3.58	66	SW3	9.00	2.51	N/A	2144	598	1123	-525	0	-525	HDU2	2635	3233	0	3233.4	2
												1123	-525	0	-525	HDU2	2635	3233	0	3233.4	2
R10	255	0	255	3.83	66	SW3	9.00	2.35	N/A	2294	598	1097	-499	0	-499	HDU2	2635	3233	0	3233.4	2
												1436	-838	0	-838	HDU2	3705	4303	0	4303.4	2
R11	305	0	305	4.58	66	SW3	9.00	1.96	N/A	2743	598	111	487	0	487	(2)CS16	400	998	0	998.39	2
												501	97	0	97	(2)CS18	1195	1793	0	1793.4	2
R12	305	0	305	4.58	66	SW3	9.00	1.96	N/A	2743	598	501	97	0	97	(2)CS18	1195	1793	0	1793.4	2
Lt				20.50								111	487	0	487	(2)CS16	400	998	0	998.39	2
	3004	0	3004																		
3	1502	0	1502	5.75	261	SW2	9.00	1.57	N/A	13518	2351	140	2211	0	2211	(2)CS16	400	2751	0	2751	2
												1529	822	0	822	CS16	4480	6831	0	6831	3
4	1502	0	1502	5.75	261	SW2	9.00	1.57	N/A	13518	2351	1529	822	0	822	CS16	4480	6831	0	6831	3
Lt				11.50								140	2211	0	2211	(2)CS16	400	2751	0	2751	2
o	1.0																				

Roof SW's

Transverse (E/W)

WALL		F (lb)	V (abv)	V (total)	L (ft)	v (plf)		h (ft)	h/l	va'	M ot (lbft)	OT (lb)	DL max (lb)	T (lb)	T (abv)	T Total (lbs)	HD	TL (lb)	C (lb)	C (abv)	C Total (lbs)	Studs 2x6
R15		1326	0	1326	19.75	67	SW1	9.00	0.46	N/A	11934	604	1946	-1341	0	-1341	none	1305	1909	0	1909.3	2
													1292	-687	0	-687	HDU2	400	1004	0	1004.3	2
F		5582		5582																		
R16		1867	0	1867	8.33	224	SW3	9.00	1.08	N/A	16802	2016	203	1814	0	1814	(2)CS16	400	2416	0	2416.2	2
													203	1814	0	1814	HDU5	400	2416	0	2416.2	2
R17		2987	0	2987	13.33	224	SW3	9.00	0.68	N/A	26883	2016	465	1551	0	1551	(2)CS18	460	2476	0	2476.2	2
													324	1692	0	1692	(2)CS18	400	2416	0	2416.2	2
R18		728	0	728	3.25	224	SW4	9.00	2.77	N/A	6553	2016	79	1937	0	1937	(2)CS14	400	2416	0	2416.2	2
	Lt				24 92								229	1787	0	1787	(2)CS14	490	2506	0	2506.2	2
F		5194	0	5194																		
R19		1636	0	1636	4.75	344	SW4	9.00	1.89	N/A	14721	3099	311	2789	0	2789	(2)CS14	400	3499	0	3499.2	2
													992	2108	0	2108	HDU5	1335	4434	0	4434.2	2
R20		3558	0	3558	10.33	344	SW4	9.00	0.87	N/A	32025	3099	932	2167	0	2167	HDU5	1335	4434	0	4434.2	2
	Lt				15.08								338	2761	0	2761	(2)CS14	360	3459	0	3459.2	2
R21		2842	0	2842	20.75	137	SW1	9.00	0.43	N/A	25578	1233	1500	-268	0	-268	CS22	400	1633	0	1632.7	2
													2490	-1258	0	-1258	None	3085	4318	0	4317.7	3
F		1928	0	1928																		
R22		964	0	964	13.33	72	SW2	9.00	0.68	N/A	8676	651	324	327	0	327	CS20	400	1051	0	1050.7	2
													663	-12	0	-12	None	1090	1741	0	1740.7	2
R23		964	0	964	13.33	72	SW2	9.00	0.68	N/A	8676	651	663	-12	0	-12	None	1090	1741	0	1740.7	2
	Lt				26.67								324	327	0	327	CS20	400	1051	0	1050.7	2

Rho 1.0

Second Floor SW's

Longitudinal (N/S)

WALL	F (lb)	V (abv)	V (total)	L (ft)	v (plf)		h (ft)	h/l	va'	M ot (lbft)	OT (lb)	DL max (lb)	T (lb)	T (abv)	T Total (lbs)	HD	TL (lb)	C (lb)	C (abv)	C Total (lbs)	Studs 2x6
F	3398	3004	6402																		
S1	1699	1502	3201	5.75	557	SW4	11.00	1.91	N/A	35211	6124	171	5953	2211	8164	HDU14	600	6724	2351	9074.6	6x8 DF
												270	5854	822	6676	HDU11	4480	10604	2351	12955	6x8 DF
S2	1699	1502	3201	5.75	557	SW4	11.00	1.91	N/A	35211	6124	270	5854	822	6676	HDU11	4480	10604	2351	12955	6x8 DF
Lt				11.50								171	5953	2211	8164	HDU14	600	6724	2351	9074.6	6x8 DF
F	2635	926	3561																		
S3	742	261	1002	3.92	256	SW3	11.00	2.81	N/A	11025	2815	116	2699	0	2699	MSTCM60	600	3415	0	3414.9	2
												116	2699	332	3030	MSTCM60	600	3415	575	3989.8	3
S4	1893	666	2559	10.00	256	SW3	11.00	1.10	N/A	28149	2815	297	2518	0	2518	MSTCM60	600	3415	0	3414.9	2
Lt				13.92								297	2518	0	2518	MSTCM60	600	3415	0	3414.9	2
Transverse (E/W)																					
S5	3096	2842	5938	25.50	233	SW1	11.00	0.43	N/A	65318	2561	2050	511	-268	244	HDU14	600	3161	1233	4394.2	3
												3139	-578	0	-578	None	3525	6086	0	6086.5	3
F	3408	5194	8602																		
S6	2095	3193	5289	6.92	765	SW6	11.00	1.59	N/A	58175	8411	829	7582	2167	9749	HDU14	2610	11021	3099	14120	6x8 DF
												2341	6070	1877	7947	HDU11	5894	14305	1877	16182	6x8 DF
S7	1313	2001	3313	4.33	765	SW6	11.00	2.54	N/A	36447	8411	307	8104	2789	10893	HDU14	600	9011	3099	12110	6x8 DF
Lt				11.25								646	7765	2108	9873	HDU14	2610	11021	3099	14120	6x8 DF
F	3744	5582	9326																		
S8	1872	2791	4663	5.33	874	SW6	11.00	2.06	N/A	51293	9617	1894	7734	1081	8814	HDU14	5011	14629	1081	15709	6x8 DF
												2634	6984	1024	8008	HDU14	7353	16970	1024	17994	6x8 DF
S9	1872	2791	4663	5.33	874	SW6	11.00	2.06	N/A	51293	9617	2256	7362	-992	6370	CMST12	5895	15512	-992	14521	6
Lt				10.67								1446	8172	-1510	6662	CMST12	4525	14142	-1510	12632	6

Basement SW's

Transverse (E/W)

WALL	F (lb)	V (abv)	V (total)	L (ft)	v (plf)		h (ft)	h/l	va'	M ot (lbft)	OT (lb)	DL max (lb)	T (lb)	T (abv)	T Total (lbs)	HD	TL (lb)	C (lb)	C (abv)	C Total (lbs)	Studs 2x6
M1	9326	0	9326	10.17	917	SW1	9.00	0.89	N/A	83934	8256	6683	1573	6345	7918		19196	27452	6345	33797	HSS 3x3
												1466	6790	0	6790		2595	10851	0	10851	6x8 DF

SECTION 4: FOUNDATION

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South Line Strong walls

1

L = 2'9" Can use SSW 24X11

		Max	Min
Max axial = 1265D, 495S, 1235L	+(2)(355D, 149S, 365L)	1975D, 795S, 1965L	710D, 300S, 730L
		Max ↓ 4.045K	↓ 1.48K
		Max shear = 3205#S Seismic & wind	Max shear = 3475#S Seismic, 3710#S Wind

2

L = 2'10" can use SSW 24X11

Max axial	Min	Middle
1955D	710D	1345D
795S	300S	550S
1965L	730L	1345L
↑	↑	↑
Shear = 3205 Seismic & Wind	3475#S Seismic, 3710#S Wind	

3

L = 2'5" can use SSW 24X11

Max axial = 1345D, 550S, 1345L

3475 – Seismic

3710 – Wind.

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South Line Strong Walls continued

Wind shear = 2723 #S (.6 W)

Seismic = 3480 #S (.7E) — ρ=1.3 ; 4524 #S

use ~~3~~ 2 & 1 wider footings because ~~chimney~~ retaining walls

Load each = 2262 #S

T = (v·h/B) B = 18¼"

$$T = \frac{(2262)(129.25)}{18.25} = 16019 \text{ \#S } (\rho=1.3)$$

.7E + .6D = 15,594 #S (Asd) 1" ϕ anchor bolt

1.0E + .9D = 22,245 #S (Lrfd)

$\frac{1}{2} \phi V_c$ = 22.25K min Ac = 593.3 in² 24" X 25" 21" X 29"

20X22

T = 22,245

v = (¼)(4524) = 1131

30X21

PAB 8 18" embedment

T = 22,245 ✓

v = 1131 ✓

PAB8 too short to reach

T = 15.6K use CNW1

good for 17K

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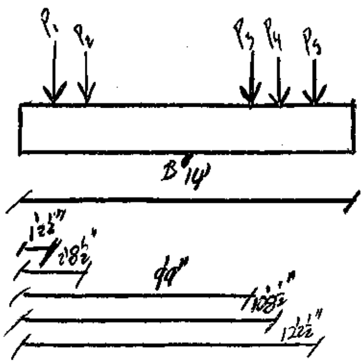
footings under South Line Strong walls

$V = 22.25k$ (1.0E) try 30X24 SW = 729

$M = 24.884$ k.ft (1.0E) $M = 17.41$ k.ft (Asd)

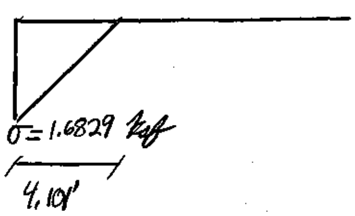
	↓ .7E	← .6D+.7E	→ .6D+.7E
P_1 = 355D, 16019E +		16232	-15806
P_2 = 1620D, 16019E −		-15047	16991
P_3 = 1265D		759	759
P_4 = 355D, 16019E +		16232	-15806
P_5 = 635D, 16019E −		-15638	16400

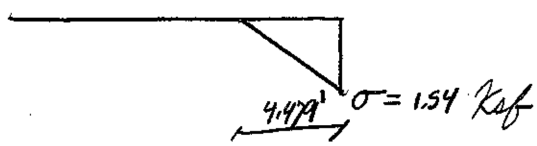


P↓ = 2538 (.6D)

M_{ot_B} = 48.6 k.ft ←E

M_{ot_B} = 47.51 k.ft E→





.6D+.7E

(1.6) = Lrfd

V = 15.17 k

V = 24.28 k

M = 20.27 k.ft

M = 32.43 k.ft

.6D+.7E

(1.6) = Lrfd

V = 15.27 k

V = 24.43 k

M = 22.35 k.ft

M = 35.76 k.ft

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footing under South Line Strong Walls - Continued

$V = 24.43K$
$M = 35.76 K.ft$

$\frac{1}{2}\phi V_c = 27.0K$

$\phi M_n = 70.26 K.ft$
(4 # T & B) - $A_{s\,min}$ = 7#4 Grade 60

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1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: South Line Strong
Location:
Fastening description: Walls

2. Input Data & Anchor Parameters

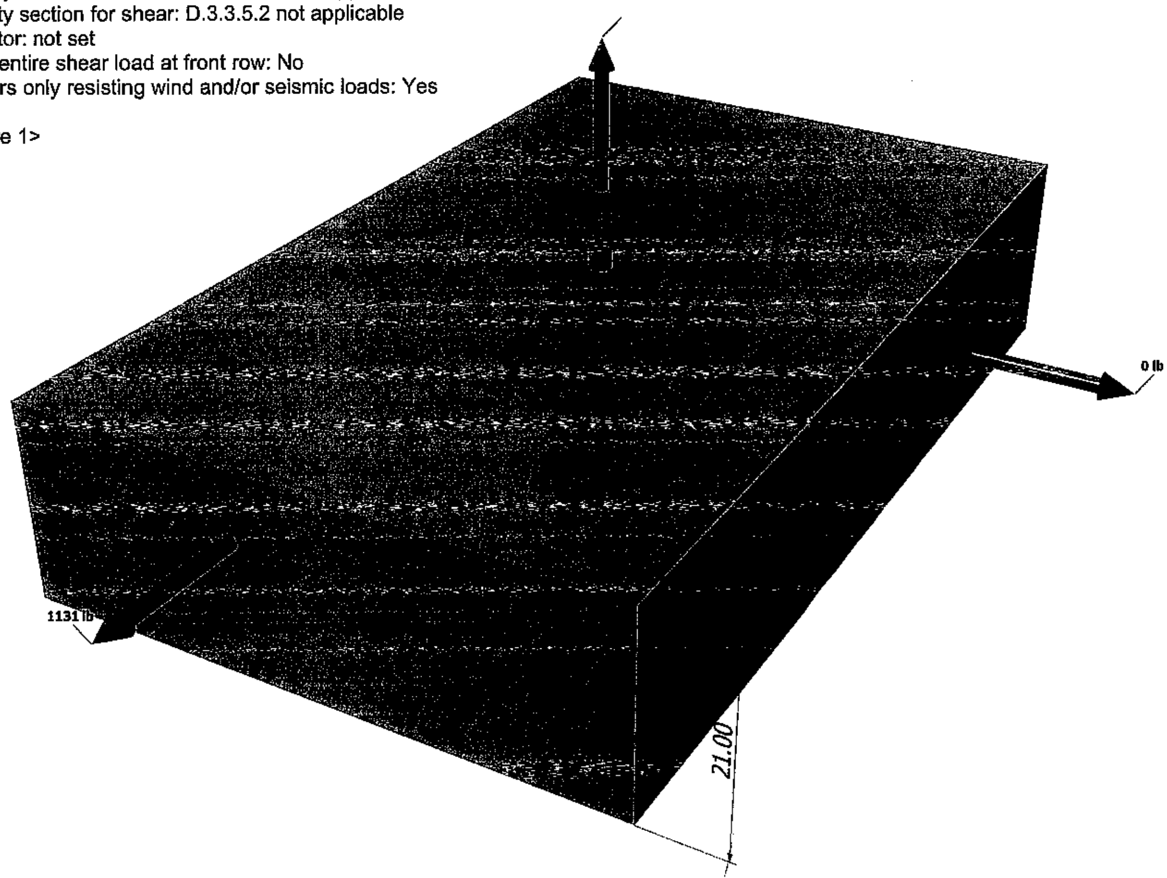
General
Design method:ACI 318-11
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 18.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 20.63
C_{min} (inch): 6.00
S_{min} (inch): 6.00

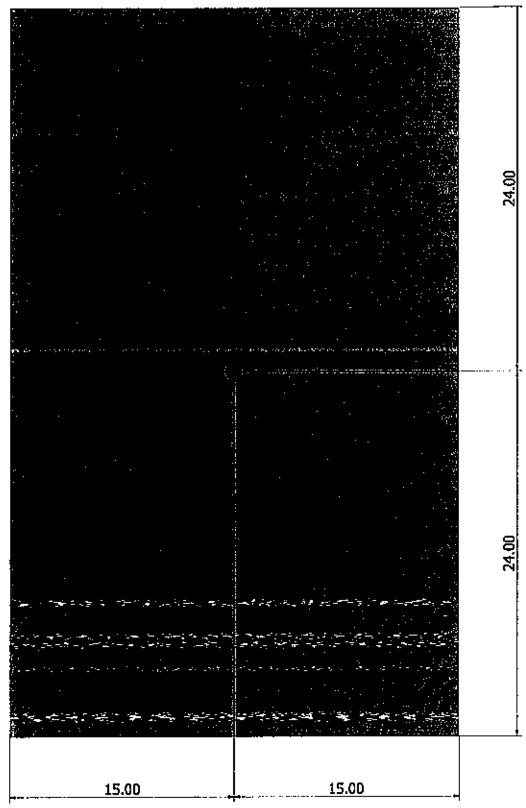
Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 21.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: D.3.3.4.2 not applicable
Ductility section for shear: D.3.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

<Figure 1>



<Figure 2>



Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB8 (1"Ø)

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	22245.0	1131.0	0.0	1131.0
Sum	22245.0	1131.0	0.0	1131.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 22245
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. D.5.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
35150	0.75	26363

5. Concrete Breakout Strength of Anchor in Tension (Sec. D.5.2)

$N_b = 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3}$ (Eq. D-7)

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	2500	16.000	81275

$0.75\phi N_{cb} = 0.75\phi\ (A_{Nc} / A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. D.4.1 & Eq. D-3)

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
1440.00	2304.00	15.00	0.888	1.00	1.000	81275	0.70	23668

6. Pullout Strength of Anchor in Tension (Sec. D.5.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}N_p = 0.75\phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. D.4.1, Eq. D-13 & D-14)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	5.15	2500	0.70	54117

8. Steel Strength of Anchor in Shear (Sec. D.6.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
21090	1.0	0.65	13709

9. Concrete Breakout Strength of Anchor in Shear (Sec. D.6.2)

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. D-33 & Eq. D-34)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
8.00	1.000	1.00	2500	14.00	23572

$\phi V_{cbx} = \phi\ (A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. D.4.1 & Eq. D-30)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbx} (lb)
630.00	882.00	0.914	1.000	1.000	23572	0.70	10776

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. D-33 & Eq. D-34)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
8.00	1.000	1.00	2500	15.00	26143

$\phi V_{cbx} = \phi\ (2)(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. D.4.1 & Eq. D-30)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbx} (lb)
945.00	1012.50	1.000	1.000	1.035	26143	0.70	35359

10. Concrete Pryout Strength of Anchor in Shear (Sec. D.6.3)

$\phi V_{cp} = \phi k_{cp}N_{cb} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. D.4.1 & Eq. D-40)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
2.0	1440.00	2304.00	0.888	1.000	1.000	81275	0.70	63115

11. Results

Interaction of Tensile and Shear Forces (Sec. D.7.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	22245	26363	0.84	Pass
Concrete breakout	**22245**	**23668**	**0.94**	**Pass (Governs)**
Pullout	22245	54117	0.41	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	1131	13709	0.08	Pass
T Concrete breakout x+	**1131**	**10776**	**0.10**	**Pass (Governs)**
|| Concrete breakout y-	**1131**	**35359**	**0.03**	**Pass (Governs)**
Pryout	1131	63115	0.02	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. D.7..1	0.94	0.00	94.0 %	1.0	Pass

PAB8 (1"Ø) with hef = 18.000 inch meets the selected design criteria.

12. Warnings

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.4.3 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.5.3 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

Strong walls - SWs Garage

2 sections $L = 1'9''$ $H = 11'0'' = 132''$ top of slab to bottom of ceiling

SSW 21X11 $L = 21'' = 19'' \checkmark$ $h = 129\frac{1}{4}''$ Loses $2\frac{3}{4}''$ of height, Ripped 4X Blocking + replace Sill PL

$V_{wind} = 3963$ #S (.6 w)

$V_{seismic} = 3553$ #S (.7E) — 4618 #S (Ω=1.3)

Max Axial west end = 1735D 805S 2060L + (1.75)(397D 174S 440L) | 2630D 1110S 2830L

Max $\downarrow$ = 5.585 K Less than 7.5K — max shear = 2305 #S Seismic or Wind

Max Axial East end = 2810D 1555S 2055L + 1.75(397D 174S 440L) | 3505D 1860S 2845L

Max $\downarrow$ = 7.033K Less than 7.5K — max shear = 2305 #S Seismic or Wind

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Garage strong walls Continued

$T = (V \cdot h / B) - P/2$ $\quad B = 15\frac{1}{4}''$ for SSW21

$V = 1981$ #S (wind) .6W

$V = 2309$#S Seismic (.7E, Ω=1.3) Max shear = 2305#S 4 #S... ✓

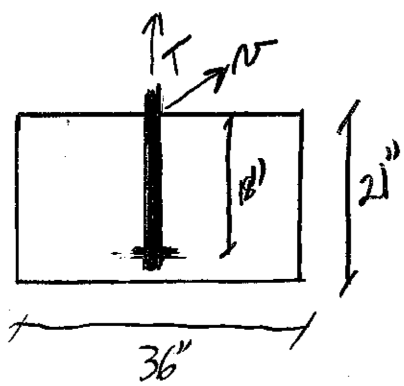
$\frac{(2309)(129.25)}{15.25} = 19569$ #S $\quad P/2$ (from wall only) = 347#SD(1.0)

.7E − .6D = 19360#S (Asd) use 1"φ anchor bolts

1.0E − .9D = 27643#S (Lrfd)

Looking @ $\frac{1}{2}\phi V_c = 27.64k$ min $A_c = 737.2\ in^2$ 31"x24"

or 36x21



Embed PAB8H 18"

Axial = 27643 ✓

$V = 3298/2 = 1649$ ✓

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footings under Garage Strong walls

$V = 27.64^k$ · 1.0E

$M = 36.28^{k \cdot ft}$ 1.0E $M = 25.4^{k \cdot ft}$ (Asd)

$D = 21''$ $\frac{1}{2}\phi V_c = 28.35^k$

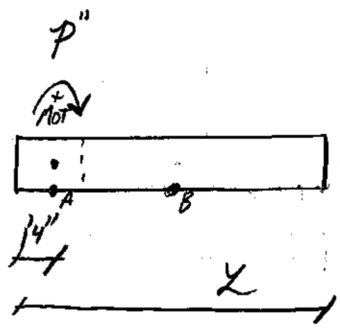
$W = 36''$

(Min)

Axial = 2630D SW footing / ft = 761

.6D .9D .6 .9

1578 2367 456 685



$M_{ot}^{+} = 25.4^{k \cdot ft} - (P)(\frac{L}{2} - 1'4'')$

$M_{ot}^{-} = 25.4^{k \cdot ft} + (P)(\frac{L}{2} - 1'4'')$

$P'' = 2630D + (L)(SW)$

when $L = 11'$

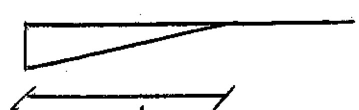
$M_{ot}^{-} = 25.4^{k \cdot ft} + (1.578)(5.5 - 1.33) = 31.975^{k \cdot ft}$

6.575 k·ft

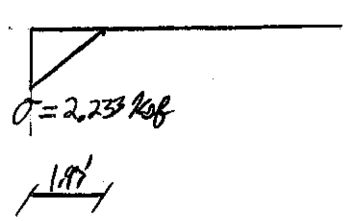
$M_{ot}^{+} = 25.4 - 6.575$ k·ft

$= 18.825$ k·ft

$P_T = 1.578 + (11)(.456) = 6.594^k$



$\sigma = .553$ ksf



$P_R = (\frac{36}{12})(2.233)(\frac{1.97}{2}) = 6.598^k$ ✓

$M_R = (6.598)(5.5 - \frac{1.97}{3}) = 31.956^{k \cdot ft}$ ✓

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footings under Garage Walls - Continued

M_{ot}^{+} (Asd)

$V = 18.68K$

$M = 27.49$ k.ft

1.6(Asd) = Lrfd

$V = 29.88K$

$M = 43.98$ k.ft

M_{ot}^{-} (Asd)

$V = 15.76K$

$M = 31.98$ k.ft

(1.6)(Asd) = Lrfd

$V = 25.23K$

$M = 51.16$ k.ft

$V = 29.98K$

$M = 51.16$ k.ft

Make 36" X 24"

$\frac{1}{2}\phi V_c = 32.47K$

$\phi M_n = 70.48$ k.ft

(4) #4 T & B Grade 60 = As min

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SIMPSON Strong-Tie® Anchor Designer™ Software
Version 2.5.6582.53

Company:	Harriott & Valentine	Date:	1/24/2018
Engineer:	Jonathan Carlson	Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

1.Project Information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: Garage Strong Wall
Location:
Fastening description:

2. Input Data & Anchor Parameters

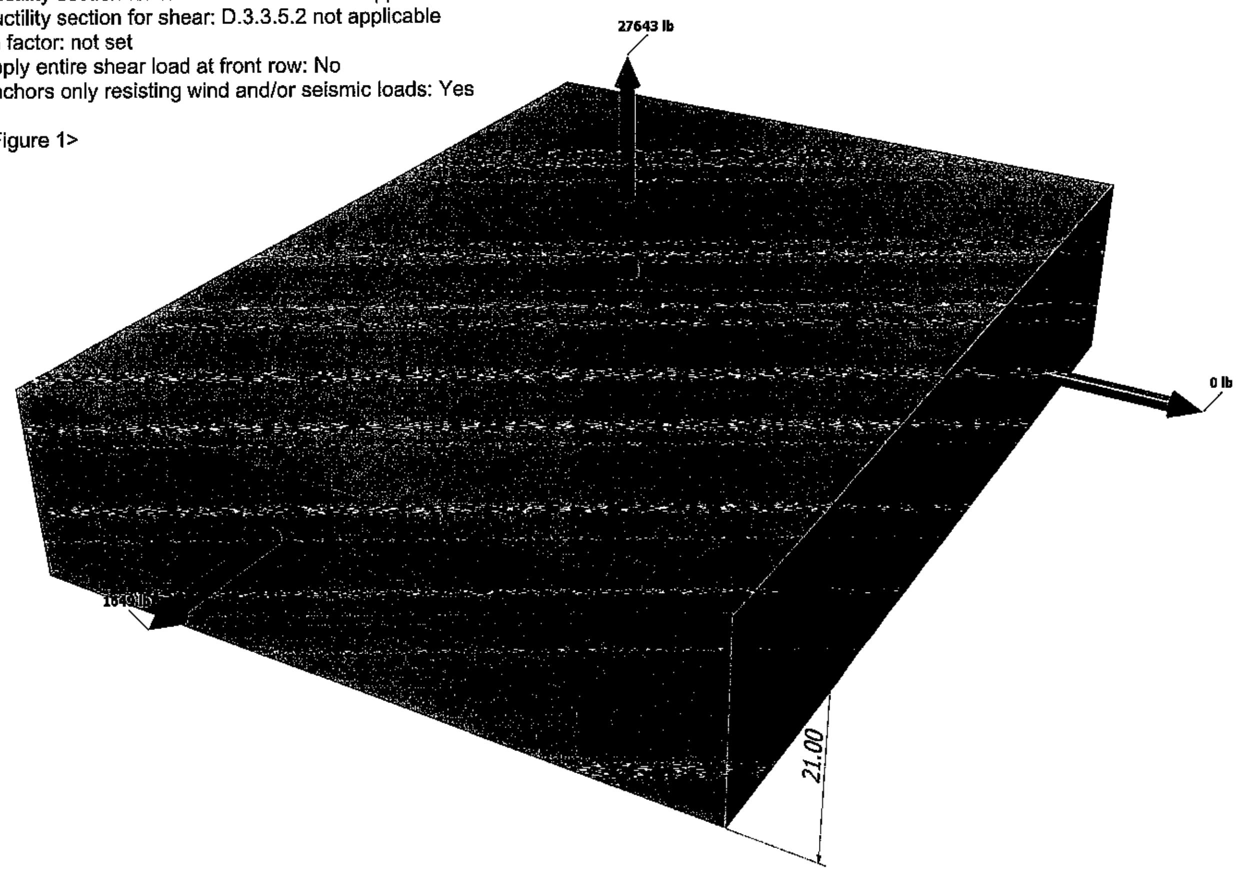
General
Design method:ACI 318-11
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB_H
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 18.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 20.63
C_{min} (inch): 6.00
S_{min} (inch): 6.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 21.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

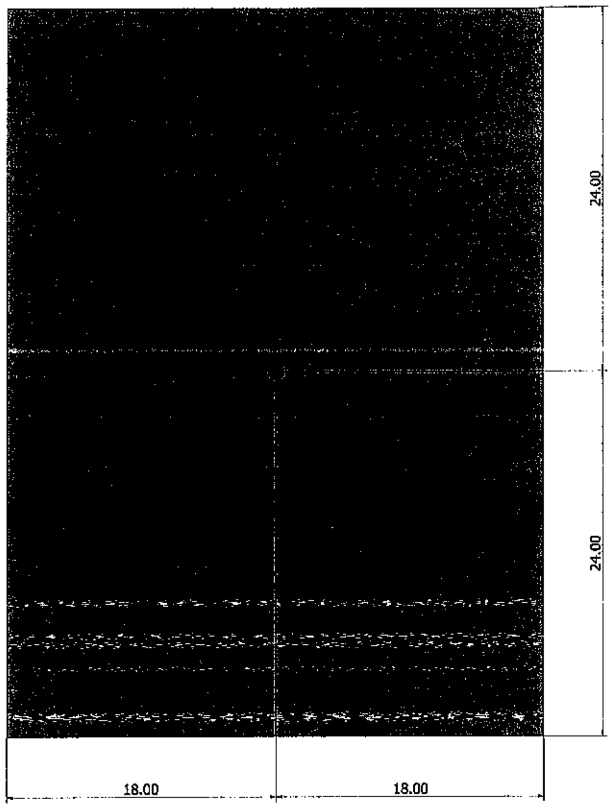
Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: D.3.3.4.2 not applicable
Ductility section for shear: D.3.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

<Figure 1>



Company:	Harriott & Valentine	Date:	1/24/2018
Engineer:	Jonathan Carlson	Page:	2/5
Project:			
Address:			
Phone:			
E-mail:			

<Figure 2>



Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB8H (1"Ø)

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	27643.0	1649.0	0.0	1649.0
Sum	27643.0	1649.0	0.0	1649.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 27643
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. D.5.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
72720	0.75	54540

5. Concrete Breakout Strength of Anchor in Tension (Sec. D.5.2)

$N_b = 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3}$ (Eq. D-7)

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	2500	16.000	81275

$0.75\phi N_{cb} = 0.75\phi\,(A_{Nc}/A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. D.4.1 & Eq. D-3)

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
1728.00	2304.00	18.00	0.925	1.00	1.000	81275	0.70	29602

6. Pullout Strength of Anchor in Tension (Sec. D.5.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}N_p = 0.75\phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. D.4.1, Eq. D-13 & D-14)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	5.15	2500	0.70	54117

8. Steel Strength of Anchor in Shear (Sec. D.6.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
43630	1.0	0.65	28360

9. Concrete Breakout Strength of Anchor in Shear (Sec. D.6.2)

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. D-33 & Eq. D-34)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
8.00	1.000	1.00	2500	14.00	23572

$\phi V_{cbx} = \phi\ (A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. D.4.1 & Eq. D-30)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbx} (lb)
756.00	882.00	0.957	1.000	1.000	23572	0.70	13537

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. D-33 & Eq. D-34)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
8.00	1.000	1.00	2500	16.00	28800

$\phi V_{cbx} = \phi\ (2)(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. D.4.1 & Eq. D-30)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbx} (lb)
1008.00	1152.00	1.000	1.000	1.069	28800	0.70	37716

10. Concrete Pryout Strength of Anchor in Shear (Sec. D.6.3)

$\phi V_{cp} = \phi k_{cp}N_{cb} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. D.4.1 & Eq. D-40)

k_{cp}	A_{Nc} (in²)	A_{Nco} (in²)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
2.0	1728.00	2304.00	0.925	1.000	1.000	81275	0.70	78938

11. Results

Interaction of Tensile and Shear Forces (Sec. D.7.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	27643	54540	0.51	Pass
Concrete breakout	**27643**	**29602**	**0.93**	**Pass (Governs)**
Pullout	27643	54117	0.51	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	1649	28360	0.06	Pass
T Concrete breakout x+	**1649**	**13537**	**0.12**	**Pass (Governs)**
|| Concrete breakout y-	**1649**	**37716**	**0.04**	**Pass (Governs)**
Pryout	1649	78938	0.02	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. D.7..1	0.93	0.00	93.4 %	1.0	Pass

PAB8H (1"Ø) with hef = 18.000 inch meets the selected design criteria.

12. Warnings

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.4.3 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.5.3 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

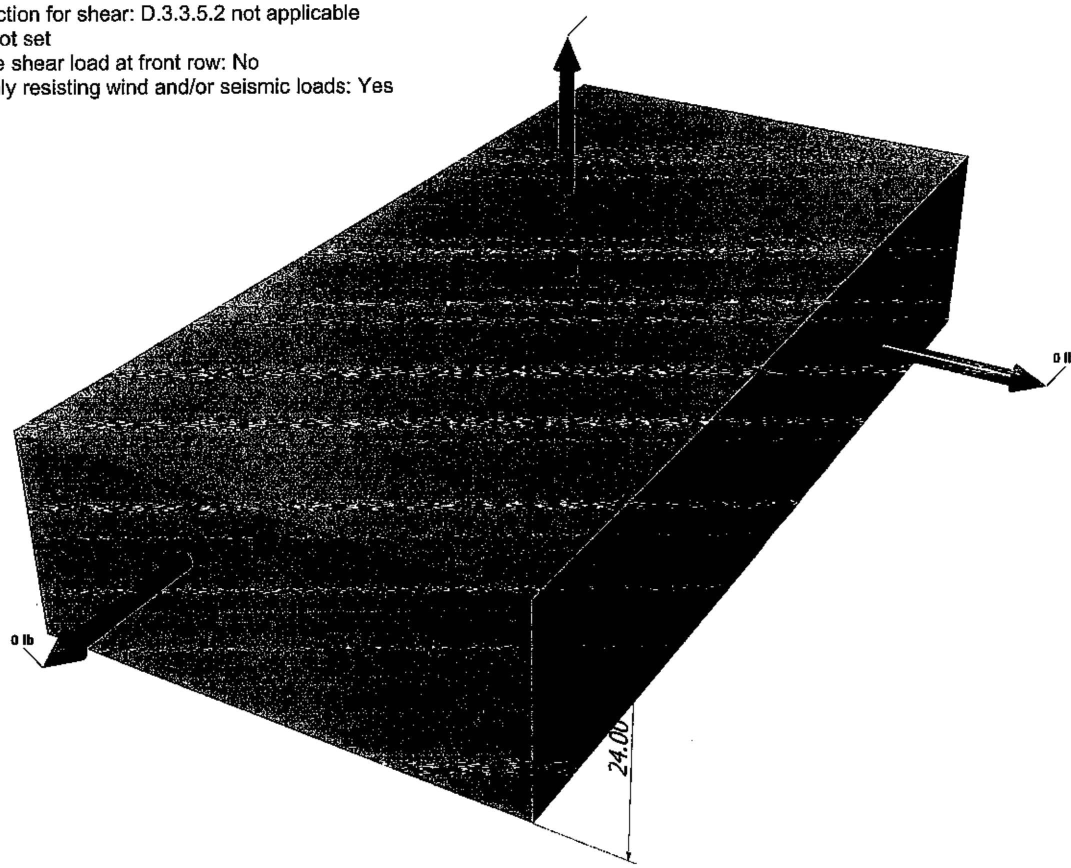
General
Design method:ACI 318-11
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 16.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 18.63
C_{min} (inch): 6.00
S_{min} (inch): 6.00

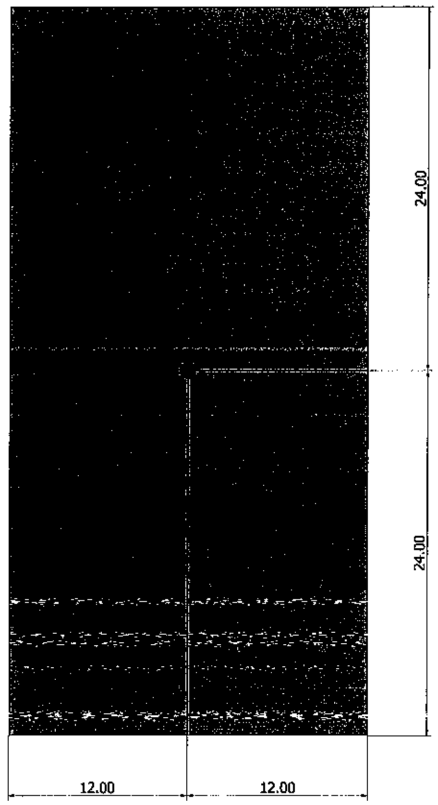
Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 24.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: D.3.3.4.2 not applicable
Ductility section for shear: D.3.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

<Figure 1>



<Figure 2>



Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB8 (1"Ø)

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	12280.0	0.0	0.0	0.0
Sum	12280.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 12280
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. D.5.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
35150	0.75	26363

5. Concrete Breakout Strength of Anchor in Tension (Sec. D.5.2)

$N_b = 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3}$ (Eq. D-7)

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	2500	16.000	81275

$0.75\phi N_{cb} = 0.75\phi\ (A_{Nc} / A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. D.4.1 & Eq. D-3)

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
1152.00	2304.00	12.00	0.850	1.00	1.000	81275	0.70	18134

6. Pullout Strength of Anchor in Tension (Sec. D.5.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}N_p = 0.75\phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. D.4.1, Eq. D-13 & D-14)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	5.15	2500	0.70	54117

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	12280	26363	0.47	Pass
Concrete breakout	**12280**	**18134**	**0.68**	**Pass (Governs)**
Pullout	12280	54117	0.23	Pass

PAB8 (1"Ø) with hef = 16.000 inch meets the selected design criteria.

12. Warnings

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.4.3 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.5.3 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

SIMPSON Strong-Tie® Anchor Designer™ Software
Version 2.5.6582.53

Company:	Harriott & Valentine	Date:	1/29/2018
Engineer:	Jonathan Carlson	Page:	1/4
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: East wall, upper Hdu2's
Location:
Fastening description:

2. Input Data & Anchor Parameters

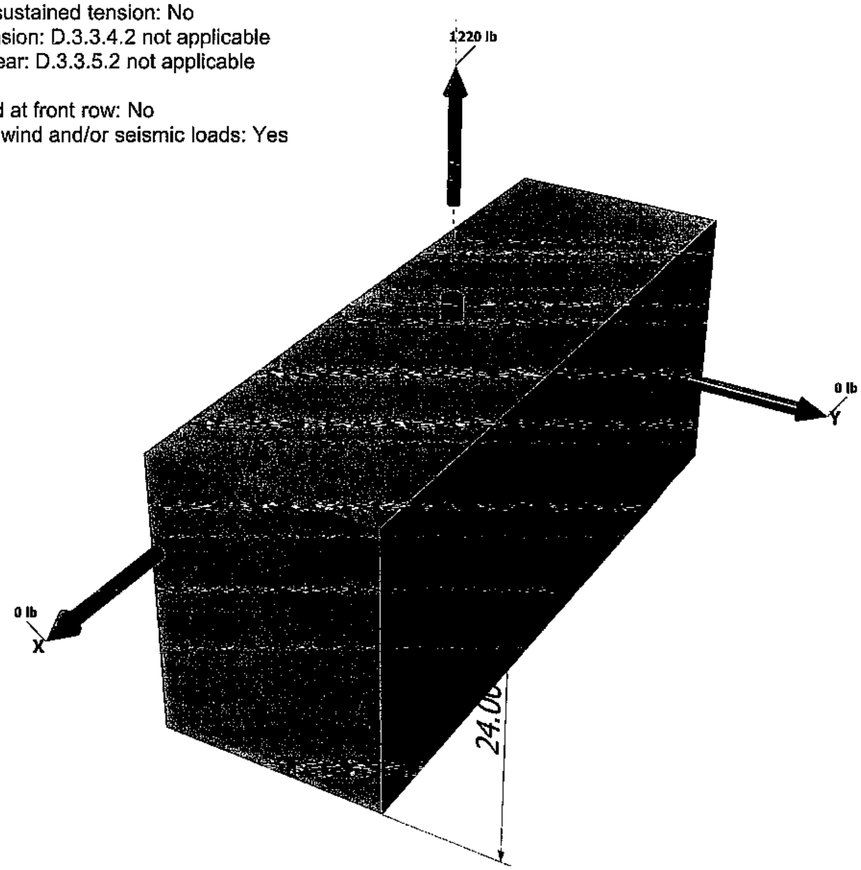
General
Design method:ACI 318-11
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 10.000
Code report: ICC-ES ESR-2508
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 13.13
c_{ac} (inch): 17.02
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 24.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: No
Ductility section for tension: D.3.3.4.2 not applicable
Ductility section for shear: D.3.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

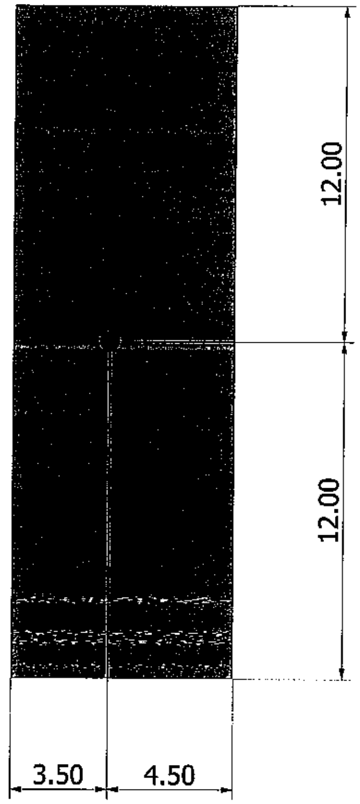
<Figure 1>



SIMPSON Strong-Tie® Anchor Designer™ Software
Version 2.5.6582.53

Company:	Harriott & Valentine	Date:	1/29/2018
Engineer:	Jonathan Carlson	Page:	2/4
Project:			
Address:			
Phone:			
E-mail:			

<Figure 2>



Recommended Anchor
Anchor Name: SET-XP® - SET-XP w/ 5/8"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-2508

Company:	Harriott & Valentine	Date:	1/29/2018
Engineer:	Jonathan Carlson	Page:	3/4
Project:			
Address:			
Phone:			
E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	1220.0	0.0	0.0	0.0
Sum	1220.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 1220
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. D.5.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13110	0.75	9833

5. Concrete Breakout Strength of Anchor in Tension (Sec. D.5.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. D-6)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	8.000	19233

$0.75\phi N_{cb} = 0.75\phi\,(A_{Nc}/A_{Nco})\,\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. D.4.1 & Eq. D-3)

A_{Nc} (in²)	A_{Nco} (in²	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
192.00	576.00	3.50	0.788	1.00	1.000	19233	0.65	2461

6. Adhesive Strength of Anchor in Tension (Sec. D.5.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat} \alpha_{N,seis}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\alpha_{N,seis}$	$\tau_{k,cr}$ (psi)
435	1.72	1.00	1.00	748

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. D-22)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	748	0.63	10.000	14691

$0.75\phi N_a = 0.75\phi\,(A_{Na}/A_{Na0})\,\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}$ (Sec. D.4.1 & Eq. D-18)

A_{Na} (in²)	A_{Na0} (in²)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{p,Na}$	N_{a0} (lb)	ϕ	$0.75\phi N_a$ (lb)
128.74	258.98	8.05	3.50	0.830	1.000	14691	0.65	2957

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	1220	9833	0.12	Pass
Concrete breakout	**1220**	**2461**	**0.50**	**Pass (Governs)**
Adhesive	1220	2957	0.41	Pass

SET-XP w/ 5/8"Ø F1554 Gr. 36 with hef = 10.000 inch meets the selected design criteria.

12. Warnings

- When cracked concrete is selected, concrete compressive strength used in concrete breakout strength in tension, adhesive strength in tension and concrete pryout strength in shear for SET-XP adhesive anchor is limited to 2,500 psi per ICC-ES ESR-2508 Section 5.3.

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.4.3 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.5.3 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

Anchor Bolts – MF 1

Max ↓ Asd = 29.539 K (1.1365D + 1.75E)

Max ↑ Asd = 20.463 K (.4635D + 1.75E)

Max ↓ Lrfd = 45.053 K 1.395D + L + .2S + 2.5E

Max ↑ Lrfd = 28.997 K .705D + 2.5E

Bearing on concrete

$$\frac{45.06K}{(.65)(.85)(2.5)} = 32.63 \text{ in}^2$$

W18X50 D = 18" BF = 7.5"

Min area = (20")(8") = 160 in^2 ✓

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Seattle, Washington 98101-2447
tel. 206 624 4760 | fax 447 6971

Moment frame base plate

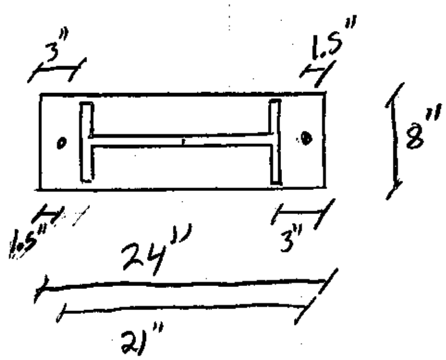
Max ↓ (Asd) = 28.92 K   Max ↓ (Lrfd) = 43.18 K

Max ↑ (Asd) = 20.59 K   Max ↑ (Lrfd) = 29.28 K

Required bearing = $\frac{(43.18)}{(.65)(2.5)} = 26.57 \text{ in}^2$ Area Column = 18 X 7.5 ✓

$(.55)(2.5)(135)\left[1 - \left(\frac{(.8)(18)}{(32)(108)}\right)^2\right] = (185.6\,K)(\phi)$ ✓ pass axial well

Base PL = (18 + 6)" X 8" = 24 X 8



Section 14-5

Base PL

t_{min} = Max (.355", .029", .287", .407")

Bending PL axial ↓

Make 1/2" thick

Bending PL tension

T = 20.59/2 = 10.295 K

@ 1.5" = 1.286 K·ft

Make 5/8" thick

M_2 = 1.4 K·ft ≥ 1.286 K·ft ✓

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Seattle, Washington 98101-2447
tel. 206 624 4760 | fax 447 6971

F

MOMENT FRAME BASEPLATES

END COLUMNS

E = ± 7.8^k
W = ± 5^k

D = 14.9 k
L = 10.2^k
S = 6.0 k
E = ± 13^k
W = ± 7.7^k

MIDDLE COLUMN

E = ± 13.3 k
W = ± 8.5^k

D = 5.04^k
L = 1.6^k
S = 1.9 k
E = 0
W = 0

0.9D + 1.0E

± 7.8^k E

13.4^k D
± 13^k E

0.9D + 1.0E

± 13.3^k E

4.5^k D
± 0^k E

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tel. 206 624 4760 | fax 447 6971

SIMPSON Strong-Tie® Anchor Designer™ Software
Version 2.5.6582.53

Company:	Harriott & Valentine	Date:	2/7/2018
Engineer:	Jonathan Carlson	Page:	1/4
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-11
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 18.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 20.63
C_{min} (inch): 6.00
S_{min} (inch): 6.00

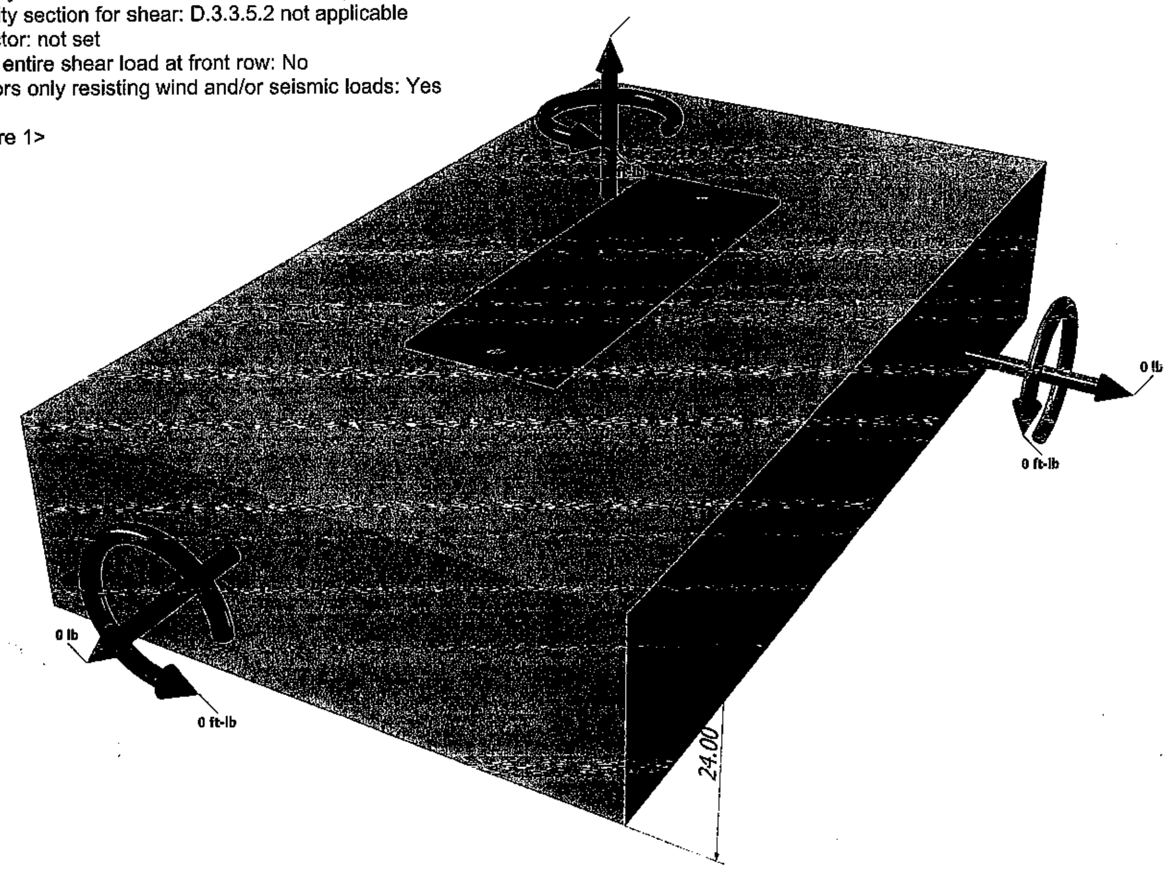
Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: D.3.3.4.2 not applicable
Ductility section for shear: D.3.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 24.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

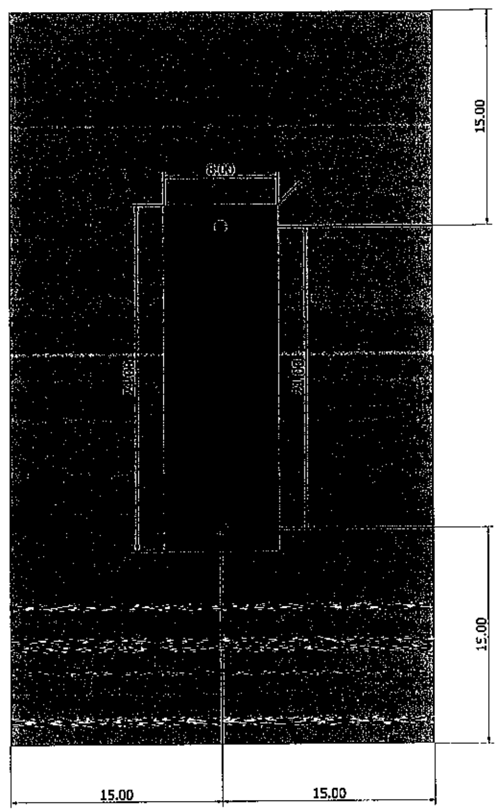
Base Plate
Length x Width x Thickness (inch): 24.00 x 8.00 x 0.25
Yield stress: 36000 psi

Profile type/size: W18X50

<Figure 1>



<Figure 2>



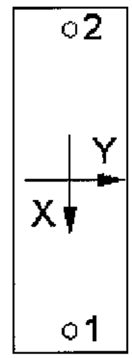
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB8 (1"Ø)

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	14644.0	0.0	0.0	0.0
2	14644.0	0.0	0.0	0.0
Sum	29288.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 29288
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. D.5.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
35150	0.75	26363

5. Concrete Breakout Strength of Anchor in Tension (Sec. D.5.2)

$N_b = 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3}$ (Eq. D-7)

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	2500	10.000	37133

$0.75\phi N_{cbg} = 0.75\phi\,(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. D.4.1 & Eq. D-4)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cbg}$ (lb)
1530.00	900.00	15.00	1.000	1.000	1.00	1.000	37133	0.70	33141

6. Pullout Strength of Anchor in Tension (Sec. D.5.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}N_p = 0.75\phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. D.4.1, Eq. D-13 & D-14)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	5.15	2500	0.70	54117

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	14644	26363	0.56	Pass
Concrete breakout	**29288**	**33141**	**0.88**	**Pass (Governs)**
Pullout	14644	54117	0.27	Pass

PAB8 (1"Ø) with hef = 18.000 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.707 inch
Warning: input base plate thickness does not meet required base plate thickness.

12. Warnings

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.4.3 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.5.3 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

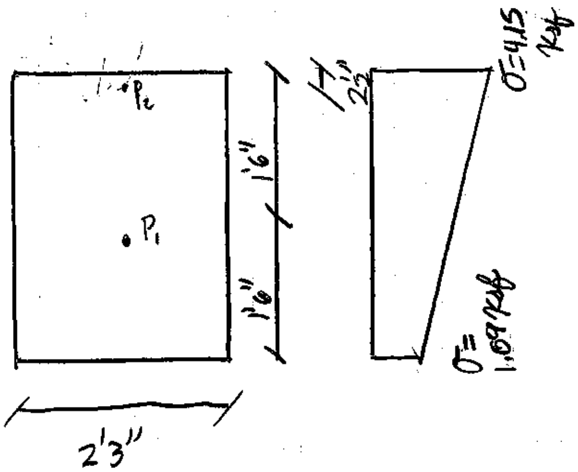
Pad → Basement Corner

$P_1 = 5020D$ $7689L$ $P_2 = 1515D$ $2485L$

$P = 6535D$
$10170L$
7658

$D+L - 16.71K$

$M_{ot} = 5.16$ K.ft



2 way shear for P_2
15.75 K ✓

Depth = 12"

(Asd)
$v = 6.93$ K
$m = 3.98$ K.ft

·(1.6) = (Lrfd)
$V_u = 11.08$ K
$M_u = 6.36$ K.ft

$\frac{1}{2}\phi\ V_c = 12.15$ K

$\phi\ M_n = 14.21$ K.ft
(2 #4 T & B)

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Strip footing

PLF = 480D / 715L Make 9" deep, 12" wide provides 4.5 KLF

Max P = 4.8K

$\frac{1}{2}\phi V_c$ = 4.05 K ✓ ϕM_n = 8.46 K, ft

$A_{s\,min}$ = (2)#4

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Egress window

$w_w = \frac{23D}{38S}{90L} + 113D + 99D + \frac{10D}{28L} \Big| \frac{245D}{38S}{118L}$

$1.2D + 1.6L + .5S$

$w_u = 502$ plf

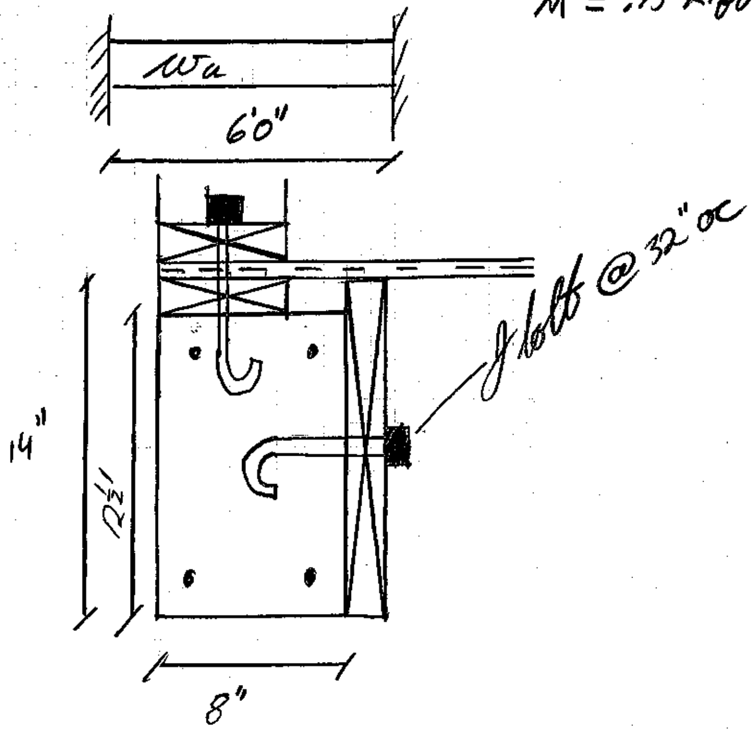
$V = 1.506$ K

$M = .75$ K·ft

$\frac{1}{2}\phi V_c = 4.2$ K ✓

$\phi M_n = 14.23$ K·ft ✓

(2)(#4 T)



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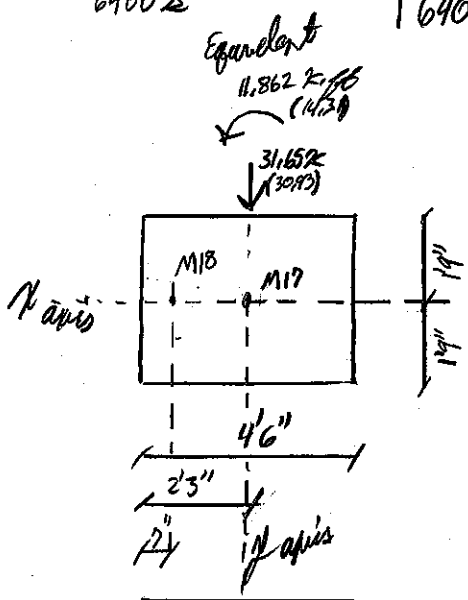
F

footing under M17 & M18

P = 11,895 D + 2715 D | 14,610 D
10,450 L + 5870 L | 16,320 L
6400 S | 6400 S

Max ↓ = 31.65 K at = 11,862 K·ft

D+L ↓ = 30.93 K at = 14.31 K·ft

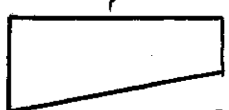


Depth = 24"

2 Way shear M18 = 65.7 K ✓

1.2D+1.6L+1.0S (P_u = 50.04 K, M_u = 21.08 K·ft)

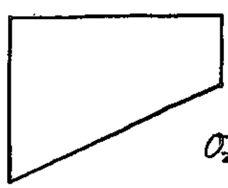
X	Y
V = 34.1 K	V = 27.1 K
M = 41.07 K·ft	M = 30.46 K·ft



−D+.25S+.75L ·(1.6)

X	Y
V = 33.84 K	V = 27.1 K
M = 40.44 K·ft	M = 30.96 K·ft

σ_1 = 3.187 ksf σ_2 = 1.179 ksf



−D+L ·(1.6)

X	Y
V = 34.57 K	V = 26.9 K
M = 41.75 K·ft	M = 30.3 K·ft

σ_2 = .926 ksf

σ_1 = 3.349 ksf

X axis

V = 34.6 K $\frac{1}{2}\phi V_c$ = 37.8 K

M = 41.75 K·ft ϕM_n = 88 K·ft

As min = 10 #4 Grade 60

5 T&B

Y axis

V = 27.1 K $\frac{1}{2}\phi V_c$ = 48.6 K

M = 31 K·ft ϕM_n = 105.77 K·ft

As min = 12 #4

Grade 60

6 T&B

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Base PL – M17 P = 11,893D
10,450L
6,400S

Max ↓ = 24.25k (Asd)
37.39k (Lrfd)

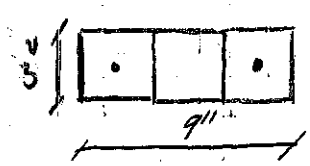
Bearing Area =

$$\frac{(37.39)}{(.65)(.85)(2.5)} = 27\ in^2$$

σ = .898 ksi make 9X3½

σ =

V = 8.08 k
M = 1.01 k·ft



1" PL V₂ = 18.9 k
M₂ = 1.57 k·ft

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Pad footings

Soil Bearing = 4.5 Ksf

1' square Max P = 4.5 K = 7.2 K (Lfd)

depth = 9"

1 way shear = 3.6 K — $\frac{1}{2}\phi V_c$ = 4.05 K

2 Way shear = 7.2 K — $\frac{1}{2}\phi V_c$ (2 way) = 16.88 K (4x4) = 19.57 K (6x6)

M_u = 1.8 K.ft — ϕM_n = 4.64 K.ft (2 #4) Grade 40

1'6" square max P = 10.125 K = 16.2 K (Lfd)

depth = 12"

1 way shear = 8.1 K — $\frac{1}{2}\phi V_c$ = 8.1 K

2 way shear = 16.2 K — $\frac{1}{2}\phi V_c$ (2 way) = 27.9 K (4x4) ✓

M_n = 6.08 K.ft — ϕM_n = 12.26 K.ft (3 #4) Grade 40

2' square max P = 18 K = 28.8 K (Lfd)

depth = 1'4"

1 way shear = 14.4 K — $\frac{1}{2}\phi V_c$ = 14.4 K

2 way shear = 28.8 K — $\frac{1}{2}\phi V_c$ = 46.8 K (4x4)

M_n = 14.4 K.ft — ϕM_n = 25.78 K.ft (4 #4) Grade 40

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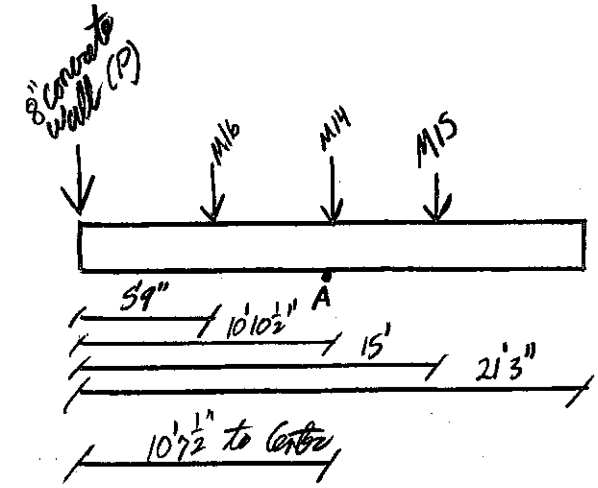
Strip footing | E ↔ W



E going East

M16↓ M14↓ M15↑

E going West

M16↑ M14↑ M15↓

Estimated 24"x24"

	Min (Asd)	Max (Asd)	Min (Lrfd)	Max (Lrfd)	M' (.6D+.7E)	.9D+1.0E
M16	-7.91 k	9.09 k	-11.27 k	14.67 k	8.79 k	12.6
M14	-6.38 k	10.57 k	-9.03 k	16.71 k	9.01 k	12.96
M15	-7.52 k	31.22 k	-10.31 k	41.99 k	19.65 k	28.51

All min P are .6D+.7E / .9D−1.0E } take Min as { .6D+.7E / .9D+1.0E }

E → (West) (M+ ↻)

$M_{ot} = -(P)(10.625') + (7.91)(10.625 - 5.75) - (6.38)(10.875 - 10.625) + (19.65)(15 - 10.625)$

P=0 M15=19.65 L=2.96' σ=4.308 ksf M_{ot}=122.9 P_{net} = 5.36

M15=31.22 L=10.47' σ= 2.32 ksf M_{ot}=173.5 P_{net}=16.93

E← P=0 M14=9.01, M16=8.79 L'=19.39' σ=.91 ksf M_{ot}=73.49 P_{net}=10.28

East P=0 M14=10.57, M16=9.09 L'=20.42' σ=.956 ksf M_{ot}=79.57 P_{net}=12.14

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Strip footing Continued

ℰ →
(West)

	(Asd)	(1.6) = Lrfd
Light	$V = 11.72k$	$V = 18.75k$
	$M = 60.31 k\cdot ft$	$M = 96.5 k\cdot ft$
Heavy	$V = 18.19k$	$V = 29.1k$ @ M15
	$M = 65.88 k\cdot ft$	$M = 105.4 k\cdot ft$

24X24
$\frac{1}{2}\phi V_c = 21.6k$
$\phi M_n = 120.2 k\cdot ft$
(3)#4 T&B

$21.6k - 29.1k =$
$7.5k$
$\frac{21.6}{1.6} = 13.5k$
$V = 12.99k$ @ 17.5'

stirrups between
15' − 1' (good measure)
& 17.5' + 1' (good measure)

← ℰ
(East)

	(Asd)	(1.6) = (Lrfd)
Light	$V = 7.3k$	$V = 11.68k$
	$M = 22.07 k\cdot ft$	$M = 35.3 k\cdot ft$
Heavy	$V = 8.18k$	$V = 13.09k$
	$M = 25.92 k\cdot ft$	$M = 40.46$

#3 hoop, Grade 40

$\frac{A_s \cdot f_y \cdot d}{S} = 7.5k$

(k/in², in², in, in) ✓ (units)

$\frac{(40)(.11)(2)(24/2)}{S\ in} = 7.5k$

$S = 14.08$ in make
@ 12" OC = 8.8k (V_s)

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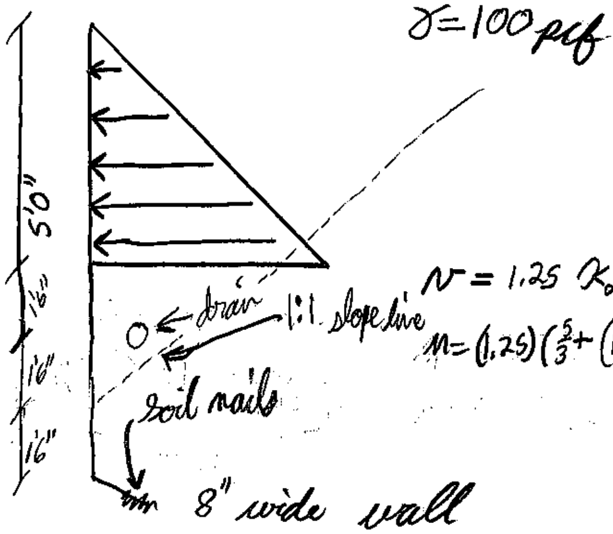
Catchment Wall



$v = 1.25$ klf — Asd

$M = (1.25)(\frac{5}{3} + (1\frac{1}{2})(3)) = 7.708$ k·ft

Lrfd

$v = 2.0$ klf

$M = 12.33$ k·ft

$\frac{1}{2}\phi V_c = 3.6$ klf

$\phi M_n = 13.28$ k·ft

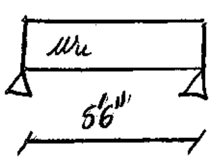
Grade 60 long CL @ 3" oc

$D = 3.75$" #6

$3" - .75 = 2.25"$ — can still fit in wall

$w_u = 1250$ plf

tolerance = ± $\frac{1}{2}$" of wall CL



1.6H

$v = 5.5$ K

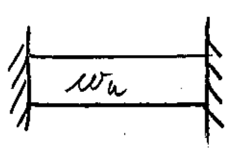
$M = 11.56$ K·ft

12" deep by 8" width — fail

2' width $\frac{1}{2}\phi V_c = 7.2$ K

8" depth $\phi M_n = 10.29$ k·ft

Grade 60 long CL (4) #4 spacing 3.375"



1.6H

$v = 5.5$ K

$M = 5.04$ K·ft

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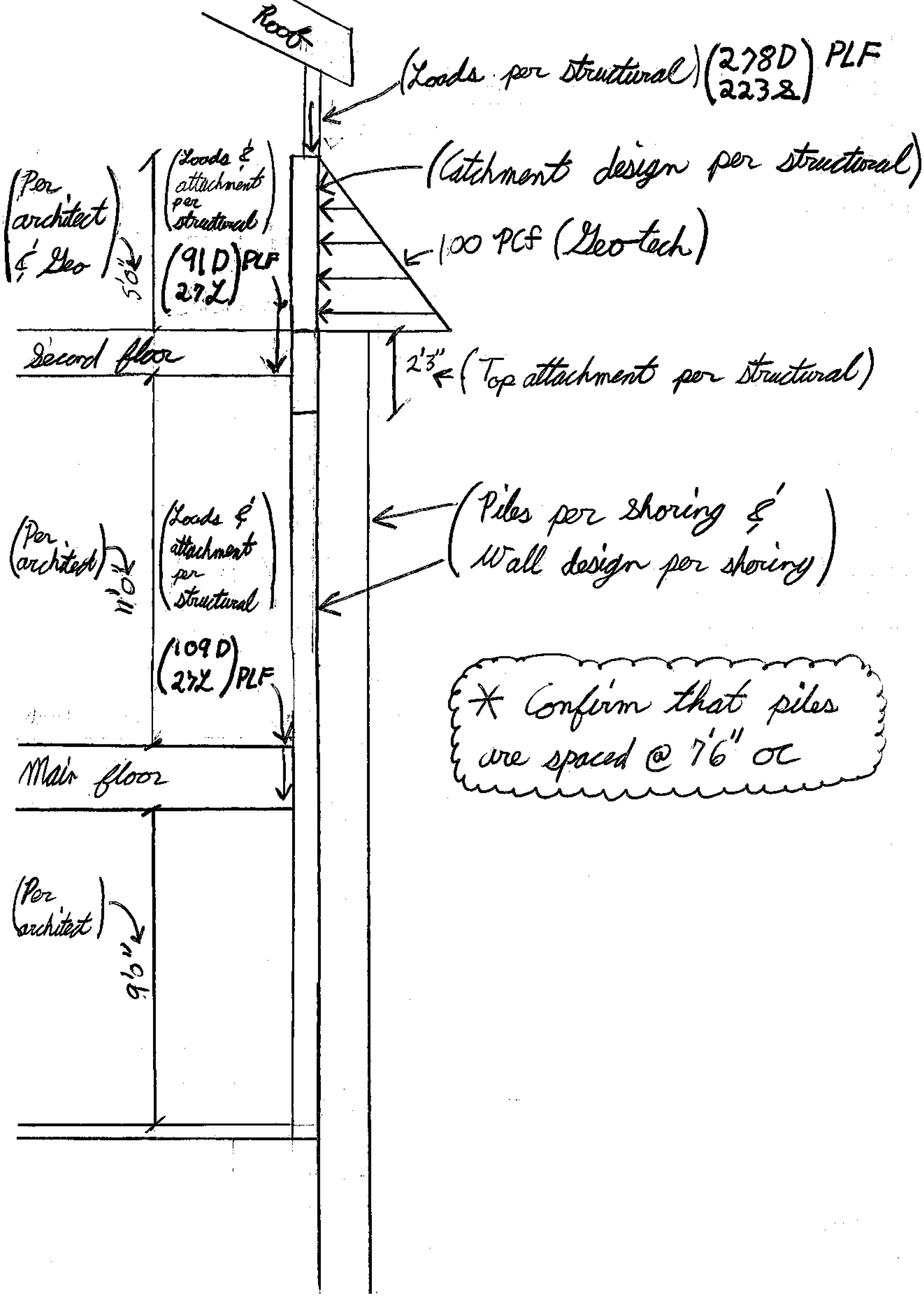

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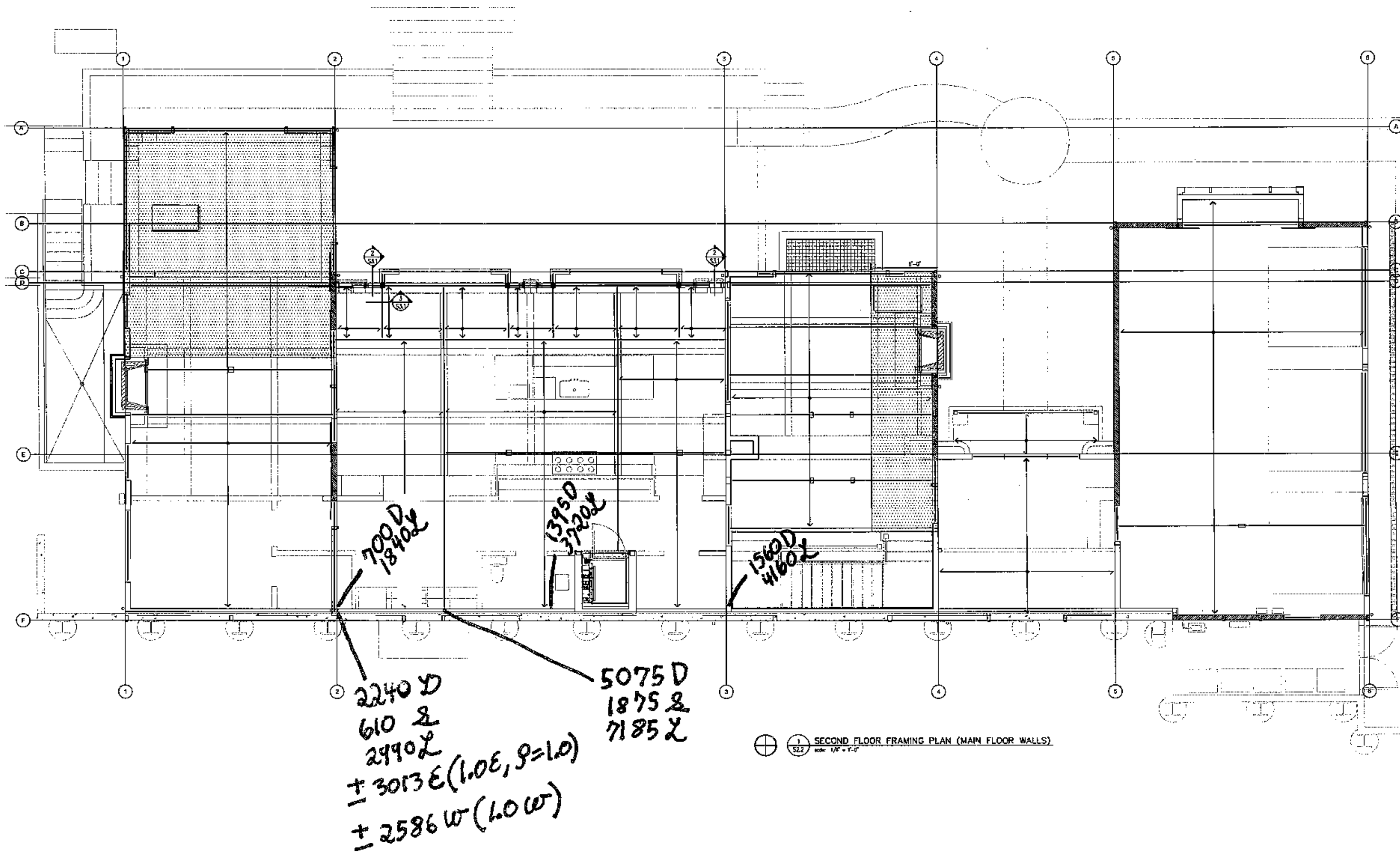
* Red are point Loads acting at the Second floor
* Green are point loads acting at the Main floor



HVE

NOT FOR CONSTRUCTION

Tangled Ride Residence

SECOND FLOOR FRAMING PLAN

S2.2

SIMPSON Strong-Tie®

Anchor Designer™ Software
Version 2.5.6582.53

Company:	Harriott & Valentine	Date:	1/19/2018
Engineer:	Jonathan Carlson	Page:	1/4
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: Gravity to pile
Location:
Fastening description:

2. Input Data & Anchor Parameters

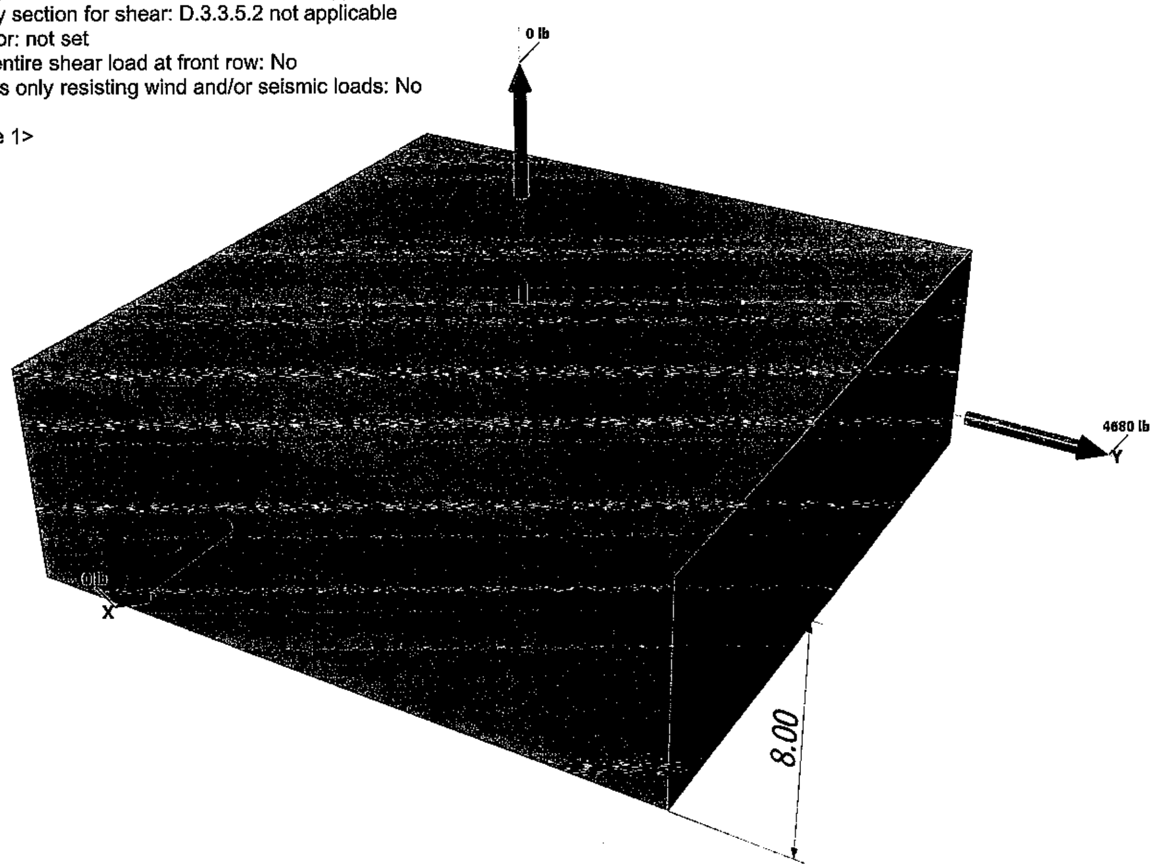
General
Design method:ACI 318-11
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AWS Type A
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 5.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 6.38
C_{min} (inch): 1.38
S_{min} (inch): 2.50

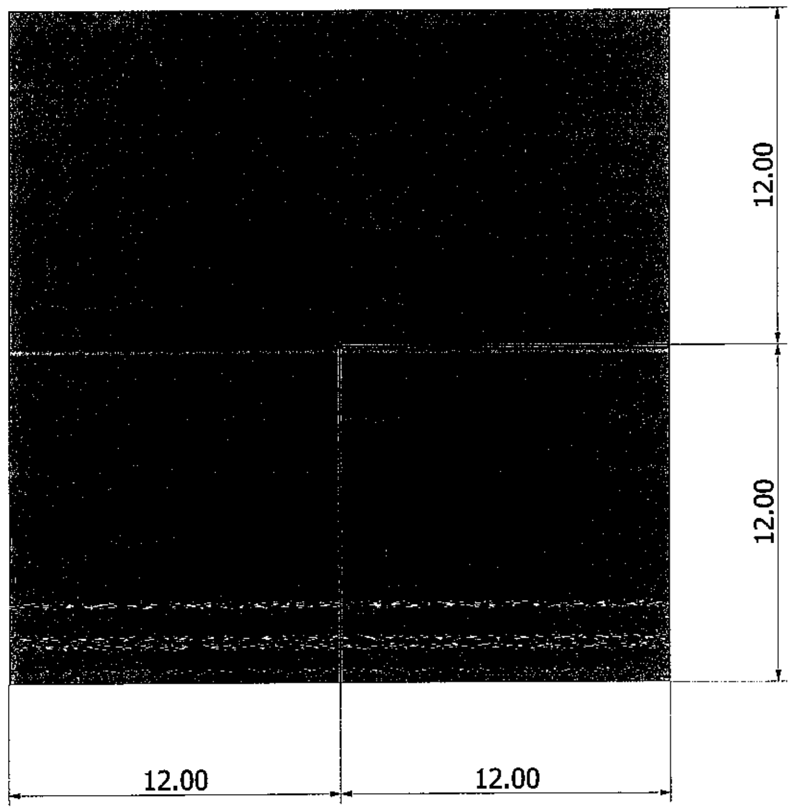
Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 8.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: D.3.3.4.2 not applicable
Ductility section for shear: D.3.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

<Figure 1>



<Figure 2>



Recommended Anchor
Anchor Name: Headed Stud - 5/8"Ø AWS Type A Headed Stud

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	0.0	4680.0	4680.0
Sum	0.0	0.0	4680.0	4680.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

8. Steel Strength of Anchor in Shear (Sec. D.6.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
18715	1.0	0.65	12165

9. Concrete Breakout Strength of Anchor in Shear (Sec. D.6.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. D-33 & Eq. D-34)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
5.00	0.625	1.00	2500	8.00	9490

$\phi V_{cby} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. D.4.1 & Eq. D-30)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cby} (lb)
192.00	288.00	1.000	1.000	1.225	9490	0.70	5424

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. D-33 & Eq. D-34)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
5.00	0.625	1.00	2500	8.00	9490

$\phi V_{cby} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. D.4.1 & Eq. D-30)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cby} (lb)
192.00	288.00	1.000	1.000	1.225	9490	0.70	10848

10. Concrete Pryout Strength of Anchor in Shear (Sec. D.6.3)

$\phi V_{cp} = \phi k_{cp}N_{cb} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. D.4.1 & Eq. D-40)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
2.0	225.00	225.00	1.000	1.000	1.000	13416	0.70	18783

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	4680	12165	0.38	Pass
T Concrete breakout y+	**4680**	**5424**	**0.86**	**Pass (Governs)**
|| Concrete breakout x-	**4680**	**10848**	**0.43**	**Pass (Governs)**
Pryout	4680	18783	0.25	Pass

5/8"Ø AWS Type A Headed Stud with hef = 5.000 inch meets the selected design criteria.

12. Warnings

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.4.3 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.5.3 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

SIMPSON Strong-Tie® Anchor Designer™ Software Version 2.5.6582.53

Company:	Harriott & Valentine	Date:	1/19/2018
Engineer:	Jonathan Carlson	Page:	1/4
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: catchment attachment
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-11
Units: Imperial units

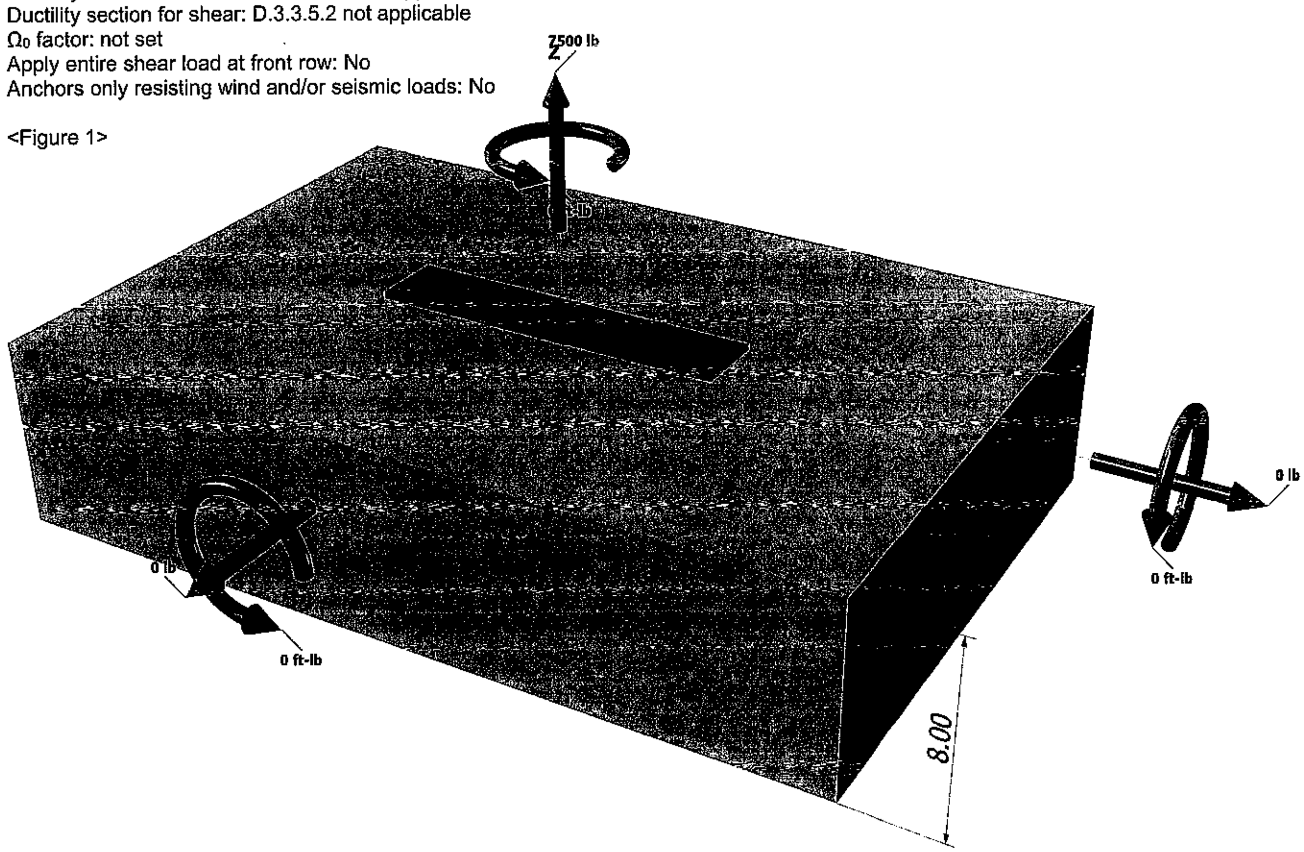
Anchor Information:
Anchor type: Cast-in-place
Material: AWS Type A
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 5.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 6.38
C_{min} (inch): 1.38
S_{min} (inch): 2.50

Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: D.3.3.4.2 not applicable
Ductility section for shear: D.3.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

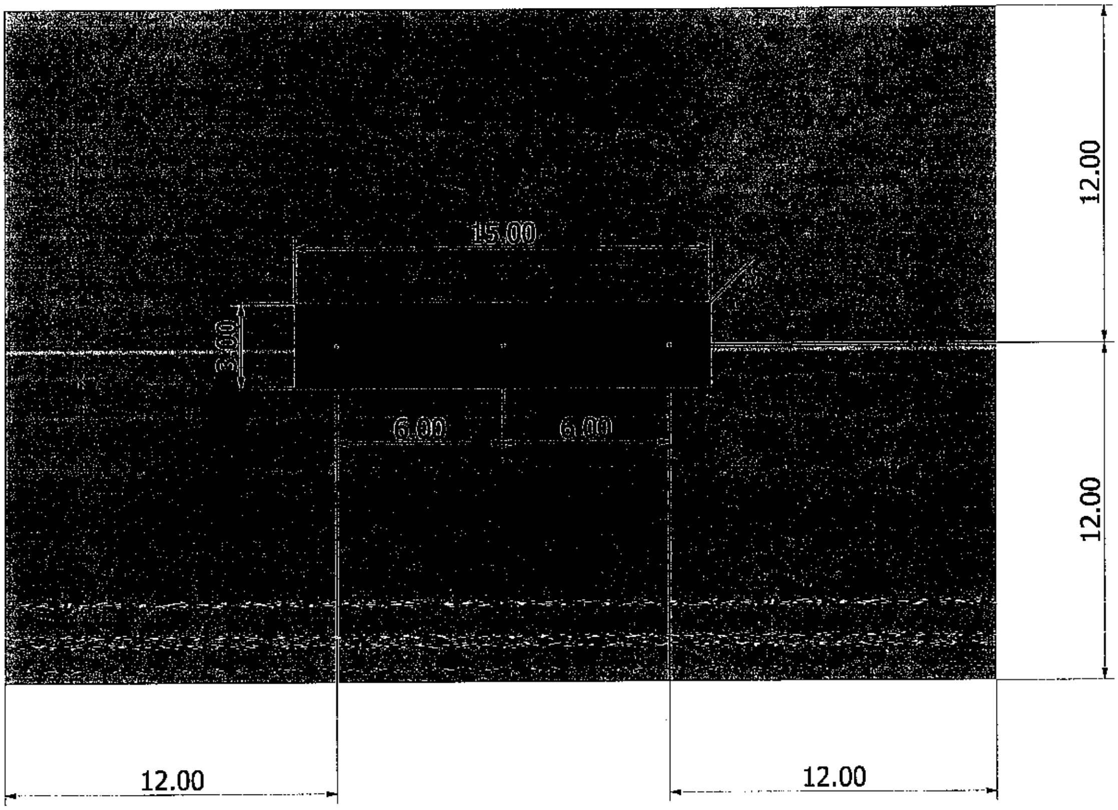
Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 8.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 3.00 x 15.00 x 0.25

<Figure 1>



<Figure 2>



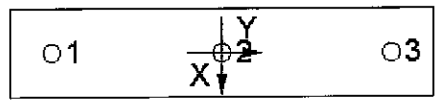
Recommended Anchor
Anchor Name: Headed Stud - 5/8"Ø AWS Type A Headed Stud

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	2500.0	0.0	0.0	0.0
2	2500.0	0.0	0.0	0.0
3	2500.0	0.0	0.0	0.0
Sum	7500.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 7500
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. D.5.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
18715	0.75	14036

5. Concrete Breakout Strength of Anchor in Tension (Sec. D.5.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. D-6)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	2500	5.000	13416

$0.75\phi N_{cbg} = 0.75\phi\,(A_{Nc}/A_{Nco})\,\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. D.4.1 & Eq. D-4)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cbg}$ (lb)
405.00	225.00	12.00	1.000	1.000	1.00	1.000	13416	0.70	12679

6. Pullout Strength of Anchor in Tension (Sec. D.5.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}N_p = 0.75\phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. D.4.1, Eq. D-13 & D-14)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	0.92	2500	0.70	9660

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	2500	14036	0.18	Pass
Concrete breakout	**7500**	**12679**	**0.59**	**Pass (Governs)**
Pullout	2500	9660	0.26	Pass

5/8"Ø AWS Type A Headed Stud with hef = 5.000 inch meets the selected design criteria.

12. Warnings

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.4.3 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.5.3 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.