STRUCTURAL CALCULATIONS

FOR

# STABILIZATION

TO

# NAVARRO INN

NAVARRO RIVER REDWOODS STATE PARK

MENDOCINO COUNTY, CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
ALEX L. ROOD
No. S 004492
Exp. 12/31/10
STRUCTURAL
STATE OF CALIFORNIA

5/16/2011


**FULCRUM STRUCTURAL ENGINEERING**
665 THIRD ST, STE 333
SAN FRANCISCO, CA 94107
415.543.0161

**LOAD TABLE** 08-37: NAVARRO INN

All materials will be redwood, unless otherwise note.

**FLOOR**

| | JOISTS | BEAMS |
|---|---|---|
| FLOOR FINISH (RED WOOD) | 2.5 | 2.5 |
| 1x T&G D.F. | 3.0 | 3.0 |
| 2x @ 16" O.C. | 2.2 | 2.2 |
| INSULATION | 1.8 | 1.8 |
| 1x WOOD | 3.0 | 3.0 |
| BEAM | - | - |
| MISCELLENEOUS | 1.5 | 1.5 |

| TOTAL | | JOISTS | BEAMS | |
|---|---|---|---|---|
| | DL | 14.0 | 14.0 | (psf) |
| | LL | 40.0 | - | (psf) |

**WALLS**

| | EXTERIOR WALL | INTERIOR WALL |
|---|---|---|
| WALL FINISH | 2.0 | - |
| 1" VERT WD | 3.0 | - |
| 1x BEAD BOARD | 3.0 | 3.0 |
| STUDS 2X4 @ 16" | 1.1 | 1.1 |
| INSULATION | 0.5 ~~1.6~~ | - |
| MISCELLENEOUS | ~~1.3~~ | 1.9 |

| TOTAL | EXTERIOR WALL | INTERIOR WALL | |
|---|---|---|---|
| | ~~12.0~~ 10 | 6.0 | DL (psf) |

PARTITION WALL

3/4" RED WOOD = 2 psf.
(3) 2X3. = 0.3 psf.

2.5 psf.

| FULCRUM STRUCTURAL ENGINEERING | Job No.: 08-37 | By: HZ | Date: 9/10/08 | 1.2 / 13 |
|---|---|---|---|---|
| | Name : NAVARRO INN. | Chk: | Date: | |

PARTITION WALL

3/4" RED WOOD = 2 psf
(3) 2X3 = 0.3 psf

2.5 psf.

TOTAL PARTITION WALL @ 2nd FLOOR, L = 15' X 12 = 180'

WT OF PARTITION WALL = 180' X 9' X 2.5 psf = 4050 #

FLOOR AREA = 50' X 27.5' = 1375 ft$^2$.

WALL DL = 4050 # / 1375 ft$^2$ = 3 psf.

1.3/13

| | |
|---|---|
| **PROJECT :** NAVARRO INN | JOB #: 08-37 |
| **ENGR :** HZ | DATE: 8/14/2008 |
| | SHEET: |

**Design Load Table -**

ROOF PITCH = 10.8 : 12

* MULTIPLIER FOR SLOPED ROOF 1.35

| DEAD LOAD | | PLY | | RAFTERS | | TRUSS WEBS | | CLG JOIST | | SEISMIC | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | flat | sloped | flat | sloped | flat | sloped | flat | sloped | flat | sloped |
| ROOFING | **(psf)** | **6.0** | **8.1** | **6.0** | **8.1** | **6.0** | **8.1** | | | **6.0** | **8.1** |
| PLY | **(psf)** | **1.1** | **1.5** | **1.1** | **1.5** | **1.1** | **1.5** | | | **1.1** | **1.5** |
| SKIP SHEATHING | **(psf)** | **1.2** | **1.6** | **1.2** | **1.6** | **1.2** | **1.6** | | | **1.2** | **1.6** |
| MISC | **(psf)** | **0.5** | **0.7** | **0.5** | **0.7** | **0.5** | **0.7** | | | **0.5** | **0.7** |
| RAFTERS | **(psf)** | | | **1.1** | **1.5** | **1.1** | **1.5** | | | **1.1** | **1.5** |
| TRUSS WEBS | **(psf)** | | | | | **0.0** | **0.0** | | | **0.0** | **0.0** |
| CEILING JOISTS | **(psf)** | | | | | | | **1.9** | **1.9** | **1.9** | **1.9** |
| INSULATION | **(psf)** | | | | | | | **1.1** | **1.1** | **1.1** | **1.1** |
| 3/8" PLY | **(psf)** | | | | | | | **0.0** | **0.0** | **0.0** | **0.0** |
| MISC | **(psf)** | | | | | | | ~~**0.7**~~ | **0.7** | **0.7** | **0.7** |
| TOTAL DEAD LOAD | **(psf)** | 8.8 | **11.8** | 9.9 | **13.3** | 9.9 | **13.3** | 3.7 | **3.7** | 13.6 | **17.0** |
| LIVE LOAD | **(psf)** | **16** | **16** | **16** | **16** | **16** | **16** | **16** | **16** | **16** | **16** |

Notes:

1.
2.
3.

SEISMIC

FROM SOIL REPORT, THE SITE LATITUDE IS 39.19

& LONGITUDE IS −123.76, SITE CLASS C, $S_s = 1.581g$

$F_a = 1.0$, $F_v = 1.3$

$S_{MS} = F_a S_s = 1.581$ $\quad S_{DS} = 2/3\, S_{MS} = 1.054$

$C_s = S_{DS} / (R/I) = \frac{1.054}{(6.5/1)} = 0.162$

(SEE PRECEDING PAGE)

$V = C_s W = 0.162\,(78\ k) = 12.6\ k$

ASD: $0.7 \times 12.6 = 8.8\ k$

75% OF CODE SEISMIC $\Rightarrow 0.75 \times 8.8\ k = 6.6\ k$

# Seismic Base Shear Coefficient

| | | |
|---|---|---|
| Applicable Building Code: | 2007 California Building Code/2006 International Building Code w/ references to ASCE 7-05 | |
| Earthquake Load: | $E = E_h + E_v = r\ Q_E + 0.2\ S_{DS}\ D$ | eqtn 12.4-1 |

Equivalent Lateral Force Procedure

| | | |
|---|---|---|
| Earthquake Load (Base Shear): Q | = V = Cs * W | eqtn 12.8-1 |
| Cs | = ( $S_{DS}$ * I / R ) | eqtn 12.8-2 |

| | | |
|---|---|---|
| Soil site class type | = C | |
| Lateral resisting structure type | = All other structural sytesms | |
| Redundancy Factor, ρ | = 1.00 | Section 12.3.4 |
| Importance Factor, I | = 1.00 | Table 11.5-1 |
| Response Modification Coefficient, R | = 4.0 | Table 12.2-1 |
| Building Height, $h_n$ | = 30 | |
| Spectral Response Acceleration, $S_S$ | = 1.581 | USGS Hazmaps |
| Spectral Response Acceleration, $S_1$ | = 0.817 | USGS Hazmaps |
| MCE Spectral Response Acceleration, $S_{MS}$ | = Fa x Ss = 1.00 x 1.581 = 1.581 | eqtn 11.4-1 |
| MCE Spectral Response Acceleration, $S_{M1}$ | = Fv x S1 = 1.30 x 0.817 = 1.062 | eqtn 11.4-2 |
| Design spectral acceleration, $S_{DS}$ | = 2/3 x SMS = 2/3 x 1.581 = 1.054 | eqtn 11.4-3 |
| Design spectral acceleration, $S_{D1}$ | = 2/3 x SM1 = 2/3 x 1.062 = 0.708 | eqtn 11.4-4 |
| **Vertical effect, $E_v$** | = 0.2 x 1.054 x DL = 0.211DL | eqtn 12.4-4 |
| Approximate Fundamental Period, $T_a$ | = Ct x hn^.75 = 0.02x30.0 ft ^0.75 = 0.26 sec | eqtn 12.8-7 |
| $T_o$ | = 0.2xSD1/SDS = 0.2x0.708/1.054 = 0.13 sec | Sect 11.4.5 |
| $T_s$ | = SD1/SDS = 0.708/1.054 = 0.67 sec | Sect 11.4.5 |
| Long Period, $T_L$ | = 8.00 sec | Figure 22-15 |
| Base Shear, $Q_E$ | = (1.05 x 1.00 / 4.0) x W = 0.264W | eqtn 12.8-1 |
| Max Base Shear, $Q_E$ | = 0.71 / [ 0.26 x ( 4.00 / 1.00 )] = 0.690W | eqtn 12.8-3 |
| Min Base Shear, $Q_E$ | = .01W | eqtn 12.8-5 |
| Min Base Shear, $Q_E$ | = 0.5 x 0.817 / ( 4.0 / 1.00 ) = 0.102W | eqtn 12.8-6 |

**Governing Value, $Q_E$ =** 0.264 W (@ Strength)

**$\rho Q_E$ =** 0.264 W (@ Strength)

# Vertical Distribution of Seismic Base Shear

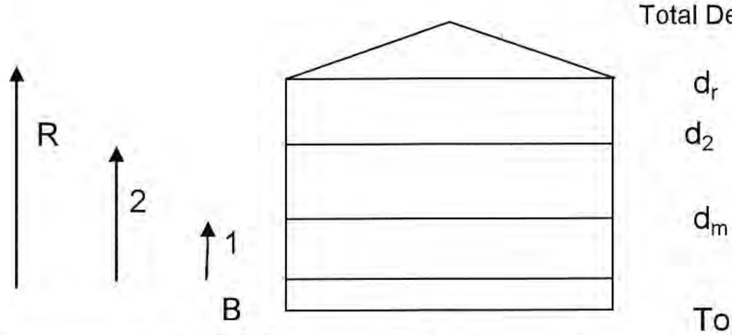
Applicable Building Code: 2007 California Building Code/2006 International Building Code w/ references to ASCE 7-05

Applicable Equation: $F_X = C_{VX} V$ (12.8-11)

USE 75% OF THE CODE SEISMIC LOADS

Total Design Base Shear (V) = 10237 lbs (@ ASD

k = 1.00




| | WALLS | | | | DIAPHRAGMS | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Level | $h_x$ (ft) | Lumped Weight Including Walls $W_x$ (lbs) | $W_x h_x^K$ | $F_x = \frac{V(W_x h_x^K)}{\Sigma(W_x h_x^K)}$ (lbs) | Diaphragm Weight $w_{dx}$** (lbs) | $F_{dx(12.10\text{-}1)} = \frac{(\Sigma F_{x\&up}) w_{dx}}{(\Sigma w_{x\&up})}$ (lbs) | $F_{dx(MIN)} = .2 S_{DS} I W_{px}$ (lbs) | $F_{dx(MAX)} = .4 S_{DS} I W_{px}$ (lbs) | $F_{dx(GOV)}$*= (lbs) |
| Roof | 22.00 | 30000 | 660000 | 5687 | 30000 | 5687 | 6324 | 12648 | 6324 |
| 3rd Flr | 12.00 | 44000 | 528000 | 4550 | 44000 | 6087 | 9275 | 18550 | 9275 |
| | Σ= | 74000 | 1188000 | 10237 | | | | | |

* NOTE: The governing force on the diaphragm shown does not include any off-center intermediate shear wall loads from above.

** NOTE: The diaphragm weight shown does not have any wall weights subtracted out (reasonable for wood-framed residential) For masonry structures, subtract out wall weight parallel to loading direction to avoid excessive conservatism (i.e. only walls perpendicular to the loading direction need to be included here)

1-7/13

- **Part 8 - California Historical Building Code**

    - *Section 8-812, Table 8-8A & Table 8-8B* (Missing tables from July publication)

*Chapter 8-8, Section 8-812*

*Table 8-8A ALLOWABLE VALUES FOR EXISTING MATERIALS*

| EXISTING MATERIALS OR CONFIGURATIONS OF MATERIALS[1] | ALLOWABLE VALUES<br>x14.594 for N/m |
|---|---|
| 1. Horizontal diaphragms[2] | |
| 1.1 Roofs with straight sheathing and roofing applied directly to the sheathing | 100 lbs. Per foot for seismic shear |
| 1.2 Roofs with diagonal sheathing and roofing applied directly to the sheathing | 250 lbs. Per foot for seismic shear |
| 1.3 Floors with straight tongue-and-groove sheathing | 100 lbs. Per foot for seismic shear |
| 1.4 Floors with straight sheathing and finished wood flooring with board edges offset or perpendicular | 500 lbs. Per foot for seismic shear |
| 1.5 Floors with diagonal sheathing and finished | 600 lbs. Per foot for seismic shear |
| 2. Crosswalls[2,3] | |
| 2.1 Plaster on wood or metal lath | Per side: 200 lbs. Per foot for seismic shear |
| 2.2 Plaster on gypsum lath | 175 lbs. Per foot for seismic shear |
| 2.3 Gypsum wallboard, unblocked edges | 75 lbs. Per foot for seismic shear |
| 2.4 Gypsum wallboard, blocked edges | 125 lbs. Per foot for seismic shear |
| Existing footings, wood frming, structural steel and reinforced steel | |
| 3.1 Plain concrete footings | $f'_c$=1,500 psi (10.34 MPa) unless otherwise shown by tests[4] |
| 3.2 Douglas fir wood | Allowable stress same as D.F. No. 1[4] |
| 3.2 Reinforcing steel | $f_t$=18,000 ibs. Per square inch (124.1 M/mm$^2$) maximum |
| 3.4 Structural steel | $f_t$=200.00 lbs. Per square inch (137.9 N/mm$^2$ maximum[4] |

[1]Material must be sound and in good condition.
[2]A one-third increase in allowable stress is not allowed.
[3]Shearvalues of these materials may be combined, except the total combined value shall not exceed 300 pounds per foot (4380 N/m).
[4]Stresses given may be increased for combinations of loads as specified in the regular code.

*Table 8-8B ALLOWABLE VALUES OF NEW MATERIALS USED IN CONNECTION WITH EXISTING CONSTRUCTION*

| NEW MATERIALS OR CONFIGURATIONS OF MATERIALS | ALLOWABLE VALUES[1] |
|---|---|
| 1. Horizontal diaphragms[2] | |
| Plywood sheathing nailed directly over existing straight sheathing with ends of plywood sheets bearing on joists or rafters and edges of ply- wood located on center of individual sheathing boards | 225 lbs. Per foot (3283 N/m) |
| Plywood sheathing nailed directly over existing diagonal sheathing with ends of plywood sheets bearing on joists or rafters | 375 lbs. Per foot (5473 N/m) |
| 1.3 Plywood sheathing nailed directly over existing straight or diagonal sheathing with ends of ply- wood sheets bearing on joists or rafters with edges of plywood located over new blocking and nailed to provide a minimum nail penetra-tion | 75 percent of the values specified in the regular code |

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| | Name : NAVARRO INN | Chk: | Date: | |

LATERAL DESIGN

DESIGN FOR (E) VERT SHTHG AT ROOF

CAP. $v = 100^{\#/\prime}$ ← SEE SECTION 8-8 TABLE 8-6A OF CBC, ATTACHED

$V = 5687^{\#}$ ← SEE VERT DIST SHT

- REDUCE FOR R = 6.5, SEE PREVIOUS PAGE FOR EXPLANATION.

$V = 5687 \times 4/6.5$

$= 3500^{\#}$

ROOF

N/S

GL (C) $V = 3500/2 = 1750^{\#}$

$v = 1750/(8+8+2+2)$

$= 87.5^{\#/\prime} < 100^{\#/\prime}$

∴ (E) SHTHG OK

GL (A) $V = 1750^{\#}$

$v = 1750/(8+6.5+7)$

$= 81^{\#/\prime} < 100^{\#/\prime}$

∴ (E) SHTG OK

## LATERAL DESIGN CONT

ROOF

E/W

GL (2) $V = 3500/2 = 1750^{\#}$

$v = 1750/(7+5+5+3+4+5+3)$

$= 55^{\#/\prime}$

GL (4) $V = 1750^{\#}$

$v = 1750/7+5+5+3+4+5+7$

$= 55^{\#/\prime}$

1st FLR

- USE LOADING BASED ON R=4

N/S

GL (A) $V = 10.24^{k}/4 = 2.56^{k}$

- ASSUME (2) FLAT STRAP BRACE FRM

$v = 2.56/2 = \underline{1.3^{k}}$

$T_{OT} = 1.3^{k} \times 9/6 = 1950^{\#}$

USE HDU2

LATERAL DESIGN CONT

2nd FLR

N/S

GL (B) V = 10.24/2 = 5.12^k
v = 5.12/11' = 465
USE 10-3 PLY SW

OT = 465 x 9' = 4.19^K
USE HDU5

GL (C) V = 10.24/4 = 2.56^k
USE SSW 24x9
CAP = 3586 > 2560

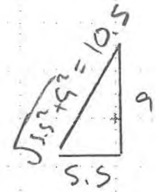
E/W

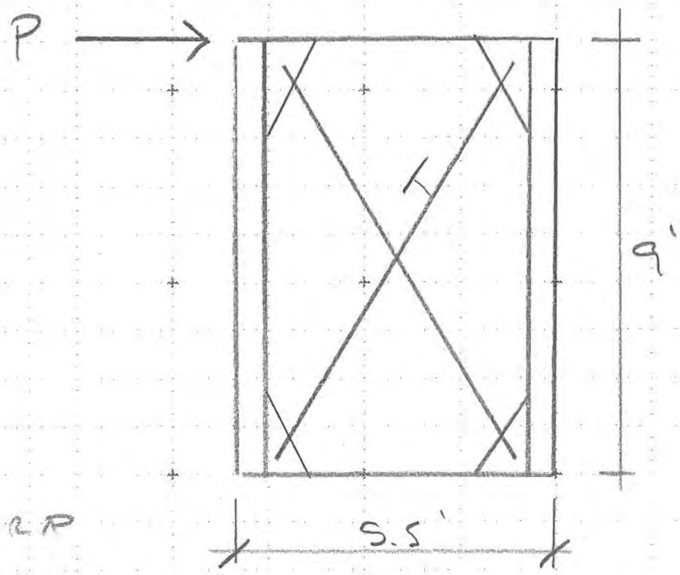
GL (2) V = 10.24/4 = 2.56
USE SSW 24x9

GL (3) V = 10.24/2 = 5.12^k
v = 5.12/11' = 465 #/'
USE 10-3 w/ HDU5

GL (4) V = 10.24/4 = 2.56^k
USE (2) FLAT STRAP BRACED FRMS
OR USE 2 SSW 24x9

# FLAT STRAP FRM




$P = 1500^{\#}$ @ ASD

$T = 1500 \times 10.5/5.5$
$= 2863^{\#}$

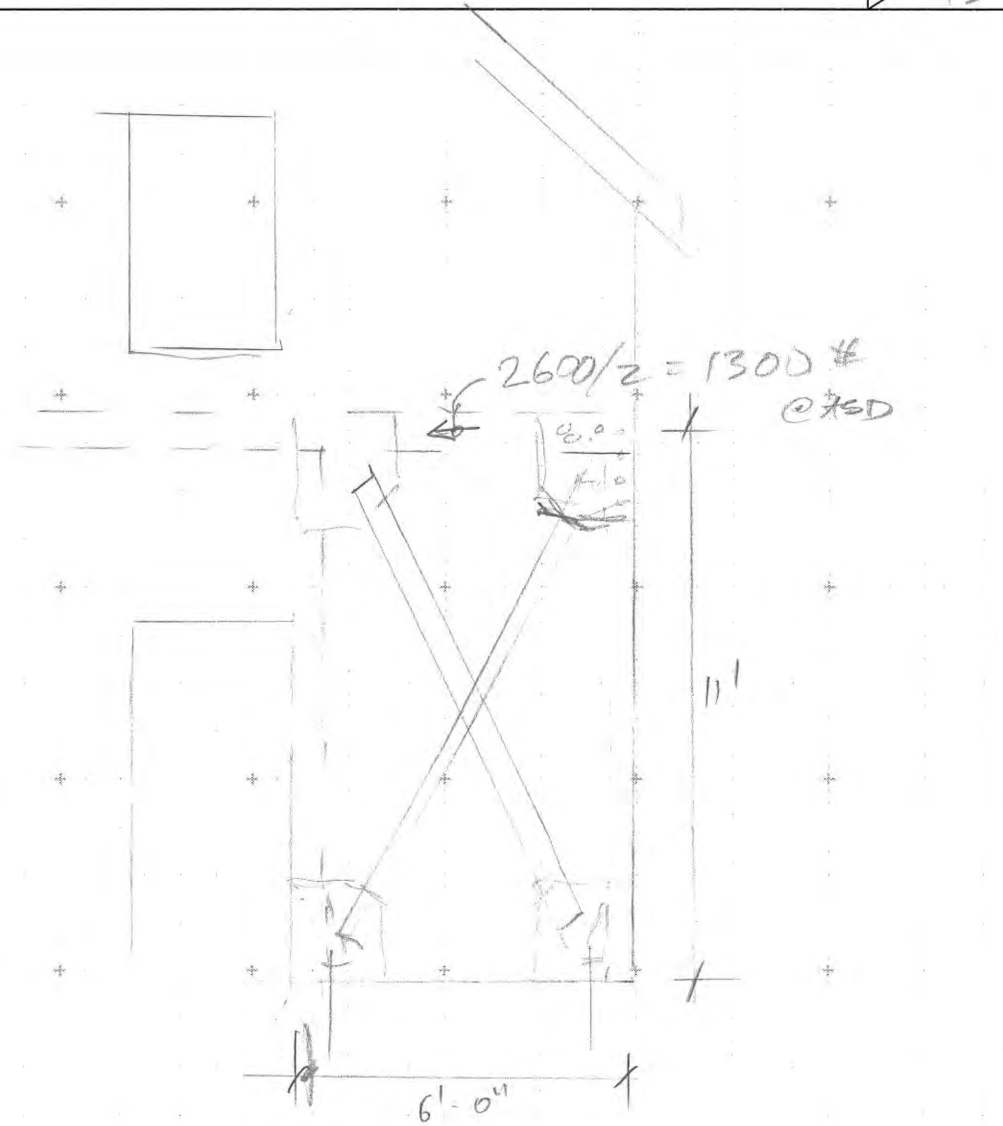
$A_{BRACE} = \dfrac{2863}{30{,}000 \times 0.6}$
$= 0.16\ in^2$

- USE 14GA STRAP

WIDTH $= 0.16/0.75$
$= 2.13$

**MAKE STRAP 14 GA × 3" WIDE**

## DESIGN WELD




DIAG. LENGTH $= (6^2 + 11^2)^{1/2} = 12.53'$

$T_{STRAP} = 1300^{\#} \times \dfrac{12.53}{6} = 2{,}714^{\#}$

SAY SS, $F_y = 30$ ksi

$A \geq \dfrac{2.714^k}{30 \times 0.60} \geq 0.151^{\square''} \Rightarrow 3'' \times 0.05'' \Rightarrow 3'' \times 16$ GA

t 0.0635

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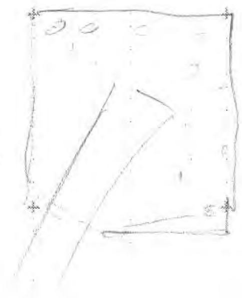
| Job No.: | By: | Date: |
| --- | --- | --- |
| Name : | Chk: | Date: |



\# SDS @ GUSSET



LOAD @ DIAG

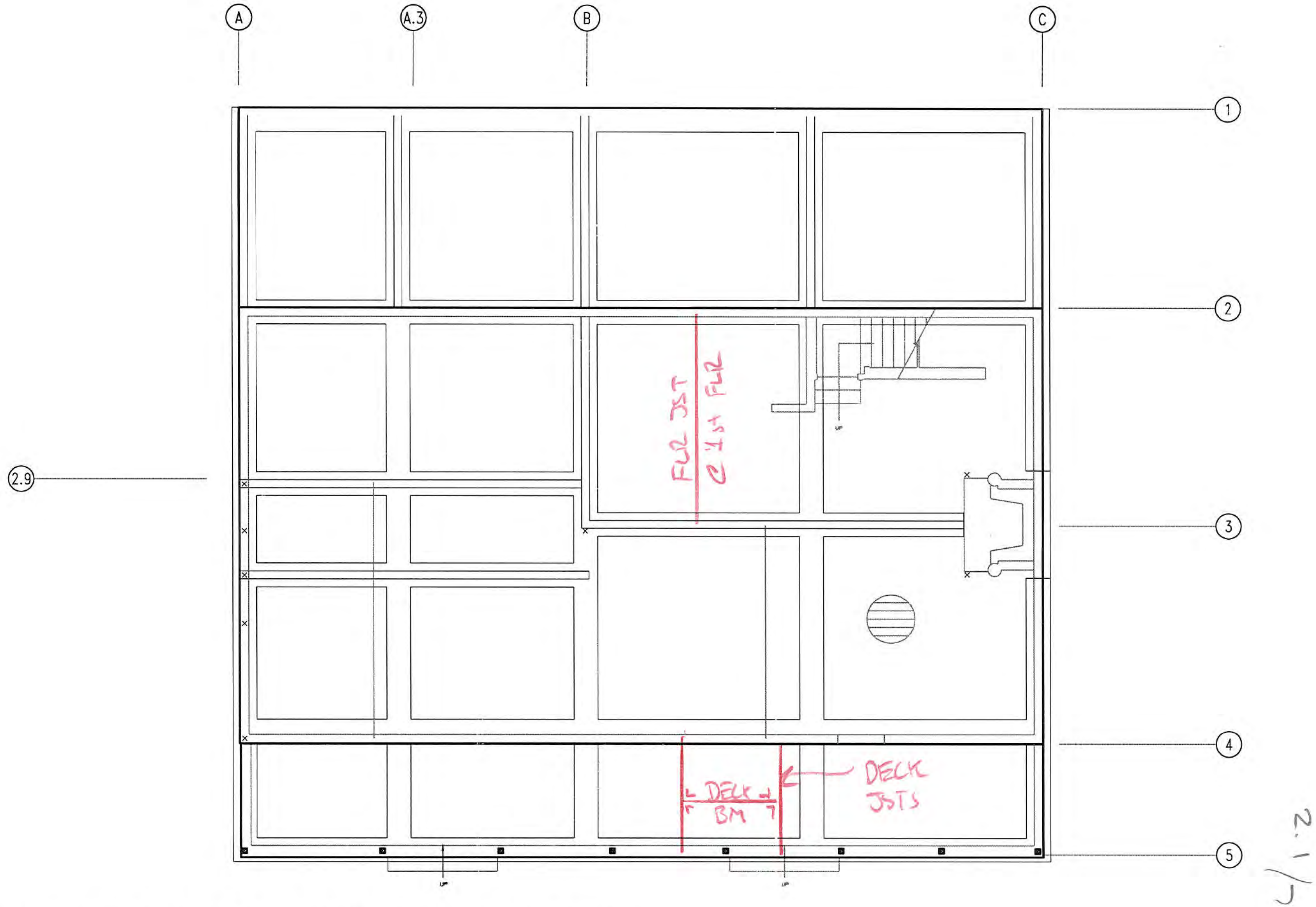
$= 2{,}714^{\#} \times \overset{\Omega_0}{2.0}$

$= 5{,}428^{\#}$

$V_{HORIZ} = 5{,}428^{\#} \times \frac{6}{12.53} = 2{,}600^{\#} / 250 \times 1.33 = 8$ SDS

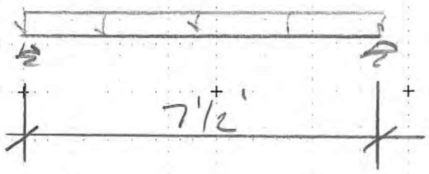
$V_{VERT} = 5428^{\#} \times \frac{11}{12.53} = 4{,}765^{\#} / 250 \times 1.33 = 14$ SDS

VERTICAL CALCULATION


A
A.3
B
C
1
2
2.9
3
4
5
FLR JST
@ 1st FLR
DECK BM
DECK JSTS
2.1/7

BEAMS FND

DECK BM




$w_D = 10\text{psf}(8') = 80^{\#/\prime}$

$w_L = 40\text{psf}(8') = 320^{\#/\prime}$

USE 4x8 @ 8'-0" OC

SEE COMP PRINTOUT

DECK JSTS


$w_D = 8\text{psf} \times 16/12 = 11^{\#/\prime}$

$w_L = 100\text{psf} \times 16/12 = 133^{\#/\prime}$

USE 2x8 DF #1 @16

SEE COMP PRINTOUT

2.3/7

Title :
Dsgnr:
Description :

Scope :

Job #
Date: 9:22AM, 11 AUG 09

Rev: 580006
User: KW-0606676, Ver 5.8.0, 1-Dec-2003
(c)1983-2003 ENERCALC Engineering Software

# Timber Beam & Joist



**Description** FRONT DECK

## Timber Member Information

Code Ref: 1997/2001 NDS, 2000/2003 IBC, 2003 NFPA 5000. Base allowables are user defined

| | | DECK BM | DECK JSTS | |
|---|---|---|---|---|
| **Timber Section** | | 4x8 | 2x8 | |
| Beam Width | in | 3.500 | 1.500 | |
| Beam Depth | in | 7.250 | 7.250 | |
| Le: Unbraced Length | ft | 0.00 | 0.00 | |
| TImber Grade | | Douglas Fir - Larch, No.1 | Douglas Fir - Larch, No.1 | Douglas Fir - Larch, No.1 |
| Fb - Basic Allow | psi | 1,000.0 | 1,000.0 | |
| Fv - Basic Allow | psi | 180.0 | 180.0 | |
| Elastic Modulus | ksi | 1,700.0 | 1,700.0 | |
| Load Duration Factor | | 1.000 | 1.000 | |
| Member Type | | Sawn | Sawn | |
| Repetitive Status | | No | Repetitive | |

## Center Span Data

| | | | | |
|---|---|---|---|---|
| Span | ft | 7.50 | 8.00 | |
| Dead Load | #/ft | 80.00 | 11.00 | |
| Live Load | #/ft | 320.00 | 133.00 | |

## Cantilever Span

| | | | | |
|---|---|---|---|---|
| Span | ft | | | 1.50 |
| Uniform Dead Load | #/ft | | | 16.00 |
| Uniform Live Load | #/ft | | | |
| Point #1 DL | lbs | | | |
| LL | lbs | | | 150.00 |
| @ X | ft | | | 1.500 |

## Results

| Ratio = | | 0.8467 | 0.7623 | |
|---|---|---|---|---|
| Mmax @ Center | in-k | 33.75 | 13.82 | |
| @ X = | ft | 3.75 | 4.00 | |
| Mmax @ Cantilever | in-k | 0.00 | 0.00 | |
| fb : Actual | psi | 1,100.7 | 1,052.0 | |
| Fb : Allowable | psi | 1,300.0 | 1,380.0 | |
| | | **Bending OK** | **Bending OK** | |
| fv : Actual | psi | 74.5 | 68.0 | |
| Fv : Allowable | psi | 180.0 | 180.0 | |
| | | **Shear OK** | **Shear OK** | |

## Reactions

| | | | | |
|---|---|---|---|---|
| @ Left End DL | lbs | 300.00 | 44.00 | |
| LL | lbs | 1,200.00 | 532.00 | |
| Max. DL+LL | lbs | 1,500.00 | 576.00 | |
| @ Right End DL | lbs | 300.00 | 44.00 | |
| LL | lbs | 1,200.00 | 532.00 | |
| Max. DL+LL | lbs | 1,500.00 | 576.00 | |

## Deflections

| | | Deflection OK | Deflection OK | |
|---|---|---|---|---|
| Center DL Defl | in | -0.030 | -0.013 | |
| L/Defl Ratio | | 2,986.0 | 7,668.7 | |
| Center LL Defl | in | -0.121 | -0.151 | |
| L/Defl Ratio | | 746.5 | 634.3 | |
| Center Total Defl | in | -0.151 | -0.164 | |
| Location | ft | 3.750 | 4.000 | |
| L/Defl Ratio | | 597.2 | 585.8 | |
| Cantilever DL Defl | in | | | 0.000 |
| Cantilever LL Defl | in | | | 0.000 |
| Total Cant. Defl | in | | | 0.000 |
| L/Defl Ratio | | | | 0.0 |

FULCRUM STRUCTURAL ENGINEERING

| Job No.: | 08-37 | By: | JB | Date: | 12/16/08 | 24 |
|---|---|---|---|---|---|---|
| Name : | NAVARRO IN | Chk: | | Date: | | 7 |

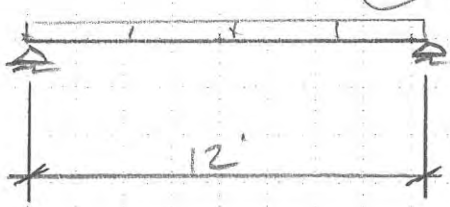
## FLR JSTS

### FLR JOIST @ 1st FLR

- ASSUME LL = 100 psf FOR ASSEMBLY LOADING & MATCH (E) 3½ x 7¼ DF FLR JST

$w_D = 14\text{psf} \times 20''/12'' = 23\ \#/'$ (SPCG)

$w_L = 100\text{psf}\ 20''/12'' = 167\ \#/'$




USE 3½ x 7¼ DF #1 FLR JST @ 20" OC

SEE COMP PRINTOUT

2.5/7

| | |
|---|---|
| **Title :** | **Job #** |
| **Dsgnr:** | **Date:** 5:14PM, 16 DEC 08 |
| **Description :** | |
| **Scope :** | |



# Timber Beam & Joist

**Description** Floor Joists

## Timber Member Information

Code Ref: 1997/2001 NDS, 2000/2003 IBC, 2003 NFPA 5000. Base allowables are user defined

| | | Floor Joist |
|---|---|---|
| **Timber Section** | | 4x8 |
| Beam Width | in | 3.500 |
| Beam Depth | in | 7.250 |
| Le: Unbraced Length | ft | 0.00 |
| TImber Grade | | Douglas Fir - Larch, No.1 |
| Fb - Basic Allow | psi | 1,000.0 |
| Fv - Basic Allow | psi | 180.0 |
| Elastic Modulus | ksi | 1,700.0 |
| Load Duration Factor | | 1.000 |
| Member Type | | Sawn |
| Repetitive Status | | Repetitive |

## Center Span Data

| | | |
|---|---|---|
| Span | ft | 12.00 |
| Dead Load | #/ft | 23.00 |
| Live Load | #/ft | 167.00 |

## Results

| | | |
|---|---|---|
| Ratio = | | 0.8953 |
| Mmax @ Center | in-k | 41.04 |
| @ X = | ft | 6.00 |
| fb : Actual | psi | 1,338.5 |
| Fb : Allowable | psi | 1,495.0 |
| | | **Bending OK** |
| fv : Actual | psi | 60.9 |
| Fv : Allowable | psi | 180.0 |
| | | **Shear OK** |

## Reactions

| | | | |
|---|---|---|---|
| @ Left End | DL | lbs | 138.00 |
| | LL | lbs | 1,002.00 |
| | Max. DL+LL | lbs | 1,140.00 |
| @ Right End | DL | lbs | 138.00 |
| | LL | lbs | 1,002.00 |
| | Max. DL+LL | lbs | 1,140.00 |

## Deflections

Deflection OK

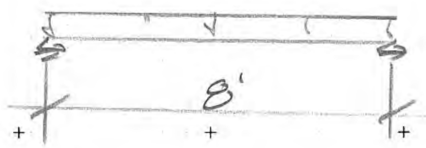
| | | |
|---|---|---|
| Center DL Defl | in | -0.057 |
| L/Defl Ratio | | 2,535.6 |
| Center LL Defl | in | -0.412 |
| L/Defl Ratio | | 349.2 |
| Center Total Defl | in | -0.469 |
| Location | ft | 6.000 |
| L/Defl Ratio | | 306.9 |

# BEAMS @ PORCH

## PORCH HDR

$w_D = 12psf(1.5 + 6.5/2) = 57^{\#/'}$

$w_L = 20psf(1.5 + 6.5/2) = 95^{\#/'}$



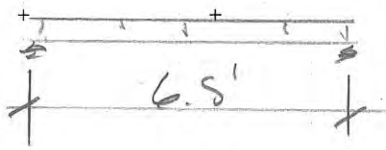

USE FLAT 4x6 DF, SS

SEE COMP PRINTOUT

## PORCH RAFTERS

$w_D = 12psf(16/12) = 16^{\#/'}$

$w_L = 20psf(16/12) = 27^{\#/'}$




USE 2x4 DF #1 @ 16"

SEE COMP PRINTOUT

27/7

Title :
Dsgnr:
Description :

Scope :

Job #
Date: 4:34PM, 5 AUG 09



# Timber Beam & Joist



**Description** FRONT DECK

## Timber Member Information

Code Ref: 1997/2001 NDS, 2000/2003 IBC, 2003 NFPA 5000. Base allowables are user defined

| | | DECK BM | PORCH HDR | RAFTERS |
|---|---|---|---|---|
| Timber Section | | 4x8 | 4x4 | 2x4 |
| Beam Width | in | 3.500 | 5.500 | 1.500 |
| Beam Depth | in | 7.250 | 3.500 | 2.750 |
| Le: Unbraced Length | ft | 0.00 | 0.00 | 0.00 |
| Timber Grade | | Douglas Fir - Larch, No.1 | Douglas Fir - Larch, Select | Douglas Fir - Larch, No.1 |
| Fb - Basic Allow | psi | 1,000.0 | 1,500.0 | 1,000.0 |
| Fv - Basic Allow | psi | 180.0 | 180.0 | 180.0 |
| Elastic Modulus | ksi | 1,700.0 | 1,900.0 | 1,700.0 |
| Load Duration Factor | | 1.000 | 1.250 | 1.250 |
| Member Type | | Sawn | Sawn | Sawn |
| Repetitive Status | | No | No | Repetitive |

## Center Span Data

| | | DECK BM | PORCH HDR | RAFTERS |
|---|---|---|---|---|
| Span | ft | 7.50 | 8.00 | 6.50 |
| Dead Load | #/ft | 80.00 | 57.00 | 16.00 |
| Live Load | #/ft | 320.00 | 95.00 | 27.00 |

## Cantilever Span

| | | DECK BM | PORCH HDR | RAFTERS |
|---|---|---|---|---|
| Span | ft | | | 1.50 |
| Uniform Dead Load | #/ft | | | 16.00 |
| Uniform Live Load | #/ft | | | |
| Point #1 DL | lbs | | | |
| LL | lbs | | | 150.00 |
| @ X | ft | | | 1.500 |

## Results

| Ratio = | | 0.8467 | 0.6931 | 0.7153 |
|---|---|---|---|---|
| Mmax @ Center | in-k | 33.75 | 14.59 | 2.62 |
| @ X = | ft | 3.75 | 4.00 | 3.20 |
| Mmax @ Cantilever | in-k | 0.00 | 0.00 | -2.92 |
| fb : Actual | psi | 1,100.7 | 1,299.5 | 1,542.3 |
| Fb : Allowable | psi | 1,300.0 | 1,875.0 | 2,156.3 |
| | | Bending OK | Bending OK | Bending OK |
| fv : Actual | psi | 74.5 | 44.0 | 62.1 |
| Fv : Allowable | psi | 180.0 | 225.0 | 225.0 |
| | | Shear OK | Shear OK | Shear OK |

## Reactions

| | | DECK BM | PORCH HDR | RAFTERS |
|---|---|---|---|---|
| @ Left End DL | lbs | 300.00 | 228.00 | 49.23 |
| LL | lbs | 1,200.00 | 380.00 | 87.75 |
| Max. DL+LL | lbs | 1,500.00 | 608.00 | 136.98 |
| @ Right End DL | lbs | 300.00 | 228.00 | 78.77 |
| LL | lbs | 1,200.00 | 380.00 | 272.37 |
| Max. DL+LL | lbs | 1,500.00 | 608.00 | 351.13 |

## Deflections

| | | Deflection OK | Deflection OK | Ratio > 240 ! |
|---|---|---|---|---|
| Center DL Defl | in | -0.030 | -0.141 | -0.127 |
| L/Defl Ratio | | 2,986.0 | 682.3 | 614.7 |
| Center LL Defl | in | -0.121 | -0.234 | -0.245 |
| L/Defl Ratio | | 746.5 | 409.4 | 317.9 |
| Center Total Defl | in | -0.151 | -0.375 | -0.372 |
| Location | ft | 3.750 | 4.000 | 3.224 |
| L/Defl Ratio | | 597.2 | 255.9 | 209.6 |
| Cantilever DL Defl | in | | | 0.080 |
| Cantilever LL Defl | in | | | -0.171 |
| Total Cant. Defl | in | | | -0.091 |
| L/Defl Ratio | | | | 397.7 |

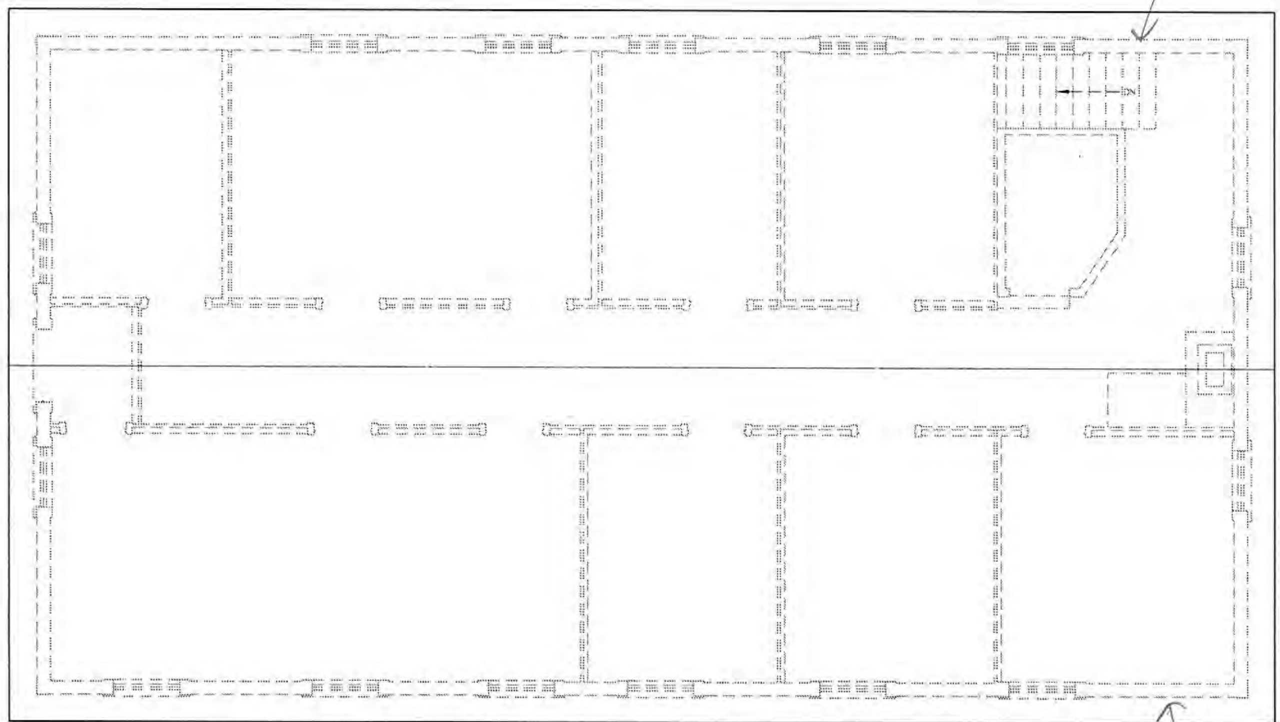
LOADING ON GRID LINES &

RETANING WALL CALC.

ROOF DL

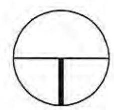
$W_{DL} = 17\,psf\,(15') = 108$

$W_{LL} = 20\,psf\,(15') = 300\,plf.$



$W_{DL} = 108\,plf$

$W_{LL} = 300\,plf.$




A ROOF FRAMING PLAN

SCALE: 1/4" = 1'-0"

3.1/17

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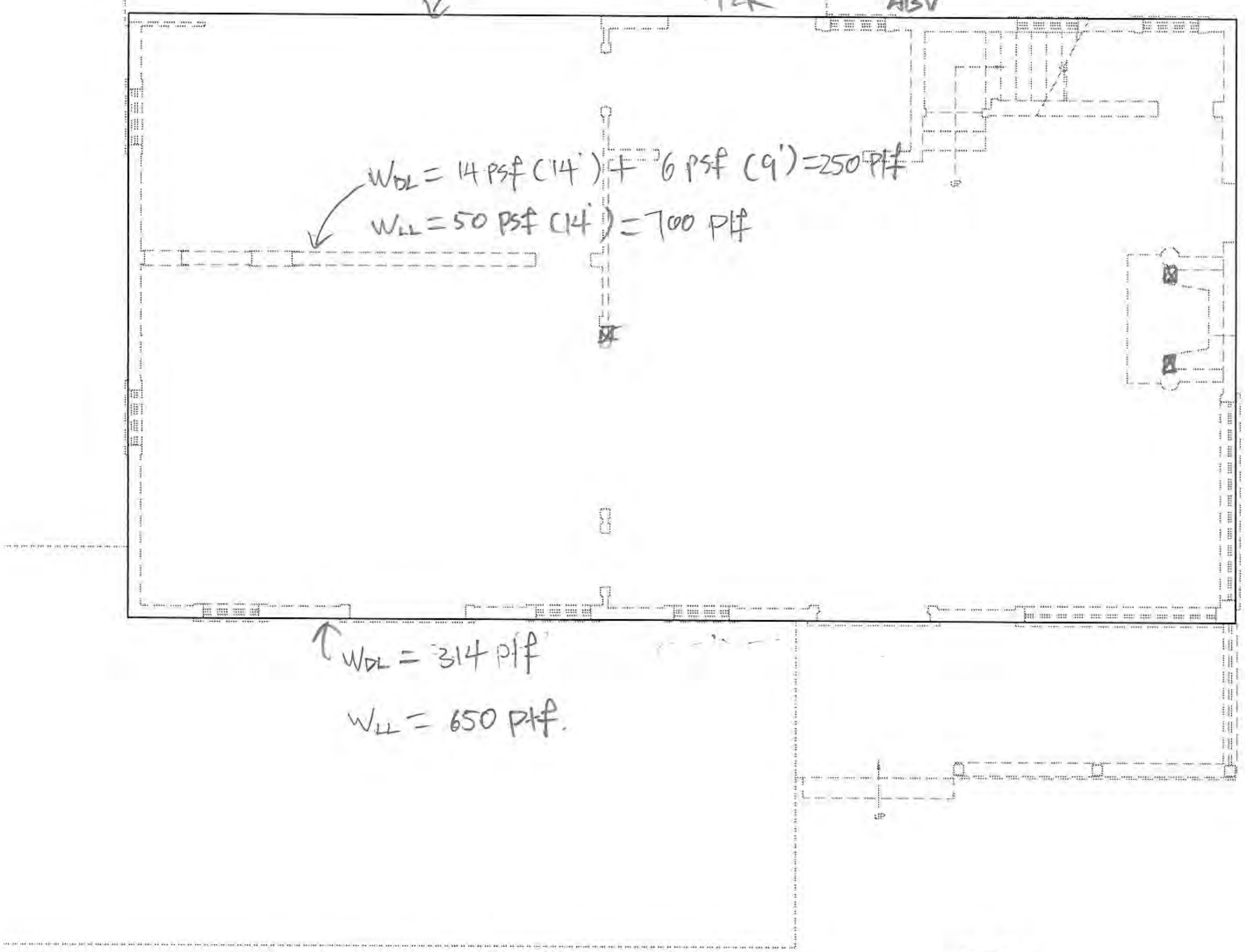
$W_{DL} = (14)\ psf\,(7') + 12\ psf\,(9') + 108\ plf = 314\ plf$

FLR, WALL, ABV

$W_{LL} = 50\ psf\,(7') + 300\ plf = 650\ plf$

FLR, ABV



$W_{DL} = 14\ psf\,(14') + 6\ psf\,(9') = 250\ plf$

$W_{LL} = 50\ psf\,(14') = 700\ plf$

$W_{DL} = 314\ plf$

$W_{LL} = 650\ plf.$

## A SECOND FLOOR FRAMING PLAN

SCALE: 1/4" = 1'-0"

PROJECT & REAL NORTH

3.2/17

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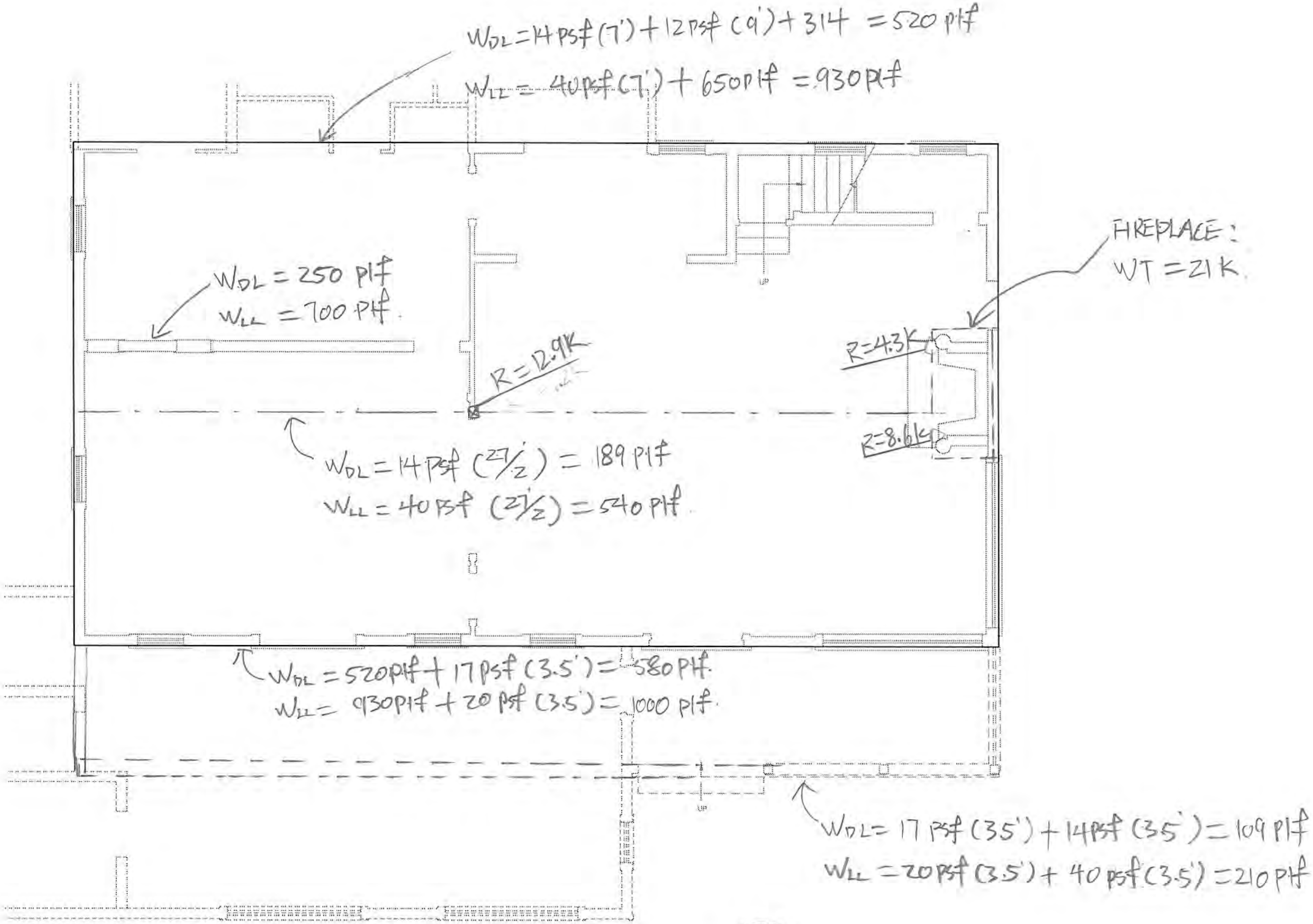


A FOUNDATION/ FIRST FLOOR FRAMING PLAN

SCALE: 1/4" = 1'-0"

PROJECT & REAL NORTH

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FOUNDATION WT ON GL's.

GL A.

WT = 150 pcf (8"/12)(4')(43') = 17.2k

GL A4, B, B2, B6, C ⇒ SEE GL A

GL 5.

WT = 150 pcf (8"/12)(5.5')(51') = 28.1 K

GL4

WT = 150 pcf (12"/12)(5')(51') = 38.3 K.

GL 3.

WT = 150 pcf (12"/12)(4')(24') = 14.4 K.

GL 2.9.

WT = 150 pcf (12"/12)(4')(20') = 12 K

GL 2

WT = 150 pcf (18"/12)(2.5')(51') = 28.6$^{k}$.

GL 1

WT = 150 pcf (18/12)(3')(51') + 150 pcf (10"/12)(6')(51')
= 34$^{k}$ + 38$^{k}$ = 72$^{k}$

TOTAL FND WT = 17.2k×5 + 28.1k + 38.3k + 14.4k + 12k + 28.6k + 72k + 44$^{k}$ + 13.5 = 337

CONC SLAB : WT = 150 pcf (7/12)(50'×10') = 44$^{K}$

FIREPLACE : WT = 150 pcf (4'×7.5'×3') = 13.5k

TOTAL WEIGHT ON GL'S. (SEE PREVIOUS PAGE FOR LOADING INFO)

GL A & C

$WT = \underset{\text{(SEE PREVIOUS PAGE)}}{17.2k} + \underset{\text{WALL DL}}{12\,psf}\,(20')(34') = 23k$

GL A4, B2, B6

$WT = 17.2k$

GL 1 $\quad WT = 72^k$

GL 2 $\quad WT = 28.6^k + \overbrace{(520 + 930)}^{\text{(DL+LL, SEE KEY PLAN.)}}\,plf\,(50') = 101.1\,k$

GL 2.9 $\quad WT = 12k + \underbrace{(250 + 700)}_{DL+LL}(20') = 31\,k$

GL 3 $\quad WT = 14.4k + \overbrace{(189 + 540)}\,plf\,(27') = 34\,k$

GL 4. $\quad WT = 38.3k + \underbrace{(580 + 1000)}_{DL+LL}\,plf\,(50') = 117.3k$

GL 5. $\quad WT = 28.1k + \overbrace{(109 + 210)}\,plf\,(50') = 44.1\,k$

FIREPLACE $= 13.5k + 21k = 34.5k$

TOTAL WEIGHT $= 23 \times 2 + 17.2 \times 3 + 72 + 101^k + 31 + 34 + 117.3 + 44.1 + 34.5 + 44^k = 575k$

FRICTIONAL FORCE $= 575k \times 0.4 = 230k$

FROM SOIL REPORT.

W/O SLAB, GRID 1 & ALL GRADE BEAM BTW GL 1 & 2,

TOTAL WEIGHT =

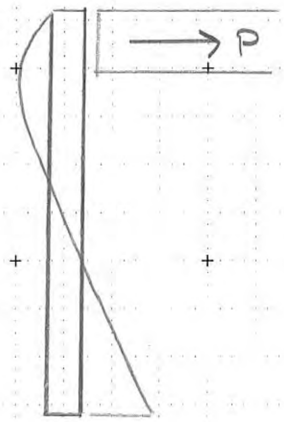
$= 575^k - \underset{\text{GL 1 WALL}}{72\,k} - \underset{\text{CONC SLAB}}{44^k} - 150\,pcf\,(8''/12)(4')(8')(\overset{\#}{5}) = 443^k$

FRICTIONAL FORCE $= 443\,k \times 0.4 = 177^k$

# PIER DESIGN

\# OF PIERS

$P = 177^k / 7 = 25^k$




- ASSUME FORCES INDUCED BY LATERAL SPREADING & EQUIVALENT FLUID WT FROM RETAINING WALL DO NOT ACT SIMULTAINIOUSLY

DETERMINE MOMENT ON PIER DUE TO LATERAL SPREADING

$M = 455^{k\text{-}ft}$ ← SEE EXCEL SPDSHT

SIZE REINF

USE (18) #10 BARS

SEE COMP PRINTOUT

## Unconstrained Pier:

### INPUT:

| | | | |
| --- | --- | --- | --- |
| Pier Dia = | 24 | in | Dia |
| Pier Dia Factor = | 2 | | Dia_Fac |
| Dist of P From Ground Surface, y = | 4.00 | ft | y |
| Depth Below Surface to Neglect, Dneg = | 2 | ft | Dneg |
| P, unfactored = | 25.0 | k | P |
| S1,initial = | 0 | psf | S1_init |
| passive, pass = | 300 | pcf | pass |
| Over Turning Stability Factor = | 1 | | OT_Fac |

### REQUIRED DEPTH OF PIER:

| | | | | | |
| --- | --- | --- | --- | --- | --- |
| b = | Dia*Dia_Fac/12 | = | 4 | ft | b_ |
| h = | y+Dneg | = | 6.0 | ft | h_ |
| S1 = | S1_init+(dp_var/3)*pass | = | 1460 | psf | S1_ |
| A = | (2.34*P*OT_Fac*1000)/(S1_*b_) | = | 10.02 | | A_ |
| d, variable = | | = | 14.6 | ft | dp_var |
| d, calc'd = | (A_/2)*(1+(1+(4.36*h_)/A_)^0.5) | = | 14.5 | ft | dp_calc |

**Embedment From Surface =**

| | | | | |
| --- | --- | --- | --- | --- |
| | dp_calc+Dneg | = | **16.5** | ft |

### UTIMATE MOMENT IN PIER:

| | | | | | |
| --- | --- | --- | --- | --- | --- |
| S1 = | S1_init+(dm_var/3)*pass | = | 1390 | psf | S1_m |
| A = | (2.34*P*1000)/(S1_m*b_) | = | 10.52 | | A_m |
| d, variable = | | = | 13.9 | ft | dm_var |
| d, calc'd = | (A_m/2)*(1+(1+(4.36*y)/A_m)^0.5) | = | 13.8 | ft | dm_calc |
| Mn, @ 0.34*d = | | | | | |
| | P*(h_+dm_calc*0.34) | = | 267.6 | kip-ft | Mn |
| **Mu =** | 1.7*Mn | = | **454.9** | kip-ft | Mu |

3.9/17

Title :
Dsgnr:
Description :

Scope :

Job #
Date: 6:12PM, 28 JUL 09

Rev: 580002
User: KW-0606676, Ver 5.8.0, 1-Dec-2003


# Circular Concrete Column



**Description** PIER AT RETAINING WALL

## General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

| | | | | | |
|---|---|---|---|---|---|
| Diameter | 24.000 in | f'c | 2,500.0psi | Total Height | 15.000 ft |
| Number of Bars | 18 | Fy | 60,000.0 psi | Unbraced Length | 0.000 ft |
| Bar Size | 10 | Seismic Zone | 4 | Eff. Length Factor | 1.000 |
| Total Rebar Area | 22.860 in2 | LL & ST Loads Act Separate | | Column is BRACED | |
| Rebar Percent | 5.053 % | Spiral Ties Used | | | |
| Bar Cover | 2.500 in | | | | |

## Loads

Note: Load factoring supports 2003 IBC and 2003 NFPA 5000 by virtue of their references to ACI 318-02 for concrete design. Factoring of entered loads to ultimate loads within this program is according to ACI 318-02 C.2

| | Dead Load | Live Load | Short Term | Eccentricity |
|---|---|---|---|---|
| **Axial Loads** | 1.000 k | 1.000 k | k | in |
| **Applied Moments...** | | | | |
| @ Top | k-ft | 455.000 k-ft | k-ft | |
| @ Bottom | k-ft | k-ft | k-ft | |

## *Summary*

**Column is OK**

Column Diameter= 24.00in, with 18 #10 Bars

| | ACI C-1 | ACI C-2 | ACI C-3 |
|---|---|---|---|
| **Applied Pu : Max Factored** | **3.10** k | **1.40** k | **0.90** k |
| **Allowable Pn * Phi @ Design Ecc.** | **3.22** k | **1,336.11** k | **1,336.11** k |
| M-critical | 773.50 k-ft | 0.15 k-ft | 0.10 k-ft |
| Combined Eccentricity | ###.#### in | 1.3200 in | 1.3200 in |
| Magnification Factor | 1.00 | 1.00 | 1.00 |
| Design Eccentricity | ###.#### in | 1.3200 in | 1.3200 in |
| Magnified Design Moment | 773.50 k-ft | 0.15 k-ft | 0.10 k-ft |
| Po * 0.85 | 1,941.70 k | 1,941.70 k | 1,941.70 k |
| P : Balanced | 481.52 k | 481.52 k | 481.52 k |
| Ecc : Balanced | 18.8202 in | 18.8202 in | 18.8202 in |

## Slenderness *per ACI 318-02 Section 10.12 & 10.13*

Actual k Lu / r 0.000 Elastic Modulus 2,850.0 ksi Beta 0.850

| | ACI Eq. C-1 | ACI Eq. C-2 | ACI Eq. C-3 |
|---|---|---|---|
| Neutral Axis Distance | 9.4920 in | 27.2820 in | 27.2820 in |
| Phi | 0.8956 | 0.7000 | 0.7000 |
| Max Limit kl/r | 34.0000 | 34.0000 | 34.0000 |
| Beta = M:sustained/M:max | 0.4516 | 1.0000 | 1.0000 |
| Cm | 1.0000 | 1.0000 | 1.0000 |
| EI / 1000 | 0.00 | 0.00 | 0.00 |
| Pc : pi^2 E I / (k Lu)^2 | 0.00 | 0.00 | 0.00 |
| alpha: MaxPu / (.75 Pc) | 0.0000 | 0.0000 | 0.0000 |
| Delta | 1.0000 | 1.0000 | 1.0000 |
| Ecc: Ecc Loads + Moments | ###.#### | 1.3200 | 1.3200 in |
| Design Ecc = Ecc * Delta | **0.0000** | **0.0000** | **0.0000** in |

## ACI Factors (per ACI 318-02, applied internally to entered loads)

| | | | | | |
|---|---|---|---|---|---|
| ACI C-1 & C-2 DL | 1.400 | ACI C-2 Group Factor | 0.750 | Add"l "1.4" Factor for Seismic | 1.400 |
| ACI C-1 & C-2 LL | 1.700 | ACI C-3 Dead Load Factor | 0.900 | Add"l "0.9" Factor for Seismic | 0.900 |
| ACI C-1 & C-2 ST | 1.700 | ACI C-3 Short Term Factor | 1.300 | | |
| ....seismic = ST * : | 1.100 | | | | |

## Spiral Tie Requirements *per 97 UBC 1910.9.3*

| | | | |
|---|---|---|---|
| Spiral Tie Bar Size # | **3** | Min. Spiral Reinforcement Ratio | **0.011** |
| Gross Area of Column | **452.39** in2 | | |
| Core Area Within Spirals | **283.53** in2 | Max Spiral Tie Spacing | **2.074** in |

| Job No.: 08-37 | By: JB | Date: | 3.10 |
|---|---|---|---|
| Name : NAVARRO INN | Chk: | Date: | 17 |

FULCRUM STRUCTURAL ENGINEERING

# RETAINING WALL

DESIGN RETAINING WALL STEM ASSUMING AT LEAST 2:1 SLOPE INCLINATION

EQUIVALENT FLUID WT = 55pcf ← TABLE 1 OF SOILS REPORT

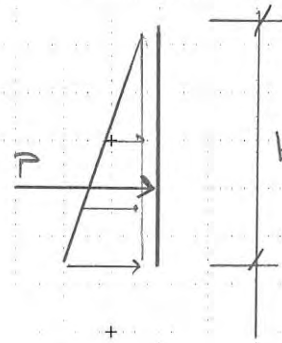
H = 7' MAX

$P = 55pcf\,(7^2/2) = 1350^{\#}$

$= 1350 \times 1.6 = 2156^{\#}$

$M = 1350^{\#} \times 7/3 = 3150^{\#-1}$

$= 3150 \times 1.6 = 5040^{\#-1}$




SAY 10" STEM

$A_s = \dfrac{5.04}{4 \times 7} = 0.18\ in^2$

USE # 4 @ 12" OC

| FULCRUM STRUCTURAL ENGINEERING | Job No.: 08-37 | By: JB | Date: 7/28/09 | 3.11 |
|---|---|---|---|---|
| | Name: NAVARRO INN | Chk: | Date: | 17 |

## RETAINING WALL

### DESIGN GB w/ TORSION

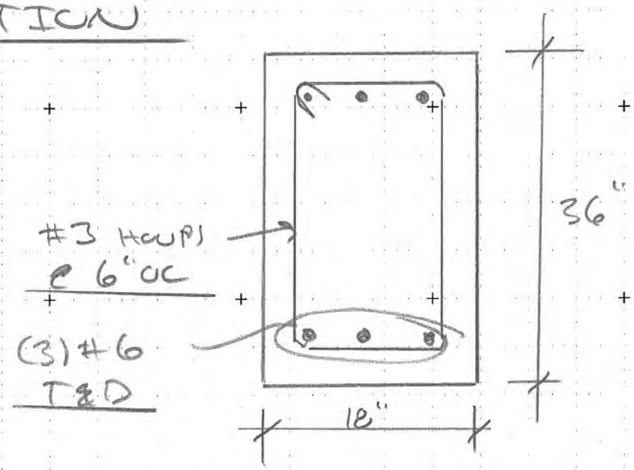
LARGEST SPAN

TORSION (UNFACTORED) $= 3150^{\#\text{-}1} \times 13'/2 = 20.5^{k\text{-}1}$

$20.5^{k\text{-}1} \times 1.6 = 32.8^{k\text{-}1}$

$V_u = 150\text{pcf}\,(10/12)(7')(10') = 8.8^{k}$

DRAW SECTION




SEE EXCEL SPRSHT

3.12/17

# CONCRETE TORSION

LOCATION: TORSION BEAM AT RETAINING WALL

**Basic Input**

| | | | |
|---|---|---|---|
| b = | 18 | in | b |
| Side Cover = | 2 | in | Cov_side |
| h = | 36 | in | h |
| Top Cover = | 1.5 | in | Cov_top |
| Bot cover = | 3 | in | Cov_bot |
| Slab to Right Thk = | 0 | in | Slab_thk_r |
| Slab to Left Thk = | 0 | in | Slab_thk_l |
| Mu = | | ft-kips | Mu |
| Vu = | 8.8 | kips | Vu |
| Equil Tu = | 32.8 | ft-kips | Tu_equil |
| Theta = | 45 | deg | Theta |
| Phi, torsion = | 0.85 | | Phi_t |
| Phi, shear = | 0.85 | | Phi_v |
| Phi, bend = | 0.9 | | Phi_b |
| f'c = | 3,000 | psi | fc |
| fy = | 60,000 | psi | fy |
| fyv = | 40,000 | psi | fyv |

**"d"**

| | | | | |
|---|---|---|---|---|
| Trial longit bar size = | 6 | long_bar_trial | | |
| Trial Stirrup bar size = | 3 | stir_trial | | |
| d from top = | h-Cov_bot-stir_trial/8-long_bar_trial/8 = | | 31.875 in | d_top |
| d from bot = | h-Cov_top-stir_trial/8-long_bar_trial/8 = | | 33.375 in | d_bot |
| min d = | MIN(d_top,d_bot) = | | 31.875 in | d_min |

**Neglect Torsion?**

| | | | |
|---|---|---|---|
| Slab Overhang Projection to Right = | 0 | in | Flng_r |
| Slab Overhang Projection to Left = | 0 | | Flng_l |

| | | | | |
|---|---|---|---|---|
| Acp = | b*h = | b*h+Slab_thk_r*Flng_r+Slab_thk_l*Flng_l = | 648 in^2 | Acp |
| Pcp = | 2*(b+h) = | 2*(b+h) +2*Flng_r+2*Flng_l = | 108 in | Pcp |
| Threshold Torsion = | | Phi_t*(fc^0.5)*((Acp^2)/Pcp)*(1/(1000*12)) = | 15.08 ft-kips | Threshld_T |

thus: **Consider Torsion**
IF(Tu_equil<Threshld_T,"Neglect Torsion","Consider Torsion")

**Compatibility Torsion?** n ("y" or "n") Compatibility

| | | | |
|---|---|---|---|
| Compatibility Torsion = | 4 * Threshld_T = | 60.3 ft-kips | Tu_comp_calc |
| Design Torsion = | IF(Compatibility="y", MIN(Tu_equil,Tu_comp_calc),Tu_equil) = | **32.8** ft-kips | Tu |

**Is the Section Big Enough to Resist the Torsion?**

| | | | |
|---|---|---|---|
| Aoh = | (b-2*Cov_side-stir_trial/8)*(h-Cov_top-Cov_bot-stir_trial/8) = | 424 in^2 | Aoh |
| ph = | 2*((b-2*Cov_side-stir_trial/8) + (h-Cov_top-Cov_bot-stir_trial/8)) = | 90 in | ph |

| | | | | |
|---|---|---|---|---|
| ACI Eqn 11-18: | Shear & Torsion Interaction: | Vu*1000/(b*d_min) = | 15 psi | Vu_strs |
| | | (Tu*12*1000*ph)/(1.7*Aoh^2) = | 115 psi | Tu_strs |
| | | (Vu_strs^2+Tu_strs^2)^0.5 = | 116 psi | Vu_Tu_Int |
| | Stress Lim = | Phi_t*(2*fc^0.5 + 8*fc^0.5) = | 466 psi | Strs_Lim |

thus: **O.K - Section Large Enough.**
IF(Vu_Tu_Int<=Strs_Lim,"O.K - Section Large Enough.","N.G. - Increase Section")

3.13/17

## Compute the Stirrup Area Required for Shear

| | | | |
|---|---|---|---|
| Vc = | 2*fc^0.5*b*d_min/1000 = | 62.9 kips | Vc |
| Vs = | IF((Vu/Phi_v) >=Vc,Vu/Phi_v-Vc,0) | 0.0 kips | Vs |
| Av/s = | Vs*1000/(fyv*d_min) = | 0.000 | Av_per_s |

## Compute the Stirrup Area Required for Torsion

| | | | |
|---|---|---|---|
| Tn = | Tu*12*1000/Phi_t = | 463,059 in-lb | Tn |
| Ao = | 0.85*Aoh = | 360 in^2 | Ao |
| Cot (Theta) = | 1/TAN(Theta*PI()/180) = | 1.00 | Cot_Theta |
| At/s = | Tn*(Cot_Theta)/(2*Ao*fyv) = | **0.016** | At_per_s |

## Add the Stirrup Areas and Select Sirrups

| | | | |
|---|---|---|---|
| Av+t / s = | Av_per_s+2*At_per_s = | 0.0321 | Avt_per_s |
| Min Stirrups Av+t / s = | 50*b/fyv = | 0.0225 | min_Avt_per_s |
| thus, Av+t / s for design = | MAX(Avt_per_s,min_Avt_per_s) = | 0.0321 | design_Avt_per_s |

Stirrups: Bar # = 3 Stir_Bar
No Legs = 2 No_Legs Av+t = No_Legs*((Stir_Bar/8)^2*PI()/4) = 0.22 in^2 Avt

s <= Avt/design_Avt_per_s = 6.9 in s_reqd

min s: 12 in
ph/8 = 11.1875 in ph_ovr_8

s max = MIN(s_reqd,12,ph_ovr_8) = 6.9 in s_max

select s = 6 in s **OK**

thus: **No.3 Closed Stirrups @ 6 in oc**

## Design Longitudinal Reinforcement for Torsion

| | | | |
|---|---|---|---|
| Al = | At_per_s*ph*(fyv/fy)*Cot_Theta^2 = | 0.96 in^2 | Al_calc |
| Al, min = | 5*fc^0.5*Acp/fy - (((Stir_Bar/8)^2*PI()/4)/s)*ph*(fyv/fy) = | 1.86 in^2 | Al_min |
| thus, Al = | MAX(Al_calc,Al_min) = | **1.86** in^2 | Al |
| Area of steel around perimeter = | Al/(ph) | 0.021 in^2/in | Al_perim |

Max spacing of Longitudinal Bars at Sides = 12 in Max_Long_Spcg

Min Longit Bar diameter:

| | | | |
|---|---|---|---|
| Min Area of Longit Bar | Max_Long_Spcg*Al_perim = | 0.249 in^2 | Al_area |
| Min Longit Bar Dia = | ((4*Al_area)/PI())^0.5 = | 0.56 in | Al_Long_Bar_Dia_Calcd |
| s/24 = | | 0.250 in | Min_Long_Bar_Dia_Calc |

Min Bar Dia = MAX(0.375,Al_Long_Bar_Dia_Calcd,Min_Long_Bar_Dia_Calc) = **0.56** in

Select Size of Side Bars = # 5 Bar Side_Bar_Dia
Selected Side Bar Diameter = Side_Bar_Dia/8 = 0.625
O.K.

Area of Steel to Add to Top and Bot Reinf = Al_perim*(b-2*Cov_side + 12) = 0.54 in^2 Add_Torsion_Reinf

Provide min (1) bar at corners
Number of Top and Bot Reinf Used:
( **3** ) # **6** = 1.33 in^2 Bend_Reinf

Effective Bending Reinf = Bend_Reinf - Add_Torsion_Reinf = 0.785 in^2 Eff_Bend_Reinf

Effective Moment Capacity , Phi*Mn = Eff_Bend_Reinf * 4 * d_min = **100.1** ft kips

## GRADE BEAMS

DESIGN N/S SPANNING GB @ REAR

$DL = 150\,psf\,(7/12)(14') + 150\,pcf\,(12''/12'')(3')$
$= 1.7^{k/l} \times 1.2 = 2^{k/l}$

$LL = 40\,psf\,(14') = 560^{\#/l} \times 1.6 = 0.9^{k/l}$

- ASSUME TORQUE DUE TO RETAINING WALL & TORSION DM IS TRANSFERED ENTIRELY INTO GB

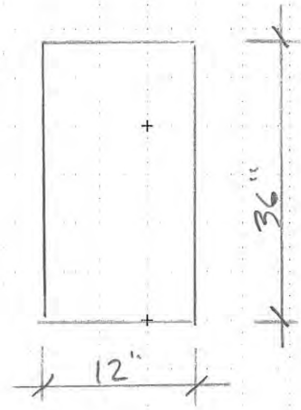
$M = 32.8^{k\text{-}l}$ (FACTORED)

$M_{MAX} = 61.8^{k\text{-}l}$ ← SEE COMP PRINTOUT
$V_{MAX} = 18.9^{k}$ ← " " "

SIZE REINF

$A_s = \dfrac{61.8}{4 \times 30} = 0.515\,\square''$

USE (2) #6 BARS, T&B




CHK SHEAR REINF

$\phi V_n/2 = 0.75 \times \sqrt{2500} \times 12 \times 30$
$= 13.5^{k} < 18.9^{k}$ ∴ SHEAR REQD

USE #3 CLOSED HOOPS @ 12" OC

3.15/17

Title :
Dsgnr:
Description :

Scope :

Job #
Date: 6:49PM, 28 JUL 09



# Single Span Beam Analysis

**Description** N/S GRADE BEAM AT REAR OF BLDG

## General Information

| | | | |
|---|---|---|---|
| Center Span | 11.00 ft | Moment of Inertia | 1,000.000 in4 |
| Left Cantilever | ft | Elastic Modulus | 29,000 ksi |
| Right Cantilever | ft | Beam End Fixity | Pin-Pin |

## Uniform Loads

| On Center Span... | | On Left Cantilever... | | On Right Cantilever... | |
|---|---|---|---|---|---|
| # 1 | 2.000 k/ft | # 1 | k/ft | # 1 | k/ft |
| # 2 | 0.900 k/ft | # 2 | k/ft | # 2 | k/ft |

## Moments

| | | | | |
|---|---|---|---|---|
| Magnitude | 32.80 k-ft | k-ft | k-ft | k-ft |
| Location | 11.000 ft | ft | ft | ft |

## Query Values

| | | | | | |
|---|---|---|---|---|---|
| **Center Location** | 0.000 ft | **Left Cant** | 0.000 ft | **Right Cant** | 0.000 ft |
| Moment | 0.00 k-ft | | 0.00 k-ft | | 0.00 k-ft |
| Shear | 18.93 k | | 0.00 k | | 0.00 k |
| Deflection | 0.00000 in | | 0.00000 in | | 0.00000 in |

## *Summary*

**Moments...**

| | | | |
|---|---|---|---|
| Max + @ Center | 61.80 k-ft | at | 6.53 ft |
| Max - @ Center | 0.00 k-ft | at | 0.00 ft |
| @ Left Cant | 0.00 k-ft | | |
| @ Right Cant | 0.00 k-ft | | |
| **Maximum =** | **61.80** k-ft | | |

**Shears...**

| | |
|---|---|
| @ Left | 18.93 k |
| @ Right | 12.97 k |
| Maximum | 18.93 k |

**Reactions...**

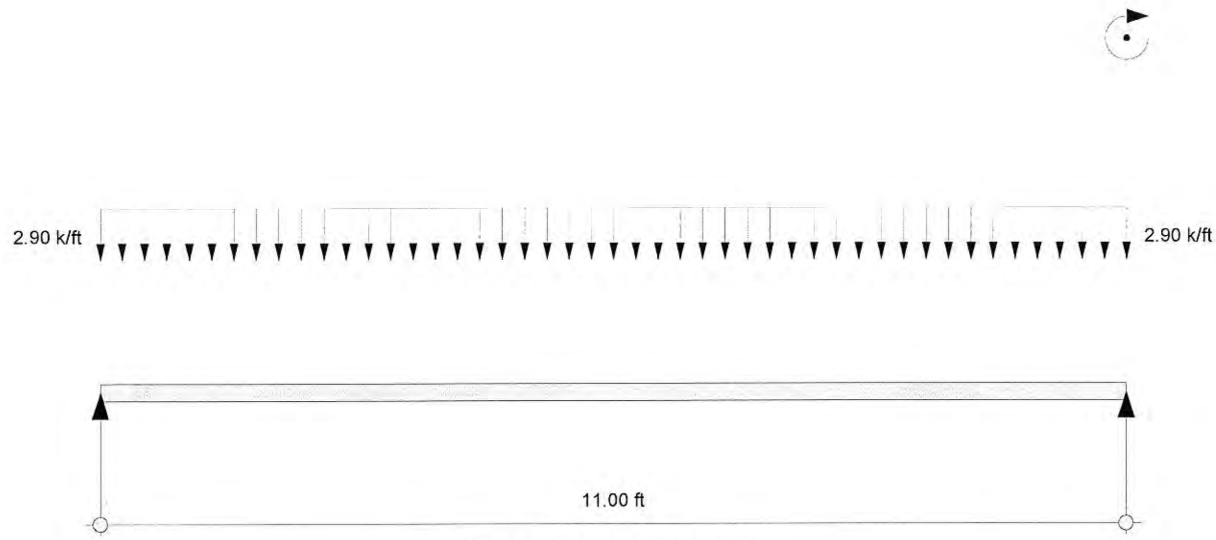
| | |
|---|---|
| @ Left | 18.93 k |
| @ Right | 12.97 k |

**Deflections...**

| | | | |
|---|---|---|---|
| @ Center | -0.048 in | at | 5.75 ft |
| @ Left Cant. | 0.000 in | at | 0.00 ft |
| @ Right Cant | 0.000 in | at | 0.00 ft |

3.16/17

32.80 k-ft




Mmax = 61.79 k-ft at 6.52 ft from left

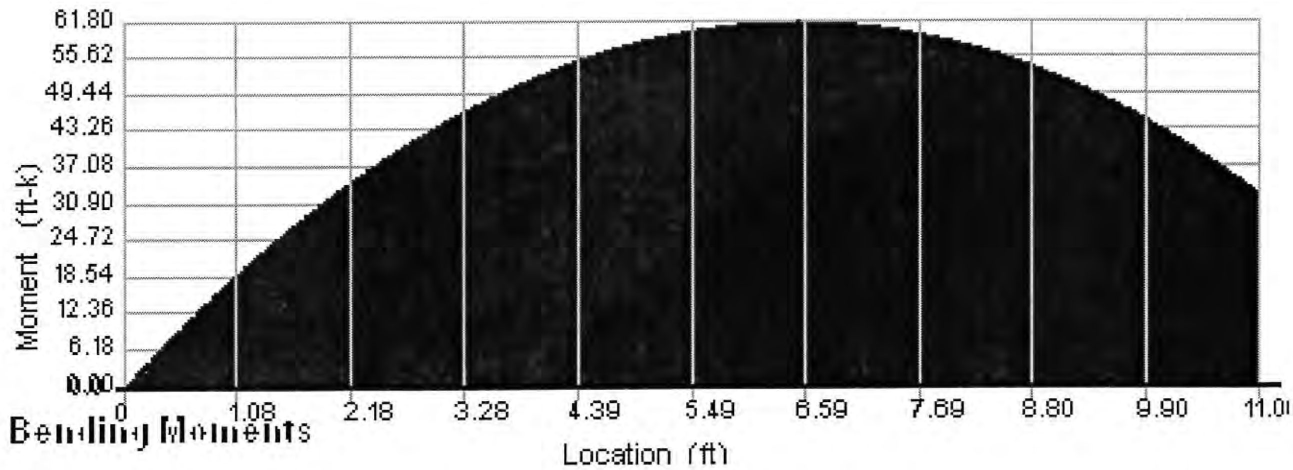
Dmax = -0.0478 in at 5.75 ft from left

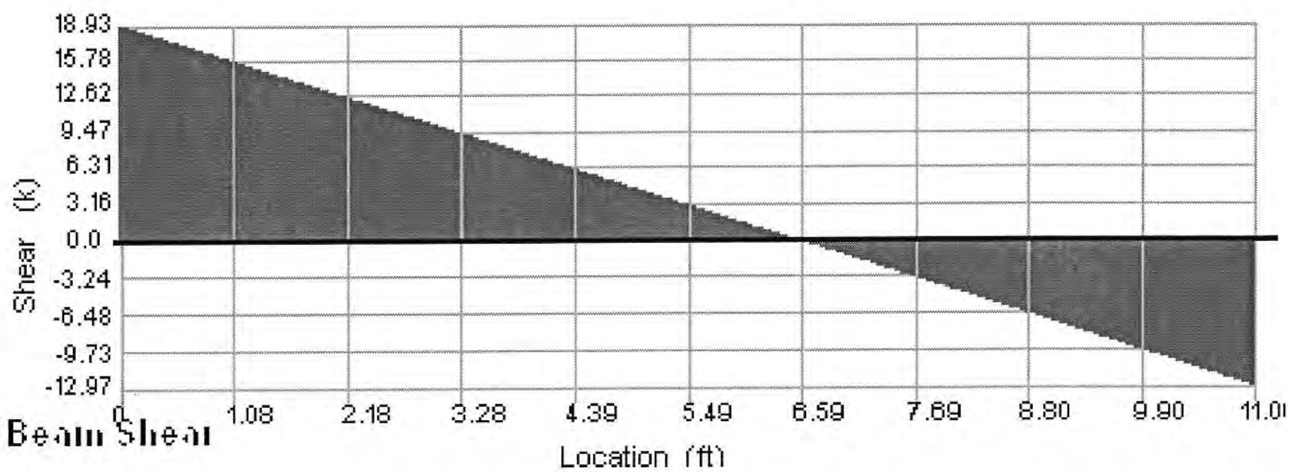
Rl = 18.931 k

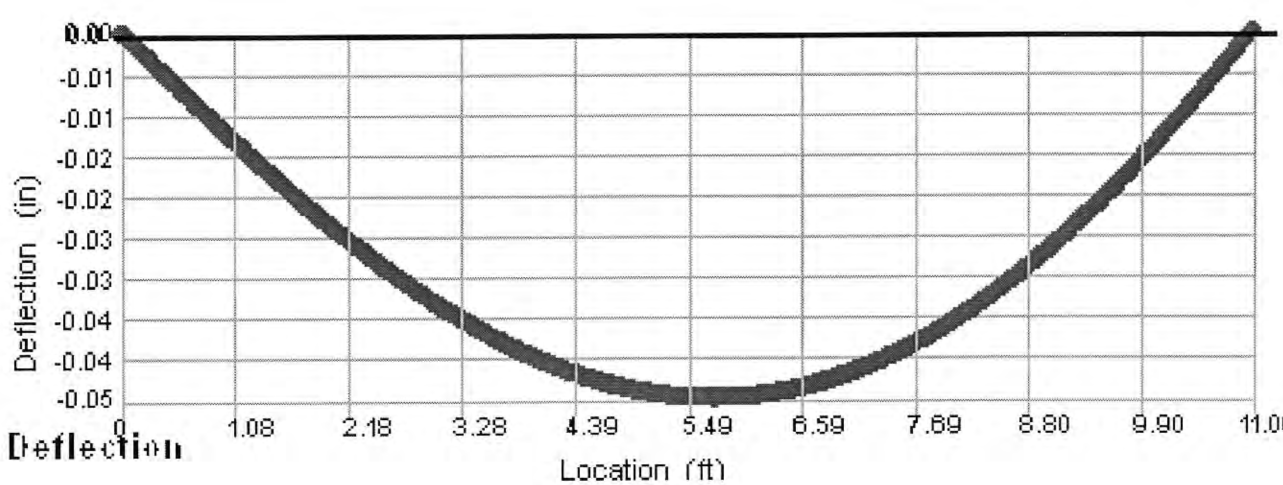
Vmax @ left = 18.931 k

Rr = 12.968 k

Vmax @ rt = 12.968 k


61.80
55.62
49.44
43.26
37.08
30.90
24.72
18.54
12.36
6.18
0.00
Moment (ft-k)
0
1.08
2.18
3.28
4.39
5.49
6.59
7.69
8.80
9.90
11.0
Bending Moments
Location (ft)


18.93
15.78
12.62
9.47
6.31
3.16
0.0
-3.24
-6.48
-9.73
-12.97
Shear (k)
0
1.08
2.18
3.28
4.39
5.49
6.59
7.69
8.80
9.90
11.0
Beam Shear
Location (ft)


0.00
-0.01
-0.01
-0.02
-0.02
-0.03
-0.03
-0.04
-0.04
-0.05
Deflection (in)
0
1.08
2.18
3.28
4.39
5.49
6.59
7.69
8.80
9.90
11.0
Deflection
Location (ft)