State of Utah

GARY R. HERBERT
Governor

SPENCER J. COX
Lt. Governor

Department of Administrative Services
KIMBERLY K. HOOD
Executive Director

Division of Facilities Construction and Management
ERIC R. THOLEN
Director

Addendum No. 1

Date:	May 10, 2016
To:	A/E Firms
From:	Matt Boyer – Project Manager, DFCM
Reference:	BDO Campus Bay 2 Buildout-Ogden/Weber ATC Utah College of Applied Technology DFCM Project No. 17026240
Subject:	**<u>Addendum No. 1</u>**

Pages			
	Addendum Cover Sheet	2	pages
	Structural Evaluation	74	pages
	Total	76	pages

Note: This Addendum shall be included as part of the Contract Documents. Items in this Addendum apply to all drawings and specification sections whether referenced or not involving the portion of the work added, deleted, modified, or otherwise addressed in the Addendum. Acknowledge receipt of this Addendum in the space provided on the Bid Form. Failure to do so may subject the Bidder to Disqualification.

1.1 SCHEDULE: There are no project schedule changes.

1.2 GENERAL ITEMS:

1.2.1 QUESTION: In the management plan and statement of qualification section of our proposal, "Tab 2 and Tab 3" both require an organizational chart. I assume you only need one. Which tab would you like to see this under?
ANSWER: Please provide only one copy of the organizational chart in the management plan. List the corresponding responsibility of the management plan for that team member.

1.2.2 As part of the build out of the warehouse space, OWATC is interested in upgrading the structure framework for possible future photovoltaic array over Bay #2. This would be a structural upgrade to the west 1/3rd of the existing Tiber

frame structure to increase the loading to accommodate a rack mounted solar P.V. system. This desire of the college is to incorporate the necessary structural improvements during this phase of construction to facilitate a solar P. V. system that would generate enough power to offset at least ½ to 2/3 of the power necessary for operation of the BDO Facility. On other PV projects, PV arrays this size have increased the roof load by seven pounds per square foot. It is anticipated that the future PV system would need to generate around 440kW to meet the 2/3 power offset goal of the building operations.
The intent would be to complete design for such a structural improvement now as part of the design agreement, and then show these structural upgrades as a bid alternate to the construction documents.

The college has consulted with a Structural Engineer, who is of the opinion that a simple steel structure, from the exterior wall to midpoint of the west bay is where the structure would need to be installed to provide additional structural support to allow the installation of a Solar P.V. array on the West Bay of Bay #2.

Please see the attached structural evaluation that was done during the reroof & seismic upgrade project.

ARW ENGINEERS
structural consultants

Structural Calculations

For

OWATC BDO Bldg 10A
Roof Upgrades

Project Number: 15118

July 22, 2015

PROFESSIONAL STRUCTURAL ENGINEER
Zachary C. Hansen
No. 7749124-2203
7/22/15
STATE OF UTAH

DIVISION OF FACILITIES CONSTRUCTION AND MANAGEMENT
REVIEWED FOR CODE COMPLIANCE
SIGNATURE:
DATE: 11/16/2015

Prepared by
ARW Engineers
1594 West Park Circle
Ogden, Utah 84404

ARW ENGINEERS
structural consultants

STRUCTURAL CALCULATIONS

FOR

OWATC BDO Bldg 10A Roof Upgrades

Client: Bott Pantone Architects

Project Number: 15118

DESIGN CRITERIA

GOVERNING CODE: IBC 2012

GENERAL: Risk Category = II

SEISMIC: Seismic Upgrade Per ASCE 41-06 Basic Safety Objective

DESIGN LOADS

ROOFS: DL = 15 psf SL = 30 psf

A R W ENGINEERS
structural consultants

CALCULATIONS INDEX

DESIGN CRITERIA

ROOF DEAD LOAD

SINGLE PLY MEMBRANE	2.0 psf
RIGID INSULATION (4")	1.0 psf
15/32" OSB OVERLAY	2.0 psf
1x12 DIAG. SHEATHING	2.5 psf
2x12 @ 24" o.c.	2.2 psf
SUSPENDED CEILING	2.0 psf
MISC	1.5 psf

TL = 13.2 psf SAY 15 psf

ROOF SNOW LOAD

$P_g = 43$ psf

$I_s = 1.0$

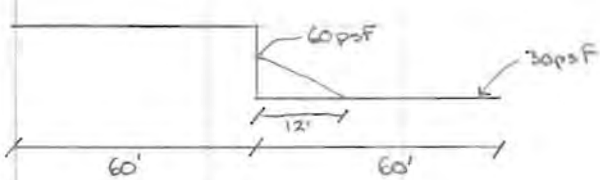
$C_e = 1.0$

$C_t = 1.0$

$P_f = 0.7(1.0)(1.0)(1.0)(43\text{psf}) = 30$ psf

$P_f = 30$ psf

DRIFT @ CLERESTORY: (SEE SPREADSHEET CALCS)



SEISMIC CRITERIA

EVALUATE/UPGRADE BUILDING TO BSE-2 @ COLLAPSE PREVENTION
\+
BSE-1 @ LIFE SAFETY PER ASCE41-06

BSE-2N ⇒ $S_{XS} = 1.422g$
(MCE) $S_{X1} = 0.744g$

BSE-1N ⇒ $S_{XS} = 0.948g$
(2/3 MCE) $S_{X1} = 0.496g$

DESIGN CRITERIA (CONT'D)

WALL DEAD LOAD (EXTERIOR WALL)

METAL SIDING	1.0 psf
1x12 SHEATHING	2.5 psf
3x6 STUDS @ 24" o.c.	1.8 psf
5/8" GYPSUM	2.8 psf
15/32" OSB	1.7 psf
MISC.	1.5 psf

} — FUTURE UPGRADES

TL = 11.3 psf SAY 12 psf **WALL DL = 12 psf**

FIRE WALL DEAD LOAD

3x8 STUDS	2.5 psf
(2) SIDES 1x12 SHEATHING	5.0 psf
(2) SIDE T+G 1x DECK	5.0 psf
3 LAYER 1/2" GYP. EA SIDE	13.2 psf

TL = 25.7 psf **FIRE WALL DL = 26 psf**

3

USGS Design Maps Summary Report

User–Specified Input

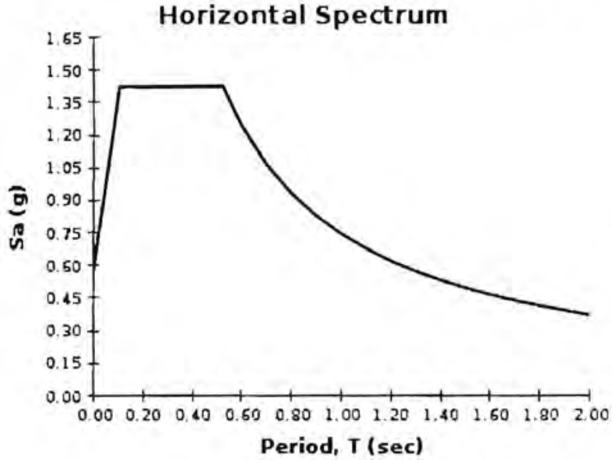
Building Code Reference Document ASCE 41-13 Retrofit Standard, BSE-2N
(which utilizes USGS hazard data available in 2008)

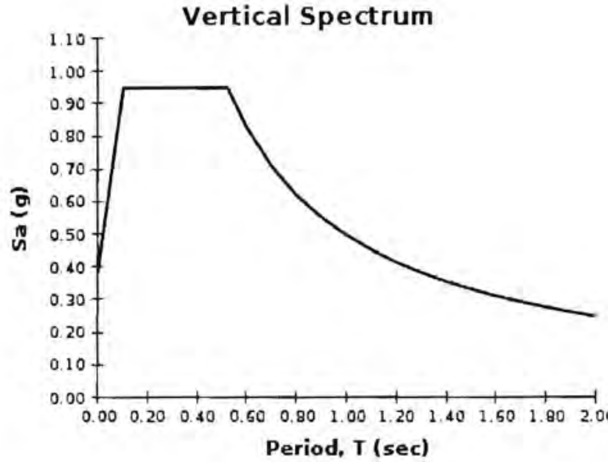
Site Coordinates 41.2588°N, 112.00037°W

Site Soil Classification Site Class D – "Stiff Soil"

USGS–Provided Output

$S_{S,BSE-2N}$	1.422 g	$S_{XS,BSE-2N}$	1.422 g
$S_{1,BSE-2N}$	0.495 g	$S_{X1,BSE-2N}$	0.744 g





Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

USGS Design Maps Summary Report

User–Specified Input

Building Code Reference Document ASCE 41-13 Retrofit Standard, BSE-1N
(which utilizes USGS hazard data available in 2008)

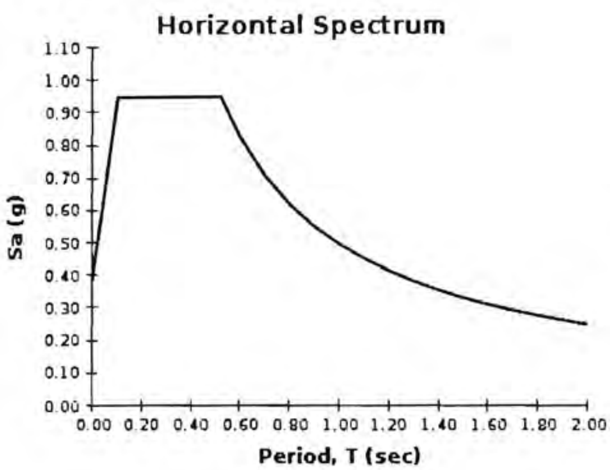
Site Coordinates 41.2588°N, 112.00037°W

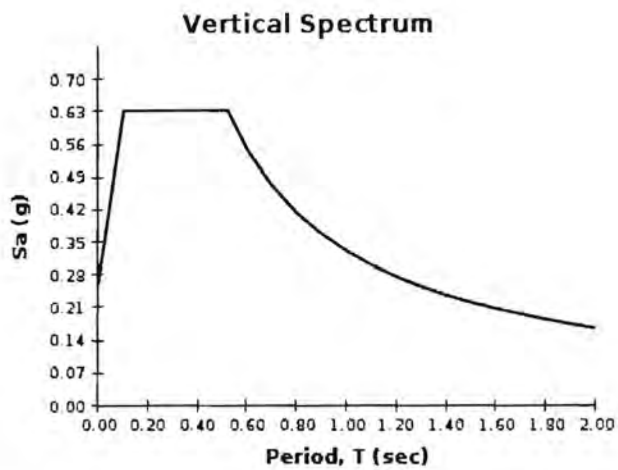
Site Soil Classification Site Class D – "Stiff Soil"

USGS–Provided Output

$S_{XS,BSE-1N}$ 0.948 g

$S_{X1,BSE-1N}$ 0.496 g

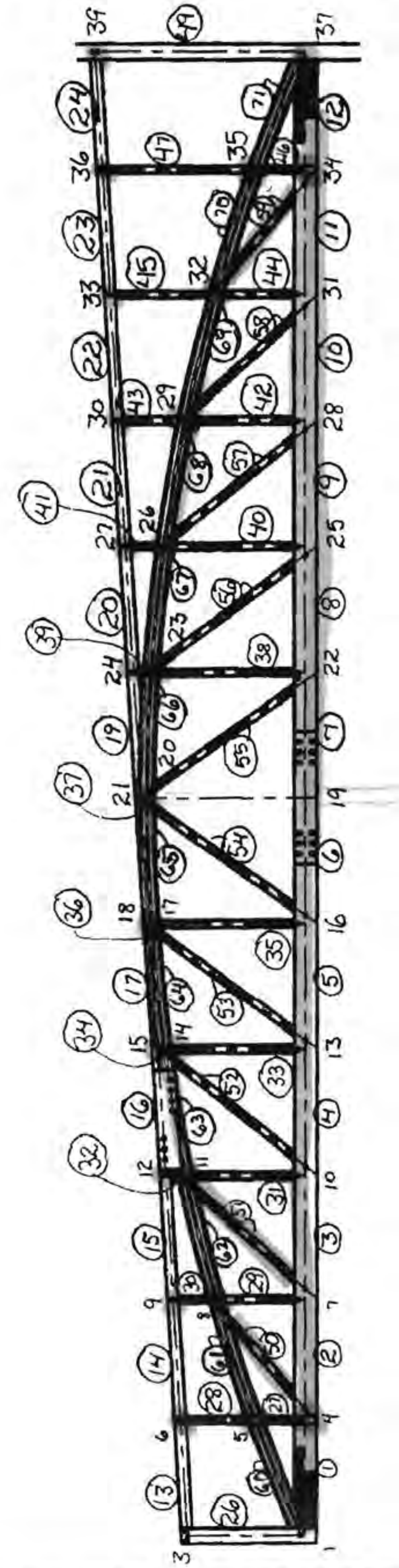




ROOF FRAMING

ARW Engineers
STRUCTURAL AND CIVIL

Project No. 43367 Sheet No. SD-4
6
Project
Prepared By Date 1-11-94
Client



BAD CONNECTIONS

OVERSTRESSED MEMBERS

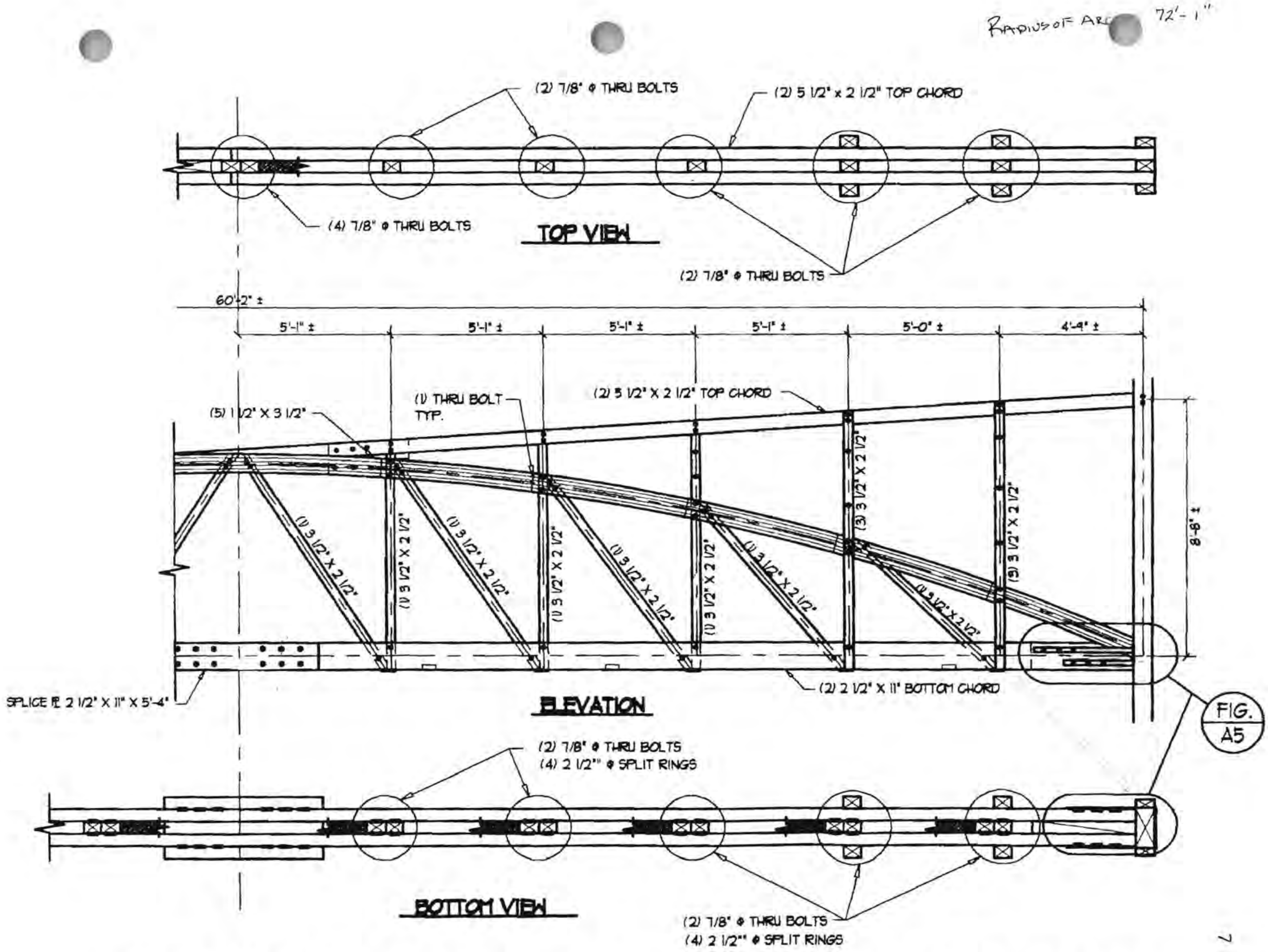


FIGURE A9 (CONTINUED) TRUSS GEOMETRY, MEMBER SIZES & CONNECTION CONFIGURATION FOR SIDE BAY TRUSS

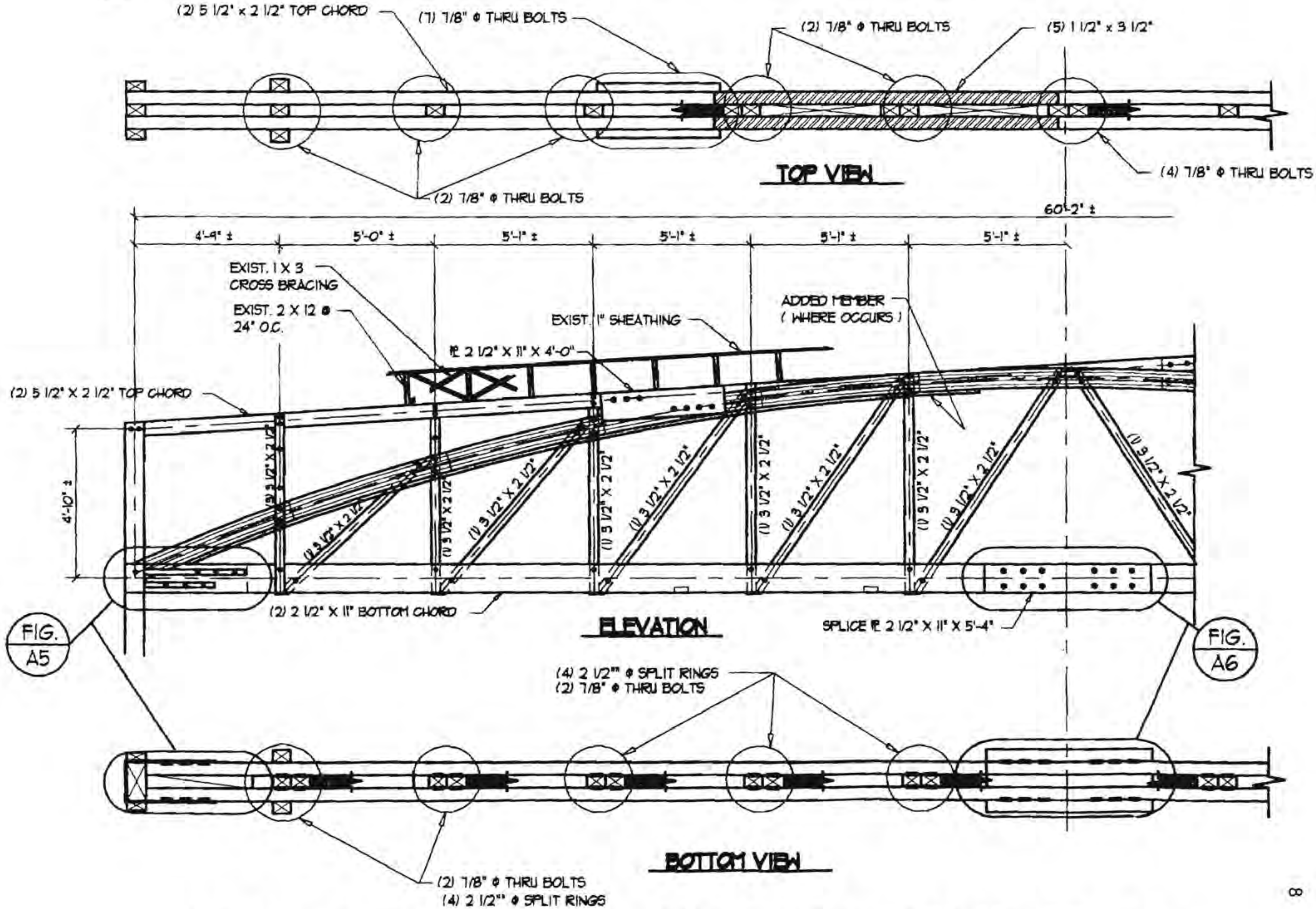
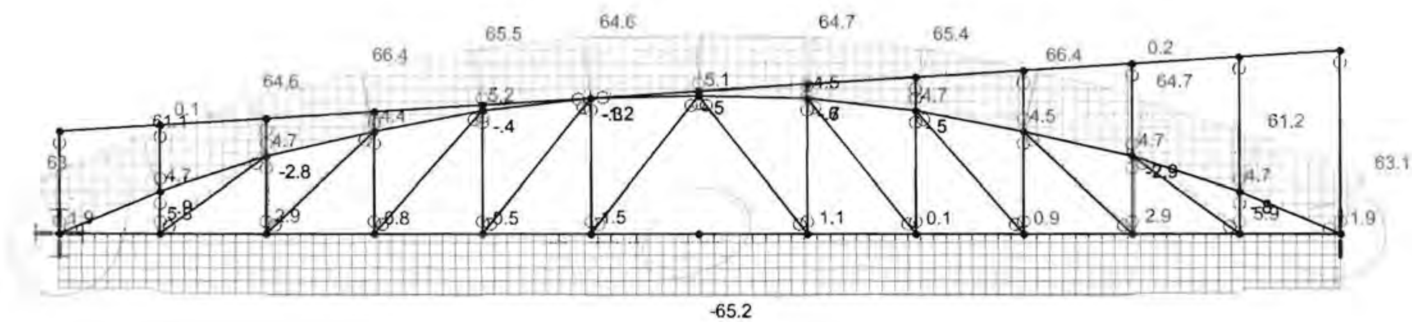


FIGURE A3 : TRUSS GEOMETRY, MEMBER SIZES & CONNECTION CONFIGURATION FOR SIDE BAY TRUSS

DL + SL
15psf + 30psf



Results for LC 3, DL + SL
Member Axial Forces (k)

SK - 1

Apr 15, 2015 at 2:03 PM

Bow String Truss Model Template.r3d

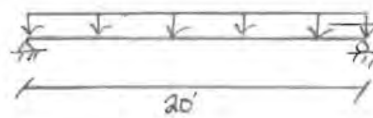
ARW ENGINEERS
structural consultants

Project No. 15118 Sheet No. 10
Project OWATC BLDG 10A
Prepared By ZCH Date

2x12 JOIST

JOIST LOADING (D.F. #1 GRADE)

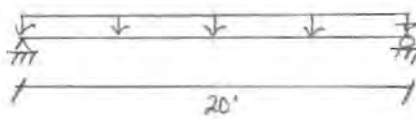
W/ UNIFORM SNOW:



$W_{DL} = 15psf(2') = 30plf$
$W_{SL} = 30psf(2') = 60plf$

CSR = 1.075

W/ SNOW DRIFT LOAD:



$W_{DL} = 30plf$
$W_{SL} = 60plf + 55psf(2') = 170 PLF$

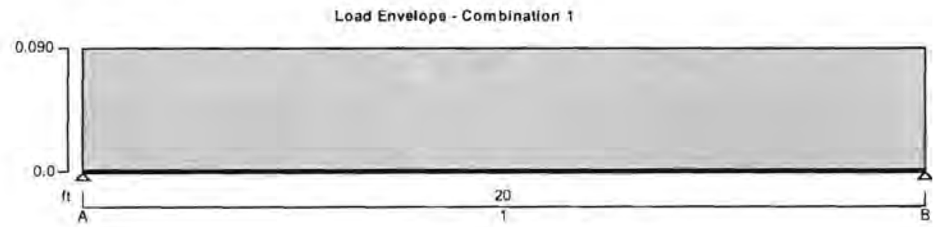
CSR = 2.390

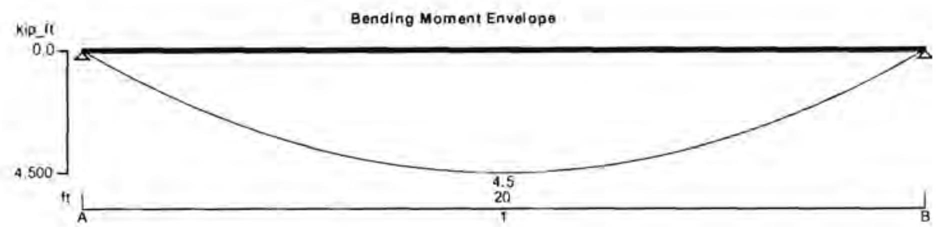
∴ UPGRADE W/ 1¾" x 11¼" LVL @ EA. 2x12 LOCATION UNDER DRIFT

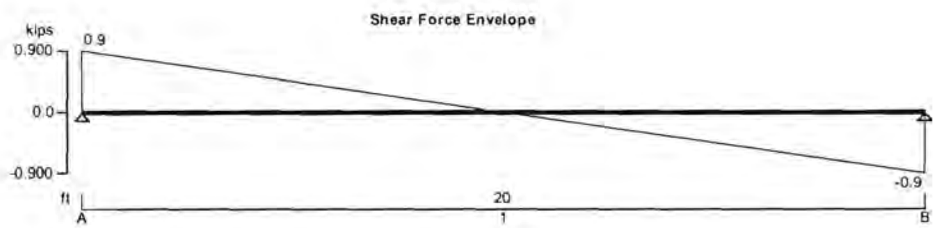
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead full UDL 30 lb/ft

Snow full UDL 60 lb/ft

Load combinations

Load combination 1	Support A	Dead × 1.00
		Snow × 1.00
	Span 1	Dead × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **4500** lb_ft	M_{min} = **0** lb_ft
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **4500** lb_ft	
Maximum shear	F_{max} = **900** lb	F_{min} = **-900** lb
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **900** lb	
Total load on member	W_{tot} = **1800** lb	
Reaction at support A	R_{A_max} = **900** lb	R_{A_min} = **900** lb
Unfactored dead load reaction at support A	R_{A_Dead} = **300** lb	
Unfactored snow load reaction at support A	R_{A_Snow} = **600** lb	
Reaction at support B	R_{B_max} = **900** lb	R_{B_min} = **900** lb
Unfactored dead load reaction at support B	R_{B_Dead} = **300** lb	
Unfactored snow load reaction at support B	R_{B_Snow} = **600** lb	

Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **12** in
Dressed depth of sections	d = **11.25** in
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **1.5** in

Table 4A - Reference design values for visually graded dimension lumber (2"-4" thick)

Species, grade and size classification	Douglas Fir-Larch, No.1 & Btr grade, 2" & wider
Bending parallel to grain	F_b = **1200** lb/in^2
Tension parallel to grain	F_t = **800** lb/in^2
Compression parallel to grain	F_c = **1550** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1800000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **112500** lb/in^2

Member details

Service condition	**Dry**
Length of bearing	L_b = **2.5** in
Load duration	**Two months**

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	A = N × b × d = **16.87** in^2

Section modulus $S_x = N \times b \times d^2 / 6 = \mathbf{31.64}$ in^3

$S_y = d \times (N \times b)^2 / 6 = \mathbf{4.22}$ in^3

Second moment of area $I_x = N \times b \times d^3 / 12 = \mathbf{177.98}$ in^4

$I_y = d \times (N \times b)^3 / 12 = \mathbf{3.16}$ in^4

Adjustment factors

Load duration factor - Table 2.3.2 $C_D = \mathbf{1.15}$

Temperature factor - Table 2.3.3 $C_t = \mathbf{1.00}$

Size factor for bending - Table 4A $C_{Fb} = \mathbf{1.00}$

Size factor for tension - Table 4A $C_{Ft} = \mathbf{1.00}$

Size factor for compression - Table 4A $C_{Fc} = \mathbf{1.00}$

Flat use factor - Table 4A $C_{fu} = \mathbf{1.20}$

Incising factor for modulus of elasticity - Table 4.3.8 $C_{iE} = \mathbf{1.00}$

Incising factor for bending, shear, tension & compression - Table 4.3.8

$C_i = \mathbf{1.00}$

Incising factor for perpendicular compression - Table 4.3.8

$C_{ic\_perp} = \mathbf{1.00}$

Repetitive member factor - cl.4.3.9 $C_r = \mathbf{1.15}$

Bearing area factor - eq.3.10-2 $C_b = (L_b + 0.375 \text{ in}) / L_b = \mathbf{1.15}$

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom}) = \mathbf{6.00}$

- Beam is fully restrained

Beam stability factor - cl.3.3.3 $C_L = \mathbf{1.00}$

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b = \mathbf{719}$ lb/in^2

Applied compression stress perpendicular to grain $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b) = \mathbf{240}$ lb/in^2

$f_{c\_perp} / F_{c\_perp}' = \mathbf{0.334}$

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = \mathbf{1587}$ lb/in^2

Actual bending stress $f_b = M / S_x = \mathbf{1707}$ lb/in^2

$f_b / F_b' = \mathbf{1.075}$

FAIL - Design bending stress is less than actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_t \times C_i = \mathbf{207}$ lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A) = \mathbf{80}$ lb/in^2

$f_v / F_v' = \mathbf{0.386}$

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection $E' = E \times C_{ME} \times C_t \times C_{iE} = \mathbf{1800000}$ lb/in^2

Design deflection $\delta_{adm} = 0.0056 \times L_{s1} = \mathbf{1.344}$ in

Bending deflection $\delta_{b\_s1} = \mathbf{1.011}$ in

Shear deflection $\delta_{v\_s1} = \mathbf{0.034}$ in

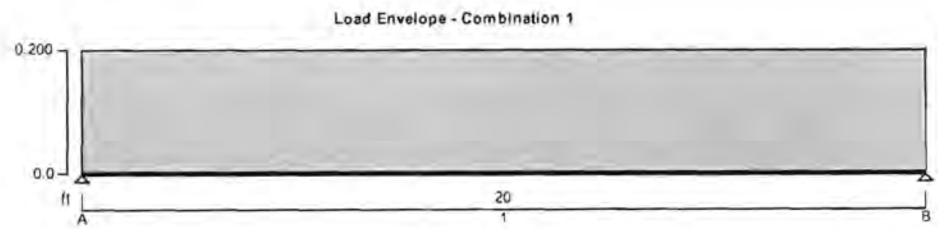
Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{1.045}$ in

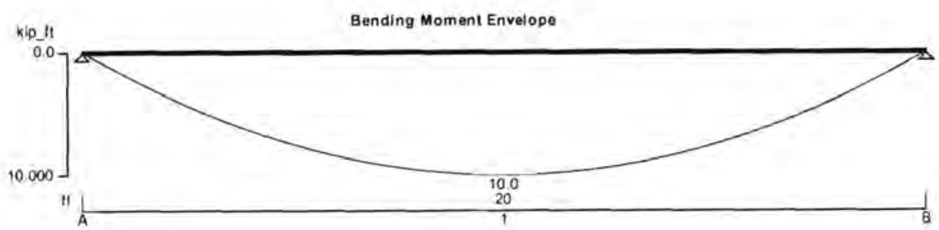
$\delta_a / \delta_{adm} = \mathbf{0.778}$

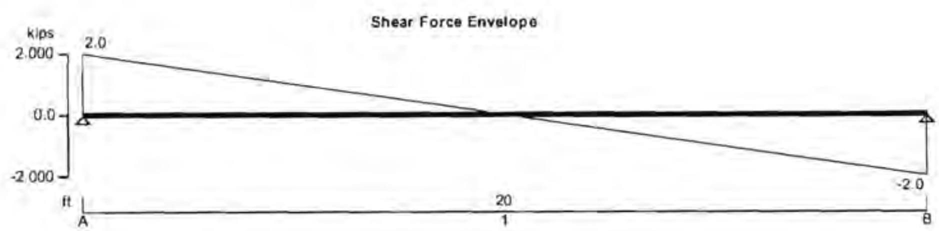
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead full UDL 30 lb/ft

Snow full UDL 170 lb/ft

Load combinations

Load combination 1	Support A	Dead × 1.00
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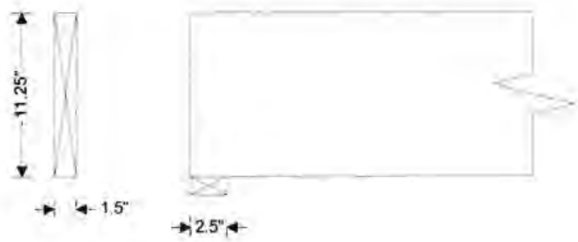
	Span 1	Dead × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **10000** lb_ft	M_{min} = **0** lb_ft
Design moment	$M = max(abs(M_{max}),abs(M_{min}))$ = **10000** lb_ft	
Maximum shear	F_{max} = **2000** lb	F_{min} = **-2000** lb
Design shear	$F = max(abs(F_{max}),abs(F_{min}))$ = **2000** lb	
Total load on member	W_{tot} = **4000** lb	
Reaction at support A	R_{A_max} = **2000** lb	R_{A_min} = **2000** lb
Unfactored dead load reaction at support A	R_{A_Dead} = **300** lb	
Unfactored snow load reaction at support A	R_{A_Snow} = **1700** lb	
Reaction at support B	R_{B_max} = **2000** lb	R_{B_min} = **2000** lb
Unfactored dead load reaction at support B	R_{B_Dead} = **300** lb	
Unfactored snow load reaction at support B	R_{B_Snow} = **1700** lb	



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Dressed depth of sections	d = **11.25** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **1.5** in

Table 4A - Reference design values for visually graded dimension lumber (2"-4" thick)

Species, grade and size classification	Douglas Fir-Larch, No.1 & Btr grade, 2" & wider
Bending parallel to grain	F_b = **1200** lb/in^2
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Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1800000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **112500** lb/in^2

Member details

Service condition	**Dry**
Length of bearing	L_b = **2.5** in
Load duration	**Two months**

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **16.87** in^2

Section modulus $S_x = N \times b \times d^2 / 6 = \mathbf{31.64}$ in^3

$S_y = d \times (N \times b)^2 / 6 = \mathbf{4.22}$ in^3

Second moment of area $I_x = N \times b \times d^3 / 12 = \mathbf{177.98}$ in^4

$I_y = d \times (N \times b)^3 / 12 = \mathbf{3.16}$ in^4

Adjustment factors

Load duration factor - Table 2.3.2 $C_D = \mathbf{1.15}$

Temperature factor - Table 2.3.3 $C_t = \mathbf{1.00}$

Size factor for bending - Table 4A $C_{Fb} = \mathbf{1.00}$

Size factor for tension - Table 4A $C_{Ft} = \mathbf{1.00}$

Size factor for compression - Table 4A $C_{Fc} = \mathbf{1.00}$

Flat use factor - Table 4A $C_{fu} = \mathbf{1.20}$

Incising factor for modulus of elasticity - Table 4.3.8 $C_{iE} = \mathbf{1.00}$

Incising factor for bending, shear, tension & compression - Table 4.3.8

$C_i = \mathbf{1.00}$

Incising factor for perpendicular compression - Table 4.3.8

$C_{ic\_perp} = \mathbf{1.00}$

Repetitive member factor - cl.4.3.9 $C_r = \mathbf{1.15}$

Bearing area factor - eq.3.10-2 $C_b = (L_b + 0.375 \text{ in}) / L_b = \mathbf{1.15}$

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom}) = \mathbf{6.00}$

- Beam is fully restrained

Beam stability factor - cl.3.3.3 $C_L = \mathbf{1.00}$

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b = \mathbf{719}$ lb/in^2

Applied compression stress perpendicular to grain $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b) = \mathbf{533}$ lb/in^2

$f_{c\_perp} / F_{c\_perp}' = \mathbf{0.742}$

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = \mathbf{1587}$ lb/in^2

Actual bending stress $f_b = M / S_x = \mathbf{3793}$ lb/in^2

$f_b / F_b' = \mathbf{2.390}$

FAIL - Design bending stress is less than actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_t \times C_i = \mathbf{207}$ lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A) = \mathbf{178}$ lb/in^2

$f_v / F_v' = \mathbf{0.859}$

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection $E' = E \times C_{ME} \times C_t \times C_{iE} = \mathbf{1800000}$ lb/in^2

Design deflection $\delta_{adm} = 0.0056 \times L_{s1} = \mathbf{1.344}$ in

Bending deflection $\delta_{b\_s1} = \mathbf{2.247}$ in

Shear deflection $\delta_{v\_s1} = \mathbf{0.076}$ in

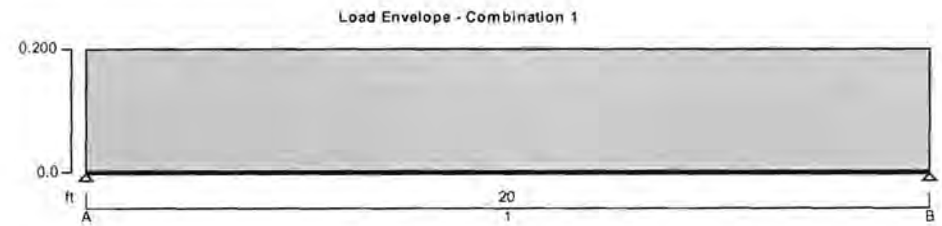
Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{2.323}$ in

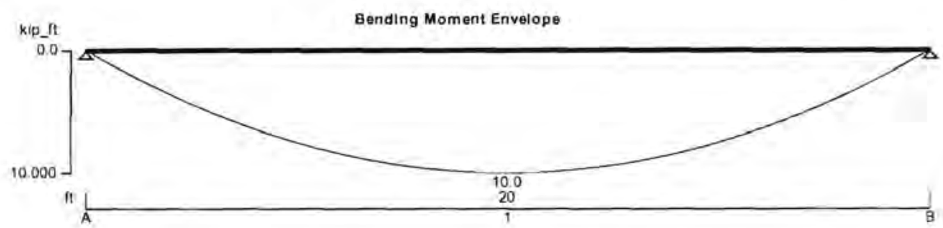
$\delta_a / \delta_{adm} = \mathbf{1.729}$

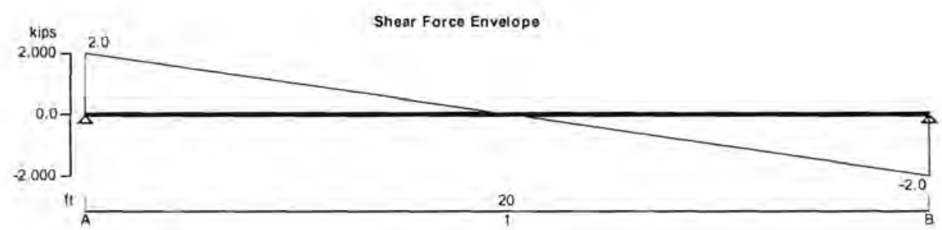
STRUCTURAL COMPOSITE LUMBER BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead full UDL 30 lb/ft

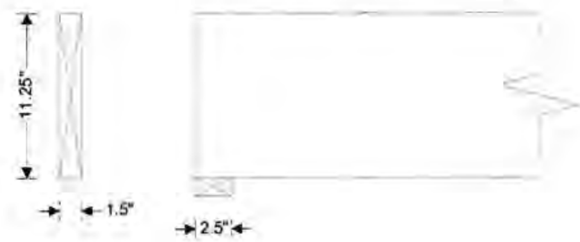
Snow full UDL 170 lb/ft

Load combinations

Load combination 1	Support A	Dead × 1.00
		Snow × 1.00
	Span 1	Dead × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **10000** lb_ft	M_{min} = **0** lb_ft
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **10000** lb_ft	
Maximum shear	F_{max} = **2000** lb	F_{min} = **-2000** lb
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **2000** lb	
Total load on member	W_{tot} = **4000** lb	
Reaction at support A	R_{A_max} = **2000** lb	R_{A_min} = **2000** lb
Unfactored dead load reaction at support A	R_{A_Dead} = **300** lb	
Unfactored snow load reaction at support A	R_{A_Snow} = **1700** lb	
Reaction at support B	R_{B_max} = **2000** lb	R_{B_min} = **2000** lb
Unfactored dead load reaction at support B	R_{B_Dead} = **300** lb	
Unfactored snow load reaction at support B	R_{B_Snow} = **1700** lb	



Composite section details

Breadth of composite section	b = **1.5** in
Depth of composite section	d = **11.25** in
Number of composite sections in member	N = **1**
Overall breadth of composite member	b_b = N × b = **1.5** in

Reference design values for structural composite lumber

Composite type and grade	Microllam LVL, 1.9E-2600Fb grade
Bending parallel to grain	F_b = **2600** lb/in^2
Tension parallel to grain	F_t = **1555** lb/in^2
Compression parallel to grain	F_c = **2510** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **750** lb/in^2
Shear parallel to grain	F_v = **285** lb/in^2
Modulus of elasticity	E = **1900000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **118750** lb/in^2
Average density	ρ = **42** lb/ft^3

Member details

Service condition	**Dry**
Length of bearing	L_b = **2.5** in
Load duration	**Two months**

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	A = N × b × d = **16.87** in^2
Section modulus	S_x = N × b × d^2 / 6 = **31.64** in^3

	$S_y = d \times (N \times b)^2 / 6 = \mathbf{4.22}$ in^3
Second moment of area	$I_x = N \times b \times d^3 / 12 = \mathbf{177.98}$ in^4
	$I_y = d \times (N \times b)^3 / 12 = \mathbf{3.16}$ in^4

Adjustment factors

Load duration factor - Table 2.3.2	$C_D = \mathbf{1.15}$
Temperature factor - Table 2.3.3	$C_t = \mathbf{1.00}$
Size factor for bending	$C_{Fb} = (12 \text{ in} / \max(d, 3.5 \text{ in}))^{0.136} = \mathbf{1.01}$
Size factor for shear	$C_{Fv} = \mathbf{1.00}$
Repetitive member factor - cl.8.3.7	$C_r = \mathbf{1.04}$
Length factor	$C_{Len} = \mathbf{1.00}$
Bearing area factor - eq.3.10-2	$C_b = (L_b + 0.375 \text{ in}) / L_b = \mathbf{1.15}$
Depth-to-breadth ratio	$d / (N \times b) = \mathbf{7.50}$
Effective laterally unsupported span length	$l_e = \mathbf{1}$ ft
Slenderness ratio for bending members - eq.3.3-5	$R_b = \min(\sqrt{[l_e \times d / (N \times b)^2]}, 50) = \mathbf{7.746}$
Adjusted bending design value for bending	$F_b^* = F_b \times C_D \times C_M \times C_t \times C_{Fb} \times C_r = \mathbf{3137}$ lb/in^2
Adjusted modulus of elasticity for member stability	$E' = E \times C_M \times C_t = \mathbf{1900000}$ lb/in^2
Critical buckling design value for bending	$F_{bE} = 1.2 \times E' / R_b^2 = \mathbf{38000}$ lb/in^2
Beam stability factor - eq.3.3-6	

$$C_L = [1 + (F_{bE} / F_b^*)] / 1.9 - \sqrt{[([1 + (F_{bE} / F_b^*)] / 1.9)^2 - (F_{bE} / F_b^*) / 0.95]} = \mathbf{1.00}$$

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain	$F_{c\_perp}' = F_{c\_perp} \times C_t \times C_b = \mathbf{863}$ lb/in^2
Applied compression stress perpendicular to grain	$f_{c\_perp} = R_{A\_max} / (N \times b \times L_b) = \mathbf{533}$ lb/in^2
	$f_{c\_perp} / F_{c\_perp}' = \mathbf{0.618}$

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress	$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_r = \mathbf{3123}$ lb/in^2
Actual bending stress	$f_b = M / S_x = \mathbf{3793}$ lb/in^2
	$f_b / F_b' = \mathbf{1.214}$

FAIL - Design bending stress is less than actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress	$F_v' = F_v \times C_D \times C_t \times C_{Fv} = \mathbf{328}$ lb/in^2
Actual shear stress - eq.3.4-2	$f_v = 3 \times F / (2 \times A) = \mathbf{178}$ lb/in^2
	$f_v / F_v' = \mathbf{0.542}$

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection	$E' = E \times C_M \times C_t = \mathbf{1900000}$ lb/in^2
Design deflection	$\delta_{adm} = 0.0056 \times L_{s1} = \mathbf{1.344}$ in
Bending deflection	$\delta_{b\_s1} = \mathbf{2.129}$ in
Shear deflection	$\delta_{v\_s1} = \mathbf{0.072}$ in
Total deflection	$\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{2.201}$ in
	$\delta_a / \delta_{adm} = \mathbf{1.638}$

FAIL - Design deflection exceeds total deflection

BOTTOM CHORD

(2) $2\frac{1}{2}'' \times 11''$ BOTTOM CHORDS
D.F. #1 + BETTER

TOTAL TENSION LOAD = 67 K
MOMENT = 2.8 K-ft (33.6 K-in)

CHECK GROSS AREA TENSION

$A = 2(2.5'')(11'') = 55\,in^2$

$I = \frac{2.5(11)^3}{12}(2) = 555\,in^4$

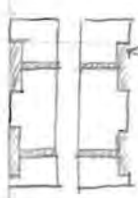
$S = \frac{2.5(11)^2}{6}(2) = 101\,in^3$

$C_D = 1.15$ $\quad F_t = 800$ psi
$C_M = 1.0$
$C_t = 1.0$
$C_F = 1.0$
$C_i = 1.0$

$\therefore F'_t = 800\,psi(1.15) = 920\,psi$

$F_{t\,ACTUAL} = \frac{67\,K}{55\,in^2} = 1{,}218\,psi$ $\qquad CSR = \frac{1{,}218}{920} = 1.32 \therefore$ NO GOOD

CHECK NET AREA TENSION (@ SPLIT RING SPLICE)



(4) 4" Ø SPLIT RING (½" DEEP)
W/ 1 1/16" Ø BOLT

AREA REMOVED = $4[(4'')(.5) + 2''(.75'')] = 14\,in^2$

$A_{NET} = 55\,in^2 - 14\,in^2 = 41\,in^2$

$F_{t\,ACTUAL} = \frac{67K}{41\,in^2} = 1{,}634\,psi$ $\qquad CSR = \frac{1{,}634}{920} = 1.78$ $\therefore$ NO GOOD

BOTTOM CHORD UPGRADE

* REINFORCE BOTTOM CHORD W/ STEEL TO TAKE ALL AXIAL LOAD, USE A36 STEEL

BOTTOM CHORD AXIAL TENSION FORCE = 70K (SEE RISA)

REQ'D AREA OF STEEL = $\frac{70K(1.67)}{36 \text{ ksi}} = 3.25 \text{ in}^2$

USE (2) L4x4x1/4" ANGLES OR (4) 1 1/8" Ø BARS

TRY 50KSI BARS – $A_{S\,REQ'D} = \frac{70K(1.67)}{50 \text{ ksi}} = 2.34 \text{ in}^2$

USE (4) 7/8" Ø BARS

CHECK IF BOTTOM CHORD IS ADEQUATE TO TAKE MOMENT FROM AXIAL LOADS IN WEB MEMBERS

MAX MOMENT = 9.84 K-FT = 118.1 K-in (SEE RISA)

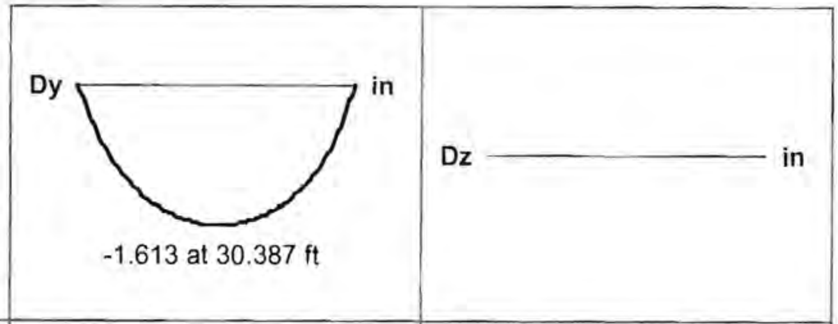
$S = \frac{2.5''(11'')^2}{6}(2) = 101 \text{ in}^3$

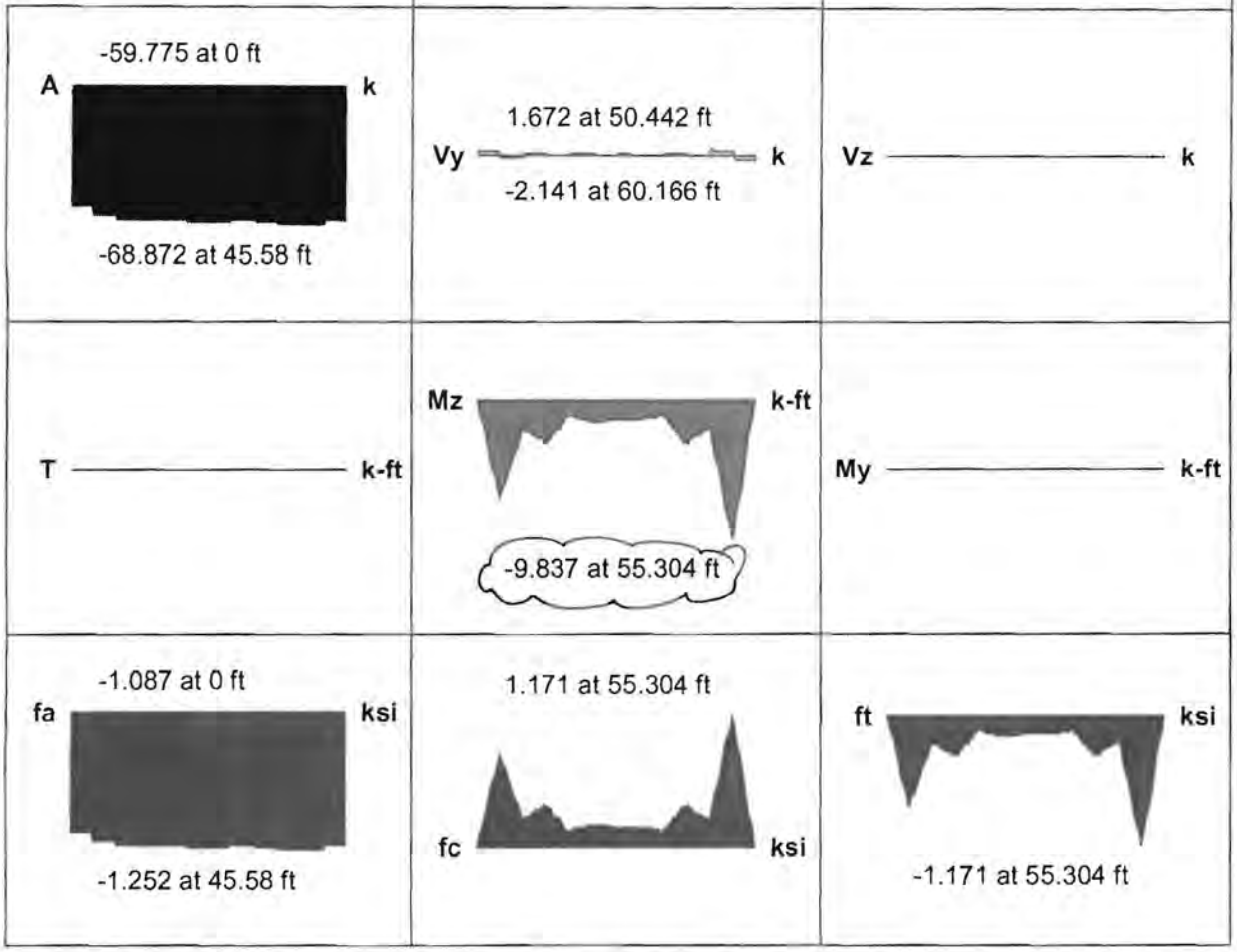
$f_b = 118.1\text{k-in}/101 \text{ in}^3 = 1.17 \text{ ksi}$

$F_b' = 1.66 \text{ ksi}$ (SEE RISA OUTPUT) ∴ BOTTOM CHORD IS OKAY TO TAKE MOMENT.

Beam: ***M15***

Shape: **2-2.5X11FS**
Material: **DF**
Length: **60.166 ft**
I Joint: **N1**
J Joint: **N13**
LC 3: DL + SL
Code Check: **2.204 (bending)**
Report Based On 100 Sections





AF&PA NDS-12: ASD Code Check

Max Bending Check	**2.204**	Max Shear Check	**0.282 (y)**
Location	**55.304 ft**	Location	**60.166 ft**
Equation	**3.9-1**	Max Defl Ratio	**L/447**

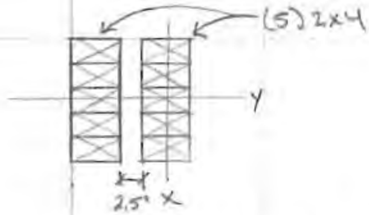
CD **1.15** RB **5.138** CL **.998**
Cr **1** Cfu **1.2** CP **.133** Kf **.6**

	(ksi)	Cm	Ct	CF
Fc'	.237	1	1	1
Ft'	.92	1	1	1
Fb1'	1.377	1	1	1
Fb2'	1.656	1	1	1
Fv'	.207	1	1	
E'	1800	1	1	

	y-y	z-z
Lb	**15 ft**	**5 ft**
le/d	**36**	**5.455**
Sway	**No**	**No**

Le-Bending Top **5 ft**
Le-Bending Bot **5 ft**

ARCHED TOP CHORD



FORCE IN TOP CHORD => P_u = 72K (COMP)
V_u = 300 lbs

NOTE: ANALYZE TOP CHORD AS (2) SEPERATE MEMBERS BRACED @ 5'-0" o.c. AND AS (1) MEMBER BRACED @ 30'.

SINGLE SIDE BRACED @ 5'-0" o.c. LOAD => P_u = 72K/2 = 36.0 K
V_u = 300lb/2 = 150 lbs

CSR = 1.023 ∴ SINGLE MEMBERS ARE ADEQUATE @ 5'-0" o.c.

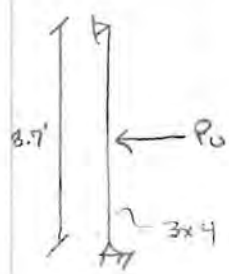
BOTH MEMBERS BRACED @ 30'-0" o.c.

IDEALIZE AS (5) 2x8 FOR CALCS

BRACED @ 30' CSR = 6.17 K ∴ NO GOOD

IF TOP CHORD IS BRACED @ 10'-0" o.c. THEN CSR = 0.996

CHECK IF WEBS CAN BRACE TOP CHORD:



P_u = 72 K (0.01) = 720 lbs

WEB IS NOT ADEQUATE, ∴ BRACE TOP CHORD @ 10'-0" o.c.

* SEE FOLLOWING PAGES FOR MORE DETAILED CALCS

TOP CHORD FORCES

Member Section Forces

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-	z-z Moment[k-
1	3	M47A	1	.148	2.153	0	0	0	0
2			2	.08	1.077	0	0	0	-1.922
3			3	.011	0	0	0	0	-2.562
4			4	-.058	-1.077	0	0	0	-1.922
5			5	-.126	-2.153	0	0	0	0
6	3	M48A	1	.157	2.267	0	0	0	0
7			2	.085	1.133	0	0	0	-2.129
8			3	.013	0	0	0	0	-2.839
9			4	-.059	-1.133	0	0	0	-2.129
10			5	-.131	-2.267	0	0	0	0
11	3	M49	1	.14	2.304	0	0	0	0
12			2	.066	1.152	0	0	0	-2.201
13			3	-.007	0	0	0	0	-2.934
14			4	-.081	-1.152	0	0	0	-2.201
15			5	-.154	-2.304	0	0	0	0
16	3	M50	1	.098	2.304	0	0	0	0
17			2	.024	1.152	0	0	0	-2.201
18			3	-.049	0	0	0	0	-2.935
19			4	-.123	-1.152	0	0	0	-2.201
20			5	-.196	-2.304	0	0	0	0
21	3	M51	1	.006	2.304	0	0	0	0
22			2	-.068	1.152	0	0	0	-2.201
23			3	-.141	0	0	0	0	-2.935
24			4	-.214	-1.152	0	0	0	-2.201
25			5	-.288	-2.304	0	0	0	0
26	3	M52	1	-.139	2.304	0	0	0	0
27			2	-.212	1.152	0	0	0	-2.201
28			3	-.286	0	0	0	0	-2.935
29			4	-.359	-1.152	0	0	0	-2.201
30			5	-.433	-2.304	0	0	0	0
31	3	M53	1	-.002	2.304	0	0	0	0
32			2	-.076	1.152	0	0	0	-2.201
33			3	-.149	0	0	0	0	-2.935
34			4	-.223	-1.152	0	0	0	-2.201
35			5	-.296	-2.304	0	0	0	0
36	3	M54	1	.081	2.304	0	0	0	0
37			2	.007	1.152	0	0	0	-2.201
38			3	-.066	0	0	0	0	-2.935
39			4	-.14	-1.152	0	0	0	-2.201
40			5	-.213	-2.304	0	0	0	0
41	3	M55	1	.126	2.304	0	0	0	0
42			2	.053	1.152	0	0	0	-2.201
43			3	-.021	0	0	0	0	-2.935
44			4	-.094	-1.152	0	0	0	-2.201
45			5	-.168	-2.304	0	0	0	0
46	3	M56	1	.151	2.304	0	0	0	0
47			2	.078	1.152	0	0	0	-2.201
48			3	.004	0	0	0	0	-2.934
49			4	-.069	-1.152	0	0	0	-2.201
50			5	-.143	-2.304	0	0	0	0
51	3	M57	1	.159	2.267	0	0	0	0
52			2	.087	1.133	0	0	0	-2.129
53			3	.015	0	0	0	0	-2.839
54			4	-.057	-1.133	0	0	0	-2.129
55			5	-.13	-2.267	0	0	0	0
56	3	M58	1	.147	2.153	0	0	0	0

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-	z-z Moment[k-
57			2	.079	1.077	0	0	0	-1.922
58			3	.01	0	0	0	0	-2.562
59			4	-.058	-1.077	0	0	0	-1.922
60			5	-.127	-2.153	0	0	0	0

$V_u = 2.3\text{ K}$

$M_u = 3.0\text{ K-ft}$

THIS DOES NOT CONSIDER SNOW DRIFT LOAD. SEE DRIFT UPGRADE CALCS FOR THIS LOAD.

Bottom Chord Forces

Member Section Forces

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Mome...	z-z Moment[k-ft]
1	3	M15	1	-59.775	1.504	0	0	0	0
2			2	-66.027	-.351	0	0	0	-2.939
3			3	-66.947	-.075	0	0	0	-1.367
4			4	-67.483	.357	0	0	0	-2.978
5			5	-66.742	-2.141	0	0	0	0

$T_U = 68K$ (TENSION)

$V_U = 2.1K$

$M_U = 3.0$ K-ft

ARCHED CHORD FORCES

Member Section Forces

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Mome...	z-z Moment[k-ft]
1	3	M23	1	67.232	0	-.032	0	0	0
2			2	67.234	0	-.016	0	-.031	0
3			3	67.236	0	0	0	-.041	0
4			4	67.238	0	.016	0	-.031	0
5			5	67.24	0	.032	0	0	0
6	3	M24	1	69.639	0	-.032	0	0	0
7			2	69.642	0	-.016	0	-.032	0
8			3	69.645	0	0	0	-.042	0
9			4	69.648	0	.016	0	-.032	0
10			5	69.652	0	.032	0	0	0
11	3	M20	1	62.939	0	-.032	0	0	0
12			2	62.938	0	-.016	0	-.031	0
13			3	62.936	0	0	0	-.041	0
14			4	62.934	0	.016	0	-.031	0
15			5	62.932	0	.032	0	0	0
16	3	M19	1	66.209	0	-.032	0	0	0
17			2	66.206	0	-.016	0	-.032	0
18			3	66.203	0	0	0	-.042	0
19			4	66.2	0	.016	0	-.032	0
20			5	66.197	0	.032	0	0	0
21	3	M22	1	65.127	0	-.032	0	0	0
22			2	65.128	0	-.016	0	-.031	0
23			3	65.128	0	0	0	-.041	0
24			4	65.128	0	.016	0	-.031	0
25			5	65.129	0	.032	0	0	0
26	3	M21	1	64.672	0	-.032	0	0	0
27			2	64.671	0	-.016	0	-.031	0
28			3	64.671	0	0	0	-.041	0
29			4	64.671	0	.016	0	-.031	0
30			5	64.67	0	.032	0	0	0
31	3	M18	1	64.42	0	-.032	0	0	0
32			2	64.416	0	-.016	0	-.032	0
33			3	64.413	0	0	0	-.042	0
34			4	64.409	0	.016	0	-.032	0
35			5	64.405	0	.032	0	0	0
36	3	M25	1	69.604	0	-.032	0	0	0
37			2	69.608	0	-.016	0	-.032	0
38			3	69.612	0	0	0	-.042	0
39			4	69.615	0	.016	0	-.032	0
40			5	69.619	0	.032	0	0	0
41	3	M26	1	69.467	0	-.032	0	0	0
42			2	69.472	0	-.016	0	-.032	0
43			3	69.477	0	0	0	-.042	0
44			4	69.483	0	.016	0	-.032	0
45			5	69.488	0	.032	0	0	0
46	3	M17	1	61.679	0	-.032	0	0	0
47			2	61.673	0	-.016	0	-.032	0
48			3	61.668	0	0	0	-.042	0
49			4	61.663	0	.016	0	-.032	0
50			5	61.657	0	.032	0	0	0
51	3	M16	1	64.415	0	-.03	0	0	0
52			2	64.409	0	-.015	0	-.029	0
53			3	64.403	0	0	0	-.039	0
54			4	64.396	0	.015	0	-.029	0
55			5	64.39	0	.03	0	0	0
56	3	M27	1	71.822	0	-.03	0	0	0

RISA TECHNOLOGIES
Company
Designer
Job Number
Model Name

Apr 22, 2015
Checked By:____

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Mome...	z-z Moment[k-ft]
57			2	71.828	0	-.015	0	-.029	0
58			3	71.834	0	0	0	-.039	0
59			4	71.841	0	.015	0	-.029	0
60			5	71.847	0	.03	0	0	0

$P_U = 72\text{ K}$ (COMPRESSION)

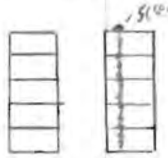
$V_U = 320\text{ lbs}$

Bow string truss

Arched chord analysis (single chord analysis) [Braced out of plane at every panel point]

(5) 2x4s (2) sets panel points = 5'-3" max

screws @ 9"oc staggered



★ analyze as bolted built up column per NDS 2012 15.3.4
by 1/4" ϕ screws

d) ∴ bolt spacing in a row = 4(1/4) = 1" o.c. min
6(1.5) = 9" o.c. max ←

e) ∴ spacing between rows = 1.5(1/4) = 0.375" min
10(1/4) = 2.5" max ←

g) d > 3t_m
3.5" > 4.5" ∴ only (1) row of bolts req'd

use 1/4"ϕ screws @ 9"oc staggered

$$C_p = k_f\left[\frac{1+(F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1+(F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}\right] =$$

pin-pin	fix-fix
↓	↓
0.62	0.86

$F^*_c = F_c * C_D^{1.15} * C_M * C_t * C_F^{1.15} * C_i$
$F^*_c = 1550 * 1.15 * 1.15$
$F^*_c = 2050$ psi

$F_{cE}/F_c^* =$ 0.82 (pin-pin) 1.93 (fix-fix)

$F_{cE} = \dfrac{0.822\,E_{min}'}{(l_e/d)^2} \Rightarrow$ 1674.4 (pin-pin) 3963 (fixed-fixed)

$E_{min}' = E_{min} * C_T$ ← 1.0 conservative
$E_{min}' = 660{,}000$ psi

15.3.2.1 And Appendix G → pin-pin $k_e l_e$ → $k_e l_e = 5.25' * 12 = 63"$
$d = 3.5"$

fix-fix $k_e = 0.65$ suggested, NDS Appendix G & 15.3.2.1
$k_e l_e = 63"(0.65) = 41"$

$C = 0.8$ sawn lumber $k_f = 1.0$

Arched chord analysis cont.

$F'_c = 2050\,psi \times C_p$ → 0.62

$F'_c = 1271\,psi$ (pin-pin) 1765 (fix-fix)

$Area = 3.5'' \times 5(1.5'') = 26.25\,in^2$

Axial load from RISA model

Pin-Pin : $1271\,lb/in^2\,(26.25\,in^2) = \underline{33.4\ kips} \ngtr 35\ kips$

Fix-fix : $1765\,lb/in^2\,(26.25\,in^2) = \underline{46.3\ kips} > 35\ kips$

fix-fix

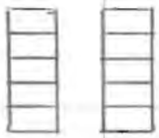
The Arched chords will act between panel points as somewhere between a pinned-pinned condition and a fixed-fixed condition since the chords are continuous (and not spliced at the panel points). Since the chord is continuous it will behave closer to a fixed-fixed condition. As shown by the analysis, the fixed-fixed condition column has a capacity of 46.3 kips which is larger than the 35 kip applied axial load.

Therefore, the existing arched chords are ok if they are braced out-of-plane at each panel point.

Bow String Truss

Arched Chord analysis (double chord analysis) [Braced out-of-plane at every other panel point]

(5) 2x4s (2) sets panel points @ 5'-3" o.c.



* Spaced Columns (NDS 2012 15.2)

15.2.3.2

a) $l_1/d_1 \le 80 \Rightarrow \frac{10.6'(12'')}{3.5''} = 36 \quad \therefore ok$

b) $l_2/d_2 \le 50 \Rightarrow \frac{10.6'(12'')}{5(1.5'')} = 17 \quad \therefore ok$

c) $l_3/d_1 \le 40 \Rightarrow \frac{5.3'(12'')}{3.5''} = 18 \quad \therefore ok$

$$C_p = \frac{1+(F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1+(F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} = \begin{array}{cc} \text{pin-pin} & \text{fix-fix} \\ \underline{\underline{0.5}} & \underline{\underline{0.79}} \end{array}$$

$F_c^* = 2050\,psi$ (from before)

$$F_{cE} = \frac{0.822 K_x E_{min}}{(l_e/d)^2} = \begin{array}{cc} \text{pin-pin} & \text{fix-fix} \\ 1235\,psi & 2922\,psi \end{array}$$

$K_x = 3.0$ = fixity condition "b"

$$\text{pin-pin} = \frac{l_e}{d} = \frac{10.6'(12'')}{3.5''} = 36.3$$

15.2.3.1 $\longrightarrow$ $\text{fix-fix} = \frac{\overset{\text{Appendix G}}{k} \cdot l_e}{d} = \frac{0.65(10.6'(12''))}{3.5''} = 23.6$

$$\frac{F_{cE}}{F_c^*} = \begin{array}{cc} \text{pin-pin} & \text{fix-fix} \\ 0.6 & 1.4 \end{array}$$

$c = 0.8$ = sawn lumber

double chord analysis cont...

$F'c = 2050\text{psi} * C_p$

$C_{p_{\text{pin-pin}}} = 0.5$, $C_{p_{\text{fix-fix}}} = 0.79$

$F'c_{\text{pin}} = 1025 \text{ psi}$

$F'c_{\text{fix}} = 1619.5 \text{ psi}$

$\text{Area} = 3.5' \times 5(1.5'') \times 2 \text{ chords} = 52.5 \text{in}^2$

Axial Load from Risa

Pin-pin : $1025\text{psi}(52.5\text{in}^2) = 53.8^{\text{kips}} \not> 35^k \times 2 = 70^{\text{kips}}$

fix-fix : $1619.5\text{psi}(52.5\text{in}^2) = 85^{\text{kips}} > 70^{\text{kips}}$ $\therefore$ ok

unbraced length req'd to achieve 70^{kips}

$l_e = 8.7^{ft}$ (see spreadsheet)

Bow string Truss

Arched chord analysis (alternate)

- check buckling in individual ply of arched chord.
  - 1-ply = (1) 2x4
  - Total axial compression : 70 kips (from RISA model)

# of plies : 10

$70^{kips} / 10_{plies} = 7$ kips per ply

NDS 3.6 & 3.7 - simple solid column check

$$C_p = \frac{1 + (F_{cE}/F^*_c)}{2c} - \sqrt{\left[\frac{1 + F_{cE}/F^*_c}{2c}\right]^2 - \frac{F_{cE}/F^*_c}{c}} \cong 1.0$$

$c = 0.8$

$F^*_c = 2050_{psi}$ (from before)

$F_{cE} = \frac{0.822\, E'_{min}}{(l_e/d)^2} \Rightarrow 2119$

$E'_{min} = 660{,}000$ psi (from before), try adding bolts @ 24" o.c.

$\frac{l_e}{d} = \frac{24"}{1.5} = 16$

pin: $\frac{F_{cE}}{F^*_c} \cong 1.0$

$2050_{psi}\ (\overset{C_p}{1.0})\ (1.5" \times 3.5") = 10762^{\#} > 7^{kips}$ ∴ ok

∴ use thru bolts @ 24" o.c.

↳ use bolts @ 18" o.c. to be conservative

Wood Column

\15118 - OWATC BDO 10A Structural Upgrade\Engineering\Calculations\Other\15118 owatc bdo 10a upgrade.ec6
ENERCALC, INC. 1983-2015, Build:6.15.7.8, Ver:6.15.7.8

Lic. # : KW-06002489 **Licensee : ARW ENGINEERS**

Description : arched chord brace (check 1-ply)

Code References

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method :	Allowable Stress Design	Wood Section Name	2x4
End Fixities	Top Fixed, Bottom Fixed	Wood Grading/Manuf.	Graded Lumber
Overall Column Height	10.50 ft	Wood Member Type	Sawn

(Used for non-slender calculations)

Wood Species	Douglas Fir - Larch		
Wood Grade	No.1 & Better		
Fb - Tension	1200 psi	Fv	180 psi
Fb - Compr	1200 psi	Ft	800 psi
Fc - Prll	1550 psi	Density	31.2 pcf
Fc - Perp	625 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1800	1800	1800 ksi
Minimum	660	660	

Exact Width	1.50 in
Exact Depth	3.50 in
Area	5.250 in^2
Ix	5.359 in^4
Iy	0.9844 in^4

Allow Stress Modification Factors	
Cf or Cv for Bending	1.50
Cf or Cv for Compression	1.150
Cf or Cv for Tension	1.50
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No (non-glb only)

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Fully braced against buckling along Y-Y Axis

$C_p = 1.0$

Applied Loads

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

Column self weight included : 11.944 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.50 ft, S = 7.0 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.7493 : 1
Load Combination	+D+S+H
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are . . .	
Applied Axial	7.012 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	1,782.50 psi
PASS Maximum Shear Stress Ratio =	0.0 : 1
Load Combination	+0.60D+0.70E+0.60H
Location of max.above base	10.50 ft
Applied Design Shear	0.0 psi
Allowable Shear	180.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension
Cf or Cv : Size based factors	1.500	1.150	

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+H	1.000	1.000	0.001276	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+L+H	1.000	1.000	0.001276	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+Lr+H	1.000	1.000	0.001276	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+S+H	1.000	1.000	0.7493	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750Lr+0.750L+H	1.000	1.000	0.001276	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750L+0.750S+H	1.000	1.000	0.5623	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.60W+H	1.000	1.000	0.001276	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.70E+H	1.000	1.000	0.001276	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750Lr+0.750L+0.450W+H	1.000	1.000	0.001276	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750L+0.750S+0.450W+H	1.000	1.000	0.5623	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750L+0.750S+0.5250E+H	1.000	1.000	0.5623	PASS	0.0 ft	0.0	PASS	10.50 ft
+0.60D+0.60W+0.60H	1.000	1.000	0.000766	PASS	0.0 ft	0.0	PASS	10.50 ft

Wood Column

Description : arched chord brace (check 1-ply)

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D+0.70E+0.60H	1.000	1.000	0.000766	PASS	0.0 ft	0.0	PASS	10.50 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base
+D+H		k		k	0.012 k
+D+L+H		k		k	0.012 k
+D+Lr+H		k		k	0.012 k
+D+S+H		k		k	7.012 k
+D+0.750Lr+0.750L+H		k		k	0.012 k
+D+0.750L+0.750S+H		k		k	5.262 k
+D+0.60W+H		k		k	0.012 k
+D+0.70E+H		k		k	0.012 k
+D+0.750Lr+0.750L+0.450W+H		k		k	0.012 k
+D+0.750L+0.750S+0.450W+H		k		k	5.262 k
+D+0.750L+0.750S+0.5250E+H		k		k	5.262 k
+0.60D+0.60W+0.60H		k		k	0.007 k
+0.60D+0.70E+0.60H		k		k	0.007 k
D Only		k		k	0.012 k
Lr Only		k		k	k
L Only		k		k	k
S Only		k		k	7.000 k
W Only		k		k	k
E Only		k		k	k
H Only		k		k	k

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+0.450W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.450W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.60W+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

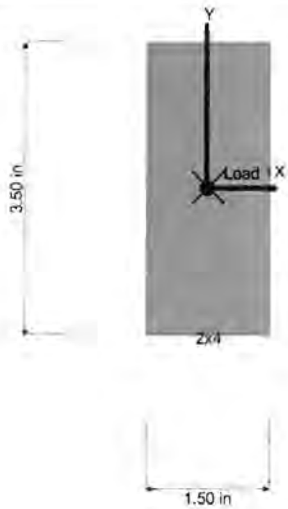
Wood Column

\15118 - OWATC BDO 10A Structural Upgrade\Engineering\Calculations\Other\15118 owatc bdo 10a upgrade.ec6
ENERCALC, INC. 1983-2015, Build:6.15.7.8, Ver:6.15.7.8

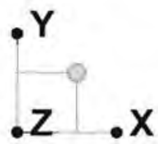
Lic. # : KW-06002489 **Licensee : ARW ENGINEERS**

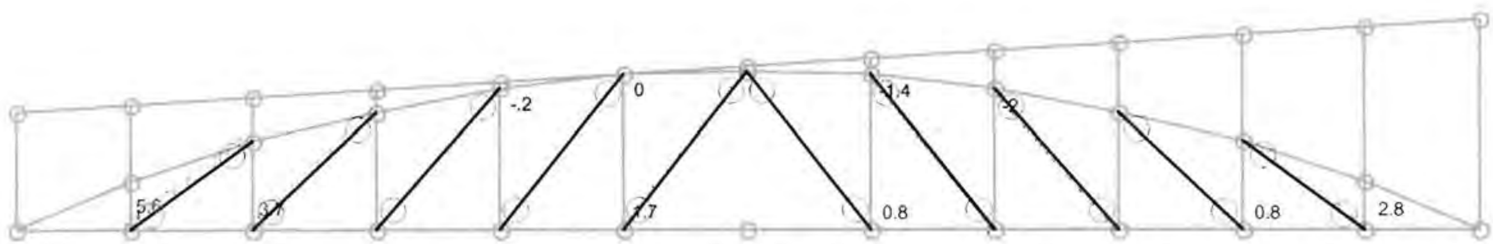
Description : arched chord brace (check 1-ply)

Sketches



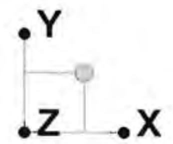
Loads are total entered value. Arrows do not reflect absolute direction.

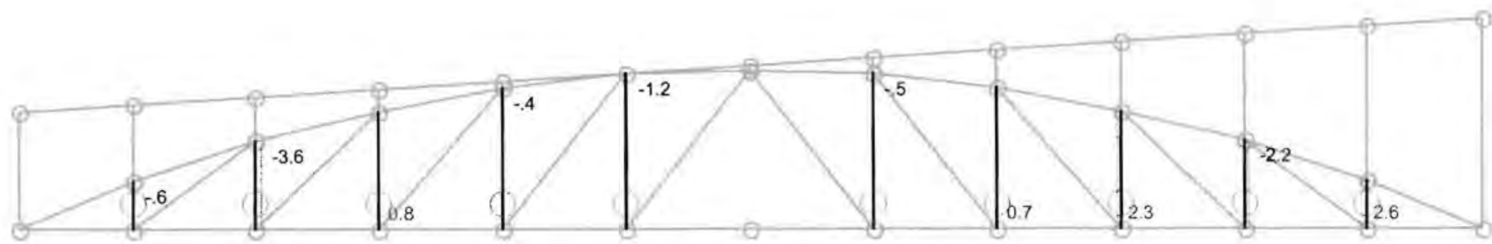




Results for LC 3, DL + SL
Member Axial Forces (k)

		SK - 2
		June 9, 2015 at 2:19 PM
	Worse Case Diag. Web Axial Loads	Bow String Truss Model Pinned Arch.r3d



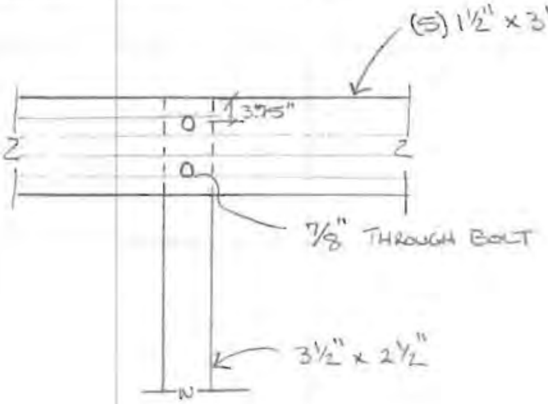


Results for LC 3, DL + SL
Member Axial Forces (k)

		SK - 1
		June 9, 2015 at 2:00 PM
	Worse Case Vertical Web Axial Loads	Bow String Truss Model Pinned Arch.r3d

BOLT CAPACITY OF WEBS TO TRUSS CHORDS

TRUSS TO ARCHED CHORD



$Z_{\parallel}$ = 3270 lbs (WEB)

$Z_{\perp}$ = 1470 lbs (ARCH CHORD)

WEB MEMBER CAPACITY:

EDGE DISTANCE => l/D = 2.5/7/8 = 2.86 ∴ MIN. EDGE = 1.5(7/8) = 1.3"

ACTUAL = 1.75" ∴ O.K.

END DISTANCE => FOR TENSION LOAD => C_Δ=0.5 END = 3.5(0.875in) = 3"

C_Δ=1.0 END = 7(0.875in) = 6"

ACTUAL = 3.75" ∴ C_Δ=0.5

FOR COMPRESSION LOAD => C_Δ=1.0 END = 4(0.875in) = 3.5"

ACTUAL = 3.75" ∴ C_Δ=1.0

MIN SPACING => TENSION + COMP. C_Δ=1.0 END = 4(0.875in) = 3.5"

ACTUAL = 3.5" ∴ C_Δ=1.0

GROUP ACTION FACTOR = 0.99 (PER TABLE 10.3.6.A)

WEB CONNECTION CAPACITY =>

TENSION = $Z'_{\parallel}$ = 1.15(3270lbs)(0.5)(0.99)(2) = 3.72 K

COMPRESSION = $Z'_{\parallel}$ = 1.15(3270lbs)(1.0)(0.99)(2) = 7.4 K

BOLT CAPACITY OF WEBS TO TRUSS CHORDS (CONT'D)

TRUSS TO ARCHED CHORD (CONT'D)

CHORD MEMBER CONNECTION CAPACITY:

EDGE DISTANCE => MIN. EDGE = 4(0.875in) = 3.5"

ACTUAL = 3.75" ∴ O.K.

END DISTANCE => $C_\Delta = 0.5$ = 2(0.875in) = 1.75in

$C_\Delta = 1.0$ = 4(0.875in) = 3.5 in

ACTUAL = LARGE

∴ $C_\Delta = 1.0$

ONLY USE (1) BOLT SINCE EDGE DISTANCE IS ONLY O.K. FOR TENSION OR COMP LOADS.

CHORD CONNECTION CAPACITY =>

TENSION OR COMP = $Z'_\perp$ = 1.15(1470 lbs)(1.0)(2 SIDES) = 3.38 K

∴ CONNECTION IS ADEQUATE FOR MOST MEMBERS. PROVIDE STEEL SADDLE CLAMPS AT JOINTS TO STRENGTHEN CONNECTION. THAT WAY THE EDGE DIST. REQUIREMENTS WILL NOT MATTER AND BOTH BOLTS CAN BE UTILIZED.

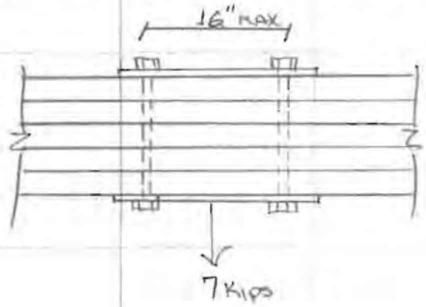
CAPACITY OF BOTH BOLTS =>

TENSION OR COMP. = $Z'_\perp$ = 1.15(1470 lbs)(0.99)(2 SIDES)(2 BOLTS) = 6.8K

∴ CONNECTION IS ADEQUATE AT ALL JOINTS.

CONNECTION SADDLE

* DESIGN SADDLE FOR 7 KIPS @ JOINTS N31 + N35 ON RISA



MIN PL THICKNESS:

$$M_U = \frac{7(16in)}{8} = 14\ K\text{-}in$$

$$Z_{REQ'D} = \frac{14\ K\text{-}in(1.67)}{50\ Ksi} = 0.47\ in^3$$

$$t_{min} = \sqrt{\frac{0.97\ in^3(4)}{3.5}} = 0.75\ in$$

∴ USE 3/4" PL SADDLE

@ REMAINING JOINTS, DESIGN SADDLE FOR 3.7 KIPS

MIN PL THICKNESS

$$M_U = \frac{3.7K(16in)}{8} = 7.4\ K\text{-}in$$

$$Z_{REQ'D} = \frac{7.4\ K\text{-}in(1.67)}{50\ Ksi} = 0.25\ in^3$$

$$t_{min} = \sqrt{\frac{0.25\ in^3(4)}{3.5}} = 0.5\ in$$

∴ USE 1/2" PL SADDLE

BOLT CAPACITY:

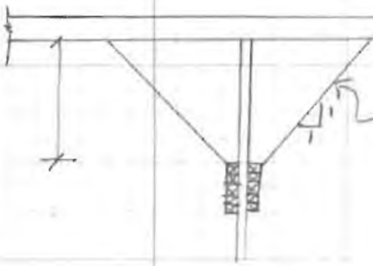
FIND MIN. BOLT DIA. => $\frac{3.5\ Kips(1.67)}{36\ Ksi} = 0.162\ in^2$ (TENSILE NET AREA)

∴ USE 5/8" Ø BOLTS @ JOINTS N31 + N35

@ REMAINING JOINTS, USE => $\frac{3.7\ Kips(1.67)}{2(36\ Ksi)} = 0.0858\ in^2$

∴ USE 1/2" Ø BOLTS

ARCHED CHORD BRACING



BRACE @ 10'o.c. APPROX.

LOAD IN TOP CHORD: MAX COMP. LOAD = 70 KIPS

REQ'D BRACE LOAD = 70K(0.004) = 280 lbs (HORIZ.)

LOAD IN BRACE = 1.41(280 lbs) = 392 lbs

MAX. BRACE LENGTH = 1.41(4.3) = 6.1'

USE 2x4 BRACE CSR = 0.372

BRACE ATTACHMENT TO CHORD AND JOIST:

16d BOX NAIL CAPACITY => W = 103 lbs Cd = 1.15

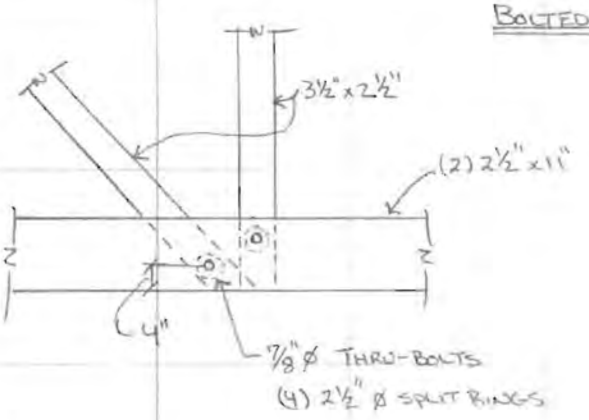
W' = 103(1.15) = 118 lbs

REQ'D NAILS = 373 lbs/118 = 3.2 ∴ 4 NAILS

SIMPSON SDS25312 SCREW G.F. 340(1.15) = 391 lbs ∴ (1) LAG NEEDED

USE SIMPSON SDS25312 SCREW

BOLTED CAPACITY OF WEBS TO TRUSS BOTTOM CHORDS



$C_g = 1.0$

$C_D = 1.15$

$C_t = 1.0$

$C_m = 1.0$

$C_d = 1.0$ (LAGS ARE NOT USED)

DIAG. MEMBER/BOTTOM CHORD CAPACITY

$P = 2730$ lb/SPLIT RING

$Q = 1940$ lb/SPLIT RING

GEOMETRY FACTOR:

DIAG MEMBER ⇒ $C_\Delta = 1.0$

BOTTOM CHORD ⇒ $C_\Delta = 1.0$

DIAG. MEMBER CONN. STRENGTH:

(2) SPLIT RINGS

$P' = 2{,}730\text{ lb}(2)(1.15)(1.0) = 6{,}279\text{ lbs}$

BOTTOM CHORD CONN. STRENGTH:

$P' = 2730\text{ lb}(2)(1.15)(1.0) = 6{,}279\text{ lbs}$

$Q' = 1{,}940\text{ lb}(2)(1.15) = 4{,}462\text{ lbs}$

$@45° = N' = \dfrac{6{,}279\text{ lbs}(4{,}462\text{ lb})}{6{,}279\text{ lb}\sin^2(45°) + 4{,}462\text{ lbs}\cos^2(45°)} = 5{,}217\text{ lbs}$ ✓ CONTROLS

WORST CASE LOAD IS 3.7 KIPS (SEE RISA) ∴ CONNECTION STRENGTH IS ADEQUATE.

BOLTED CAPACITY OF WEBS TO BOTTOM CHORD (CONT'D)

VERT MEMBER/BOTTOM CHORD CAPACITY

P = 2730 lb/SPLIT RING

Q = 1940 lb/SPLIT RING

GEOMETRY FACTOR:

VERT MEMBER ⇒ $C_\Delta = 1.0$

BOTTOM CHORD ⇒ $C_\Delta = 1.0$

VERT. MEMBER CONN. STRENGTH:

$P' = 2{,}730\ lb(2)(1.15)(1.0) = 6{,}279\ lbs$

BOTTOM CHORD CONN. STRENGTH:

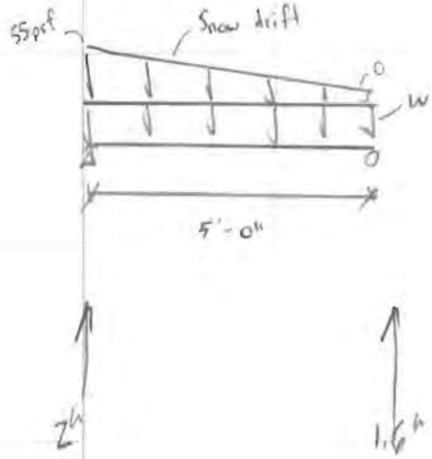
$Q' = 1{,}940\ lb(2)(1.15)(1.0) = 4{,}462\ lbs.$ ✓ CONTROLS

WORSTE CASE LOAD IS 3.6 Kips
(SEE RISA)

∴ CONNECTION STRENGTH IS ADEQUATE

Top chord where snow drift occurs

(E) top chord = (2) 5½ × 2½ spanning 5'-0"



W
DL: 15psf (20') = 300plf
SL: 30psf (20') = 600plf

See TEDDS attached

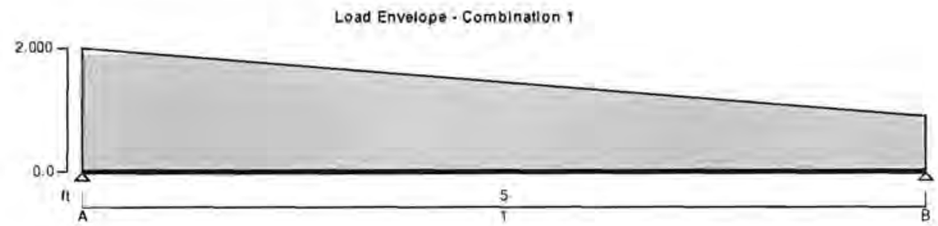
Use (2) 2x6 DF #2 sisters

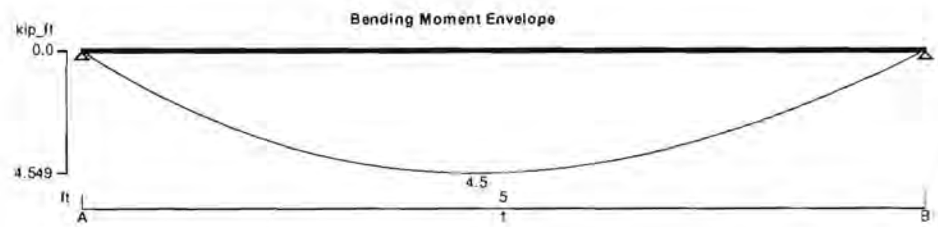
Tedds ARW Engineers 1594 W Park Circle Ogden, UT 84404	Project OWATC Bldg 10A				Job Ref. 15118 44	
	Section Top Chord with snow drift				Sheet no./rev. 1	
	Calc. by AJH	Date 6/11/2015	Chk'd by	Date	App'd by	Date

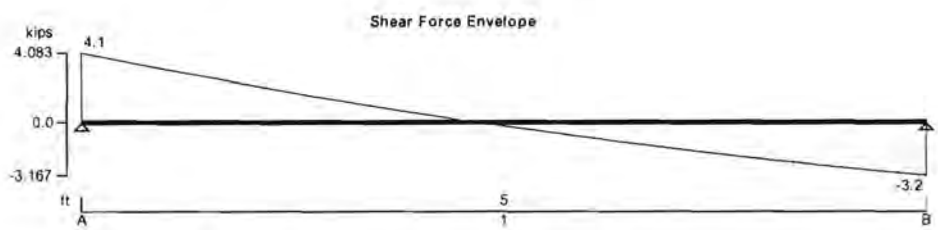
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2012 using the ASD method

TEDDS calculation version 1.6.03







Applied loading

Beam loads

	Dead full UDL 300 lb/ft
	Snow full UDL 600 lb/ft
snow drift	Snow partial VDL 1100 lb/ft at 0.00 in to 0 lb/ft at 60.00 in

Load combinations

Load combination 1	Support A	Dead × 1.00
		Snow × 1.00
	Span 1	Dead × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Snow × 1.00

Tedds ARW Engineers 1594 W Park Circle Ogden, UT 84404	Project OWATC Bldg 10A				Job Ref. 15118 45	
	Section Top Chord with snow drift				Sheet no./rev. 2	
	Calc. by AJH	Date 6/11/2015	Chk'd by	Date	App'd by	Date

Analysis results

Maximum moment M_{max} = **4549** lb_ft M_{min} = **0** lb_ft

Design moment $M = max(abs(M_{max}),abs(M_{min}))$ = **4549** lb_ft

Maximum shear F_{max} = **4083** lb F_{min} = **-3167** lb

Design shear $F = max(abs(F_{max}),abs(F_{min}))$ = **4083** lb

Total load on member W_{tot} = **7250** lb

Reaction at support A R_{A_max} = **4083** lb R_{A_min} = **4083** lb

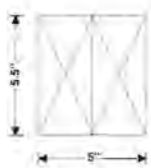
Unfactored dead load reaction at support A R_{A_Dead} = **750** lb

Unfactored snow load reaction at support A R_{A_Snow} = **3333** lb

Reaction at support B R_{B_max} = **3167** lb R_{B_min} = **3167** lb

Unfactored dead load reaction at support B R_{B_Dead} = **750** lb

Unfactored snow load reaction at support B R_{B_Snow} = **2417** lb



Sawn lumber section details

Nominal breadth of sections b_{nom} = **3** in

Dressed breadth of sections b = **2.5** in

Nominal depth of sections d_{nom} = **6** in

Dressed depth of sections d = **5.5** in

Number of sections in member N = **2**

Overall breadth of member $b_b = N \times b$ = **5** in

Species, grade and size classification Douglas Fir-Larch, No.1 & Btr grade, 2" & wider

Bending parallel to grain F_b = **1200** lb/in^2

Tension parallel to grain F_t = **800** lb/in^2

Compression parallel to grain F_c = **1550** lb/in^2

Compression perpendicular to grain F_{c_perp} = **625** lb/in^2

Shear parallel to grain F_v = **180** lb/in^2

Modulus of elasticity E = **1800000** lb/in^2

Modulus of elasticity, stability calculations E_{min} = **660000** lb/in^2

Mean shear modulus $G_{def} = E / 16$ = **112500** lb/in^2

Member details

Service condition **Dry**

Length of bearing L_b = **0.5** in

Load duration **Two months**

Section properties

Cross sectional area of member $A = N \times b \times d$ = **27.50** in^2

Section modulus $S_x = N \times b \times d^2 / 6$ = **25.21** in^3

$S_y = d \times (N \times b)^2 / 6$ = **22.92** in^3

Second moment of area $I_x = N \times b \times d^3 / 12$ = **69.32** in^4

$I_y = d \times (N \times b)^3 / 12 = \mathbf{57.29}$ in^4

Adjustment factors

Load duration factor - Table 2.3.2 $C_D = \mathbf{1.15}$

Temperature factor - Table 2.3.3 $C_t = \mathbf{1.00}$

Size factor for bending - Table 4A $C_{Fb} = \mathbf{1.30}$

Size factor for tension - Table 4A $C_{Ft} = \mathbf{1.30}$

Size factor for compression - Table 4A $C_{Fc} = \mathbf{1.10}$

Flat use factor - Table 4A $C_{fu} = \mathbf{1.15}$

Incising factor for modulus of elasticity - Table 4.3.8

$C_{iE} = \mathbf{1.00}$

Incising factor for bending, shear, tension & compression - Table 4.3.8

$C_i = \mathbf{1.00}$

Incising factor for perpendicular compression - Table 4.3.8

$C_{ic\_perp} = \mathbf{1.00}$

Repetitive member factor - cl.4.3.9 $C_r = \mathbf{1.00}$

Bearing area factor - cl.3.10.4 $C_b = \mathbf{1.00}$

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom}) = \mathbf{1.00}$

- Beam is fully restrained

Beam stability factor - cl.3.3.3 $C_L = \mathbf{1.00}$

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b = \mathbf{625}$ lb/in^2

Applied compression stress perpendicular to grain $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b) = \mathbf{1633}$ lb/in^2

$f_{c\_perp} / F_{c\_perp}' = \mathbf{2.613}$

FAIL - Design compressive stress is less than applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = \mathbf{1794}$ lb/in^2

Actual bending stress $f_b = M / S_x = \mathbf{2166}$ lb/in^2

$f_b / F_b' = \mathbf{1.207}$

FAIL - Design bending stress is less than actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_t \times C_i = \mathbf{207}$ lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A) = \mathbf{223}$ lb/in^2

$f_v / F_v' = \mathbf{1.076}$

FAIL - Design shear stress is less than actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection $E' = E \times C_{ME} \times C_t \times C_{iE} = \mathbf{1800000}$ lb/in^2

Design deflection $\delta_{adm} = 0.0056 \times L_{s1} = \mathbf{0.336}$ in

Bending deflection $\delta_{b\_s1} = \mathbf{0.163}$ in

Shear deflection $\delta_{v\_s1} = \mathbf{0.021}$ in

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{0.185}$ in

$\delta_a / \delta_{adm} = \mathbf{0.550}$

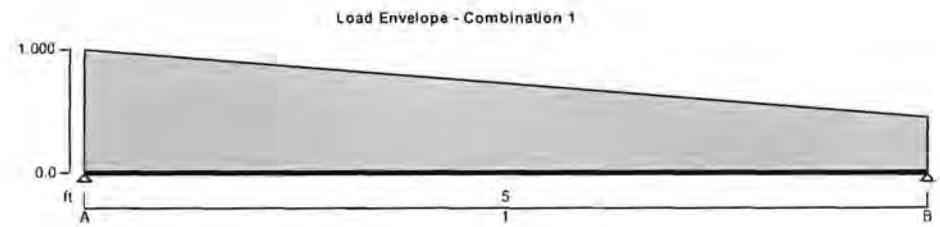
PASS - Design deflection is less than total deflection

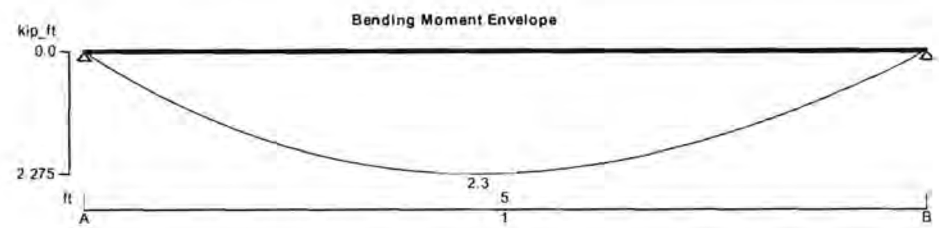
Tedds ARW Engineers 1594 W Park Circle Ogden, UT 84404	Project OWATC Bldg 10A				Job Ref. 15118	48
	Section Top Chord with snow drift UPGRADE				Sheet no./rev. 1	
	Calc. by AJH	Date 6/11/2015	Chk'd by	Date	App'd by	Date

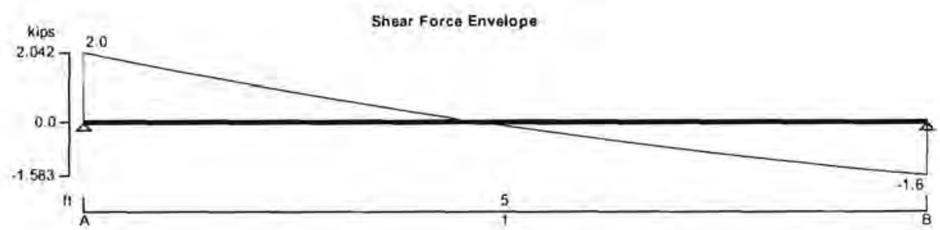
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2012 using the ASD method

TEDDS calculation version 1.6.03







Applied loading

Beam loads

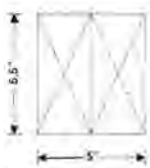
	Dead full UDL 300 lb/ft
	Snow full UDL 600 lb/ft
snow drift	Snow partial VDL 1100 lb/ft at 0.00 in to 0 lb/ft at 60.00 in

Load combinations

Load combination 1	Support A	Dead × 0.50
		Snow × 0.50
	Span 1	Dead × 0.50
		Snow × 0.50
	Support B	Dead × 0.50
		Snow × 0.50

Analysis results

Maximum moment	M_{max} = **2275** lb_ft	M_{min} = **0** lb_ft
Design moment	$M = \max(\text{abs}(M_{max}),\text{abs}(M_{min}))$ = **2275** lb_ft	
Maximum shear	F_{max} = **2042** lb	F_{min} = **-1583** lb
Design shear	$F = \max(\text{abs}(F_{max}),\text{abs}(F_{min}))$ = **2042** lb	
Total load on member	W_{tot} = **3625** lb	
Reaction at support A	R_{A_max} = **2042** lb	R_{A_min} = **2042** lb
Unfactored dead load reaction at support A	R_{A_Dead} = **750** lb	
Unfactored snow load reaction at support A	R_{A_Snow} = **3333** lb	
Reaction at support B	R_{B_max} = **1583** lb	R_{B_min} = **1583** lb
Unfactored dead load reaction at support B	R_{B_Dead} = **750** lb	
Unfactored snow load reaction at support B	R_{B_Snow} = **2417** lb	



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **3** in
Dressed breadth of sections	b = **2.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **2**
Overall breadth of member	$b_b = N \times b$ = **5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition	**Dry**
Length of bearing	L_b = **1.5** in
Load duration	**Two months**

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **27.50** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **25.21** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **22.92** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **69.32** in^4

$I_y = d \times (N \times b)^3 / 12 = \mathbf{57.29}$ in^4

Adjustment factors

Load duration factor - Table 2.3.2 $C_D = \mathbf{1.15}$

Temperature factor - Table 2.3.3 $C_t = \mathbf{1.00}$

Size factor for bending - Table 4A $C_{Fb} = \mathbf{1.30}$

Size factor for tension - Table 4A $C_{Ft} = \mathbf{1.30}$

Size factor for compression - Table 4A $C_{Fc} = \mathbf{1.10}$

Flat use factor - Table 4A $C_{fu} = \mathbf{1.15}$

Incising factor for modulus of elasticity - Table 4.3.8

$C_{iE} = \mathbf{1.00}$

Incising factor for bending, shear, tension & compression - Table 4.3.8

$C_i = \mathbf{1.00}$

Incising factor for perpendicular compression - Table 4.3.8

$C_{ic\_perp} = \mathbf{1.00}$

Repetitive member factor - cl.4.3.9 $C_r = \mathbf{1.00}$

Bearing area factor - cl.3.10.4 $C_b = \mathbf{1.00}$

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom}) = \mathbf{1.00}$

- Beam is fully restrained

Beam stability factor - cl.3.3.3 $C_L = \mathbf{1.00}$

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b = \mathbf{625}$ lb/in^2

Applied compression stress perpendicular to grain $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b) = \mathbf{272}$ lb/in^2

$f_{c\_perp} / F_{c\_perp}' = \mathbf{0.436}$

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = \mathbf{1346}$ lb/in^2

Actual bending stress $f_b = M / S_x = \mathbf{1083}$ lb/in^2

$f_b / F_b' = \mathbf{0.805}$

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_t \times C_i = \mathbf{207}$ lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A) = \mathbf{111}$ lb/in^2

$f_v / F_v' = \mathbf{0.538}$

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection $E' = E \times C_{ME} \times C_t \times C_{iE} = \mathbf{1600000}$ lb/in^2

Design deflection $\delta_{adm} = 0.0056 \times L_{s1} = \mathbf{0.336}$ in

Bending deflection $\delta_{b\_s1} = \mathbf{0.184}$ in

Shear deflection $\delta_{v\_s1} = \mathbf{0.024}$ in

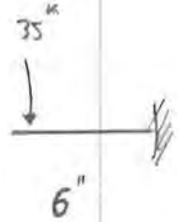
Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{0.208}$ in

$\delta_a / \delta_{adm} = \mathbf{0.618}$

PASS - Design deflection is less than total deflection

Plate bending check for new saddle for new tension rods @ bottom chord

worst case load : 35^k

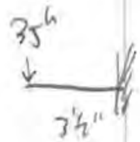


$M = 35^k(6") = 210^{k \cdot in}$

$\sigma = \frac{M\ell}{S} \Rightarrow \quad S_{req'd} = \frac{M\,\Omega}{\sigma} = \frac{(210^{k \cdot in})(1.67)}{36\,ksi} = 9.7\ in^3$

try 3/4" thick $\Rightarrow \quad S : \frac{bh^2}{6} = \frac{(12")(.75)^2}{6} = 1.125\ in^3$

$\therefore$ stiffeners req'd



$M: 35^k(3.5") : 123^{k \cdot in}$

$S_{req'd} = 5.7\ in^3$

weld check for rod to PL on detail 1/52.2 $\quad 70^k/4 = 17.5^k$

$l_{req'd} = \frac{17.5^k}{4(.928)} : 4.7"$

Bottom chord upgrade

$E_{wood} = 1{,}800{,}000$ psi (DF #1 & btr)

$E_{steel} = 29{,}000{,}000$ psi

Loads in BC. axial tension

DL: 24^k } from RISA model

TL: 69^k

Length of B.C. = 60 ft ⇒ 720 inches

Area of B.C. = (2) 2.5×11" ⇒ $A = 55 \text{ in}^2$

Strains

Wood: $\varepsilon_{w_{DL}} = \dfrac{\delta}{L}$

$$\delta_{DL} = \frac{PL}{AE} \Rightarrow \frac{(24^k)(720")}{(55 \text{ in}^2)(1{,}800)} \Rightarrow \delta_{DL} = 0.17"$$

$$\varepsilon_{w_{DL}} = \frac{0.17"}{720"} = 2.4 \times 10^{-4}$$

Find allowable strain in wood:

F_t = 800 psi

$$F_t = 800\text{psi}(55 \text{ in}^2)(\overset{C_D}{1.15})(\overset{C_F}{1.1}) = 55.7 \text{ kip allowable load}$$

$$\delta_{w_{allow}} = \frac{55.7^k (720")}{(55 \text{ in}^2)(1800 \text{ ksi})} = 0.41 \text{ in}$$

$$\varepsilon_{w_{allow}} = \frac{\delta_{w_{allow}}}{L} = \frac{0.41 \text{ in}}{720 \text{ in}} = 5.6 \times 10^{-4}$$

Bottom Chord Upgrade cont

$$A_{steel} = (4)\,\pi\left(\frac{7/8''}{2}\right)^2 = 2.41\,in^2$$

Strains

Steel:

$$\varepsilon_{S_{req'd}} = \frac{\delta}{L} \qquad \delta_{req'd} = \frac{PL}{AE} = \frac{(60^k)\,720''}{(2.41\,in^2)(29000)} = 0.62\,in$$

$$\varepsilon_{S_{req'd}} = \frac{0.62\,in}{720\,in} = 8.62\times10^{-4}$$

$$\varepsilon_{S_{req'd}} - \varepsilon_{w_{allow}} = \varepsilon_{pre\ tension}$$

$$\varepsilon_{pretension} = 8.62\times10^{-4} - 5.6\times10^{-4} = 3.0\times10^{-4}$$

$$\delta_{pretension} = \overset{\varepsilon}{(3.0\times10^{-4})}\,\overset{L}{(720\,in)} = 0.217\,in$$

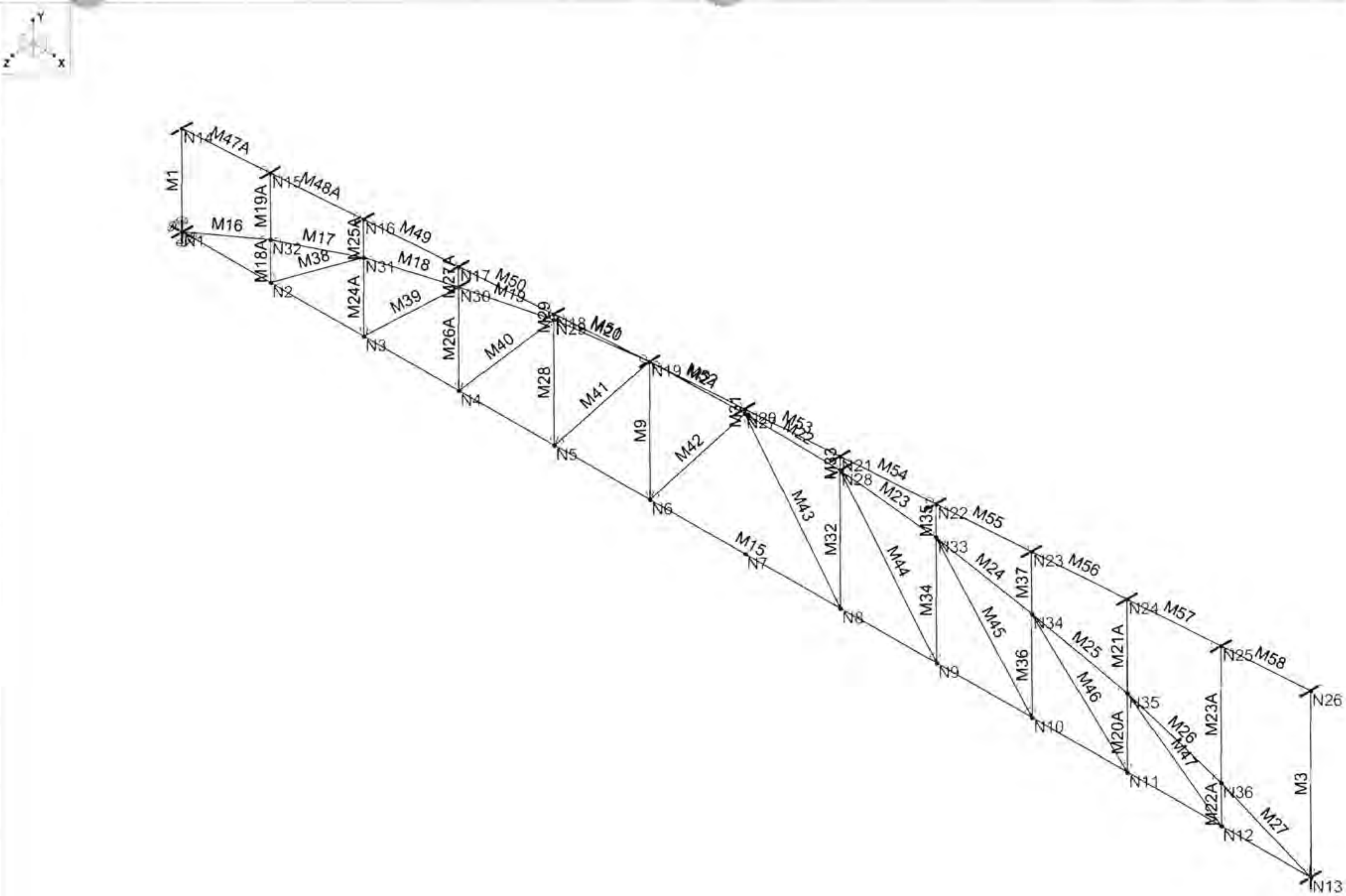
$$P_{pretension} = \frac{\delta_{pretension}\,AE}{L} = \frac{(0.217\,in)(2.41\,in^2)(29000\,ksi)}{720\,in}$$

$$P_{pretension} = 21.1\ kips$$

$$/\,(4)\ rods = 5.3\ kips$$

say 6 kips per rod

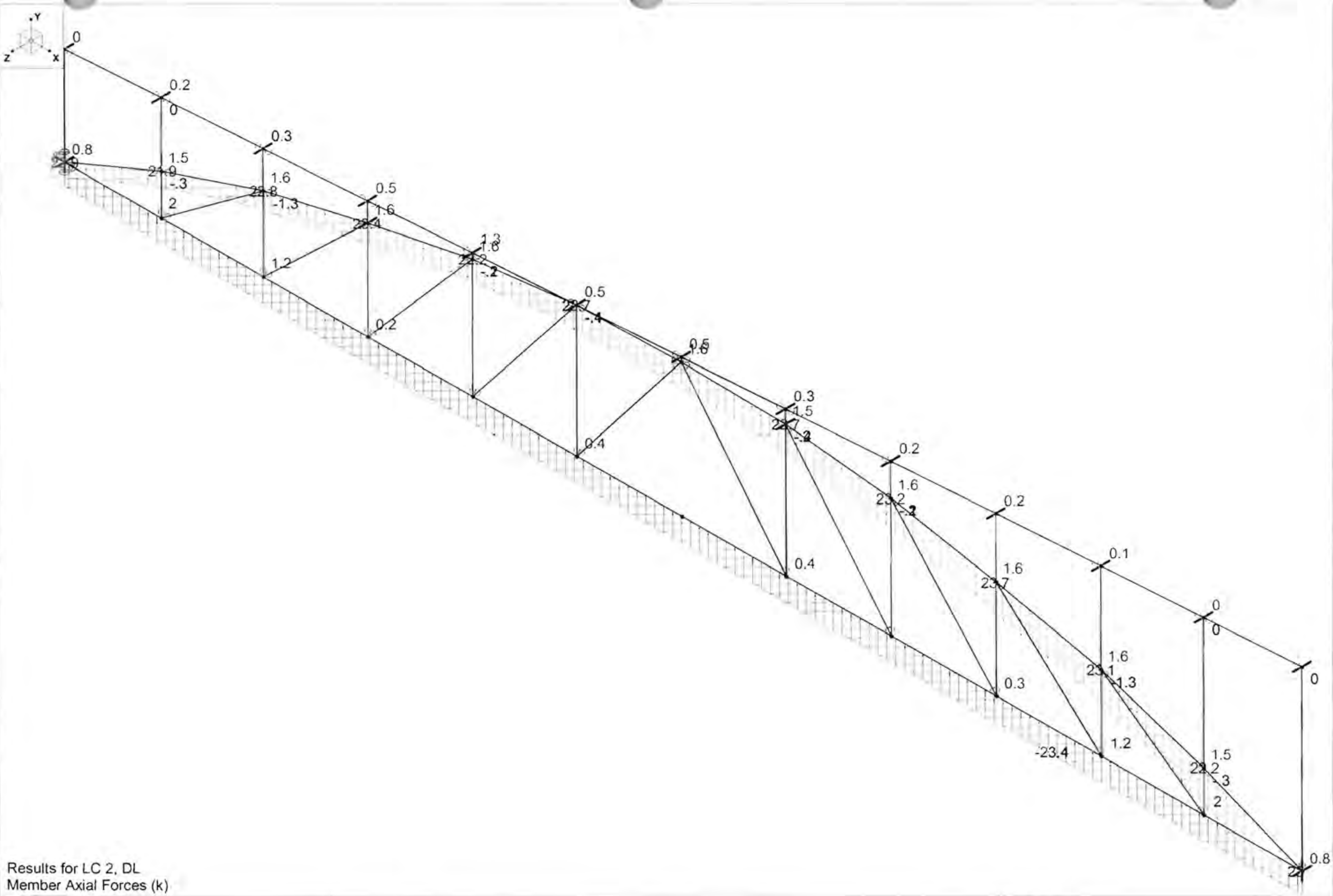
21.1 kips < 24 kips ∴ ok, Bottom chord wood members still in axial tension.



SK - 1

July 20, 2015 at 2:27 PM

Bow String Truss Model Pinned Arch.r3d

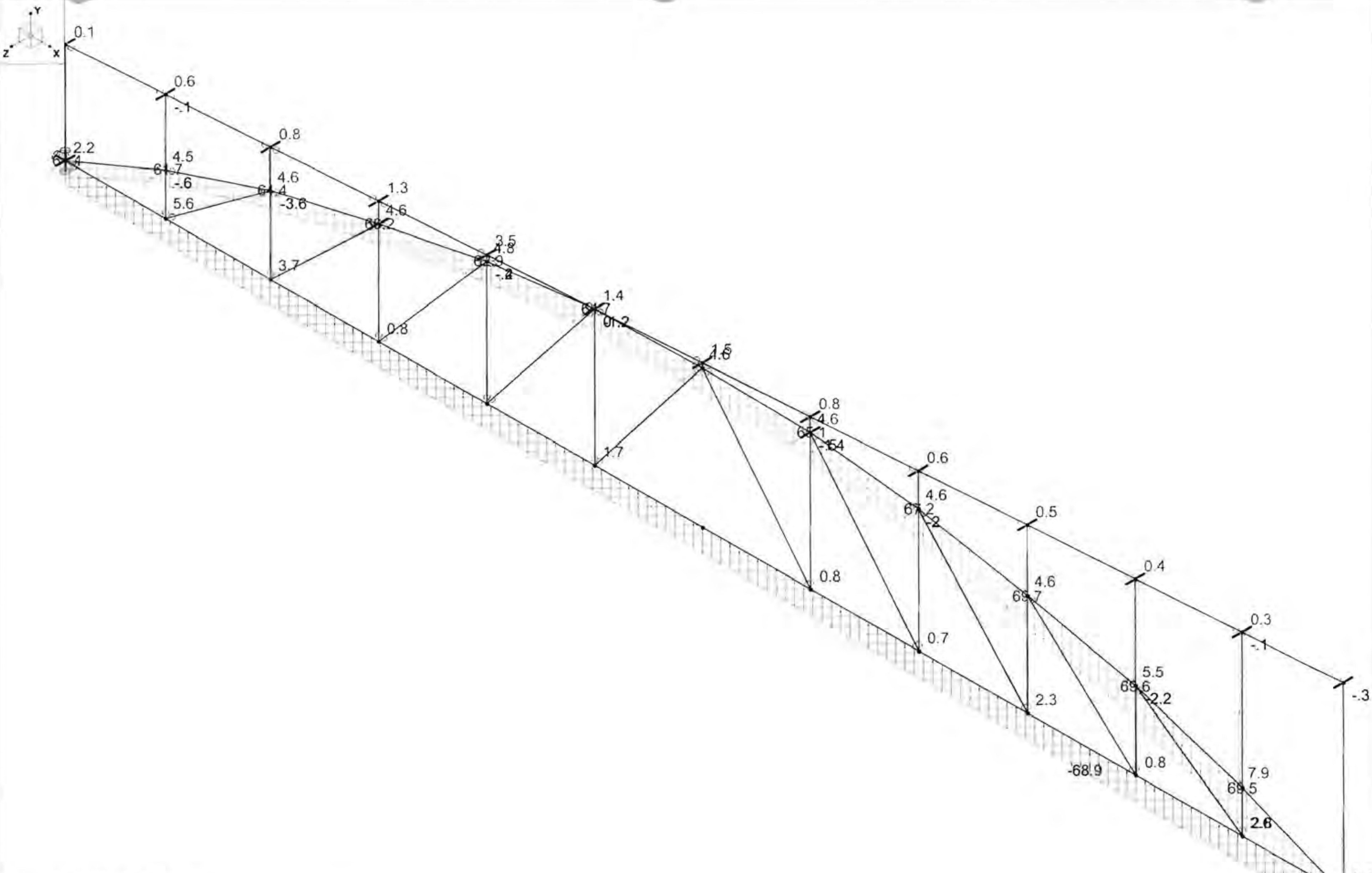


Results for LC 2, DL
Member Axial Forces (k)

SK - 2

July 20, 2015 at 2:28 PM

Bow String Truss Model Pinned Arch.r3d



Results for LC 3, DL + SL
Member Axial Forces (k)

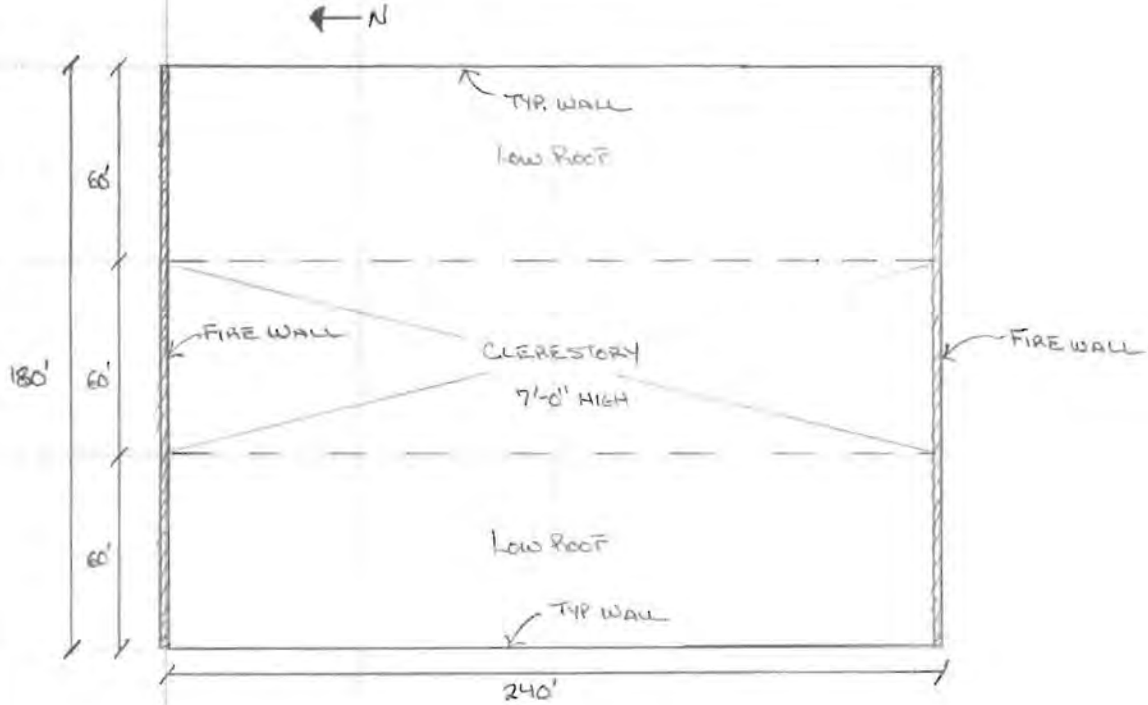
SK - 3

July 20, 2015 at 2:29 PM

Bow String Truss Model Pinned Arch.r3d

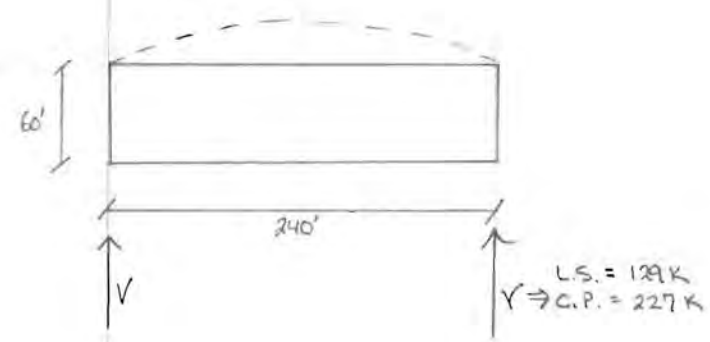
LATERAL ANALYSIS

LATERAL ANALYSIS



NOTE: LATERAL ANALYSIS WILL BE BY ASCE 41-06 W/ APPLICABLE UPGRADES

EAST-WEST DIRECTION



USE 16d TO GET 1½" PENETRATION INTO FRAMING

USE 15/32" OSB OVERLAY W/ 16d NAILS @ 4" o.c.

SEE ATTACHED CALCS

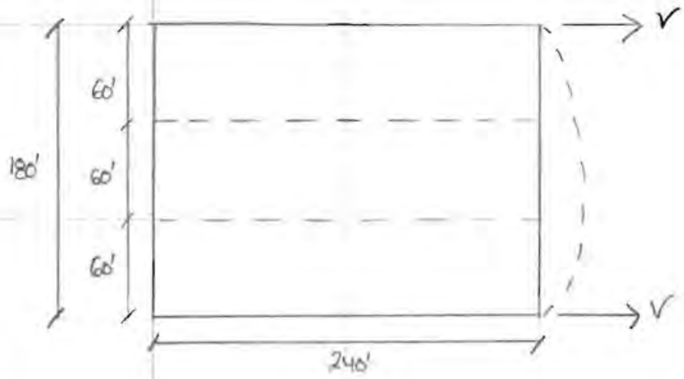
ARW ENGINEERS
structural consultants

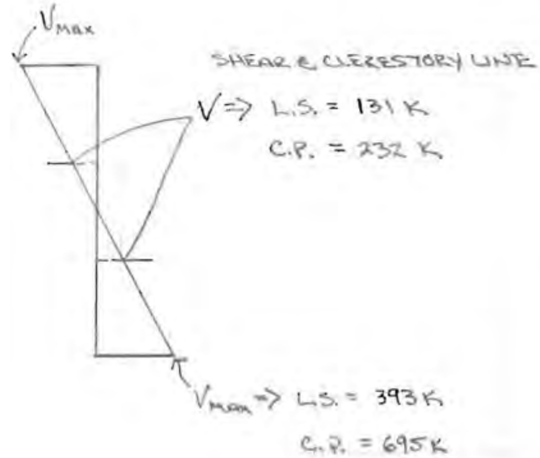
Project No. 15118 Sheet No.

Project OWATC BLDG 10A
Prepared By ZCH Date

LATERAL ANALYSIS (CONT'D)

NORTH - SOUTH DIRECTION



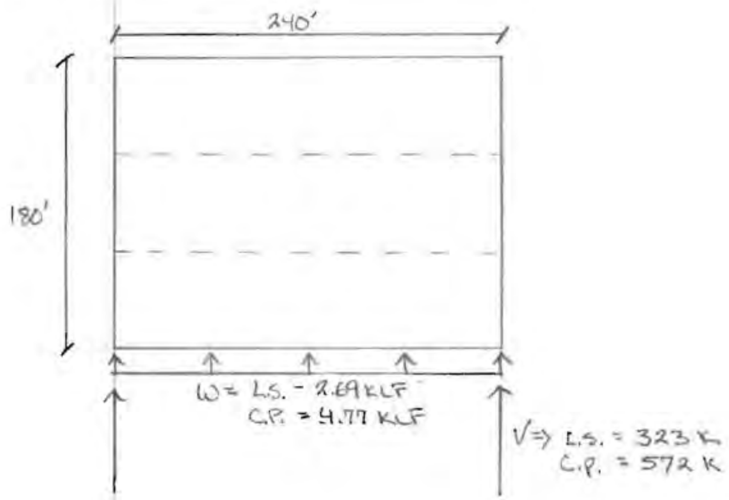


USE 15/32" OSB OVERLAY W/ 16d NAILS @ 4" o.c.

(SEE ATTACHED CALCS)

DIAPH. CHORD FORCES

EAST - WEST DIRECTION:



FORCE-CONTROLLED ACTION

CHORD FORCE => (DEFORMATION CONTROLLED)

$$L.S. = \frac{2.69\,KLF(240)^2}{8(180')} = 108\ K$$

$$C.P. = \frac{4.77\,KLF(240)^2}{8(180)} = 190\ K$$

$$Q_{UF} = \quad L.S. = \frac{108K}{1.05(1.77)} = 58\ K$$

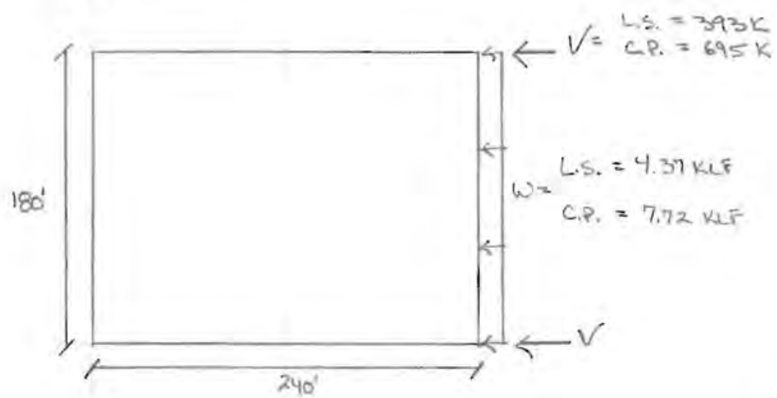
$$C.P. = \frac{190\ K}{1.23(1.07)(2.67)} = 57\ K$$

$\therefore$ CHORD FORCE $Q_{UF} = 58K$

LATERAL ANALYSIS (CONT'D)

DIAPH. CHORD FORCES (CONT'D)

N-S DIRECTION: (OVERALL)



FORCE CONTROLLED ACTION

CHORD FORCE ⇒ (DEFORMATION CONTROLLED) $L.S. = \frac{4.37(180)^2}{8(240)} = 74K$ $Q_{UF} = L.S. = \frac{74K}{1.05(1.77)} = 40K$

$C.P. = \frac{7.72(180)^2}{8(240)} = 130K$ $C.P. = \frac{130K}{1.23(1.01)(2.61)} = 39K$

CHORD FORCE $Q_{UF} = 40K$

CHECK CAPACITY OF 3x6 TOP PL:

$F_t = 800\,psi$ $F'_t = 800(1.3)(2.16) = 2{,}246\,psi$

$C_F = 1.3$

$K_F = 2.16$ EXPECTED STRENGTH = $2{,}246\,psi(1.5) = 3{,}370\,psi$

$\lambda = 1.0$

$A = 13.75\,in^2$ $T_U = 3{,}370\,psi(13.75\,in^2) = 46.3K$

$L.S. = mT_U = 46.3K(2.5) = 116K > 108K$ ∴ GOOD

$C.P. = mT_U = 46.3K(3.0) = 140K < 190K$ ∴ NO GOOD TRY STRAPS

FIND CAPACITY OF SIMPSON CMSTI2:

EXPECTED STRENGTH → $T_U = 14{,}625\,lbs$

$L.S. = mT_U = 6(14.6K) = 88K$

$C.P. = mT_U = 8(14.6K) = 117K$

∴ USE (2) CMSTI2 STRAPS @ EA. SIDE

ARW ENGINEERS structural consultants

Project No. 15118 Sheet No. 61

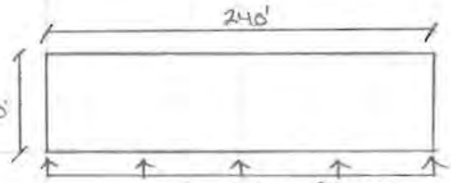
Project OWATC BLDG 10A

Prepared By ZCH Date

LATERAL ANALYSIS (CONT'D)

DIAPH CHORD FORCES (CONT'D)

CLERESTORY CHORDS:



w = L.S. = .98 KLF

C.P. = 1.73 KLF

WEIGHT

ROOF = 15 psf(60')(240') = 216 K

WALLS = 12 psf(3.5')(240')(2) = 20.2 K

TOTAL W = 236 K

L.S. = 236 K(1.0) = 236 K

C.P. = 236 K(1.76) = 415 K

CHORD FORCE ⇒ $L.S. = \frac{.98(240)^2}{8(60)} = 118\ K$

$C.P. = \frac{1.73(240)^2}{8(60)} = 208\ K$

∴ USE (2) CMSTI2 STRAPS

27-Apr-15

ENGINEERS

ASCE 41-06 PSUEDO LATERAL FORCE 1^{ST} ITERATION

Version Date: July 25, 2012

JOB TITLE: OWATC BDO 10A Upgrade
BUILDING LOCATION: BDO Ogden, UT

JOB #: 15118
PREPARED BY: ZCH

ASCE 41-06 Psuedo Lateral Force

Building Period (ASCE 41-06 Section 3.3.1.2)

Building Height (H): 27.0 ft
Period Coefficient (Ct): 0.02
Height Coefficient (β): 0.75

Bldg. Period (T): 0.24 sec

Mapped Acceleration Factors (ASCE 41-06 Section 1.6.1)

	BSE-1 (2/3*MCE)		BSE-2 (MCE)	
	S_s =	1.42	S_s =	1.42
	S_1 =	0.50	S_1 =	0.50
Soil Factors	F_a =	1.00	F_a =	1.00
	F_v =	1.50	F_v =	1.50
	Scale Factor =	0.67	Scale Factor =	1.00

Base Shear Coefficient (BSE-1 (2/3*MCE) - ASCE 41-06 Section 3.3.1)

Response Acceleration Parameters: (ASCE 41-06 Section 1.6.1 & Section 1.6.2)

S_{XS} = Scale Factor * S_s * F_V = 0.95

S_{X1} = Scale Factor * S_1 * F_V = 0.50

$$T_s = \frac{S_{x1}}{S_{xs}}$$ T_s = 0.52 sec

T_0 = ... To = 0.10 sec

Effective viscous damping ratio (Section 1.6.1.5.1) β = 0.05

$$B_1 = \frac{4}{5.6 - \ln(100 * \beta)}$$ B_1 = 1.00

S_a = 0.95

BSE-1 (2/3*MCE) Psuedo Lateral Force - Section 3.3.1.3.1

Modification Factors:

DCR_{max} = 1.00

Effective mass factor C_m = 1 (ASCE 41-06 Table 3-1)
Site Class Factor a = 60 (Section 3.3.1.3)

$$R = \frac{DCR_{max}}{1.5} * C_m \geq 1.0$$ R = 1.00

$$C_1 = \begin{cases} 1.0 & T > 1.0 \\ 1 + \left(\frac{R-1}{aT^2}\right) & T \leq 1.0 \end{cases}$$ C_1 = 1.00

$$C_2 = \begin{cases} 1.0 & T > 0.7 \\ 1 + \frac{1}{800}\left(\frac{R-1}{T}\right)^2 & T \leq 0.7 \end{cases}$$ C_2 = 1.00

BSE-1 (2/3*MCE) Psuedo Lateral Force Coefficient $V = C_1 * C_2 * C_m * S_a * W$ V = 0.95 x W

ENGINEERS

ASCE 41-06 PSUEDO LATERAL FORCE (CONT'D)

Version Date: July 25, 2012

27-Apr-15
1:39 PM

JOB TITLE: OWATC BDO 10A Upgrade
BUILDING LOCATION: BDO Ogden, UT

JOB #: 15118
PREPARED BY: ZCH

Base Shear Coefficient (BSE-2 (MCE) - ASCE 41-06 Section 3.3.1)

Response Acceleration Parameters: (ASCE 41-06 Section 1.6.1 & Section 1.6.2)

S_{XS} = Scale Factor * S_1 * F_V = 1.42

S_{X1} = Scale Factor * S_1 * F_V = 0.74

$$T_s = \frac{S_{x1}}{S}$$ T_s = 0.52 sec

$$T_o = 0.2 * T_s$$ To = 0.10 sec

Effective viscous damping ratio (Section 1.6.1.5.1) β = 0.05

$$B_1 = \frac{4}{5.6 - \ln(100 * \beta)}$$ B_1 = 1.00

See Section 1.6.1.5 for Sa calculation) S_a = 1.42

BSE-2 (MCE) Psuedo Lateral Force - Section 3.3.1.3.1

Modification Factors:

DCR_{max} = 1.00

Effective mass factor (Table 3-1) C_m = 1 (ASCE 41-06 Table 3-1)
Site Class Factor a = 60 (Section 3.3.1.3)

$$R = \frac{DCR_{max}}{1.5} * C_m \geq 1.0$$ R = 1.00

$$C_1 = \begin{cases} 1.0 & T > 1.0 \\ 1 + \left(\frac{R-1}{aT^2}\right) & T \leq 1.0 \end{cases}$$ C_1 = 1.00

$$C_2 = \begin{cases} 1.0 & T > 0.7 \\ 1 + \frac{1}{800}\left(\frac{R-1}{T}\right)^2 & T \leq 0.7 \end{cases}$$ C_2 = 1.00

BSE-2 (MCE) Psuedo Lateral Force Coefficient

$$V = C_1 * C_2 * C_m * S_a * W$$ V = 1.42 x W

ENGINEERS

ASCE 41-06 DIAPHRAGM ANALYSIS / DESIGN

27-Apr-15
2:27 PM

Version Date: September 3, 2014 Author: Scott Porter Reviewed by: Troy M. Dye, SE

JOB TITLE: QWATC BDO 10A Upgrade
BUILDING LOCATION: BDO Ogden, UT
DIAPHRAGM AREA: Typical Diaphragm in the East West Direction

JOB #: 15118
PREPARED BY: ZCH

ASCE 41-06 - Roof Diaphragm Design / Analysis

Roof Weights: Design Mode: Upgrade Component Analysis/Upgrade Factor: **1.00**

Roof Dead Load= 15 psf
Roof N-S Dimension = 240 ft
Roof E-W Dimension = 60 ft
Roof Seismic Weight = 15 psf

Flat Roof Snow Load = 30 psf
Roof Snow Load = 30 psf
Total Roof Weight = 216000 lbs

Tributary Wall Weights - Earthquake in N-S Direction

	Wall Mark	Wall Weight (psf)	Tributary Wall Length (ft)	Tributary Wall Height (ft)	Total Wall Weight (lbs)
	1				0
	2				0
	3				0
	4				0
	5				0
	6				0
	7				0
	8				0
	9				0
	10				0

Total Wall Weights N-S Direction = 0 lbs

Total Seismic Weight (TSW) N-S Direction = **216000** lbs

Tributary Wall Weights - Earthquake in E-W Direction

	Wall Mark	Wall Weight (psf)	Tributary Wall Length (ft)	Tributary Wall Height (ft)	Total Wall Weight (lbs)
	1	12	240	11	31680
	2	12	240	3.5	10080
	3				0
	4				0
	5				0
	6				0
	7				0
	8				0
	9				0
	10				0

Total Wall Weights E-W Direction = 41760 lbs

Total Seismic Weight (TSM) E-W Direction = **257760** lbs

Diaphragm Design/Analysis N-S Direction

	Life Safety		Collapse Prevention	
Performance Level	Life Safety		Collapse Prevention	
Seismic Hazard Level	BSE-1		BSE-2	
C_1=	1		1	
C_2=	1		1	
C_m=	1		1	
S_a=	0.95		1.42	
Pseudo Lateral Force Factor: $C_1C_2C_mS_a$=	0.95		1.42	
Pseudo Lateral Force: V_{NS}=	205200	lbs	306720	lbs

$V = C_1C_2C_mS_a \times TSW$

Length of Diaphragm Boundary: L= ft

Shear Force at Boundary Edge:

NOT APPLICABLE. SEE FOLLOWING SHEET.

Q_{UD_NS}= #DIV/0! lb/ft Q_{UD_NS}= #DIV/0! lb/ft

Diaphragm Capacity: Chorded? (Y/N): Y

$v_{s\ nominal}$ = lb/ft (LRFD nominal capacity w/ Φ=1.0 from NDS tables. ASCE 41-06 C8.5.8.2 states that this is equivalent to 2.0*ASD values.)
$v_{s\ expected}$ = 0 lb/ft

(Enter Diaphragm Thickness and Nailing)

m_{LS} =

m_{CP} =

(Factor obtained from ASCE 41-06 Table 8-3. Factor is based on diaphragm Length/Width ratio, diaphragm type, chorded/unchorded, blocked/unblocked)

k = 0.75 (Knowledge Factor, see ASCE 41-06 Section 2.2.6.4)

$m_{LS}*k*Q_{CE\_NS}$= 0 lb/ft $m_{CP}*k*Q_{CE\_NS}$= 0 lb/ft

Demand-to-Capacity Ratio (DCR): (If DCR values are above 1.5 psuedo force for entire building will increase.)

	Life Safety	Collapse Prevention
DCR_{NS}=	#DIV/0!	#DIV/0!
1.00*Qud/(mkQce) =	#DIV/0!	#DIV/0!
Status	#DIV/0!	#DIV/0!

ENGINEERS

ASCE 41-06 DIAPHRAGM ANALYSIS/DESIGN
Version Date September 3, 2014

27-Apr-15
2:27 PM

JOB TITLE: OWATC BDO 10A Upgrade
BUILDING LOCATION: BDO Ogden, UT
DIAPHRAGM AREA: Typical Diaphragm in the East West Direction

JOB #: 15118
PREPARED BY: ZCH

ASCE 41-06 - Roof Diaphragm Design/Analysis (Cont'd)

Diaphragm Design/Analysis E-W Direction

		BSE-1			BSE-2		
	Hazard Level	BSE-1		Hazard Level	BSE-2		
Pseudo Lateral Force Factor:	$C_1C_2C_mS_a$=	**0.95**		$C_1C_2C_mS_a$=	**1.42**		
Pseudo Lateral Force:	V_{EW}=	**244872**	lbs	V_{EW}=	**366019**	lbs	$V = C_1C_2C_mS_a$ x TSW
Length of Diaphragm Boundary:	L=	**60**	ft				
Shear Force at Boundary Edge:							
	Q_{UD_EW}=	**2041**	lb/ft	Q_{UD_EW}=	**3050**	lb/ft	

Diaphragm Capacity: Chorded? (Y/N): Y

$\nu_{s\ nominal}$ = 770 lb/ft (LRFD nominal capacity w/ Φ=1.0 from NDS tables. ASCE 41-06 C8.5.8.2 states that this is equivalent to 2.0*ASD values.)
$\nu_{s\ expected}$ = 1155 lb/ft

15/32" Overlay w/ 10d nails @ 4"o.c.

m_{LS} = 2.5
m_{CP} = 3
(Factor obtained from ASCE 41-06 Table 8-3. Factor is based on diaphragm Length/Width ratio, diaphragm type, chorded/unchorded, blocked/unblocked)

k = 1 (Knowledge Factor, see ASCE 41-06 Section 2.2.6.4)

m_{LS}*k*Q_{CE_EW}= **2887.5** lb/ft m_{CP}*k*Q_{CE_EW}= **3465** lb/ft

Demand-to-Capacity Ratio (DCR): (If DCR values are above 1.5 psuedo force for entire building will increase.)

	BSE-1		BSE-2
DCR_{EW}=	**1.77**	DCR_{EW}=	**2.64**
1.00*Qud/(mkQce) =	0.71	1.00*Qud/(mkQce) =	0.88
Status	Okay	Status	Okay

ENGINEERS

ASCE 41-06 DIAPHRAGM ANALYSIS / DESIGN

Version Date: September 3, 2014 Author: Scott Porter Reviewed by: Troy M. Dye, SE

27-Apr-15
2:27 PM

JOB TITLE: OWATC BDO 10A Upgrade
BUILDING LOCATION: BDO Ogden, UT
DIAPHRAGM AREA: Typical Diaphragm in the North South Direction

JOB #: 15118
PREPARED BY: ZCH

ASCE 41-06 - Roof Diaphragm Design / Analysis

Roof Weights: Design Mode: Upgrade Component Analysis/Upgrade Factor: 1.00

Roof Dead Load= 15 psf
Roof N-S Dimension = 240 ft
Roof E-W Dimension = 180 ft
Roof Seismic Weight = 15 psf

Flat Roof Snow Load = 30 psf
Roof Snow Load = 30 psf
Total Roof Weight = 648000 lbs

Tributary Wall Weights - Earthquake in N-S Direction

	Wall Mark	Wall Weight (psf)	Tributary Wall Length (ft)	Tributary Wall Height (ft)	Total Wall Weight (lbs)
	1	26	180	15	70200
	2	26	180	15	70200
	3				0
	4				0
	5				0
	6				0
	7				0
	8				0
	9				0
	10				0

Total Wall Weights N-S Direction = 140400 lbs

Total Seismic Weight (TSW) N-S Direction = **788400** lbs

Tributary Wall Weights - Earthquake in E-W Direction

	Wall Mark	Wall Weight (psf)	Tributary Wall Length (ft)	Tributary Wall Height (ft)	Total Wall Weight (lbs)
	1				0
	2				0
	3				0
	4				0
	5				0
	6				0
	7				0
	8				0
	9				0
	10				0

Total Wall Weights E-W Direction = 0 lbs

Total Seismic Weight (TSM) E-W Direction = **648000** lbs

Diaphragm Design/Analysis N-S Direction

Performance Level		Life Safety		Collapse Prevention
Seismic Hazard Level		BSE-1		BSE-2
	C_1=	1	C_1=	1
	C_2=	1	C_2=	1
	C_m=	1	C_m=	1
	S_a=	0.95	S_a=	1.42
Pseudo Lateral Force Factor:	$C_1C_2C_mS_a$=	0.95	$C_1C_2C_mS_a$=	1.42
Pseudo Lateral Force:	V_{NS}=	**748980** lbs	V_{NS}=	**1119528** lbs

$V = C_1C_2C_mS_a \times TSW$

Length of Diaphragm Boundary: L= 240 ft

Shear Force at Boundary Edge:

Q_{UD_NS}= **1560** lb/ft Q_{UD_NS}= **2332** lb/ft

Diaphragm Capacity: Chorded? (Y/N): Y

$v_{s\ nominal}$ = 770 lb/ft (LRFD nominal capacity w/ Φ=1.0 from NDS tables. ASCE 41-06 C8.5.8.2 states that this is equivalent to 2.0*ASD values.)
$v_{s\ expected}$ = 1155 lb/ft

15/32" Overlay w/ 10d nails @ 4"o.c.

m_{LS} = 2.5
(Factor obtained from ASCE 41-06 Table 8-3. Factor is based on diaphragm Length/Width ratio, diaphragm type, chorded/unchorded, blocked/unblocked)
m_{CP} = 3

k = 1 (Knowledge Factor, see ASCE 41-06 Section 2.2.6.4)

$m_{LS}*k*Q_{CE\_NS}$= **2887.5** lb/ft $m_{CP}*k*Q_{CE\_NS}$= **3465** lb/ft

Demand-to-Capacity Ratio (DCR): (If DCR values are above 1.5 psuedo force for entire building will increase.)

DCR_{NS}=	1.35	DCR_{NS}=	2.02
1.00*Qud/(mkQce) =	0.54	1.00*Qud/(mkQce) =	0.67
Status	Okay	Status	Okay

2ND ITERATION

27-Apr-15

ENGINEERS

ASCE 41-06 PSUEDO LATERAL FORCE

Version Date: July 25, 2012

JOB TITLE: OWATC BDO 10A Upgrade
BUILDING LOCATION: BDO Ogden, UT

JOB #: 15118
PREPARED BY: ZCH

ASCE 41-06 Psuedo Lateral Force

Building Period (ASCE 41-06 Section 3.3.1.2)

Building Height (H): 27.0 ft
Period Coefficient (Ct): 0.02
Height Coefficient (β): 0.75

Bldg. Period (T): 0.24 sec

Mapped Acceleration Factors (ASCE 41-06 Section 1.6.1)

	BSE-1 (2/3*MCE)		BSE-2 (MCE)	
	S_s =	1.42	S_s =	1.42
	S_1 =	0.50	S_1 =	0.50
Soil Factors	F_a =	1.00	F_a =	1.00
	F_v =	1.50	F_v =	1.50
	Scale Factor =	0.67	Scale Factor =	1.00

Base Shear Coefficient (BSE-1 (2/3*MCE) - ASCE 41-06 Section 3.3.1)

Response Acceleration Parameters: (ASCE 41-06 Section 1.6.1 & Section 1.6.2)

S_{XS} = Scale Factor * S_1 * F_V = 0.95

S_{X1} = Scale Factor * S_1 * F_V = 0.50

$$T_s = \frac{S_{x1}}{S_{xs}}$$ T_s = 0.52 sec

T_o ... To = 0.10 sec

Effective viscous damping ratio (Section 1.6.1.5.1) β = 0.05

$$B_1 = \frac{4}{5.6 - \ln(100 * \beta)}$$ B_1 = 1.00

S_a = 0.95

BSE-1 (2/3*MCE) Psuedo Lateral Force - Section 3.3.1.3.1

Modification Factors:

DCR_{max} = 1.77

Effective mass factor C_m = 1 (ASCE 41-06 Table 3-1)
Site Class Factor a = 60 (Section 3.3.1.3)

$$R = \frac{DCR_{max}}{1.5} * C_m \geq 1.0$$ R = 1.18

$$C_1 = \begin{cases} 1.0 & T > 1.0 \\ 1 + \left(\frac{R-1}{aT^2}\right) & T \leq 1.0 \end{cases}$$ C_1 = 1.05

$$C_2 = \begin{cases} 1.0 & T > 0.7 \\ 1 + \frac{1}{800}\left(\frac{R-1}{T}\right)^2 & T \leq 0.7 \end{cases}$$ C_2 = 1.00

BSE-1 (2/3*MCE) Psuedo Lateral Force Coefficient $V = C_1 * C_2 * C_m * S_a * W$ V = 1.00 x W

ENGINEERS

ASCE 41-06 PSUEDO LATERAL FORCE (CONT'D)

Version Date: July 25, 2012

27-Apr-15
2:31 PM

JOB TITLE: OWATC BDO 10A Upgrade
BUILDING LOCATION: BDO Ogden, UT

JOB #: 15118
PREPARED BY: ZCH

Base Shear Coefficient (BSE-2 (MCE) - ASCE 41-06 Section 3.3.1)

Response Acceleration Parameters: (ASCE 41-06 Section 1.6.1 & Section 1.6.2)

S_{XS} = Scale Factor * S_1 * F_V = 1.42

S_{X1} = Scale Factor * S_1 * F_V = 0.74

$$T_s = \frac{S_{x1}}{S}$$ T_s = 0.52 sec

$$T_o = 0.2 * T_s$$ To = 0.10 sec

Effective viscous damping ratio (Section 1.6.1.5.1) β = 0.05

$$B_1 = \frac{4}{5.6 - \ln(100 * \beta)}$$ B_1 = 1.00

See Section 1.6.1.5 for Sa calculation) S_a = 1.42

BSE-2 (MCE) Psuedo Lateral Force - Section 3.3.1.3.1

Modification Factors:

DCR_{max} = 2.64

Effective mass factor (Table 3-1) C_m = 1 (ASCE 41-06 Table 3-1)
Site Class Factor a = 60 (Section 3.3.1.3)

$$R = \frac{DCR_{max}}{1.5} * C_m \geq 1.0$$ R = 1.76

$$C_1 = \begin{cases} 1.0 & T > 1.0 \\ 1 + \left(\frac{R-1}{aT^2}\right) & T \leq 1.0 \end{cases}$$ C_1 = 1.23

$$C_2 = \begin{cases} 1.0 & T > 0.7 \\ 1 + \frac{1}{800}\left(\frac{R-1}{T}\right)^2 & T \leq 0.7 \end{cases}$$ C_2 = 1.01

BSE-2 (MCE) Psuedo Lateral Force Coefficient

$$V = C_1 * C_2 * C_m * S_a * W$$ V = 1.76 x W

ENGINEERS

ASCE 41-06 DIAPHRAGM ANALYSIS / DESIGN

Version Date: September 3, 2014 Author: Scott Porter Reviewed by: Troy M. Dye, SE

27-Apr-15
2:35 PM

JOB TITLE: OWATC BDO 10A Upgrade
BUILDING LOCATION: BDO Ogden, UT
DIAPHRAGM AREA: Typical Diaphragm in the East West Direction

JOB #: 15118
PREPARED BY: ZCH

ASCE 41-06 - Roof Diaphragm Design / Analysis

Roof Weights: Design Mode: Upgrade Component Analysis/Upgrade Factor: 1.00

Roof Dead Load=	15	psf	Flat Roof Snow Load =	30	psf
Roof N-S Dimension =	240	ft	Roof Snow Load =	30	psf
Roof E-W Dimension =	60	ft	*Total Roof Weight =*	216000	lbs
Roof Seismic Weight =	15	psf			

Tributary Wall Weights - Earthquake in N-S Direction

	Wall Mark	Wall Weight (psf)	Tributary Wall Length (ft)	Tributary Wall Height (ft)	Total Wall Weight (lbs)
	1				0
	2				0
	3				0
	4				0
	5				0
	6				0
	7				0
	8				0
	9				0
	10				0

Tributary Wall Weights - Earthquake in E-W Direction

	Wall Mark	Wall Weight (psf)	Tributary Wall Length (ft)	Tributary Wall Height (ft)	Total Wall Weight (lbs)
	1	12	240	11	31680
	2	12	240	3.5	10080
	3				0
	4				0
	5				0
	6				0
	7				0
	8				0
	9				0
	10				0

Total Wall Weights N-S Direction = 0 lbs *Total Wall Weights E-W Direction =* 41760 lbs

Total Seismic Weight (TSW) N-S Direction = **216000** lbs *Total Seismic Weight (TSM) E-W Direction =* **257760** lbs

Diaphragm Design/Analysis N-S Direction

		Life Safety		Collapse Prevention
Performance Level		Life Safety		Collapse Prevention
Seismic Hazard Level		BSE-1		BSE-2
	C_1=	1.05	C_1=	1.23
	C_2=	1	C_2=	1.01
	C_m=	1	C_m=	1
	S_a=	0.95	S_a=	1.42
Pseudo Lateral Force Factor:	$C_1C_2C_mS_a$=	0.9975	$C_1C_2C_mS_a$=	1.764066
Pseudo Lateral Force	V_{NS}=	**215460** lbs	V_{NS}=	**381038** lbs

$V = C_1C_2C_mS_a \times TSW$

Length of Diaphragm Boundary L= ft

Shear Force at Boundary Edge:

NOT APPLICABLE - SEE FOLLOWING PAGE

Q_{UD_NS}= **#DIV/0!** lb/ft Q_{UD_NS}= **#DIV/0!** lb/ft

Diaphragm Capacity: Chorded? (Y/N): Y

$v_{s\,nominal}$ = lb/ft (LRFD nominal capacity w/ Φ=1.0 from NDS tables. ASCE 41-06 C8.5.8.2 states that this is equivalent to 2.0*ASD values.)

$v_{s\,expected}$ = 0 lb/ft

(Enter Diaphragm Thickness and Nailing)

m_{LS} =

m_{CP} =

(Factor obtained from ASCE 41-06 Table 8-9. Factor is based on diaphragm Length/Width ratio, diaphragm type, chorded/unchorded, blocked/unblocked)

k = 0.75 (Knowledge Factor, see ASCE 41-06 Section 2.2.6.4)

$m_{LS}*k*Q_{CE\_NS}$= 0 lb/ft $m_{CP}*k*Q_{CE\_NS}$= 0 lb/ft

Demand-to-Capacity Ratio (DCR): (If DCR values are above 1.5 psuedo force for entire building will increase.)

DCR_{NS}=	**#DIV/0!**	DCR_{NS}=	**#DIV/0!**
1.00*Qud/(mkQce) =	#DIV/0!	1.00*Qud/(mkQce) =	#DIV/0!
Status	#DIV/0!	Status	#DIV/0!

ENGINEERS

ASCE 41-06 DIAPHRAGM ANALYSIS/DESIGN

Version Date: September 3, 2014

27-Apr-15
2:35 PM

JOB TITLE: OWATC BDO 10A Upgrade
BUILDING LOCATION: BDO Ogden, UT
DIAPHRAGM AREA: Typical Diaphragm in the East West Direction

JOB #: 15118
PREPARED BY: ZCH

ASCE 41-06 - Roof Diaphragm Design/Analysis (Cont'd)

Diaphragm Design/Analysis E-W Direction

	Hazard Level	BSE-1		Hazard Level	BSE-2		
Pseudo Lateral Force Factor:	$C_1C_2C_mS_a$=	1.00		$C_1C_2C_mS_a$=	1.76		
Pseudo Lateral Force:	V_{EW}=	257116	lbs	V_{EW}=	454706	lbs	$V = C_1C_2C_mS_a \times TSW$
Length of Diaphragm Boundary:	L=	60	ft				
Shear Force at Boundary Edge:							
	Q_{UO_EW}=	2143	lb/ft	Q_{UO_EW}=	3789	lb/ft	

Diaphragm Capacity: Chorded? (Y/N): Y

$v_{s\ nominal}$ = 770 lb/ft (LRFD nominal capacity w/ Φ=1.0 from NDS tables. ASCE 41-06 C8.5.8.2 states that this is equivalent to 2.0*ASD values.)

$v_{s\ expected}$ = 1155 lb/ft

15/32" Overlay w/ 10d nails @ 4"o.c.

m_{LS} = 3

m_{CP} = 4

(Factor obtained from ASCE 41-06 Table 8-3. Factor is based on diaphragm Length/Width ratio, diaphragm type: chorded/unchorded, blocked/unblocked)

k = 1 (Knowledge Factor, see ASCE 41-06 Section 2.2.6.4)

m_{LS}*k*Q_{CE_EW}= 3465 lb/ft m_{CP}*k*Q_{CE_EW}= 4620 lb/ft

Demand-to-Capacity Ratio (DCR): (If DCR values are above 1.5 psuedo force for entire building will increase.)

DCR_{EW}=	1.77	DCR_{EW}=	2.64
1.00*Qud/(mkQce) =	0.62	1.00*Qud/(mkQce) =	0.82
Status	Okay	Status	Okay

ENGINEERS

ASCE 41-06 DIAPHRAGM ANALYSIS / DESIGN

27-Apr-15
2:35 PM

Version Date: September 3, 2014 Author: Scott Porter Reviewed by: Troy M. Dye, SE

JOB TITLE: OWATC BDO 10A Upgrade
BUILDING LOCATION: BDO Ogden, UT
DIAPHRAGM AREA: Typical Diaphragm in the North South Direction

JOB #: 15118
PREPARED BY: ZCH

ASCE 41-06 - Roof Diaphragm Design / Analysis

Roof Weights: Design Mode: Upgrade Component Analysis/Upgrade Factor: 1.00

Roof Dead Load=	15	psf	Flat Roof Snow Load =	30	psf
Roof N-S Dimension =	240	ft	Roof Snow Load =	30	psf
Roof E-W Dimension =	180	ft	*Total Roof Weight =*	648000	lbs
Roof Seismic Weight =	15	psf			

Tributary Wall Weights - Earthquake in N-S Direction

	Wall Mark	Wall Weight (psf)	Tributary Wall Length (ft)	Tributary Wall Height (ft)	Total Wall Weight (lbs)
	1	26	180	15	70200
	2	26	180	15	70200
	3				0
	4				0
	5				0
	6				0
	7				0
	8				0
	9				0
	10				0

Total Wall Weights N-S Direction = 140400 lbs

Total Seismic Weight (TSW) N-S Direction = **788400** lbs

Tributary Wall Weights - Earthquake in E-W Direction

	Wall Mark	Wall Weight (psf)	Tributary Wall Length (ft)	Tributary Wall Height (ft)	Total Wall Weight (lbs)
	1				0
	2				0
	3				0
	4				0
	5				0
	6				0
	7				0
	8				0
	9				0
	10				0

Total Wall Weights E-W Direction = 0 lbs

Total Seismic Weight (TSM) E-W Direction = **648000** lbs

Diaphragm Design/Analysis N-S Direction

		Life Safety			Collapse Prevention	
Performance Level		Life Safety			Collapse Prevention	
Seismic Hazard Level		BSE-1			BSE-2	
	$C_1=$	1.05		$C_1=$	1.23	
	$C_2=$	1		$C_2=$	1.01	
	$C_m=$	1		$C_m=$	1	
	$S_a=$	0.95		$S_a=$	1.42	
Pseudo Lateral Force Factor:	$C_1C_2C_mS_a=$	0.9975		$C_1C_2C_mS_a=$	1.764066	
Pseudo Lateral Force:	$V_{NS}=$	**786429**	lbs	$V_{NS}=$	**1390790** lbs	$V = C_1C_2C_mS_a \times TSW$

Length of Diaphragm Boundary: $L=$ 240 ft

Shear Force at Boundary Edge:

$Q_{UD\_NS}=$ **1638** lb/ft $Q_{UD\_NS}=$ **2897** lb/ft

Diaphragm Capacity: Chorded? (Y/N): Y

$v_{s\,nominal}=$ 770 lb/ft (LRFD nominal capacity w/ Φ=1.0 from NDS tables. ASCE 41-06 C8.5.8.2 states that this is equivalent to 2.0*ASD values.)

$v_{s\,expected}=$ 1155 lb/ft

15/32" Overlay w/ 10d nails @ 4"o.c.

$m_{LS}=$ 3

$m_{CP}=$ 4

(Factor obtained from ASCE 41-06 Table 8-3. Factor is based on diaphragm Length/Width ratio, diaphragm type, chorded/unchorded, blocked/unblocked)

$k=$ 1 (Knowledge Factor, see ASCE 41-06 Section 2.2.6.4)

$m_{LS}{*}k{*}Q_{CE\_NS}=$ **3465** lb/ft $m_{CP}{*}k{*}Q_{CE\_NS}=$ **4620** lb/ft

Demand-to-Capacity Ratio (DCR): (If DCR values are above 1.5 psuedo force for entire building will increase.)

	Life Safety		Collapse Prevention
$DCR_{NS}=$	1.35	$DCR_{NS}=$	2.02
1.00*Qud/(mkQce) =	0.47	1.00*Qud/(mkQce) =	0.63
Status	Okay	Status	Okay