antaring Menza ones	Project				Job Ref.	
	Section				Sheet no./rev.	
ASES					1 (of 81
112 Wilson Drive. Port Jefferson. NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				

ENGINEERING CALCULATIONS

Location:

Prepared for:

Prepared by:

A.S. Engineering Services, P.C. 112 Wilson Drive Port Jefferson, New York, 11777

Engineer of Record:

d united with a damage	Project				Job Ref.	
ASES	Section				Sheet no./rev. 2 of 81	
112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259	Calc. by AS	Date 5/15/2012	Chk'd by	Date	App'd by	Date

Table of Contents

	2
	3
WIND PRESSURE CALCS	6
WIND PERPENDICULAR TO RIDGE	6
WIND PARALLEL TO RIDGE	9
SEISMIC CALCS	13
SNOW LOADS	15
MOMENT CONNECTION CALCS	18
SHEARWALL CALCS	28
INDIVIDUAL BEAM ANALYSIS	38
FOOTING DESIGN	81

and state Means Odg.	Project					Job Ref.	
ANULUNA	Section					Sheet no./rev.	
ASES						3	of 81
112 Wilson Drive, Port Jefferson, NY 11777	, Calc. by		Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	A	S	5/15/2012				
						ŀ	
	l		ING PARA	METERS			
PER 2009 IRC							
WIND SPEED = 90 MPH							
EXP C							
SEISMIC - SEE BELOW							
FROM CALCS, MAXIMUM WIN	D PRESSUF	RE ARE	AS FOLLOWS				
ROOM INPUT							
PROJECTION	50.00	FT					
EAVE HEIGTH	8.00	FT					
RIDGE HEIGHT	16.00	FT					
FRONT WALL LENGTH	95.00	FT					
	10.00	nof fo		Windword			
WIND PRESSORES	12.00	psi io	r	Looward V			
	9.00	psillo		Side	/vaii		
	11.61	psf fo	r	Wall			
	14.16	psf fo	r	Roof			
WIND BASE SHEAR CALCUL	ATIONS						
	X-Direction	n Surfa	ce Area =	600	sa ft for	peaked wall	
	Y-Direction	n Surva	ce Area =	950	sa ft for	side wall	
Therefore.	X-Direction	Wind	Shear, V _{wx} =	13169	lbs		
	Y-Direction	Wind	Shear. V _{wY} =	12240	lbs		
	1		,	0			
NOTE THAT CORNER ZONE P	RESSURES	WERE	UTILIZED FOR	R ENTIRE STRU	UCTURE, TI	HEREFORE DESI	GN IS
CONSERVATIVE							
BASE SHEAR:							
Vx = 13.1K							
Vy = 12.3K							
BASE SHEAR MAIN HOUSE (E	XCLUDES (GARAG	E) :				
Vx = 13.1K							
Vy = 8.4K							

A MARTIN WARRANT	Project				Job Ref.	
ASES	Section				Sheet no./rev.	of 81
A.S. Engineering Services, P.C.		1	1			
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				

SHEAR AT ROOF:

Vx = 6.55K Vy = 4.2K

MAIN ROOF DIAPHRAGM APPROXIMATELY 30'x65' 1/2 TO EACH SIDE THEREFORE LOADS AS FOLLOWS: Vx = 3.3K

Vy= 2.1K

SHEAR PER FOOT OF WALL: vx = 3.3/65 = 51 PLF vy = 70 PLF

THEREFORE DESIGN HORIZONTAL DIAPHRAGM TO RESIST THESE LOADS

LOADS ARE TRANSFERRED TO PERIMETER WALLS AND INTERNAL STEEL BEAM

APPLY FULL LOAD TO BEAM 3.3K SINCE SPLIT IN HALFT BETWEEN BEAM AND REAR WALL

IF APPLY LOAD AT TOP OF COLUMN, H = 11'

MOMENT TO BE RESISTED = 36.3K-FT = 435.6 K-IN

FOR STEEL COLUMN

Sreq = 15.7

USE HSS7x5x1/2 S=17.3 AND I = 50.7

CHECK DELTA = (P*L^3)/48EI = .11 OKAY LESS THAN .01 X H = 0.11

SEE RISA CALCS

NOW CHECK SHEAR WALLS

CHECK WORST CASE X FIRST V = 70 PLF

INSTEAD DUMPED FULL 3.3 KIPS INTO WALL ALONG REAR MASTER BEDROOM - OK

AGAIN DESIGN IS CONSERVATIVE SINCE SINGLE WALL CAN RESIST FULL SHEAR LOAD AND ALL WALLS ARE CONSTRUCTED IN SAME MANNER

and and a state of the state of	Project				Job Ref.	
ASES	Section	Sheet no./rev. 5 of 81				
AS Ingrowing Service PC.	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				

SEE TEDDS OUTPUT

NOW CHECK Y-DIRECTION WALLS

2.1K EACH WALL

SEE TEDDS OUTPUT

CALC SEISMIC BASE SHEAR

TOTAL WEIGHT STRUCTURE

LENGHT EXTERIOR WALLS = 272'x12x10 AVE HEIGHT = 32640 LBS

LENGTH INTERIOR WALL BETWEEN KITCHEN AND GARAGE = 20x12x10=2400 LBS

ROOF = 3000 SF x 10 PSF = 30000 LBS

TOTAL WEIGHT STRUCTURE = 65040 LBS

SEISMIC BASE SHEAR = 7.5 KIPS

NOW TAKE 1/2 WALL WEIGHT + FULL ROOF FOR LOAD TO WALL = 47.5K

FORCE TO WALLS = 5.5 KIPS

0.5 TO EACH WALL = 2.75 KIPS

RECHECK TEDDS SHEAR WALL CALCS

and station Minera Dag	Project				Job Ref.	
	Section				Sheet no./rev.	
ASES					6	of 81
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
		DDECOUD				
	WIND	PRESSUR	E CALCS			
		PENDICUL		ЭF		
<u>WIND LOAD (ASCE 7 – 05)</u>						on version 1.2.02
					TEDDS calculati	on version 1.2.03
Structure is a building						
Structure is Pigid						
Mean roof height		b – 15 0 ft				
Horizontal dimension parallel to wind	4.	I = 13.0 ft				
Horizontal dimension parallel to wind	а, 1·	B = 92 0 ft				
Roof angle:	4,	θ = 32.0 π				
Wind force resisting element is n	art of main win	d force resistin	a system			
Structure is enclosed			g system			
Structure is low rise						
Dress dure						
Procedure		Catagory 2				
Occupancy category (table 1-1);		Category = 2				
Basic wind speed (sect. 0.5.4),		V = 90.0 mpn	Brono			
Importance factor (table 6-1):		l = 1.00	erione			
Exposure category (sect. 6.5.6):		C				
Wind directionality factor:		⊂ K ₄ = 0.85				
Topographic factor:		$K_{zt} = 1.00$				
·			Design p	rocedure - analy	tical procedu	re (Method2)
Velocity pressure at mean roof heid	ot 'b'(ASCE 7-0	5 cl 6 5 10)		-	•	, ,
Case of loading system (table 6-3):		Case = 1				
Velocity pressure exposure coefficie	ent:	$K_{\rm h} = 0.85$				
Velocity pressure at mean roof heigh	nt 'h':	$q_{\rm h} = 0.00256 \times$	$K_h \times K_{7} \times K_d \times \Lambda$	$/^2 \times I \times 1 \text{ psf} / \text{mp}$	h ² = 14.98 psf	:
Design wind proceure for MWERS	f low rise onelog	and and partially			rooduro)	
Velocity prossure of moon roof heid	n low-rise enclos	sed and partially $a_1 = 14.98$ pcf	enciosed buildi	ngs (allemative p	nocedure)	
		q _h = 14.30 psi				
External and internal pressure coeffi	icients (fig. 6-5)					
Positive internal pressure coefficient	t;	GC _{pi_pos} = 0.18	_			
Negative internal pressure coefficier	nt;	GC _{pi_neg} = -0.18	3			
Building surface 1						
External pressure coeff. for surface	1 (fig. 6-10);	$GC_{pf_1} = 0.51$				
With positive GC _{pi} ;		$p_{1_{S1}} = q_h \times ((G$	iC _{pf_1}) - (GC _{pi_pos}	s)) = 4.98 psf		
With negative GC _{pi} ;		$p_{2_S1} = q_h \times ((G_{1}))$	iC _{pf_1}) - (GC _{pi_neg}	g)) = 10.38 psf		
Building surface 2						
External pressure coeff. for surface	2 (fig. 6-10);	GC _{pf_2} = -0.69				
With positive GC _{pi} ;		$p_{1_{S2}} = q_h \times ((G_{S2}))$	Cpf_2) - (GCpi_pos	s)) = -13.03 psf		
With negative GC _{pi} ;		$p_{2_{S2}} = q_h \times ((G_{S2}))$	Cpf_2) - (GCpi_nec	g)) = -7.64 psf		

Water and Manual Strate	Project				Job Ref.	
a bed ateda	Castion				Chaot no /roy	
ASES	Section				Sneet no./rev.	7 of 91
A.S. Engineering Gervices, P.C.	Calc by	Date	Chk'd by	Date	App'd by	
112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259	AS	5/15/2012	Clika by	Dale	лрр и Бу	Date
	7.0	0,10,2012				
Building surface 3						
External pressure coeff. for surface	3 (fig. 6-10);	GC _{pf_3} = -0.47				
With positive GC _{pi} ;		$p_{1_{S3}} = q_h \times ((C_{S3}))$	GC _{pf_3}) - (GC _{pi_}	_{_pos})) = -9.67 psf		
With negative GC _{pi} ;		$p_{2_S3} = q_h \times ((C_{1}))$	GC _{pf_3}) - (GC _{pi_}	_neg)) = -4.27 psf		
Building surface 4						
External pressure coeff. for surface	4 (fig. 6-10);	GC _{pf_4} = -0.41				
With positive GC _{pi} ;		$p_{1_S4} = q_h \times ((C_{1_S4}))$	GC _{pf_4}) - (GC _{pi_}	_pos)) = -8.86 psf		
With negative GC _{pi} ;		$p_{2_S4} = q_h \times ((C_{1}))$	GC _{pf_4}) - (GC _{pi_}	_ _{neg})) = -3.47 psf		
Building surface 5						
External pressure coeff. for surface	5 (fig. 6-10);	GC _{pf_5} = -0.45				
With positive GC _{pi} ;		$p_{1_S5} = q_h \times ((C_{1}))$	GC _{pf_5}) - (GC _{pi}	_pos)) = -9.44 psf		
With negative GC _{pi} ;		$p_{2_{S5}} = q_h \times ((C_{S5}))$	GC _{pf_5}) - (GC _{pi_}	_neg)) = -4.05 psf		
Building surface 6						
External pressure coeff. for surface	6 (fig. 6-10);	GC _{pf 6} = -0.45				
With positive GC _{pi} ,		$p_{1_{S6}} = q_h \times ((0))$	GC _{pf_6}) - (GC _{pi_}	_{pos})) = -9.44 psf		
With negative GC _{pi} ;		$p_{2_S6} = q_h \times ((0$	GC _{pf_6}) - (GC _{pi_}	_neg)) = -4.05 psf		
Building surface 1E						
External pressure coeff. for surface	1E (fig. 6-10);	GC _{pf 1E} = 0.77				
With positive GC _{pi} ,		$p_{1_{S1E}} = q_h \times (($	(GC _{pf_1E}) - (GC	; ; _{pi_pos})) = 8.91 psf	f	
With negative GC _{pi} ;		$p_{2_{S1E}} = q_h \times (($	(GC _{pf_1E}) - (GC	; pi_neg)) = 14.30 p	sf	
Building surface 2E						
External pressure coeff. for surface	2E (fig. 6-10);	GC _{pf 2E} = -1.07	7			
With positive GC _{pi} ;		$p_{1_{S2E}} = q_h \times (($	(GC _{pf_2E}) - (GC	; pi_pos)) = -18.73 p	osf	
With negative GC _{pi} ;		$p_{2_S2E} = q_h \times (($	(GC _{pf_2E}) - (GC	; ; _{pi_neg})) = -13.33 p	osf	
Building surface 3E						
External pressure coeff. for surface	3E (fig. 6-10);	GC _{pf 3E} = -0.67	7			
With positive GC _{pi} ,		$p_{1_{S3E}} = q_h \times (($	(GC _{pf_3E}) - (GC	; ; _{pi_pos})) = -12.71 p	osf	
With negative GC _{pi} ;		$p_{2_S3E} = q_h \times (($	(GC _{pf_3E}) - (GC	; p _{i_neg})) = -7.32 ps	sf	
Building surface 4E						
External pressure coeff. for surface	4E (fig. 6-10);	GC _{pf 4E} = -0.61	1			
With positive GC _{pi} ;		$p_{1_{S4E}} = q_h \times (($	(GC _{pf_4E}) - (GC	; ; _{pi_pos})) = -11.87 p	osf	
With negative GC _{pi} ,		$p_{2_S4E} = q_h \times (($	(GC _{pf_4E}) - (GC	; p _{i_neg})) = -6.47 ps	sf	
Note: - As per Section 6.1.	4.1, the wind l	oad to be used	in the design	of the MWFRS	shall be not le	ess than 10 psf

Note: - As per Section 6.1.4.1, the wind load to be used in the design of the MWFRS shall be not less than 10 psf multiplied by the area of the building or structure projected onto a vertical plane normal to the wind direction.





Project				Job Ref.	
Section				Sheet no./rev.	f 01
A Standard Baracan PC.	Date	Chk'd by	Date	App'd by	Date
12 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259 A	S 5/15/2012		Date	, ibb a by	Duto
	6	6		(4) (4)	
	6	2	5		
	D B2			APS -	
	B		5 Letiono	i Mud	
Being Designed Reference	<	B Referen	ce Beiny		
Corner	Tanalanalia	Corne	r		
Trepoveres Dire	IOrsional Load		n al Dina ati		
Iransverse Dire	ction	Longitud	nal Directi	on	
W	IND PARALLEL	. TO RIDGE			
WIND LOAD (ASCE 7 - 03)				TEDDS calculation	n version 1.2.03
Classification summary					
Structure is a building					
Structure is Rigid					
Mean roof height;	h = 15.0 ft				
Horizontal dimension parallel to wind;	L = 92.0 ft				
Horizontal dimension normal to wind;	B = 42.0 ft				
Roof angle;	θ = 18.0 deg				
Wind force resisting element is part of ma Structure is enclosed	ain wind force resisti	ing system			
Structure is low rise					
Procedure					
Procedure Occupancy category (table 1-1);	Category = 2				
Procedure Occupancy category (table 1-1); Basic wind speed (sect. 6.5.4);	Category = 2 V = 90.0 mph				
Procedure Occupancy category (table 1-1); Basic wind speed (sect. 6.5.4); Region;	Category = 2 V = 90.0 mph Non-Hurrica	ne Prone			
Procedure Occupancy category (table 1-1); Basic wind speed (sect. 6.5.4); Region; Importance factor (table 6-1);	Category = 2 V = 90.0 mph Non-Hurrica I = 1.00	ne Prone			
Procedure Occupancy category (table 1-1); Basic wind speed (sect. 6.5.4); Region; Importance factor (table 6-1); Exposure category (sect. 6.5.6);	Category = 2 V = 90.0 mph Non-Hurricat I = 1.00 C	ne Prone			
Procedure Occupancy category (table 1-1); Basic wind speed (sect. 6.5.4); Region; Importance factor (table 6-1); Exposure category (sect. 6.5.6); Wind directionality factor;	Category = 2 V = 90.0 mph Non-Hurricat I = 1.00 C K _d = 0.85	ne Prone			
Procedure Occupancy category (table 1-1); Basic wind speed (sect. 6.5.4); Region; Importance factor (table 6-1); Exposure category (sect. 6.5.6); Wind directionality factor; Topographic factor;	Category = 2 V = 90.0 mph Non-Hurrican I = 1.00 C $K_d = 0.85$ $K_{zt} = 1.00$	ne Prone			
Procedure Occupancy category (table 1-1); Basic wind speed (sect. 6.5.4); Region; Importance factor (table 6-1); Exposure category (sect. 6.5.6); Wind directionality factor; Topographic factor;	Category = 2 V = 90.0 mph Non-Hurrican I = 1.00 C $K_d = 0.85$ $K_{zt} = 1.00$	ne Prone Design µ	procedure - ar	nalytical procedur	e (Method2)
Procedure Occupancy category (table 1-1); Basic wind speed (sect. 6.5.4); Region; Importance factor (table 6-1); Exposure category (sect. 6.5.6); Wind directionality factor; Topographic factor; Velocity pressure at mean roof height 'h' (AS	Category = 2 V = 90.0 mph Non-Hurrical I = 1.00 C $K_d = 0.85$ $K_{zt} = 1.00$ CE 7-05, cl. 6.5.10)	ne Prone Design µ	procedure - ar	nalytical procedur	e (Method2)
Procedure Occupancy category (table 1-1); Basic wind speed (sect. 6.5.4); Region; Importance factor (table 6-1); Exposure category (sect. 6.5.6); Wind directionality factor; Topographic factor; Velocity pressure at mean roof height 'h'(AS Case of loading system (table 6-3);	Category = 2 V = 90.0 mph Non-Hurrican I = 1.00 C $K_d = 0.85$ $K_{zt} = 1.00$ CE 7-05, cl. 6.5.10) Case = 1	ne Prone Design µ	procedure - ar	nalytical procedur	e (Method2)
Procedure Occupancy category (table 1-1); Basic wind speed (sect. 6.5.4); Region; Importance factor (table 6-1); Exposure category (sect. 6.5.6); Wind directionality factor; Topographic factor; Velocity pressure at mean roof height 'h'(AS Case of loading system (table 6-3); Velocity pressure exposure coefficient;	Category = 2 V = 90.0 mph Non-Hurrical I = 1.00 C $K_d = 0.85$ $K_{zt} = 1.00$ CE 7-05, cl. 6.5.10) Case = 1 $K_h = 0.85$	ne Prone Design µ	procedure - ar	nalytical procedur	e (Method2)

a manufacture stars on a	Project Job Ref.					
ASES	Section				Sheet no./rev.	of 81
112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259	Calc. by AS	Date 5/15/2012	Chk'd by	Date	App'd by	Date
Design wind pressure for MWFRS of Velocity pressure at mean roof height	f low-rise enclo nt 'h';	sed and partially q _h = 14.98 psf	enclosed buildi	ngs (alternative p	procedure)	
External and internal pressure coeff Positive internal pressure coefficient Negative internal pressure coefficier	cients (fig. 6-5 ;; nt;) GC _{pi_pos} = 0.18 GC _{pi_neg} = -0.18	3			
Building surface 1 External pressure coeff. for surface With positive GC _{pi} ; With negative GC _{pi} ;	1 (fig. 6-10);	$GC_{pf_{-1}} = 0.51$ $p_{1_{-}S1} = q_h \times ((G_{-}p_{2_{-}S1} = q_h \times ((G_{-}p_{-}))))$	iC _{pf_1}) - (GC _{pi_pos} iC _{pf_1}) - (GC _{pi_neg})) = 4.98 psf)) = 10.38 psf		
Building surface 2 External pressure coeff. for surface With positive GC _{pi} ; With negative GC _{pi} ;	2 (fig. 6-10);); $GC_{pf_2} = -0.69$ $p_{1_S2} = q_h \times ((GC_{pf_2}) - (GC_{pi_pos})) = -13.03 \text{ psf}$ $p_{2_S2} = q_h \times ((GC_{pf_2}) - (GC_{pi_neg})) = -7.64 \text{ psf}$				
Building surface 3 External pressure coeff. for surface With positive GC _{pi} ; With negative GC _{pi} ;	3 (fig. 6-10);	$\begin{aligned} GC_{pf_3} &= \textbf{-0.47} \\ p_{1_S3} &= q_h \times ((GC_{pf_3}) - (GC_{pi_pos})) = \textbf{-9.67} \text{ psf} \\ p_{2_S3} &= q_h \times ((GC_{pf_3}) - (GC_{pi_neg})) = \textbf{-4.27} \text{ psf} \end{aligned}$				
Building surface 4 External pressure coeff. for surface With positive GC _{pi} ; With negative GC _{pi} ;	4 (fig. 6-10);	; $GC_{pf_{4}} = -0.41$ $p_{1_{54}} = q_h \times ((GC_{pf_{4}}) - (GC_{pi_{pos}})) = -8.86 \text{ psf}$ $p_{3_{54}} = q_h \times ((GC_{pf_{4}}) - (GC_{pi_{pos}})) = -3.47 \text{ psf}$				
Building surface 5 External pressure coeff. for surface With positive GC _{pi} ; With negative GC _{pi} ;	5 (fig. 6-10);	$GC_{pf_5} = -0.45$ $p_{1_S5} = q_h \times ((G_{p_2_S5} = q_h \times ((G_{p_1} = 0))))$	iC _{pf_5}) - (GC _{pi_pos} iC _{pf_5}) - (GC _{pi_neg})) = -9.44 psf)) = -4.05 psf		
Building surface 6 External pressure coeff. for surface With positive GC _{pi} ; With negative GC _{pi} ;	6 (fig. 6-10);	$GC_{pf_6} = -0.45$ $p_{1_S6} = q_h \times ((G_{p_2_S6} = q_h \times ((G_{p_1} = (G_{p_1} = q_h \times ((G_{p_1} = (G_{p_1} = (G_{p_1} = (G_{p_1} = (G_{p_1} \times ((G_{p_1} = (G_{p_1} = (G_{p_1} \times ((G_{p_1} = (G_{p_1} \times ((G_{p_1} \times (G_{p_1}$	iC _{pf_6}) - (GC _{pi_pos} iC _{pf_6}) - (GC _{pi_neg})) = -9.44 psf)) = -4.05 psf		
Building surface 1E External pressure coeff. for surface With positive GC _{pi} ; With negative GC _{pi} ;	1E (fig. 6-10);	$GC_{pf_{1E}} = 0.77$ $p_{1_{S1E}} = q_h \times ((0, p_{2_{S1E}}) = q_h \times ((0, p_{1_{S1E}})))$	GC _{pf_1E}) - (GC _{pi_f} GC _{pf_1E}) - (GC _{pi_}	_{bos})) = 8.91 psf _{heg})) = 14.30 psf		
Building surface 2E External pressure coeff. for surface With positive GC _{pi} ; With negative GC _{pi} ;	2E (fig. 6-10);	$GC_{pf_{2E}} = -1.07$ $p_{1_{S2E}} = q_h \times ((0))$ $p_{2_{S2E}} = q_h \times ((0))$	GC _{pf_2E}) - (GC _{pi_f} GC _{pf_2E}) - (GC _{pi_r}	_{bos})) = -18.73 psf _{neg})) = -13.33 psf		
Building surface 3E External pressure coeff. for surface With positive GC _{pi} ; With negative GC _{pi} ;	3E (fig. 6-10);	$GC_{pf_3E} = -0.67$ $p_{1_S3E} = q_h \times ((0))$ $p_{2_S3E} = q_h \times ((0))$	GC _{pf_3E}) - (GC _{pi_f} GC _{pf_3E}) - (GC _{pi_f}	_{bos})) = -12.71 psf _{heg})) = -7.32 psf		

A MARTINE MERCE AND	Project				Job Ref.	
ASES	Section				Sheet no./rev. 11 of 81	
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				

Building surface 4E

External pressure coeff. for surface 4E (fig. 6-10);

With positive GC_{pi};

 $\begin{aligned} GC_{pf_{-}4E} &= \textbf{-0.61} \\ p_{1_S4E} &= q_h \times ((GC_{pf_{-}4E}) - (GC_{pi_pos})) = \textbf{-11.87} \text{ psf} \end{aligned}$

With negative GC_{pi};

 $p_{2_S4E} = q_h \times ((GC_{pf_4E}) - (GC_{pi_neg})) = -6.47 \text{ psf}$

Note: - As per Section 6.1.4.1, the wind load to be used in the design of the MWFRS shall be not less than 10 psf multiplied by the area of the building or structure projected onto a vertical plane normal to the wind direction.



Basic Load Cases - Transverse Direction





Torsional Load Case

Transverse Direction

Longitudinal Direction

and station means Day	Project				Job Ref.	
ATLENTA	Section				Sheet no./rev.	
ASES					13	of 81
112 Wilson Drive Port Jefferson NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
						1
	9	SEISMIC CA	LCS			
SEISMIC FORCES (ASCE 7-05)						
					Tedds calculati	on version 3.0.01
Site parameters						
Site class:		D				
Mapped acceleration parameters (S	ection 11.4.1)	_				
at short period;	,	S _S = 1.027				
at 1 sec period;		S ₁ = 0.354				
Site coefficientat short period (Table	e 11.4-1);	F _a = 1.1				
at 1 sec period (Table 11.4-2);	,. ,.	F _v = 1.7				
Spectral response acceleration par	meters					
at short period (Eq. 11.4-1):	lineleis	Suc - E × So	- 1 110			
at short period (Eq. $11.4-1$),		$S_{MS} = \Gamma_a \times S_S$	- 0 500			
at i sec penou (Eq. 11.4-2),		$\mathbf{S}_{M1} = \mathbf{F}_V \times \mathbf{S}_1 =$	= 0.555			
Design spectral acceleration parameters	eters (Sect 11.4	4.4)				
at short period (Eq. 11.4-3);		$S_{DS} = 2/3 \times S_{DS}$	Б _{МЅ} = 0.746			
at 1 sec period (Eq. 11.4-4);		$S_{D1} = 2/3 \times S$	_{M1} = 0.399			
Seismic design category						
Occupancy category (Table 1-1);		Ш				
Seismic design category based on s	hort period res	ponse accelerat	ion (Table 11.6	-1)		
		D				
Seismic design category based on (sec period res	sponse accelerat	tion (Table 11.6	6-2)		
		D				
Seismic design category;		D				
Approximate fundamental period						
Height above base to highest level of	of building;	h _n = 14 ft				
From Table 12.8-2:						
Structure type;		All other syste	ms			
Building period parameter C _t ;		$C_t = 0.02$				
Building period parameter x;		x = 0.75				
		T O "``	A 11.4.5.Y			
Approximate fundamental period (E	q 12.8-7);	$I_a = C_t \times (h_n)^*$	× 1sec / (1ft)^=	U.145 Sec		
Building fundamental period (Sect 1	2.8.2);	$ = _a = 0.145$	sec			
Long-period transition period;		$I_{L} = 16 \text{ sec}$				
Seismic response coefficient						
Seismic force-resisting system (Tab	le 12.14-1);	A. Bearing_Wa	all_Systems			
		13.Light-frame	d walls sheath	ed with wood par	nels ratedfor shr	/stl
Response modification factor (Table	e 12.14-1);	R = 6.5				
Seismic importance factor (Table 11	.5-2);	l _e = 1.000				
Seismic response coefficient (Sect	12.8.1.1)					
Calculated (Eq 12.8-2);		$C_{s_{calc}} = S_{DS} / ($	(K / I _e)= 0.115			

OVALIDA	1127 × 06		Project				Job Ref.	
	100							
			Section				Sheet no /rev	
AS	FS		Coolion				14	of 91
A.S. Engineerte	g Services, P.C.						14	
112 Wilson Drive, Por	t Jefferson, N	IY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-5	60-0259		AS	5/15/2012				
	0.0.0			0 0 1/				
Maximum (Eq 1	2.8-3);			$C_{s_{max}} = S_{D1} / ($	$I \times (R / I_e)) =$	0.424		
Minimum (Eq 12	2.8-5,Supp	o. No. 2);		$C_{s_{min}} = max(0)$	$044 \times S_{DS} \times I_{e}$	_e ,0.01) = 0.033		
Seismic respon	se coefficie	ent;		C _s = 0.115				
Seismic base sl	near (Sect	12.8.1)						
Effective seismi	c weight o	f the struc	ture;	W = 65.0 kips				
Seismic respon	se coefficie	ent:		C _s = 0.115				
Seismic base sl	near (Eq.1 [.]	2 8-1) [.]		$V = C_0 \times W = 7$	5 kips			
		2.0 1),		· = 0; / · · · = ·				
SEISIVIIC ACCE	LERATIO		IE I EKS FROM	0363				
Contermi	inous 48	Statos						
2000 Inte	rnotional	Duilding	Codo					
Zin Code	00045	Building	CODE					
Zip Code	= 90240	Anala	stiene Ce and					
Spectral	Response	e Accelei	trations 5s and	151				
Ss and S	1 = Mapp	bed Spec	tral Accelerati	on values				
Data are	based or	n a 0.05 c	deg grid spacii	ng				
_								
Period	Centroid	d Sa						
(sec)	(g)							
0.2	1.027	(Ss)						
1.0	0.354	(S1)						
Period	Maximu	ım Sa						
(sec)	(g)							
0.2	1.057	(Ss)						
1.0	0.371	(S1)						
Period	Minimur	m Sa						
(sec)	(g)							
0.2	1.024	(Ss)						
1.0	0.352	(S1)						
		(/						

Project					Job Ref.		
Admit atmit	Section				Sheet no./rev.		
ASES					1	5 of 81	
12 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date	
(C) 631-560-0259	AS	5/15/2012					
		SNOW LO	ADS				
SNOW LOADING (ASCE7-10)					TEDDS calcul	ation version .	
Building details							
Roof type;		Monopitch					
Width of roof;		b = 42.00 ft					
Slope of roof 1;		a = 18.00 deg					
Ground snow load							
Ground snow load;		p _g = 25.00 lb/	ft ²				
Density of snow;		g = min(0.13	p _g / 1ft + 14	lb/ft ³ , 30lb/ft ³)	= 17.25 lb/ft ³		
Terrain type;		В					
Exposure condition (Table 7-2);		Fully expose	d 🖌				
Exposure factor (Table 7-2);		C _e = 0.90					
Thermal condition (Table 7-3);		Structures ke	pt just above	e freezing			
Thermal factor (Table 7-3);		C _t = 1.10					
Importance category (Table 1-1);		Ш					
Importance factor (Table 7-4);		I _s = 1.00					
Flat roof snow load (Sect 7.3);		$p_f = 0.7 \ C_e$	$C_t ightharpoonup I_s ightharpoonup p_g =$	= 17.33 lb/ft ²			
Cold roof slope factor ($C_t > 1.0$)							
Roof surface type;		Non slippery					
Ventilation;		Ventilated					
Thermal resistance (R-value);		R = 38.00;°F I	n ft ² / Btu				
	line);	C _s = 1.00					
Roof slope factor Fig 7-2b (solid l							
Roof slope factor Fig 7-2b (solid I Monoslope							





and the second second second	Project				Job Ref.	
Andatat	Section				Sheet no./rev.	
ASES					1	8 of 81
12 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
	MOME			LCS		
ALTHOUGH CALCS DO NOT EXA	CTLY MATCH	I CONDITION, TH	IEY ARE VAL	ID - ONLY DIF	FERENCE IS CO	DLUMN
SIZE/TYPE. WELD OF PLATES T	O COLUMNS	IS MORE THAN	ADEQUATE			
SEE FOLLOWING SHEETS FOR						

a sugar the second second	Project				Job Ref.	
And-una	Section				Sheet no./rev.	
ASES					1	9 of 81
112 Wilson Drive Port Jefferson NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
	Connec	tion 1			45	D
	Connects	tion 1 Column/Bea	m <mark>Flan</mark> ge Plat	e Moment	AS Connection	D 1
	Connect S Mater	tion 1 Column/Beau	m Flange Plat	e Moment	AS Connection	D
	Connect s Materi Colum	tion 1 Column/Beau ial Properties: on W8X3	m Flange Plat	e Moment Fy=50.00 ksi	AS Connection F _u = 65.00 ksi	D

					· · ·	u	
		Plate	P0.38X4.00X12.00	A36	$F_y = 36.00 \text{ ksi}$	F _u = :	58.00 ksi
	e 1	Moment Plate	P0.50X10.25X9.50	A36	$F_y = 36.00 \text{ ksi}$	F _u = :	58.00 ksi
		Input Data:					
		Load	15.00 kips 436.00 kips in	Us	er Input Load		
		Top Column D	450.00 mps-m	Us	er Input Top Coli	mm Dis	
		Column Force	30.00 kips	Us	er Input Column .	Force	
		Story Shear	0.00 kips	U_{2}	er Input Story She	sar	
Note: Unless specified, all code refe	from AISC 360-05	Collapse	All	📱 Expand	I All		
Limit State		Require	ed Availab	le	Unity Check	R	lesult
Geometry Restrictions at Be	am					P	ASS
Check Min Bolt Spacing	Pass		Condition: S	min >=	(2+2/3) * dboli		(J3.3)
Smin	3.00 in		Min bolt space	cing			
dbolt	0.75 in		Bolt diameter	*3			
Check Max Bolt Spacing	Pass		Condition: S,	max<=	min(12.00 in, 2	(4*t)	(J3.5a
Smax	3.00 in		Max bolt spa	cing			
t	0.38 in		Thickness of	govern	ing element (Pl	ate)	
Check Min Edge Distance	Pass		Condition: E	D _{min} >	$= ED_{allow}$		(J3.4
Check Max Edge Distance	Pass		Condition: E	D _{max} <	= min (6.00 in	, 12*t)	(J3.5)
Geometry Restrictions at Fla	ange Be	am				P	ASS
Chask Min Balt Spasing	Pare		Condition: S	>=	(2+2/3) * A.		(73 3

Pass	Condition: $S_{min} \ge (2+2/3) * d_{bolt}$	(J3.3)
3.00 in	Min bolt spacing	
0.75 in	Bolt diameter	
Pass	Condition: $S_{max} \le min(12.00 \text{ in, } 24*t)$	(J3.5a)
7.25 in	Max bolt spacing	
0.50 in	Thickness of governing element (Moment 1	Plate)
Pass	Condition: $ED_{min} > = ED_{allow}$	(J3.4)
Pass	Condition: $ED_{max} \le min (6.00 in, 12*t)$	(J3.5)
	Pass 3.00 in 0.75 in Pass 7.25 in 0.50 in Pass Pass	Pass Condition: $S_{min} >= (2+2/3) * d_{bolt}$ 3.00 in Min bolt spacing 0.75 in Bolt diameter Pass Condition: $S_{max} <= min(12.00 in, 24*t)$ 7.25 in Max bolt spacing 0.50 in Thickness of governing element (Moment 1) Pass Condition: $ED_{min} >= ED_{allow}$ Pass Condition: $ED_{max} <= min (6.00 in, 12*t)$

Flange Plate Weld Limitations

Check Weld Min Size Pass PASS

understan Means Anny	Project				Job Ref.	
Anthata	Section				Sheet no./rev.	
ASES					20	of 81
on Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
	0.25 :		III's laborate			
D	0.25 m		Weia size	8		
Dmin	0.19 m		Min size allowea			
nin	0.50 m		Controlling mem	ber thickness		
Check Weld Min Leng	gth Pass		Condition: wp >	= 4 * D		
D	0.25 m		Weld size			
Wp	10.25 in		Plate width	100 0 0		
Check Weld Max Len	gth Pass		Condition: wp <=	= 100 * D		
D	0.25 in		Weld size			
Wp	10.25 in		Plate width			
Column Weld Limitation	15				PASS	
Weld Max/Min Size, L	ength		(J2.2b)			
Check Weld Min Size	Pass					
D	0.25 in		Weld size			
D _{min}	0.19 in		Min size allowed			
t _{min}	0.38 in		Controlling mem	ber <mark>thickness</mark>		
Check Weld Min Leng	gth Pass		Condition: L _{min}	>= 4*D		
D	0.25 in		Weld size			
L _{min}	12.00 in		Min weld segmen	nt length		
Check Weld Max Len	gth Pass		Condition: L _{max}	<= 100*D		
D	0.25 in		Weld size			
L _{max}	12.00 in		Max weld segme	nt length		
Shear Plate Weld Streng	th at Column	15.00 kips	78.38 kips	0.19	PASS	
$\operatorname{Rn}/\Omega = 2 * C_1 * \alpha * 0.5$	928 * D ₁₆ * L					
Double Fillet						
c ₁	1.00		Electrode strengt	th coefficient		
α	0.88		Base material pr	oration factor		
D ₁₆	4.00		Weld fillet size in	n sixteenths of an	inch	
L	12.00 in		Weld length per :	side		
Rn/Ω	78.38 kips		Weld strength	Metal State and I		2
Beam Web Shear Yield		15.00 kips	129.01 kip	os 0.12	PASS	
$R_n = 0.6 * F_y * A_g * C_y$			$\Omega = 1.50$	(G2-1)		
Fy	50.00 ksi		Minimum yield st	tress of material		
Ag	6.45 in ²		Gross area subje	ect to shear		
C _v	1.00		Web shear coeffi	cient (G2-2)		
R_n/Ω	129.01 kips	9	Shear yield stren	gth		
		15.00 kips	64 80 kips	0.23	PASS	1
Vert. Plate Shear Yield			a transferra			

AND THE DR. MILLION MILLION	Project				Job Ref.	
a tot atota	Section				Sheet no./rev.	
ASES					21	of 8
As Concerned Brock PC. Ison Drive. Port Jefferson. NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
-	den a selator a					
Fy	36.00 ksi		Minimum yield str	ess of material		
Ag	4.50 in ²		Gross area subjec	t to shear		
R _n /Ω	64.80 kips		Shear yield streng	th		
Beam Web Shear Ruptur	re	15.00 kips	98.82 kips	0.15	PASS	
$\mathbf{R}_{\mathbf{n}} = 0.6 \text{ *} \mathbf{F}_{\mathbf{u}} \text{*} \mathbf{A}_{\mathbf{nv}}$			Ω = 2.0 (J4-4)		
Fu	65.00 ksi		Minimum tensile s	tress of material		
Anv	5.07 in ²		Net area subject to	o shear		
R _n /Ω	98.82 kips		Shear rupture stre	ngth		
Vert. Plate Shear Ruptur	.e	15.00 kips	55.46 kips	0.27	PASS	
$\mathbf{R}_{\mathbf{n}} = 0.6 \mathbf{F}_{\mathbf{u}} \mathbf{A}_{\mathbf{n}\mathbf{v}}$			Ω = 2.0	(J4-4)		
Fu	58.00 ksi		Minimum tensile s	tress of material		
Anv	3.19 in ²		Net area subject to	o shear		
R _n /Ω	55.46 kips		Shear rupture stre	ngth		
Beam Web Block Shear		15.00 kips	81.48 kips	0.18	PASS	
$\mathbf{R}_{\mathbf{n}} = [\min(0.6^{+}\mathbf{F}_{\mathbf{u}}^{+}\mathbf{A}_{\mathbf{n}\mathbf{v}}^{+}]$	$(0.6*F_y*A_g) + U$	bs *Fu *Ant]	$\Omega = 2.00$	(34-5)		
Ag	4.69 in ²		Gross area subjec	t to shear		
Anv	3.48 in ²		Net area subject to	o shear		
Ubs	1.00		Uniform tension si	tress factor		
Ant	0.42 in ²		Net area subject to	o tension		
Fu	65.00 ksi		Minimum tensile s	tress of material		
Fy	50.00 ksi		Minimum yield str	ess of material		
R _n /Ω	81.48 kips		Block shear streng	<u>zth</u>		
Vert. Plate Block Shear		15.00 kips	59.52 kips	0.25	PASS	
$R_n = [\min(0.6*F_u*A_{nv})]$	$(0.6*F_y*A_g) + U$	bs*Fu*Ant]	Ω = 2.00 ((34-5)		
Ag	3.94 in ²		Gross area subjec	t to shear		
Anv	2.79 in ²		Net area subject to	o shear		
Ubs	1.00		Uniform tension si	tress factor		
Ant	0.59 in ²		Net area subject to	o tension		
Fu	58.00 ksi		Minimum tensile s	tress of material		
Fy	36.00 ksi		Minimum yield str	ess of material		
R _n /Ω	59.52 kips		Block shear streng	<i>şth</i>		
		15.00 kins	42.41 kips	0.35	PASS	
Bolt Shear at Beam Web		10.00 Mps				
Bolt Shear at Beam Web R _n = F _{nv} *A _b *N _{bolt} *C		15.00 kips	$\Omega = 2.00$	(J3-1)		
Bolt Shear at Beam Web R _n = F _{nv} *A _b *N _{bolt} *C F _{nv}	48.00 ksi	15.00 kips	$\Omega = 2.00$ (Shear stress N type	(J3-1) e		

at sugar	A STATE OF THE OWNER	Project				JOD KET.	
A	AL STRAG	Section				Sheet no./rev.	
A	SES					22 of 81	
2 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259		Calc. by AS	Date 5/15/2012	Chk'd by	Date	App'd by	Date
12							
	N _{bolt}	4		Number of bol	ts		
	С	1.00		Eccentricity coefficient			
	R_n/Ω	42.41 kips	6	Bolt shear rup	ture strength		
В	olt Bearing at Beam	Web	15.00 kips	92.43 ki	ps 0.16	PASS	
	$R_n = N_{rows} * N_{cols} * R$	n-spacing		$\Omega = 2.00$	(J3-6a)		
	Nrows	1		Number of row	vs of bolts		
	Ncols	4		Number of bol	ts per row		
	d	0.75 in		Bolt diameter	46)		
	t	0.40 in		Thickness of m	aterial		
	Fu	65.00 ksi		Minimum tensi	ile stress of materia	l	
	L _{c-spacing}	2.19 in		Vertical distan	ce from edges of ad	ljacent holes	
	R _{n-spacing}	46.22 kips		Strength at spo	aces = min(R _{n-spacin}	ng-tearout ^e R _{n-}	
	Rnhearing	46 22 kips		Bearing = 2.4	*d*t*F,,		
	R _{n-spacing-tearout}	67.40 kips		Tear out at spa	aces = $1.2*L_{\odot}$	*t*F,	
	R_n/Ω	92.43 kips		Bolt bearing st	trength		
В	olt Bearing at Vert. I	Plate	15.00 kips	73.00 ki	ps 0.21	PASS	
	$R_n = N_{rows} + [R_{n-edge}]$	+(N _{cols} -1)*R _{n-spa}	_{cing}]	<u>Ω</u> = 2.00	(J3-6a)		
	Nrows	1		Number of row	vs of bolts		
	Ncols	4		Number of bol	ts per row		
	d	0.75 in		Bolt diameter			
	t	0.38 in		Thickness of m	aterial		
	Fu	58.00 ksi		Minimum tensi	ile stress of materia	1	
	L _{c-edge}	1.09 in		Vertical distan material	ce from edge of hol	e to edge of	
	L _{c-spacing}	2.19 in		Vertical distan	ce from edges of ad	ljacent holes	
	R _{n-edge}	28.55 kips		Strength at edg	ze = min(R _{n-edge-tea}	wour Rn-bearing)	
	R _{n-spacing}	39.15 kips		Strength at spa	nces = min(R _{n-spacin}	ng-tearout R _{n-}	
	R _{n-bearing}	39.15 kips		Bearing = 2.4	*d*t*F _u		
	R	28.55 kips		Tear out at edg	$ge = 1.2 * L_{condec} * t * L_{condec}$	F,,	
	R _{n-spacing teanout}	57.09 kips		Tear out at spa	$aces = 1.2*L_{acces}$	_*t*F,	
	R_n/Ω	73.00 kips		Bolt bearing st	trength	5 "	
B	olt Shear at Flange P	late	436.00 kin	s-in 996.561	cips-in 0.44	PASS	-
	$R_{p} = F_{pv} + A_{b} + N_{bolt}$	C	1	$\Omega = 2.00$	(J3-1)		
	F _{nv}	48.00 ksi		Shear stress N	type		
	A	0.44 in2		Area of holt			
	0	0.44 III		1			1

RISAConnection 1.1

2 Wilson Drive, Port Jefferso (C) 631-560-0259	on, NY 11777	alc. by AS	Date 5/15/2012	Chk'd by	Date	Sheet no./rev. 23 App'd by	of 81
2 Wilson Drive, Port Jefferso (C) 631-560-0259	on, NY 11777 Ca	alc. by AS	Date 5/15/2012	Chk'd by	Date	23 App'd by	of 81
2 Wilson Drive, Port Jefferso (C) 631-560-0259 Nbolt C R_b/Ω	Ca	alc. by AS	Date 5/15/2012	Chk'd by	Date	App'd by	
(C) 631-560-0259		AS 6	5/15/2012				Date
N _{bolt} C R _n /Ω		6					
N _{bolt} C R _n /Ω		6					
C R _n /Ω				Number of bolts			1
R_n/Ω		1.00		Eccentricity coef	fficient		
		63.62 kips		Bolt shear ruptu	re strength		
d _m		15.66 in		Moment arm bet	ween the flange for	ces	
M _n /Ω		996.56 kips-	in	$Moment = R_n * a$	d _m		
Bolt Bearing	g at Beam Flan	ge	436.00 kip	s-in 3326.35 k	ips-in 0.13	PASS	1
$R_n = N_r$	ows*[Rn-edge+(Nc	ols -1)*R _{n-space}	ing]	Ω = 2.00	(J3-6a)		
Nrows		2		Number of rows	of bolts		
Ncols		3		Number of bolts	per row		
d		0.75 in		Bolt diameter			
t		0.66 in		Thickness of mat	terial		
Fu		65.00 ksi		Minimum tensile	stress of material		
L _{c-edge}		1.09 in		Distance from ea	ige of hole to edge o	of material	
L _{c-spacin}	ng	2.19 in		Distance betwee	n adjacent hole edg	res	
R _{n-edge}		56.73 kips		Strength at edge	$= min(R_{n-edge-tearo})$	nut Rn-bearing)	
R _{n-spaci}	ng	77.80 kips		Strength at space	$es = min(R_{n-spacing})$	tearout Rn-	
R _{n-beari}	ng	77.80 kips		Bearing = 2.4*d	$*t*F_u$		
R _{n-edge}	tearout	56.73 kips		Tear out at edge	$= 1.2 * L_{c-edge} * t * F_t$	1	
R _{n-spaci}	ng-tearout	113.47 kips		Tear out at space	es = 1.2*L _{c-spacing} *	t*F _u	
R_n/Ω		212.34 kips		Bolt bearing stre	ingth		
dm		15.66 in		Moment arm bet	ween the flange for	ces	
M_n/Ω		3326.35 kip	s-in	$Moment = R_n * d$	d _m		
Bolt Bearin	g at Flange Plat	te	436.00 kip	s-in 2397.64 k	tips-in 0.18	PASS	
$R_n = N_r$	ows*[Rn-edge+(Nc	ols -1)*R _{n-spac}	ing]	$\Omega = 2.00$	(J3-6a)		
Nrows		2		Number of rows	of bolts		
Ncols		3		Number of bolts	per row		
d		0.75 in		Bolt diameter			
t		0.50 in		Thickness of mai	terial		
Fu		58.00 ksi		Minimum tensile	stress of material		
L _{c-edge}		1.09 in		Distance from ea	dge of hole to edge	of material	
L _{c-spaci}	ng	2.19 in		Distance betwee	n adjac <mark>e</mark> nt hole edg	res	
R _{n-edge}		38.06 kips		Strength at edge	$= min(R_{n-edge-tearc})$	nut Rn-bearing)	
R _{n-spaci}	ng	52.20 kips		Strength at space	es = min(R _{n-spacing}	-tearout Rn-	
R ,	62	52 20 kins		Bearing = $2.4*d$	***F		
R -	ing	38.06 bios		Tear out at edge	= 1.2*L . ***F		

_

A MARTIN MILES & Date of the second s	Project				Job Ref.		
Andustria	Section				Sheet no./rev.		
ASES					24 of 81		
son Drive, Port Jefferson, NY 11777	Calc. by D	late	Chk'd by	Date	App'd by	Date	
(C) 631-560-0259	AS	5/15/2012					
NECT							
R _{n-spacing-tearout}	76.13 kips		Tear out at space	ces = 1.2*L _{e-spacing}	*t*F _u		
R _n /Ω	142.46 kips		Bolt bearing strength				
dm	16.83 in		Moment arm between the flange forces				
M_n/Ω	2397.64 kips-	in	$Moment = R_n *$	d _m			
Beam Flange Block She	ar	436.00 kip	s-in 2872.681	kips-in 0.15	PASS		
$R_n = 2 * [min(0.6*F_1)]$	$_{a}^{\star}A_{av}, 0.6^{\star}F_{y}^{\star}A_{g}) +$	Ubs*Fu*Ant]	Ω = 2.00	(J4-5)			
Ag	4.99 in ²		Gross area subj	iect to shear			
A _{nv} 3.53 in ²			Net area subjec	t to shear			
Ubs	1.00		Uniform tension stress factor Net area subject to tension				
Ant	0.70 in ²						
Fu	F _u 65.00 ksi		Minimum tensile stress of material				
Fy	F _y 50.00 ksi		Minimum yield	stress of material			
R_n/Ω	R _n /Ω 183.38 kips		Block shear stre	ength			
dm	15.66 in		Moment arm between the flange forces				
$M_{\rm h}/\Omega$	2872.68 kips-	in	$Moment = R_n^*$	d _m			
Flange Plate Block She	ar	436.00 kip	s-in 1881.801	kips-in 0.23	PASS		
$R_n = 2 * [min(0.6*F_n)]$	$^{*}A_{nv}, 0.6*F_{y}*A_{g}) +$	Ubs*Fu*Ant]	Ω = 2.00	(J4-5)			
Ag	3.75 in ²		Gross area subj	lect to shear			
Anv	2.66 in ²		Net area subjec	t to shear			
Ubs	1.00		Uniform tension	n stress factor			
Ant	0.53 in ²		Net area subjec	t to tension			
Fu	58.00 ksi		Minimum tensil	le stress of material			
Fy	36.00 ksi		Minimum yield	stress of material			
R_n/Ω	111.81 kips		Block shear stre	ength			
dm	16.83 in		Moment arm be	tween the flange for	ces		
M_n/Ω	1881.80 kips-	in	$Moment = R_n *$	d _m			
Flange Plate Tearout		436.00 kip	s-in 2918.95	kips-in 0.15	PASS		
$\mathbf{R}_{\mathbf{n}} = [\min(0.6*\mathbf{F}_{\mathbf{u}}*\mathbf{A}$	$_{\rm nv}$, $0.6*F_y*A_g$) + U _b	*Fu*Ant]	$\Omega = 2.00$	(34-5)			
Ag	7.50 in ²		Gross area subj	ject to shear			
Anv	5.31 in ²		Net area subjec	t to shear			
Ubs	1.00		Uniform tension	n stress factor			
Ant	3.19 in ²		Net area subjec	rt to tension			
Fu	58.00 ksi		Minimum tensil	le stress of material			
Fy	36.00 ksi		Minimum yield	stress of material			
New York Control of Co	100 C 100 C 100 C 100 C			10.000			

a manufacture and a second	Project				Job Ref.	
ASES	Section		Sheet no./rev. 25. of 81			
A.S. Engineering Garaces P.C.					20	
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				

dm	16.83 in	Moment arm between the flange forces
M_n/Ω	2918.95 kips-in	$Moment = R_n * d_m$
Flange Plate Weld Strength	436.00	kips-in 1502.43 kips-in 0.29 PASS
$Rn/\Omega = 1.5 \pm 2 \pm \alpha \pm 0.928$	* C ₁ * D ₁₆ * w _p	
Double Fillet	3446C 1360 1515	
dm	16.83 in	Moment arm. $d_m = d_b + t_p$
a	0.78	Base material proration factor
c ₁	1.00	Electrode strength coefficient
D ₁₆	4.00	Weld fillet size in sixteenths of an inch
wp	10.25 in	Plate width
db	16.33 in	Beam overall depth
t _p	0.50 in	Plate thickness
Rn/Ω	89.27 kips	Weld strength
Mn/Ω	1502.43 kips-in	$Moment = R_n * d_m$
Flange Plate Tension Yield	436.00	kips-in 1859.36 kips-in 0.23 PASS
$R_n = F_v^* A_g$		$\Omega = 1.67$ (J4-1)
F	36.00 ksi	Minimum yield stress of material
A,	5.13 in ²	Gross area subject to shear
R /Ω	110.48 kips	Tensile vield strength
d	16.83 in	Moment arm between the flange forces
M_{u}/Ω	1859.36 kips-in	$Moment = R_n * d_m$
Beam Flange Tension Rupt	ure 436.00	kips-in 2872.68 kips-in 0.15 PASS
$R_n = F_u A_e$		$\Omega = 2.0$ (J4-2)
F.	65.00 ksi	Minimum tensile stress of material
A	5.64 in ²	Effective area subject to tension = $min(A_{u}, 0.85A_{a})$
A	6.81 in ²	Gross area subject to tension
A_	5.64 in ²	Net area subject to tension
R_/Ω	183 38 kins	Tensile moture strength
d_	15 66 in	Moment arm between the flange forces
M_/Ω	2872.68 kins-in	$Moment = R_{u} * d_{u}$
n		e are a viz Area a construction of the second se
Flange Plate Tension Ruptu	re 436.00	kips-in 2074.30 kips-in 0.21 PASS
$R_n = F_u * A_e$		$\Omega = 2.0$ (J4-2)
Fu	58.00 ksi	Minimum tensile stress of material
A _e	4.25 in ²	Effective area subject to tension = $min(A_{\eta'} 0.85A_g)$
Ag	5.13 in ²	Gross area subject to tension
	1 25 :2	Max man automatica terration

AND ALLING MARTIN DAVE	Project	Job Ref.				
A ANAL JULIA	Section				Sheet no./rev.	
ASES					26	of 8'
5. Engraving Bencer, PC Drive, Port Jefferson, NY 11777 (C) 631-560-0259	Calc. by AS	Date 5/15/2012	Chk'd by	Date	App'd by	Date
An						ĺ
R_n/Ω	123.25 kips		Tensile ruptur	e strength		
dm	16.83 in		Moment arm b	etween the flange for	rces	
M_n/Ω	2074.30 kip	s-in	$Moment = R_n$	* d _m		
Flange Plate Compressio	n				PASS	ĺ
Check condition : KL	r <= 25.0		(J4-6)			
K	0.65		Effective lengt	h factor		
L	2.00 in		Unbraced leng	ζth Marine		
r KL/r	9.01 9.01		Plate slendern	ess		
Column Flange Bending		436.00 kip	s-in 771.661	kips-in 0.57	PASS	Ì
$R_{n} = 6.25 \star t_{f}^{2} \star F_{y}$			$\Omega = 1.67$	(J10-1)		
dend	36.58 in		Distance from	concentrated force t	o top of column	
tf	0.50 in		Column flange	thickness		
F _v	50.00 ksi		Minimum yield			
R_n/Ω	45.85 kips		Column flange	local bending		
de	16.83 in		Moment arm f	rom centerlines force	ឋ	
M_n/Ω	771.66 kips-	in	$Moment = R_n$	* d _c		
Column Web Yielding		436.00 kip	s-in 946.941	kips-in 0.46	PASS	
$\mathbf{R}_{\mathbf{n}} = (5 \star \mathbf{k} + \mathbf{N}) \star \mathbf{F}_{\mathbf{y}} \star$	t _w		$\Omega = 1.50$	(J10-2)		
dend	36.58 in		Distance from	concentrated force t	o top of column	
d _{col}	8.12 in		Column depth			
k	0.89 in		Distance from toe of the filler	outer face of the flan	ige to the web	
N	1.00 in		Length of bear	ing		
F _y	50.00 ksi		Minimum yield	d stress <mark>of</mark> column		
t _w	0.31 in		Column web t	hickness		
R_n/Ω	56.26 kips		Column web l	ocal yielding		
d _c	16.83 in		Moment arm f	rom centerlines force	u	
M _n /Ω	946.94 kips-	in	$Moment = R_n$	* d _c		
Column Web Buckling		436.00 kip	s-in 1368.11	kips-in 0.32	PASS	1
$R_n = 24 + t_w^3 + (E + F_s)$.) ^{0.5} / h		Ω = 1.67	(J10-8)		
dend	36.58 in		Distance from	concentrated force t	o top of column	
d _{col}	8.12 in		Column depth			
t _w	0.31 in		Column web t	hickness		
Fy	50.00 ksi		Minimum yiel	d stress of column		
T	20000 00 1	- 1	14.1.1. 6.1	11 11 C 1		

AND AVER MENTER AND	Project	Job Ref.	Job Ref.			
A INL-MILA	Section				Sheet no./rev.	
ASES					27	of 81
Vilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
I				au - 100 P		Ĩ
h	6.34 in		Clear distance be	etween flanges h=0	d-2*k _{des}	
R _n /Ω	81.29 kips	1	Column web com	pression buckling		
d _c	16.83 in	1	Moment arm fron	n centerlines force	ប	
M _n /Ω	1368.11 kips	-in -	Moment = R _n * d	le .		
Column Web Crippling		436.00 kips-	in 1164.65 ki	ps-in 0.37	PASS	Ī
$R_n = 0.8 t_w^{2} (1+3)^{1/3}$	$(t_{col})^* (t_w/t_f)^{1.5})^* (E^{-1})^{1.5}$	$F_{y}^{*}t_{f}^{\prime}/t_{w}^{0.5}$	$\Omega = 2.00$	(J10-4)		
dend	36.58 in		Distance from co	ncentrated force t	o top of column	
N/d _{col}	0.12		Bearing length to	column depth rat	io	
d _{col}	8.12 in		Column depth			
t _w	0.31 in		Column web thici	kness		
t _f	0.50 in		Column flange th	ickness		
N	1.00 in		Length of bearing	3		
Fy	50.00 ksi	-	Minimum yield st	ress of column		
E	29000.00 ksi		Modulus of elasti	city of column		
R _n /Ω	69.20 kips		Column web crip	pling capacity		
d _c	16.83 in		Moment arm fron	n centerlines force	s	
M_n/Ω	1164.65 kips	-in -	Moment <mark>= R_n * d</mark>	c		
Column Panel Zone She	ar	436.00 kips-	in 761.04 kip	s-in 0.57	PASS	1
$\mathbf{R_n} = 0.60 * \mathbf{F_y} * \mathbf{d} * \mathbf{t_y}$	$(P_r <= 0.4 * P_c)$		$\Omega = 1.67$	(J10-9)		
Pr	30.00 kips	82	Axial force in the	column at the con	mection	
Pc	309.00 kips		$P_c = 0.6 * P_y = 0.6 *.$	F,*A		
Fy	50.00 ksi		Minimum yield st	rress of column		
A	10.30 in ²		Column cross-see	ctional area		
d	8.12 in		Column depth			
t _w	0.31 in		Column web thici	kness		
R_n/Ω	45.22 kips		Web panel zone c	apacity		
de	16.83 in		Moment arm from	n centerlines force	s.	
M_n/Ω	761.04 kips-i	n	Moment = $R_n * d_c$			



and the interest of the states	Project Job Ref.								
And state	0								
ACEC	Section		Sheet no./rev.	-1.04					
A.S. Engineering Services,P.C.					29	0181			
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by		Chk'd by	Date	Арр'а бу	Date			
(C) 631-560-0259	AS	5/15/2012							
Cross-sectional area of end posts;		A _e = 16.5 in ²							
Hole diameter;		Dia = 1 in							
Net cross-sectional area of end post	s;	A _{en} = 13.5 in ²							
Nominal collector size;		2 x 2" x 6"							
Dressed collector size;		2 x 1.5" x 5.5"							
Service condition;		Dry							
Temperature;		100 degF or le	SS						
From NDS Supplement Table 4A - F	Reference desid	n values for visu	ually graded d	imension lumbe	r (2" - 4" thick)				
Species, grade and size classification	on;	, Douglas Fir-La	rch, stud grad	e, 2" & wider					
Specific gravity;	,	G = 0.50	, j						
Tension parallel to grain;		$F_t = 450 \text{ lb/in}^2$							
Compression parallel to grain;		F _c = 850 lb/in ²							
Modulus of elasticity;		E = 1400000 lk	o/in ²						
Minimum modulus of elasticity;		E _{min} = 510000	lb/in ²						
Sheathing details									
Sheathing material:		7/16" wood pa	nel sheathin	a					
Fastener type:		8d common nails at 4" centers							
			Evenue Ohiere		and Develo				
From SDPWS Table 4.3A Nominal C	Unit Snear Cap	x = 760 lb/ft	-Frame Shear	vvalis - vvood-b	ased Panels				
Nominal unit shear capacity for w	ind docian:	$v_s = 700 \text{ Ib/ft}$							
Annarent shear wall shear stiffne	inu uesign,	$V_{\rm W} = 10005$ lb/lt G ₂ = 22 kins/ir							
	33,	$O_a = 22$ Kips/ii							
Loading details									
Dead load acting on top of panel;		D = 307 ID/It							
Rool live load acting on top of panel	,	$L_r = 223 \text{ ID/II}$							
Sell weight of parlet,	nanol:	$S_{wt} = 12$ ID/II							
In plane seismic load acting at head of	of papel.	V = 3.5 kips							
Seismic response coefficient:	or parier,	$C_q = 0.2$							
		U S – U.Z							
From IBC 2009 cl.1605.3.1 Basic loa	ad combination	S N/							
Load combination no.1;		D + W							
Load combination no.2;		D + 0.7E							
Load combination no.3;		D + 0.75VV + 0	$.75L_{\rm f} + 0.75L_{\rm r}$						
Load combination no.4;			0.70Lf + 0.70L	r					
Load combination no.6;									
		0.00 + 0.7 E							
Adjustment factors		0 1 00							
Load duration factor -1 able 2.3.2;		$C_{\rm D} = 1.60$							
Size factor for tension – Table 4A;	40.	$C_{Ft} = 1.00$							
Size factor for compression – Table	4A;	$C_{Fc} = 1.00$							
Wet service factor for tension – Tabl		$C_{Mt} = 1.00$							
Wet service factor for modulus of all	- Table 4A;	$G_{Mc} = 1.00$							
wet service factor for modulus of ela	aslicity – Table	4/1 Cure = 1 00							
		$C_{ME} = 1.00$							

and the Merry Designed	Project				Job Ref.				
A Anti Anti A	Section				Sheet no./rev.				
ASES					3	0 of 81			
AS bigming Server PC.	Calc. by	Date	Chk'd by	Date	App'd by	Date			
(C) 631-560-0259	AS	5/15/2012							
Temperature factor for tension – T	able 2.3.3;	$C_{tt} = 1.00$							
I emperature factor for compression	n - 1 able 2.3.3	$C_{tc} = 1.00$							
I emperature factor for modulus of	elasticity – Tat	ole 2.3.3							
Incising factor – cl 4 3 8		$C_{tE} = 1.00$							
Buckling stiffness factor $- cl 4 4 2^{\circ}$		$C_{T} = 1.00$							
Adjusted modulus of elasticity:		$E_{\rm min}' = E_{\rm min} \times C$		× C _T – 510000	nsi				
Critical buckling design value:		$E_{min} = E_{min} \times C$ $E_{rc} = 0.822 \times 10^{-10}$	$=$ $(h/d)^2$	~ 01 = 310000 - 1376 nei	psi				
Peference compression design value,	ue:	$F^* = F \times C_2 \times C_2$	$-\min \left(\prod u \right) $	- 1370 psi	nei				
For sawn lumber:	iue,			$F_{\rm C} \wedge C_{\rm I} = 1300$	psi				
Column stability factor – egn 3.7	′-1·	C = 0.0	/ F. [*])) / (2 ´ c		$(F_{*}^{*}))/(2 - c)^{2}$	· (E ₁ = / E ₁ *) / c)			
	-1,	= 0 70							
From CDDWC Table 4.2.4 Movimu		Accest Dation							
FIGHT SDPWS Table 4.3.4 Maximum	in Snear wair								
Segment 1 shear wall length:		5.5							
Shear wall aspect ratio:		$b_1 = 4.3$ fr							
Segment 2 shear wall length:		b ₂ = 5.5 ft							
Shear wall aspect ratio:		h / b ₂ = 1.455							
Segment 3 shear wall length;		$b_3 = 4$ ft							
Shear wall aspect ratio;		$h / b_3 = 2$							
Segmented shear wall capacity									
Maximum shear force under seism	ic loading:	$V_{s max} = 0.7 \times$	$(E_{a} + C_{s} \times (A))$	< S _{wt} + (D + +	L _r) × b)) = 4.213	kips			
Shear capacity for seismic loading	:	$V_s = V_s \times (b_1 + $	$b_2 + b_3) / 2 = 1$	5.32 kips					
		$V_{s max} / V_s = 0.$	792						
		PASS - Shear	capacity for	seismic load	exceeds maxim	um shear force			
Maximum shear force under wind	oading;	$V_{w_{max}} = W = 3$	3.3 kips						
Shear capacity for wind loading;		$V_w = v_w \times (b_1 +$	$V_w = v_w \times (b_1 + b_2 + b_3) / 2 = 7.455$ kips						
		$V_{w_{max}} / V_{w} = 0$.443						
		PASS - Sh	ear capacity	for wind load	exceeds maxim	um shear force			
Chord capacity for segment 1									
Shear wall aspect ratio;		h / b ₁ = 1.778							
Shear force for maximum tension;		$V = 0.7 \times (E_q +$	$-C_s \times (A \times S_{wt})$	+ (D + + L_r) ×	(ab)) = 4.213 kips				
Axial force for maximum tension;		$P = 0.6 \times (D + $	S _{wt} × h) = 24 1	.8 lb/ft					
Maximum tensile force in chord;		$T = V \times h / (b_1$	+ b ₂ + b ₃) - P	× b ₁ / 2 = 1.86	3 kips				
Maximum applied tensile stress;		$f_t = T / A_{en} = 13$	38 lb/in ²						
Design tensile stress;		$F_t' = F_t \times C_D \times$	$C_{Mt} \times C_{tt} \times C_{Ft}$	\times C _i = 720 lb/i	n ²				
		$f_t / F_t' = 0.192$							
		PASS - Desig	n tensile stre	ess exceeds n	naximum applied	d tensile stress			
Shear force for maximum compres	sion;	$V = 0.7 \times (E_q +$	$-C_s \times (A \times S_{wt})$	+ (D + + L_r) ×	(b)) = 4.213 kips				
Axial force for maximum compress	ion;	$P = (D + S_{wt} \times$	h) = 403 lb/ft						
Maximum compressive force in ch	ord;	$C = V \times h / (b_1$	$+ b_2 + b_3) + F$	$2 \times b_1 / 2 = 3.31$	14 kips				
Maximum applied compressive str	ess;	$f_{c} = C / A_{e} = 20$)1 lb/in ²		2				
Design compressive stress;		$F_c' = F_c \times C_D \times$	$C_{\text{Mc}} \times C_{\text{tc}} \times C$	$F_{C} \times C_{i} \times C_{P} = 9$	945 lb/in [∠]				

Summer and Man And	Project				Job Ref.		
ASES	Section					Sheet no./rev. 31 of 81	
112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259	Calc. by AS	Date 5/15/2012	Chk'd by	Date	App'd by	Date	

	f _c / F _c ' = 0.213
PASS	- Design compressive stress exceeds maximum applied compressive stress
Chord capacity for segment 2	
Shear wall aspect ratio;	h / b ₂ = 1.455
Shear force for maximum tension;	$V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_r) \times b)) = \textbf{4.213 kips}$
Axial force for maximum tension;	$P = 0.6 \times (D + S_{wt} \times h) = 241.8 \text{ lb/ft}$
Maximum tensile force in chord;	T = V \times h / (b ₁ + b ₂ + b ₃) - P \times b ₂ / 2 = 1.743 kips
Maximum applied tensile stress;	$f_t = T / A_{en} = 129 \text{ lb/in}^2$
Design tensile stress;	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \textbf{720} \text{ Ib/in}^2$
	f _t / F _t ' = 0.179
	PASS - Design tensile stress exceeds maximum applied tensile stress
Shear force for maximum compression;	$V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_r) \times b)) = \textbf{4.213 kips}$
Axial force for maximum compression;	$P = (D + S_{wt} \times h) = 403 \text{ lb/ft}$
Maximum compressive force in chord;	$C = V \times h / (b_1 + b_2 + b_3) + P \times b_2 / 2 = 3.516$ kips
Maximum applied compressive stress;	$f_c = C / A_e = 213 \text{ lb/in}^2$
Design compressive stress;	F_{c} ' = $F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} \times C_{P}$ = 945 lb/in ²
	f _c / F _c ' = 0.225
PASS	- Design compressive stress exceeds maximum applied compressive stress

Chord capacity for segment 3 Shear wall aspect ratio; Shear force for maximum tension; Axial force for maximum tension; Maximum tensile force in chord; Maximum applied tensile stress; Design tensile stress;

Shear force for maximum compression; Axial force for maximum compression; Maximum compressive force in chord; Maximum applied compressive stress; Design compressive stress;

Collector capacity Maximum shear force in collector; Unit shear above opening; Unit shear below opening; Maximum tensile force in collector; Maximum applied tensile stress; Design tensile stress;

$$\begin{split} h / b_3 &= 2 \\ V &= 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_r) \times b)) = \textbf{4.213} \text{ kips} \\ P &= 0.6 \times (D + S_{wt} \times h) = \textbf{241.8} \text{ lb/ft} \\ T &= V \times h / (b_1 + b_2 + b_3) - P \times b_3 / 2 = \textbf{1.924} \text{ kips} \\ f_t &= T / A_{en} = \textbf{143} \text{ lb/in}^2 \\ F_t' &= F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \textbf{720} \text{ lb/in}^2 \\ f_t / F_t' = \textbf{0.198} \\ \textbf{PASS - Design tensile stress exceeds maximum applied tensile stress} \\ V &= 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_r) \times b)) = \textbf{4.213} \text{ kips} \\ P &= (D + S_{wt} \times h) = \textbf{403} \text{ lb/ft} \\ C &= V \times h / (b_1 + b_2 + b_3) + P \times b_3 / 2 = \textbf{3.213} \text{ kips} \\ f_c &= C / A_e = \textbf{195} \text{ lb/in}^2 \\ F_c' &= F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \textbf{945} \text{ lb/in}^2 \\ f_c / F_c' &= \textbf{0.206} \end{split}$$

PASS - Design compressive stress exceeds maximum applied compressive stress

or;	$V_{max} = max(V_{s_max}, V_{w_max}) = 4.213 \text{ kips}$
	v _a = V _{max} / b = 165.219 lb/ft
	$v_b = V_{max} / (b_1 + b_2 + b_3) =$ 300.935 lb/ft
tor;	$T = b_1 \times v_b - P_{o1} \times v_a = 0.611$ kips
;	$f_t = T / (2 \times A_s) = 37 \text{ lb/in}^2$
	$F_t = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = 720 \text{ lb/in}^2$
	f _t / F _t ' = 0.051
	PASS - Design tensile stress exceeds maximum applied tensile stress

and and the set offer	Project				Job Ref.		
ACCO	Section				Sheet no./rev.		
AS. Engineering Starvices, P.C.					32	2 of 81	
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date	
(C) 631-560-0259	AS	5/15/2012					
Maximum compressive force in coll	ector;	$C = (P_{o1} + w_{o1})$	\times v _a - b ₁ \times v _b	= 0.794 kips			
Maximum applied compressive stre	SS;	$f_c = C / (2 \times A_s)$) = 48 lb/in ²				
Column stability factor;		C _P = 1.00					
Design compressive stress;		$F_c' = F_c \times C_D \times$	$C_{Mc} imes C_{tc} imes C_{tc}$	$=_{c} \times C_{i} \times C_{P} = 1$	1 360 lb/in ²		
		$f_c / F_c' = 0.035$					
	PASS - D	esign compressi	ive stress exc	eeds maximu	ım applied comp	ressive stress	
Deflection							
Design shear force;		$V = E_{q} + C_{s} \times ($	$A \times S_{wt} + (D +$	+ L _r) × b) = 6	.019 kips		
Induced unit shear;		$v = V / (b_1 + b_2)$	+ b ₃) = 429.9	08 lb/ft	8 lb/ft		
Vertical elongation of wall ancho	rage;	$D_a = 0.25$ in					
Shear wall deflection – Eqn. 4.3-1	•	d _{sw} = 8 ´ v ´ h	³ / (E ´ A _e ´ (l	D ₁ + b ₂ + b ₃)) +	v´h/(1000´G	i_a) + h \dot{D}_a /	
		(b ₁ + b ₂ + b ₃) =	= 0.208 in				
Deflection limit;		$\delta_{\text{limit}} = h / 240 =$	= 0.4 in				
		$\delta_{sw} / \delta_{limit} = 0.5$	21				
			PASS - Shea	ar wall deflect	ion is less than c	leflection limit	



INTERIOR MERTE DATE	Project				Job Ref.		
A-11-0-4							
ACCE	Section				Sheet no./rev.		
		1	1		34	of 81	
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date	
(C) 631-560-0259	AS	5/15/2012					
Total area of wall;		$A = h \times b - w_{o1}$	\times h _{o1} - w _{o2} \times h	₀₂ - w ₀₃ × h ₀₃ = 309	.99 ft ²		
Panel construction							
Nominal stud size:		2" x 6"					
Dressed stud size;		1.5" x 5.5"					
Cross-sectional area of studs;		$A_s = 8.25 \text{ in}^2$					
Stud spacing;		s = 16 in					
Nominal end post size;		2 x 2" x 6"					
Dressed end post size;		2 x 1.5" x 5.5"					
Cross-sectional area of end posts;		A _e = 16.5 in ²					
Hole diameter;		Dia = 1 in					
Net cross-sectional area of end post	s;	A _{en} = 13.5 in ²					
Nominal collector size;		2 x 2" x 6"					
Dressed collector size;		2 x 1.5" x 5.5"					
Service condition;		Dry					
Temperature;		100 degF or les	ss				
From NDS Supplement Table 4A - F	Reference desig	n values for visu	ally graded di	mension lumber (2	" - 4" thick)		
Species, grade and size classification	n;	Douglas Fir-La	rch, stud grade	e, 2" & wider			
Specific gravity;		G = 0.50	, U				
Tension parallel to grain;		$F_t = 450 \text{ lb/in}^2$					
Compression parallel to grain;		$F_{c} = 850 \text{ lb/in}^{2}$					
Modulus of elasticity;		E = 1400000 lb	/in ²				
Minimum modulus of elasticity;		E _{min} = 510000	b/in ²				
Sheathing details							
Sheathing material;		7/16" wood pa	nel sheathing	3			
Fastener type;		8d common n	ails at 4" cent	ers			
From SDPWS Table 4.3A Nominal U	Jnit Shear Capa	acities for Wood-	Frame Shear	Walls - Wood-base	ed Panels		
Nominal unit shear capacity for se	eismic design;	$v_s = 760 \text{ lb/ft}$					
Nominal unit shear capacity for w	ind design;	v _w = 1065 lb/ft					
Apparent shear wall shear stiffnes	ss;	G _a = 22 kips/ir	1				
Loading details							
Dead load acting on top of panel;		D = 50 lb/ft					
Roof live load acting on top of panel	;	$L_r = 50 \text{ lb/ft}$					
Self weight of panel;		$S_{wt} = 12 \text{ lb/ft}^2$					
In plane wind load acting at head of	panel;	W = 2.2 kips					
In plane seismic load acting at head	of panel;	E _q = 3 kips					
Seismic response coefficient;		$C_{s} = 0.2$					
From IBC 2009 cl.1605.3.1 Basic loa	ad combinations	6					
Load combination no.1;		D + W					
Load combination no.2;		D + 0.7E					
Load combination no.3;		D + 0.75W + 0.	75L _f + 0.75L _r				
Load combination no.4;		D + 0.525E + 0	.75L _f + 0.75L _r				
Load combination no.5;		0.6D + W					
Load combination no.6;		0.6D + 0.7E					

anorthing miners Days	Project				Job Ref.	
	Section			Sheet no./rev.		
		1		1	35	of 81
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
Adjustment factors						
Load duration factor – Table 2.3.2:		C _D = 1.60				
Size factor for tension – Table 4A:		C _{Ft} = 1.00				
Size factor for compression – Table	4A;	C _{Fc} = 1.00				
Wet service factor for tension – Tabl	e 4A;	C _{Mt} = 1.00				
Wet service factor for compression -	- Table 4A;	C _{Mc} = 1.00				
Wet service factor for modulus of ela	sticity – Table	4A				
		C _{ME} = 1.00				
Temperature factor for tension – Tab	ole 2.3.3;	$C_{tt} = 1.00$				
Temperature factor for compression	– Table 2.3.3;	C _{tc} = 1.00				
Temperature factor for modulus of e	lasticity – Table	2.3.3				
		C _{tE} = 1.00				
Incising factor – cl.4.3.8;		$C_i = \textbf{1.00}$				
Buckling stiffness factor – cl.4.4.2;		$C_{T} = 1.00$				
Adjusted modulus of elasticity;		$E_{min}' = E_{min} \times C$	$C_{\text{ME}} \times C_{\text{tE}} \times C_{\text{i}} \times C_{\text{i}}$	C _T = 510000 psi		
Critical buckling design value;		$F_{cE} = 0.822 \times E$	$E_{min}' / (h / d)^2 = 4$	49 psi		
Reference compression design value	e;	$F_c^* = F_c \times C_D \times$	$C_{Mc} \times C_{tc} \times C_{Fc}$	× C _i = 1360 psi		
For sawn lumber;		c = 0.8				
Column stability factor – eqn.3.7-1	l;	$C_{P} = (1 + (F_{cE}))$	/ F _c *)) / (2 ´ c) –	Ö([(1 + (F _{cE} / F _c *)) / (2 ´ c)] ² - (l	F _{cE} / F _c *) / c)
		= 0.30				
From SDPWS Table 4.3.4 Maximum	Shear Wall As	pect Ratios				
Maximum shear wall aspect ratio:		3.5				
Segment 1 shear wall length;		b ₁ = 4.5 ft				
Shear wall aspect ratio;		h / b ₁ = 3.111				
Segment 2 shear wall length;		b ₂ = 4.5 ft				
Shear wall aspect ratio;		h / b ₂ = 3.111				
Segment 3 shear wall length;		b ₃ = 5.5 ft				
Shear wall aspect ratio;		h / b ₃ = 2.545				
Segment 4 shear wall length;		b ₄ = 2.5 ft				
Shear wall aspect ratio;		h / b ₄ = 5.6				
Segmented shear wall capacity						
Maximum shear force under seismic	loading;	$V_{s max} = 0.7 \times 0.10$	$E_{\alpha} + C_{s} \times (A \times S)$	$S_{wt} + (D + + L_r) \times$	b)) = 2.971 kip	S
Shear capacity for seismic loading:	5,	$V_s = v_s \times (2 \times b)$	$b_1^2 / h + 2 \times b_2^2 /$	$h + 2 \times b_3^2 / h) / 2$	2 = 3.841 kips	
, , , , , , , , , , , , , , , , , , ,		$V_{s max} / V_{s} = 0.$	773			
		PASS - Shear	capacity for se	ismic load exce	eds maximun	n shear force
Maximum shear force under wind loa	ading;	$V_{w max} = W = 2$. 2 kips			
Shear capacity for wind loading:	0	$V_w = v_w \times (2 \times 1)$	$b_{1}^{2}/h + 2 \times b_{2}^{2}/h$	$/h + 2 \times b_3^2 / h) /$	2 = 5.382 kips	
		$V_{w max} / V_{w} = 0$.409	- ,	·	
		PASS - She	ear capacity for	wind load exce	eds maximun	n shear force
Chord capacity for segment 1						
Shear wall aspect ratio		h / b₁ = 3₋111				
Shear force for maximum tension:		$V = 0.7 \times (F_{-} +$	C × (A × S +	$(D + + -) \times h)) -$	2.971 kins	
Axial force for maximum tension:		$P = 0.6 \times (D + 1)$	$S_{wt} \times h$ – 130 g	$(\underline{b}, \underline{b}, \underline{b}) = (\underline{b}, \underline{b})$		
		. – 0.0 × (D +		10/11		

SectionSectionSection12 Witeo Dres, Port Jetter no. rev.36 of 81Carle by ASDateAge to by Stit/2012Carle by ASDateAge to by Asia force for maximum compression; $F_1 = F_1 < C_0 $	and the state of t	Project				Job Ref.		
Image: contract of the set	a tablestate	Section				Sheet no./rev	·	
12 Windo Dire, Port Jefferson, NY 11777Cat: by ASDateApple by bitsDate12 Windo Dire, Port Jefferson, NY 11777 (c) attractoriesCat: by ASDateApple by bitsDateMaximum tensile force in chord; Maximum applied tensile stress; Design tensile stress;T = V × h / (b; + b ₂ + b ₃) · P × b ₁ / 2 = 2.574 kips Fi = Fi × C ₀ + C ₁ = 2.574 kips Fi = Fi × C ₀ + C ₁ > 2.971 kips PASS - Design tensile stress exceeds maximum applied tensile stress fi = C / A ₀ = 204 lb/n² fi / Fi = 0.265 PASS - Design compressive stress; Fi = Fi × C ₀ + 2 = 3.359 kips fi = C / A ₀ = 204 lb/n² fi / Fi = 0.264 fi = C = V × h / (b + b ₂ + b ₃) + P × b / 2 = 3.359 kips fi = C / A ₀ = 204 lb/n² fi / Fi = 0.263Design compressive stress; Design compressive stress; Chord capacity for segment 2 Shear valia spect ratio; Naximum tensile force in romaximum tension; Asaid force for maximum tension; P = 0.6 × (D + b ₂ + b ₃) · P × b ₂ / 2 = 2.574 kips fi = T / A ₀ = 191 lb/n² fi / Fi = 0.265 PASS - Design tensile stress; Fi = Fi × C ₀ = 720 lb/n² fi / Fi = 0.493PASS - Design tensile stress; Design tensile stress; Shear force for maximum compression; P = (0 + C ₀ × C + V ₀ × C ₀ + C ₀ ×	ASES					3	36 of 81	
NormalityAS5/15/2012Maximum tensile force in chord; Maximum applied tensile stress; $T = V \times h / (b_1 + b_2 + b_3) - P \times b_1 / 2 = 2.574$ kips $f_1 = T / A_{en} = 191 bin^2$ $f_1 / F_1 = 0.265$ PASS - Design tensile stress exceeds maximum applied tensile stressShear force for maximum compression; Maximum compressive stress; $P = (D + S_{ex} \times h) = 2.871 kips$ $P = (D + S_{ex} \times h) = 2.18 lbftMaximum compressive stress;Design tensile stress stress exceeds maximum applied compressive stress;F_1 = F_1 \times C_0 \times S_{ex} \times C_{ex} \times C_e \times $	AS Connecting Service PC	Calc. by	Date	Chk'd by	Date	App'd by	Date	
Maximum tensile force in chord; Maximum applied tensile stress; $T = V \times h / (b_1 + b_2 + b_3) - P \times b_1 / 2 = 2.574$ kips $f_1 = T / A_{em} = 191 lbin^2$ $F_1 = F_1 \times C_0 \times C_{bk} \times C_n \times C_n \times C_n + C_n = 720 lbin^2$ $(1/F_1 = 0.265)$ Design tensile stress; $F_1 = F_1 \times C_0 \times C_{bk} \times C_n \times C_n \times C_n + (D + L_n) \times b) = 2.971$ kips Axial force for maximum compression; $V = 0.7 \times (E_n + C_n \times (A \times S_{m} + (D + L_n) \times b)) = 2.971$ kips Axial force for maximum compressive stress; $L_n = C / A_n = 204$ lb/m² $L_n / F_n = 0.493$ Design compressive stress; $F_n = F_n \times C_n \times C_m \times C_m \times C_n \times C_$	(C) 631-560-0259	AS	5/15/2012					
Maximum applied tensile stress: $f_1 = T / A_{en} = 191 \text{ lb/m}^2$ Design tensile stress: $F_1 = F_1 \times C_0 \times C_{la} \times C_0 \times C_1 \times C_1 \times C_2 = 720 \text{ lb/m}^2$ $I_1 / F_1 = 0.265$ PASS - Design tensile stress exceeds maximum applied tensile stress Shear force for maximum compression: $V = 0.7 \times (E_q + C_q \times (A \times S_{at} + (D + L) \times b)) = 2.971 \text{ kips}$ Maximum applied compressive stress; $E_1 < I / A_{en} = 204 \text{ lb/m}^2$ Design compressive stress; $E_1 < I / A_{en} = 204 \text{ lb/m}^2$ Design compressive stress; $E_1 < I / A_{en} = 204 \text{ lb/m}^2$ Design compressive stress; $E_1 < I / A_{en} = 204 \text{ lb/m}^2$ Design compressive stress; $E_1 < I / A_{en} = 204 \text{ lb/m}^2$ Design compressive stress; $E_1 < I / A_{en} = 204 \text{ lb/m}^2$ Design compressive stress; $E_1 < I / A_{en} = 204 \text{ lb/m}^2$ Shear wall aspect ratio; $h / b_2 = 3.111$ Shear vall aspect ratio; $h / b_2 = 3.111$ Shear off core in maximum tension; $P = 0.5 \times C_{bn} \times C (a \times C_{bn} \times C_{bn} = 202 \text{ lb/m}^2$ Maximum applied tensile stress; $E_1 = T / A_{en} = 191 \text{ lb/m}^2$ Design tensile stress; $E_1 = T / A_{en} = 191 \text{ lb/m}^2$ Design tensile stress; $E_1 = T / A_{en} = 191 \text{ lb/m}^2$ Axial force for maximum compression; $V = 0.7 \times (E_q + C_q \times (A \times S_{at} + (D + L) \times b)) = 2.971 \text{ kips}$ Axial force for maximum compression; $V = 0.7 \times (E_q + C_q \times (A \times S_{at} + (D + L) \times b)) = 2.971 \text{ kips}$ Axial force for maximum compression; $V = 0.7 \times (E_q + C_q \times (A \times S_{at} + (D + L) \times b)) = 2.971 \text{ kips}$ Axial force for maximum compressio	Maximum tensile force in chord;		$T = V \times h / (b \cdot$	1 + b ₂ + b ₃) - P	×b ₁ / 2 = 2.57	4 kips		
Design tensile stress; $F_i^c = F_i \times C_0 \times C_{hk} \times C_n \times C_i = 720 \text{ lb/m}^2$ $f_i / F_i^c = 0.265$ PASS - Design tensile stress exceeds maximum applied tensile stress Shear force for maximum compression; $P = (D + S_n \times h) = 218 \text{ lb/lt}$ Maximum compressive stress; $f_c = C / A_n = 204 \text{ lb/m}^2$ Design compressive stress; $f_c = C / A_n = 204 \text{ lb/m}^2$ Design compressive stress; $f_c = C / A_n = 204 \text{ lb/m}^2$ Design compressive stress; $f_c = C / A_n = 204 \text{ lb/m}^2$ Design compressive stress; $f_c = C / A_n = 204 \text{ lb/m}^2$ Design compressive stress; $f_c = C / A_n = 204 \text{ lb/m}^2$ Design compressive stress; $f_c = C / A_n = 204 \text{ lb/m}^2$ Design compressive stress; $f_c = C / A_n = 204 \text{ lb/m}^2$ Design compressive stress; $f_c = C / A_n = 204 \text{ lb/m}^2$ Design compressive stress; $f_c = C / A_n = 204 \text{ lb/m}^2$ Design compressive stress; $F_c^c = F_n \times C_0 \times C_{hk} \times C_{h$	Maximum applied tensile stress;		$f_t = T / A_{en} = 1$	91 lb/in ²				
$ f_1/F_1 = 0.265 $ $ PASS - Design tensile stress exceeds maximum applied tensile stress exceeds maximum applied tensile stress exceeds maximum compression; P = (D + S_{st} \times h) = 218 b/lt Maximum compressive force in chord; C = V \times h/(b_1 + b_2 + b_3) + P \times h/(2 + 3.359 kips Maximum applied compressive stress; f_c = C/A_c = 204 bin^2 Design compressive stress; F_c^+ = F_c \times C_0 \times C_{bt} \times C_{b$	Design tensile stress;		$F_t' = F_t \times C_D \times$	$c C_{Mt} imes C_{tt} imes C_F$	$t_t \times C_i = 720$ lb/i	n ²		
PASS - Design tensile stress exceeds maximum applied tensile stressShear force for maximum compression; $V = 0.7 \times (E_n + C_n \times (A \times S_{nt} + (D + + L_1) \times b)) = 2.971$ kipsMaximum compressive stress; $E = C / A_n = 204$ lb/n²Maximum applied compressive stress; $E_n \in C / A_n = 204$ lb/n²Design compressive stress; $E_n \in C / A_n = 204$ lb/n²Design compressive stress; $E_n \in C / A_n = 204$ lb/n²Design compressive stress; $E_n \in C / A_n = 204$ lb/n²Design compressive stress; $E_n \in C / A_n = 204$ lb/n²Design compressive stress; $E_n \in C / A_n = 204$ lb/n²Design compressive stress; $E_n \in C / A_n = 204$ lb/n²Chord capacity for segment 2h / $b_2 = 3.111$ Shear vall aspect ratic;h / $b_2 = 3.114$ Shear vall aspect ratic;h / $b_2 = 3.114$ Shear vall aspect ratic;h / $b_2 = 4.214$ Maximum tension; $V = 0.7 \times (E_n + C_n \times (A \times S_{nt} + (D + + L_1) \times b)) = 2.971$ kipsAxial force for maximum tension; $P = 0.6 \times (D + S_{nt} \times h) = 130.8$ lb/ntMaximum tensile force in chord; $T = V \times h / (b_1 + b_2 + b_1) + V > b_2 / 2 = 2.574$ kipsMaximum applied tensile stress; $E_n \in C / A_n \times S_n \times (D_n + C_n + D_n) + D > b_2 / 2 = 2.574$ kipsMaximum compression; $V = 0.7 \times (E_n + C_n \times (A \times S_{nt} + (D + + L_1) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_n + C_n \times (A \times S_{nt} + (D + + L_1) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_n + C_n \times (A \times S_{nt} + (D + + L_1) \times b)) = 2.971$ kipsAxial force for maximum tension;			$f_t / F_t' = 0.265$					
Shear force for maximum compression; $V = 0.7 \times (E_q + C_x \times (A \times S_{st} + (D + L_u) \times b)) = 2.971$ kipsAxial force for maximum compression; $P = (D + S_{st} \times h) = 218$ lb/ttMaximum compressive stress; $f_c = C / A_c = 204$ lb/n²Design compressive stress; $F_c = F_c \times C_0 \times C_{bb} \times C_c \times C_c \times C_c = 413$ lb/n² $I_c / F_c' = 0.493$ $PASS - Design compressive stress exceeds maximum applied compressive stressChord capacity for segment 2h / b_c = 3.111Shear wall aspect ratio;h / b_c = 3.111Shear force for maximum tension;\Psi = 0.5 \times (C + S_{at} \times h) = 130.8 lb/ttMaximum applied tensile stress;f_i = T / A_{an} = 191 lb/n²Maximum applied tensile stress;f_i = T / A_{an} = 191 lb/n²Maximum compressive force in chord;T = V \times h / (b_1 + b_2 + b_2) + P \times b_2 / 2 = 2.574 kipsMaximum applied tensile stress;f_i = T / A_{an} = 191 lb/n²Maximum compressive force in chord;T = V \times h / (b_1 + b_2 + b_2) + P \times b_2 / 2 = 2.574 kipsMaximum compressive force in chord;T = V \times h / (b_1 + b_2 + b_2) + P \times b_2 / 2 = 2.574 kipsMaximum compressive force in chord;T = V \times h / (b_1 + b_2 + b_2) + P \times b_2 / 2 = 2.574 kipsMaximum compressive force in chord;T = V \times h / (b_1 + b_2 + b_2) + P \times b_2 / 2 = 3.359 kipsMaximum compressive force in chord;T = V \times h / (b_1 + b_2 + b_2) + P \times b_2 / 2 = 3.359 kipsMaximum applied compressive stress;F_0 = C A_{ac} > 204 lb/n²Maximum applied compressive stress;F_0 = C A_{ac} > C + b_{ac} \times C + C \times C_{bc} \times C_{bc} \times C_{bc} = 2.515Shear wall aspect ratio;h / b_$			PASS - Desi	gn tensile str	ess exceeds n	naximum applie	d tensile stress	
Axial force for maximum compression; $P = (D + S_{ex} \times h) = 218$ lb/ltMaximum compressive force in chord; $C = V \times h/(b_1 + b_2 + b_3) + P \times b_1/2 = 3.359$ kipsMaximum applied compressive stress; $I_c = C / A_e = 204$ lb/n²Design compressive stress; $I_c = F_c \times C_0 \times C_{bx} \times$	Shear force for maximum compress	sion;	$V = 0.7 imes (E_q$	+ $C_s \times (A \times S_w$	$_{rt}$ + (D + + L _r) ×	b)) = 2.971 kips		
Maximum compressive force in chord; $C = V \times h / (b_1 + b_2 + b_3) + P \times b_1 / 2 = 3.359 kips$ Maximum applied compressive stress; $I_c = C / A_o = 204 lbin^2$ Design compressive stress; $I_c = C / A_o = 204 lbin^2$ $I_c / F_c^2 = 0.493$ $PASS - Design compressive stress exceeds maximum applied compressive stressChord capacity for segment 2h / b_2 = 3.111Shear valia aspect ratio;h / b_2 = 3.111Shear valia aspect ratio;h / b_2 = 3.111Shear valia force for maximum tension;V = 0.7 \times (E_q + C_s \times (A \times S_{str} + (D + + L_i) \times b)) = 2.971 kipsAxial force for maximum tension;P = 0.6 \times (D + S_{str} \times h) = 130.8 lb/tMaximum applied tensile stress;I_i = T / A_m = 191 lbin^2Design tensile stress;I_i = T \times D_i / C_i \times C_i \times C_i \times C_i = 22.574 kipsMaximum applied compression;P = 0.6 \times (D + S_{str} \times h) = 130.8 lb/tAxial force for maximum compression;V = 0.7 \times (E_q + C_s \times (A \times S_{str} + (D + + L_i) \times b)) = 2.971 kipsAxial force for maximum compression;V = 0.7 \times (E_q + C_s \times (A \times S_{str} + (D + + L_i) \times b)) = 2.971 kipsAxial force for maximum compression;V = 0.7 \times (E_q + C_s \times (A \times S_{str} + (D + + L_i) \times b)) = 2.971 kipsPaisor compressive stress;I_c \subset I = V \times h / (b_1 + b_2 + b_3) + P \times b_2 / 2 = 3.359 kipsAxial force for maximum compression;V = 0.7 \times (E_q + C_s \times (A \times S_{str} + (D + + L_i) \times b)) = 2.971 kipsPASS - Design compressive stress;I_c \subset I = C_{str} \times D_s / C_s \times C_s$	Axial force for maximum compressi	on;	$P = (D + S_{wt})$	< h) = 218 lb/ft				
Maximum applied compressive stress; $f_c = C / A_e = 204 \ lb/ln^2$ Design compressive stress; $F_c' = F_c \times C_0 \times C_{loc} \times C_{rc} \times C_r \times C_c = 413 \ lb/ln^2$ $f_c / F_c' = 0.493$ PASS - Design compressive stress exceeds maximum applied compressive stress Chord capacity for segment 2Shear wall aspect ratio; $h / b_2 = 3.111$ Shear force for maximum tension; $V = 0.7 \times (E_q + C_a \times (A \times S_{wl} + (D + + L_l) \times b)) = 2.971 \ kips$ Axial force for maximum tension; $V = 0.7 \times (E_q + C_a \times (A \times S_{wl} + (D + + L_l) \times b)) = 2.971 \ kips$ Maximum applied tensile stress; $f_i = T / A_{en} = 191 \ lb/ln^2$ Design tensile stress; $f_i = T / A_{en} = 191 \ lb/ln^2$ Notice for maximum compression; $V = 0.7 \times (E_q + C_a \times (A \times S_{wl} + (D + + L_l) \times b)) = 2.971 \ kips$ Axial force for maximum compression; $V = 0.7 \times (E_q + C_a \times (A \times S_w + (D + + L_l) \times b)) = 2.971 \ kips$ Axial force for maximum compression; $V = 0.7 \times (E_q + C_a \times (A \times S_w + (D + + L_l) \times b)) = 2.971 \ kips$ Axial force for maximum tempression; $V = 0.7 \times (E_q + C_a \times (A \times S_w + (D + + L_l) \times b)) = 2.971 \ kips$ Axial force for maximum tension; $V = 0.7 \times (E_q + C_a \times (A \times S_w + (D + + L_l) \times b)) = 2.971 \ kips$ PASS - Design compressive stress; $F_c' = F_c \times C_0 \times C_{loc} \times C_{loc} \times C_{loc} \times C_{loc} = 241 \ lb/ln^2$ tay for the maximum tension; $V = 0.7 \times (E_q + C_a \times (A \times S_w + (D + + L_l) \times b)) = 2.971 \ kips$ Axial force for maximum tension; $V = 0.7 \times (E_q + C_a \times (A \times S_w + (D + + L_l) \times b)) = 2.971 \ kips$ Axial force for maximum tension; $V = 0.7 \times (E_q + C_a \times (A \times S_w + (D + + L_l) \times b)) = 2.971 \ kips$ <td>Maximum compressive force in cho</td> <td>rd;</td> <td>$C = V \times h / (b$</td> <td>1 + b2 + b3) + F</td> <td>$P \times b_1 / 2 = 3.35$</td> <td>59 kips</td> <td></td>	Maximum compressive force in cho	rd;	$C = V \times h / (b$	1 + b2 + b3) + F	$P \times b_1 / 2 = 3.35$	5 9 kips		
Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_b \times C_b \times C_b \times C_p = 413 \text{ lb/m}^2$ $f_c/F_c' = 0.493$ $PASS - Design compressive stress exceeds maximum applied compressive stress Chord capacity for segment 2 Shear force for maximum tension; V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971 \text{ kips}Axial force for maximum tension;P = 0.6 \times (0 + S_{wt} \times b) = 130.6 \text{ lb/lt}Maximum applied tensile stress;f_t = 1 / A_{an} = 191 \text{ lb/m}^2Design tensile stress;F_t' = F_t \times C_D \times C_{bk} \times C_t \times C_T \times C_T = 720 \text{ lb/m}^2f_t/F_t' = 0.285PASS - Design tensile stress exceeds maximum applied tensile stress Shear force for maximum compression; V = 0.7 \times (E_q + C_s \times (A \times S_w + (D + + L_t) \times b)) = 2.971 \text{ kips}Axial force for maximum compression;V = 0.7 \times (E_q + C_s \times (A \times S_w + (D + + L_t) \times b)) = 2.971 \text{ kips}Pass - Design tensile stress exceeds maximum applied tensile stress Shear force for maximum compression; V = 0.7 \times (E_q + C_s \times (A \times S_w + (D + + L_t) \times b)) = 2.971 \text{ kips}Pass - Design compressive stress;F_c' = F_c \times C_D \times C_{bk} \times C_b \times C_c \times C_c \times C_t \times C_p = 413 \text{ lb/m}^2f_c/F_c' = 0.493PASS - Design compressive stress exceeds maximum applied compressive stress Chord capacity for segment 3 Shear force for maximum tension; Axial force for maximum tension; P = 0.6 \times (D + S_{wt} \times h) = 130.8 \text{ lb/ft}Maximum applied tensile stress;f_t = T/A_{an} = 188 \text{ lb/m}^2Pass - Design tensile stress exceeds maximum applied tensile stress Shear force for maximum tension; P = 0.6 \times (D + S_{wt} \times h) = 2.971 \text{ kips}Pass - Design tensile stress;f_t/F_t' = 0.258PASS - Design tensile stress exceeds maximum applied tensile stress Shear force for maximum tension; P = (D + S_{wt} \times h) = 2.8 \text{ lb/ft}Maximum applied tensile stress;f_t = T/A_{an} = 188 \text{ lb/ft}Maxi$	Maximum applied compressive stre	SS;	$f_c = C / A_e = 2$	04 lb/in ²				
$f_c / F_c' = 0.493$ $PASS - Design compressive stress exceeds maximum applied compressive stress Chord capacity for segment 2 Shear value aspect ratio; h / b_2 = 3.111 Shear force for maximum tension; V = 0.7 \times (E_n + C_n \times (A \times S_{wt} + (D + + L) \times b)) = 2.971 \text{ kips} Axial force for maximum tension; P = 0.6 \times (D + S_{wt} \times h) = 130.8 \text{ lb/lt} Maximum tensile force in chord; T = V \times h / (b_1 + b_2 + b_3) + P \times b_2 / 2 = 2.574 \text{ kips} Maximum applied tensile stress; f_1 = T / A_{ann} = 191 \text{ lb/ln}^2 Design tensile stress; F_1 = T / A_{ann} = 191 \text{ lb/ln}^2 Design tensile stress; PASS - Design tensile stress exceeds maximum applied tensile stress exceeds maximum applied tensile stress; PASS - Design tensile stress; Pars - 204 \text{ lb/ln}^2 Maximum compressive stress; F_1 = C / A_a = 204 \text{ lb/ln}^2 F_1 = F_a \times 204 \text{ lb/ln}^2 F_1 = F_a < 204 \text{ lb/ln}^2 PASS - Design compressive stress exceeds maximum applied compressive stress exceeds maximum applied compressive stress; F_1 = F_1 \times C_0 \times C_{hc} \times C_{hc} \times C_h $	Design compressive stress;		$F_c' = F_c \times C_D$	\times C _{Mc} \times C _{tc} \times C	$C_{Fc} \times C_i \times C_P = 4$	113 lb/in ²		
PASS - Design compressive stress exceeds maximum applied compressive stressChord capacity for segment 2Shear wall aspect ratio;h / b ₂ = 3.111Shear force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{stt} + (D + L_i) \times b)) = 2.971$ kipsAxial force for maximum tension; $P = 0.6 \times (D + S_{stt} \times h) = 130.8$ lb/ltMaximum tensile force in chord; $T = V \times h / (b_1 + b_2 + b_3) \cdot P \times b_2 / 2 = 2.574$ kipsMaximum applied tensile stress; $f_t = 7 / A_{an} = 191$ lb/ln ² Design tensile stress; $f_t = 7 / A_{an} = 191$ lb/ln ² Axial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{stt} + (D + + L_s) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{stt} + (D + + L_s) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{stt} + (D + + L_s) \times b)) = 2.971$ kipsMaximum applied compressive stress; $f_e = C / A_e = 204$ lb/ln ² Maximum applied compressive stress; $F_c^2 = F_c \times C_D \times C_{Mc} \times C_c \times C_c \times C_c \times C_c \times C_c = 413$ lb/ln ² tay, f_c = 0.493h / b_3 = 2.545Shear force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{stt} + (D + + L_s) \times b)) = 2.971$ kipsAxial force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{stt} + (D + + L_s) \times b)) = 2.971$ kipsAxial force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{stt} + (D + + L_s) \times b)) = 2.971$ kipsAxial force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{stt} + (D + + L_s) \times b)) = 2.971$ kipsAxial force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{stt} + (D + + L_s) \times$			$f_c / F_c' = 0.493$	3				
Chord capacity for segment 2 Shear wall aspect ratic; Shear force for maximum tension; Axial force for maximum tension; Maximum applied tensile stress; Shear force for maximum compression; Axial force for maximum compression; Axial force for maximum tension; P = 0.6 × (0 + S _{wt} × h) = 130.8 lb/ft T = V × h / (b ₁ + b ₂ + b ₃) - P × b ₂ / 2 = 2.574 kips f ₁ = T / A _{wn} = 191 lb/m ² F ₁ ¹ = F ₁ × C _D × C _{Mt} × C _R × C _R = C _R × C _R = 720 lb/in ² f ₁ / F ₁ ¹ = 0.265 PASS - Design tensile stress exceeds maximum applied tensile stress Shear force for maximum compression; Axial force for maximum compression; Axial force for maximum compression; P = (D + S _m × h) = 218 lb/ft C = V × h / (b ₁ + b ₂ + b ₃) + P × b ₂ / 2 = 3.359 kips f ₆ = C / A _e = 204 lb/in ² F ₆ ² = F ₂ × C _D × C _{Mt} × C _R × C _R × C _R = C ₁ × C _P = 413 lb/in ² f ₆ / F ₆ ² = 0.493 PASS - Design compressive stress; Chord capacity for segment 3 Shear force for maximum tension; Axial force for maximum tension; P = 0.6 × (D + S _{wt} × h) = 130.8 lb/ft Maximum applied tensile stress; F ₁ = T / A _{en} = 186 lb/m ² F ₁ + T / K ₂ = 0.2509 kips f ₁ = T / A _{en} = 186 lb/m ² F ₁ + T / A _{en} = 186 lb/m ² F ₁ + T / A _{en} = 186 lb/m ² F ₁ = C / A _e = 210 lb/ln ² f ₁ / F ₁ = 0.258 PASS - Design tensile stress exceeds maximum applied tensile stress Shear force for maximum compression; P = (D + S _{wt} × h) = 218 lb/ft Axial force for maximum compression; P = (D + S _{wt} × h) = 218 lb/lt Maximum compressive stress; F ₁ = T / A _{en} = 210 lb/ln ² F ₁ = F ₁ × C _D × C _{Mt} × C ₁ = 2 = 3.468 kips Maximum applied com		PASS - D	esign compress	sive stress ex	ceeds maximu	im applied com	pressive stress	
Shear wall aspect ratio;h / b_2 = 3.111Shear force for maximum tension; $V = 0.7 \times [E_n + C_n \times (A \times S_{nt} + (D + +L_1) \times b)] = 2.971$ kipsAxial force for maximum tension; $P = 0.6 \times (D + S_{nt} \times h) = 130.8$ lb/ftMaximum tensile force in chord; $T = V \times h / (b_1 + b_2 + b_3) + P \times b_2 / 2 = 2.574$ kipsMaximum applied tensile stress; $f_i = T / A_{n-1} = 191$ lb/n²Design tensile stress; $F_i = F_i \times C_D \times C_{ht} \times C_n \times C$	Chord capacity for segment 2							
Shear force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_l) \times b)) = 2.971$ kipsAxial force for maximum tension; $P = 0.6 \times (D + S_{wt} \times h) = 130.8$ lb/ftMaximum applied tensile stress; $T = V \times h / (b_1 + b_2 + b_3) - P \times b_2 / 2 = 2.574$ kipsMaximum applied tensile stress; $F_1^i = F_1 \times C_0 \times (M_1 \times C_{H} \times $	Shear wall aspect ratio;		h / b ₂ = 3.111					
Axial force for maximum tension; $P = 0.6 \times (D + S_{stt} \times h) = 130.8$ lb/ftMaximum tensile force in chord; $T = V \times h / (b_1 + b_2 + b_3) - P \times b_2 / 2 = 2.574$ kipsMaximum applied tensile stress; $f_i = T / A_{en} = 191$ lb/n²Design tensile stress; $f_i = T_i \times D_0 \times C_{hh} \times C_{fr} \times C_{fr} \times C_{fr} = 720$ lb/in²Axial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_i) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_i) \times b)) = 2.971$ kipsAxial force for maximum compression; $P = (D + S_{wt} \times h) = 218$ lb/ftMaximum applied compressive stress; $F_c^- = F_c \times C_0 \times C_{hc} \times C_{hc} \times C_{hc} \times C_{hc} \times C_{p} = 413$ lb/in²Maximum applied tensile stress; $F_c^- = -5.45$ Chord capacity for segment 3 $h / b_3 = 2.545$ Shear force for maximum tension; $P = 0.6 \times (D + S_{wt} \times h) = 130.8$ lb/ftMaximum applied tensile stress; $h / b_3 = 2.545$ Chord capacity for segment 3 $h / b_3 = 2.545$ Shear force for maximum tension; $P = 0.6 \times (D + S_{wt} \times h) = 130.8$ lb/ftMaximum applied tensile stress; $f_i = T / A_{en} = 186$ lb/in²Design tensile stress; $f_i = T / A_{en} = 186$ lb/in²Shear force for maximum compression; $V = 0.7 \times (E_q + C_s (A \times S_{wt} + (D + + L_i) \times b)) = 2.971$ kipsAxial force for maximum tension; $V = 0.7 \times (E_q + C_s (A \times S_{wt} + (D + + L_i) \times b)) = 2.971$ kipsAxial force for maximum tension; $V = 0.7 \times (E_q + C_s (A \times S_{wt} + (D + + L_i) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_q + C$	Shear force for maximum tension;		$V = 0.7 imes (E_q$	+ $C_s \times (A \times S_w)$	$_{rt}$ + (D + + L _r) ×	b)) = 2.971 kips		
Maximum tensile force in chord; Maximum applied tensile stress; $T = V \times h / (b_1 + b_2 + b_3) - P \times b_2 / 2 = 2.574$ kips $f_1 = T / A_{en} = 191 lb/n^2$ Design tensile stress; $f_1 = T / A_{en} = 191 lb/n^2$ Design tensile stress; $F_1 = F_1 \times C_D \times C_{M} \times C_R \times C_F_1 \times C_F_1 = 720 lb/in^2$ Shear force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_1) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_1) \times b)) = 2.971$ kipsMaximum applied compressive stress; $F_c = C / A_e = 204 lb/n^2$ Design compressive stress; $F_c = C / A_e = 204 lb/n^2$ PASS - Design compressive stress; $F_c = C / A_e = 204 lb/n^2$ Pass - Design compressive stress; $F_c = C / A_e = 204 lb/n^2$ Pass - Design compressive stress; $F_c = C / A_e = 204 lb/n^2$ Pass - Design compressive stress; $F_c = C / A_e = 204 lb/n^2$ Pass - Design compressive stress; $F_c = C / A_e = 204 lb/n^2$ Shear wall aspect ratic; $h / b_3 = 2.545$ Shear wall aspect ratic; $h / b_3 = 2.545$ Shear force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_1) \times b)) = 2.971 kipsAxial force for maximum tension;P = 0.6 \times (D + S_{wt} \times h) = 130.8 lb/ltMaximum applied tensile stress;f_1 = T / A_{en} = 186 lb/n^2Design tensile stress;f_1 = T / A_{en} = 186 lb/n^2Design tensile stress;f_1 = T / A_{en} = 186 lb/n^2Design tensile stress;f_2 = T A_{en} = 186 lb/n^2Design tensile stress;f_2 = T A_{en} = 186 lb/n^2Design $	Axial force for maximum tension;		$P = 0.6 \times (D + $	+ S _{wt} × h) = 13	0.8 lb/ft			
Maximum applied tensile stress; $f_i = T / A_{en} = 191 \text{ lb/n}^2$ Design tensile stress; $F_i' = F_i \times C_D \times C_{Mi} \times C_{Hi} \times C_{Fi} \times C_i = 720 \text{ lb/in}^2$ $f_i / F_i' = 0.265$ PASS - Design tensile stress exceeds maximum applied tensile stress Shear force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wit} + (D + + L_i) \times b)) = 2.971 \text{ kips}$ Axial force for maximum compressives; $P = (D + S_{wit} \times h) = 218 \text{ lb/ft}$ Maximum applied compressive stress; $F_c = C / A_e = 204 \text{ lb/in}^2$ Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_F_c \times C_i \times C_P = 413 \text{ lb/in}^2$ $f_c = C / A_e = 204 \text{ lb/in}^2$ $F_c' = -0.493$ PASS - Design compressive stress; $F_c' = -0.493$ PASS - Design compressive stress; $h / b_3 = 2.545$ Shear wall aspect ratio; $h / b_3 = 2.545$ Shear force for maximum tension; $P = 0.6 \times (D + S_{wit} \times h) = 130.8 \text{ lb/ft}$ Maximum applied tensile stress; $f_i = T / A_{en} = 186 \text{ lb/in}^2$ Design tensile stress; $f_i = T_i \times C_D \propto C_{Min} \propto C_{tit} \propto C_{F1} \times C_i = 720 \text{ lb/in}^2$ f_i / F_i' = 0.258 $F_{ASS} - Design tensile stress exceeds maximum applied tensile stress;f_i = F_i \times C_D \propto C_{Min} \propto C_{tit} \propto C_{F1} \times C_{F1} \times C_i = 720 \text{ lb/in}^2f_i / F_i' = 0.258F_{ASS} - Design tensile stress exceeds maximum applied tensile stress;f_i = F_i \times C_D \propto C_{Min} \propto C_{tit} \propto C_{F1} \propto C_F \times C_i = 720 \text{ lb/in}^2f_i / F_i' = 0.258F_{ASS} - Design tensile stress exceeds maximum applied tensile stress;f_i = F_i \times C_D \propto C_{Min} \propto C_{it} \propto C_{F1} \propto C_{F1} \propto C_{F1} \approx C_{F1} \times C_{F$	Maximum tensile force in chord;		$T = V \times h / (b + b)$	1 + b ₂ + b ₃) - P	$x b_2 / 2 = 2.574$	4 kips		
Design tensile stress; $F_{1}^{t} = F_{1} \times C_{D} \times C_{Mt} \times C_{tt} \times C_{F_{1}} \times C_{i} = 720 \text{ lb/in}^{2}$ $f_{t}/F_{t}^{i} = 0.265$ $PASS - Design tensile stress exceeds maximum applied tensile stress$ Shear force for maximum compression; Maximum applied compressive stress; Design compressive stress; Design compressive stress; Design compressive stress; Chord capacity for segment 3 Shear force for maximum tension; Axial force for maximum tension; Maximum applied tensile stress; Design tensile stress; Fig. = T / A _{en} = 186 lb/in ² Fig. = Fig. CD × CMt × CHt	Maximum applied tensile stress;		$f_t = T / A_{en} = 1$	1 91 lb/in ²				
$f_{1}/F_{1}^{2} = 0.265$ $PASS - Design tensile stress exceeds maximum applied tensile stress$ Shear force for maximum compression; Axial force for maximum compression; $V = 0.7 \times (E_{q} + C_{s} \times (A \times S_{wt} + (D + + L_{t}) \times b)) = 2.971 \text{ kips}$ $P = (D + S_{wt} \times h) = 218 \text{ lb/ft}$ $C = V \times h / (b_{1} + b_{2} + b_{3}) + P \times b_{2} / 2 = 3.359 \text{ kips}$ $f_{0} = C / A_{0} = 204 \text{ lb/in}^{2}$ $F_{0}^{-} = F_{0} \times C_{D} \times C_{te} \times C_{F_{0}} \times C_{i} \times C_{P} = 413 \text{ lb/in}^{2}$ $F_{0}^{-} = F_{0} \times C_{D} \times C_{te} \times (A \times S_{wt} + (D + + L_{t}) \times b)) = 2.971 \text{ kips}$ $P = (D + S_{wt} \times h) = 2.18 \text{ lb/in}^{2}$ $F_{0}^{-} = F_{0} \times C_{D} \times C_{te} \times C_{F_{0}} \times C_{i} \times C_{P} = 413 \text{ lb/in}^{2}$ $F_{0}^{-} = -6 \times C_{D} \times C_{te} \times C_{F_{0}} \times C_{i} \times C_{P} = 413 \text{ lb/in}^{2}$ $F_{0}^{-} = 0.493$ $PASS - Design compressive stress exceeds maximum applied compressive stress$ Chord capacity for segment 3 Shear force for maximum tension; $V = 0.7 \times (E_{q} + C_{s} \times (A \times S_{wt} + (D + + L_{t}) \times b)) = 2.971 \text{ kips}$ $Axial force for maximum tension;$ $P = 0.6 \times (D + S_{wt} \times h) = 130.8 \text{ lb/ft}$ $T = V \times h / (b_{1} + b_{2} + b_{3}) + P \times b_{3} / 2 = 2.509 \text{ kips}$ $f_{1} = T / A_{en} = 186 \text{ lb/in}^{2}$ $Pass - Design tensile stress;$ $F_{1}^{-} = F_{c} C_{D} \times C_{Mt} \times C_{Ht} \times C_{Ft} \times C_{i} = 720 \text{ lb/in}^{2}$ $f_{1} / F_{1}^{-} = 0.258$ $PASS - Design tensile stress exceeds maximum applied tensile stress exceeds maximum applied tensile stress exceeds maximum applied tensile stress;$ $F_{0} = (D + S_{wt} \times h) = 218 \text{ lb/ft}$ $Axial force for maximum compression;$ $V = 0.7 \times (E_{q} + C_{s} \times (A \times S_{wt} + (D + + L_{t}) \times b)) = 2.971 \text{ kips}$ $PASS - Design tensile stress exceeds maximum applied tensile$	Design tensile stress;		$F_t' = F_t \times C_D \times$	$C_{Mt} \times C_{tt} \times C_{F}$	$t_t \times C_i = 720$ lb/i	n ²		
PASS - Design tensile stress exceeds maximum applied tensile stressShear force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_i) \times b)) = 2.971$ kipsAxial force for maximum compression; $P = (D + S_{wt} \times h) = 218$ lb/ftMaximum compressive force in chord; $C = V \times h / (b_1 + b_2 + b_3) + P \times b_2 / 2 = 3.359$ kipsMaximum applied compressive stress; $f_c = C / A_e = 204$ lb/in ² Design compressive stress; $F_c = C \times C_D \times C_{hc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = 413$ lb/in ² $f_c / F_c' = 0.493$ PASS - Design compressive stress exceeds maximum applied compressive stress Chord capacity for segment 3Shear wall aspect ratio; $h / b_3 = 2.545$ Shear force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum tension; $P = 0.6 \times (D + S_{wt} \times h) = 130.8$ lb/ftMaximum applied tensile stress; $f_1 = T / A_{en} = 186$ lb/in ² Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{FT} \times C_i = 720$ lb/in ² $f_t / F_t' = 0.258$ PASS - Design tensile stress exceeds maximum applied tensile stress Shear force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.$			$f_t / F_t' = 0.265$					
Shear force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum compression; $P = (D + S_{wt} \times h) = 218$ lb/ftMaximum compressive force in chord; $P = (D + S_{wt} \times h) = 218$ lb/ftMaximum applied compressive stress; $F_c = C / A_e = 204$ lb/in ² Design compressive stress; $F_c = C / A_e = 204$ lb/in ² PASS - Design compressive stress; $F_c = -C / A_e = 204$ lb/in ² PASS - Design compressive stress; $F_c = 0.493$ PASS - Design compressive stress exceeds maximum applied compressive stressChord capacity for segment 3 $h / b_3 = 2.545$ Shear force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum tension; $P = 0.6 \times (D + S_{wt} \times h) = 130.8$ lb/ftMaximum applied tensile stress; $F_t = F_t \times C_D \times C_{ht} \times C_{Ft} \times C_t = 720$ lb/in ² Maximum applied tensile stress; $F_t = F_t \times C_D \times C_{ht} \times C_{Ft} \times C_t = 720$ lb/in ² Maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum compression; $F_t = F_t \times C_D \times C_{ht} \times C_{ft} \times C_{ft} \times C_t = 720$ lb/in ² Shear force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum compression; $F_t = F_t \times C_D \times C_{ht} \times C_{ft} \times C_{ft} \times C_{ft} = 720$ lb/in ² Axial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2$			PASS - Desi	gn tensile str	ess exceeds n	naximum applie	d tensile stress	
Axial force for maximum compression; $P = (D + S_{wt} \times h) = 218$ lb/ftMaximum compressive force in chord; $P = (D + S_{wt} \times h) = 218$ lb/ftMaximum applied compressive stress; $f_c = C / A_c = 204$ lb/in ² Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_F c \times C_i \times C_P = 413$ lb/in ² $f_c / F_c' = 0.493$ PASS - Design compressive stress exceeds maximum applied compressive stress Chord capacity for segment 3 $h / b_3 = 2.545$ Shear force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum tension; $P = 0.6 \times (D + S_{wt} \times h) = 130.8$ lb/ftMaximum applied tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_t \times C_F \times C_t = 2.509$ kipsMaximum applied tensile stress; $F_t' = T / A_{en} = 186$ lb/in ² Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_t \times C_F \times C_t = 720$ lb/in ² Shear force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum compression; $F_t' = F_t \times C_D \times C_{Mt} \times C_t \times C_F \times C_t = 720$ lb/in ² Shear force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) $	Shear force for maximum compress	sion;	$V = 0.7 \times (E_q)$	+ $C_s \times (A \times S_w)$	$_{t}$ + (D + + L _r) ×	b)) = 2.971 kips		
Maximum compressive force in chord; Maximum applied compressive stress; $C = V \times h / (b_1 + b_2 + b_3) + P \times b_2 / 2 = 3.359$ kips $f_c = C / A_e = 204$ lb/in²Design compressive stress; $F_c = C / A_e = 204$ lb/in² $F_c ' = F_c \times C_D \times C_{Mc} \times C_{Ic} \times C_{Fc} \times C_i \times C_P = 413$ lb/in² $f_c / F_c ' = 0.493$ PASS - Design compressive stress exceeds maximum applied compressive stress Chord capacity for segment 3Shear wall aspect ratio;Shear force for maximum tension; Maximum tensile force in chord; Maximum applied tensile stress; $P = 0.6 \times (D + S_{wt} \times h) = 130.8$ lb/ftMaximum applied tensile stress; $F_t ' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_t = 720$ lb/in² $f_t / F_t ' = 0.258$ Shear force for maximum compression; Axial force for maximum compression; Axial force for maximum compression; $P = (D + S_{wt} \times h) = 218$ lb/ftMaximum applied compressive stress; $P = (D + S_{wt} \times h) = 218$ lb/ftDesign tensile stress; $P = (D + S_{wt} \times h) = 218$ lb/ftDesign tompressive force in chord; $P = (D + S_{wt} \times h) = 218$ lb/ftMaximum compression; Maximum applied compressive stress; $P = (D + S_{wt} \times h) = 218$ lb/ftDesign compressive stress; $P = (D + S_{wt} \times h) = 218$ lb/ftDesign compressive stress; $P = (D - K_{wt} \times h) = 210$ lb/in² $F_c ' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_F \times C_i \times C_P = 413$ lb/in² $f_c = C / A_e = 210$ lb/in²	Axial force for maximum compressi	on;	$P = (D + S_{wt})$	< h) = 218 lb/ft				
Maximum applied compressive stress; $f_c = C / A_e = 204 \text{ lb/in}^2$ Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = 413 \text{ lb/in}^2$ $f_c / F_c' = 0.493$ PASS - Design compressive stress exceeds maximum applied compressive stress Chord capacity for segment 3Shear wall aspect ratio;Shear force for maximum tension;Axial force for maximum tension;Maximum tensile force in chord;Maximum applied tensile stress;Design tensile stress;Design tensile stress;Design tensile stress;Shear force for maximum compression;Naximum compression;Axial force for maximum compression;Axial force for maximum compression;N = 0.7 × (Eq + Cs × (A × Swt + (D + + Lq) × b)) = 2.971 kipsPelos × (D × Da) × D × Da) × D × Da × Da × Da × D	Maximum compressive force in cho	rd;	$C = V \times h / (b)$	$_{1} + b_{2} + b_{3} + b_{3}$	$P \times b_2 / 2 = 3.35$	5 9 kips		
Design compressive stress: $F_{c} = F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} \times C_{P} = 413 \text{ lb/in}^{2}$ $f_{c} / F_{c} = 0.493$ PASS - Design compressive stress exceeds maximum applied compressive stress Chord capacity for segment 3 Shear wall aspect ratio; Shear force for maximum tension; Axial force for maximum tension; Maximum applied tensile stress; Design tensile stress; Design tensile stress; Shear force for maximum compression; Axial force for maximum compression; Shear force for maximum compression; Axial force for maximum compression; Shear force for maximum compression; Axial force for maximum compressive force in chord; Axial force for maximum compressive stress; Design compressive stress; Baximum applied compressi	Maximum applied compressive stre	ss;	$f_c = C / A_e = 2$	04 lb/in²		2		
$f_c / F_c' = 0.493$ PASS - Design compressive stress exceeds maximum applied compressive stress Chord capacity for segment 3 Shear wall aspect ratio; Shear force for maximum tension; Axial force for maximum tension; Maximum applied tensile stress; Design tensile stress; Shear force for maximum compression; Axial force for maximum compression; Shear force for maximum compression; Axial force for maximum compressive stress; Design compressive stress; Des	Design compressive stress;		$F_c' = F_c \times C_D$	$\times C_{Mc} \times C_{tc} \times C_{tc}$	$C_{Fc} \times C_i \times C_P = 4$	113 lb/in ²		
Chord capacity for segment 3 Shear wall aspect ratio; Axial force for maximum tension; Axial force for maximum tension; Maximum tensile force in chord; Maximum applied tensile stress; Design tensile stress; Shear force for maximum compression; Shear force for maximum compression; Shear force for maximum compression; Axial force for maximum compression; Maximum applied compressive stress; Chord capacity for segment 3 Maximum applied tensile stress; Shear force for maximum compression; Axial force for maximum compression; Maximum applied compressive stress; Shear force for maximum compression; Maximum applied compressive stress; Chord capacity force in chord; Maximum applied compressive stress; Design		5400 5	$f_{c} / F_{c}' = 0.493$	3				
Choice dapacity for segment 3Shear wall aspect ratio; $h / b_3 = 2.545$ Shear force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_r) \times b)) = 2.971$ kipsAxial force for maximum tension; $P = 0.6 \times (D + S_{wt} \times h) = 130.8$ lb/ftMaximum applied tensile stress; $T = V \times h / (b_1 + b_2 + b_3) - P \times b_3 / 2 = 2.509$ kipsDesign tensile stress; $f_t = T / A_{en} = 186$ lb/in ² Design tensile stress; $F_t' = R_t \times C_D \times C_{Mt} \times C_{tt} \times C_t = 720$ lb/in ² f_t / F_t' = 0.258 $PASS - Design tensile stress exceeds maximum applied tensile stressShear force for maximum compression;V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_r) \times b)) = 2.971 kipsAxial force for maximum compression;P = (D + S_{wt} \times h) = 218 lb/ftAxial force for maximum applied compressive stress;C = V \times h / (b_1 + b_2 + b_3) + P \times b_3 / 2 = 3.468 kipsf_c = C / A_e = 210 lb/in2F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_F \times C_i \times C_P = 413 lb/in2$	Chard consolity for comment 2	PASS - D	esign compress	sive stress ex	ceeds maximu	im applied com	pressive stress	
Shear force for maximum tension; Axial force for maximum tension; Maximum tensile force in chord; Maximum applied tensile stress; Design tensile stress; Shear force for maximum compression; Axial force for maximum compressive stress; Design compressive stress; Design compressive stress; $f_c = C / A_e = 210 lb/in^2$ $f_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_Fc \times C_i \times C_P = 413 lb/in^2$ $f_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_Fc \times C_i \times C_P = 413 lb/in^2$	Shear wall aspect ratio:		h / ha - 2 545					
Axial force for maximum tension; $V = 0.7 \times (Eq + C_S \times (A \times O_M + (D + + E_f) \times O)) = 2.31 \text{ kps}$ Axial force for maximum tension; $P = 0.6 \times (D + S_{wt} \times h) = 130.8 \text{ lb/ft}$ Maximum applied tensile force in chord; $T = V \times h / (b_1 + b_2 + b_3) - P \times b_3 / 2 = 2.509 \text{ kps}$ Design tensile stress; $f_t = T / A_{en} = 186 \text{ lb/in}^2$ Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = 720 \text{ lb/in}^2$ f_t / F_t' = 0.258PASS - Design tensile stress exceeds maximum applied tensile stressShear force for maximum compression; $V = 0.7 \times (Eq + C_S \times (A \times S_{wt} + (D + + L_f) \times b)) = 2.971 \text{ kips}$ Axial force for maximum compression; $V = 0.7 \times (Eq + C_S \times (A \times S_{wt} + (D + + L_f) \times b)) = 2.971 \text{ kips}$ Axial force for maximum compression; $P = (D + S_{wt} \times h) = 218 \text{ lb/ft}$ Maximum applied compressive stress; $F_c = C / A_e = 210 \text{ lb/in}^2$ Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = 413 \text{ lb/in}^2$ $f_c / F_c' = 0.500$ $F_c = 0.500$	Shear force for maximum tension:		$V = 0.7 \times (E_{2})$. + (D + + -) ×	h)) - 2 971 kins		
Axial force for maximum tension, Maximum applied tensile stress; $T = 0.0 \times (D + G_{MT} \times N) = 130.0$ lb/ltMaximum tensile force in chord; Maximum applied tensile stress; $T = V \times h / (b_1 + b_2 + b_3) - P \times b_3 / 2 = 2.509$ kips $f_t = T / A_{en} = 186$ lb/ln ² Design tensile stress; $F_t = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = 720$ lb/ln ² $f_t / F_t' = 0.258$ Shear force for maximum compression; Maximum compressive force in chord; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsP = (D + S_{wt} \times h) = 218 lb/ft $C = V \times h / (b_1 + b_2 + b_3) + P \times b_3 / 2 = 3.468$ kips $f_c = C / A_e = 210$ lb/ln ² Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = 413$ lb/ln ²	Avial force for maximum tension:		$\nabla = 0.7 \times (L_q)$	$+ O_{\rm S} \wedge (\Lambda \wedge O_{\rm W})$	π + (D + + Lr) ∧ Λ 8 lb/ft	b)) – 2.97 i kips		
Maximum applied tensile stress; $f_t = v \times h/(b_1 + b_2 + b_3) + P \times b_3/2 = 2.509$ kipsMaximum applied tensile stress; $f_t = T / A_{en} = 186$ lb/in ² Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = 720$ lb/in ² $f_t / F_t' = 0.258$ PASS - Design tensile stress exceeds maximum applied tensile stress Shear force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_t) \times b)) = 2.971$ kipsAxial force for maximum compression; $P = (D + S_{wt} \times h) = 218$ lb/ftMaximum compressive force in chord; $C = V \times h / (b_1 + b_2 + b_3) + P \times b_3 / 2 = 3.468$ kipsMaximum applied compressive stress; $f_c = C / A_e = 210$ lb/in ² Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_F \times C_i \times C_P = 413$ lb/in ² $f_c = C + Q = 500$ $f_c = C + Q = 500$	Maximum tensile force in chord:		T = V ~ h / /h	$-\mathbf{O}_{Wt} \times \Pi = \mathbf{I} \mathbf{J}$	$v \cdot v = 10/11$	9 kins		
Maximum applied tensile stress; $F_t = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_F t \times C_i = 720 \text{ lb/in}^2$ Design tensile stress; $F_t = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = 720 \text{ lb/in}^2$ Shear force for maximum compression; $PASS - Design tensile stress exceeds maximum applied tensile stressAxial force for maximum compression;V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_r) \times b)) = 2.971 \text{ kips}Maximum compressive force in chord;P = (D + S_{wt} \times h) = 218 \text{ lb/ft}Maximum applied compressive stress;f_c = C / A_e = 210 \text{ lb/in}^2Design compressive stress;F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_F \times C_I \times C_P = 413 \text{ lb/in}^2f_c = C + 0.500$	Maximum applied tensile stress:		$\mathbf{I} = \mathbf{V} \times \mathbf{I} \mathbf{I} / (\mathbf{D})$ $\mathbf{f}_{\mathbf{I}} = \mathbf{T} / \mathbf{A} = \mathbf{I}$	1 + U2 + U3) - P 1 86 lb/in ²	$^{03}/2 = 2.303$	a viha		
$F_{t} = F_{t} \times G_{b} \times G_{Mt} \times G_{Ft} \times G_{F$	Design tensile stress:		$r_t = 1 / A_{en} = 1$ $F_c' = F_c \times C_c \times C_c$			n ²		
Int r = 0.230PASS - Design tensile stress exceeds maximum applied tensile stressShear force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_r) \times b)) = 2.971$ kipsAxial force for maximum compression; $P = (D + S_{wt} \times h) = 218$ lb/ftMaximum compressive force in chord; $C = V \times h / (b_1 + b_2 + b_3) + P \times b_3 / 2 = 3.468$ kipsMaximum applied compressive stress; $f_c = C / A_e = 210$ lb/in ² Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_F \times C_I \times C_P = 413$ lb/in ² $f_c = C / A_e = 200$	שבטועוז ופווטווב גוופטט,		$\Gamma_t = \Gamma_t \times CD \times f_t / F_t' = 0.259$		$t \wedge \mathbf{U}_{1} = \mathbf{I} \mathbf{Z} \mathbf{U} \mathbf{I} \mathbf{U} / \mathbf{I}$			
Shear force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + + L_r) \times b)) = 2.971$ kipsAxial force for maximum compression; $P = (D + S_{wt} \times h) = 218$ lb/ftMaximum compressive force in chord; $C = V \times h / (b_1 + b_2 + b_3) + P \times b_3 / 2 = 3.468$ kipsMaximum applied compressive stress; $f_c = C / A_e = 210$ lb/in ² Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_F \times C_i \times C_P = 413$ lb/in ² $f_c = C / A_e = 200$ lb/in			PASS - Desi	an tencile etr	ass arcoads n	naximum annlio	d tensile stress	
Axial force for maximum compression; $P = (D + S_{wt} \times h) = 218 \text{ lb/ft}$ Maximum compressive force in chord; $C = V \times h / (b_1 + b_2 + b_3) + P \times b_3 / 2 = 3.468 \text{ kips}$ Maximum applied compressive stress; $f_c = C / A_e = 210 \text{ lb/in}^2$ Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_F \times C_i \times C_P = 413 \text{ lb/in}^2$	Shear force for maximum compress	sion.	$V = 0.7 \times (F$	+ $C_{\alpha} \times (\Delta \times S)$	$+ (D + + L) \vee$	b)) = 2 971 kine		
Maximum compressive force in chord; $C = V \times h / (b_1 + b_2 + b_3) + P \times b_3 / 2 = 3.468$ kipsMaximum applied compressive stress; $f_c = C / A_e = 210$ lb/in ² Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_F \times C_i \times C_P = 413$ lb/in ² $f_c = C / A_e = 200$ lb/in	Axial force for maximum compress	on:	$P = (D \pm S)$	(h) = 218 lh/ft		<i>5)) – 2.31</i> Kips		
Maximum applied compressive stress; $C = V \times II / (D_1 + D_2 + D_3) + P \times D_3 / 2 = 3.400 \text{ kps}$ Maximum applied compressive stress; $f_c = C / A_e = 210 \text{ lb/in}^2$ Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_F \times C_i \times C_P = 413 \text{ lb/in}^2$ $f_c = C / A_e = 210 \text{ lb/in}^2$	Maximum compressive force in che	urd.	$\Gamma = (D + S_{Wt})$		Dvh./2-244	8 kine		
Design compressive stress; $F_c = F_c \times C_p \times C_{Hc} \times C_{Fc} \times C_i \times C_P = 413 \text{ lb/in}^2$	Maximum applied compressive torce in cho	iu,	$\mathbf{U} = \mathbf{v} \times \mathbf{n} / (\mathbf{D})$	$_1 + u_2 + u_3) + 1$	$- \times u_3 / 2 = 3.40$	kih2		
$\Gamma_{\rm C} = \Gamma_{\rm C} \times O_{\rm D} \times O_{\rm MC} \times O_{\rm FC} \times O_{\rm f} \times O_{\rm P} = 413 \text{ ID/III}$	Design compressive stress:	33,	$I_{c} = C / A_{e} = Z$			113 lb/in ²		
	Design compressive stress,		$\mathbf{r}_{c} = \mathbf{r}_{c} \times \mathbf{O}_{D}$		$V_{FC} \times U_i \times U_P = 4$			
understand Means Obsers	Project Job Ref.							
--	---	--	--	-------------------------------	--------------------	------------------	--	--
a deal and a deal a	Section				Shoot po /rov			
ASES	Section				Sheet no./rev.	7 of 81		
A.S. Engineering Services, P.C.	Calc by	Date	Chk'd by	Date	App'd by			
112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259		5/15/2012	Clikeby	Date	App a by	Dale		
(0) 031-300-0233	AU	3/13/2012						
	PASS - L	Design compres	sive stress ex	ceeds maximu	m applied comp	pressive stress		
Chord capacity for segment 4								
Shear wall aspect ratio;	h / b ₄ = 5.6							
•	Seg	ment not consid	lered, shear w	all aspect ratio	o exceeds maxii	num allowable		
Collector capacity	-			-				
Maximum shear force in collector:		$V_{max} = max(V)$	(° max, (Vw max) =	2.971 kips				
Unit shear above opening:	$v_a = V_{max} / b =$	= 118.831 lb/ft						
Unit shear below opening:	$v_b = V_{max} / (b)$	$(1 + b_2 + b_3) = 2$	04.882 lb/ft					
Maximum tensile force in collector;	$T = (b_1 + b_2 + b_3) \times v_b - P_{03} \times v_a = 0.594 \text{ kins}$							
Maximum applied tensile stress;	$f_t = T / (2 \times A_s) = 36 \text{ lb/in}^2$							
Design tensile stress:	$F_t = F_t \times C_D$	$< C_{Mt} \times C_{tt} \times C_{F}$	t × Ci = 720 lb/ir	1 ²				
5		$f_t / F_t' = 0.050$)					
		PASS - Desi	ign tensile str	ess exceeds m	aximum applied	l tensile stress		
Maximum compressive force in colle	ctor;	$C = max((P_{o1})$	+ w _{o1}) × v _a - b	$_1 \times v_b$, 0 kips) =	0 kips			
Maximum applied compressive stres	$f_c = C / (2 \times A)$	h_{s}) = 0 lb/in ²						
Column stability factor;		C _P = 1.00						
Design compressive stress;		$F_{c}' = F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} \times C_{P} = \textbf{1360 lb/in}^{2}$						
		$f_{c} / F_{c}' = 0.00$	D					
	PASS - L	Design compres	sive stress ex	ceeds maximu	m applied comp	pressive stress		
Deflection								
Design shear force;		$V = E_q + C_s \times$	$(A \times S_{wt} + (D +$	+ + L_r) × b) = 4 .	.244 kips			
Induced unit shear;		$v = V / (b_1 + b_2 + b_3) = 292.688$ lb/ft						
Vertical elongation of wall anchora	age;	D _a = 0.25 in						
Shear wall deflection - Eqn. 4.3-1;		$d_{sw} = 8 \cdot v \cdot h^3 / (E \cdot A_e \cdot (b_1 + b_2 + b_3)) + v \cdot h / (1000 \cdot G_a) + h \cdot D_a / $						
		$(b_1 + b_2 + b_3)$	= 0.472 in					
Deflection limit;		$\delta_{\text{limit}} = h / 240 = 0.7$ in						
		$\delta_{sw} / \delta_{limit} = 0.674$						
			PASS - She	ar wall deflecti	ion is less than o	deflection limi		



and arian minor of	Project				Job Ref.		
ATURITA	Section				Sheet no./rev.		
ASES			39 of 81				
112 Wilson Drive. Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date	
(C) 631-560-0259	AS	5/15/2012					
		LOWER ROOM	F DEAD LOAD	WITH GREEN RO	JOF - Dead	UDL 0.165	
			.00 In to 408.00				
		SNOW LOAD		AFIERS - Snow	UDL 0.225	KIPS/ft from	
		0.00 in to 408					
				AFIERS W/ GREI	EN ROOF L	OAD - Dead	
		UDL 0.307 KIP	s/ft from 0.00	in to 408.00 in			
Load combinations							
Load combination 1		Support A		Dead × 1.00)		
				Live × 1.00			
				Roof live × 1	1.00		
			Snow × 1.00	00			
				Dead × 1.00)		
				$Live \times 1.00$			
				Roof live × 1	1.00		
				Snow \times 1.00)		
		Support B		Dead \times 1.00	Dead × 1.00 Live × 1.00		
				Live \times 1.00			
				Roof live × 1	1.00		
		Snow × 1			00		
Analysis results							
Maximum moment;		M _{max} = 134.2 k	.ips_ft;	$M_{min} = 0 kips$	s_ft		
Maximum shear;		V _{max} = 15.8 kip	s;	V _{min} = -15.8	kips		
Deflection;		$\delta_{max} = 1$ in;		$\delta_{min} = 0$ in			
Maximum reaction at support A;		R _{A_max} = 15.8 k	kips;	R _{A_min} = 15.	B kips		
Unfactored dead load reaction at	support A;	$R_{A_Dead} = 12 $ k	ips				
Unfactored snow load reaction at	support A;	$R_{A_{snow}} = 3.8$	kips				
Maximum reaction at support B;		R _{B_max} = 15.8 k	kips;	R _{B_min} = 15.	B kips		
Unfactored dead load reaction at	support B;	$R_{B_{Dead}} = 12 k$	ips				
Unfactored snow load reaction at	support B;	$R_{B_{Snow}} = 3.8$ I	kips				
Section details							
Section type;		W 16x67 (AIS	C 13th Edn 20	05)			
ASTM steel designation;		A992					
Steel yield stress;		F _y = 50 ksi					
Steel tensile stress;		F _u = 65 ksi					
Modulus of elasticity;		E = 29000 ksi					

IN DISTANCE PROPERTY OF THE PR	Project				Job Ref.	
Adapt antinate	Section				Sheet no /rev	,
ASES	occion				2	10 of 81
112 Wilson Drive Port Jefferson NV 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
		→ -0.67				
	Ť	→				
	6.3"	-	▲ −0.4"			
		E.				
		− 0.6 <u>1</u>				
	<u> </u>	<u>↓</u>				
		 ∢ 1	0.2"			
Safety factors						
Safety factor for tensile yielding;		$\Omega_{\rm ty} = 1.67$				
Safety factor for compression:		$\Omega_{\rm tr} = 2.00$				
Safety factor for flexure:		$\Omega_{c} = 1.67$				
Safety factor for shear:		$\Omega_{\rm v} = 1.57$				
		120 - 1100				
		Span 1 has co	ontinuous late	ral bracing		
Classification of apotions for local b	onding Socti			rai bracing		
Classification of flanges in flexure -	Table B4.1 (ca	ase 1) $h(2\times t) = 7$	67			
Limiting ratio for compact section:		$D_f / (2 \times l_f) = I$.07 [E/E]_015			
Limiting ratio for non-compact section,	on:	$\lambda_{\text{pff}} = 0.38 \times \sqrt{16}$	[/ F.] – 24 08	Compa	act	
			- / i yj - 24.00 ,	, compe		
Width to thickness ratio:	Die D4.1 (Case	$(d - 2 \times k) / t$	- 35 85			
Limiting ratio for compact section:		$\lambda_{out} = 3.76 \times 10^{-10}$	_ 33.83 √[F / FJ] = 90.5	55		
Limiting ratio for non-compact section,	on:	$\lambda_{rwf} = 5.70 \times N$	([= / F _v] = 137 .	.27: Compa	act	
3	,			, [-	Section is con	npact in flexure
Design of members for shear - Cha	opter G					
Required shear strength;	1	V _r = max(abs)	(V _{max}), abs(V _m	_{iin})) = 15.794 kij	ps	
Web area;		$A_w = d \times t_w = 0$	6.439 in ²			
Web plate buckling coefficient;		$k_v = 5$				
Web shear coefficient - eq G2-2;		$C_v = 1.000$				
Nominal shear strength - eq G2-1;		$V_n = 0.6 \times F_y$	$\times A_{\rm w} \times C_{\rm v} = 19$	3.155 kips		
Allowable shear strength;		$V_c = V_n / \Omega_v =$	128.770 kips	_		
		PASS - A	Allowable she	ear strength ex	xceeds required	shear strength

	-						
a subscription Miner Days	Project		Job Ref.	Job Ref.			
-Anti-state	Section				Sheet no./rev.		
ASES As Engineering Sarvices, PIC.		1			4	41 of 81	
112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259	Calc. by AS	Date 5/15/2012	Chk'd by	Date	App'd by	Date	
Design of members for flexure in the Required flexural strength;	e major axis - C	hapter F M _r = max(abs(M _{s1 max}), abs((M _{s1 min})) = 134	.246 kips_ft		
Yielding - Section F2.1			01_maxy /	(0,)	1 -		
Nominal flexural strength for yiel	ding - eq F2-1;	$\mathbf{M}_{nyld} = \mathbf{M}_{p} = \mathbf{F}_{p}$, ´ Z_x = 541.6	67 kips_ft			
Nominal flexural strength;		$M_n = M_{nyld} = 54$	1.667 kips_ft	i			
Allowable flexural strength;	vable flexural strength; M_c			_ft			
		PASS - Allow	able flexura	i strength exce	eeas requirea fie	exural strength	
Design of members for vertical defle	e roof live and s	snow loads					
Limiting deflection;		$\delta_{\text{lim}} = L_{s1} / 360$	= 1.133 in				
Maximum deflection span 1;		$\delta = \max(abs(\delta_{t}))$	_{max}), abs(δ _{min})) = 1.01 in			
		PAS	SS - Maximul	m deflection d	oes not exceed o	deflection limit	



and the state of t	Project				Job Ref.	
And state	0					
ASES	Section				Sheet no./rev.	2 of 01
A.S. Engineering Services, P.C.	Colo by	Data	Chlid by	Data	4.	
112 Wilson Drive, Port Jefferson, NY 11777		5/15/2012	Crik a by	Date	Арр а бу	Dale
(6) 031-300-0239	70	5/15/2012				
Load combinations						
Load combination 1		Support A		Dead $ imes$ 1.00)	
				$\text{Live} \times 1.00$		
				Snow × 1.00	C	
				Roof Live $ imes$	0.00	
		Span 1		Dead $ imes$ 1.00)	
		-		Live \times 1.00		
				Snow × 1.00	0	
				Roof Live ×	0.00	
		Support B		Dead \times 1.00)	
				Live $\times 1.00$		
				$Snow \times 1.00$	n	
				Boof Live x	0.00	
					0.00	
Analysis results						
Maximum moment;		$M_{max} = 2394$ lb	_ft;	$M_{min} = 0 \ Ib_{-}$	ft	
Design moment;		M = max(abs)/h	VI _{max}),abs(IVI _{mir}	$(1) = 2394 \text{ ID}_{\Pi}$	lh	
Maximum snear;		$F_{max} = 332$ ID; F = max(aba)(F)		$F_{min} = -332$	D	
Total load on member:		r = max(abs(r = 1064 lb))	max), abs(Fmin)) = 332 ID		
Reaction at support A:		R = 532 lt		B) lh	
Unfactored dead load reaction at su	upport A:	R_{A} max = 307 l	b	rta_min = co		
Unfactored snow load reaction at si	upport A:	R_{A} show = 225	≈ b			
Unfactored roof live load reaction a	t support A;	$R_{A,Booflive} = 18$	0 lb			
Reaction at support B;		R _{B max} = 532 lb);	R _{B min} = 532	2 lb	
Unfactored dead load reaction at su	upport B;	R _{B Dead} = 307 I	b	_		
Unfactored snow load reaction at se	upport B;	R _{B_Snow} = 225	b			
Unfactored roof live load reaction a	t support B;	R _{B_Roof Live} = 18	0 lb			
T I						
DI L						
-11.2	XI <			>		
. ■ L		\triangleleft				
	4 −1.5	"-▶				
Sawn lumber section details						
Nominal breadth of sections:		h – 2 in				
Dressed breadth of sections:		b = 1.5 in				
Nominal denth 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:		$b_{\rm b} = N \times h = 1$	5 in			
		$D_{\rm D} = 11 \wedge D = 1$	•			

and strange and	Project				Job Ref.			
ANU-UNA	Section				Sheet no./rev.			
ASES					44	of 81		
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date		
(C) 631-560-0259	AS	5/15/2012						
Table 44 - Reference design values	for visually grad	ded dimension l	umber (2"-4" t	hick)				
Species, grade and size classification	or visually grad	Douglas Fir-I a	rch No 2 area	he 2" & wider				
Bending parallel to grain:	1,	E ₁ = 900 lb/in ²	icii, No.2 giac					
Tension parallel to grain;		$F_{b} = 575 \text{ lb/in}^{2}$						
Compression parallel to grain:		$F_c = 1350 \text{ lb/in}^2$						
Compression perpendicular to grain:		$F_{c} = 625 \text{ lb}$	/in ²					
Shear parallel to grain:		$F_v = 180 \text{ lb/in}^2$						
Modulus of elasticity:		E = 1600000 lb	/in ²					
Mean shear modulus:		$G_{def} = E / 16 =$	100000 lb/in ²					
Nombor dotoilo								
		Dru						
Service condition,		Diy						
Length of bearing,		$L_b = 4 \parallel 1$						
The beam is one of three or more repetitive members								
		15						
Section properties			12 2 1 2					
Cross sectional area of member;	Cross sectional area of member;			3				
Section modulus;		$S_x = N \times b \times d^2$	/ 6 = 31.64 in	2				
		$S_y = d \times (N \times b)$) ² / 6 = 4.22 in	13				
Second moment of area;		$I_x = N \times b \times d^3$	(12 = 177.98)	in⁴				
		$I_y = d \times (N \times b)^2$	³ / 12 = 3.16 ir	1 ⁴				
Adjustment factors								
Load duration factor - Table 2.3.2;		C _D = 1.00						
Temperature factor - Table 2.3.3;		C _t = 1.00						
Size factor for bending - Table 4A;		C _{Fb} = 1.00						
Size factor for tension - Table 4A;		C _{Ft} = 1.00						
Size factor for compression - Table 4	A;	C _{Fc} = 1.00						
Flat use factor - Table 4A;		C _{fu} = 1.20						
Incising factor for modulus of elasticit	y - Table 4.3.8	; C _{iE} = 1.00						
Incising factor for bending, shear, ter	ision & compre	ssion - Table 4.	3.8					
		C _i = 1.00						
Incising factor for perpendicular comp	pression - Tabl	e 4.3.8						
		C _{ic_perp} = 1.00						
Repetitive member factor - cl.4.3.9;		$C_r = 1.15$						
Bearing area factor - eq.3.10-2;		$C_{\rm b} = 1.00$	\ 0.00					
Depth-to-breadth ratio;		$d_{nom} / (N \times b_{nom})$	h) = 6.00					
- Beam is fully restrained		0 4 00						
Beam stability factor - cl.3.3.3,		$C_{\rm L} = 1.00$						
Bearing perpendicular to grain - cl.3.	10.2			2				
Design compression perpendicular to	o grain;	$F_{c_perp}' = F_{c_perp}$	$\times C_t \times C_i \times C_t$	_o = 625 lb/in ²				
Applied compression stress perpendi	Applied compression stress perpendicular to grain; $f_{c_perp} = R_{B_max} / (N \times b \times L_b) = 89 \text{ lb/in}^2$							
		f _{c_perp} / F _{c_perp} ' =	= 0.142					
	PASS - Des	ign compressi	ve stress exc	eeds applied com	pressive stre	ss at bearing		

and the second second	Project Job Ref.								
a data a taka	Section				Sheet no /rev	Sheet no /rev			
ASES	Geolon				45	of 81			
A.S. Engineering Gervices, P.C.	Calc. by	Date	Chk'd by	Date	App'd by	Date			
112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259	AS	5/15/2012							
(-)		0,10,2012							
Strength in bending - cl.3.3.1					_				
Design bending stress;		$\textbf{F}_{b}{}^{\prime} = \textbf{F}_{b} \times \textbf{C}_{D} \times \textbf{C}_{t} \times \textbf{C}_{L} \times \textbf{C}_{Fb} \times \textbf{C}_{i} \times \textbf{C}_{r} = \textbf{1035} \text{ lb/in}^{2}$							
Actual bending stress;		$f_b = M / S_x = 908 \text{ lb/in}^2$							
		f _b / F _b ' = 0.877							
		PAS	S - Design be	ending stress e	exceeds actual be	ending stress			
Strength in shear parallel to grain - o	3.4.1								
Design shear stress;		$F_v{'}=F_v\times C_D\times$	$C_t \times C_i = 180$	lb/in ²					
Actual shear stress - eq.3.4-2;		$f_v = 3 \times F / (2 \times F)$	< A) = 47 lb/in ²	:					
		f _v / F _v ' = 0.263							
			PASS - Desi	gn shear stres	s exceeds actual	shear stress			
Deflection - cl.3.5.1									
Modulus of elasticity for deflection:		$E' = E \times C_{ME} \times$	$C_t \times C_{iF} = 160$	00000 lb/in ²					
Design deflection:		$\delta_{\text{adm}} = 0.003 \times$	l = 0.648 in						
Bending deflection:		$v_{adm} = 0.003 \times L_{s1} = 0.040 \text{ III}$							
Shear deflection:		$\delta_{\rm v,s1} = 0.027 \text{ in}$							
Total deflection:		$\delta_{V_s1} = 0.021$ II	- 0.683 in						
Total deflection,		$\delta_a = 0_{D_s1} + 0_{V_s}$	51 – 0.003 mi						
		0a / 0adm - 1.03	IS FAII	- Design defle	oction exceeds to	tal deflection			
				Designation					
NEXT CHECKED SAME RAFTERS	BUT WITH LA	RGE NOTCH - 0	ONLY FOR SH	HEAR					
STRUCTURAL WOOD BEAM ANA	LYSIS & DESI	<u>GN (NDS 2005)</u>							
In accordance with the ASD method	1								
					TEDDS calculat	ion version 1.5.04			
		Load Envelope - Comb	ination 1						
0.058									
0.0									
ft L		18	3] B				



adarina mesara	Project				Job Ref.				
AND ADDA	Section				Sheet no./rev.				
ASES					47	of 81			
AS Industry Smoot AC	Calc. by	Date	Chk'd by	Date	App'd by	Date			
(C) 631-560-0259	AS	5/15/2012							
	<u>.</u>								
Design shear;		F = max(abs(F	max),abs(Fmin))) = 519 lb					
Total load on member;		$W_{tot} = 1038 \text{ lb}$							
Reaction at support A;		$R_{A_{max}} = 519$ lb);	R _{A_min} = 519	lb				
Unfactored dead load reaction at su	pport A;	R _{A_Dead} = 294 I	b						
Unfactored snow load reaction at su	ipport A;	R _{A_Snow} = 225 I	R _{A_Snow} = 225 lb						
Unfactored roof live load reaction at	support A;	R _{A_Roof Live} = 18	0 lb						
Reaction at support B;		$R_{B_{max}} = 519$ lb);	R _{B_min} = 519	lb				
Unfactored dead load reaction at su	pport B;	R _{B_Dead} = 294 I	b						
Unfactored snow load reaction at su	ipport B;	R _{B_Snow} = 225 I	b						
Unfactored roof live load reaction at	support B;	R _{B_Roof Live} = 18	0 lb						
1									
7.25									
→ 1.5" ◀─									
	∢ —4" →								
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} = 8 in							
Dressed depth of sections;		d = 7.25 in							
Number of sections in member;		N = 1							
Overall breadth of member:		$b_{\rm b} = \mathbf{N} \times \mathbf{b} = 1.5$ in							
	<i>.</i>		- (0" 4"						
Table 4A - Reference design values	for visually gra	aded dimension i	umber (2"-4"						
Species, grade and size classification	on;	Douglas Fir-La	rcn, No.2 gra	ide, 2" & wider					
Bending parallel to grain;		$F_b = 900 \text{ ID/IN}$							
		$F_t = 575 \text{ ID/III}$	2						
Compression parallel to grain,		$F_c = 1330 \text{ ID/III}$	/in ²						
Compression perpendicular to grain	,	$F_{c_perp} = 623 \text{ ID}$	/IN						
Shear parallel to grain;		$F_v = 180 \text{ ID/IN}$	1:-2						
Modulus of elasticity,				2					
Mean snear modulus;		$G_{def} = E / 16 =$	100000 ID/IN						
Member details									
Service condition;		Dry							
Length of bearing;		$L_b = 4$ in							
Load duration;		Ten years							
The beam is one of three or more re	petitive memb	ers							
Section properties									
Cross sectional area of member;		$A = N \times b \times d =$	10.87 in ²						
Section modulus;		$S_x = N \times b \times d^2$	/ 6 = 13.14 i	n ³					

and shink Misse of	Project				Job Ref.					
	Section				Sheet no./rev.					
ASES					48	of 81				
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date				
(C) 631-560-0259	AS	5/15/2012								
			$\frac{1}{2}$	3						
		$S_y = d \times (IN \times D)$	0) / 6 = 2.72 in	4						
Second moment of area;		$I_x = N \times b \times d^3 / 12 = 47.63$ in								
		$I_y = d \times (N \times b)$	° / 12 = 2.04 ir	l'						
Adjustment factors										
Load duration factor - Table 2.3.2;		C _D = 1.00								
Temperature factor - Table 2.3.3;		C _t = 1.00								
Size factor for bending - Table 4A;		C _{Fb} = 1.20								
Size factor for tension - Table 4A;		C _{Ft} = 1.20								
Size factor for compression - Table 4	IA;	C _{Fc} = 1.05								
Flat use factor - Table 4A;		C _{fu} = 1.15								
Incising factor for modulus of elastici	ty - Table 4.3.8	B; C _{iE} = 1.00								
Incising factor for bending, shear, ter	nsion & compre	ession - Table 4.	3.8							
		C _i = 1.00								
Incising factor for perpendicular com	pression - Tabl	le 4.3.8								
		C _{ic_perp} = 1.00								
Repetitive member factor - cl.4.3.9;	C _r = 1.15									
Bearing area factor - eq.3.10-2;	$C_{b} = (L_{b} + 0.37)$	$(5 \text{ in}) / L_b = 1.0$	9							
Depth-to-breadth ratio;	$d_{nom} / (N \times b_{nom})$	n) = 4.00								
- Beam is fully restrained										
Beam stability factor - cl.3.3.3;		C _L = 1.00								
Bearing perpendicular to grain - cl.3.	10.2									
Design compression perpendicular to	o grain;	$F_{c_perp}' = F_{c_perp}$	$\mathbf{x} \times \mathbf{C}_{t} \times \mathbf{C}_{i} \times \mathbf{C}_{b}$	= 684 lb/in ²						
Applied compression stress perpend	icular to grain;	$f_{c_perp} = R_{A_max}$	/ (N \times b \times L _b) =	= 86 lb/in ²						
		f _{c_perp} / F _{c_perp} ' =	= 0.126							
	PASS - Des	sign compressi	ve stress exc	eeds applied co	ompressive stre	ss 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 = 1242	lb/in ²					
Actual bending stress;		$f_b = M / S_x = 21$	1 32 lb/in ²							
		f _b / F _b ' = 1.717								
		FAIL -	Design bendi	ng stress is les	s than actual be	ending stress				
Strength in shear parallel to grain - c	1341									
Design shear stress:			$C_{1} \times C_{2} = 180$	h/in ²						
Actual shear stress - eq $3.4-2^{\circ}$		$f = 3 \times E / (2 \times E)$	$(\Delta) = 72$ lb/in ²	6/111						
		$f_{\rm e} = 0.398$	$(\mathbf{r}_{i}) = \mathbf{r}_{i} \mathbf{E}_{i} \mathbf{E}_{i}$							
		iy / i y = 0.330	PASS - Desid	n shear stress	exceeds actual	shear stress				
Deflection of 2.5.1				,						
Deflection - Cl.3.5.1			0.00 400							
Design deflection;		$E = E \times C_{ME} \times$	$U_t \times U_{iE} = 160$	0000 ID/IN						
Design deflection;		$\delta_{adm} = 0.003 \times L_{s1} = 0.648$ in								
Bending deflection;		$o_{b_{s1}} = 2.406$ in								
Shear deflection;	$\delta_{v_{s1}} = 0.042$ in									
Total deflection;	Total deflection;			$\delta_a = \delta_{b_s1} + \delta_{v_s1} = 2.448 \text{ in}$						
		δ_a / δ_{adm} = 3.77	8							

HARTLING MESTE ANNI	Project				Job Ref.	
a tull atuta	Section		Sheet no./rev.			
ASES				49 of 81		
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
			FAI	L - Design defl	ection exceeds	total deflection
NOTE - PURPOSE OF THIS CALC	WAS ONLY 1	TO CHECK SHE	AR AT NOTC	H OF RAFTER		



all the state of t	Project					Job Ref.		
A A A A A A A A A A A A A A A A A A A	Section				Sheet no./rev.			
ASES					51	of 81		
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date		
(C) 631-560-0259	AS	5/15/2012						
				Snow v 1.0	0			
					0 00			
		Creat 1			0.00			
		Span		Dead × 1.00	J			
				Live × 1.00	0			
				Show × 1.00	0 00			
		Querra ent D		Roof Live ×	0.00			
		Support B			J			
				Live × 1.00	.00			
				Snow × 1.00	0			
				Roof Live ×	0.00			
Analysis results								
Maximum moment;		M _{max} = 2070 lb	_ft;	$M_{min} = 0 \text{ Ib}_{-}$	ft			
Design moment;		M = max(abs(N	M _{max}),abs(M _{min}	n)) = 2070 lb_ft				
Maximum shear;		$F_{max} = 460 \text{ lb};$		F _{min} = -460	lb			
Design shear;		F = max(abs(F))	_{max}),abs(⊦ _{min)}) = 460 lb				
l otal load on member;		$VV_{tot} = 920 \text{ ID}$						
Reaction at support A;	Innort A.	$R_{A_{max}} = 460$ If); h	R _{A_min} = 460	di c			
Unfactored show load reaction at su	ipport Α; ipport Δ:	$R_{A}_{Dead} = 134$	b					
Linfactored roof live load reaction at a	support A:	R_{A} show = 300 l	34.lh					
Reaction at support B:	oupport A,	$R_{R} = 460$ lb):	R _{R min} = 46 () lb			
Unfactored dead load reaction at su	ipport B;	$R_{\text{B} \text{ bead}} = 154 \text{ lb}$						
Unfactored snow load reaction at su	ipport B;	$R_{B \text{ snow}} = 306 \text{ lb}$						
Unfactored roof live load reaction at	support B;	R _{B_Roof Live} = 23	4 lb					
Ē								
11.26				>				
<u>*</u> L		1						
-	I ← 1.5"	∠						
	A -4-							
Sawn lumber section details								
Nominal breadth of sections;		b _{nom} = 2 in						
Dressed breadth of sections;		b = 1.5 in						
Nominal depth of sections;		$a_{nom} = 12 \text{ in}$						
Number of sections in member		u = 11.23 m N = 1						
Overall broadth of member		$\mathbf{n} = \mathbf{I}$	5 in					
		$b_b = N \times b = 1.$	J					
Table 4A - Reference design values	s for visually gra	ded dimension I	umber (2"-4"	thick)				
Species, grade and size classification	on;	Douglas Fir-La	rch, No.2 gra	de, 2" & wider				
Bending parallel to grain;		r _b = 900 lb/in ²						

and arian messa and	Project				Job Ref.					
	Section				Sheet no./rev.					
ASES					52	of 81				
112 Wilson Drive. Port Jefferson. NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date				
(C) 631-560-0259	AS	5/15/2012								
l'ension parallel to grain;		$F_t = 5/5 \text{ ID/IN}$	2							
Compression parallel to grain;		$F_c = 1350 \text{ ib/in}$	1:-2							
Compression perpendicular to grain	,	$F_{c_perp} = 623 ID/$	(III)							
Shear parallel to grain;		$F_v = 180 \text{ lb/ln}^2$								
Moon choor modulus:		E = 1600000 m	100000 lb/ip ²							
mean shear modulus,		$G_{def} = E / 10 =$								
Member details										
Service condition;		Dry								
Length of bearing;		$L_b = 4$ in								
Load duration;		Ten years								
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;		$S_x = N \times b \times d^2$	$S_x = N \times b \times d^2 / 6 = 31.64 \text{ in}^3$							
		$S_y = d \times (N \times b)^2 / 6 = 4.22 in^3$								
Second moment of area:		$I_x = N \times b \times d^3 / 12 = 177.98 in^4$								
		$I_v = d \times (N \times b)^3$	3 / 12 = 3.16 in ⁴							
Adjustment factors		, , ,								
Load duration factor - Table 2.3.2:		$C_{2} = 1.00$								
Temperature factor - Table 2.3.2,		$C_{\rm D} = 1.00$								
Size factor for bending - Table 4A:		$C_{t} = 1.00$								
Size factor for tension - Table 4Λ :		$C_{\rm Ft} = 1.00$								
Size factor for compression - Table	1.0.	C _{Ec} = 1.00								
Flat use factor - Table 4A	<i>T</i> , ,	$C_{fu} = 1.20$								
Incising factor for modulus of elastic	ity - Table 4 3 8	: Cir = 1.00								
Incising factor for bending shear te	nsion & compre	ssion - Table 4 :	3.8							
		$C_i = 1.00$	3.0							
Incising factor for perpendicular corr	pression - Tabl	e 4 3 8								
		$C_{ic, perp} = 1.00$								
Repetitive member factor - cl.4.3.9:		Cr = 1.15								
Bearing area factor - eg.3.10-2:		$C_{\rm b} = (L_{\rm b} + 0.37)$	5 in) / L _b = 1.09							
Depth-to-breadth ratio:		$d_{\text{ser}} / (N \times h_{\text{ser}}) = 6.00$								
- Beam is fully restrained			,,							
Beam stability factor - cl.3.3.3;		C _L = 1.00								
Bearing perpendicular to grain - cl 3	10.2									
Design compression perpendicular t	o grain:	F '_F	$\times \mathbb{C} \times \mathbb{C} \times \mathbb{C} =$	681 lb/in^2						
Applied compression stress perpend	licular to grain:	$c_{perp} - \Gamma_{c_{perp}}$	$(\Lambda \cup_{t} \land \cup_{t} \land \cup_{b} = 7)$	7 lb/in ²						
Applied complession sitess perpend	ilcular to grain,	$f_{c_perp} = R_{A_max}$	$(\mathbf{N} \times \mathbf{D} \times \mathbf{L}_{\mathbf{b}}) = \mathbf{I}$							
$I_{c_perp} / \Gamma_{c_perp} = 0.112$						ee at hoaring				
	FAJJ - Des	ign compressi	re suess excee	as applied colli	Diessive sile	ss at nearnig				
Strength in bending - cl.3.3.1										
Design bending stress;		$F_b' = F_b \times C_D \times$	$C_t \times C_L \times C_{Fb} \times C_{Fb}$	$C_i \times C_r = 1035 \text{ lb/}$	in ⁻					
Actual bending stress;		$f_b = M / S_x = 78$	5 lb/in [∠]							
		$f_b / F_b' = 0.758$								

and this is a strange of the	Project				Job Ref.	
AUTOR .	Section				Sheet no./rev.	
					53	3 of 81
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
		PAS	S - Desian b	endina stress	exceeds actual b	endina stress
Otropath in choose possibility are in			e 200.g.r.s	onung on ooo		onung on ood
Strength in shear parallel to grain -	CI.3.4.1		00 400	lle /: ²		
Design snear stress;		$F_v = F_v \times C_D $	$C_t \times C_i = 180$	2 ID/IN		
Actual shear stress - eq.3.4-2;		$I_v = 3 \times F / (2 >$	(A) = 41 ID/IN			
		$I_v / F_v = 0.227$	BASS Doc	ian choor otro	an avanada antur	l choor strag
			PASS - Des	ign snear stre	ss exceeds actua	li snear stress
Deflection - cl.3.5.1				2		
Modulus of elasticity for deflection;		$E' = E \times C_{ME} \times$	$C_t \times C_{iE} = 16$	00000 lb/in²		
Design deflection;		δ_{adm} = 0.003 $ imes$	L _{s1} = 0.648 ir	ı		
Bending deflection;		$\delta_{b_{s1}} = 0.640$ in	ו			
Shear deflection;		$\delta_{v_s1} = 0.027$ ir	ו			
Total deflection;		$\delta_{a} = \delta_{b_s1} + \delta_{v_}$	_{s1} = 0.666 in			
		δ_a / δ_{adm} = 1.02	28			
			FAIL	- Design def	lection exceeds to	otal deflection
AGAIN - DEFLECTION FAILS BY L	LESS THAN 39					
AGAIN - DEFLECTION FAILS BY I NOW CHECK AGAIN FOR SHEAR <u>STRUCTURAL WOOD BEAM AN</u>	LESS THAN 39 R DUE TO NOT ALYSIS & DES	76 MEREFORE (176 ONLY 176 (NDS 2005)				
AGAIN - DEFLECTION FAILS BY I NOW CHECK AGAIN FOR SHEAR STRUCTURAL WOOD BEAM ANA In accordance with the ASD method	LESS THAN 39 R DUE TO NOT ALYSIS & DES d	% HIEREFORE (FCH ONLY SIGN (NDS 2005)			TEDDS colouid	ation vorsion 1.5.0
AGAIN - DEFLECTION FAILS BY I NOW CHECK AGAIN FOR SHEAR STRUCTURAL WOOD BEAM ANA In accordance with the ASD method	LESS THAN 39 R DUE TO NOT ALYSIS & DES d	76 MEREFORE (176 ONLY 176 (NDS 2005)			TEDDS calcula	ation version 1.5.04
AGAIN - DEFLECTION FAILS BY I NOW CHECK AGAIN FOR SHEAR STRUCTURAL WOOD BEAM ANA In accordance with the ASD method	LESS THAN 39 R DUE TO NOT <u>ALYSIS & DES</u> d	CH ONLY	ination 1		TEDDS calcula	ation version 1.5.04
AGAIN - DEFLECTION FAILS BY I NOW CHECK AGAIN FOR SHEAR STRUCTURAL WOOD BEAM AND In accordance with the ASD method $0.050 ext{ } ex$	LESS THAN 34 R DUE TO NOT ALYSIS & DES d	TCH ONLY SIGN (NDS 2005) Load Envelope - Comb	ination 1		TEDDS calcula	ation version 1.5.04
AGAIN - DEFLECTION FAILS BY I NOW CHECK AGAIN FOR SHEAR STRUCTURAL WOOD BEAM AND In accordance with the ASD method $0.050 \int_{t_A}^{0.050} \int_{$	LESS THAN 39 R DUE TO NOT ALYSIS & DES d	TCH ONLY SIGN (NDS 2005) Load Envelope - Comt	ination 1		TEDDS calcula	ation version 1.5.04
AGAIN - DEFLECTION FAILS BY I NOW CHECK AGAIN FOR SHEAR STRUCTURAL WOOD BEAM AND In accordance with the ASD method $0.050 ext{ } ex$	LESS THAN 34 R DUE TO NOT ALYSIS & DES d	CH ONLY SIGN (NDS 2005) Load Envelope - Comb	ination 1		TEDDS calcula	ation version 1.5.04
AGAIN - DEFLECTION FAILS BY I NOW CHECK AGAIN FOR SHEAR STRUCTURAL WOOD BEAM ANA In accordance with the ASD method	LESS THAN 34 R DUE TO NOT ALYSIS & DES d	TCH ONLY SIGN (NDS 2005) Load Envelope - Comt 1 1 1 1 1 1 1 1 1 1 1 1 1	velope		TEDDS calcula	ation version 1.5.04
AGAIN - DEFLECTION FAILS BY I NOW CHECK AGAIN FOR SHEAR STRUCTURAL WOOD BEAM AND In accordance with the ASD method	LESS THAN 34 R DUE TO NOT ALYSIS & DES d	TCH ONLY SIGN (NDS 2005) Load Envelope - Comb	ination 1 8 velope		TEDDS calcula	ation version 1.5.04
AGAIN - DEFLECTION FAILS BY I NOW CHECK AGAIN FOR SHEAR STRUCTURAL WOOD BEAM AN/ In accordance with the ASD method	LESS THAN 34 R DUE TO NOT ALYSIS & DES d	TCH ONLY SIGN (NDS 2005) Load Envelope - Comb Bending Moment En	velope		TEDDS calcula	ation version 1.5.04
AGAIN - DEFLECTION FAILS BY I NOW CHECK AGAIN FOR SHEAR STRUCTURAL WOOD BEAM AND In accordance with the ASD method $0.050 \int_{t_A}^{0.050} \int_{$	LESS THAN 34 R DUE TO NOT ALYSIS & DES d	1 CH ONLY SIGN (NDS 2005) Load Envelope - Comt Bending Moment En	ination 1		TEDDS calcula	ation version 1.5.04
AGAIN - DEFLECTION FAILS BY I NOW CHECK AGAIN FOR SHEAR STRUCTURAL WOOD BEAM AND In accordance with the ASD method 0.050 0.050 0.050 0.050 0.050 0.050 0.050 0.050 0.050 0.050 0.050 0.050 0.050 0.050 1 1 1 1	LESS THAN 34 R DUE TO NOT ALYSIS & DES d	2 THEREFORE C	velope		TEDDS calcula	ation version 1.5.04
AGAIN - DEFLECTION FAILS BY I NOW CHECK AGAIN FOR SHEAR STRUCTURAL WOOD BEAM AND In accordance with the ASD method 0.050 0.050 0.0 t_{A}	LESS THAN 34 R DUE TO NOT ALYSIS & DES d	CH ONLY SIGN (NDS 2005) Load Envelope - Comt Bending Moment En 2	velope		TEDDS calcula	ation version 1.5.04
AGAIN - DEFLECTION FAILS BY I NOW CHECK AGAIN FOR SHEAR STRUCTURAL WOOD BEAM AND In accordance with the ASD method 0.050 0.050 0.0 t_A $kip_h t_A$	LESS THAN 34 R DUE TO NOT ALYSIS & DES d	CH ONLY	velope		TEDDS calcula	ation version 1.5.04



04 ^{0,110,0} 105 17 0.0	Project				Job Ref.		
	Section				Sheet no./rev.		
ASES					55	of 81	
A.S. Engineering Sarvices (PC).	Calc by	Date	Chk'd by	Date	App'd by	Date	
112 Wilson Drive, Port Jefferson, NY 11777			Clikaby	Date	дрр а бу	Date	
(C) 631-560-0259	AS	5/15/2012					
				\leq			
				\geq			
	◄ —4" — ►						
Sawn lumber section details							
Nominal breadth of sections;		$b_{nom} = 2$ in					
Dressed breadth of sections;		b = 1.5 in					
Nominal depth of sections;		$d_{nom} = 8$ in					
Dressed depth of sections;		d = 7.25 in					
Number of sections in member;		N = 1					
Overall breadth of member;		$b_b = N \times b = 1.5$	5 in				
Table 4A - Reference design values	for visually grad	ded dimension lu	umber (2"-4" thi	ck)			
Species, grade and size classification	on.	Douglas Fir-La	rch No 2 grade	2" & wider			
Bending parallel to grain:	,	$F_{\rm b} = 900 \text{lb/in}^2$	grade				
Tension parallel to grain;		$F_{t} = 575 \text{ lb/in}^{2}$					
Compression parallel to grain;		$F_{2} = 1350 \text{ lb/in}^{2}$	2				
Compression perpendicular to grain,		$F_{0} = 625 \text{ lb}$	/in ²				
Shear parallel to grain:	,	$F_{\rm u} = 180 \text{ lb/in}^2$					
Modulus of elasticity:		E - 1600000 lb	/in ²				
Mean shear modulus:		E = 1000000 m	100000 lb/in ²				
Member details							
Service condition;		Dry					
Length of bearing;		$L_b = 4$ in					
Load duration;		ien years					
The beam is one of three or more re	epetitive membe	rs					
Section properties							
Cross sectional area of member;		$A = N \times b \times d =$	10.87 in ²				
Section modulus;		$S_x = N \times b \times d^2$	/ 6 = 13.14 in ³				
		$S_v = d \times (N \times b)$	$^{2}/6 = 2.72 \text{ in}^{3}$				
Second moment of area:		$I_x = N \times b \times d^3$, 12 = 47.63 in ⁴				
		$l_v = d \times (N \times h)^2$	$^{3}/12 = 2.04 \text{ in}^{4}$				
Adjustment factors		,					
Aujustinent lactors		$C_{-} = 1.00$					
Temperature factor - Table 2.3.2;		$C_{\rm D} = 1.00$					
Piero footon footon footon - Table 2.3.3;		$C_t = 1.00$					
Size factor for bending - Table 4A;		$C_{Fb} = 1.20$					
Size factor for tension - Table 4A;		C _{Ft} = 1.20					
Size factor for compression - Table	4A;	C _{Fc} = 1.05					
Flat use factor - Table 4A;		C _{fu} = 1.15					

and tarian men a ange	Project				Job Ref.	
a tatlatata	Section				Sheet no /rev	
ASES	Section				56	6 of 81
A.S. Engineering Services, PC.	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
Incising factor for modulus of elasti	city - Table 4.3 8	3 [.] C _{i∈} = 1 00				
Incising factor for bending, shear, th	ension & compre	ession - Table 4.	3.8			
inclosing factor for Softanig, critar, t		$C_i = 1.00$				
Incising factor for perpendicular co	mpression - Tab	le 4.3.8				
c		C _{ic_perp} = 1.00				
Repetitive member factor - cl.4.3.9;	;	C _r = 1.15				
Bearing area factor - eq.3.10-2;		$C_{b} = (L_{b} + 0.37)$	75 in) / L _b = 1.	09		
Depth-to-breadth ratio;		d_{nom} / (N $ imes$ b_{nor}	m) = 4.00			
- Beam is fully restrained						
Beam stability factor - cl.3.3.3;		$C_{\text{L}}=\textbf{1.00}$				
Bearing perpendicular to grain - cl.	3.10.2					
Design compression perpendicular	to grain;	$F_{c perp}' = F_{c per}$	$_{\rm p} \times {\rm C_t} \times {\rm C_i} \times {\rm C}$	$S_{\rm b} = 684 \text{lb/in}^2$		
Applied compression stress perper	ndicular to grain;	$f_{c,perp} = R_{A,max}$	$/(N \times b \times L_b)$	= 74 lb/in ²		
	3 <i>i</i>	f _{c perp} / F _{c perp} '	= 0.109			
	PASS - Des	sign compressi	ive stress exc	ceeds applied	compressive str	ess at bearing
Strength in bending - cl.3.3.1						
Design bending stress;		$F_{b}' = F_{b} \times C_{D} \times$	$\mathbf{C}_{t} \times \mathbf{C}_{L} \times \mathbf{C}_{Fb}$	$\times C_i \times C_r = 124$	12 lb/in ²	
Actual bending stress;		$f_{b} = M / S_{x} = 18$	836 lb/in ²			
		f _b / F _b ' = 1.478				
		FAIL -	Design bend	ing stress is le	ess than actual b	ending stress
Strength in shear parallel to grain -	cl.3.4.1					
Design shear stress;		$F_v' = F_v \times C_D \times$	$C_t \times C_i = 180$	lb/in ²		
Actual shear stress - eq.3.4-2;		$f_v = 3 \times F / (2 >$	< A) = 62 lb/in	2		
		f _v / F _v ' = 0.342				
			PASS - Desi	ign shear stres	s exceeds actua	l shear stress
Deflection - cl.3.5.1						
Modulus of elasticity for deflection:		$E' = E \times C_{ME} \times$	$C_t \times C_{iF} = 160$	00000 lb/in ²		
Design deflection:		$\delta_{adm} = 0.003 \times$	L _{s1} = 0.648 ir)		
Bending deflection:		$\delta_{\rm b}$ s1 = 2.344 it	า			
Shear deflection:		$\delta_{\rm v, s1} = 0.041$ ir	า			
Total deflection:		$\delta_a = \delta_{h s1} + \delta_v$	_{s1} = 2.385 in			
		$\delta_a / \delta_{adm} = 3.68$	30			
			FAIL	Design defle	ection exceeds t	otal deflectior
CHECK WAS ONLY FOR SHEAR						



Section 112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259 Calc. by AS 5/15/2012 Chk'd by Date Span 1	Sheet no./rev. 58 of 81 App'd by Date Live × 1.00 × 1.00
Section 112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259 Calc. by AS Date 5/15/2012 Chk'd by Date Date Roof L Snow Span 1 Dead	Sheet no./rev. 58 of 81 App'd by Date .ive × 1.00 × 1.00 × 1.00 × 1.00
Calc. by Date Chk'd by Date 112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259 Calc. by Date 5/15/2012 Date Roof L Snow Span 1 Dead	58 of 81 App'd by Date Live × 1.00 × 1.00 × 1.00 × 1.00
112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259 Calc. by AS Date Chk'd by Date Roof L Snow Span 1 Dead	App'd by Date Date Live × 1.00 × 1.00 × 1.00
(C) 631-560-0259 AS 5/15/2012 Roof L Snow Span 1 Dead	ive × 1.00 × 1.00 × 1.00
Roof L Snow Span 1 Dead	Live × 1.00 × 1.00 × 1.00
Snow Span 1 Dead	× 1.00 × 1.00 × 1.00
Show Span 1 Dead	× 1.00 × 1.00
Span 1 Dead	× 1.00
Live ×	1.00
Root L	live × 1.00
Snow	× 1.00
Support B Dead	× 1.00
Live ×	1.00
Roof L	$ive \times 1.00$
Snow	× 1.00
Analysis results	
Maximum moment; M _{max} = 66230 lb_ft; M _{min} =	-90 lb_ft
Design moment; $M = max(abs(M_{max}),abs(M_{min})) = 66230 lb_{Min}$	_ft
Maximum shear; $F_{max} = 10000$ lb; $F_{min} =$	-10367 lb
Design shear; $F = max(abs(F_{max}), abs(F_{min})) = 10367$ lb	
Total load on member; $W_{tot} = 20368$ lb	
Reaction at support A; $R_{A_{max}} = 10000 \text{ lb};$ $R_{A_{min}}$	= 10000 lb
Unfactored dead load reaction at support A; $R_{A_Dead} = 5629$ lb	
Unfactored live load reaction at support A; $R_{A_Live} = 4371$ lb	
Reaction at support B; $R_{B_{max}} = 10367 \text{ lb};$ $R_{B_{min}}$	= 10367 lb
Unfactored dead load reaction at support B; $R_{B_Dead} = 5828$ lb	
Unfactored live load reaction at support B; $R_{B_{Live}} = 4539$ ib	
$[\mathbf{x}, \mathbf{x}] = \mathbf{x} + \mathbf{x} +$	
Composite section details	
Breadth of composite section; $b = 1.5$ in	
Depth of composite section; $d = 20$ in	
Number of composite sections in member; $N = 4$	
Overall breadth of composite member; $b_b = N \times b = 6$ in	
Reference design values for structural composite lumber	
Composite type and grade; Microllam LVL, 1.9E-2600Fb grade	
Bending parallel to grain; $F_b = 2600 \text{ lb/in}^2$	
Tension parallel to grain; $F_t = 1555 \text{ lb/in}^2$	
Compression parallel to grain; $F_c = 2510 \text{ lb/in}^2$	
Compression perpendicular to grain; $F_{c_perp} = 750 \text{ lb/in}^2$	
Shear parallel to grain; $F_v = 285 \text{ lb/in}^2$	

advision means of a	Project				Job Ref.	
AUTORA .	Section				Sheet no./rev.	
ASES					59	of 81
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
Modulus of elasticity:		E - 190000 lk	v/in ²			
Mean shear modulus:		$G_{dof} = F / 16 =$	118750 lb/in ²			
Average density:		$0 = 42 \text{ lb/ft}^3$				
Nombor detaile		p 12				
		Dry				
Length of bearing:		L – Alin				
Length of bearing,		L _b = 4 III				
		Tell years				
Section properties		A Nissia sal	100 00 i= ²			
Cross sectional area of member;		$A = N \times D \times d =$	= 120.00 In			
Section modulus;		$S_x = N \times D \times d$	$76 = 400.00 \text{ ln}^{2}$	3		
		$S_y = d \times (N \times b)$) ⁻ / 6 = 120.00 ir	4		
Second moment of area;		$I_x = N \times b \times d^{\circ}$	/ 12 = 4000.00 ir	1		
		$I_y = d \times (N \times b)^{\circ}$	° / 12 = 360.00 ir	י'		
Adjustment factors						
Load duration factor - Table 2.3.2;		C _D = 1.00				
Temperature factor - Table 2.3.3;		C _t = 1.00	0.13	16		
Size factor for bending;		$C_{Fb} = (12 \text{ in } / \text{ m})$	nax(d, 3.5 in)) ^{0.13}	¹⁰ = 0.93		
Size factor for shear;		$C_{Fv} = (12 \text{ in } / \text{ d})$	$)^{0.130} = 0.93$			
Repetitive member factor - cl.8.3.7;		$C_r = 1.00$				
Length factor;		$C_{\text{Len}} = 1.00$				
Bearing area lactor - eq.3.10-2;		$C_b = 1.00$				
Depth-to-breadth ratio;		$d / (N \times b) = 3.$	33			
- Beam is fully restrained Beam stability factor - cl 3 3 3:		$C_{1} = 1.00$				
Beam stability factor - cl.3.3.3,		$C_{L} = 1.00$				
Bearing perpendicular to grain - cl.3	.10.2					
Design compression perpendicular	to grain;	$F_{c_perp} = F_{c_perp}$	$5 \times C_t \times C_b = 750$			
Applied compression stress perpend	dicular to grain;	$f_{c_perp} = R_{B_max}$	$/(N \times b \times L_b) = 4$	132 lb/in ²		
	-	t _{c_perp} / ⊢ _{c_perp} ' =	= 0.576			
	PASS - Des	ign compressi	ve stress excee	as applied com	oressive stre	ss at bearing
Strength in bending - cl.3.3.1						
Design bending stress;		$F_{b}' = F_{b} \times C_{D} \times$	$C_t \times C_L \times C_{Fb} \times C_{Fb}$	$C_r = 2426 \text{ lb/in}^2$		
Actual bending stress;		$f_{b} = M / S_{x} = 19$	187 lb/in²			
		$f_{\rm b} / F_{\rm b}' = 0.819$				
		PAS	S - Design bend	ling stress exce	eds actual be	nding stress
Strength in shear parallel to grain - o	cl.3.4.1					
Design shear stress;		$F_v' = F_v \times C_D \times$	$C_t \times C_{Fv} = 266 \text{ lk}$	o∕in²		
Actual shear stress - eq.3.4-2;		$f_v = 3 \times F / (2 \times$: A) = 130 lb/in ²			
		f _v / F _v ' = 0.487				
			PASS - Design	shear stress ex	ceeds actual	shear stress
Deflection - cl.3.5.1				_		
Modulus of elasticity for deflection;		$E'=E\timesC_M\timesC$	C _t = 1900000 lb/i	n ²		
Design deflection;		δ_{adm} = 0.004 \times	L _{s1} = 1.272 in			

IN DISCOUNTER ADA	Project				Job Ref.	
Anthatula	Section				Sheet no./rev.	
ASES					60	of 81
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
Bending deflection:		δ _{b s1} = 1.101 ir	1			
Shear deflection;		δ _{v s1} = 0.067 in	l			
Total deflection;		$\delta_a = \delta_{b_s1} + \delta_{v_s}$	₁ = 1.168 in			
		δ_a / δ_{adm} = 0.91	9			
			PASS - L	Design deflection	າ is less than to	tal deflection

advarian misza ada	Project				Job Ref.	Job Ref.	
ATTLETTE	Section				Sheet no./rev.		
ASES					61	of 81	
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date	
(C) 631-560-0259	AS	5/15/2012					
		•	•	•	•	•	

DINING ROOM HEADER

STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.04



In the second second second	Project				Job Ref.	
a deal and the	Castion				Chaot no /roy	
ASES	Section				Sneet no./rev	7. 62 of 81
A.S. Engineering Dervices, PCL		Data	Chk'd by	Data	App'd by	02 01 8 1
112 Wilson Drive, Port Jefferson, NY 11777		5/15/2012	Clik d by	Dale	Арр и бу	Dale
(0) 031-300-0239	70	5/15/2012				
		Span 1		Dead × 1	.00	
				Live \times 1.0	00	
		Support B		$Dead \times 1$.00	
				Live × 1.0	00	
Analyzia reculto						
Analysis results		M _ 5527 lb	. ft.	M – 0	lh ft	
Maximum moment,		$M_{max} = 5537 \text{ ID}$	$(M_{\rm obs})$	$W_{min} = 0$	ID_II	
Maximum shear:		F = -2768 lb	vi _{max}),abs(ivi _{mi}	$n)) = 5557 \text{ ID}_11$	768 lb	
Dosign shoar:		$F_{max} = 2708 \text{ ID},$ $F = max(abc)(F_{max})$, babe(E)	$\Gamma_{min} = -21$		
Total load on member:		r = max(abs(r)	max),aDS(rmin)) = 2766 lb		
Position of support A:		$VV_{tot} = 3337$ ID	lb.	D (769 lb	
Reaction at support A,	pport A.	$R_{A_{max}} = 2700$	ID,	$R_{A_{min}} = 4$		
Unfactored live load reaction at su	ppon A,	$R_{A_{Dead}} = 1000$				
Prostion of support P:	port A,	$R_{A_{Live}} = 900 \text{ IL}$) Ib:	P		
Linfactored doed load reaction at su	nnort R.	$R_{B_{max}} = 2100$	ib,	$R_B_{min} = 4$	2700 10	
Linfactored live load reaction at sup	ppon B,	$R_{B_{Dead}} = 1000$				
Official ive load reaction at sup	port B,	$RB_{Live} = 300 \text{ fc}$, ,			
	 ∢ −4".	_ →				
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} = 10 in				
Dressed depth of sections;		d = 9.25 in				
Number of sections in member;		N = 3				
Overall breadth of member;		$b_b = N \times b = 4.$	5 in			
Table 4A - Reference design values	for visually gra	ded dimension I	umber (2"-4"	thick)		
Species, grade and size classification	on;	Douglas Fir-La	rch, No.2 gra	de, 2" & wider		
Bending parallel to grain;		$F_{b} = 900 \text{ lb/in}^{2}$				
Tension parallel to grain;		$F_t = 575 \text{ lb/in}^2$	_			
Compression parallel to grain;		F _c = 1350 lb/in	2			
Compression perpendicular to grain	;	$F_{c_{perp}} = 625 \text{ lb}$	/in ²			
Shear parallel to grain;		$F_v = 180 \text{ lb/in}^2$	0			
Modulus of elasticity;		E = 1600000 lk	o∕in [∠]	0		
Mean shear modulus;		$G_{def} = E / 16 =$	100000 lb/in ²	-		
Member details						
Service condition;		Dry				

ADDALING MEST ADDA	Project				Job Ref.	
. Anti-title .	Section				Sheet no./rev.	
ASE S		T	I	1	63	of 81
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
Length of bearing:		L _b = 4 in				
Load duration;		Ten years				
Section properties						
Cross sectional area of member;		$A = N \times b \times d =$	41.62 in ²			
Section modulus;		$S_x = N \times b \times d^2$	/ 6 = 64.17 in ³			
		$S_y = d \times (N \times b)$	$)^{2} / 6 = 31.22 \text{ in}^{3}$			
Second moment of area;		$I_x = N \times b \times d^3$	/ 12 = 296.79 in ⁴	l l		
		$I_y = d \times (N \times b)$	³ / 12 = 70.24 in ²	1		
Adjustment factors						
Load duration factor - Table 2.3.2;		C _D = 1.00				
Temperature factor - Table 2.3.3;		$C_t = 1.00$				
Size factor for bending - Table 4A;		C _{Fb} = 1.10				
Size factor for tension - Table 4A;		C _{Ft} = 1.10				
Size factor for compression - Table	4A;	$C_{\text{Fc}} = \textbf{1.00}$				
Flat use factor - Table 4A;		C _{fu} = 1.20				
Incising factor for modulus of elastic	ity - Table 4.3.8	; C _{iE} = 1.00				
Incising factor for bending, shear, te	ension & compre	ssion - Table 4.	3.8			
		C _i = 1.00				
Incising factor for perpendicular cor	npression - Tabl	e 4.3.8				
Popotitivo mombor factor ol 4.3.0:		$C_{ic_{perp}} = 1.00$				
Bearing area factor - eg 3 10-2:		$C_r = 1.13$				
Depth-to-breadth ratio:		$d_{\text{port}} / (N \times b_{\text{port}})$	-1.67			
- Beam is fully restrained			1) = 1.07			
Beam stability factor - cl.3.3.3;		C _L = 1.00				
Bearing perpendicular to grain - cl.3	.10.2					
Design compression perpendicular	to grain;	$F_{c_perp}' = F_{c_perp}$	$\mathbf{b} \times \mathbf{C}_{\mathrm{t}} \times \mathbf{C}_{\mathrm{i}} \times \mathbf{C}_{\mathrm{b}} =$	625 lb/in ²		
Applied compression stress perpen	dicular to grain;	f _{c_perp} = R _{B_max}	$/(N \times b \times L_b) = 1$	54 lb/in ²		
		f _{c_perp} / F _{c_perp} ' =	= 0.246			
	PASS - Des	ign compressi	ve stress excee	eds applied com	pressive stre	ss at bearing
Strength in bending - cl.3.3.1						
Design bending stress;		$F_{b}' = F_{b} \times C_{D} \times$	$C_t \times C_L \times C_{Fb} \times 0$	C _i × C _r = 1139 lb/	in ²	
Actual bending stress;		$f_{b} = M / S_{x} = 10$)35 lb/in ²			
		$f_{b} / F_{b}' = 0.909$				
		PAS	S - Design bend	ling stress exce	eds actual be	nding stress
Strength in shear parallel to grain -	cl.3.4.1					
Design shear stress;		$F_v' = F_v \times C_D \times$	$C_t \times C_i = 180 \text{ lb/}$	in ²		
Actual shear stress - eq.3.4-2;		$f_v = 3 \times F / (2 \times F)$	(A) = 100 lb/in ²			
		$f_v / F_v' = 0.554$				
			PASS - Design	shear stress ex	ceeds actual	shear stress
Deflection - cl.3.5.1						
Modulus of elasticity for deflection;		$E'=E\times C_{ME}\times$	$C_t \times C_{iE} = 16000$	100 lb/in ²		

and the second design of the second	Project				Job Ref.	
a dati atada	Section				Sheet no./rev.	
ASES					64	of 81
AS transmit an and the second	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
		<u> </u>	0.000 ·			
Design deflection;		$\delta_{adm} = 0.003 \times$	L _{s1} = 0.288 in			
Bending deflection;		$o_{b_{s1}} = 0.134 \text{ In}$				
Total deflection:		$\delta_{v_s1} = 0.019$ in	. - 0 153 in			
Total denection,		$\delta_a = 0_{b_s1} + 0_{V_s}$ $\delta_a / \delta_{adm} = 0.53$	1 – 0.155 m 3			
			PASS - De	sian deflection is	s less than to	tal deflection
				sign deneedon is		



water and Merry Days	Project				Job Ref.	
Anti-Anta	Section				Sheet no./rev.	6 of 91
A.S. Engineering Services, PIC.					6	6 01 81
112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259	Calc. by AS	Date 5/15/2012	Chk'd by	Date	App'd by	Date
				Live × 1.	00	
		Support B		Dead $ imes$ 1	1.00	
				Live × 1	00	
					00	
Analysis results		M _ 69502	h ft.	M – 0	lh ff	
Design moment:		$M_{max} = 00505$	u_n, A) abe(M	$(N_{min} = 0)$	10_11	
Maximum shear:		F = 10149	(imax),ab3(ivimi	n)) = 00303 lb_lt F ·1	0140 lb	
Design shear:		F = max(abs)(F)	,) abs(F)) – 10149 lb	0143 15	
Total load on member:		W 20297 It) = 10143 15		
Reaction at support A:		$R_{A} = 10149$	lh.	RA min =	10149 lb	
Unfactored dead load reaction at st	upport A.	RA_max = 1014	lb.	TTA_min =		
Unfactored live load reaction at sur	oport A:	$R_{A_beau} = 4388$	h			
Reaction at support B:	port A,	$R_{R_{max}} = 10149$) lb:	RR min =	10149 lb	
Unfactored dead load reaction at st	upport B:	R _{B Dood} = 5761	lb			
Unfactored live load reaction at sur	oport B:	$R_{B \perp ivo} = 4388$	b			
	port D,		~			
	 ¹ ¹ ¹ ¹ ¹ ¹ ¹ ¹	→ 4" ←				
Composite section details						
Breadth of composite section;		b = 1.75 in				
Depth of composite section;		d = 18 in				
Number of composite sections in m	nember;	N = 4				
Overall breadth of composite memb	ber;	$b_b = N \times b = 7$	n			
Reference design values for structu	iral composite li	umber				
Composite type and grade;		Microllam LVL	1.9E-2600F	o grade		
Composite type and grade; Bending parallel to grain;		Microllam LVL $F_b = 2600$ lb/in	1.9E-2600FI 2	o grade		
Composite type and grade; Bending parallel to grain; Tension parallel to grain;		$\label{eq:result} \begin{aligned} \text{Microllam LVL} \\ \text{F}_{\text{b}} &= \textbf{2600} \text{ lb/in} \\ \text{F}_{\text{t}} &= \textbf{1555} \text{ lb/in} \end{aligned}$	1.9E-2600FI	o grade		
Composite type and grade; Bending parallel to grain; Tension parallel to grain; Compression parallel to grain;		$\begin{aligned} \text{Microllam LVL} \\ F_{b} &= \textbf{2600} \text{ lb/in} \\ F_{t} &= \textbf{1555} \text{ lb/in}^{2} \\ F_{c} &= \textbf{2510} \text{ lb/in} \end{aligned}$	1.9E-2600Fl	o grade		
Composite type and grade; Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain	n;	$\label{eq:higher} \begin{array}{l} \mbox{Microllam LVL} \\ \mbox{F}_{b} = {\bf 2600} \mbox{ lb/in} \\ \mbox{F}_{t} = {\bf 1555} \mbox{ lb/in} \\ \mbox{F}_{c} = {\bf 2510} \mbox{ lb/in} \\ \mbox{F}_{c_perp} = {\bf 750} \mbox{ lb} \end{array}$	1.9E-2600Fl 2 2 /in ²	o grade		
Composite type and grade; Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain;	n;	$\begin{aligned} \text{Microllam LVL} \\ F_{b} &= 2600 \text{ lb/in} \\ F_{t} &= 1555 \text{ lb/in}^{2} \\ F_{c} &= 2510 \text{ lb/in} \\ F_{c_perp} &= 750 \text{ lb} \\ F_{v} &= 285 \text{ lb/in}^{2} \end{aligned}$	1.9E-2600FH	o grade		
Composite type and grade; Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain; Modulus of elasticity;	n;	$\begin{aligned} & \text{Microllam LVL} \\ & F_b = 2600 \text{ lb/in} \\ & F_t = 1555 \text{ lb/in} \\ & F_c = 2510 \text{ lb/in} \\ & F_{c_perp} = 750 \text{ lb} \\ & F_v = 285 \text{ lb/in}^2 \\ & E = 1900000 \text{ lb} \end{aligned}$	1.9E-2600Fl 2 /in ² //in ²	o grade		
Composite type and grade; Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain; Modulus of elasticity; Mean shear modulus;	n;	Microllam LVL $F_b = 2600$ lb/in $F_t = 1555$ lb/in ² $F_c = 2510$ lb/in $F_{c_perp} = 750$ lb $F_v = 285$ lb/in ² E = 1900000 lB $G_{def} = E / 16 =$	1.9E-2600Fl 2 /in ² /in ² 118750 lb/in ²	o grade		
Composite type and grade; Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain; Modulus of elasticity; Mean shear modulus; Average density;	n;	$\begin{array}{l} \mbox{Microllam LVL} \\ F_b = 2600 \mbox{ lb/in} \\ F_t = 1555 \mbox{ lb/in}^2 \\ F_c = 2510 \mbox{ lb/in} \\ F_{c_perp} = 750 \mbox{ lb} \\ F_v = 285 \mbox{ lb/in}^2 \\ E = 1900000 \mbox{ lk} \\ G_{def} = E \slash 16 = \\ \rho = 42 \mbox{ lb/ft}^3 \end{array}$	1.9E-2600Fl 2 /in ² //in ² 118750 lb/in ²	o grade		
Composite type and grade; Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain; Modulus of elasticity; Mean shear modulus; Average density; Member details	n;	$\begin{aligned} & \text{Microllam LVL} \\ F_b = 2600 \text{ lb/in} \\ F_t = 1555 \text{ lb/in}^2 \\ F_c = 2510 \text{ lb/in}^2 \\ F_c = 2510 \text{ lb/in}^2 \\ F_v = 285 \text{ lb/in}^2 \\ E = 1900000 \text{ ll} \\ G_{def} = E / 16 = \\ \rho = 42 \text{ lb/ft}^3 \end{aligned}$	1.9E-2600Fl 2 /in ² /in ² 118750 lb/in ²	o grade		
Composite type and grade; Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain; Modulus of elasticity; Mean shear modulus; Average density; Member details Service condition;	n;	$\label{eq:result} \begin{array}{l} \mbox{Microllam LVL} \\ F_b = 2600 \mbox{ lb/in} \\ F_t = 1555 \mbox{ lb/in}^2 \\ F_c = 2510 \mbox{ lb/in} \\ F_{c_perp} = 750 \mbox{ lb} \\ F_v = 285 \mbox{ lb/in}^2 \\ E = 1900000 \mbox{ lb} \\ G_{def} = E \mbox{ / } 16 = \\ \rho = 42 \mbox{ lb/ft}^3 \end{array}$	1.9E-2600Fl 2 /in ² /in ² 118750 lb/in ²	o grade		
Composite type and grade; Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain; Modulus of elasticity; Mean shear modulus; Average density; Member details Service condition; Length of bearing;	n;	$\begin{aligned} & \text{Microllam LVL} \\ F_b &= 2600 \text{ lb/in} \\ F_t &= 1555 \text{ lb/in}^2 \\ F_c &= 2510 \text{ lb/in} \\ F_{c_perp} &= 750 \text{ lb} \\ F_v &= 285 \text{ lb/in}^2 \\ E &= 1900000 \text{ lk} \\ G_{def} &= E / 16 = \\ \rho &= 42 \text{ lb/ft}^3 \end{aligned}$	1.9E-2600Fl 2 /in ² /in ² 118750 lb/in ²	o grade		

ADDALTION MENTE OFF.	Project				Job Ref.	
	Section				Sheet no./rev.	
ASES					67	of 81
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
Section properties						
Cross sectional area of member:		$A = N \times b \times d =$	= 126.00 in ²			
Section modulus:		$S_x = N \times b \times d^2$	/ 6 = 378.00 i	n ³		
,		$S_v = d \times (N \times b)$	$)^{2}/6 = 147.00$) in ³		
Second moment of area;		$I_x = N \times b \times d^3$, / 12 = 3402.00) in⁴		
		$I_v = d \times (N \times b)$	³ / 12 = 514.5 0	0 in⁴		
Adjustment factors		, , ,				
Load duration factor - Table 2.3.2:		C _D = 1.00				
Temperature factor - Table 2.3.3:		$C_t = 1.00$				
Size factor for bending;		$C_{Fb} = (12 \text{ in } / \text{ n})$	nax(d, 3.5 in)) ⁽	^{0.136} = 0.95		
Size factor for shear:		$C_{Fv} = (12 \text{ in } / \text{ d})$	$^{0.136} = 0.95$			
Repetitive member factor - cl.8.3.7;		C _r = 1.00	,			
Length factor;		C _{Len} = 1.00				
Bearing area factor - eq.3.10-2;		$C_{b} = (L_{b} + 0.37)$	′5 in) / L _b = 1.0)9		
Depth-to-breadth ratio;		d / (N × b) = 2 .	57			
- Beam is fully restrained						
Beam stability factor - cl.3.3.3;		C _L = 1.00				
Bearing perpendicular to grain - cl.3.1	0.2					
Design compression perpendicular to	grain;	Fc_perp' = Fc_perp	$0 \times C_t \times C_b = 82$	20 lb/in ²		
Applied compression stress perpendi	cular to grain;	$f_{c_perp} = R_{A_max}$	/ (N $ imes$ b $ imes$ L _b) =	= 362 lb/in ²		
		fc_perp / Fc_perp' =	= 0.442			
	PASS - Des	ign compressi	ve stress exc	eeds applied co	ompressive stre	ss at bearing
Strength in bending - cl.3.3.1						
Design bending stress;		$F_b' = F_b \times C_D \times$	$C_t \times C_L \times C_{Fb}$	× C _r = 2461 lb/in	2	
Actual bending stress;		$f_b = M / S_x = 21$	1 75 lb/in ²			
		$f_{b} / F_{b}' = 0.884$				
		PAS	S - Design be	nding stress ex	ceeds actual be	ending stress
Strength in shear parallel to grain - cl.	.3.4.1					
Design shear stress;		$F_v{'}=F_v\times C_D\times$	$C_t \times C_{Fv} = 270$) lb/in ²		
Actual shear stress - eq.3.4-2;		$f_v = 3 \times F / (2 \times F)$	(A) = 121 lb/in	1 ²		
		$f_v / F_v' = 0.448$				
			PASS - Desig	gn shear stress	exceeds actual	shear stress
Deflection - cl.3.5.1						
Modulus of elasticity for deflection;		$E' = E \times C_M \times C_M$	Ct = 1900000 I	b/in ²		
Design deflection;		$\delta_{adm} = 0.004 \times$	L _{s1} = 1.296 in			
Bending deflection;		$\delta_{b_{s1}} = 1.391$ ir	1			
Shear deflection;		$\delta_{v_s1} = 0.066$ in	1			
Total deflection;		$\delta_a = \delta_{b_s1} + \delta_{v_s}$	₃₁ = 1.457 in			
		$\delta_a / \delta_{adm} = 1.12$	4			
			FAIL	- Design deflec	tion exceeds to	tal deflection

Indivision means Days	Project				Job Ref.	
- Anti-stude	Section		Sheet no./rev.	Sheet no./rev.		
ASES	Calc. by	Date	Chk'd by	Date	6 App'd by	B of 81
112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259	AS	5/15/2012		Dailo	, pp & ~)	

an and the Man and the	Project		Job Ref.			
ASES	Section				Sheet no./rev. 69 of 81	
112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259	Calc. by AS	Date 5/15/2012	Chk'd by	Date	App'd by	Date

6x6 HEADER

STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.04



and the second state of the	Project				Job Ref.	
Section					Sheet no./rev.	
AS Engineering Services P.C.					70	of 81
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
				$1 ive \times 1.00$		
		Support P			0	
		Support B			0	
				Live × 1.00		
Analysis results						
Maximum moment;		M _{max} = 2427 lb	_ft;	M _{min} = 0 lb_	_ft	
Design moment;		M = max(abs(N	/I _{max}),abs(M _{mi}	n)) = 2427 lb_ft		
Maximum shear;		F _{max} = 1618 lb;		F _{min} = -161	B lb	
Design shear;		F = max(abs(F	_{max}),abs(F _{min})) = 1618 lb		
Total load on member;		W _{tot} = 3236 lb				
Reaction at support A;		R _{A_max} = 1618	b;	R _{A_min} = 16	18 lb	
Unfactored dead load reaction at su	ipport A;	R _{A_Dead} = 943 I	C			
Unfactored live load reaction at sup	port A;	$R_{A_Live} = 675$ lb				
Reaction at support B;		R _{B_max} = 1618	b;	R _{B_min} = 16	18 lb	
Unfactored dead load reaction at su	ipport B;	R _{B_Dead} = 943 I	0			
Unfactored live load reaction at sup	port B;	$R_{B_{Live}} = 675$ lb				
↓ (c) (c) (c) (c) (c) (c) (c) (c)	4"	+				
Sawn lumber section details						
Nominal breadth of sections;		$b_{nom} = 6$ in				
Dressed breadth of sections;		b = 5.5 in				
Nominal depth of sections;		$d_{nom} = 6$ in				
Dressed depth of sections;		d = 5.5 in				
Number of sections in member;		N = 1				
Overall breadth of member;		$b_b = N \times b = 5.$	5 in			
Table 4D - Reference design values	s for visually gra	ded timbers (5">	5" and large	r)		
Species, grade and size classificati		- 1		, da Daata and timele	ore	
	on;	Douglas Fir-La	rch, No.1 gra	de, Posts and timb		
Bending parallel to grain;	on;	Douglas Fir-La F _b = 1400 lb/in	rch, No.1 gra 2	de, Posts and timb	513	
Bending parallel to grain; Tension parallel to grain;	on;	Douglas Fir-La $F_b = 1400 \text{ lb/in}$ $F_t = 925 \text{ lb/in}^2$	rch, No.1 gra 2	de, posts and timo		
Bending parallel to grain; Tension parallel to grain; Compression parallel to grain;	on;	Douglas Fir-La $F_b = 1400 \text{ lb/in}$ $F_t = 925 \text{ lb/in}^2$ $F_c = 1500 \text{ lb/in}$	rch, No.1 gra 2	de, Posts and timb		
Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain	on; ;;	$\begin{array}{l} \text{Douglas Fir-La} \\ F_{b} = 1400 \text{ lb/in} \\ F_{t} = 925 \text{ lb/in}^{2} \\ F_{c} = 1500 \text{ lb/in} \\ F_{c_perp} = 405 \text{ lb} \end{array}$	rch, No.1 gra ² /in ²	de, Posts and timb		
Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain;	on; ı;	$\begin{array}{l} \text{Douglas Fir-La} \\ F_{b} = 1400 \text{ lb/in} \\ F_{t} = 925 \text{ lb/in}^{2} \\ F_{c} = 1500 \text{ lb/in} \\ F_{c_perp} = 405 \text{ lb} \\ F_{v} = 150 \text{ lb/in}^{2} \end{array}$	rch, No.1 gra ² /in ²	de, Posts and timb		
Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain; Modulus of elasticity;	on; ı;	Douglas Fir-La $F_b = 1400 \text{ lb/in}$ $F_t = 925 \text{ lb/in}^2$ $F_c = 1500 \text{ lb/in}^2$ $F_{c_perp} = 405 \text{ lb}$ $F_v = 150 \text{ lb/in}^2$ E = 1600000 lb	rch, No.1 gra 2 /in ² //in ²	de, Posts and timb		
Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain; Shear parallel to grain; Modulus of elasticity; Mean shear modulus;	on; ı;	Douglas Fir-La $F_b = 1400 \text{ lb/in}$ $F_t = 925 \text{ lb/in}^2$ $F_c = 1500 \text{ lb/in}^2$ $F_{c_perp} = 405 \text{ lb}$ $F_v = 150 \text{ lb/in}^2$ E = 1600000 lb $G_{def} = E / 16 = 100000 \text{ lb}$	rch, No.1 gra 2 /in ² //in ² 100000 lb/in ²	de, Posts and timb		
Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain; Shear parallel to grain; Modulus of elasticity; Mean shear modulus; Member details	on; ı;	Douglas Fir-La $F_b = 1400 \text{ lb/in}^2$ $F_t = 925 \text{ lb/in}^2$ $F_c = 1500 \text{ lb/in}^2$ $F_{c_perp} = 405 \text{ lb}$ $F_v = 150 \text{ lb/in}^2$ E = 1600000 lk $G_{def} = E / 16 = 100000 \text{ lk}^2$	rch, No.1 gra 2 /in ² //in ² 100000 lb/in ²	de, Posts and timb		
Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain; Modulus of elasticity; Mean shear modulus; Member details Service condition:	on; ;	Douglas Fir-La $F_b = 1400 \text{ lb/in}$ $F_t = 925 \text{ lb/in}^2$ $F_c = 1500 \text{ lb/in}^2$ $F_{c_perp} = 405 \text{ lb}$ $F_v = 150 \text{ lb/in}^2$ E = 1600000 lk $G_{def} = E / 16 =$ Dry	rch, No.1 gra 2 /in ² //in ² 100000 lb/in ²	de, Posts and timb		
Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain; Modulus of elasticity; Mean shear modulus; Member details Service condition; Length of bearing;	on; ı;	Douglas Fir-La $F_b = 1400 \text{ lb/in}^2$ $F_c = 925 \text{ lb/in}^2$ $F_c = 1500 \text{ lb/in}^2$ $F_{c_perp} = 405 \text{ lb}^2$ $F_v = 150 \text{ lb/in}^2$ $E = 1600000 \text{ lk}^2$ $G_{def} = E / 16 = 0$ Dry $L_b = 4 \text{ in}^2$	rch, No.1 gra 2 /in ² //in ² 100000 lb/in ²	de, Posts and timb		
Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain; Modulus of elasticity; Mean shear modulus; Member details Service condition; Length of bearing; Load duration;	on; ı;	Douglas Fir-La $F_b = 1400 \text{ lb/in}^2$ $F_t = 925 \text{ lb/in}^2$ $F_c = 1500 \text{ lb/in}^2$ $F_{c_perp} = 405 \text{ lb}$ $F_v = 150 \text{ lb/in}^2$ E = 1600000 lk $G_{def} = E / 16 =$ Dry $L_b = 4 \text{ in}$ Ten years	rch, No.1 gra 2 /in ² //in ² 100000 lb/in ²	de, Posts and timb		

and alling means of	Project				Job Ref.			
	Section	n				Sheet no./rev.		
ASES					71	of 81		
AS Engineering Service, PC.	Calc. by	Date	Chk'd by	Date	App'd by	Date		
(C) 631-560-0259	AS	5/15/2012						
Section properties								
Cross sectional area of member;		$A = N \times b \times d =$	30.25 in ²					
Section modulus;		$S_x = N \times b \times d^2$	/ 6 = 27.73 in ³					
		$S_y = d \times (N \times b)$	$r^{2}/6 = 27.73 \text{ in}^{3}$					
Second moment of area;		$I_x = N \times b \times d^3 / 12 = 76.26 in^4$						
		$I_v = d \times (N \times b)^3$	³ / 12 = 76.26 in ⁴					
Adjustment festers		, , ,						
Augustiment factors		$C_{-} = 1.00$						
Tomporatura factor Table 2.3.2,		$C_{\rm D} = 1.00$						
Size factor for bonding Table 4D:		$C_{t} = 1.00$						
Size factor for tonsion Table 4D,		$C_{Fb} = 1.00$						
Size factor for compression - Table 4D,		$C_{Ft} = 1.00$						
Size factor for compression - Table 2	iD,	$C_{Fc} = 1.00$						
Flat use factor - Table 4D,	tu Tabla 4.2.9	$C_{fu} = 1.00$						
Incising factor for honding, shoer, to	ly - Table 4.3.0	$C_{\rm iE} = 1.00$						
incising factor for bending, shear, ter	ISION & COMPLE	$C_{1} = 1.00$	0.0					
Inciding factor for perpendicular com	proposion Tabl	$C_{i} = 1.00$						
incising factor for perpendicular com		-100						
Popotitivo mombor factor ol 4.2.0:		$C_{ic_perp} = 1.00$						
Repetitive member factor - eq 3 10-2:	$C_r = 1.00$ $C_r = (1 + 0.37)$	5 in) / 1 = 1.09						
Depth to broadth ratio:	$d_{\rm b} = (L_{\rm b} + 0.57)$	$(100)^{-1} = 1.03$						
Beem is fully restrained	Unom / (IN × Dnom) = 1.00						
- Dealli is fully restrained		$C_{1} = 1.00$						
Deam stability factor - 0.3.3.3,		CL = 1.00						
Bearing perpendicular to grain - cl.3.	10.2			2				
Design compression perpendicular t	$F_{c_{perp}} = F_{c_{perp}}$	$\times C_t \times C_i \times C_b =$	443 lb/in ²					
Applied compression stress perpendicular to grain;		; $f_{c_perp} = R_{A_max} / (N \times b \times L_b) = 74 \text{ lb/in}^2$						
		f _{c_perp} / F _{c_perp} ' =	• 0.166					
	PASS - Des	ign compressiv	e stress excee	ds applied com	pressive stre	ss at bearing		
Strength in bending - cl.3.3.1								
Design bending stress;		$F_{b}' = F_{b} \times C_{D} \times$	$C_t \times C_L \times C_{Fb} \times C_{Fb}$	$C_i \times C_r = 1400 \text{ lb/}$	in ²			
Actual bending stress; $f_b = M / S_x = 1050 \text{ lb/in}^2$								
		$f_{b} / F_{b}' = 0.750$						
		PASS	6 - Design bend	ling stress exce	eds actual be	nding stress		
Strength in shear parallel to grain - c	1341							
Design shear stress:		$F_{v}' = F_{v} \times C_{D} \times$	C _t × C _i = 150 lb/i	n ²				
Actual shear stress $_{-}$ og 3 $_{-}$?		$f = 3 \times E / (2 \times E)$	$(A) = 80 \text{ lb/in}^2$					
		$f_v = 5 \times 1 / (2 \times 1)^2$						
		1 ₀ / 1 ₀ = 0.333	PASS - Design	shaar strass av	roods actual	shaar strass		
			, AGG Design			511641 311633		
Deflection - cl.3.5.1				2				
Modulus of elasticity for deflection;		$E' = E \times C_{ME} \times C_{ME}$	$C_t \times C_{iE} = 16000$	00 lb/in ⁻				
Design deflection;		$\delta_{adm}=0.003\times$	L _{s1} = 0.216 in					
Bending deflection;		$\delta_{b_{s1}} = 0.129$ in						

and a state of the second second	Project				Job Ref.	
A DIFINE	Section				Sheet no./rev.	
ASES AS. Engineering Garview, PC.			1	T	72	of 81
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
Shear deflection:		δ – 0 012 in				
Total deflection:		$\delta_{v} = \delta_{v} \circ 1 + \delta_{v}$	4 = 0 140 in			
		$\delta_2 / \delta_{2dm} = 0.65$	0			
			PASS - De	sian deflection is	s less than to	tal deflection
				0		
an and the second second	Project		Job Ref.			
--	----------------	-------------------	----------------------------	------	----------	------
ASES	Section		Sheet no./rev. 73 of 81			
112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259	Calc. by AS	Date 5/15/2012	Chk'd by	Date	App'd by	Date

PORCH HEADER

STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.04



WHATTERSON MENTER OFFICE	Project				Job Ref.		
adatt atata	Section				Sheet no /rev		
ASES	Section				74 of 81		
AS Engraving Service, PC.	Calc. by	Date	Chk'd by	Date	App'd by	Date	
112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259	AS	5/15/2012					
<u>,</u> ,							
				$Live \times 1.00$	1		
		Support B		Dead imes 1.0	0		
				$Live \times 1.00$	1		
Analysis results							
Maximum moment;		M _{max} = 4539 lb	_ft;	$M_{min} = 0 \ Ib_{j}$	_ft		
Design moment;		M = max(abs(N	/I _{max}),abs(M _{mi}	n)) = 4539 lb_ft			
Maximum shear;		F _{max} = 1252 lb;		F _{min} = -125	2 lb		
Design shear;		F = max(abs(F	_{max}),abs(F _{min})) = 1252 lb			
Total load on member;		$W_{tot} = 2504 \text{ lb}$					
Reaction at support A;		R _{A_max} = 1252	lb;	R _{A_min} = 12	52 lb		
Unfactored dead load reaction at su	pport A;	R _{A_Dead} = 382 I	b				
Unfactored live load reaction at supp	port A;	R _{A_Live} = 870 lb)				
Reaction at support B;		R _{B_max} = 1252	lb;	R _{B_min} = 12	52 lb		
Unfactored dead load reaction at su	pport B;	R _{B_Dead} = 382 I	b				
Unfactored live load reaction at supp	port B;	R _{B_Live} = 870 lb)				
↓	↓	 !'→					
Sawn lumber section details							
Nominal breadth of sections;		$b_{nom} = 6$ in					
Dressed breadth of sections;		b = 5.5 in					
Nominal depth of sections;		$d_{nom} = 10$ in					
Dressed depth of sections;		d = 9.5 in					
Number of sections in member;		N = 1					
Overall breadth of member;		$b_b = N \times b = 5.$	5 in				
Table 4D - Reference design values	for visually gra	ded timbers (5")	< 5" and large	r)			
Species, grade and size classification	n;	Douglas Fir-La	rch, Dense N	o.2 grade, Beams	and stringers		
Bending parallel to grain;		F _b = 1000 lb/in	2				
Tension parallel to grain;		$F_t = 500 \text{ lb/in}^2$					
Compression parallel to grain;		$F_c = 700 \text{ lb/in}^2$	<i>r</i> 2				
Compression perpendicular to grain	,	$F_{c_{perp}} = 730 \text{ lb}$	/IN				
Modulus of elasticity:		$F_v = 170$ lb/lfl	/in ²				
Mean shear modulus:		$\Box = \mathbf{I} + 0$	87500 lb/in ²				
wember details		Dry					
Length of bearing:		uy l⊾=4.in					
Longer of boaring,							

	ADAPTICH MEASE OFF.	Project				Job Ref.	
	ANULUNA	Section				Sheet no./rev.	
	ASES					75	of 81
	112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
	(C) 631-560-0259	AS	5/15/2012				
	Lood duration:						
			Tell years				
	Section properties			2			
	Cross sectional area of member;		$A = N \times b \times d =$	• 52.25 in ²			
	Section modulus;		$S_x = N \times b \times d^2$	/6 = 82.73 in ^o			
			$S_y = d \times (N \times b)$) ² / 6 = 47.90 in ³			
	Second moment of area;		$I_x = N \times b \times d^3 / d^3$	′ 12 = 392.96 in [∙]	4		
			$I_y = d \times (N \times b)^{\gamma}$	° / 12 = 131.71 ir	1 ⁴		
	Adjustment factors						
	Load duration factor - Table 2.3.2;		C _D = 1.00				
	Temperature factor - Table 2.3.3;		$C_t = 1.00$				
	Size factor for bending - Table 4D;		$C_{\text{Fb}} = \textbf{1.00}$				
	Size factor for tension - Table 4D;		C _{Ft} = 1.00				
	Size factor for compression - Table 4	4D;	C _{Fc} = 1.00				
	Flat use factor - Table 4D;		C _{fu} = 1.00				
	Incising factor for modulus of elastic	ty - Table 4.3.8	; C _{iE} = 1.00				
	Incising factor for bending, shear, te	nsion & compre	ssion - Table 4.	3.8			
		·	$C_i = 1.00$				
	incising factor for perpendicular com	pression - Table	e 4.3.8				
	Popotitivo mombor factor ol 4.2.0;		$C_{ic_{perp}} = 1.00$				
	Bearing area factor - eg 3 10-2		$C_r = 1.00$	$5 in) / L_1 = 1.09$			
	Depth-to-breadth ratio:		$d_{\rm b} = (L_{\rm b} + 0.5)$	(-1.67)			
	- Beam is fully restrained) = 1.01			
	Beam stability factor - cl.3.3.3:		C ₁ = 1.00				
		10.2	01				
	Bearing perpendicular to grain - ci.s.				709 lb/in ²		
	Applied compression stress perpendicular t	o grain,	$\Gamma_{c_{perp}} = \Gamma_{c_{perp}}$	$(X \cup U_t \times U_i \times U_b) =$	790 ID/III		
	Applied compression stress perpend	licular to grain,	$I_{c_perp} = R_{A_max}$	$(\mathbf{N} \times \mathbf{D} \times \mathbf{L}_{\mathbf{b}}) = \mathbf{J}$	0 7 10/10		
		PASS - Des	ion compression	- 0.07 1 Va strass avcaa	ds annlied com	nrassiva stra	ss at hoaring
		1 A00 - Des	igh complessi			pressive sue.	ss at bearing
	Strength in bending - cl.3.3.1					• 2	
	Design bending stress;		$F_b = F_b \times C_D $	$C_t \times C_L \times C_{Fb} \times C_{Fb}$	$C_i \times C_r = 1000 \text{ ID/}$	IN	
	Actual bending stress,		$I_b = IVI / S_x = 03$				
			1b / Fb = 0.050	S - Design beng	ling stross avea	ode actual be	ndina stross
			r Au	s - Design bend	ing suess exce	eus actual be	inuning suless
	Strength in shear parallel to grain - c	1.3.4.1			. 2		
	Design shear stress;		$F_v = F_v \times C_D \times$	$C_{\rm t} \times C_{\rm i} = 170 \text{lb/l}$	in ⁻		
	Actual shear stress - eq.3.4-2;		$f_v = 3 \times F / (2 \times f_v)$	A) = 36 lb/in ⁻			
			$T_v / F_v = 0.211$	BASS Design		ando ontual	abaar atraaa
1				rass - Design	snear stress ex	ceeus actual	Shear Stress
	Deflection - cl.3.5.1				2		
	Modulus of elasticity for deflection;		$E' = E \times C_{ME} \times$	$C_t \times C_{iE} = 14000$	00 lb/in [∠]		
	Design deflection;		$\delta_{adm} = 0.004 \times$	L _{s1} = 0.696 in			

and the second second	Project				Job Ref.	
ASES	Section				Sheet no./rev. 76	of 81
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by Date Chk'd by			Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
Bending deflection:		$\delta_{\rm b, s1} = 0.312$ in	1			
Shear deflection;		$\delta_{v s1} = 0.014$ in	I			
Total deflection;		$\delta_a = \delta_{b_s1} + \delta_{v_s}$	a1 = 0.327 in			
		δ_a / δ_{adm} = 0.46	9			
			PASS - D	esign deflectio	on is less than to	tal deflectior



and taxing win ya anan	Project				Job Ref.		
a deal and a deal	Opatian				Sheet no./rev.		
ASES	Section						
A.S. Engineering Services P.C.	Calc by	Data Chkid by		Date	App'd by	Date	
112 Wilson Drive, Port Jefferson, NY 11777		5/15/2012	Clike by	Date	лрр а ру	Dale	
(0) 031-300-0239	70	3/13/2012					
				Live × 1.00			
		Support B		Dead imes 1.00			
				$Live \times 1.00$			
Analysis results							
Maximum moment:		M = 2427 lb	ft·	$M_{min} = 0 \mathbf{b} $	't		
Design moment:		M = max(abs(M = max))	_n, /may) abs(Mmin))	= 2427 lb ft	ι.		
Maximum shear:	ximum shear:			$F_{min} = -1618$	lb		
Design shear:			max).abs(Fmin)) =	= 1618 lb			
Total load on member:	I load on member;						
Reaction at support A:		$R_{A} = 1618$	lb:	R ₄ min = 161	8 lb		
Unfactored dead load reaction at support A;		R _{A Dead} = 943	b				
Unfactored live load reaction at support A;		RA Live = 675 lb					
Reaction at support B;		R _{B max} = 1618	lb:	R _B min = 161	8 lb		
Unfactored dead load reaction at support B;		R _{B Dead} = 943	b				
Unfactored live load reaction at support B;							
	 ⊲ —4"						
Sown lumber costion details							
Nominal broadth of soctions:		h – G in					
Dressed breadth of sections:		$D_{nom} = 0 III$					
Nominal depth of sections:		D = J.J III					
Dressed depth of sections:	Nominal depth of sections;						
Number of sections in member:		d _{nom} = 6 in d = 5-5 in					
		d _{nom} = 6 in d = 5.5 in N = 1					
Overall breadth of member:		$d_{nom} = 6$ in d = 5.5 in N = 1 $b_h = N \times b = 5$	5 in				
Overall breadth of member;		$d_{nom} = 6 \text{ in}$ $d = 5.5 \text{ in}$ $N = 1$ $b_b = N \times b = 5.$	5 in				
Overall breadth of member; Table 4D - Reference design values	s for visually gra	$d_{nom} = 6$ in d = 5.5 in N = 1 $b_b = N \times b = 5.$ added timbers (5")	5 in (5" and larger)	Pooto and timbo	r0		
Overall breadth of member; Table 4D - Reference design values Species, grade and size classification Bending parallel to grain:	s for visually gra on;	$d_{nom} = 6$ in d = 5.5 in N = 1 $b_b = N \times b = 5.$ aded timbers (5") Douglas Fir-La $F_{v} = 1400$ lb/in	5 in (5" and larger) rch, No.1 grade	e, Posts and timbe	rs		
Overall breadth of member; Table 4D - Reference design values Species, grade and size classification Bending parallel to grain; Tension parallel to grain;	s for visually gra on;	$d_{nom} = 6$ in d = 5.5 in N = 1 $b_b = N \times b = 5.$ aded timbers (5") Douglas Fir-La $F_b = 1400$ lb/in $F_c = 925$ lb/in ²	5 in 5" and larger) rch, No.1 grade 2	e, Posts and timbe	rs		
Overall breadth of member; Table 4D - Reference design values Species, grade and size classification Bending parallel to grain; Tension parallel to grain; Compression parallel to grain;	s for visually gra on;	$d_{nom} = 6 \text{ in}$ $d = 5.5 \text{ in}$ $N = 1$ $b_b = N \times b = 5.$ aded timbers (5") Douglas Fir-La $F_b = 1400 \text{ lb/in}$ $F_t = 925 \text{ lb/in}^2$ $F_a = 1500 \text{ lb/in}$	5 in (5" and larger) rch, No.1 grade 2	e, Posts and timbe	rs		
Overall breadth of member; Table 4D - Reference design values Species, grade and size classification Bending parallel to grain; Tension parallel to grain; Compression parallel to grain;	s for visually gra on;	$d_{nom} = 6$ in d = 5.5 in N = 1 $b_b = N \times b = 5.$ aded timbers (5") Douglas Fir-La $F_b = 1400$ lb/in $F_t = 925$ lb/in ² $F_c = 1500$ lb/in $F_c = 0.000$ lb/in	5 in 5" and larger) rch, No.1 grade 2 /in ²	e, Posts and timbe	rs		
Overall breadth of member; Table 4D - Reference design values Species, grade and size classification Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain:	s for visually gra on; n;	$d_{nom} = 6$ in d = 5.5 in N = 1 $b_b = N \times b = 5.$ aded timbers (5") Douglas Fir-La $F_b = 1400$ lb/in $F_t = 925$ lb/in ² $F_c = 1500$ lb/in $F_{c_perp} = 405$ lb $F_v = 150$ lb/in ²	5 in (5" and larger) rch, No.1 grade 2 /in ²	e, Posts and timbe	rs		
Overall breadth of member; Table 4D - Reference design values Species, grade and size classification Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain; Modulus of elasticity:	s for visually gra on; n;	$d_{nom} = 6$ in d = 5.5 in N = 1 $b_b = N \times b = 5.$ aded timbers (5") Douglas Fir-La $F_b = 1400$ lb/in $F_t = 925$ lb/in ² $F_c = 1500$ lb/in ² $F_v = 150$ lb/in ² E = 1600000 lb	5 in (5" and larger) rch, No.1 grade 2 /in ² /in ²	e, Posts and timbe	rs		
Overall breadth of member; Table 4D - Reference design values Species, grade and size classification Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain; Shear parallel to grain; Modulus of elasticity; Mean shear modulus;	s for visually gra on; n;	$d_{nom} = 6$ in d = 5.5 in N = 1 $b_b = N \times b = 5.$ aded timbers (5") Douglas Fir-La $F_b = 1400$ lb/in $F_t = 925$ lb/in ² $F_c = 1500$ lb/in $F_{c_perp} = 405$ lb $F_v = 150$ lb/in ² E = 1600000 lk $G_{def} = E / 16 = 100000$	5 in (5" and larger) rch, No.1 grade 2 /in ² 0/in ² 100000 lb/in ²	e, Posts and timbe	rs		
Overall breadth of member; Table 4D - Reference design values Species, grade and size classification Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain Shear parallel to grain; Modulus of elasticity; Mean shear modulus; Member details	s for visually gra on; n;	$d_{nom} = 6$ in d = 5.5 in N = 1 $b_b = N \times b = 5.$ aded timbers (5") Douglas Fir-La $F_b = 1400$ lb/in $F_t = 925$ lb/in ² $F_c = 1500$ lb/in ² $F_c = 1500$ lb/in ² $F_v = 150$ lb/in ² E = 1600000 lb $G_{def} = E / 16 = 10000$	5 in (5" and larger) rch, No.1 grade 2 /in ²)/in ² 100000 lb/in ²	e, Posts and timbe	rs		
Overall breadth of member; Table 4D - Reference design values Species, grade and size classification Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain; Shear parallel to grain; Modulus of elasticity; Mean shear modulus; Member details Service condition:	s for visually gra on; n;	d _{nom} = 6 in d = 5.5 in N = 1 b _b = N × b = 5. aded timbers (5") Douglas Fir-La F _b = 1400 lb/in F _t = 925 lb/in ² F _c = 1500 lb/in F _{c_perp} = 405 lb F _v = 150 lb/in ² E = 1600000 lk G _{def} = E / 16 =	5 in 5" and larger) rch, No.1 grade 2 /in ² b/in ² 100000 lb/in ²	e, Posts and timbe	rs		
Overall breadth of member; Table 4D - Reference design values Species, grade and size classification Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain; Shear parallel to grain; Modulus of elasticity; Mean shear modulus; Member details Service condition; Length of bearing;	s for visually gra on; n;	d _{nom} = 6 in d = 5.5 in N = 1 b _b = N × b = 5. aded timbers (5") Douglas Fir-La F _b = 1400 lb/in F _t = 925 lb/in ² F _c = 1500 lb/in F _{c_perp} = 405 lb F _v = 150 lb/in ² E = 1600000 lk G _{def} = E / 16 = Dry L _b = 4 in	5 in (5" and larger) rch, No.1 grade 2 /in ² /in ² 100000 lb/in ²	e, Posts and timbe	rs		
Overall breadth of member; Table 4D - Reference design values Species, grade and size classification Bending parallel to grain; Tension parallel to grain; Compression parallel to grain; Compression perpendicular to grain; Shear parallel to grain; Modulus of elasticity; Mean shear modulus; Member details Service condition; Length of bearing; Load duration:	s for visually gra on; n;	dnom = 6 in d = 5.5 in N = 1 bb = N × b = 5. aded timbers (5"> Douglas Fir-La Fb = 1400 lb/in Ft = 925 lb/in ² Fc = 1500 lb/in ² Fc = 1500 lb/in ² E = 1600000 lb Gdef = E / 16 = Dry Lb = 4 in Ten years	5 in (5" and larger) rch, No.1 grade 2 /in ² 100000 lb/in ²	e, Posts and timbe	rs		

oddarinu mesze na	Project				Job Ref.		
	Section				Sheet no./rev.		
ASES					79 of 81		
AS Incharting Survice ALL	Calc. by	Date	Chk'd by	Date	App'd by	Date	
(C) 631-560-0259	AS	5/15/2012					
Section properties							
Cross sectional area of member;		$A = N \times b \times d =$	30.25 in ²				
Section modulus;		$S_x = N \times b \times d^2$	/ 6 = 27.73 in ³				
		$S_y = d \times (N \times b)$	$r^{2}/6 = 27.73 \text{ in}^{3}$				
Second moment of area;		$I_x = N \times b \times d^3$ /	12 = 76.26 in ⁴				
		$I_v = d \times (N \times b)^3$	³ / 12 = 76.26 in ⁴				
Adjustment fasters		,					
Adjustment factors		C 1.00					
Load duration factor - Table 2.3.2,		$C_{\rm D} = 1.00$					
Size fector for hending Table 4D		$C_t = 1.00$					
Size factor for bending - Table 4D;		$C_{Fb} = 1.00$					
Size factor for tension - Table 4D;		$C_{Ft} = 1.00$					
Size factor for compression - Table 4	iD;	$C_{Fc} = 1.00$					
Flat use factor - Table 4D;	. .	$C_{fu} = 1.00$					
Incising factor for modulus of elastici	ty - Table 4.3.8	; C _{iE} = 1.00					
Incising factor for bending, shear, ter	nsion & compre	ssion - Table 4.	3.8				
		$C_i = 1.00$					
Incising factor for perpendicular com	pression - Table	e 4.3.8					
		C _{ic_perp} = 1.00					
Repetitive member factor - cl.4.3.9;		$C_r = 1.00$					
Bearing area factor - eq.3.10-2;		$C_{\rm b} = (L_{\rm b} + 0.37)$	$5 \text{ in}) / L_b = 1.09$				
Depth-to-breadth ratio;		d _{nom} / (N × b _{nom}) = 1.00				
- Beam is fully restrained							
Beam stability factor - cl.3.3.3;		C _L = 1.00					
Bearing perpendicular to grain - cl.3.	10.2						
Design compression perpendicular to	o grain;	$F_{c_perp}' = F_{c_perp}$	$\times C_t \times C_i \times C_b =$	443 lb/in ²			
Applied compression stress perpend	icular to grain;	$f_{c_{perp}} = R_{A_{max}}$	$(N \times b \times L_b) = 7$	4 lb/in ²			
		f _{c_perp} / F _{c_perp} ' =	0.166				
	PASS - Des	ign compressiv	e stress excee	ds applied com	pressive stre	ss at bearing	
Strength in bending - cl.3.3.1							
Design bending stress:		$F_{h}' = F_{h} \times C_{D} \times$	$C_{+} \times C_{+} \times C_{E_{+}} \times C_{-}$	$C_{i} \times C_{r} = 1400 \text{ lb/}$	in ²		
Actual bending stress:		$f_{\rm b} = M / S_{\rm c} = 10$	50 lb/in^2				
Actual boliang brood,		$f_{\rm b} / F_{\rm b}' = 0.750$					
		PASS	S - Desian bend	lina stress exce	eds actual be	ndina stress	
			2 Doolgii Sona			inanig en eee	
Strength in shear parallel to grain - c	1.3.4.1			2			
Design shear stress;		$F_v = F_v \times C_D \times$	$C_t \times C_i = 150$ lb/i	n			
Actual shear stress - eq.3.4-2;		$f_v = 3 \times F / (2 \times$	A) = 80 lb/in ²				
		$f_v / F_v' = 0.535$					
			PASS - Design	shear stress ex	ceeds actual	shear stress	
Deflection - cl.3.5.1							
Modulus of elasticity for deflection;		$E'=E\times C_{ME}\times$	$C_t \times C_{iE} = 16000$	00 lb/in ²			
Design deflection;		$\delta_{adm}=0.003\times$	L _{s1} = 0.216 in				
Bending deflection;		δ _{b s1} = 0.129 in					

and a state of the	Project				Job Ref.	
AUTONIA.	Section				Sheet no./rev.	
ASES AS. Engineering Garvices, PIC.		1	1	T	80	of 81
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
Shear deflection:		δ – 0 012 in				
Total deflection:		$\delta_{v_{2}} = \delta_{v_{2}} + \delta_{v_{2}}$	4 = 0 140 in			
		$\delta_a = 0.65$				
			PASS - Des	sian deflection is	s less than to	tal deflection
				J		

and a state of the	Project				Job Ref.	
Adult stute	Section				Sheet no./rev.	
			1		8	1 of 81
112 Wilson Drive, Port Jefferson, NY 11777	Calc. by	Date	Chk'd by	Date	App'd by	Date
(C) 631-560-0259	AS	5/15/2012				
		FOOTING DE	SIGN			
FOOTING SIMPLY TOOK REATION	ONS FOR BEA	MS AND DIVIDE	D BY ALLOW	ABLE SOIL BE	ARING PRESSU	RE - NO
OVERTURNING IS APPLIED TO	ANY FOOTING	S AND UPLIFT IS	S MINIMAL			