EQUILIBRIUM
ENGINEERS LLC

STRUCTURAL CALCULATIONS

December 14, 2010

Submitted to
Jon R Jurgens & Associates, Inc
15455 NW Greenbrier Parkway
Beaverton, Oregon

Project
Providence Office Park 2
Parking Structure Expansion
Portland, Oregon

EE LLC Job No 10057

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POP 2 PARKING ADDITION
10-199336-CO

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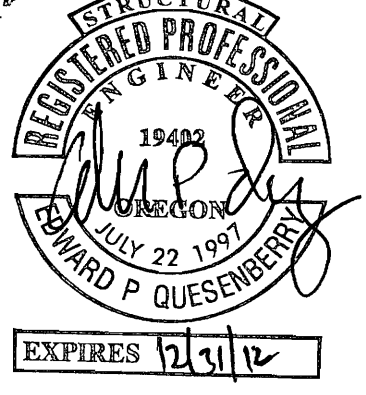

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DEC 14 2010
By

December 14, 2010

Tom Wesel
Jon R Jurgens & Associates
15455 NW Greenbrier Parkway Suite 260
Beaverton, OR 97006

RE Providence Office Park 2
Parking Structure Expansion
EE LLC Job No 10057

Dear Tom

Attached please find calculation sheets 1 through 98 which verify the structural adequacy of the two story expansion to the existing Providence Office Park 2 Parking Structure as shown on drawings PSS1 0 through PSS6 4 dated December 10, 2010 Design is based on the provisions of the 2009 International Building Code with State of Oregon Amendments

Please call if you have any questions

Sincerely,

Ed Quesenberry, S E
Principal

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EQUILIBRIUM ENGINEERS LLC

Providence Health System
Providence Office Park II
Parking Structure Expansion

STRUCTURAL NARRATIVE

General

The existing building is a five story, 212,000 square foot structure with 4 complete levels above grade. The four stories above grade are steel framed structure with buckling restrained braced frames. The one full story below grade is a concrete structure. The exterior cladding system consists of a combination of brick veneer or metal panel over light gage metal stud framing and cable rail.

The proposed expansion adds two complete levels to the existing building. The structural system of the two new levels will be an extension of the existing structural system. Since the structure was originally designed and permitted to receive 5 additional stories, the current proposed expansion does not trigger any structural upgrades of the existing structure or its foundation.

Steel Structure

The framing of the expansion consists of composite steel wide flange and castellated beams supporting a 3 inch composite steel deck with 3½ inch concrete topping.

Lateral System

Lateral wind and seismic forces are resisted by steel buckling restrained braced frames above Tier 1 and load bearing special reinforced concrete shear walls in both orthogonal building directions below Tier 1. The BRB frame sizes

Design Criteria

Code 2009 International Building Code with Oregon 2010 Amendments

BRBF Design AISC Seismic Provisions AISC 341-05

Design Criteria (cont)

Seismic
Occupancy Category II
Seismic Site Class D
Seismic Design Category D
Importance Factor = 1 0
System Coefficient R=7 0 (BRBF)
Ss=0 97g S1= 33g
Sds=0 72g Sd1=0 38g
Cs= 043 in each direction (10 Tier Building)

Wind 80 MPH Exposure B, I=1 0, per 1998 OSSC

Lateral Design Approach

The steel framed portion of the building was analyzed using a 3D model built in RAMFrame Both a linear static and a modal response spectrum analysis were performed The response spectrum analysis was done as prescribed in ASCE7-05 Section 11 4 5 using site parameters provided by the Geotechnical Engineer Forces and displacements from the response spectrum analysis were used in the design of the frame elements and connections

The buckling restrained braces (BRBs) were modeled with user defined sections of various cross sectional areas based on initial force estimates Initial brace forces and frame displacements from the 3D model were shared with Star Seismic, and brace size and stiffness iterations were made until both model and actual brace stiffnesses coincided within 10 percent Both the cross sectional area and modulus of elasticity assigned to the BRBs in the computer model were modified from the base value based on input from Star Seismic The final modeled cross sectional areas (Asc), modulus of elasticity (E'), and brace forces are specified on the drawings so that braces can be designed and built to match the modeled structural stiffness

It should be noted that due to the long building period, the frame design was controlled by drift A reasonable threshold that is approximately 50% of the code allowable limits was established and used throughout the design

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PHS OSC Parking Structure
EE LLC Job No 07063

Top Tier Loads (PSF)

	Deck	Beam	Girder
3 Deck + 3 1/2 Conc Topping	64		
Castellated Beams		3	
Girder			3
Mech/Elect	2		
Fire Sprinklers	3		
Misc	2		
Dead Load	71	74	77
Snow Load	25	25	25
Parking LL(Reducible)	40	40	40

Seismic DL 80 psf plus perimeter cladding weight

Middle Tier Loads (PSF)

	Deck	Beam	Girder
3 Deck + 3 1/2 Conc Topping	64		
Castellated Beams		3	
Girder			3
Mech/Elect	2		
Fire Sprinklers	3		
Misc	2		
Dead Load	71	74	77
Parking LL(Reducible)	40	40	40

Seismic DL 80 psf plus perimeter cladding weight

Lower Tier 1 and Tier 1 Loads (PSF)

	Tees	Girder	Column
3 inch Topping Slab	38		
28' Double Tees	65		
Girder		10	
Column			10
Mech/Elect	2		
Fire Sprinklers	3		
Misc	2		
Dead Load	110	120	130
Parking LL(Reducible)	40	40	40

Seismic DL 132 PSF plus perimeter wall weight

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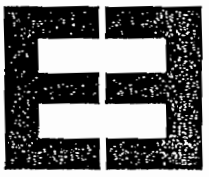

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PHS OSC Parking Structure
EE LLC Job No 07063

Tier 1 Terrace (PSF)

	Tees	Girder	Column
6 inch Average Topping Slab	75		
28 Double Tees	65		
Girder		10	
Column			10
Mech/Elect	2		
Fire Sprinklers	3		
Misc	2		
Dead Load	147	157	167
Parking LL(Reducible)	100	100	100

Add l Weight of Planters = (120 psf x height)

At Paver and Gravel Areas Additional Weight = 45 psf

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Providence Office Park
Parking Structure

Gravity Beam/Girder Framing Design

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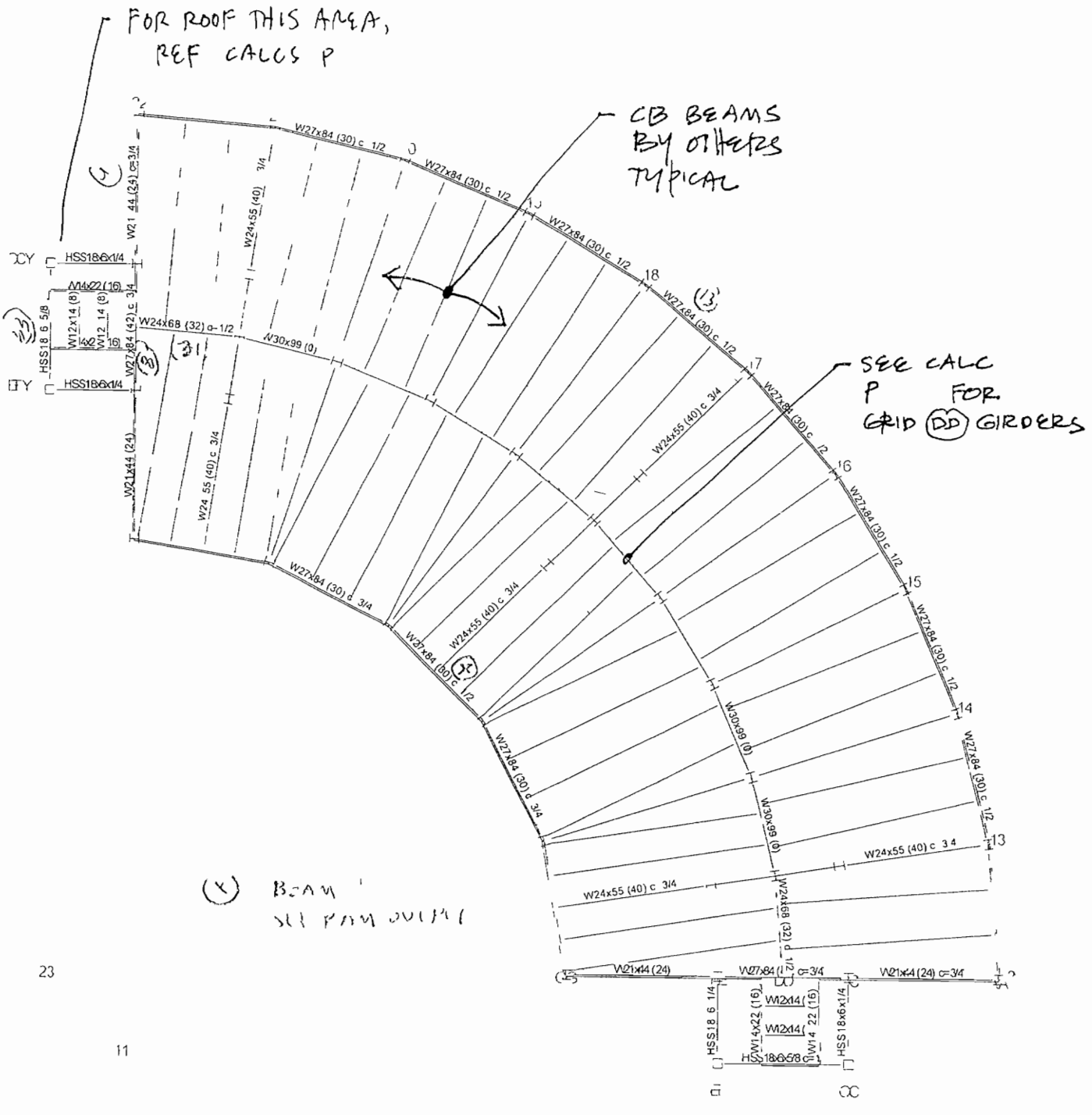
Floor Map

RAM Steel v12 1
Equilibrium Engineers LLC
DataBase 07063parkaltR2
Building Code IBC

10/30/08 12 07 01
Steel Code AISC LRFD

Floor Type Typical



BEAM DESIGN BASED ON UNREDUCED LIVE LOADS

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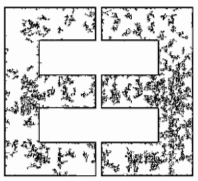

ENGINEERS LLC

December 14, 2010

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Parking Structure Expansion
EE LLC Job No 10057

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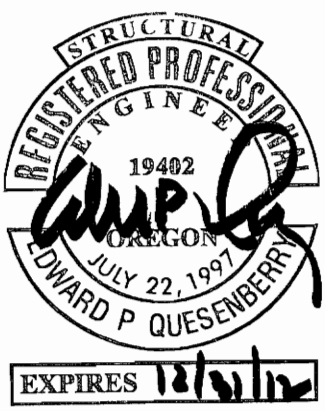

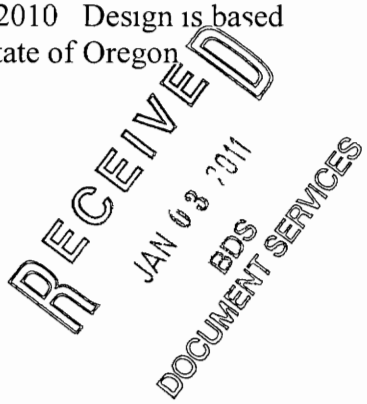

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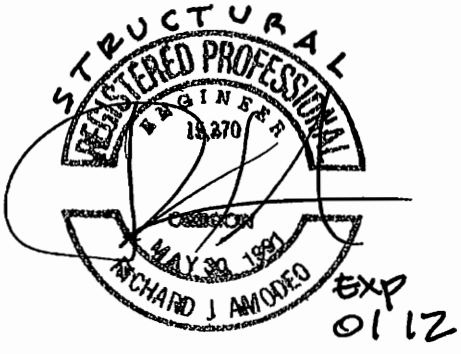
ENGINEERING

STRUCTURAL CALCULATIONS

PROJECT: POP 2 Parking Garage Expansion
Construction Life Safety Plan
PROJECT No.: A10200
DATE: February 25, 2011

PLAN REVIEW #2 RESPONSE



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Comment 3L	15 – 32
Staging	33 – 34

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CHECK SEISMIC ANALYSIS DURING ERECTION STAGE AND IMMEDIATELY POST ERECTION

DL OF FLOORSYSTEM AND FRAMING - 15psf.

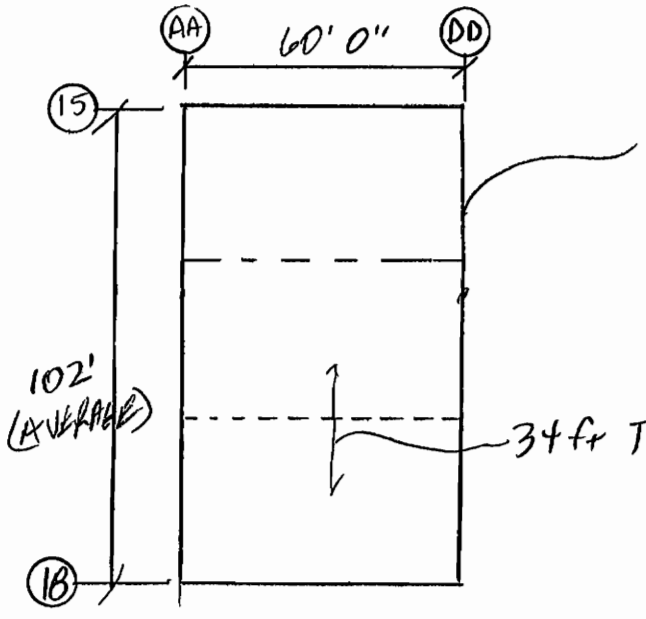
$S_{DS} = 0.72$

$R = 2.5$

$$C_S = \frac{S_{DS}}{R} I = \frac{0.72}{2.5}(1.0) = 0.288$$

LOOK AT UPPER TWO TIERS OF BUILDING INDIVIDUALLY TAKE AN EXAMPLE BUILDING SECTION:

ZONE 2 TIERS 6 & 7 BETWEEN GRIDS 15 & 18 AND AA & DD CHECK AS A RECTANGULAR SECTION AND TWO STORY LOAD DISTRIBUTION:



SEISMIC DL = 15psf (60')(102')
= 91.8 kips
@ TIER 6 & TIER 7

SEE ATTACHED SPREAD-SHEET FOR VERTICAL LOAD DISTRIBUTION.

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POP 2 Parking Garage Expansion
Seismic Vertical Distribution Portion of Erected Steel

Vertical Distribution

Sds	0 72
V	0 288 W
Iseismic	1

		VERTICAL DISTRIBUTION OF SHEAR				DIAPHRAGM SHEAR						
	Total Dead Load (kips)	hx (ft)	wi*hi (ft kips)	Cvx	Fx (kips)	MINIMUM DIAPHRAGM FORCE	MAXIMUM DIAPHRAGM FORCE	Fi dia	Sum of wi	Sum of Fi	CALCULATED FORCE	FINAL DIAPHRAGM FORCE
Upper Tier of Existing	0 0	0	0	0 00	**0 0**	0 0	0 0	0 0	183 6	52.9	0 0	**0 0**
Tier 6	91 8	11	1009 8	0 33	**17 6**	13 2	26 4	17 6	183 6	52.9	26 4	**26.4**
Tier 7	91 8	22	2019 6	0 67	**35.3**	13 2	26 4	35 3	91 8	35 3	35 3	**26 4**
	183 6	kips	3029 4	1 00	52 9							
	52 9	**kips**										

$\frac{35.3 \text{ kips}}{102 \text{ ft}} = 347 \text{ plf}$

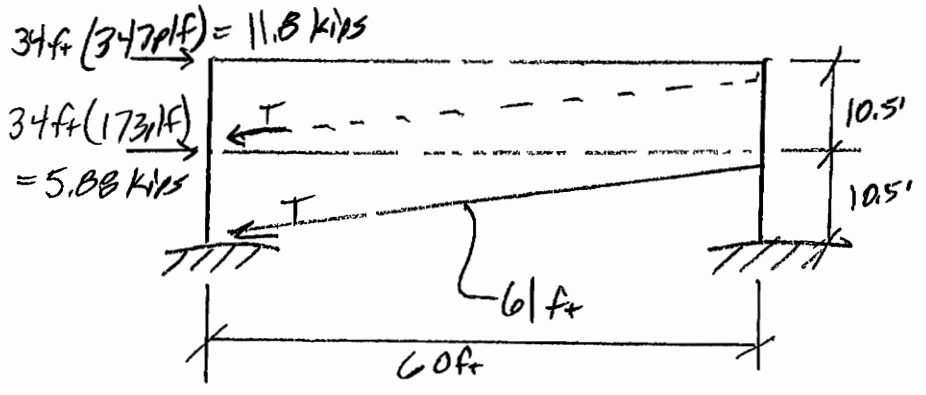
$\frac{26.4 \text{ k}}{102 \text{ ft}} = 259 \text{ plf}$

$\frac{17.6 \text{ k}}{102 \text{ ft}} = 173 \text{ plf}$

2/34

CHECK BRACE DESIGN ON CABLE BRACES

BASE SHEAR SEISMIC DISTRIBUTION



LOWER BRACE DESIGN FORCE = $\frac{11.8\,kips + 5.88\,kips\left(\frac{61}{60}\right)}{1.4}$

= 12.9 kips

SO THE (2) ½" Ø CABLE BRACES WILL BE ADEQUATE FOR SEISMIC LOADS IN THE TYPICAL CASES.

DIAPHRAGM CAPACITY CHECK UNDER SEISMIC LOADS:

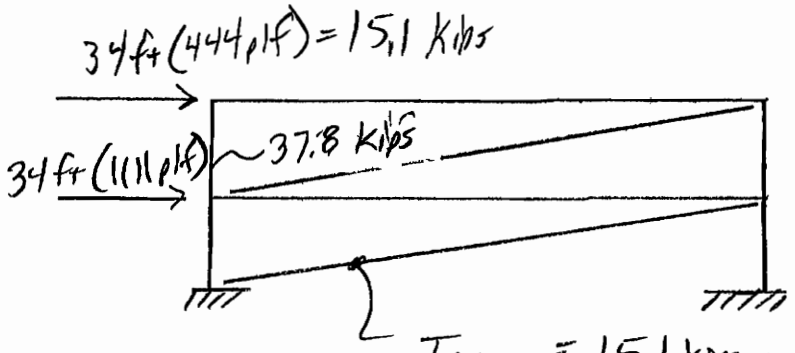
$259\,plf \times \frac{34'}{2} / 60' = 74\,plf$ OK

CONCLUSION: CODE SEISMIC LOADS FOR ERECTED STEEL CONDITION WITHOUT CONCRETE FILL IS ON THE ORDER OF MAGNITUDE AS THE WIND ANALYSIS THAT WAS PERFORMED BY PILLAR CONSULTING GROUP (STEEL ERECTORS ENGINEER)

CHECK CONDITION WHERE 1 LEVEL OF PARKING GARAGE WILL HAVE WET CONCRETE DURING AN EARTHQUAKE)

$S_{DL} = 75psf(60')(102') = 459$ kips

SEE ATTACHED LOAD DISTRIBUTION SPREADSHEETS.



$$T_{BRACE} = \frac{15.1\text{ kips} + 37.8\text{ kips}}{1.4}\left(\frac{61'}{60'}\right)$$

$= 38.5$ kips $\gg$ 16 kips Allowable

BUT THIS IS FOR CODE SEISMIC EVENT WITH A 2500 YEAR RETURN PERIOD

RUN SAME ANALYSIS WITH 0.2 DURATION FACTOR TO ACCOUNT FOR REDUCED PROBABILITY OF CODE LEVEL SEISMIC EVENT IN NEXT SIX MONTHS
REFERENCE ASCE 37-02 SECTION 6.5
SEE ATTACHED.

POP 2 Parking Garage Expansion
Seismic Vertical Distribution Portion of Erected Steel

Vertical Distribution

COOL SEISMIC CHECK:

Sds	0 72
V	0 288 W
Iseismic	1

		VERTICAL DISTRIBUTION OF SHEAR				DIAPHRAGM SHEAR						
	Total Dead Load (kips)	hx (ft)	wi*hi (ft kips)	Cvx	Fx (kips)	MINIMUM DIAPHRAGM FORCE	MAXIMUM DIAPHRAGM FORCE	Fi dia	Sum of wi	Sum of Fi	CALCULATED FORCE	FINAL DIAPHRAGM FORCE
Upper Tier of Existing	0 0	0	0	0 00	**0 0**	0 0	0 0	0 0	550 8	158 6	0 0	**0 0**
Tier 6	459 0	11	5049	0 71	**113 3**	66 1	132 2	113 3	550 8	158 6	132.2	**132.2**
Tier 7	91 8	22	2019 6	0 29	**45 3**	13 2	26 4	45 3	91 8	45 3	45 3	**26 4**
	550 8	kips	7068 6	1 00	158 6							
	158 6	**kips**										

$\frac{113.3 \text{ kips}}{102 \text{ ft}} = 1111 \text{ plf}$

$\frac{132.2 \text{ kips}}{102 \text{ ft}} = 1296 \text{ plf}$

$\frac{45.3 \text{ kips}}{102 \text{ ft}} = 444 \text{ plf}$

$\frac{26.4 \text{ kips}}{102 \text{ ft}} = 259 \text{ plf}$ ← THIS MATCHES ROOF DIAPHRAGM CHECK FROM INITIAL CODE CHECK

POP 2 Parking Garage Expansion
Seismic Vertical Distribution Portion of Erected Steel

Vertical Distribution

Sds	0 18
V	0 072 W
Iseismic	1

THIS IS AT 20% of DESIGN ACCELERATION DUE TO 6 MONTH EARTHQUAKE DURATION.

	VERTICAL DISTRIBUTION OF SHEAR					DIAPHRAGM SHEAR						
	Total Dead Load (kips)	hx (ft)	wi hi (ft kips)	Cvx	Fx (kips)	MINIMUM DIAPHRAGM FORCE	MAXIMUM DIAPHRAGM FORCE	Fi dia	Sum of wi	Sum of Fi	CALCULATED FORCE	FINAL DIAPHRAGM FORCE
Upper Tier of Existing	0 0	0	0	0 00	**0 0**	0 0	0 0	0 0	550 8	39 7	0 0	**0 0**
Tier 6	459 0	11	5049	0 71	**28 3**	16 5	33 0	28 3	550 8	39 7	33 0	**33 0**
Tier 7	91 8	22	2019 6	0 29	**11 3**	3 3	6 6	11 3	91 8	11 3	11 3	**6 6**
	550 8	kips	7068 6	1 00	39 7							
	39 7	kips										

$\frac{28.3 \text{ kips}}{102 \text{ ft}} = 277 \text{ plf}$

$\frac{11.3 \text{ kips}}{102 \text{ ft}} = 111 \text{ plf}$

$\frac{33.0 \text{ kips}}{102 \text{ ft}} = 324 \text{ plf}$

$\frac{6\ 6 \text{ kips}}{102 \text{ ft}} = 65 \text{ plf}$

STANDARD

6 4 2 Thermal, Exposure, and Slope Factors

The thermal factor, C_t, and the exposure factor C_e, shall be for the conditions that will exist during construction If a range of conditions will exist during construction, a series of load calculations shall be made to cover the range of thermal and exposure fac tors to be expected The slope factor, C_s, shall be deter mined based on the construction-phase values of C_t and C_e

6 4 3 Drainage

Where drainage provisions may become blocked during construction (e g by freezing) the extra loads created by such blockages shall be included

6 4 4 Loads in Excess of the Design Value

Surfaces on which snow and ice accumulate shall be monitored and any loads in excess of construction phase design loads shall be removed before construc tion proceeds

6 5 Earthquake

Where required by the governmental authority having jurisdiction, or by the owner, or by the design engineer of record, seismic design of temporary structures and supports shall be performed If so required earthquake loads shall be calculated in accordance with procedures in the 1995 edition of ASCE 7-95 All structures shall be treated as Category II, per Table 1-1 of ASCE 7 95, regardless of the group classification of the completed structure

6 5 1 Applicability

Earthquake loads need not be considered unless the acceleration coefficient, A_a read from Map 9-1 of ASCE 7-95, exceeds 0 15 If A_a exceeds 0 15, earth quake loads need not be considered if the horizontal design loads from wind or other causes (nominal load

COMMENTARY

C6 4 2 Thermal, Exposure, and Slope Factors

The values of the thermal factor, C_t, are determined for the conditions that will exist during construction These conditions may be quite different from those that will exist once the building is occupied Since the value of C_t depends on whether heat is pro vided in a building and most buildings under construc tion are unheated, snow loads may be higher during construction than when the building is completed and occupied

In most circumstances the exposure factor C_e, for a roof during construction will be the same as the ex posure factor for that roof during the life of the build ing When the thermal factor changes the slope factor C_s may also change

C6 4 3 Drainage

Drainage provisions, which often rely on building heat to function properly, may become blocked with ice during construction in cold weather if the building is unheated When this occurs excess loads may accumulate on roofs Ponding instability may result It may be appropriate to install temporary heaters in drains to avoid such problems during construction

C6 4 4 Loads in Excess of the Design Value

If loads in excess of construction-phase design values are encountered during construction work within the building should be halted until the overload is eliminated Snow removal procedures must be planned to avoid overloading the structure with piles of snow or by the use of equipment too heavy for the structure

C6.5 Earthquake

The earthquake provisions of ASCE 7-95 are modeled on FEMA (1994)

C6 5 1 Applicability

The 0 15 threshold for A_a is derived as follows

1 The design acceleration for a 6-month duration (A_a^*) is approximately 20% of the A_a mapped as having a 10% chance of being exceeded in a 50-year period

STANDARD

level) exceed 0 2A_a times the weight to be braced by the temporary bracing system

Construction of detached one and two-family lightly framed dwellings not exceeding two stories in height is exempt from these earthquake requirements

This section applies to all construction except those specifically covered in Section 6 5 3

COMMENTARY

(a mean recurrence interval of about 475 years)

2 The inelastic response modification factor, R, for temporary bracing systems common in construction projects is assumed to be 2 5, consistent with a low level of ductility and redundancy Therefore, the re sponse acceleration for short-period structures (also characteristic of temporary bracing) is

$$Ca = \frac{2\,5\,A_a^*}{R} = 1\,0 \quad A_a^* = 0\,2\,A_a$$

3 The strength required for horizontal construction loads represents a lower bound strength for lateral resistance It is specified herein to be 2% of the weight at allowable stress, which would be approxi mately 3% at strength levels

4 The equivalent threshold acceleration is therefore

$$0\,2\,A_a = 0\,03$$

$$A_a = 0\,15$$

6 5 2 Use of ASCE 7-95

For use of the earthquake load provisions of ASCE 7-95, the following modifications should be made

1 The mapped values for A_a and A_v may be multi- plied by a factor less than one to represent the re duced exposure period but the factor shall not be less than 0 2 Note for regions inside the 0 4 con- tour on the map, the minimum factor should in- crease to a value of 0 4 along major faults
2 The restrictions on types of structural systems in seismic performance categories D and E do not ap ply, as long as the height of the temporary bracing system above the final seismic resisting system does not exceed 100 feet (30 m)
3 The R factor used for temporary bracing systems shall not exceed 2 5 unless the system is detailed in accordance with the provisions of ASCE 7-95
4 Only the requirements dealing with the strength of the seismic resisting structural system need be satis- fied

C6 5 2 Use of ASCE 7-95

The higher factor with the 0 4 contour is caused by the truncation of the probabilistic maps at the 0 4g ac celeration level It is generally accepted that such trun cation results in reasonable design for permanent struc tures however, the logic may not be applicable for the much shorter construction periods pertinent during construction

The drift limitations and the nonstructural provi- sions are not required for temporary structures and for structures during their construction phases

Specific provisions for design of masonry structures to resist seismic loads are contained in ASCE 5-92

6 5 3 Other Standards for Earthquake-Resistant Design

Partially complete structures of a type that are ex- cluded by the earthquake load provisions of ASCE 7- 95 and for which specifically applicable standards for earthquake resistant design exist, such as vehicular bridges, shall be designed and evaluated according to the specifically applicable standard Earthquake loads

C6 5 3 Other Standards for Earthquake-Resistant Design

ASCE 7 95 excludes certain types of structures for various reasons unique characteristics of response to ground shaking exceptionally high risk associated with poor performance, and the existence of other standards for design The provisions of Section 6 5 2 are intended to make ASCE 7 95 usable for most

CHECK CABLE BRACING AND DIAPHRAGM FOR CASE WITH " 20% OF DESIGN ACCELERATION:

$$T_{BRACE} = \frac{(277\,plf + 111\,plf)\,34'}{1.4} \times \left(\frac{61}{60}\right) = 9580\ lbs$$

THIS IS WELL UNDER THE 16 kip DOUBLE BRACE CAPACITY CURRENTLY SPECIFIED.

$$\text{DIAPHRAGM LOAD} = \frac{324\,plf \times 34'/2}{1.4(60')} = 66\,plf$$ EASILY OK

CHECK THIS REDUCED SEISMIC EVENT FOR DIAPHRAGM SPANNING BETWEEN THE PERMANENT BRACES:

$W = 324\,plf$ $L = 136\,ft$

GRID 13 TO GRID 17

$$V = \frac{136\,ft}{2}(324\,plf) = 22.1\ kips$$

$$V_{dia} = \frac{V}{60ft} = \frac{370\,plf}{1.4} = 265\,plf$$

CAPACITY OF DIAPHRAGM AT TIME THAT SLAB IS POURED IS FROM GENERAL NOTES: PSS1.0.

BUTTON PUNCHES AT 36" o.c.

½" Ø PUDDLE WELDS @12" o.c. ~~AT~~ (4 PER SUPPORT)

CAPACITY = 276 plf ∴ SEE ATTACHED

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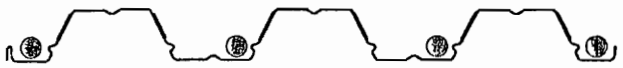
By _____ Date _____

Project No _____

W3 FORMLOK™



4 Weld Pattern at Supports
Button Punch or 1½" (*38 mm*) Top Seam Weld
Without Concrete Fill
Galvanized or Phosphatized/Painted

Allowable Diaphragm Shear Values, q (plf, *kN/m*) and Flexibility Factors, F ((in /lb)x10^6, (*mm/N*)*x10^6*)

GAGE	SIDELAP ATTACH-MENT		SPAN (ft in , *mm*)								
			6 0" *1 830*	7' 0 *2 130*	8' 0 *2 440*	9 0' *2 740*	10 0 *3 050*	11' 0' *3 350*	12' 0' *3 660*	14 0' *4 270*	16 -0 *4 880*
19	BP @ 24'	q	372 *5 43*	336 *4 90*	295 *4 31*	276 *4 03*	249 *3 63*	239 *3 49*	219 *3 20*	198 *2 89*	183 *2 67*
		F	9 2+135R *52 5+771R*	10 2+116R *58 2+662R*	11 5+101R *65 7+577R*	12 4+90R *70 8+514R*	13 8+81R *78 8+463R*	14 6+74R *83 4+423R*	16 1+67R *91 9+383R*	18 2+58R *103 9+331R*	20 3+51R *115 9+291R*
	BP @ 12	q	430 *6 28*	387 *5 65*	355 *5 18*	330 *4 82*	310 *4 52*	295 *4 31*	282 *4 12*	262 *3 82*	248 *3 62*
		F	8 2+135R *46 8+771R*	9 1+116R *52 0+662R*	9 9+101R *56 5+577R*	10 7+90R *61 1+514R*	11 5+81R *65 7+463R*	12 2+74R *69 7+423R*	12 9+67R *73 7+383R*	14 3+58R *81 7+331R*	15 5+51R *88 5+291R*
	TSW @ 24'	q	528 *7 71*	510 *7 44*	444 *6 48*	440 *6 42*	394 *5 75*	395 *5 76*	361 *5 27*	337 *4 92*	320 *4 67*
		F	7 1+135R *40 5+771R*	6 5+116R *37 1+662R*	6 6+101R *37 7+577R*	6 2+90R *35 4+514R*	6 3+81R *36 0+463R*	5 9+74R *33 7+423R*	6 0+67R *34 3+383R*	5 7+58R *32 5+331R*	5 5+51R *31 4+291R*
	TSW @ 12	q	742 *10 83*	694 *10 13*	658 *9 60*	630 *9 19*	608 *8 87*	590 *8 61*	575 *8 39*	551 *8 04*	534 *7 79*
		F	5 6+135R *32 0+771R*	5 3+116R *30 3+662R*	5 1+101R *29 1+577R*	4 9+90R *28 0+514R*	4 8+81R *27 4+463R*	4 6+74R *26 3+423R*	4 5+67R *25 7+383R*	4 3+58R *24 6+331R*	4 1+51R *23 4+291R*
18	BP @ 24"	q	478 *6 98*	429 *6 26*	376 *5 49*	350 *5 11*	316 *4 61*	300 *4 38*	276 *4 03*	248 *3 62*	227 *3 31*
		F	7 8+96R *44 5+548R*	8 7+83R *49 7+474R*	9 8+72R *56 0+411R*	10 7+64R *61 1+365R*	11 9+58R *68 0+331R*	12 7+53R *72 5+303R*	13 9+48R *79 4+274R*	15 9+41R *90 8+234R*	17 9+36R *102 2+206R*
	BP @ 12'	q	542 *7 91*	484 *7 06*	442 *6 45*	409 *5 97*	383 *5 59*	362 *5 28*	345 *5 03*	318 *4 64*	299 *4 36*
		F	7 1+96R *40 5+548R*	7 9+83R *45 1+474R*	8 7+72R *49 7+411R*	9 4+64R *53 7+365R*	10 1+58R *57 7+331R*	10 8+53R *61 7+303R*	11 5+48R *65 7+274R*	12 8+41R *73 1+234R*	14 0+36R *79 9+206R*
	TSW @ 24	q	663 *9 68*	643 *9 38*	566 *8 26*	564 *8 23*	511 *7 46*	515 *7 52*	476 *6 95*	442 *6 45*	417 *6 09*
		F	6 1+96R *34 8+548R*	5 7+83R *32 5+474R*	5 8+72R *33 1+411R*	5 5+64R *31 4+365R*	5 5+58R *31 4+331R*	5 2+53R *29 7+303R*	5 3+48R *30 3+274R*	5 1+41R *29 1+234R*	4 9+36R *28 0+206R*
	TSW @ 12'	q	912 *13 31*	859 *12 54*	822 *12 00*	794 *11 59*	773 *11 28*	756 *11 03*	744 *10 86*	710 *10 36*	685 *10 00*
		F	4 9+96R *28 0+548R*	4 7+83R *26 8+474R*	4 5+72R *25 7+411R*	4 4+64R *25 1+365R*	4 3+58R *24 6+331R*	4 1+53R *23 4+303R*	4 0+48R *22 8+274R*	3 8+41R *21 7+234R*	3 7+36R *21 1+206R*
16	BP @ 24"	q	752 *10 97*	667 *9 73*	585 *8 54*	538 *7 85*	486 *7 09*	457 *6 67*	420 *6 13*	374 *5 46*	340 *4 96*
		F	5 6+49R *32 0+280R*	6 2+42R *35 4+240R*	7 1+37R *40 5+211R*	7 8+32R *44 5+183R*	8 7+29R *49 7+166R*	9 4+27R *53 7+154R*	10 3+24R *58 8+137R*	11 9+21R *68 0+120R*	13 6+18R *77 7+103R*
	BP @ 12'	q	826 *12 05*	732 *10 68*	662 *9 66*	608 *8 87*	565 *8 25*	530 *7 73*	502 *7 33*	458 *6 68*	426 *6 22*
		F	5 2+49R *29 7+280R*	5 8+42R *33 1+240R*	6 4+37R *36 5+211R*	7 0+32R *40 0+183R*	7 7+29R *44 0+166R*	8 3+27R *47 4+154R*	8 9+24R *50 8+137R*	10 0+21R *57 1+120R*	11 2+18R *64 0+103R*
	TSW @ 24'	q	1003 *14 64*	958 *13 98*	844 *12 32*	830 *12 11*	752 *10 97*	752 *10 97*	694 *10 13*	656 *9 57*	629 *9 18*
		F	4 5+49R *25 7+280R*	4 3+42R *24 6+240R*	4 4+37R *25 1+211R*	4 2+32R *24 0+183R*	4 2+29R *24 0+166R*	4 0+27R *22 8+154R*	4 1+24R *23 4+137R*	4 0+21R *22 8+120R*	3 9+18R *22 3+103R*
	TSW @ 12	q	1327 *19 37*	1241 *18 11*	1179 *17 21*	1133 *16 53*	1098 *16 02*	1071 *15 63*	1050 *15 32*	1021 *14 90*	1003 *14 64*
		F	3 8+49R *21 7+280R*	3 6+42R *20 6+240R*	3 5+37R *20 0+211R*	3 4+32R *19 4+183R*	3 3+29R *18 8+166R*	3 3+27R *18 8+154R*	3 2+24R *18 3+137R*	3 1+21R *17 7+120R*	3 0+18R *17 1+103R*

BP = Button Punch TSW = Top Seam Weld

FINAL DETERMINATION: CONCRETE SLAB IS POURED AND STARTS TO HARDEN, AT WHAT STRENGTH DOES DIAPHRAGM GAIN ENOUGH STRENGTH TO MEET CODE.

CODE SEISMIC FORCE ON DIAPHRAGM IS 1296 plf

$V = \frac{136\,ft}{2}(1296\,plf) = 88.2\ kips$

$\div 60\,ft$

$1470\,plf$

$\div 1.4$

$1050\,plf$ ALLOWABLE DESIGN

ALLOWABLE DIAPHRAGM SHEAR FROM UBC = 2260 plf BASED ON 3000 psi STRENGTH.

PER ACI 318-05 → CHAPTER 21.9.7

$\phi V_n = 0.75(2\sqrt{f'_c} + \rho_t f_y) A_{cv}$ — $A_{cv} = 3.5" \times 12" = 42\,in^2$

$\rho_t f_y$ → 0 NEGLECT

$\phi V_n = 1470\,plf$

$\sqrt{f'_c} = \frac{1470\,plf}{0.75(2)(42\,in^2)} = 23.33$

$f'_c = 555\ psi$

$.75(2\sqrt{555})42\,in^2 = 1484\ lbs$

NEGLECTING STEEL

WITH A HIGH EARLY STRENGTH CONCRETE, DESIGN CODE COULD BE REACHED OVER A WEEKEND

ITEM 3d CONFIRM CAPACITY OF EXISTING BEAMS FOR SHORING LOADS → DETAIL 16/SS13

REF 2/PSS2.2 FOR EXISTING FRAMING:

W14x22 w/(16) STUDS
21'-0" SPAN
$\frac{9'+5.5'}{2}$ + 0.5' TRIB = 7.75'

DEAD LOAD 3" 18ga Deck w/ 3½" TOPPING 61 psf
10 psf Additional
71 psf DEAD LOAD

CONSTRUCTION LIVE LOAD - ~~100~~ 25 psf

SHORING LOAD: 76 sf x 25 psf LIVE LOAD
+ 48 sf x 150 psf LIVE LOAD
125 sf x 20 psf DEAD LOAD
11600 lbs

SEE ATTACHED ALUMA SHEET FOR ADDITIONAL INFORMATION →

PLACE THIS LOAD @ 1'-6" FROM BEAM END

BEAM WORKS AS A NON-COMPOSITE SECTION
SEE ATTACHED:
END CONNECTION = 16.21 kips
(3) 7/8" Ø A325 N → REFERENCE 1/SS1.2
C=1.75 $\frac{r_n}{\Omega} = 14.4$ kips
$\frac{R_n}{\Omega} = 25.2$ kips OK

To specify your title block on these five lines, use the SETTINGS main menu selection, choose the Printing & Title Block tab, and ent your title block information

Title
Dsgnr
Description

Job #
Date 8 18AM 8 FEB 11

Scope

Steel Beam Design

Description W14x22 Uncomposite - Plan Review Item 3D

General Information

Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements

Steel Section W14X22

		Pinned Pinned		Fy	50 00ksi
Center Span	21 00 ft	Bm Wt Added to Loads		Load Duration Factor	1 00
Left Cant	0 00 ft	LL & ST Act Together		Elastic Modulus	29 000 0ksi
Right Cant	0 00 ft				
Lu Unbraced Length	0 00 ft				

Point Loads

	# 1	# 2	# 3	# 4	# 5	# 6	# 7	
Dead Load	3 850	3 850	2 500					k
Live Load	1 360	1 360	9 100					k
Short Term								k
Location	7 000	14 000	1 500					ft

Summary

Beam OK
Static Load Case Governs Stress

Using W14X22 section Span = 21 00ft Fy = 50 0ksi
End Fixity = Pinned Pinned Lu = 0 00ft LDF = 1 000

	Actual	Allowable		
Moment	49 108 k ft	79 658 k ft	Max Deflection	0 673 in
fb Bending Stress	20 344 ksi	33 000 ksi	Length/DL Defl	591 0 1
fb / Fb	**0 616 1**		Length/(DL+LL Defl)	374 5 1
Shear	16 213 k	63 204 k		
fv Shear Stres‹	5 130 ksi	20 000 ksi		
fv / Fv	**0 257 1**			

Force & Stress Summary

	Maximum	DL Only	<< These columns are Dead + Live Load placed as noted –>> LL @ Center	LL+ST @ Center	LL @ Cants	LL+ST @ Cants	
Max M +	49 11 k ft	30 52	49 11				k ft
Max M							k ft
Max M @ Left							k ft
Max M @ Right							k ft
Shear @ Left	16 21 k	6 40	16 21				k
Shear @ Right	6 27 k	4 26	6 27				k
Center Defl	0 673 in	0 426	0 673	-0 673	0 000	0 000	in
Left Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Right Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Query Defl @	0 000 ft	0 000	0 000	0 000	0 000	0 000	in
Reaction @ Left	16 21	6 40	16 21	16 21			k
Reaction @ Rt	6 27	4 26	6 27	6 27			k

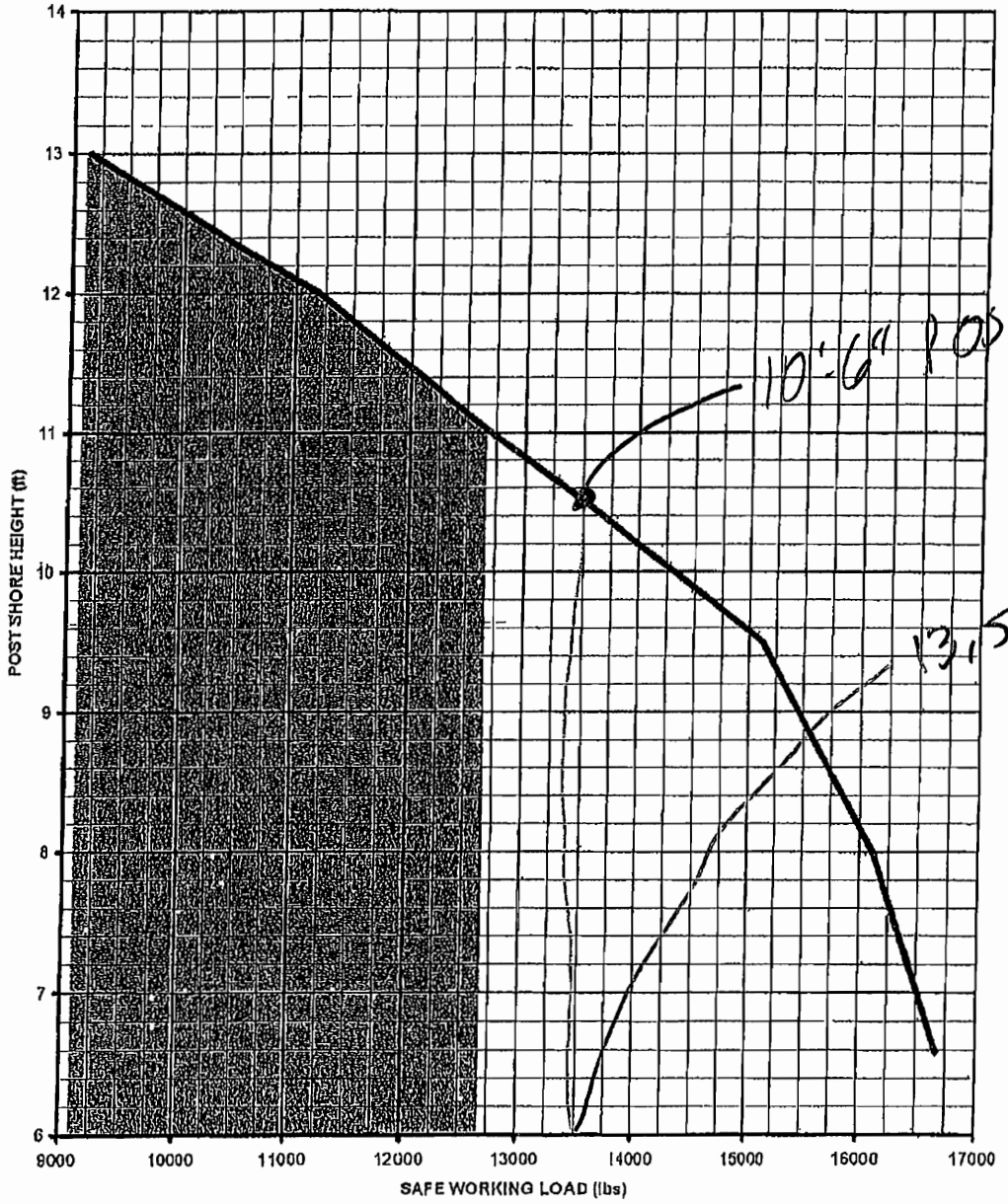
Fa calc'd per Eq E2 1 K*L/r < Cc
I Beam Passes Table B5 1 Fb per Eq F1 1, Fb = 0 66 Fy

Section Properties W14X22

Depth	13 740 in	Weight	22 04 #/ft	r xx	5 537 in
Width	5 000 in	I xx	199 00 in4	r yy	1 039 in
Web Thick	0 230 in	I yy	7 00 in4	Rt	1 250 in
Flange Thickness	0 335 in	S xx	28 967 in3		
Area	6 49 in2	S yy	2 800 in3		

#2 Heavy Duty Post Shore (Re-Shoring)

LOAD CHART FOR #2 HEAVY DUTY POST SHORE
(RE SHORING CONDITION 3.1 SAFETY FACTOR)



Note For total heights greater than 11' 0", the 2' 0" extension must be used
→ 2' 0" extension used

This information is subject to change It is intended to be used by technically skilled designers knowledgeable in the field and is to be used with other data.

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Impact Loads on Existing Structure Summary of Analysis

Possible Falling Object Scenarios

1) Columns
    a Columns will be brought in horizontal and staged near their final installation location Then, the columns will be lifted vertical immediately adjacent to their final installation location, and bolted into place
2) Beams
    a There are short W14x22 beams that weigh 400 lbs maximum that could fall during erection, with a potential of falling on end
    b There are longer Castellated beams that weigh approximately 60 lbs per foot and are 60'-0" long that are largely constrained geometrically from falling on end since the maximum lift height of 25' 0" is shorter than the beam lengths
    c There are somewhat shorter wide flange beams and girders that are still longer than the maximum lift height of 25'-0" The shortest and most critical of these girders are 28'-3" long W30x99's on grid DD Most of the other beams are closer to 40'-0" long and weigh 84 lbs per foot

Impact Loading Scenarios

1) One of the objects above (with a potential for falling on end (meaning columns and small beams) could fall on end at several critical locations
    a At Slabs on Metal Deck with Positive Moment Reinforcement (listed in order of increasing stiffness)
        i Midspan near middle of beam spans
        ii Quarter points near middle of beam spans
        iii Midspan near support of beam spans
        iv Quarter Points near support of beam spans
    b At Existing Typical castellated floor beams (listed in order of increasing stiffness)
        i Midspan
        ii $1/10^{th}$ of the span near the supports
2) One of the objects above could fall flat at several critical locations
    a At slabs on metal deck
        i Midspan of metal deck
        ii Quarter points of metal deck
    b Parallel to one steel beam
    c Perpendicular to several steel beams
        i Midspan of beam
        ii $1/10^{th}$ of the span near the supports
3) One of the long span castellated steel beams could fall and impact the slab like a hammer, imparting a large point load to the structural steel members
    a At steel beams at midspan and $1/10^{th}$ span

Design Analysis Tools for Checking the failure capacity of these slabs

1) Blodgett "Potential Energy of Falling Body on Member"
2) Basic Physics and Momentum and Impulse equations
3) Strain Energy equations

DETERMINE STIFFNESS OF BEAMS & SLAB ON METAL DECK

BEAM AT MIDSPAN: $K = \frac{P}{\Delta} = \frac{48EI}{L^3}$

BEAM AT 1/10th SPAN: $K = \frac{P}{\Delta} = \frac{3EIL}{a^2 b^2} = \frac{3EIL}{(0.1L)^2(0.9L)^2}$

$$= \frac{3EI}{0.0081 L^3}$$

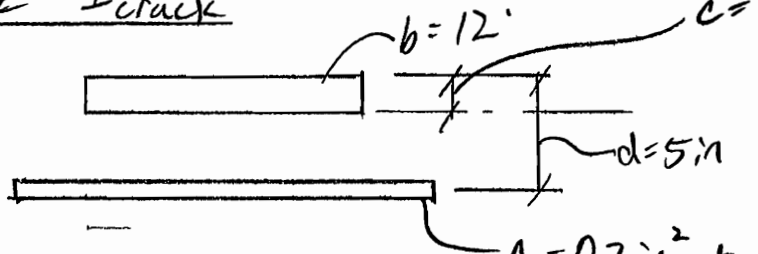
$$K = \frac{370EI}{L^3}$$

BEAM AT 1/10th THE SPAN IS 7.7 TIMES AS STIFF AS LOADS AT MIDSPAN

SLAB AT MIDSPAN $\Rightarrow \frac{48EI}{L^3} = K$

SLAB AT 1/4 PT $\Rightarrow K = \frac{3EI}{0.035 L^3} = \frac{85EI}{L^3}$

FOR SLABS, USE I_{crack}



$A_s = 0.2\,in^2 + 15.71\,in(0.0451\,in)$

$A_s = 0.9085\,in^2/ft$

$\frac{bc^2}{2} + nA_s c - nA_s d = 0 \Rightarrow c = 2.04\,in$

$n = \frac{27 \times 10^6}{57000\sqrt{3000}} = 9.29$

$I_{cr} = \frac{bc^3}{3} + nA_s(d-c)^2 = 107.9\,in^4$

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POP II Parking GARAGE

By DEL Date

Project No A10200

Sheet 17 of

USE ~~I_{crack}~~ ASSUMING BOND BREAKS BETWEEN DECK AND METAL DECK:

THIS WAS NOT USED

$C = 1.10$ in

$$I_{cr} = \frac{12(1.10)^3}{3} + 9.29(0.2)(5-1.10)^2 = 33.6 \text{ in}^4$$

CASTELLATED BEAM COMPOSITE STIFFNESS

FROM RAM STEEL → A STEEL CASTELLATED BEAM HAS AN I_{Eff} of 5480 in^4! JUST A NON COMPOSITE BEAM AS AN I of 2630 in^4

RUN SPREADSHEET ANALYSIS TO DETERMINE RANGE OF FORCES THAT MAY DEVELOP ON SYSTEM

FROM BLODGETT → $F = W_6 + \sqrt{W_6^2 + 2 k W_6 H}$

LOCATION STIFFNESSES:

BEAM MIDSPAN $K = \frac{48(29\times10^6)(5480)}{(60'\times12")^3} = 20400$ lbs/in

COMPOSITE

BEAM MIDSPAN → NON COMPOSITE $K = \frac{48(29\times10^6)(2630)}{(60)^3(12)^3} = 9810$ lbs/in

BEAM 1/10 SPAN COMPOSITE → $K = \frac{370(29\times10^6)(5480)}{(60)^3(12)^3} = 15754$

NON COMPOSITE → $K = 75607$ lbs/in

SLAB MIDSPAN & BM MIDSPAN.

$$K = \frac{48(57000\sqrt{3000})(107.9)}{(11.5'\times12)^3} = 6152 \text{ lbs/in}$$

$$\text{COMBINED STIFFNESS} = \frac{1000 \text{ lbs}}{\frac{1000 \text{ lbs}}{6152 \text{ lbs/in}} + \frac{300 \text{ lbs}}{20400 \text{ lbs/in}}}$$

$$= 5346 \text{ lbs/in}$$

SLAB 1/4 PT AT BM MIDSPAN: $K = \frac{85(3122\times10^3 \text{ psi})(107.9)}{(11.5\times12)^3}$

$K = 10900$ lbs/in

$$\text{COMBINED STIFFNESS} = \frac{1000}{\frac{1000}{10900} + \frac{750}{20400}} = 7781 \text{ lbs/in}$$

SLAB MIDSPAN AT BM 10th PT

$$\text{COMBINED STIFFNESS} = \frac{1000\ \text{lbs}}{\frac{1000\ \text{lbs}}{6152\ \text{lbs/in}} + \frac{500\ \text{lbs}}{75607\ \text{lbs/in}}}$$

$$= 5911\ \text{lbs/in}$$

SLAB 1/4 PT AT BM 10th Pt

$$\text{COMBINED STIFFNESS} = \frac{1000\ \text{lbs}}{\frac{1000\ \text{lbs}}{10900} + \frac{750}{75607}} = 9836\ \text{lbs/in}$$

BEAM STIFFNESS FOR UNIFORMLY DISTRIBUTED LOAD

$$K = \frac{WL}{\Delta} = \frac{384\,EI}{5L^3} = \frac{77\,EI}{L^3} = \frac{77(29\times10^6)(5480)}{((60')\times12''/ft)^3}$$

$$K = 32785\ \text{lbs/in}$$

$$K = 546.4\ \text{lbs/ft/in}$$

Impact Load Calculations
Varying Conditions

Condition Description	Object	Wb (lbs)	h (ft)	Impact Location	Location Stiffness K (lbs/in)	F (kips)	Notes	Applied Potential Energy (ft-kips)
Falling on End	Column	4370	5	Beam Midspan	20400	107.9	Steel Beam Check	21850
Falling on End	Column	4370	5	Beam 1/10th	75607	203.5	Steel Beam Check	21850
Falling on End	Column	4370	5	Slab Midspan at bm Midspan	5346	57.5	Slab Check	21850
Falling on End	Column	4370	5	Slab quarter point at bm midspan	7781	68 4	Slab Check	21850
Falling on End	Column	4370	5	Slab Midspan at bm 1/10th span	5911	60 2	Slab Check	21850
Falling on End	Column	4370	5	Slab Quarter point at bm 1/10th span	9836	76.3	Slab Check	21850
Falling on End	Small beam	400	25	Beam Midspan	20400	70 4	Steel Beam Check	10000
Falling on End	Small beam	400	25	Beam 1/10th	75607	135 1	Steel Beam Check	10000
Falling on End	Small beam	400	25	Slab Midspan at bm Midspan	5346	36 2	Slab Check	10000
Falling on End	Small beam	400	25	Slab quarter point at bm midspan	7781	43 6	Slab Check	10000
Falling on End	Small beam	400	25	Slab Midspan at bm 1/10th span	5911	38 1	Slab Check	10000
Falling on End	Small beam	400	25	Slab Quarter point at bm 1/10th span	9836	49 0	Slab Check	10000
Falling Flat	Column	2280	5	Beam Midspan	20400	77 0	Steel Beam Check	11400
Falling Flat	Small Beam	400	25	Beam Midspan	20400	70 4	Steel Beam Check	10000
Falling Flat perp to several beams	Large Beam	720	25	Beam Midspan	20400	94 6	Steel Beam Check	18000
Falling Flat perp to several beams	Large Beam	720	25	Beam 1/10th	75607	181 4	Steel Beam Check	18000
Falling Flat on Slab (load per ft)	Large Beam	60	25	Slab Midspan	6152	14.9	Slab Check on Per foot basis	1500
Falling Flat (load per foot)	Large Beam	60	25	Slab 1/4 point	10900	19 9	Slab Check on Per foot basis	1500
Falling Flat	Large Beam	3600	25	Main Beam	32785	269 7	Distribute Uniformly	90000
Hammer Strike	Column/Lg Beam	2185	25	Beam Midspan	20400	165 7	Steel Beam Check	54625
Hammer Strike	Column/Lg Beam	2185	25	Beam 1/10th	75607	317 0	Steel Beam Check	54625

FORCES USED FOR DESIGN CHECKS

THESE FORCES ARE BASEDON STIFFNESSES OF MEMBERS STRESSED IN THE LINEAR ELASTIC RANGE. ONCE THE MEMBERS YIELD THEIR STIFFNESSES WILL DROP AND THESE FORCES WILL DECREASE.

ANALYZE STEEL BEAMS & CONNECTIONS

USE NO PHI FACTORS. MODEL IN RAM STEEL TO ACCOUNT FOR CASTELLATED BEAMS, ETC. DECREASE LOAD BY $\frac{1.6}{0.9}$ TO ACCOUNT FOR BUILT IN LOAD & Ø FACTORS.

PT LOADS AT MIDSPAN

① 107.9 kips ÷ 1.78
② 70.4 kips
③ 77 kips
④ 94.6 kips
CONTROLS → ⑤ 165.7 kips ÷ 1.78 = 93.2 kips ENTER THIS INTO RAM

PT LOADS AT BM 1/10TH POINT

① 203.5 kips ÷ 1.78 = 115 kips
② 135.1 kips
③ 181.4 kips
CONTROLS → ④ 317 kips ÷ 1.78 = 178.1 kips ENTER THESE INTO RAM

FROM RAM STEEL → APPLIED MOMENT FOR POINT LOAD AT MIDSPAN

$M_u = 2731$ k-ft $\quad \emptyset M_n = 1027$ k-ft Approx

LARGE AMOUNT OF OVERSTRESS

PERFORM SLAB CHECKS

COLUMN REACTION → 76.3 kips

PUNCHING CHECK FROM PREVIOUS ANALYSIS

$$M_u = \frac{76.3(0.125(12))(0.75(12))}{12'} = 171.7 \text{ k.ft}$$

CONSIDER 2'x 2'-0" AREA

$$V_n = 4\sqrt{f'_c}\, b_o\, d$$

$$= 4\sqrt{3000}\,(24 \times 4)(4.25 \text{ Average } d)$$

$$V_n = 89.4 \text{ kips}$$

PUNCH THROUGH IS NOT A PROBLEM.

CHECK FLEXURE IN SLAB.

M_n $\quad A_s = 0.91 \text{ in}^2/\text{ft}$

$$a = \frac{(0.2(60) + 0.71(38))}{0.85(3)(12)} = 1.274 \text{ in}$$

$$\phi M_n = \left[0.2(60) + 0.71(38)\right] \times \left[5.5" - \frac{1.274}{2}\right] \frac{1}{12}$$

$$= 15.8 \text{ k.ft/ft}$$

NEED 11'-0" OF SLAB TO RESIST THIS

$$\rho = 0.00985 < 0.75\rho_b = 0.0160$$

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CHECK UNIFORMLY DISTRIBUTED LOAD

$14.9 \text{ kips/ft} \times 0.5 = 7.5 \text{ kips/ft}$ SHEAR

$M_u = 14.7 \text{ kips/ft} \times \frac{12'}{4} = 44.7 \text{ k-ft/ft}$

THIS AGAIN WOULD YIELD SLAB

SLAB $V_u = 2\sqrt{3000}\,(12)(5.5'') = 7230 \text{ kips/ft}$

SHEAR IS RIGHT AT LIMIT

CHECK UNIFORMLY DISTRIBUTED LOAD ON BEAM

$M_u = \left(\frac{269.7 \text{ kips}}{60'}\right)\frac{(60')^2}{8} = 2023 \text{ ft-kips}$

$+ \ 800 \text{ lbs/ft}\ (60')^2/8 = 360 \text{ ft-kips}$

STILL A LARGE FLEXURAL OVERSTRESS

CHECK SHEAR REACTION AT COLUMN

(8) 3/8 A325N BOLTS → DIRECT SHEAR

CAPACITY = 230.4 kips ULTIMATES

− 25.1 kips DEAD LOAD

205.3 kips RESERVE

- THIS IS ADEQUATE FOR ALL CASES SAVE THE HAMMER STRIKE CONDITION

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ENERGY METHODS

EXTERNAL ENERGY

POTENTIAL ENERGY = $N\Delta$ = 4.0K × 20ft = 80ft-kips

ASSUME A W30×90 FOR THIS CHECK

$Z_x = 283\ in^3$

$M_n = 50ksi\,(283\ in^3) = 14150\ in\text{-}lbs = 1180\ ft\text{-}kips$

$M = \frac{PL}{4} \Longrightarrow P = \frac{4(1180ft\text{-}kips)}{60'} = 78.667\ k\text{-}ft$

$\Delta = \frac{PL^3}{48EI} = \frac{78.7 \times (60)^3 (12)^3}{48(29\times10^3)(3610)} = 5.85\ in$

STRAIN ENERGY THAT CAN BE ABSORBED

$U = \frac{\sigma_y^2\, I\, L}{6\, E\, c^2} = \frac{\sigma_y^2}{2E}\left(\frac{I\,L}{3\,c^2}\right)$ FROM BLODGETT

$U = \frac{(50ksi)^2(3610\ in^4)(60'\times12)}{6\,(29\times10^3)ksi\,(15in)^2} = 1666\ kip\text{-}in$

$= 13.9\ kip\text{-}ft$

↑ MEMBER WILL YIELD.

WHAT ABOUT WHEN MEMBER GETS LOADED INTO THE PLASTIC RANGE

∴ REPLACE $\frac{\sigma_y^2}{2E}$ WITH $U_u = \frac{\sigma_y + \sigma_u}{2}\varepsilon_u$

$\sigma_y = 50\ ksi \quad \sigma_u = 65\ ksi \quad \varepsilon_u = 0.09$

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$$u_u = \left(\frac{50+65}{2}\right)(0.09) = 5.175 \text{ ksi}$$

← ASTM A992 HAS REPORTED ELONGATION OF 10%

$$U_u = u_u \frac{IL}{3c^2} = 5.175 \text{ ksi} \left(\frac{(3610 \text{ in}^4)(60\times12)}{3(15 \text{ in})^2}\right. = 19930 \text{ in-kips}$$

$U_u = 1660.6$ ft-kips
VERSUS 80 ft-kips
MAXIMUM
APPLIED

THIS CHECK PROVES THAT THE BEAMS WILL SURELY YIELD, BUT THE YIELDING OF STEEL HAS ENOUGH DUCTILITY TO NOT RUPTURE AT THE APPLIED LOADS.

UNIFORMLY DISTRIBUTED LOAD:

$$U = \frac{4\sigma_y^2 IL}{15Ec^2} = \frac{\sigma^2}{2E}\left(\frac{4IL}{7.5c^2}\right)$$

$$u_u = u_u\left(\frac{4IL}{7.5c^2}\right) = 5.175 \text{ ksi}\left(\frac{4(3610)60\times12}{7.5(15")^2}\right)$$

$U_u = 31684$ in-kips
$= 2657$ ft-kips

APPLYING IMPACT LOADS TO ROOF WILL DEFINITELY YIELD THE STRUCTURE BUT THE ULTIMATE RUPTURE ENERGY OF THE STEEL MEMBER IS 100 TIMES GREATER THAN YIELD ENERGY.

CHECK ULTIMATE RUPTURE CAPACITY OF CASTELLATED BEAM AT 1/10TH SPAN CONDITION

APPLIED POTENTIAL ENERGY TO YIELD MEMBER

$$U_p = \frac{1}{2} F \Delta$$

$$\Delta = \frac{P a^2 b^2}{3EIL} = \frac{P(0.9L)^2(0.1L)^2}{3EIL} = \frac{0.0081 PL^3}{3EIL}$$

$$\Delta = \frac{PL^3}{370.4EI} \qquad U_p = \frac{P^2 L^3}{740 EI}$$

FOR W30x90 EXAMPLE BEAM

$$M_{CAP} = \frac{245 in^3 (50 ksi)}{12} = 1021 \text{ ft-kips}$$

$$M_{max} = \frac{Pab}{L} \Rightarrow P_{yield} = \frac{M_{max} L}{a b} = \frac{M_{max} L}{0.09 L^2}$$

END CONNECTION HAS TO BE GOOD FOR AT LEAST THIS MINOR LOAD

$$P_{yield} = \frac{1021 \text{ ft-kips}(60')}{0.09(60)^2} = 189 \text{ kips}$$

$$U_p = \frac{P^2 L^3}{740 EI} = \frac{(189 kips)^2 (60')^3 (12)^3}{740 (29000)(3610)} = 172.1 \text{ in-kips}$$

$$U_p = 14.34 \text{ ft-kips}$$

AVAILABLE RUPTURE ENERGY = 1725 ft-kips

ENERGY TO YIELD $\times \frac{U_u}{U_y} = 14.34 kf \times \frac{51.75 ksi}{0.043 ksi}$

BEAMS WILL NOT RUPTURE

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CHECK BOLT CAPACITIES AT ULTIMATE LOAD: NO Ø FACTOR

TYPICAL CB 30 HAS (8) 7/8" Ø A325-N BOLTS

CAPACITY IN DIRECT SHEAR = $\frac{8(21.6 \text{ kips})}{0.75}$

= 230.4 kips

DEAD LOAD = 25.1 kips UNFACTORED

205.3 kips RESERVE CAPACITY

FORCE TO YIELD BEAM FROM PREVIOUS PAGE IS 189 kips

CHECK CONNECTION CAPACITY FULLY.

ASSUME 3/8" SHEAR TAB ⇝ MATCHED WEB OF W21x50 USED TO BUILD THE BEAM

MIN WELD FROM GEN NOTE:
1/4" FILLET WELD EA SIDE
- 18 x 2 x 4 (1.392) = 200 kips OK

$A_{nv} = 18(0.375) - \left(6(0.875" \times 1 1/16") \times 0.375"\right)$

= 4.5"

PLATE RUPTURE STRENGTH

$R_n = 0.6 F_u A_{nv} = 0.6(58 ksi)(4.5 in^2)$

R_n = 156.6 kips

PLATE YIELD STRENGTH:

$R_n = 0.6 F_y A_y = 0.6(36 ksi)(6.75 in^2)$

= 145.8 kips

OK WITH ADDED YIELDING FROM SLIP BOLTS AND SHEAR STUDS

OK w/ F_u = 72.5 ksi

OK w/ 7/16" PL'S

WILL YIELD BEFORE RUPTURING

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POP II Parking GARAGE

By DEL Date

Project No A10200

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AVAILABLE ENERGY DISIPATION OF SLAB ON METAL DECK.

SINCE SLAB IS UNDER REINFORCED, THE REBAR WILL YIELD BEFORE CONCRETE WILL CRUSH, SO ALL ENERGY DISIPATION WILL OCCUR ON ONE SIDE OF NEUTRAL AXIS, PERFORM PREVIOUS ANALYSIS, BUT, ENERGY DISIPATION WILL BE 1/2 OF ENERGY DISIPATION BASED ON A STEEL SECTION EQUIVALENT

ENERGY TO YIELD: P+ LOAD AT MIDSPAN:

$$U = K \frac{\sigma_y^2 I L}{E c^2} \qquad K = \frac{1}{6}$$

$$U_y = \frac{\sigma_y^2}{2E}$$

$$U = u_y \frac{I L}{3 c^2}$$

$$u_y = \frac{(36 ksi)^2}{2(29 \times 10^3) ksi}$$

$$= 24.9 psi$$

for SLAB ON METAL DECK
$F_y = 36 ksi$
$F_u = 45 ksi$
ELONGATION = 42-48% MAX

A36 HAS A 20% ELONGATION
A992 HAS AN 18% ELONGATION
A615 REBAR HAS AN ELONGATION OF 20%

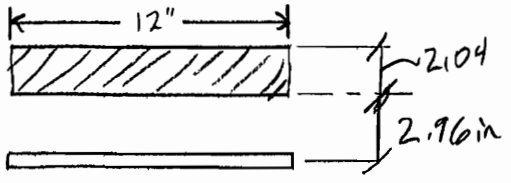
ASSUME ULTIMATE STRAIN AT ULTIMATE STRESS IS 50% OF ELONGATION USE 9% ULTIMATE STRAIN FOR ALL CALCULATIONS (CONSERVATIVE)

ENERGY TO YIELD SLAB ON METAL DECK

CONCRETE CRACKED SECTION

VERSUS

EQUIVALENT STEEL SECTION



$I_{cr} = 1079\ in^4$

$I_{ST,EQ.} = \frac{I_{cr}}{n} = 11.61\ in^4$

$c = 2.96\ in$

ENERGY TO YIELD: $L = 12'-0"$

$$U = \frac{1}{2} u_y \left[\frac{IL}{3c^2}\right] = \frac{1}{2}(24.9psi)\left[\frac{11.61in^4\,(12'\times 12''/ft)}{3\,(2.96in)^2}\right]$$

STEEL ON ONE SIDE ONLY

$= 7919\ in\text{-}lbs$

$= 66\ ft\text{-}lbs/ft$

FOR $L = 9'-0"$ $U = 50\ ft\text{-}lbs/ft$

ENERGY TO RUPTURE

$u_y = 24.9\ psi$

$$u_u = \frac{\sigma_y + \sigma_u}{2}\varepsilon_u = \left(\frac{38+45}{2}\right)0.09 = 3735\ psi$$

150 TIMES THE ENERGY ABSORBING CAPACITY

SO $U_u = 150(50) = 7500\ ft\text{-}lbs/ft$

NEED 3'-0" OF SLAB WIDTH TO SUPPORT COLUMN FALLING 5'-0" MAXIMUM HEIGHT.

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WHAT IS CORRESPONDING SHEAR IN SLAB ON METAL DECK AT YIELD. → DOES SLAB HAVE ADEQUATE SHEAR CAPACITY TO ENSURE THAT SLAB YIELDS TO DISIPATE THE ENERGY.

SLAB YIELDING OCCURS AT $2 \times 50 \text{ ft-lbs/ft} = 100 \text{ ft-lbs/ft}$
↑ CONSERVATIVE

EQUIVALENT POINT LOAD

$$U = \frac{1}{2} P \Delta$$

$$\Delta = \frac{PL^3}{48EI}$$

$$U = \frac{P^2 L^3}{96EI} =$$

$$P_y = \sqrt{\frac{96EI\,U}{L^3}} = \sqrt{\frac{96(29 \times 10^6 \text{ ksi})(11.61 \text{ in}^4)(100 \times 12)}{(9' \times 12'')^3}}$$

$P_y = 5550$ lbs FORCE TO YIELD CONCRETE SLAB

SHEAR STRENGTH OF CONCRETE SLAB

$$V_c = 2\sqrt{f'c}\, b_w d = 2\sqrt{3000}(12)(5)$$
$$= 6572 \text{ lbs}$$

SO HAVE ENOUGH SHEAR CAPACITY TO ENSURE RUPTURE DOES NOT OCCUR
→ DUCTILE FAILURE.

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SLAB 1/4 SPAN: 11'-0" SPAN CONDITION:

FORCE TO YIELD:

$$\Delta = \frac{P a^2 b^2}{3EIL} = \frac{P(0.25L)^2(0.75L)^2}{3EIL} = \frac{0.035 P L^3}{3EI}$$

$$\Delta = \frac{PL^3}{86EI} =$$

$M_{cap} = 15.8$ k-ft

$$P_{equivalent} = \frac{M_{cap} L}{a\ b} = \frac{15.8 \text{ k-ft} \cdot (11' \cancel{\text{-0"}})}{2.75 \text{ ft} (8.25')}$$

$= 7.66$ kips

$V_u = 0.75(7.66 \text{ kips}) = 5.75$ kips MAX SHEAR

SLAB HAS ENOUGH BEAM SHEAR CAPACITY TO DEVELOP YIELD STRENGTH OF MEMBER

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REVIEW OF STAGING LOADS

* CLEARLY INDICATE PARAMETERS ON DWG CLS1

* AVERAGE ℄ TO ℄ DECK SPAN @ OUTER HALF OF EXISTING BUILDING = $\frac{37'-8" + 28'-3"}{2 \times 3}$

$= 11'-0"$

* 3½" ON 3"-18 GA @ 11'-0" ⇒ (3) SPAN CAP = 276 PSF

TYPICAL CASTELLATED BEAM WT = 60 PLF @ 60'
MAXIMUM COLUMN WT. = 200 PLF @ 21'
MAXIMUM MISC. BEAM WT. = 40 PLF (L VARIES)

ERECTION STAGING GUIDELINES:

A. ALL STAGED STRUCTURAL STEEL MEMBERS SHALL BE ORIENTED IN THE E/W DIRECTION, AND SHALL CROSS A MINIMUM OF (2) EXISTING BEAMS

B. MAXIMUM LOAD SUPPORTED BY (1) EXISTING BEAM = 13.2^k @ MIDSPAN OF BEAM. THIS LIMITS THE MAXIMUM # OF STAGED COLUMNS (THE HEAVIEST MEMBER) TO 6 PER BAY.

C. MAXIMUM UNIFORM LOAD SUPPORTED BY EXISTING BEAMS = 440 PLF, THEREFORE IT IS ADEQUATE TO PLACE STAGED BEAMS @ 2'-0" O.C. OVER EXISTING BEAMS

over

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POP II PARKING
PORTLAND
OR

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NOTE: DECK CAPACITY IS GREAT ENOUGH THAT ALL MEMBERS CAN BE SPREAD OUT IN ANY DIRECTION w/o OVERSTRESSING; EXCEPT THAT A BUNDLE OF STEEL SHALL ALWAYS BE PLACED ⊥ TO THE EXISTING BEAMS.

* MINIMUM CASTELLATED $LL_{CAP} = 11' \times 40 PSF = 440 PLF$

$(M_{LL})_{CAP} = .44(60^2/8) = 198'k$

* MAXIMUM LOAD OF (1) COLUMN ON (1) EXIST BEAM WHEN COLUMN SPANS (2) BEAMS

$LOAD = .5 \times 21' \times 200 PLF = 2100\#$

* MAXIMUM POINT LOAD @ MIDSPAN OF CASTELLATED BEAM = $198'k \times 4/60 = 13.2k$

→ A BUNDLE OF (6) COLUMNS WEIGH $6 \times 2100\# = 12.6k$

$12.6k \leq 13.2k$ · OK, ALTHOUGH CONTRACTOR EXPECTS 4 MAX

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Sheet 34 of 34

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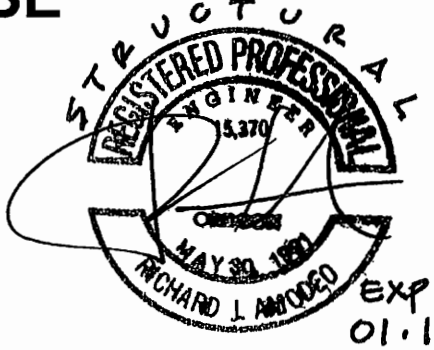
STRUCTURAL CALCULATIONS

PROJECT: POP 2 Parking Garage Expansion
Construction Life Safety Plan
PROJECT No.: A10200
DATE: February 8, 2011

PLAN REVIEW RESPONSE



Contents

Calculations	1 – 10

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ITEM 3d CONFIRM CAPACITY OF EXISTING BEAMS FOR SHORING LOADS → DETAIL 1/SS1.3

REF: 2/PSS2.2 FOR EXISTING FRAMING:

W14x22 w/(6) STUDS
21'-0" SPAN
$\frac{9'+5.5'}{2}$ + 0.5' TRIB = 7.75'

DEAD LOAD 3" 18ga Deck w/ 3½" TOPPING 61 psf
10 psf Additional
71 psf DEAD LOAD.

CONSTRUCTION LIVE LOAD - ~~100~~ 25 psf

SHORING LOAD: 76 sf x 25psf LIVE LOAD
+ 48 sf x 150psf LIVE LOAD
125 sf x 20 psf DEAD LOAD
11600 lbs

PLACE THIS LOAD @ 1'-6" FROM BEAM END.

BEAM WORKS AS A NON-COMPOSITE SECTION
SEE ATTACHED:

END CONNECTION = 16.21 kips

(3) 7/8"Ø A325 N → REFERENCE 1/SS1.2
C=1.75 $\frac{r_n}{\Omega}$ = 14.4 kips
$\frac{R_n}{\Omega}$ = 25.2 kips OK

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POP II Parking GARAGE EXPANSION By DEL Date 02/08/11
STRUCTURAL PLAN REVIEW Project No A10200
ITEM 3d Sheet 1 of

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Title
Dsgnr
Description

Job #
Date 8 18AM 8 FEB 11

Scope

Steel Beam Design

Description W14x22 Uncomposite - Plan Review Item 3D

General Information

Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements

Steel Section W14X22

		Pinned Pinned		Fy	50 00 ksi
Center Span	21 00 ft	Bm Wt Added to Loads		Load Duration Factor	1 00
Left Cant	0 00 ft	LL & ST Act Together		Elastic Modulus	29 000 0 ksi
Right Cant	0 00 ft				
Lu Unbraced Length	0 00 ft				

Point Loads

	#1	#2	#3	#4	#5	#6	#7	
Dead Load	3 850	3 850	2 500					k
Live Load	1 360	1 360	9 100					k
Short Term								k
Location	7 000	14 000	1 500					ft

Summary

Beam OK
Static Load Case Governs Stress

Using W14X22 section Span = 21 00ft Fy = 50 0ksi
End Fixity = Pinned Pinned Lu = 0 00ft LDF = 1 000

	Actual	Allowable
Moment	49 108 k ft	79 658 k ft
fb Bending Stress	20 344 ksi	33 000 ksi
fb / Fb	**0 616 1**	
Shear	16 213 k	63 204 k
fv Shear Stress	5 130 ksi	20 000 ksi
fv / Fv	**0 257 1**	

Max Deflection	0 673 in
Length/DL Defl	591 0 1
Length/(DL+LL Defl)	374 5 1

Force & Stress Summary

	Maximum	DL Only	LL @ Center	LL+ST @ Center	LL @ Cants	LL+ST @ Cants	
			<< These columns are Dead + Live Load placed as noted -->>				
Max M +	49 11 k ft	30 52	49 11				k ft
Max M -							k ft
Max M @ Left							k ft
Max M @ Right							k ft
Shear @ Left	16 21 k	6 40	16 21				k
Shear @ Right	6 27 k	4 26	6 27				k
Center Defl	-0 673 in	0 426	0 673	-0 673	0 000	0 000	in
Left Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Right Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Query Defl @	0 000 ft	0 000	0 000	0 000	0 000	0 000	in
Reaction @ Left	16 21	6 40	16 21	16 21			k
Reaction @ Rt	6 27	4 26	6 27	6 27			k

Fa calc'd per Eq E2-1 K*L/r < Cc
I Beam Passes Table B5 1, Fb per Eq F1 1 Fb = 0 66 Fy

Section Properties W14X22

Depth	13 740 in	Weight	22 04 #/ft	r xx	5 537 in
Width	5 000 in	I xx	199 00 in4	r yy	1 039 in
Web Thick	0 230 in	I yy	7 00 in4	Rt	1 250 in
Flange Thickness	0 335 in	S xx	28 967 in3		
Area	6 49 in2	S yy	2 800 in3		

PROTECTIVE COVERS OVER EXISTING ELEVATORS:

NORTH ELEVATOR WILL BE OPEN TO PUBLIC DURING INITIAL STEEL ERECTION, SO REVIEW EXISTING CAPACITY OF EXISTING ROOF: REFERENCE: 3/PSS2.5 FROM ORIGINAL DRAWINGS:

HAVE W12x14 BEAMS SPANNING 21'-0" WITH 6'-0" OF TRIBUTARY

ROOF DECK 3" 18ga N DECK SPANNING 12'-0"
CAPACITY AT $\Delta = \frac{L}{240} = 0.45"$ IS 46 psf

NEITHER OF THESE COMPONENTS HAVE ENOUGH CAPACITY TO SUPPORT CONSTRUCTION LIVE LOAD. SOUTH WALL WILL BE REMOVED SO MAY NEED TO ADD PROTECTIVE COVER INSIDE OF SHAFT.

SPAN FROM EXISTING W14x22 AND HSS18x6x1/4
W14x22 BEAM HAS ENOUGH CAPACITY AS A NON COMPOSITE SECTION
HSS18x6x1/4 HAS ENOUGH CAPACITY AS WELL.

RECOMMEND INSTALLING PLANKS ACROSS OPEN SHAFT AT TIER 5 ELEVATION. ASSUME THAT ELEVATOR CAN BE MADE TO WORK STOPPING AT TIER 4,
→ DOES NOT WORK. PUT PLANKING ON EXISTING ROOF

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Title
Dsgnr
Description

Job #
Date 9 01AM 8 FEB 11

Scope

Steel Beam Design

Description W14x22 Beam with Construction Live Load

General Information

Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements

Steel Section W14X22

		Pinned Pinned		Fy	50 00 ksi
Center Span	20 50 ft	Bm Wt Added to Loads		Load Duration Factor	1 00
Left Cant	0 00 ft	LL & ST Act Together		Elastic Modulus	29,000 0 ksi
Right Cant	0 00 ft				
Lu Unbraced Length	0 00 ft				

Distributed Loads

	# 1	# 2	# 3	# 4	# 5	# 6	# 7	
DL	0 045							k/ft
LL	0 675							k/ft
ST								k/ft
Start Location								ft
End Location								ft

Point Loads

	# 1	# 2	# 3	# 4	# 5	# 6	# 7	
Dead Load	3 600	3 600						k
Live Load	1 300	1 300						k
Short Term								k
Location	7 000	14 000						ft

Summary

Beam OK
Static Load Case Governs Stress

Using W14X22 section Span = 20 50ft Fy = 50 0ksi
End Fixity = Pinned Pinned Lu = 0 00ft LDF = 1 000

	Actual	Allowable		
Moment	72 065 k ft	79 658 k ft	Max Deflection	0 956 in
fb Bending Stress	29 855 ksi	33 000 ksi	Length/DL Defl	659 7 1
fb / Fb	**0 905 1**		Length/(DL+LL Defl)	257 4 1
Shear	12 625 k	63 204 k		
fv Shear Stress	3 995 ksi	20 000 ksi		
fv / Fv	**0 200 1**			

Force & Stress Summary

			<<-- These columns are Dead + Live Load placed as noted -->				
	Maximum	DL Only	LL @ Center	LL+ST @ Center	LL @ Cants	LL+ST @ Cants	
Max M +	72 07 k ft	27 88	72 07				k ft
Max M							k-ft
Max M @ Left							k-ft
Max M @ Right							k ft
Shear @ Left	12 39 k	4 20	12 39				k
Shear @ Right	12 63 k	4 38	12 63				k
Center Defl	0 956 in	0 373	-0 956	0 956	0 000	0 000	in
Left Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Right Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Query Defl @	0 000 ft	0 000	0 000	0 000	0 000	0 000	in
Reaction @ Left	12 39	4 20	12 39	12 39			k
Reaction @ Rt	12 63	4 38	12 63	12 63			k

Fa calc'd per Eq E2 1 K*L/r < Cc
I Beam Passes Table B5 1, Fb per Eq F1 1 Fb = 0 66 Fy

Description W14x22 Beam with Construction Live Load

Section Properties **W14X22**

Depth	13 740 in	Weight	22 04 #/ft	r xx	5 537 in
Width	5 000in	I xx	199 00 in4	r yy	1 039 in
Web Thick	0 230 in	I yy	7 00 in4	Rt	1 250 in
Flange Thickness	0 335 in	S xx	28 967 in3		
Area	6 49 in2	S yy	2 800 in3		

PROTECTIVE COVER OVER ELEVATOR:

TRY 4x6 PLANKING OVER ELEVATOR ROOF SPANNING FROM W12x14 TO W12x14.

L = 12.5' USE DOUBLE LAYER OF 4x6 DF-L#2

EXISTING W12x14 BEAMS ARE OVERSTRESSED 25%.

PUT W8x10 BEAM OVER TOP OF ROOF AND SHARE LOAD BETWEEN BEAMS.

I of W12x14 =	88.6	0.74
I of W8x10 =	30.8	0.26
	119.4	

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POP II PARKING STRUCTURE EXPANSION - STRUCTURAL PLAN REVIEW - ITEM 11.

By DBL Date 01/08/11

Project No A10260

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Title
Dsgnr
Description

Scope

Job #
Date 10 20AM 8 FEB 11

General Timber Beam

Description Construction Planking - Double Layer of 4x6 DF-I #2

General Information

Calculations are designed to 1997 NDS and 1997 UBC Requirements

Section Name	4x6	Center Span	12 50 ft	Lu	0 00 ft
Beam Width	5 500 in	Left Cantilever	ft	Lu	0 00 ft
Beam Depth	3 500 in	Right Cantilever	ft	Lu	0 00 ft
Member Type	Sawn	Douglas Fir Larch No 2			
Bm Wt Added to Loads		Fb Base Allow	875 0 psi		
Load Dur Factor	1 250	Fv Allow	95 0 psi		
Beam End Fixity	Pin Pin	Fc Allow	625 0 psi		
Wood Density	32 000 pcf	E	1 600 0 ksi		

Full Length Uniform Loads

Center	DL	5 00 #/ft	LL	37 50 #/ft
Left Cantilever	DL	#/ft	LL	#/ft
Right Cantilever	DL	#/ft	LL	#/ft

Summary

Beam Design OK

Span= 12 50ft Beam Width = 5 500in x Depth = 3 5in Ends are Pin Pin

Max Stress Ratio	0 893 1		
Maximum Moment	0 9 k ft	Maximum Shear * 1 5	0 4 k
Allowable	1 0 k ft	Allowable	2 3 k

Max Positive Moment	0 91 k ft	at	6 250 ft	Shear	@ Left	0 29 k
Max Negative Moment	0 00 k ft	at	12 500 ft		@ Right	0 29 k
Max @ Left Support	0 00 k ft			Camber	@ Left	0 000 in
Max @ Right Support	0 00 k ft				@ Center	0 243 in
Max M allow	1 02				@ Right	0 000 in

				Reactions			
fb	976 34 psi	fv	21 87 psi	Left DL	0 06 k	Max	0 29 k
Fb	1,093 75 psi	Fv	118 75 psi	Right DL	0 06 k	Max	0 29 k

Deflections

Center Span	Dead Load	Total Load	Left Cantilever	Dead Load	Total Load
Deflection	0 162 in	0 817 in	Deflection	0 000 in	0 000 in
Location	6 250 ft	6 250 ft	Length/Defl	0 0	0 0
Length/Defl	925 4	183 55	**Right Cantilever**		
Camber (using 1 5 * D L Defl)			Deflection	0 000 in	0 000 in
@ Center	0 243 in		Length/Defl	0 0	0 0
@ Left	0 000 in				
@ Right	0 000 in				

Stress Calcs

Bending Analysis

Ck	31 019	Le	0 000 ft	Sxx	11 229 in3	Area	19 250 in2
Cf	1 000	Rb	0 000	Cl	0 000		

	Max Moment	Sxx Req d	Allowable fb
@ Center	0 91 k ft	10 02 in3	1 093 75 psi
@ Left Support	0 00 k ft	0 00 in3	1 093 75 psi
@ Right Support	0 00 k ft	0 00 in3	1 093 75 psi

Shear Analysis	@ Left Support	@ Right Support
Design Shear	0 42 k	0 42 k
Area Required	3 545 in2	3 545 in2
Fv Allowable	118 75 psi	118 75 psi

Bearing @ Supports

Max Left Reaction	0 29 k	Bearing Length Req'd	0 085 in
Max Right Reaction	0 29 k	Bearing Length Req d	0 085 in

Description Construction Planking - Double Layer of 4x6 DF-I #2

Query Values

M, V, & D @ Specified Locations		Moment	Shear	Deflection
@ Center Span Location =	0 00 ft	0 00 k ft	0 29 k	0 0000 in
@ Right Cant Location =	0 00 ft	0 00 k ft	0 00 k	0 0000 in
@ Left Cant Location =	0 00 ft	0 00 k ft	0 00 k	0 0000 in

Steel Beam Design

Description W12x14 - Elevator Support Beam - Construction Live Load - Share Load with W8x10

General Information

Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements

Steel Section W12X14

		Pinned Pinned		Fy	50 00ksi
Center Span	20 50 ft	Bm Wt Added to Loads		Load Duration Factor	1 00
Left Cant	0 00 ft	LL & ST Act Together		Elastic Modulus	29 000 0ksi
Right Cant	0 00 ft				
Lu Unbraced Length	0 00 ft				

Distributed Loads

	#1	#2	#3	#4	#5	#6	#7	
DL	0 060							k/ft
LL	0 694							k/ft
ST								k/ft
Start Location								ft
End Location								ft

Summary

Beam OK
Static Load Case Governs Stress

Using W12X14 section, Span = 20 50ft Fy = 50 0ksi
End Fixity = Pinned Pinned Lu = 0 00ft LDF = 1 000

	Actual	Allowable		
Moment	40 351 k ft	40 915 k ft	Max Deflection	1 188 in
fb Bending Stress	32 545 ksi	33 000 ksi	Length/DL Defl	2 145 8 1
fb / Fb	**0 986 1**		Length/(DL+LL Defl)	207 1 1
Shear	7 873 k	47 640 k		
fv Shear Stres	3 305 ksi	20 000 ksi		
fv / Fv	**0 165 1**			

Force & Stress Summary

<< These columns are Dead + Live Load placed as noted >>

	Maximum	DL Only	LL @ Center	LL+ST @ Center	LL @ Cants	LL+ST @ Cants	
Max M +	40 35 k ft	3 89	40 35				k ft
Max M							k ft
Max M @ Left							k ft
Max M @ Right							k ft
Shear @ Left	7 87 k	0 76	7 87				k
Shear @ Right	7 87 k	0 76	7 87				k
Center Defl	1 188 in	-0 115	1 188	1 188	0 000	0 000	in
Left Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Right Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Query Defl @	0 000 ft	0 000	0 000	0 000	0 000	0 000	in
Reaction @ Left	7 87	0 76	7 87	7 87			k
Reaction @ Rt	7 87	0 76	7 87	7 87			k

Fa calc'd per Eq E2 1 K*L/r < Cc
I Beam Passes Table B5 1, Fb per Eq F1 1, Fb = 0 66 Fy

Section Properties W12X14

Depth	11 910 in	Weight	14 13 #/ft	r xx	4 615 in
Width	3 970in	I xx	88 60 in4	r yy	0 753 in
Web Thick	0 200 in	I yy	2 36 in4	Rt	0 950 in
Flange Thickness	0 225 in	S xx	14 878 in3		
Area	4 16 in2	S yy	1 189 in3		

To specify your title block on these five lines, use the SETTINGS main menu selection, choose the Printing & Title Block tab, and ent your title block information

Title
Dsgnr
Description

Scope

Job #
Date 10 24AM 8 FEB 11

Steel Beam Design

Description W8x10 Sleeper over Existing W12x14

General Information

Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements

Steel Section W8X10

		Pinned Pinned		Fy	50 00ksi
Center Span	20 50 ft	Bm Wt Added to Loads		Load Duration Factor	1 00
Left Cant	0 00 ft	LL & ST Act Together		Elastic Modulus	29 000 0ksi
Right Cant	0 00 ft				
Lu Unbraced Length	4 00 ft				

Distributed Loads

	#1	#2	#3	#4	#5	#6	#7	
DL	0 060							k/ft
LL	0 244							k/ft
ST								k/ft
Start Location								ft
End Location	20 500							ft

Summary

Beam OK
Static Load Case Governs Stress

Using W8X10 section Span = 20 50ft Fy = 50 0ksi
End Fixity = Pinned Pinned Lu = 4 00ft LDF = 1 000

	Actual	Allowable
Moment	16 498 k ft	19 188 k ft
fb Bending Stress	25 357 ksi	29 492 ksi
fb / Fb	**0 860 1**	
Shear	3 219 k	26 826 k
fv Shear Stress	2 400 ksi	20 000 ksi
fv / Fv	**0 120 1**	

Max Deflection	1 397 in
Length/DL Defl	789 3 1
Length/(DL+LL Defl)	176 1 1

Force & Stress Summary

<< These columns are Dead + Live Load placed as noted >>

	Maximum	DL Only	LL @ Center	LL+ST @ Center	LL @ Cants	LL+ST @ Cants	
Max M +	16 50 k ft	3 68	16 50				k ft
Max M							k ft
Max M @ Left							k ft
Max M @ Right							k ft
Shear @ Left	3 22 k	0 72	3 22				k
Shear @ Right	3 22 k	0 72	3 22				k
Center Defl	1 397 in	0 312	1 397	1 397	0 000	0 000	in
Left Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Right Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Query Defl @	0 000 ft	0 000	0 000	0 000	0 000	0 000	in
Reaction @ Left	3 22	0 72	3 22	3 22			k
Reaction @ Rt	3 22	0 72	3 22	3 22			k

Fa calc'd per Eq E2 2, K*L/r > Cc
I Beam Major Axis, (102,000 * Cb / Fy)^ 5 <= L/rT <= (510 000 * Cb / Fy)^ 5 Fb per Eq F1-6

Section Properties W8X10

Depth	7 890 in	Weight	10 05 #/ft	r xx	3 226 in
Width	3 940in	I xx	30 80 in4	r yy	0 840 in
Web Thick	0 170 in	I yy	2 09 in4	Rt	0 990 in
Flange Thickness	0 205 in	S xx	7 807 in3		
Area	2 96 in2	S yy	1 061 in3		

STRUCTURAL CALCULATIONS

Permit Submittal

PARKING STRUCTURE II EPANSION
STRUCTURAL STEEL ERECTION BRACING

Portland, OR

For

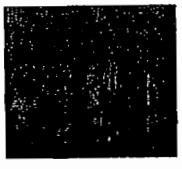
ALLIANCE INDUSTRIAL GROUP

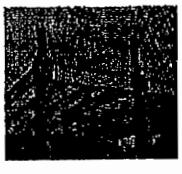
Date January 21, 2011
Project 2011005

☒ NO EXCEPTION NOTED ☐ MAKE CORRECTIONS NOTED
☐ REJECTED ☐ REVISE AND RESUBMIT

THIS REVIEW IS FOR GENERAL CONFORMANCE OF DESIGN TO LOADS AND METHODOLOGY ONLY ANY DEVIATION IN PLANS OR SPECIFICATIONS NOT CLEARLY NOTED BY T CONTRACTOR OR THEIR ENGINEER HAS NOT BEEN REVIEWED REVIEW SHALL NOT CONSTITUTE A COMPLETE CHECK OF DETAILED CALCULATIONS OR SERVE TO RELIEVE T CONTRACTOR AND THEIR ENGINEER OF CONTRACTUAL RESPONSIBILITY FOR ANY ERROR OR DEVIATION FROM CONTRACT DOCUMENTS AND BUILDING CODE REQUIREMENTS

AAI ENGINEERING
Date 02/08/11 By Derek Larsen


FACILITIES ENGINEERING


STRUCTURAL ENGINEERING

PILLAR CONSULTING GROUP, INC.

MECHANICAL ENGINEERING

PROJECT MANAGEMENT

1600 SW Western Blvd Suite #290
Corvallis Oregon 97333

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Phone (541) 752 9202
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PILLAR
CONSULTING
GROUP, INC
Professional Engineering and Project Management

ENGINEERING MEMORANDUM

TO	Andrew Haman, Alliance Industrial Group, Inc		
FROM	John Evans, PE SE	**DATE**	January 21, 2011
PROJECT	Parking Structure II Expansion – Structural Steel Erection Bracing Portland, Oregon		
SUBJECT	Permit Submittal		

We have completed our analysis and design of the structural steel erection bracing plan for the Structural Steel Erection Bracing for the Parking Structure II Expansion in Portland, Oregon Recommendations regarding erection stability bracing, column anchorage, minimum beam connections and other information will be found in the attached 8-1/2" x 11" drawing packet (32 sheets, dated 1/21/11)

We have analyzed and provided recommendations for the following conditions, in accordance with ASCE 37-02 and AISC Design Guide 10

- Stability of columns during erection
- Temporary wire rope x-bracing for lateral wind resistance during steel erection
- Stability of steel frame during deck shakeout and anchorage (loading is limited to the weight of the deck bundles and a maximum of two iron workers within any single area bounded by the building gridlines at one time)

We have not analyzed the building structure for stability or fitness-for-purpose during construction of the elevated concrete floor and roof slabs We recommend that the contractor responsible for that portion of the work retain the services of a qualified engineer to assess the structure during this phase of work Our recommendations are applicable and valid only for erection stability of the steel frame up to and including shakeout of the steel floor deck

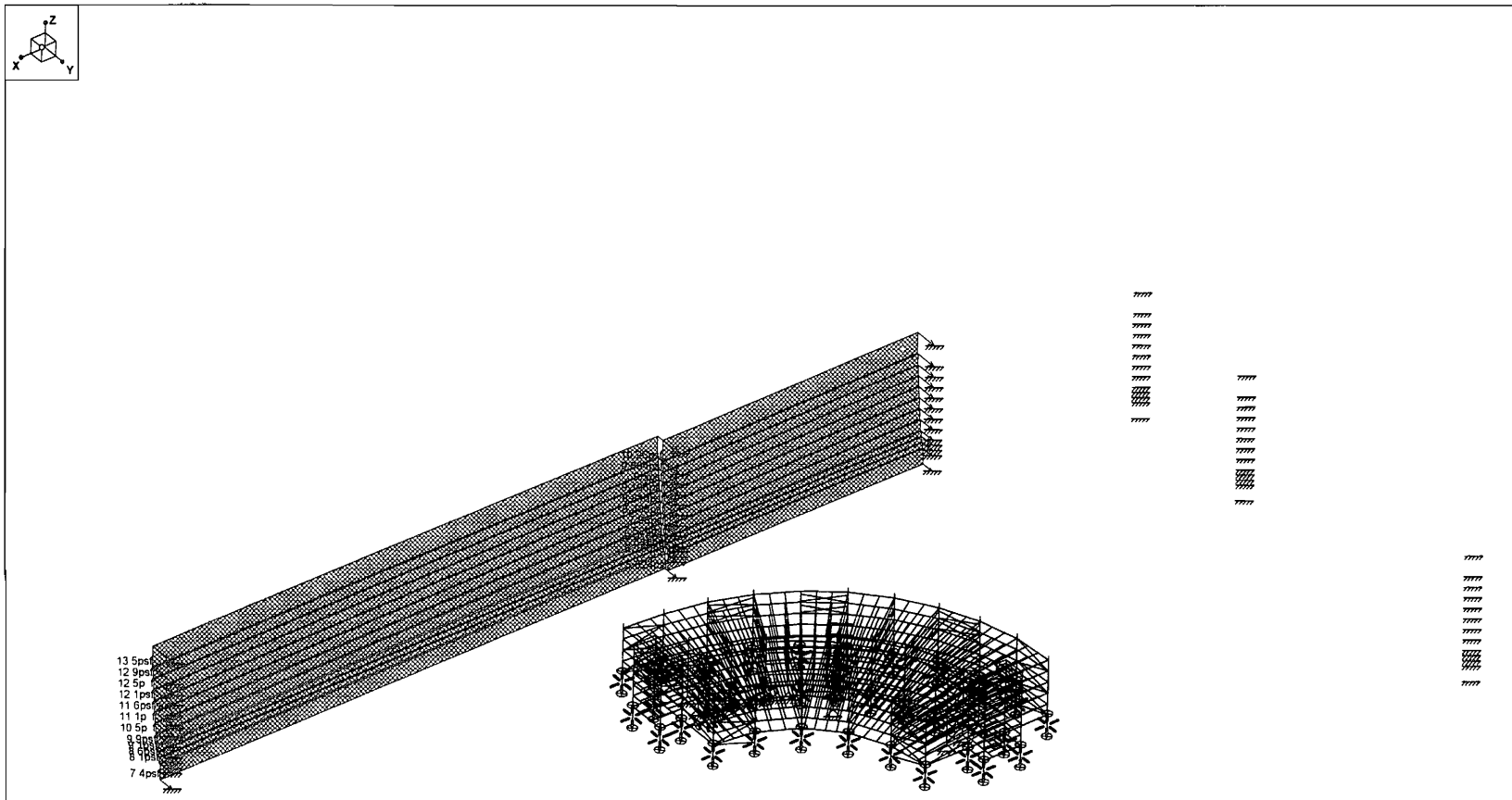
COPY TO			**PROJECT #**	20011005
VIA	☐ Fax	☐ US Mail	☐ Email	☐ N/A

1600 SW Western Blvd Suite 290 ■ Corvallis Oregon 97333 ■ www pillar inc com
Telephone (541) 752 9202 ■ Fax (541) 752 2587

Basis of Design

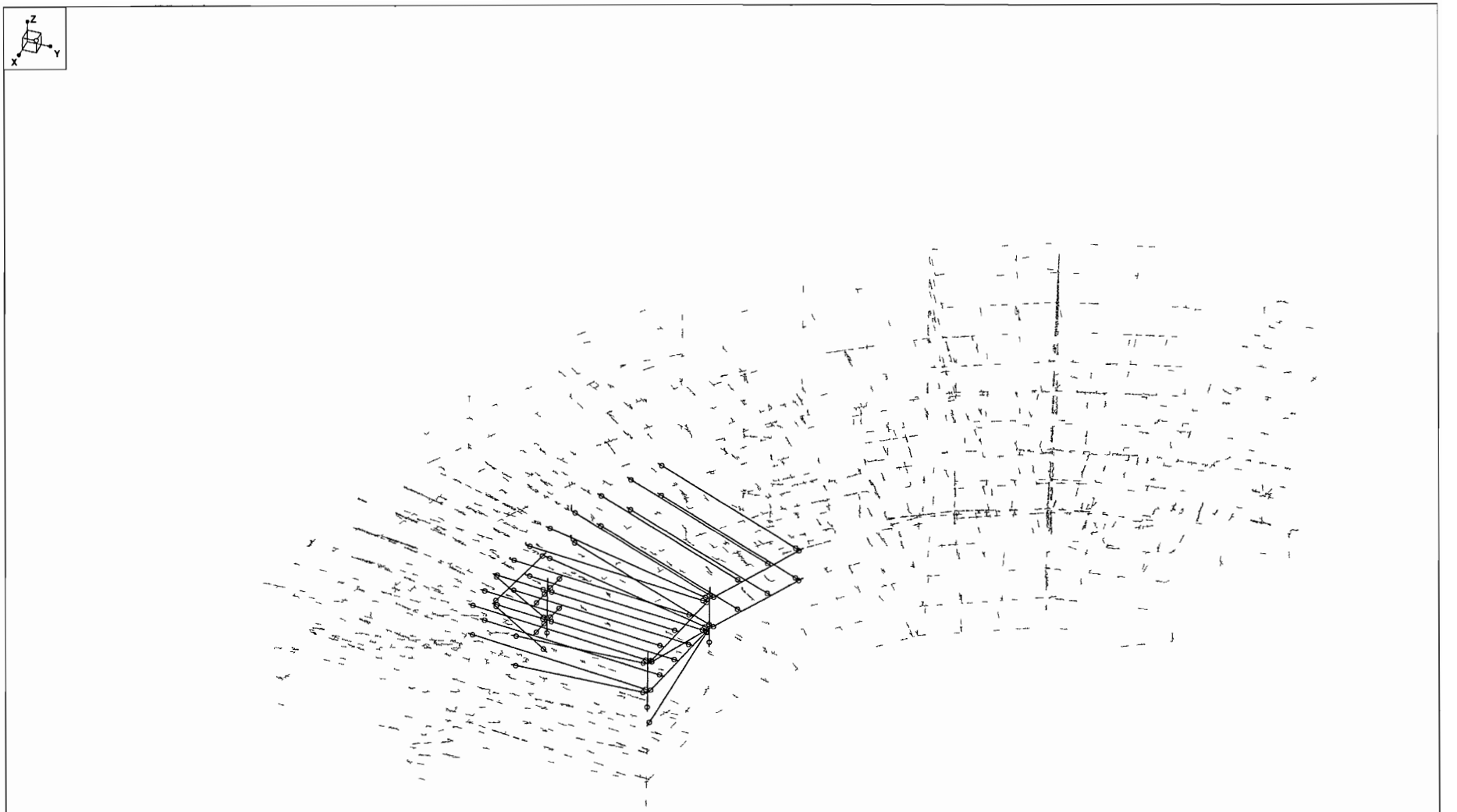
1. RISA Model created to represent existing building
    a. All existing gravity framing added
    b. All existing braced frames added
    c. Concrete floors modeled as rigid diaphragm
    d. Boundary conditions set at the base of steel structure
2. RISA Model created for each zone of erection (7 zones total)
    a. Each zone was individually added to the previous and analyzed (ie Zone 1 added to existing building and analyzed, Zone 2 added to existing +Zone 1 and analyzed, etc
    b. Column Splices modeled as fixed and splice capacity verified
3. Model Information
    a. Loads
        i. Dead Loads
            1. Self Weight of Structural Framing
            2. Metal Deck
                a. Added as line load on beam
                b. Accounts for worst case bending and shear in member
                c. Drawings indicate deck bundle zones to ensure shear is not exceeded
        ii. Live Loads
            1. (1) 250# man load added to each end of beams
                a. Accounts for shear in beams
                b. Accounts for shear and flexure in girders
        iii. Wind Loads (See wind load calculations)
            1. Modeled as an area load against the face of framing members
            2. Per ASCE 37-02 75% of wind speed was used for construction period less than 6 weeks
            3. Shielding was accounted for per AISC Design Guide 10
        iv. Load Combinations
            1. 0 5 used as arbitrary point in time (APT) factor per ASCE 37 when combining Live and Wind Loads
            2. Wind X and Y combined at 100% / 50%
    b. Cellular Beams (see calculations)
        i. Cellular beams were modeled as solid members
        ii. depth giving an equivalent solid area for wind exposure
        iii. cross sectional area giving an equivalent weight

c Wire Ropes (see calculations)
- i ½" 6x19 IWRC (EIPS) used
  1. Factor of Safety = 3 on Rope Breaking Strength (AISC DG 10)
  2. Factor of Safety = 2 on Rope Breaking x Connection Efficiency
- ii ½" 6x19 Choker Used
  1. Factor of Safety = 2 (See AISC Design Guide 10)
- iii -100 Degree F load added = +/- 600 lb prestress

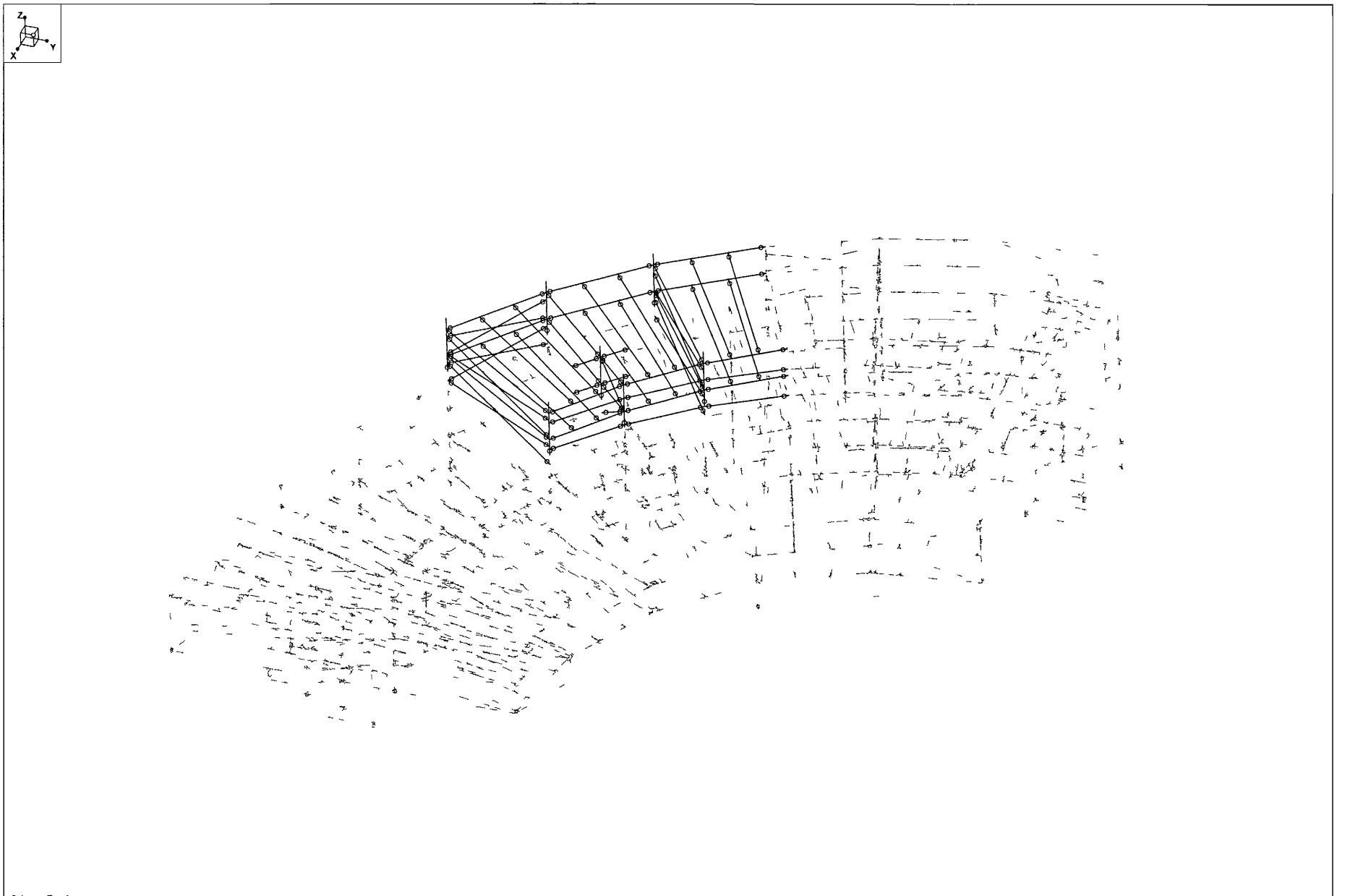


Loads BLC 3 WINDY
Soluti E lope

	WIND Y LOADING SHOWN	SK - 1
		Jan 21, 2011 at 4 00 PM
		zone 7 r3d



	ZONE 7 SHOWN IN BOLD	SK - 2
		Jan 21, 2011 at 3 57 PM
		zone 7 r3d



		SK - 3
		Jan 21 2011 at 4 09 PM
	ZONE 2 SHOWN IN BOLD	zone 2 r3d

Analysis Summary

Zone 1

Grid 13 Brace Frame – Max Axial = 15 5k
Grid AA Brace Frame N – Max Axial = 13 1k
} < 34k – (2) BOLT BRACE CONN ok

Grid AA Wire Brace – Max Axial = 3 0k — < 8k – (1) 1/2" ϕ ok

Grid 11 Wire Brace – Max Axial = 2 6k
Grid 12 Wire Brace – Max Axial = 2 7k
Grid 14 Wire Brace – Max Axial = 9 3k
Grid 15 Wire Brace – Max Axial = 4 3k
Grid CC Wire Brace – Max Axial = 5 1k
Grid DD Wire Brace – Max Axial = 3 1k
Grid EE Wire Brace – Max Axial = 3 0k
} < 16k – (2) 1/2" ϕ ok

Zone 2

Grid 13 Brace Frame – Max Axial = 11 7k
Grid AA Brace Frame N – Max Axial = 18 7k
Grid 17 Brace Frame – Max Axial = 11 9k
} < 34k – (2) BOLT BRACE CONN ok

Grid AA Wire Brace 1 – Max Axial = 2 6k
Grid AA Wire Brace 2 – Max Axial = 6 8k
Grid 16 Wire Brace – Max Axial = 12 2k
Grid 18 Wire Brace – Max Axial = 10 5k
Grid CC Wire Brace – Max Axial = 4 5k
Grid DD Wire Brace – Max Axial = 5 2k
Grid EE Wire Brace – Max Axial = 3 8k
} < 16k – (2) 1/2" ϕ ok

Zone 3

Grid 13 Brace Frame – Max Axial = 30 1k
Grid AA Brace Frame N – Max Axial = 24 4k
Grid 17 Brace Frame – Max Axial = 24 9k
Grid 21 Brace Frame – Max Axial = 12 6k
} < 34k – (2) BOLT BRACE CONN ok

Grid AA Wire Brace 1 – Max Axial = 2 9k
Grid AA Wire Brace 2 – Max Axial = 6 8k
Grid AA Wire Brace 3 – Max Axial = 8 0k
Grid 19 Wire Brace – Max Axial = 9 5k
Grid 20 Wire Brace – Max Axial = 6 6k
Grid 21 Wire Brace – Max Axial = 12 2k
Grid CC Wire Brace – Max Axial = 4 5k
Grid DD Wire Brace – Max Axial = 5 6k
Grid EE Wire Brace – Max Axial = 4 0
} < 16k – (2) 1/2" ϕ IWRC ok

Zone 4

Grid AA Brace Frame N – Max Axial = 42k
Grid AA Brace Frame E – Max Axial = 41k
} WELD BRACES $\frac{42^k}{3.71/in}$ = 12" of 1/4" FILLET = 44.4k

Grid 21 Brace Frame – Max Axial = 8 2k
Grid AA Wire Brace 1 – Max Axial = 2 3k
Grid AA Wire Brace 2 – Max Axial = 3 0k
Grid AA Wire Brace 3 – Max Axial = 2 4k
Grid 22 Wire Brace – Max Axial = 3 3k
} < 16k - (2) 1/2" Φ ok

Grid 23 Wire Brace – Max Axial = 1 1k < 8k - (1) 1/2" Φ ok

Grid CC Wire Brace – Max Axial = 2 4k
Grid DD Wire Brace – Max Axial = 1 9k
Grid EE Wire Brace – Max Axial = 1 5k
} < 16k - (2) 1/2" Φ ok

Zone 5

Grid 13 Brace Frame – Max Axial = 12 0k < 34k - 2 BOLT BRACE CONN ok
Grid AA Brace Frame N – Max Axial = 36 3k - BRACE WELDED @ ZONE 4
Grid GG Brace Frame N – Max Axial = 9 1k - < 34k - 2 BOLT BRACE CONN ok

Grid 11 Wire Brace – Max Axial = 4 2k
Grid 12 Wire Brace – Max Axial = 7 3k
Grid 14 Wire Brace – Max Axial = 6 3k
Grid CC Wire Brace – Max Axial = 6 7k
Grid DD Wire Brace – Max Axial = 8 0k
Grid EE Wire Brace – Max Axial = 6 0k
} < 16k - (2) 1/2" Φ ok

Zone 6

Grid 13 Brace Frame – Max Axial = 12 4k < 34k - (2) BOLT BRACE CONN. ok
Grid AA Brace Frame N – Max Axial = 43k - BRACE WELDED @ ZONE 4
Grid GG Brace Frame N – Max Axial = 25 8k - < 34k - (2) BOLT BRACE CONN ok

Grid 17 Brace Frame – Max Axial = 25 6k
Grid 16 Wire Brace – Max Axial = 2 0k
Grid 18 Wire Brace – Max Axial = 2 2k
Grid CC Wire Brace – Max Axial = 6 9k
Grid DD Wire Brace – Max Axial = 7 4k
Grid EE Wire Brace – Max Axial = 5 9k
} < 16k - (2) BOLT BRACE CONN ok

Zone 7

Grid AA Brace Frame N – Max Axial = 34 4k
Grid AA Brace Frame E – Max Axial = 37 0
} BRACE WELDED @ ZONE 4

Grid GG Brace Frame N – Max Axial = 34 9k
Grid GG Brace Frame E – Max Axial = 37 0
} WELD BRACE

Grid 13 Brace Frame – Max Axial = 12 3k
Grid 17 Brace Frame – Max Axial = 25 3k
Grid 21 Brace Frame – Max Axial = 15 8k
} < 34k (2) BOLT BRACE CONN ok

Grid AA Wire Brace 1 – Max Axial = 5 2k
Grid AA Wire Brace 2 – Max Axial = 5 9k
Grid AA Wire Brace 3 – Max Axial = 5 5k
Grid 19 Wire Brace – Max Axial = 7 4k
Grid 20 Wire Brace – Max Axial = 7 1k
Grid 22 Wire Brace – Max Axial = 8 9k
Grid CC Wire Brace – Max Axial = 7 1k
Grid EE Wire Brace – Max Axial = 5 3k
} < 16k (2) 1/2" Φ WIRE ok

CONSTRUCTION WIND LOADS ON STEEL FRAME

$F = q_z G C_f A_f$ <ASCE 7-05, EQ 6-28>

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$

6 WEEK DURATION → 0.75 (95 MPH)

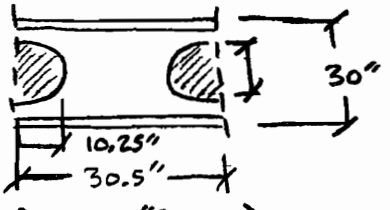
$V = 71$ mph
$K_{zt} = 1.0$
$I = 1.0$
$K_d = 0.85$

$q_z = 0.00256(1)(1)(.85)(71)^2 (K_z)$
$q_z = 10.97 (K_z)$

$G = 0.85$
C_f = ASCE 7-05, FIG 6-22

for cellular beams

$C_f = 1.4$ for solid ($w\,h/D \le 7$) (ASCE FIG 6-21)



($\approx$ 24% WT REDUCTION FROM SOLID SHAPE)

$A_g = 30"(30.5") = 915$ in²
$A_n = A_g - \pi r^2 = 915 - \pi(10.25)^2$
$A_n = 585$ in²

$E = 585/915 = 0.64$ (36% AREA REDUCTION FOR WIND)

$\therefore C_f = 1.6$ per ASCE FIG 6-22

$\frac{1.6}{1.4} = 1.14$ $\quad 1.14(0.64) = 0.73$

$\therefore$ EQUIV. "SOLID" SHAPE WIND LOAD ON CELLULAR BEAMS $= 0.73(A_g)$(WIND PRESSURE)

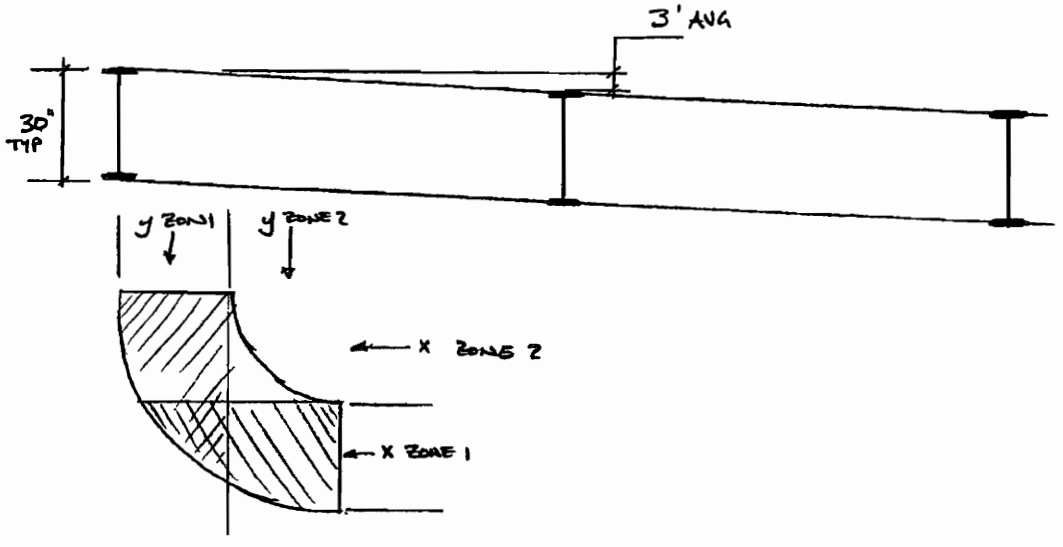
USE $F = 10.97(K_z)(0.85)(1.4)(A_f) = (K_z)(13)(A_f)$ SOLID MEMB.
$= 0.73(K_z)(13)(A_f) = (K_z)(9.5)(A_f)$ CELLULAR MEMB.
(or simply 0.73 x SOLID)

SUBJECT.	PROJECT#: 200907
	DATE.
	BY JWM
CONSTRUCTION WIND LOADS	SHEET: 1 OF

HT	Kz	SOLID MEMBER WIND LOAD (PSF)
0-15'	0.57	7.4 PSF
15'-20'	0.62	8.1
20'-25'	0.66	8 6
25'-30'	0.70	9.1
30'-40'	0 76	9 9
40'-50'	0.81	10 5
50'-60'	0.85	11.1
60'-70'	0 89	11.6
70'-80'	0.93	12 1
80'-90'	0.96	12.5
90'-100'	0 99	12 9
100'-120'	1.04	13 5

SUBJECT:	PROJECT#
	DATE
	BY.
	SHEET: 2 OF

WIND REDUCTION FOR SHIELDING
PER ASCE DG 10



X ZONE 2 & y ZONE 2 - NO SHIELDING

y ZONE 1 & X ZONE 1 - SHIELDING

a = 14' AVG } a/d = 56
d = 30"

SOLIDARITY RATIO = 0.64
PER FIG 32 OF ASCE DG 10 - n = 038
n = 8 (AVERAGE)

$A_B = (1 + 038 + (B-2)(0.38)^2)A = 2.25 A_1$, $\frac{2.25}{8} A_1 = 028 A_1$

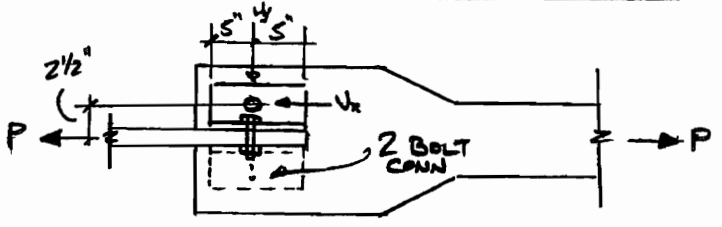
w/ n = 7	$\rightarrow A_7 = 2.1 A_x$,	$\frac{2.1}{7} A_1 = 03 A_x$
w/ n = 6	$\rightarrow A_6 = 196 A_x$	$= 033 A_x$
w/ n = 5	$\rightarrow A_5 = 181 A_x$	$= 036 A_x$
w/ n = 4	$\rightarrow A_4 = 167 A_x$	$= 042 A_x$
w/ n = 3	$\rightarrow A_3 = 1.52 A_x$	$= 051 A_x$
w/ n = 2	$\rightarrow A_2 = 1.38 A_x$	$= 0.69 A_x$
w/ n = 1	$\rightarrow A_1 = 1.0 A_x$	$= 10 A_x$

AVG DUE TO VERT STEP IN BEAMS = 0.49

USE 49% OF PROJECTED AREA OR $\frac{049}{0.64}$ = 076% ACTUAL AREA

SUBJECT	PROJECT#:
PARKING	DATE
	BY
	SHEET OF

PERMANENT BRACE CONNECTION



w/ (1) BOLT CONNECTION (¾" ϕ A490 X)

IF P = 17^k, V_x = 17^k, V_y = 17^k $\left(\frac{2.5"}{5"}\right)$ = 8.5 SRSS = 19^k

NO SHEAR LAG EFFECTS (PER COMMENTARY)

¾" ϕ A325X – CAPACITY = 16.6 × 1.2 = 199^k

BOLT BEARING @ ⅜" ϕ = ⅜ (52 2^k/in) = 19.6^k ✓ ok

w/ 2" EDGE DIST

w/ (2) BOLT CONNECTION ((2) ¾" ϕ A490X)

IF P = 34^k, V_x = $\frac{34^k}{2}$ = 17^k

NO SHEAR LAG EFFECTS (PER COMMENTARY)

¾" ϕ A490X – CAPACITY = 16.6^k × 1.2 = 199^k

BOLT BEARING @ ⅜" ϕ = 19^k (SEE ABV) ✓ ok

w/ 2" EDGE DIST

SUBJECT	PROJECT#.
POP II PARKING BRACE CONN.	DATE
	BY
	SHEET OF

WIRE ROPE

- 1/2" IWRC (EIPS)

NOMINAL BREAKING STRENGTH = 26.6^k
T_{ALLOW} = 8.87^k

- 1/2" 6x19 CHOKER
CAPACITY = 16T = 32^k @ FS = 5 (PER CATALOG)
@ FS = 2 - CAPACITY = 3.2^k (5/2) = 8^k ← GOVERNS

USE 1/2" 6x19 IWRC w/ CHOKER
CAPACITY = 8^k

OR

(2) 1/2" 6x19 IWRC w/ CHOKER
CAPACITY = 16^k

SUBJECT	PROJECT#
	DATE
	BY
	SHEET OF

Pillar Consulting Group, Inc
1600 SW Western Blvd Suite 290
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PROJECT# 200907
PROJECT POP II
DESIGNER JWM
DESCRIPTION 1/2 INCH DIA 6x19 IWRC EIPS

EFFECTIVE MODULUS & AXIAL STRETCH OF WIRE ROPE AS A FUNCTION OF AXIAL LOAD

Descrption & Assumptions

This template is calculates an effective modulus of elasticity for wire rope of a given diameter secant modulus and effective metallic area Preload and constructional stretch are incorporated into the axial deformation calculations The effective modulus is independant of the length but specific magnitude of stretch is not Ref AISC Design Guide 10 (Copyright 2003)

Wire Rope Input Data

Rope Diam	Rope Modulus	Metallic Area	Nom Brkg Str	Length	Preload	Constr Stretch	Descrption
$d_1 = 0\,50$ in	$E_1 = 15000$ ksi	$A_1 = 0\,118$ in^2	$NBS_1 = 26\,6$ kip	$L_1 = 32$ ft	$Pr_1 = 0\,0$ kip	$\%CS_1 = 0\,75\%$	1/2" dia IWRC EIPS 6x19

$FS_1 = 3$ factor of safety against NBS of rope $FS_2 = 2$ factor of safety against (NBS x connection effeciency)

Let $T_1 = 1$ kip, 2 kip NBS_1 $Eff = 80\%$ Connection efficiency of wire rope connection

$$\delta_1(L_1,T_1) = \begin{cases} \left[\frac{0\,2\,(NBS_1 - Pr_1)\,L_1}{A_1\,0\,9\,E_1} + \left[\frac{\left[T_1 - 0\,2\,(NBS_1)\right]\left[L_1 + \frac{0\,2\,(NBS_1 - Pr_1)\,L_1}{A_1\,0\,9\,E_1}\right]}{A_1\,E_1}\right] + \frac{T_1\,\%CS_1\,L_1}{0\,65\,NBS_1}\right] & \text{if } T_1 > 0\,2\,NBS_1 \\ \frac{T_1\,L_1}{A_1\,0\,9\,E_1} + \frac{T_1\,\%CS_1\,L_1}{0\,65\,NBS_1} & \text{otherwise} \end{cases}$$

$$E_{eff}(L_1,T_1) = \frac{T_1\,L_1}{A_1\,\delta_1(L_1,T_1)} \qquad E_{eff\_20\%NBS} = \frac{0\,2\,NBS_1\,L_1}{A_1\,\delta_1(L_1,0\,2\,NBS_1)} \qquad T_{20\%NBS} = 0\,2\,NBS_1$$

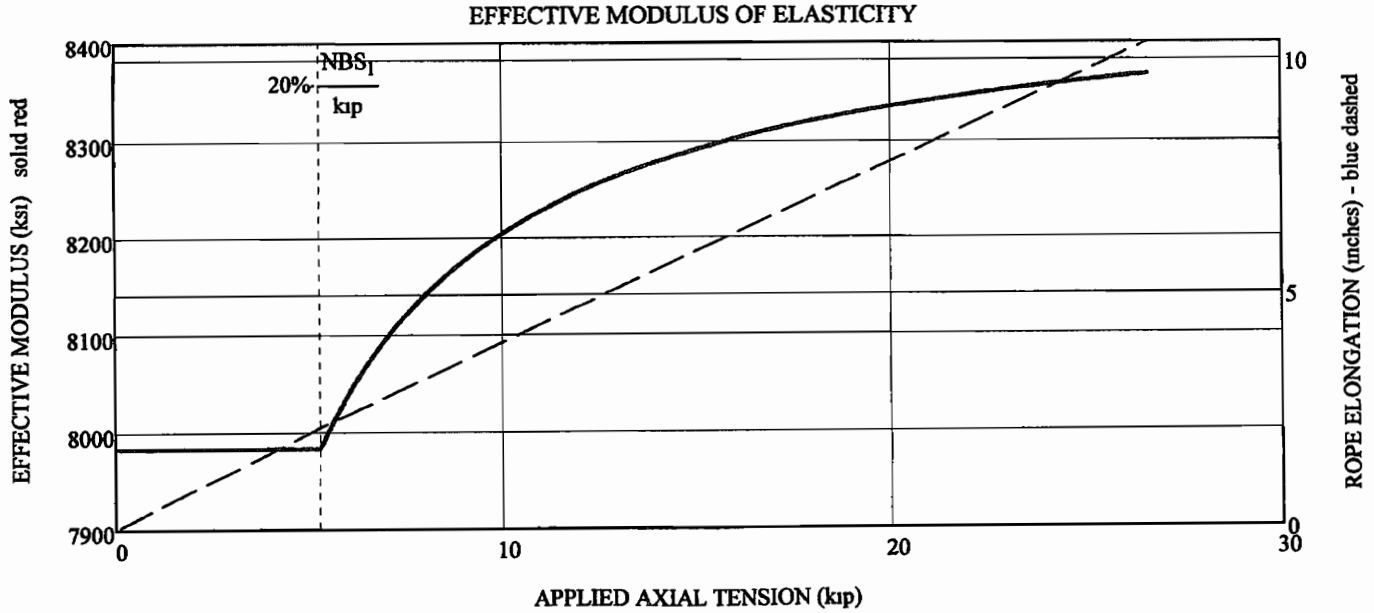
Specific Elongation Sample

$\delta_1(L_1,5\text{ kip}) = 2\,038$ in

$$d_{equiv} = \sqrt{\frac{4\,A_1}{\pi}} \qquad T_{allow} = \min\left(\frac{NBS_1}{FS_1}, \frac{NBS_1\,Eff}{FS_2}\right) \qquad f_{yequiv} = \frac{T_{allow}\,1\,67}{A_1}$$

$d_{equiv} = 0\,388$ in $f_{yequiv} = 125\,5$ ksi

$E_{eff\_20\%NBS} = 7983$ ksi $T_{20\%NBS} = 5\,32$ kip $\delta_{20\%NBS} = \delta_1(L_1, 2NBS_1) = 2\,17$ in $\delta_{NBS} = \delta_1(L_1,NBS_1) = 10\,35$ in $T_{allow} = 8\,87$ kip



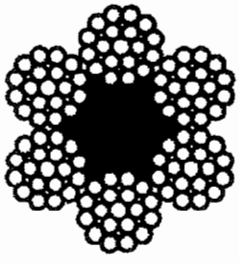
West Coast Wire Rope

WIRE ROPE

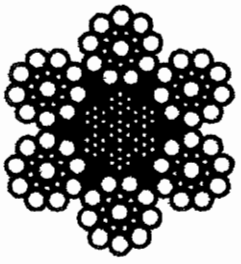
West Coast Wire Rope

GENERAL PURPOSE

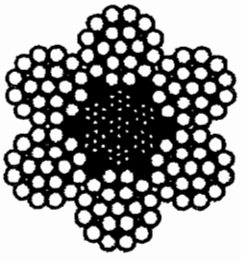
BRIGHT 6X19 CLASS (6x19, 6X21, 6X25, 6X26)



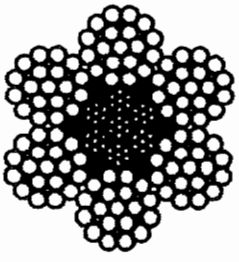
6 x 25 FILLER WIRE
FIBER CORE



6 X 19 SEALE
IWRC



6 X 25 FILLER WIRE
IWRC



6 X 26 IWRC
WARRINGTON SEALE

(IWRC INDEPENDENT WIRE ROPE CORE OR CENTER)
According to Federal Specifications RR-W-410D, preformed, right regular lay, light to heavy lubrication

Read important warnings and information on pages 6 - 7 and 12 preceding wire rope section

5/16	0 164	4 69	0 18	5 27
7/16	0 32	9 09	0 35	10.2
9/16	0 53	14 09	0 59	16 8
3/4	0 95	26 2	1 04	29 4
1	1 68	46	1 85	51 7
1 1/4	2 63	71	2 89	79 9
1 1/2	3 78	101	4 16	114
1 3/4	5 15	136	5 67	153
2 1/4	8 51	220	9 36	247
2 3/4	-	-	14	361
3 1/2	-	-	22 6	564

$\frac{NBS}{} = 266^{\#}$

NOTE Lang Lay, left lay, alternate lay seale available in some sizes
Heavy lubrication on request

*Listed for comparison only Actual operating loads may vary but should never exceed the recommended design factor or 20% of catalog Breaking Strength

SLINGS
WIRE ROPE

West Coast Wire Rope

WORKING LOAD LIMITS
HAND TUCKED SPLICE
FIBER CORE & IWRC

1-PART SLINGS
VERTICAL, CHOKER OR VERTICAL BASKET

RATED CAPACITY IN TONS OF 2,000 LBS

RATED CAPACITIES SHOWN APPLY ONLY TO 6X19 AND 6X37 CLASSIFICATION WIRE ROPE

ROPE DIA. (IN)	VERTICAL	CHOKER	BASKET
	EIPS	EIPS	EIPS
5/16	0 83	0 66	1 7
7/16	1 6	1 3	3 2
9/16	2 5	2 1	5 0
3/4	4 3	3 7	8 6
1	7 4	6 4	15
1 1/4	11	9 9	23
1 1/2	16	14	32
1 3/4	22	19	44
2	28	25	56
2 1/4	35	31	70
2 1/2	43	38	86

8K MAX CAP

13K MAX CAP

18 5K MAX @ FS=2

RATED CAPACITIES BASKET HITCH BASED ON D/d RATIO OF 15
RATED CAPACITIES BASED ON PIN DIAMETER NO LARGER THAT NATURAL EYE WIDTH OR LESS THAN THE NOMINAL SLING DIAMETER
RATED CAPACITIES BASED ON DESIGN FACTOR OF 5
HORIZONTAL SLING ANGLES LESS THAN 30 DEGREES SHALL NOT BE USED

DRAG LINES

WIND ⊥ TO GRIDS ① TEMPORARY BRACE FRAME

WIND = 11.1 (A_F) @ T5, T6 & T7

COLUMN $11.1^{psf}(15''/12'')(2)(10.5') = 291^{\#}$
GIRDERS' $11.1^{psf}(27''/12)(37.8')(3'') = \underline{2832}$
$3123^{\#}$

BEAMS ARE ∥ TO WIND — NO WIND CONTRIBUTION FROM BEAMS

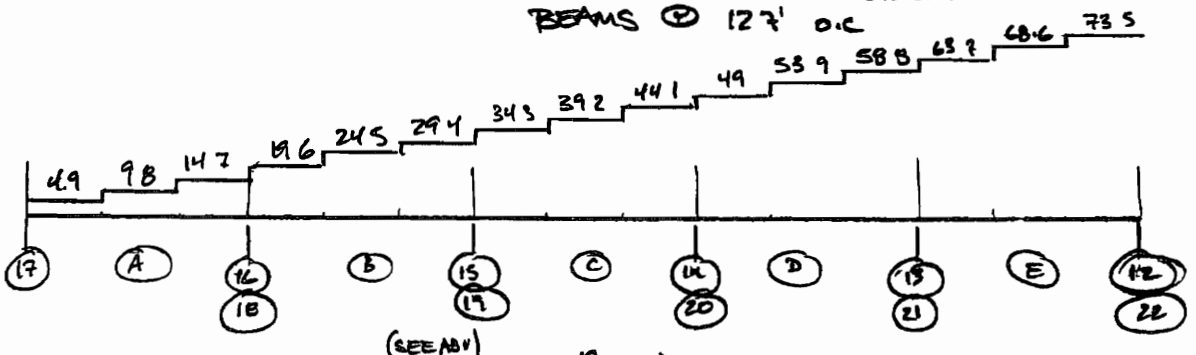
AXIAL STRESS IN BEAM = $\frac{3123^{\#}}{18^{\square''}(TYP)} = 240^{psi}$

GRAVITY = $34^{plf}(\frac{60'}{2}) + 850^{\#}_{MAN} + 625^{plf}_{SELF}(\frac{60'}{2}) = 3245^{\#}$ (EA END) NEGLIGIBLE ✓ok

(2) 7/8" ⌀ A325 IN 3/8" MIN PL ok BY INSP

WIND ∥ TO GRID AA
DUE TO STIFFNESS - ASSUME ALL LOAD GOES TO PERMANENT BRACE AT END (CONSERVATIVE)
WIND = 11.1 A_F
WIND RXN ① BM END = $0.76_{\text{SHIELDING}}(0.64)_{\text{BM OPENINGS CASTELLATED}}(30')(11.1^{psf})(60'/2) = 4.9^k$

BEAMS ② 12.7' O.C



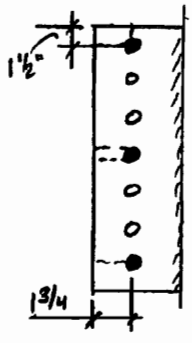
(SEE ABV)
GRAVITY = $3245^{\#} + 84^{plf}(38'/2) = 4.8^k$ ↓

SUBJECT	PROJECT#
COLLECTORS	DATE
	BY
	SHEET OF

DRAG LINES CONTINUED

WIND // TO GRID AA CONT.

TYPICAL CONNECTION W27x84



W27x84 BRACED @ 12.5 o.c

$\frac{kl}{r} = \frac{1(12.5')(12")}{207} = 72.5$

$\frac{F_{cr}}{\Omega} = 20.4^{ksi}$

$P_{max} = 20.4^{ksi}(24.8^{in^2}) = 506^k > 73.5^k$ ok

W/ NOMINAL GRAVITY LOAD
W27x84 ok

BOLTS - (3) 7/8" Ø A325 N

$\frac{R_n}{\Omega} = (14.4^k)(3) = 43.2^k$ ← GOVERNS

BEARING - @ 1¼" = 27.2^k
@ 2" = 58.3^k } INTERPOLATE → 1¾" = 44.6^k

$(3)(44.6^k)(3/8") = 50^k$

BLOCK SHEAR: @ 3 BOLTS

$A_{gv} = 1.75"(3/8")(4) = 2.625^{in^2}$
$A_{nv} = (1.75" - 13/32")(3/8")(4) = 1.922^{in^2}$
$A_{gt} = 1.5"(3/8")(2) = 1.125^{in^2}$
$A_{nt} = (1.5" - 13/32")(3/8")(2) = 0.773^{in^2}$
$U_{bs} = 1.0$
$F_u = 58^{ksi}$
$F_y = 36^{ksi}$

$\frac{R_n}{\Omega_1} = 111.7^k$, $\frac{R_n}{\Omega_2} = 101.5^k$

SUBJECT	PROJECT#.
COLLECTORS CONT	DATE
	BY:
	SHEET OF

DRAG LINES CONT

WIND ∥ TO GRID AA CONT

GRIDS 15-19 – HORIZ SHR = 34.3^k
VERT SHR = 4.8^k
SRSS = $\sqrt{34.3^2 + 4.8^2}$ = 34.6^k < 50^k
(3) 7/8" ϕ BOLTS OK

GRIDS 12-14 & 20-22
HORIZ SHR = 73.5^k
VERT SHR = 4.8^k
SRSS = $\sqrt{73.5^2 + 4.8^2}$ = 73.7^k
(6) 7/8" ϕ A325N BOLTS
BOLTS – $\frac{R_n}{\Omega}$ = 14.4^k (6) = 86.4^k √OK

BEARING
EDGED DST – (6)(44.6^k)(3/8) = 100^k √OK
SPACING – (6)(42.1^k)(3/8) = 94.7^k √OK
↑ FOR 3" o.c.
BLOCK SHEAR – OK BY INSP.
(SEE 8 BOLT COND.)

GRID GG
PER GRID A – (6) BEAMS CAN TAKE WIND LOAD W/ (3) BOLT CONDITION (COUNTING FROM MIDPOINT OF GRID LINE LENGTH
∴ PROVIDE (6) BOLTS @ GRIDS: 13, 14, 21 & 22

SUBJECT	PROJECT#:
COLLECTORS, CONT	DATE:
	BY:
	SHEET OF

DRAG LINES CONT

⊙ # GRIDS - MAX BRACE FRAME AXIAL = 30.1^k ⊙ ZONE 3, GRID 1B

BY INSP - IF BRACE FORCE = 30.1^k THEN COLLECTOR = < 30.1^k

(3) 7/8" ϕ A325 N ek

SUBJECT	PROJECT#:
COLLECTORS	DATE:
	BY
	SHEET: OF

COLUMN SPLICE

WORST CASE: W14x193 CANTILEVERS 21'

WIND = 11.6 PSF
$d = 15.5"$
$t_f = 15\frac{3}{4}"$

$M = 11.6^{psf}(15.75"/12"/1')(21')(21'/2)$

$= 3.4^{k\,ft}$ ✓ ok

(SEE ATTACHED FOR CAPACITIES)

HSS 8x8 CANTILEVERS 21'

WIND = 11.6 PSF

$M = 11.6^{psf}(8"/12"/1')(21')(21'/2) = 17^{k\,ft}$

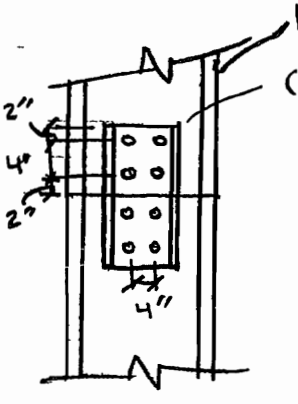
SHEAR ON BOLTS = $\frac{17^{k\cdot ft}(12"/1')}{16"}$

$= 1275^{\#}$

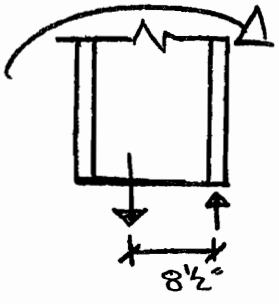
ok BY INSD

SUBJECT:	PROJECT#.
	DATE
	BY:
	SHEET: OF

COLUMN SPLICES:



COLUMN STRONG AXIS MOMENT CAPACITY OF SPLICE



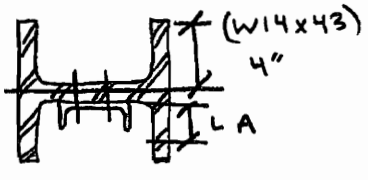
$\frac{R_n}{\Omega}$ BOLTS IN SHEAR = (2)(10.6 K) = 21.2 K

$M_{xa(MAX)}$ = (8½)(21.2) = 180 K in

$M_{xa\,MAX}$ = 15 K·ft

$\frac{R_n}{\Omega}$ BOLTS IN BRG = (52.2 K/inch)(0.285)(2)
= 29.8 K DOES NOT CONTROL

WEAK AXIS MOMENT CAPACITY OF SPLICE



(MILLED SURFACE)

t_f = 0.53" min

t_w = 0.305" min

$\frac{R_n}{\Omega}$ (BOLTS) = (4)(10.6) = 42.4 K

BRG AREA REQ'D FOR 42.4 K

$\frac{R_n}{\Omega} = \frac{1.8\,F_y\,A_{pb}}{2}$ <AISC 13th J7-1>

A_{pb} req'd = (42.4)(2)/1.8(50) = 0.94 in²

$L_A = 4" - \frac{0.305"}{2} - \frac{0.94\,in^2}{(2)(0.53")} = 2.96"$

2011005

SUBJECT:	PROJECT#. [illegible]
	DATE: 4-1-09
	BY: JWM
COLUMN SPLICE MOMENT CAP.	SHEET: 1 OF

$M_{y\phi\text{-MAX}} = (42.4^K)(2.96'') = 125\tfrac{1}{2}\ K\text{-in}$

$M_{y\phi\text{-MAX}} = 10.5\ K\text{-ft}$

CHECK TENSION CAP OF C9x15

$A_g = 4.41\ in^2$ $\quad F_y = 36\ KSI$

$A_n = 3.91\ in^2$ $\quad F_u = 58\ KSI$

$t_w = 0.285\ in$

$R_n/\Omega = \frac{F_y A_g}{1.6} = \frac{36(4.41)}{1.6} = 99^K$ OK

$R_n/\Omega = \frac{F_u A_e}{\Omega}$ $\quad A_e = UA_n$ $\quad U = 1 - \frac{\bar{x}}{l}$ $\quad \bar{x} = 0.586$, $l = 4''$

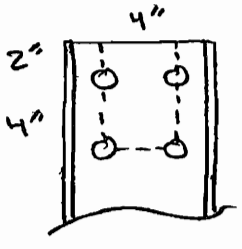
$U = 1 - \frac{.586}{4}$

$U = 0.85$

$= \frac{(58)(3.91)(0.85)}{2.0}$

$R_n/\Omega = 96.8^K$ OK

CHECK BLOCK SHEAR:



$F_y A_{gv} = (36)(3.42\ \phi'') = 123^K$ ← controls

$F_u A_{nv} = (58)(2.67\ \phi'') = 155^K$

$U_{bs} = 0.5$ STRONG AXIS moment

1.0 WK AXIS moment

$F_u A_{nt} = (58)(0.89\ \phi'') = 51.7^K$

$\frac{R_n}{\Omega}$ (STRONG AXIS) $= \frac{123 + 0.5(51.7)}{2} = 74.4^K$ (DOES NOT CONTROL)

$\frac{R_n}{\Omega}$ (WK AXIS) → DOES NOT CONTROL by INSP.

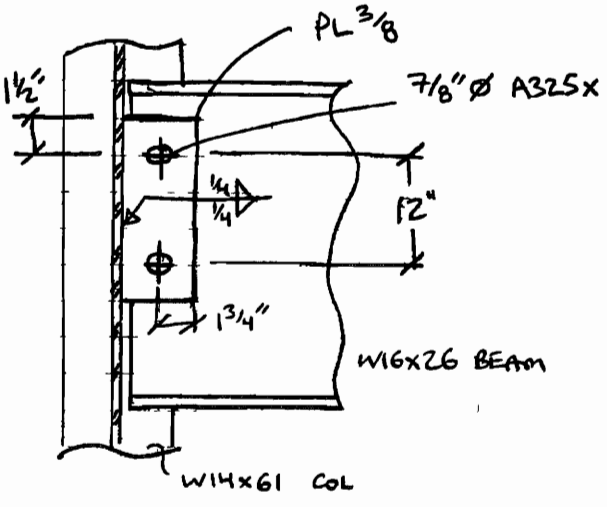
∴ MAX ALLOW MOMENT ON COLUMN SPLICE DURING ERECTION: 15 K-ft STRONG-AXIS

10½ K-ft WK-AXIS (8" flg)

SUBJECT	PROJECT#: 2011005
	DATE: 4-1-09
	BY: JWM
COL. SPLICE MOMENT CAP	SHEET: 2 OF

CONTINUOUS WORK PERIOD FRAME CHECK

TYPICAL CONNECTION:



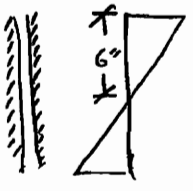
BOLT SHEAR

BOLT R_{nv}/Ω = 18K

M_n/Ω for bolts = (18)(1')

M_n/Ω = 18 K-FT (BOLTS)

FLAG WELD-TO-COL



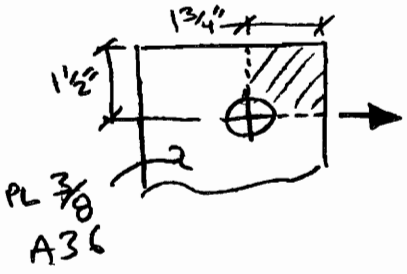
(1.5)(0.928)(4)(2) = 11.14K

33.42K

33.42K

M_n/Ω = 16.7 K-FT (WELDS)

BLOCK SHEAR OF TAB:



$0.6F_yA_{gv}$ = 0.6(36)(1.75)(0.375) = 14.18K ← controls

$0.6F_uA_{nv}$ = 0.6(58)(1.25)(0.375) = 16.31K

$U_{bs}F_uA_{nt}$ = 0.5(58)(1)(.375") = 10.875K

Ω = 2.0 $\frac{R_n}{\Omega}$ = 12.5K

$\frac{M_n}{\Omega}$ = (12.5 K)(1') = 12.5 K-FT (BLOCK SHEAR)

controls

ALLOW TEMP. MOMENT CAPACITY @ BM-TO-COL CONN ≈ 12.5 K-FT

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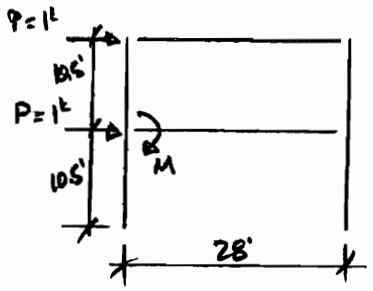
	SHEET 1 OF



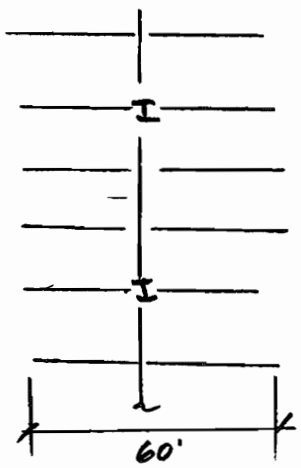
w/ FIXED BASE - $M = 4.9^{k\cdot ft}$

w/ PINNED BASE - $M = 10.6^{k\cdot ft}$ } $15.5^{k\cdot ft}$ AVERAGE

ASSUME PINNED BASE

$$P_{MAX} = \frac{12.5^{k\cdot ft}}{10.6' ft}(1^k) = 1200^{\#}$$

WIND TRIB TO FRAME



EQUIV SOLID FOR CASTELATED

$0.73(3')(30^{\#}/12^{\#})(60') + 2(14^{\#}/12^{\#})(10.5') = 341^{\#}/TIER$

$$\frac{1200^{\#}}{341^{\#}} = 3.5^{psf} = P$$

$P = q_z G C_f$, $C_f = 1.4$, $G = 0.85$

$BS = q_z (0.85)(1.4)$

$q_z = 30^{psf}$

$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$

$$V_{MAX} = \sqrt{30 / [0.00256(0.96)(1.0)(0.85)(1.0)]}$$

$= 38$ MPH

NO BRACING REQ'D FOR GUSTS < 38 MPH

USE 30 MPH

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	DATE:
	BY:
	SHEET: OF

CELLULAR BEAMS

CREATE SECTION W/ SIM WT. & WIND EXPOSURE AS CELLULAR BEAM

EQUIV SOLID AREA

CELL BEAM	WT	d	0.75d	AREA
LB30x57/68	62.5	30.6	22.3	183#
LB30x57/62	59.5	30.5	22.3	175#
LB30x57	57	30.5	22.3	168#
LB30x50/57	53.5	30.4	22.2	157#
LB30x44/50	47	30.2	22.0	138#
LB21x22/26	24	20.2	14.7	71#
LB18x22	22	17.8	13.0	65#

SUBJECT: PARKING STRUCT CELLULAR BEAMS	PROJECT#. 2011005
	DATE
	BY
	SHEET OF

SHORING @ CC.5/12, DD 3/12, CC 5/22, DD.3/22

DL: FRAMING - 5 psf
DECKING - 3 psf
MISC - 2 psf
10 psf

LL: CONST - 25 psf

TRIB AREA = 12' x 21½ = 126 ft

LOAD = (10 psf + 25 psf)(126 ft) = 4410#

SPECIFY 5^k SHORING LOAD

SUBJECT	PROJECT#.
	DATE
	BY
	SHEET OF

METAL DECK INSTALL & BRACE REMOVAL

// TO LETTER GRID

GRID AA & GG - WIND TRANSFERS DIRECTLY TO GIRDER
∴ NO DIAPHRAM FORCES

GRID DD - FRAME WILL LIKELY TAKE ALL FORCES
BUT, SAY DIAPH CANTILEVERS OFF GRIDS AA & GG

OVER (1) BAY, WIND HITS (6) BEAMS

$0.49\,(0.73)(11.1^{psf})(30''/12''/')(60'/2)(6) = 1786^{\#}/BAY$
SHIELDING EQUIV DEPTH

$\frac{1786^{\#}}{18'} = 100^{plf} < 137^{plf}$ ✓ ok

// TO # GRIDS

$11.1^{psf}(27''/12''/')(2)(37.67'(\frac{4}{2})) = 3763^{\#}$ TO FRAME LINE

$\frac{3763^{\#}}{60'} = 63^{plf} < 137^{plf}$ ✓ ok

3W x 20 GA DECK w/
1 PDF @ EA EDGE @ EA SUPPORT
& BUTTON PUNCH @ 36' O.C ok

GRID 17 BF -
BEAM AREA = $(27''/12 \times 37.67' \times 4 + 21''/12 \times 28.3' \times 4) = 537^{\#}/TIER$
COL AREA = $(10.5')(15''/12''/')(3.5) = 46^{\#}/TIER$
$583^{\#}$

$11.1^{psf}(583^{\#})(2) = 129^{k}/BRACE$ - TEMP CONN OK
2 TIERS

GRID AA BF
SAME @ GG
BEAM AREA = $(0.73(30'/12)(60')(16)) = 1752^{\#}/TIER$
COL AREA = $(10.5)(15''/12''/')(3.5)(2) = 92^{\#}$
$1844^{\#}$

$11.1(1844^{\#})(2) = 41^{k}/BRACE$ - 12" of 1/4" FILLET OK = 44^{k}
2 TIERS

SUBJECT:	PROJECT#:
	DATE
	BY
	SHEET: OF

1 PDF @ Ea Edge @ Ea Support, Button Punch @ 36" oc

Connection Variables

t	0 047	inches	Deck
l	42 67	feet	Deck
h	3	inches	Deck
w	36	inches	Deck
np	3		# Purlins
s	36	inches	BP Spacing
#P	2		# Pins
x1	16	inches	Pin Distance to Center of Deck
x2	16	inches	Pin Distance to Center of Deck
x3	0	inches	Pin Distance to Center of Deck
x4	0	inches	Pin Distance to Center of Deck
A	1		Usually, if not, its Conservative

Deck Variables

d	12	inches	flute pitch
e	2 25	inches	1/2 lower flute flat
w	3 35	inches	diag dim
f	4 5	inches	upper flute flat
I	1 203	in^4/ft	

Safety Factors

Ωc	2 35	ULT->ASD	FS Connections
Ωc	2	ULT->ASD	FS Deck

Solved Values

Qf	2329 132	lbs	pin
Qs	530 16	lbs	BP
αs	0 23		
lv	10 67	feet	span
λ	0 38		
$\alpha 1$, $\alpha 1$	0 89		
ne, ns	14 00		# BP in a deck piece
B	7 14		
N	0 33	per foot	# pins per foot
s	15 7	inches	developed flute width

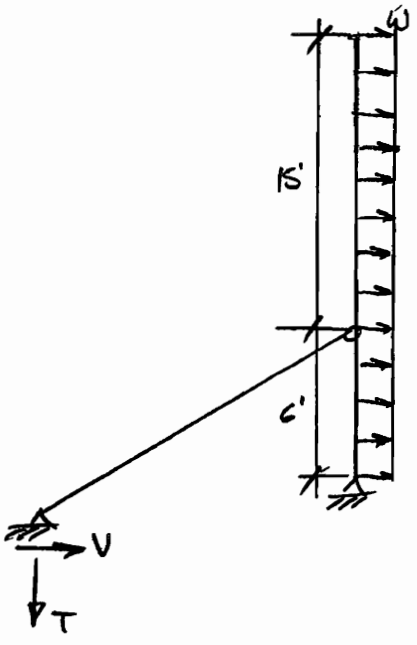
Connections

Sne	1006 8	plf
Snl	322 4	plf
Snc	348 2	plf

Deck

Snb	3096 4	plf

Governing	322 4	plf	ULT
ASD	137 2	plf	ASD



DECK = 3" w/ 3½" NWT CONC (3000#)

$w = 116^{pcf}\left(15'/12''/'\right) = 14.5^{plf}$

V = 533#
T = 267#

3/8" FORGED EYE BOLT
IN 3/8" TITEN HD ROD COUPLE

PER SIMPSON CATALOG
3/8" ϕ x 2 3/4" EMBED
ALLOW V = 1060#
ALLOW T = 640# } FS = 4.0

$\frac{533}{1060} + \frac{267}{640} = .92$ ✓ok
FS = 4/.92 = 4.34 ✓ok

FORCE IN ROPE =
$\sqrt{533^2 + 267^2} = 600^{\#}$

(1) ½" ϕ ok

w/ BRACE @ 1:1
V = 533#
T = 533#

$\frac{533}{1060} + \frac{533}{640} = 1.34$

$FS = \frac{4}{1.34} = 3.0$ ✓ok

FORCE IN BRACE = $\sqrt{2}(533^{\#})$
$= 754^{\#}$

SPECIFY 2000#

SUBJECT	PROJECT#:
	DATE:
	BY
	SHEET OF

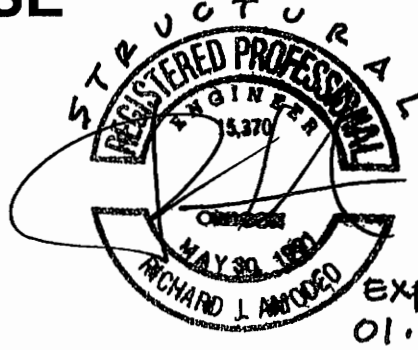
AAI aigham associates, inc.
ENGINEERING

(3)

STRUCTURAL CALCULATIONS

PROJECT: POP 2 Parking Garage Expansion
Construction Life Safety Plan
PROJECT No.: A10200
DATE: February 8, 2011

PLAN REVIEW RESPONSE



Contents

Calculations 1 – 10

10-09336 DMO 01 CO

4875 SW Griffith Drive | Suite 300 | Beaverton OR | 97005

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ITEM 3d CONFIRM CAPACITY OF EXISTING BEAMS FOR SHORING LOADS → DETAIL 1/S513

REF: 2/PSS2.2 FOR EXISTING FRAMING:

W14x22 w/(6) STUDS
21'-0" SPAN
$\frac{9'+5.5'}{2}$ + 0.5' TRIB = 7.75'

DEAD LOAD 3" 18ga Deck w/ 3½" TOPPING 61 psf
10 psf Additional
71 psf DEAD LOAD.

CONSTRUCTION LIVE LOAD - 25 psf

SHORING LOAD: 76 sf x 25psf LIVE LOAD
+ 48 sf x 150psf LIVE LOAD
125 sf x 20 psf DEAD LOAD
11600 lbs

PLACE THIS LOAD @ 1'-6" FROM BEAM END.

BEAM WORKS AS A NON-COMPOSITE SECTION
SEE ATTACHED:

END CONNECTION = 16.21 kips

(3) 7/8"Ø A325 N → REFERENCE 1/S5.2
C=1.75 $\frac{r_n}{\Omega}$ = 14.4 kips
$\frac{R_n}{\Omega}$ = 25.2 kips OK

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POP II Parking GARAGE EXPANSION By DEL Date 01/06/11
STRUCTURAL PLAN REVIEW Project No A10200
ITEM 3d Sheet 1 of

Steel Beam Design

Description W14x22 Uncomposite - Plan Review Item 3D

General Information

Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements

Steel Section W14X22

		Pinned Pinned		Fy	50 00 ksi
Center Span	21 00 ft	Bm Wt Added to Loads		Load Duration Factor	1 00
Left Cant	0 00 ft	LL & ST Act Together		Elastic Modulus	29 000 0 ksi
Right Cant	0 00 ft				
Lu Unbraced Length	0 00 ft				

Point Loads

	# 1	# 2	# 3	# 4	# 5	# 6	# 7	
Dead Load	3 850	3 850	2 500					k
Live Load	1 360	1 360	9 100					k
Short Term								k
Location	7 000	14 000	1 500					ft

Summary

Beam OK
Static Load Case Governs Stress

Using W14X22 section Span = 21 00ft Fy = 50 0ksi
End Fixity = Pinned Pinned Lu = 0 00ft LDF = 1 000

	Actual	Allowable
Moment	49 108 k ft	79 658 k ft
fb Bending Stress	20 344 ksi	33 000 ksi
fb / Fb	**0 616 1**	
Shear	16 213 k	63 204 k
fv Shear Stress	5 130 ksi	20 000 ksi
fv / Fv	**0 257 1**	

Max Deflection	0 673 in
Length/DL Defl	591 0 1
Length/(DL+LL Defl)	374 5 1

Force & Stress Summary

	Maximum	DL Only	<< These columns are Dead + Live Load placed as noted -->> LL @ Center	LL+ST @ Center	LL @ Cants	LL+ST @ Cants	
Max M +	49 11 k ft	30 52	49 11				k ft
Max M							k ft
Max M @ Left							k ft
Max M @ Right							k ft
Shear @ Left	16 21 k	6 40	16 21				k
Shear @ Right	6 27 k	4 26	6 27				k
Center Defl	-0 673 in	0 426	0 673	-0 673	0 000	0 000 in	
Left Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000 in	
Right Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000 in	
Query Defl @	0 000 ft	0 000	0 000	0 000	0 000	0 000 in	
Reaction @ Left	16 21	6 40	16 21	16 21			k
Reaction @ Rt	6 27	4 26	6 27	6 27			k

Fa calc'd per Eq E2-1 K*L/r < Cc
I Beam Passes Table B5 1 Fb per Eq F1-1 Fb = 0 66 Fy

Section Properties W14X22

Depth	13 740 in	Weight	22 04 #/ft	r xx	5 537 in
Width	5 000 in	I xx	199 00 in4	r yy	1 039 in
Web Thick	0 230 in	I yy	7 00 in4	Rt	1 250 in
Flange Thickness	0 335 in	S xx	28 967 in3		
Area	6 49 in2	S yy	2 800 in3		

PROTECTIVE COVERS OVER EXISTING ELEVATORS:

NORTH ELEVATOR WILL BE OPEN TO PUBLIC DURING INITIAL STEEL ERECTION, SO REVIEW EXISTING CAPACITY OF EXISTING ROOF: REFERENCE: 3/PSS2.5 FROM ORIGINAL DRAWINGS:

HAVE W12x14 BEAMS SPANNING 21'-0" WITH 6'-0" OF TRIBUTARY

ROOF DECK 3" 18ga N DECK SPANNING 12'-0"
CAPACITY AT $\Delta = \frac{L}{240} = 0.45"$ IS 46 psf

NEITHER OF THESE COMPONENTS HAVE ENOUGH CAPACITY TO SUPPORT CONSTRUCTION LIVE LOAD. SOUTH WALL WILL BE REMOVED SO MAY NEED TO ADD PROTECTIVE COVER INSIDE OF SHAFT.

SPAN FROM EXISTING W14x22 AND HSS18x6x1/4

W14x22 BEAM HAS ENOUGH CAPACITY AS A NON COMPOSITE SECTION

HSS18x6x1/4 HAS ENOUGH CAPACITY AS WELL.

RECOMMEND INSTALLING PLANKS ACROSS OPEN SHAFT AT TIER 5 ELEVATION. ASSUME THAT ELEVATOR CAN BE MADE TO WORK STOPPING AT TIER 4.

→ DOES NOT WORK. PUT PLANKING ON EXISTING ROOF

Description W14x22 Beam with Construction Live Load

General Information

Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements

Steel Section W14X22

		Pinned Pinned		Fy	50 00 ksi
Center Span	20 50 ft	Bm Wt Added to Loads		Load Duration Factor	1 00
Left Cant	0 00 ft	LL & ST Act Together		Elastic Modulus	29 000 0 ksi
Right Cant	0 00 ft				
Lu Unbraced Length	0 00 ft				

Distributed Loads

	#1	#2	#3	#4	#5	#6	#7	
DL	0 045							k/ft
LL	0 675							k/ft
ST								k/ft
Start Location								ft
End Location								ft

Point Loads

	#1	#2	#3	#4	#5	#6	#7	
Dead Load	3 600	3 600						k
Live Load	1 300	1 300						k
Short Term								k
Location	7 000	14 000						ft

Summary

Beam OK
Static Load Case Governs Stress

Using W14X22 section Span = 20 50ft Fy = 50 0ksi
End Fixity = Pinned Pinned Lu = 0 00ft LDF = 1 000

	Actual	Allowable		
Moment	72 065 k ft	79 658 k ft	Max Deflection	0 956 in
fb Bending Stress	29 855 ksi	33 000 ksi	Length/DL Defl	659 7 1
fb / Fb	**0 905 1**		Length/(DL+LL Defl)	257 4 1
Shear	12 625 k	63 204 k		
fv Shear Stress	3 995 ksi	20 000 ksi		
fv / Fv	**0 200 1**			

Force & Stress Summary

<< These columns are Dead + Live Load placed as noted -->>

	Maximum	DL Only	LL @ Center	LL+ST @ Center	LL @ Cants	LL+ST @ Cants	
Max M +	72 07 k ft	27 88	72 07				k-ft
Max M -							k-ft
Max M @ Left							k ft
Max M @ Right							k-ft
Shear @ Left	12 39 k	4 20	12 39				k
Shear @ Right	12 63 k	4 38	12 63				k
Center Defl	0 956 in	0 373	0 956	-0 956	0 000	0 000	in
Left Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Right Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Query Defl @	0 000 ft	0 000	0 000	0 000	0 000	0 000	in
Reaction @ Left	12 39	4 20	12 39	12 39			k
Reaction @ Rt	12 63	4 38	12 63	12 63			k

Fa calc'd per Eq E2 1 K*L/r < Cc
I Beam Passes Table B5 1 Fb per Eq F1 1 Fb = 0 66 Fy

Description W14x22 Beam with Construction Live Load

Section Properties W14X22

Depth	13 740 in	Weight	22 04 #/ft	r xx	5 537 in
Width	5 000in	I xx	199 00 in4	r yy	1 039 in
Web Thick	0 230 in	I yy	7 00 in4	Rt	1 250 in
Flange Thickness	0 335 in	S xx	28 967 in3		
Area	6 49 in2	S yy	2 800 in3		

PROTECTIVE COVER OVER ELEVATOR:

TRY 4x6 PLANKING OVER ELEVATOR ROOF SPANNING FROM W12x14 TO W12x14.

L = 12.5' USE DOUBLE LAYER OF 4x6 DF-L#2

EXISTING W12x14 BEAMS ARE OVERSTRESSED 25%.

PUT W8x10 BEAM OVER TOP OF ROOF AND SHARE LOAD BETWEEN BEAMS.

I of W12x14 =	88.6	0.74
I of W8x10 =	30.8	0.26
	119.4	

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POP II PARKING STRUCTURE EXPANSION - STRUCTURAL PLAN REVIEW - ITEM 11.

By DBL Date 01/08/11

Project No A102.00

Sheet 6 of

General Timber Beam

Description Construction Planking - Double Layer of 4x6 DF-I #2

General Information

Calculations are designed to 1997 NDS and 1997 UBC Requirements

Section Name	4x6	Center Span	12 50 ft	Lu	0 00 ft
Beam Width	5 500 in	Left Cantilever	ft	Lu	0 00 ft
Beam Depth	3 500 in	Right Cantilever	ft	Lu	0 00 ft
Member Type	Sawn	Douglas Fir Larch No 2			
Bm Wt Added to Loads		Fb Base Allow	875 0 psi		
Load Dur Factor	1 250	Fv Allow	95 0 psi		
Beam End Fixity	Pin Pin	Fc Allow	625 0 psi		
Wood Density	32 000 pcf	E	1 600 0 ksi		

Full Length Uniform Loads

Center	DL	5 00 #/ft	LL	37 50 #/ft
Left Cantilever	DL	#/ft	LL	#/ft
Right Cantilever	DL	#/ft	LL	#/ft

Summary

Beam Design OK

Span= 12 50ft, Beam Width = 5 500in x Depth = 3 5in Ends are Pin Pin

Max Stress Ratio 0 893 1

Maximum Moment	0 9 k ft	Maximum Shear * 1 5	0 4 k
Allowable	1 0 k ft	Allowable	2 3 k

Max Positive Moment	0 91 k ft	at	6 250 ft	Shear	@ Left	0 29 k
Max Negative Moment	0 00 k ft	at	12 500 ft		@ Right	0 29 k
Max @ Left Support	0 00 k ft			Camber	@ Left	0 000 in
Max @ Right Support	0 00 k ft				@ Center	0 243 in
Max M allow	1 02				@ Right	0 000 in

				Reactions			
fb	976 34 psi	fv	21 87 psi	Left DL	0 06 k	Max	0 29 k
Fb	1 093 75 psi	Fv	118 75 psi	Right DL	0 06 k	Max	0 29 k

Deflections

Center Span	Dead Load	Total Load	Left Cantilever	Dead Load	Total Load
Deflection	0 162 in	0 817 in	Deflection	0 000 in	0 000 in
Location	6 250 ft	6 250 ft	Length/Defl	0 0	0 0
Length/Defl	925 4	183 55	**Right Cantilever**		
Camber (using 1 5 * D L Defl)			Deflection	0 000 in	0 000 in
@ Center	0 243 in		Length/Defl	0 0	0 0
@ Left	0 000 in				
@ Right	0 000 in				

Stress Calcs

Bending Analysis

Ck	31 019	Le	0 000 ft	Sxx	11 229 in3	Area	19 250 in2
Cf	1 000	Rb	0 000	Cl	0 000		

	Max Moment	Sxx Req d	Allowable fb
@ Center	0 91 k ft	10 02 in3	1 093 75 psi
@ Left Support	0 00 k ft	0 00 in3	1 093 75 psi
@ Right Support	0 00 k ft	0 00 in3	1 093 75 psi

Shear Analysis	@ Left Support	@ Right Support
Design Shear	0 42 k	0 42 k
Area Required	3 545 in2	3 545 in2
Fv Allowable	118 75 psi	118 75 psi

Bearing @ Supports

Max Left Reaction	0 29 k	Bearing Length Req d	0 085 in
Max Right Reaction	0 29 k	Bearing Length Req d	0 085 in

Description Construction Planking - Double Layer of 4x6 DF-I #2

Query Values

M, V, & D @ Specified Locations		Moment	Shear	Deflection
@ Center Span Location =	0 00 ft	0 00 k ft	0 29 k	0 0000 in
@ Right Cant Location =	0 00 ft	0 00 k ft	0 00 k	0 0000 in
@ Left Cant Location =	0 00 ft	0 00 k ft	0 00 k	0 0000 in

Steel Beam Design

Description W12x14 - Elevator Support Beam - Construction Live Load - Share Load with W8x10

General Information

Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements

Steel Section W12X14

		Pinned Pinned		Fy	50 00ksi
Center Span	20 50 ft	Bm Wt Added to Loads		Load Duration Factor	1 00
Left Cant	0 00 ft	LL & ST Act Together		Elastic Modulus	29 000 0ksi
Right Cant	0 00 ft				
Lu Unbraced Length	0 00 ft				

Distributed Loads

	#1	#2	#3	#4	#5	#6	#7	
DL	0 060							k/ft
LL	0 694							k/ft
ST								k/ft
Start Location								ft
End Location								ft

Summary

Beam OK
Static Load Case Governs Stress

Using W12X14 section Span = 20 50ft Fy = 50 0ksi
End Fixity = Pinned Pinned Lu = 0 00ft LDF = 1 000

	Actual	Allowable
Moment	40 351 k ft	40 915 k ft
fb Bending Stress	32 545 ksi	33 000 ksi
fb / Fb	**0 986 1**	
Shear	7 873 k	47 640 k
fv Shear Stres‹	3 305 ksi	20 000 ksi
fv / Fv	**0 165 1**	

Max Deflection	-1 188 in
Length/DL Defl	2 145 8 1
Length/(DL+LL Defl)	207 1 1

Force & Stress Summary

	Maximum	DL Only	LL @ Center	LL+ST @ Center	LL @ Cants	LL+ST @ Cants	
		<<– These columns are Dead + Live Load placed as noted >>					
Max M +	40 35 k ft	3 89	40 35				k ft
Max M							k ft
Max M @ Left							k ft
Max M @ Right							k ft
Shear @ Left	7 87 k	0 76	7 87				k
Shear @ Right	7 87 k	0 76	7 87				k
Center Defl	1 188 in	-0 115	1 188	1 188	0 000	0 000	in
Left Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Right Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Query Defl @	0 000 ft	0 000	0 000	0 000	0 000	0 000	in
Reaction @ Left	7 87	0 76	7 87	7 87			k
Reaction @ Rt	7 87	0 76	7 87	7 87			k

Fa calc'd per Eq E2 1 K*L/r < Cc
I Beam Passes Table B5 1 Fb per Eq F1 1, Fb = 0 66 Fy

Section Properties W12X14

Depth	11 910 in	Weight	14 13 #/ft	r xx	4 615 in
Width	3 970in	I xx	88 60 in4	r yy	0 753 in
Web Thick	0 200 in	I yy	2 36 in4	Rt	0 950 in
Flange Thickness	0 225 in	S xx	14 878 in3		
Area	4 16 in2	S yy	1 189 in3		

Steel Beam Design

Description W8x10 Sleeper over Existing W12x14

General Information

Calculations are designed to AISC 9th Edition ASD and 1997 UBC Requirements

Steel Section W8X10

		Pinned Pinned		Fy	50 00ksi
Center Span	20 50 ft	Bm Wt Added to Loads		Load Duration Factor	1 00
Left Cant	0 00 ft	LL & ST Act Together		Elastic Modulus	29 000 0ksi
Right Cant	0 00 ft				
Lu Unbraced Length	4 00 ft				

Distributed Loads

	#1	#2	#3	#4	#5	#6	#7	
DL	0 060							k/ft
LL	0 244							k/ft
ST								k/ft
Start Location								ft
End Location	20 500							ft

Summary

Beam OK
Static Load Case Governs Stress

Using W8X10 section Span = 20 50ft Fy = 50 0ksi
End Fixity = Pinned Pinned Lu = 4 00ft LDF = 1 000

	Actual	Allowable		
Moment	16 498 k ft	19 188 k ft	Max Deflection	1 397 in
fb Bending Stress	25 357 ksi	29 492 ksi	Length/DL Defl	789 3 1
fb / Fb	**0 860 1**		Length/(DL+LL Defl)	176 1 1
Shear	3 219 k	26 826 k		
fv Shear Stres	2 400 ksi	20 000 ksi		
fv / Fv	**0 120 1**			

Force & Stress Summary

	Maximum	DL Only	<< These columns are Dead + Live Load placed as noted >> LL @ Center	LL+ST @ Center	LL @ Cants	LL+ST @ Cants	
Max M +	16 50 k ft	3 68	16 50				k ft
Max M -							k-ft
Max M @ Left							k ft
Max M @ Right							k ft
Shear @ Left	3 22 k	0 72	3 22				k
Shear @ Right	3 22 k	0 72	3 22				k
Center Defl	1 397 in	0 312	1 397	-1 397	0 000	0 000	in
Left Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Right Cant Defl	0 000 in	0 000	0 000	0 000	0 000	0 000	in
Query Defl @	0 000 ft	0 000	0 000	0 000	0 000	0 000	in
Reaction @ Left	3 22	0 72	3 22	3 22			k
Reaction @ Rt	3 22	0 72	3 22	3 22			k

Fa calc'd per Eq E2 2 K*L/r > Cc
I Beam Major Axis (102 000 * Cb / Fy)^ 5 <= L/rT <= (510 000 * Cb / Fy)^ 5 Fb per Eq F1-6

Section Properties W8X10

Depth	7 890 in	Weight	10 05 #/ft	r xx	3 226 in
Width	3 940in	I xx	30 80 in4	r yy	0 840 in
Web Thick	0 170 in	I yy	2 09 in4	Rt	0 990 in
Flange Thickness	0 205 in	S xx	7 807 in3		
Area	2 96 in2	S yy	1 061 in3		

WDY Structural • Civil Engineers

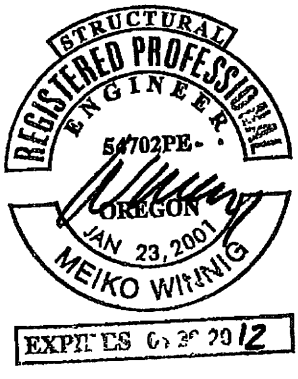
STRUCTURAL CALCULATIONS

FOR

PROVIDENCE MEDICAL CENTER POP2
CONSTRUCTION PROTECTION CANOPIES

PORTLAND, OREGON

DECEMBER 22, 2010



☒ NO EXCEPTION NOTED ☐ MAKE CORRECTIONS NOTED
☐ REJECTED ☐ REVISE AND RESUBMIT

THIS REVIEW IS FOR GENERAL CONFORMANCE OF D
LOADS AND METHODOLOGY ONLY ANY DEVIATION F
PLANS OR SPECIFICATIONS NOT CLEARLY NOTED b
CONTRACTOR OR THEIR ENGINEER HAS NOT BEEN RE
REVIEW SHALL NOT CONSTITUTE A COMPLETE C
DETAILED CALCULATIONS OR SERVE TO RELI
CONTRACTOR AND THEIR ENGINEER OF CONTRACT
RESPONSIBILITY FOR ANY ERROR OR DEVIATION
CONTRACT DOCUMENTS AND BUILDING CODE REQUIREMENTS

AAI ENGINEERING

Date 02/08/11 By Derek Larsen

DESIGN PARAMETERS **IBC 2009 with OSSC 2010 Amendments**

PEDESTRIAN PROTECTION LOAD	150 psf

CONTENTS

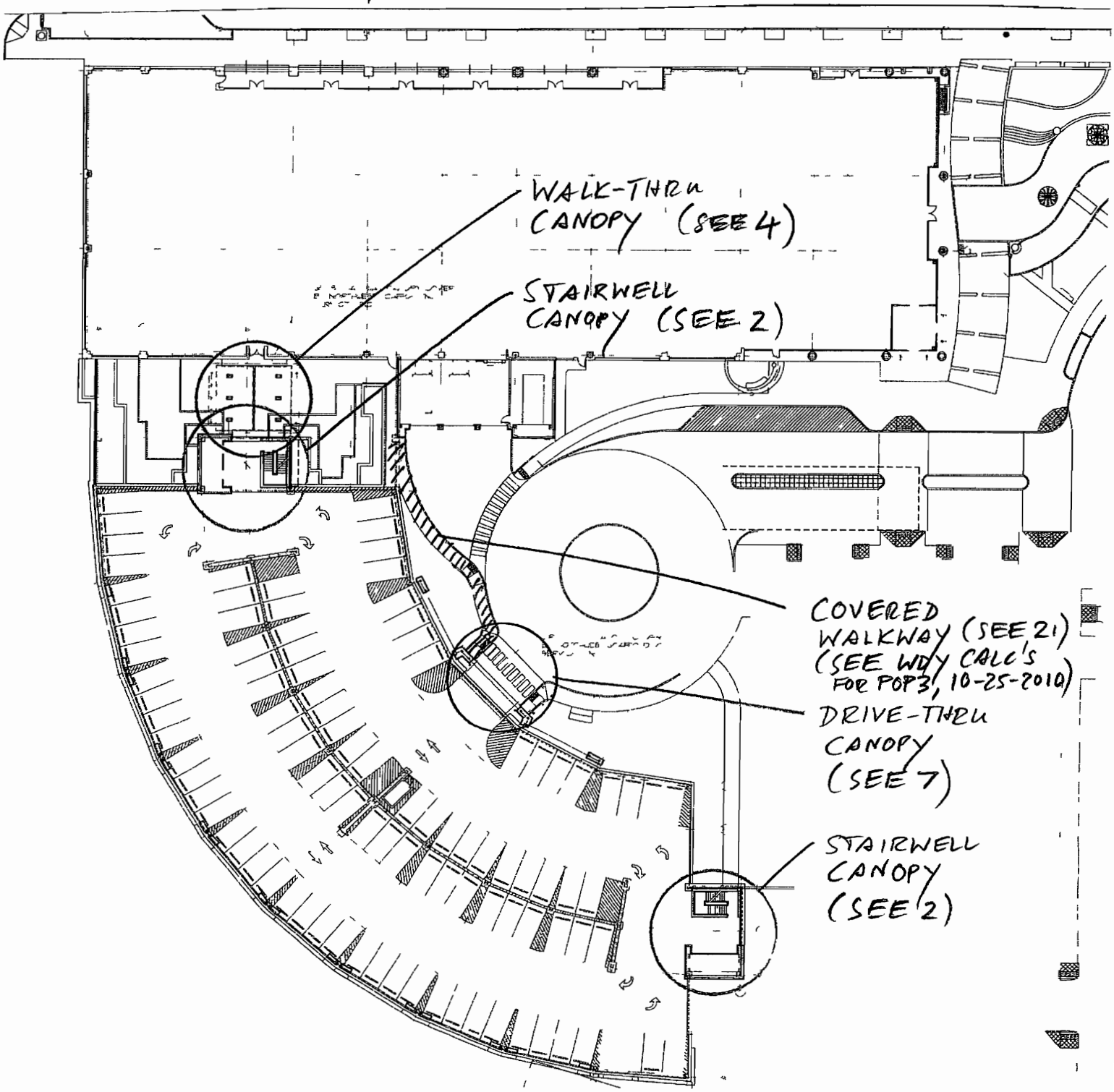
CANOPY DESIGN	1 to 29

6443 SW Beaverton Hillsdale Hwy suite 210 Portland OR 97221 • ph 503 203 8111 • fx 503 203 8122 • www wdyi com

WDY **Structural • Civil Engineers**

Job Name	**Providence POP2 Construction Protection Canopies**	Job No	**10723**	Sheet No	**1**
Client	**Northwest Scaffold Service Inc**	Date	**Nov 2010**	By	**mw**

PARTIAL SITE PLAN, N.T.S.



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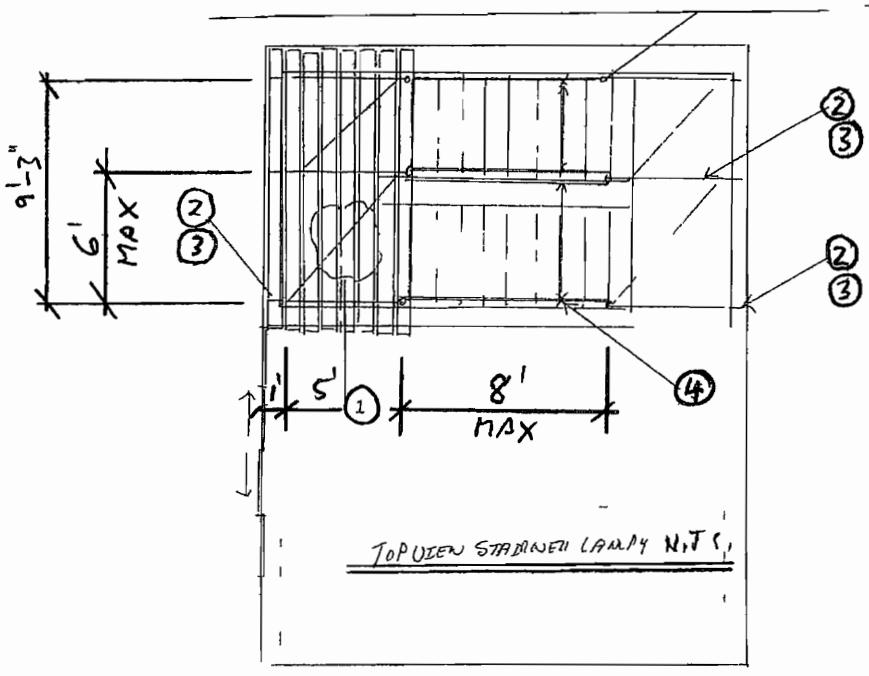
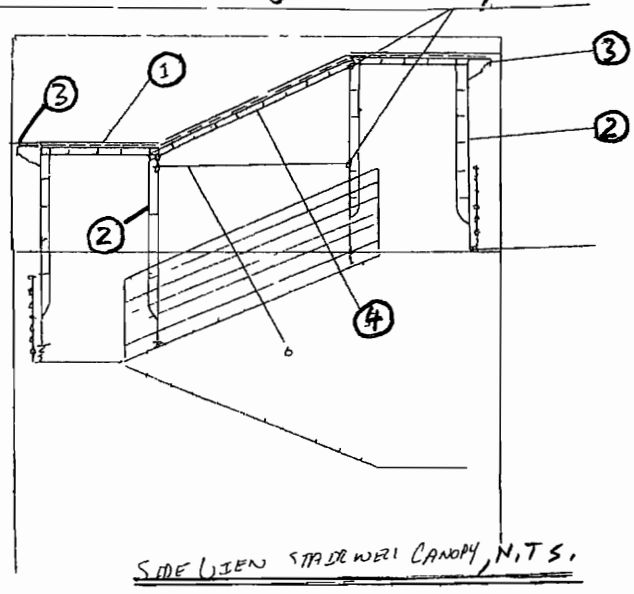
TYP. STAIRWELL CANOPY,

- PEDESTRIAN PROTECTION LL = 150 PSF, (USE S.F. = 1.33 IN CONJUNCTION W/ 150 PSF)



TOP VIEW STAIRWELL CANOPY, N.T.S.



SIDE VIEW STAIRWELL CANOPY, N.T.S.

- DESIGN,

(1) $1\frac{1}{2}$" x $9\frac{1}{2}$" MICRO-LAM PLANKING W/ $\frac{5}{8}$" CDX PLYWOOD, MAX SPAN = 6'

$\Rightarrow$ MAX. W = [(4 + 1.8) + (150)] = 155.8 PSF < 164.2 PSF = 130 PLF $\left(\frac{12}{9.5}\right)$

(PLWD + PLANKING) (SEE 9) (OK)

(2) STEPUP SCAFFOLD FRAME TF6N-5W-L5,

$\Rightarrow$ MAX. LEG LOAD = $1558\left[\left(\frac{5}{2}\right)+\left(\frac{8}{2}\right)\left(\sim\frac{10}{2}\right)\right]$ = 5,064#

< 14,565# = 4,843# $\left(\frac{4.0}{1.33}\right)$ (OK)

(SEE 10-12, TF6N-5W-L3 HAS TUBE OD = 1.625" < 1.69" = TUBE OD FOR TF6N-5W-L5)

TYP. STAIRWELL CANOPY, DESIGN, CONT.,

(3) STEPUP 12" SIDE BRACKET SBCP-12,

$\Rightarrow$ MAX. UNIFORM LOAD = $155.8(1)(6) = 935^{\#}$

$< 1{,}844^{\#} = 613^{\#}\left(\frac{4.0}{1.33}\right)$ (OK)

(SEE 13,14 SB20-SN IS 20" BRACKET > 12" BRACKET = SBCP-12)

(4) STEPUP PUTLOG TRUSS P-12, MAX. SPAN = 8'

$\Rightarrow$ MAX. $w = 155.8\left(\frac{9.25}{2}\right) = 721$ PLF

$\Rightarrow$ ALL. UNIFORM LOAD FOR P-12 PER STEPUP (SEE 15)

$= 1{,}350^{\#} \triangleq 112.5$ PLF $= \frac{1{,}350}{12}$

$\Rightarrow$ EQIVALENT $M = \frac{112.5(12)^2}{8} = 2{,}025$ FT-#

$\Rightarrow$ EQUIV ALL. w FOR 8' SPAN $= \frac{2{,}025(8)}{8^2} = 253$ PLF

$\Rightarrow$ ADJUST FOR S.F = 1.33: $253\left(\frac{4.0}{1.33}\right) = 761$ PLF > 721 PLF

(OK)

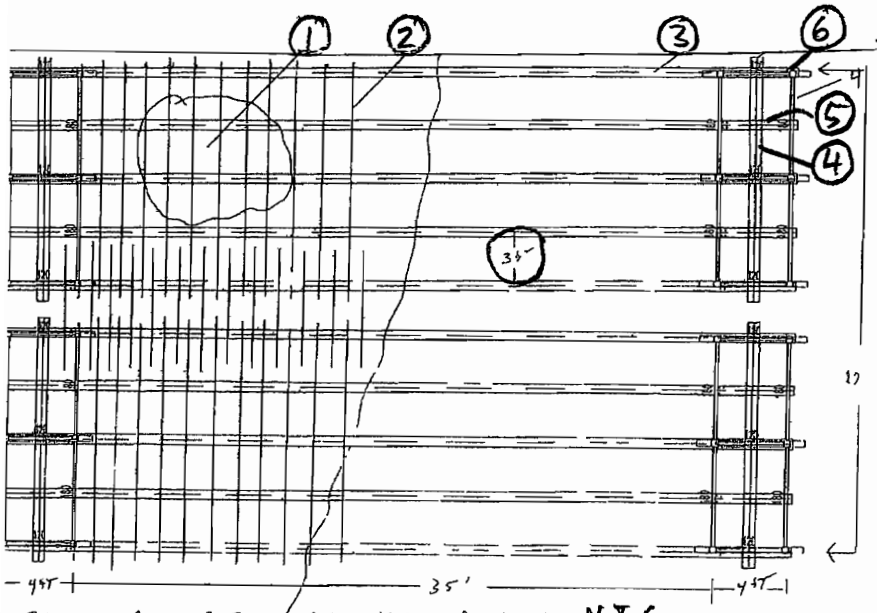
(5) STABILITY,

SCAFFOLD FRAMES ARE CONNECTED TO (E) STAIRWELL RAILING WITH STEPUP CLAMP-S

WALK-THRU CANOPY,

– LOADS: DECKING – PEDESTRIAN PROTECTION LL = 150 PSF

SUPPORT STRUCTURE – 25 PSF SNOW LOAD



TOP VIEW COVER BETWEEN POP 2 OFFICE + PARK DOG, N.T.S

– SNOW DRIFT:

$P_g = 20$ PSF ; $L_u \cong 111'$

$\Rightarrow h_d = 3.2'$, $W_d = 4 \times 3.2 = 12.8'$

$P_d = 3.2\left(\frac{13}{20} \times 20 + 14\right) = 53$ PSF

(USE S.F. = 2.5 IN CONJUNCTION W/ SNOW LOAD FOR SUPPORT STRUCTURE)

– DESIGN,

① $\frac{5}{8}$" CDX PLYWOOD, CLEAR SPAN = 16" – 5" = 11"

$\Rightarrow$ MAX. W = 150 PSF = 150 PSF (OK)

(PER APA DESIGN/CONSTRUCTION GUIDE "RESIDENTIAL & COMMERCIAL" – $\frac{5}{8}$" SHEATHING W/ FLOOR JOISTS @ 16" O.C)

② SYMONS ALUMINUM BEAMS (AB) @ 16" O.C., MAX SPAN = 6'

$\Rightarrow$ MAX. $W = \left[\left(1.8 \times \frac{16}{12} + \sim 5\right) + \left(150 \times \frac{16}{12}\right)\right] = 207.4$ PLF

WALK-THRU CANOPY, DESIGN, (2) ALUMINUM BEAMS, CONT

$\Rightarrow$ MAX. $M = \frac{207.4(6)^2}{8} = 933.3$ FT-# $< 11{,}100$ FT-# = ALL. M (OK)

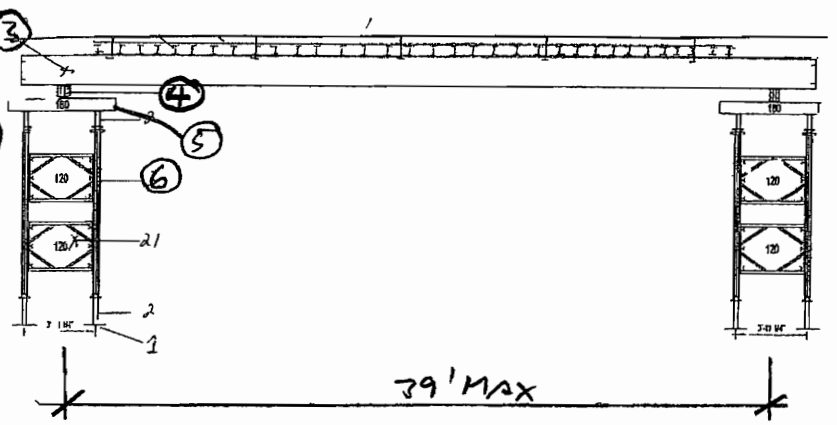
$\Rightarrow$ MAX. $R_{END} = 207.4\left(\frac{6}{2}+\sim 1\right) = 829.6^{\#} < 4{,}100^{\#} = ALL.\ R_{END}$ (OK)

$\Rightarrow$ MAX. $R_{INT} = 207.4(1.25\times 6) = 1{,}556^{\#} < 8{,}200^{\#} = ALL.\ R_{INT}$ (OK)

(SEE 16)

(3) W21x44 @ 3'0.C.,

(PROVIDED BY SYMONS, F_y = 36 KSI)



END VIEW COVER BETWEEN POP2 VALLEY & PARKING, N.T.S

$\Rightarrow$ MAX $w = \left[(1.8+5)(3) + 44\right] + \left[\left(20+\frac{3}{4}(53)\right)(3)\right] = 244$ PLF

- $(1.8+5)(3)$ → PLYWOOD & ALUMINUM STRINGERS
- 44 → BEAM SELF WEIGHT
- $\left(20+\frac{3}{4}(53)\right)(3)$ → SNOW DRIFT / FLAT ROOF SNOW } > 25PSF

$\Rightarrow$ MAX. $M = \frac{244(39)^2}{8(1{,}000)} = 46.4^{k}\text{-FT} < 163^{k}\text{-FT} = ALL.\ M$ (F_y = 36 KSI) (OK)

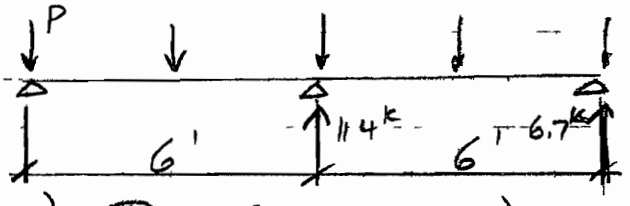
$\Rightarrow$ MAX $R = \frac{244(39)}{2(1{,}000)} = 4.76^{k}$

$\Rightarrow$ SPLICES @ 10' FROM SUPPORT. $M_{10'} \cong 35.4^{k}\text{-FT} < 68.8^{k}\text{-FT}$ (OK)
$V_{10'} \cong 2.3^{k} < 41^{k}$ (SEE 17)

WALK-THRU CANOPY, DESIGN, CONT,

(4.) DBL. TITAN 225 SUPPORT BEAM,

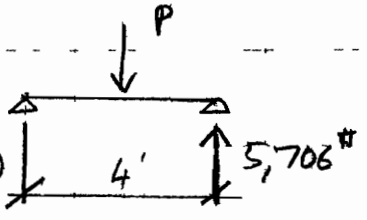
$P = 4.76^{k}$



$\Rightarrow$ MAX $M^{+} = 5,873$ FT-# $< 21,020$
MAX. $M^{-} = 5,419$ FT-# $< 21,020$
MAX. $R_{END} = 6,726^{\#} < 20,000^{\#}$
MAX. $R_{INT} = 11,411^{\#} < 20,000^{\#}$
} (OK) (SEE 18)

(5.) SINGLE TITAN 225 CROSS-BEAM,

$P_{MAX} = 11,411^{\#}$



$\Rightarrow$ MAX. $M^{+} = \frac{11,411\,(4)}{4} = 11,411$ FT-# $< 21,020$ (OK)

MAX. $R = \frac{11,411}{2} = 5,706^{\#} < 20,000^{\#}$ (OK) (SEE 18)

(6) TITEN ISCHEBECK POST SHORES,

MAX. $H \cong 18'$

MAX. $P \cong 5,706^{\#} < 11,600^{\#}$ (OK) (SEE 19)

(7.) ~5" x 5" BASE PLATE ON 2x12 WOOD PAD,

$$f_{c\perp} = \frac{1}{5^2}\left[\underbrace{5,706}_{\text{SEE ABOVE}} + \underbrace{5,94\left(2\times 1.25\times 6 + \tfrac{1}{2}\times 6\right)}_{\text{TITAN 225 BEAMS}} + \underbrace{\sim 70}_{\text{SHORE}} + \underbrace{\tfrac{1}{2}(2\times 15 + 2\times 23}_{\text{FRAMES}}\right.$$

$= 237$ PSI (OK)

WDY Structural • Civil Engineers

Job Name	Providence POP2 Construction Protection Canopies	Job No	10723	Sheet No	7
Client	Northwest Scaffold Service Inc	Date	Nov 2010	By	mw

DRIVE-THRU CANOPY,

- LOADS DECKING - PEDESTRIAN PROTECTION LL = 150 PSF
  SUPPORT STRUCTURE - 25 PSF SNOW LOAD
  - SNOW DRIFT

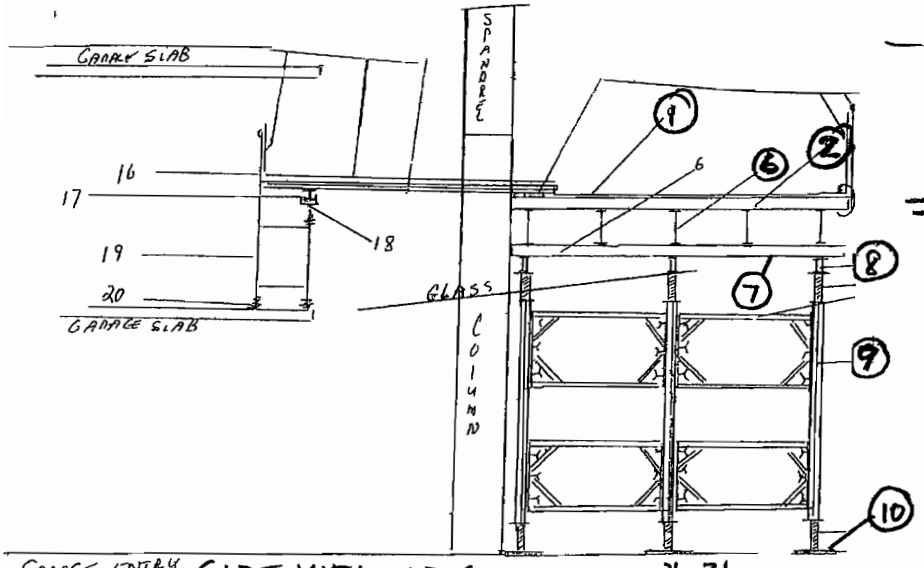
$P_G = 20$ PSF ; $e_u \cong 13.5'$

$\Rightarrow \frac{3}{4} h_d = 75 \times .8 = .6'$

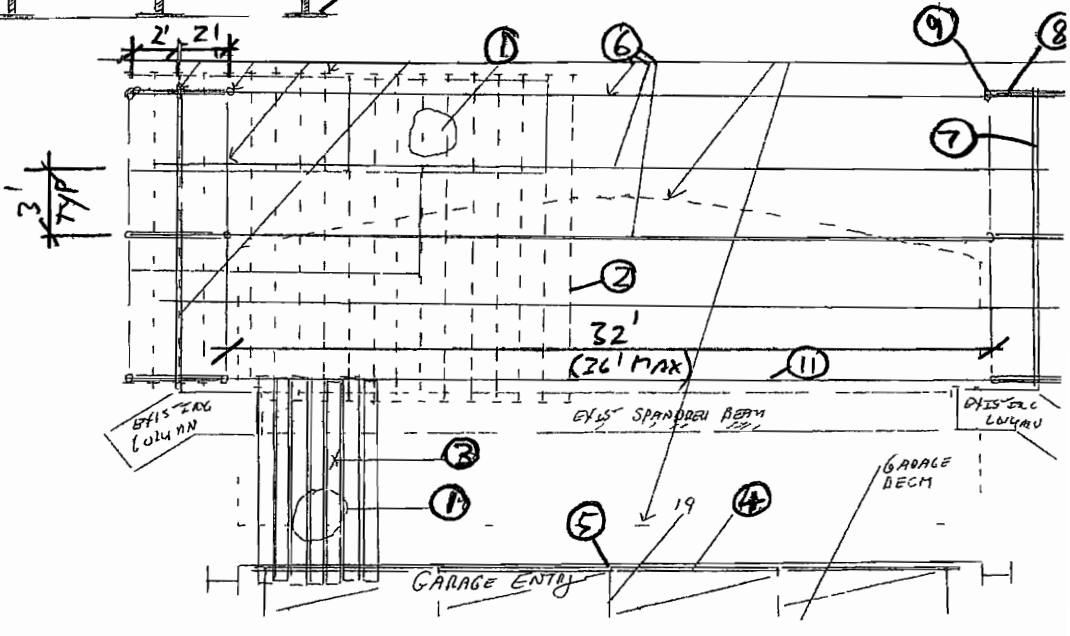
$W_d = 4 \times .6 = 2.4' < 12.8'$

$p_d = 6(.13 \times 20 + 14) = 10$ PS < 53 PS

$\Rightarrow$ USE SNOWDRIFT PER SHEET 4



GARAGE ENTRY SIDE VIEW, N.T.S.



PLAN VIEW GARAGE ENTRY CANOPY, N.T.S

- DESIGN,
  1 AND 2, SEE WALK-THRU CANOPY, ITEMS 1 AND 2

DRIVE-THRU CANOPY, DESIGN, CONT,

(3) DOUBLE $1\frac{1}{2}" \times 9\frac{1}{2}"$ MICRO-LAM PLANKING, MAX. SPAN = 9'

$\Rightarrow$ MAX. $W = [(4\times2+1.8)+(150)] = 159.8$ PSF < 455.9 PSF (OK)
$= 2 \times 60$ PSF $\left(\frac{12}{9.5}\right)\left(\frac{4.0}{1.33}\right)$ (SEE-9)

(4) SYMONS ALUMINUM BEAMS (AB) OR W8x10, MAX. SPAN = 7

$\Rightarrow$ MAX. $W = 159.8\left(\frac{9}{2}\right) = 719.1$ PLF

$\Rightarrow$ MAX $M = \frac{719.1(7)^2}{8} = 4,404$ FT-# $< 11,100$ FT-# = ALL. M (OK)
$< 16,000$ FT-# (W8x10)

$\Rightarrow$ MAX. $R_{END} = 719.1\left(\frac{7}{2}+\sim1\right) = 3,236^{\#} < 4,100^{\#} =$ ALL. R_{END} (OK)

$\Rightarrow$ MAX. $R_{INT} = 719.1\,(1.25\times7) = 6,292^{\#} < 8,200^{\#} =$ ALL. R_{INT} (OK)
(SEE 16)

(5) STEPUP SCAFFOLD FRAME NF3H-2W-L3

$\Rightarrow$ MAX. LEG LOAD $= 6,292^{\#}$ (SEE ITEM (4))
$< 14,565^{\#} = 4,843^{\#}\left(\frac{4.0}{1.33}\right)$ (OK)
(SEE 10, 11, 20, TF6H-5W-L3
HAS TUBE OD $= 1.675" < 1.69"$ =
TUBE OD FOR NF3H-2W-L3)

(6) THRU (10), SEE WALK-THRU CANOPY, ITEMS (3) THRU (7)

DRIVE-THRU CANOPY, DESIGN, CONT.,

(11) CHECK W21x44 FOR ADDED PLANKING,

SPAN = 36'

$$W \cong \Big[\underbrace{44}_{\text{SELFWEIGHT}} + \underbrace{(1.8+5)\left(\tfrac{3}{2}+1\right)}_{\text{PLWD + ALU STRINGERS}} + \underbrace{(4\times2+1.8)\left(\tfrac{9}{2}+1\right)}_{\text{DBL. PLANKING + PLWD}}\Big] + \Big[\underbrace{\left(20+\tfrac{1}{2}\left(\tfrac{8.3+2.3}{12.8}\right)(53)\right)}_{\text{FLAT SNOW}}\underbrace{6}_{\text{SNOW DRIFT}}\Big]$$

= 367 PLF

$$\Rightarrow \text{MAX. } M = \frac{367(36)^2}{8(1{,}000)} = 59.5^k\text{-FT} < 163^k\text{-FT} = \text{ALL. M}$$

(F_y = 36 KSI) OK

$$\Rightarrow \text{MAX. } R = \frac{367(36)}{2(1{,}000)} = 6.61^k$$

SPLICE OK (SEE SHEET 5)

⇒ CHECK SINGLE TITAN 225 CROSS-BEAM,

$$P \cong 6.7^k + (6.61 - 4.76) = \underline{8.6^k} < 11.4^k \text{ (SEE 6)}$$ OK

↳ SEE SHEET 6, ITEM (4.) $P = 4.76^k$

→ SEE ABOVE, $R = 6.61^k$

→ SEE SHEET 6, EXT REACTION = 6.7^k

9

SUPERIORITY IN WOOD TECHNOLOGY

MICRO=LAM is a laminated wood plank that's tough, reliable and stiff In fact, in deflection it's 29% stiffer than 2050 dense, select structural Douglas Fir That means important additional stability

MICRO=LAM scaffold planks are available in widths from 10" to 24" and con tinuous lengths to 80' in three thicknesses (1 1/2", 1 3/4" & 2 1/2") MICRO=LAM scaffold planks are cut from billets in any length you require without a pricing penalty

MICRO=LAM, Scaffold Plank Clear Span Load Table

SIZE 1-1/2"x 9 1/2"

Span Ft	4	5	6	7	8	9	10	11	12
Load Lbs/Ft	300	190	130	100	70	60	50	40	30
Conc Load @ Mid Span Lbs	600	480	400	340	300	260	240	220	200
Uniform Load Deflection In	35	55	80	1 1	1 4	1 8	2 3	2 8	3 2

SIZE 1 3/4"x11 3/4"

Span Ft	6	7	8	9	10	11	12	13	14
Load Lbs/Ft	220	160	120	100	80	70	60	50	40
Conc Load @ Mid Span Lbs	670	570	500	440	400	360	330	310	290
Uniform Load Deflection In	68	94	1 2	1 5	2 0	2 3	2 8	3 3	3 7

SIZE 2-1/2"x11 3/4"

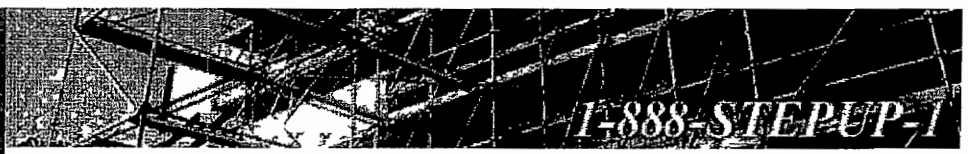
Span Ft	12	13	14	15	16	17	18	19	20
Load Lbs/Ft	110	100	80	70	65	55	50	45	40
Conc Load @ Mid Span Lbs	680	630	580	540	510	480	450	430	410
Uniform Load Deflection In	1 9	2 3	2 6	3 1	3 5	3 9	4 2	4 8	5 4

These tables are for untreated MICRO=LAM

STEPUP

STEPUP SCAFFOLD

The quality source for all your scaffolding needs



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Testing

CHEMICAL ANALYSIS

		Q345 E			AISI 512	
Element		Result	Min %	Max %	Min %	Max %
C	=	0.13		0.18	0.10	0.16
Mn		1.20		1.60	1.10	1.40
P	-	[illegible]		0.025	0.000	0.040
S	-	[illegible]		0.025	0.000	0.0[illegible]

The [illegible] ASTM A29

TENSILE TEST RESULTS

Specimen Dimensions OD x Wall, in	Area Sq ins	Yield Strength psi @ 0.2% Offset	Total Load lbs	Tensile Strength psi
1.701 x 0.000	0.455	67 600	35 525	78 100
Requirement	---	60 000 min	---	70 000 min

Tensile [illegible]

SINGLE TIER COMPRESSIVE LOAD TEST

Load at Yield	Load per Leg	Per Leg Load at 4:1 Safety Factor
77 500 lbs	19 375 lbs	4 843 lbs

Pre load

Yield at 77 500 lbs

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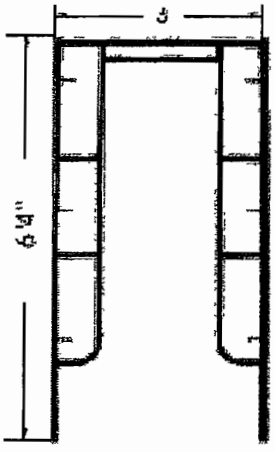
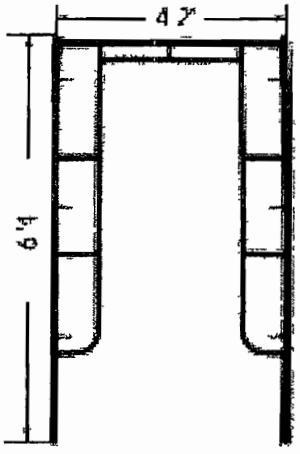
Frames

THROUGH FRAMES

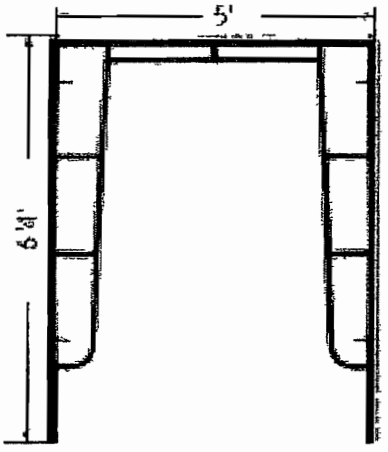
DROP-LOCK THROUGH FRAMES



TF6H-3W-L3



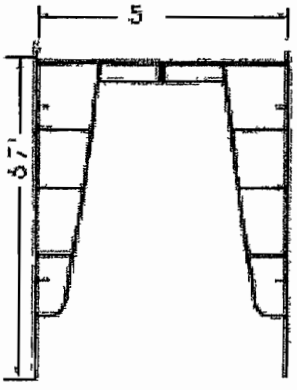
TF6H-4'2W-L3



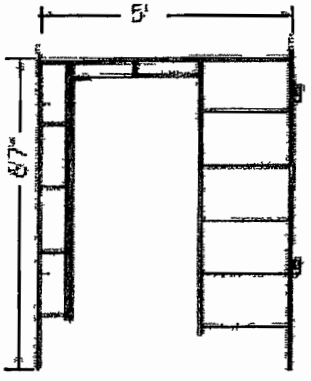
TF6H-5W-L3

MIGHTY THROUGH FRAMES

- Tube OD 1.625"
- 13 Gauge Tube
- Manganese Steel
- Powder-Coated
- Minimum Yield Strength 50,000 PSI
- Minimum Tensile Strength 65,000 PSI



TF6H-5W-L2-M



TF6H-5W-M

THROUGH FRAMES

DROP-LOCK THROUGH FRAMES

Item No.	Description	Weight (lbs)
TF6H-3W-L3	6'4"H x 3' W Walk Through Frame with Drop Lock	41.25
TF6H-4'2W-L3	6'4"H x 4'2" W Walk Through Frame with Drop Lock	42.80
TF6H-5W-L3	6'4"H x 5' W Walk Through Frame with Drop Lock	47.55

MIGHTY THROUGH FRAMES

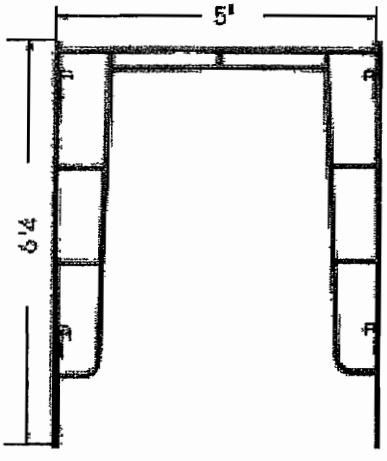
Item No.	Description	Weight (lbs)
TF6H-5W-L2-M	5' x 6'7" Walk Thru Frame Flip Lock Mighty Style	48
TF6H-5W-L2-M	5' x 6'7" Walk Thru Frame w/22" Ladder Mighty Style	53

STEPUP

Frames

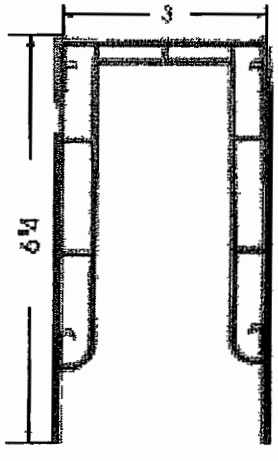
THROUGH FRAMES

V-LOCK THROUGH FRAMES

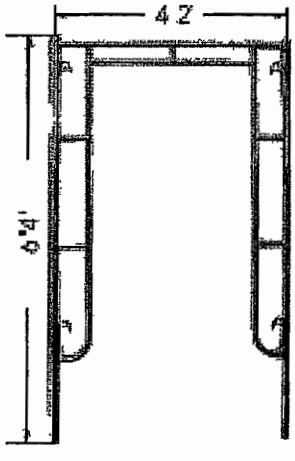


TF/- -W -5

- Tube OD 1.69"
- 13 Gauge Tube
- Manganese Steel
- Powder Coated
- Minimum Yield Strength 50,000 PSI
- Minimum Tensile Strength 65,000 PSI



TF6H- -W -5



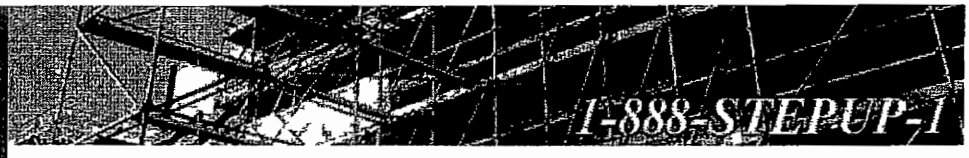
TF6H-42 V-L5

THROUGH FRAMES			
V-LOCK THROUGH FRAMES			
Part No.	Description	Steel Center	Weight (lbs)
TF6H-3W-L5	6'6"H X 3'W with V-Lock	5'	44
TF6H-42W-L5	6'6"H X 4'2"W with V-Lock	5'	46
TF6H-5W-L5	6'6"X5'W with V-Lock	5'	49
TF7H-6W-L5	Sidewalk Canopy Frame with V-Lock	2' 4'	57

STEPUP

STEPUP SCAFFOLD

The quality source for all your scaffolding needs



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Testing

CHEMICAL ANALYSIS

			Q215 A,B		Q235 A		AISI 1513	
Element		Result %	Min %	Max %	Min %	Max %	Min %	Max %
C		0.15	0.09	[illegible]	---	0.16	0.10	0.16
Mn	-	1.20	0.25	[illegible]	0.50	1.50	1.10	1.40
P	=	0.020	---	[illegible]	[illegible]	0.045	0.000	0.040
S		0.017		[illegible]	[illegible]	0.045	0.000	0.050

[illegible]

TENSILE TEST RESULTS

Specimen Dimensions OD x Wall In	Area [illegible]	[illegible]	[illegible]	[illegible]	% El (2")
1.691 0.090	0.453	75,700	37,722	82,600	17
Requirement	[illegible]	[illegible]	[illegible]	70,000 [illegible]	--

[illegible]

LOAD TEST RESULTS
SB20 SH

Test Sample#	Failure Mode	Yield Load (lbs)	Load with 1:1 Safety Factor (lbs)
1	Deformation of V-Plate (saddle)	2,[illegible]50	614
2	Deformation of V-Plate (saddle)	2,575	644

SB20-SH at load of 2,[illegible]50 lbs

[illegible] at load of [illegible]575 lbs

SB20-SH at load of [illegible],575 lbs

Configuration for testing side brackets

Test set-up for SB20-SH type bracket

Several types of scaffold frames were used as the supporting structure.
The type of frame for test purposes had no bearing on resulting loads of side brackets

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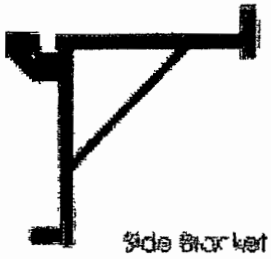
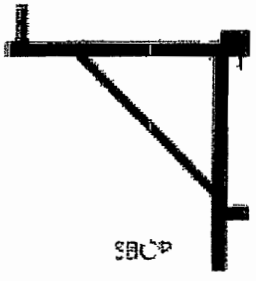
ACCESSORIES

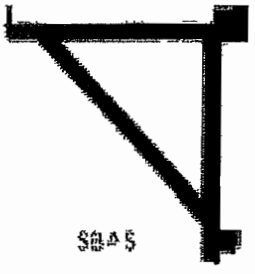
SIDE BRACKETS



Side Bracket

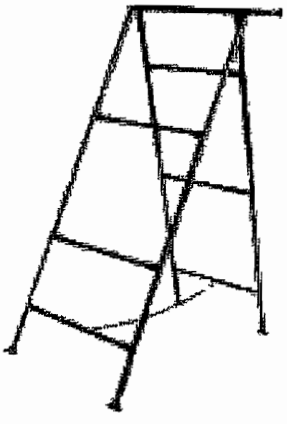


SBCP

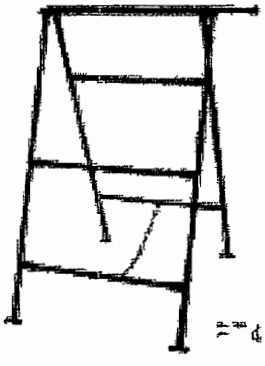


SBAS

FOLDING TRESTLE

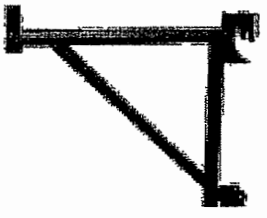


FT6



FT4

END BRACKETS



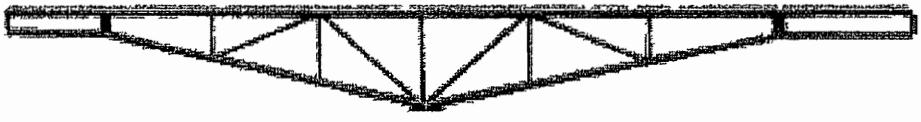
End Bracket

ACCESSORIES		
END BRACKETS		
EB20-HH	20" End Bracket with Hook Hanger	13
EB20-HHH	40" End Bracket with Hook Hanger	16
SIDE BRACKETS		
SB20-SH	20" Side Bracket w/Saddle Hanger	16
SB20-HH-NY	20" Side Bracket with Hook Hanger, New York Style	14
SB20-SH	20" Side Bracket w/Saddle Hanger	10
SB20-HH-H	20" Side Bracket with Hook Hanger, Houston	6.65
SBCP-12	12" Side Bracket with Coupling Pin	5.65
SBCP-20	20" Side Bracket with Coupling Pin	12
SBCP-24	24" Side Bracket with Coupling Pin	15
SBCP-36	36" Side Bracket with Coupling Pin	25
SBAS-20	20" Angle Iron Side Bracket with Stud	11
SB20-SH-M	20" Side Bracket, Mighty Style	9.20
FOLDING TRESTLE		
FT2	2' Folding Trestle	15
FT3	3' Folding Trestle	19
FT4	4' Folding Trestle	25
FT6	6' Folding Trestle	41
FTE2	2' Folding Trestle Extension	33
FTE3	3' Folding Trestle Extension	47

PUTLOG & ACCESSORIES

PUTLOG

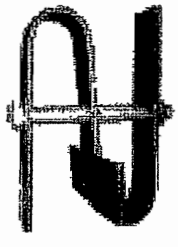
Putlog (P8 P12)



Deep Truss Putlog (P16 P22

PUTLOG ACCESSORIES

PRAH



PPH

PUTLOG ACCESSORIES		
Item No.	Description	Weight (kg)
PPH	Putlog Parallel Hanger	4.30
PRAH	Putlog Right Angle Hanger	5.05

PUTLOG					
Item No.	Description	Load*	Load**	Weight (kg)	
P8	8' Putlog	1900	1000	28	
P12	12' Putlog	1900	1000	41	
P16	16' Putlog Deep Truss	2250	1250	54	
P22	22' Putlog Deep Truss	2250	1250	68	

*B = Uniform Load MAX

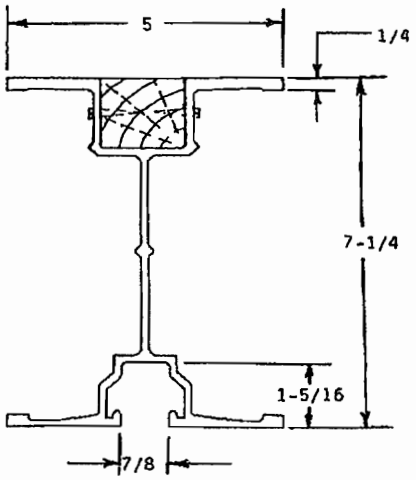
**C = Approved load concentrated in centre

ALUMINUM BEAM (AB) PROPERTIES and DESIGN LIMITS

The following Aluminum Beam properties and allowable limits may be used in design calculations for support and loading conditions which are not tabulated

PROPERTIES

Ixx	= 29 20 in (4)
Iyy	= 4 37 in (4)
Rxx	= 2 92 in
Ryy	= 1 13 in
Area	= 3 428 in ²
MOE	= 10 x 10 (6) psi
Alloy/Temper	= 6061-T6



ALLOWABLE LIMITS FOR DESIGN

End Reactions Simple or Multiple Supports without Cantilevers
R = 4100 lbs with 2" min bearing length
R = 5900 lbs with 4" min bearing length

With 6 in Min Cantilever Past Centerline of Support
R = 7400 lbs with 2" min bearing length
R = 7700 lbs when supported by another AB or AS

Intermediate Reactions
R = 9700 lbs with 8" min bearing length (two or three span condition)
R = 8200 lbs with 4" min bearing length (two or three span condition)
R = 9500 lbs with 4" min bearing length (two or three span condition)

Moment
M = 11,100 ft lbs for uniform loading
For AB used as a header (heavy concentrated load), consult a Regional Engineer for lateral stability requirements

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FX (909) 594-8253

December 21, 2010

Attn Terry Morrow
Masons Supply

RE W21x44 Roller Deck Beam Splice

Dear Mr Morrow

The purpose of this letter is to state the allowable loads that the butt splice connection in the Symons W21x44 roller deck beam can hold When using (12) ¾" A-325 speed bolts in the splice, and the shear load in the splice is 41 kips or less, the allowable moment in the splice is 68 8 kip-ft The table below shows the allowable moment for a given shear load in the splice The maximum allowable shear load in the splice is 90 kips

Shear (kips)	Allowable Moment (kip-ft)
41	68 8
50	66
60	62
70	57 5
80	52
90	43

¾" A-490 bolts may be substituted for the ¾" A-325 speed bolts in the splice
If you have any questions, please feel free to contact me at (909) 594-8201 Ext 14350

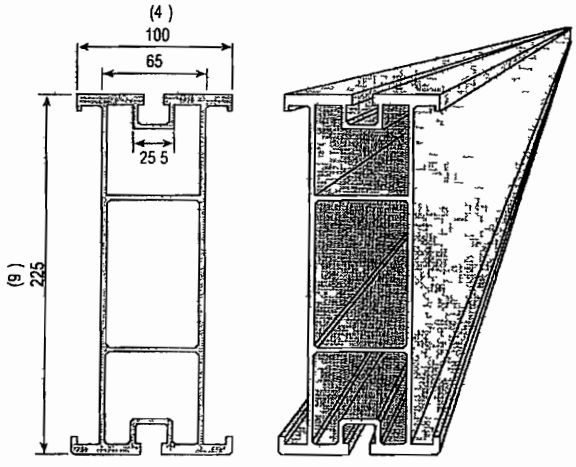
Respectfully,

Kirk J Tatman, P E
Regional Engineer
Symons by Dayton Superior

MEGASHORE beams

ISCHEBECK®
TITAN

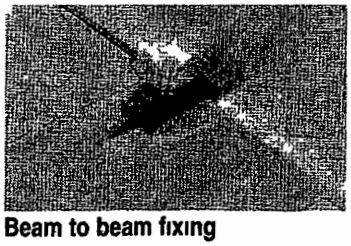
Manufactured from extruded aluminium section, MEGASHORE beams offer a lightweight, easy to handle solution for decking/soffit applications
All secondary beams are shown color coded in Ischebeck design drawings



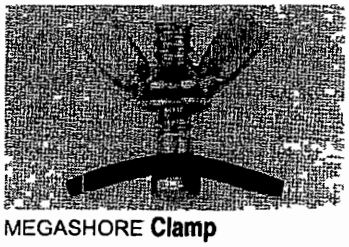
MEGASHORE **225 beam**

	Area	5 057 in²
Ixx	Moment of inertia	54 00 in⁴
Zxx	Section modulus	12 16 in³
E	Modulus of elasticity	9 799 883 lbs/in²
E I	Bending stiffness	31 623 lbs/ft²
W	Weight	5 94 lbs/ft
M	Resistance moment	21 020 lbs ft
S	Allowable shear force	20 000 lbs

length (m + feet)	length (m +feet)
1 2 = 3' 11	8 0 = 26 3
1 8 = 5 -11	9 0 = 29 6
2 4 = 7 -11	10 0 = 32 10
3 0 = 9'-10"	11 0 = 36 1
3 6 = 11 10	
4 2 = 13' 9"	
4 8 = 15 9	
5 4 = 17 9	
6 0 = 19 8	
6 4 = 21	
7 2 = 23'-7"	



Beam to beam fixing



MEGASHORE **Clamp**

Headplate to beam fixing

Megashore Loading Graphs

The allowable leg load is determined by the floor to soffit height, the number of ledger frames in height or jack extension

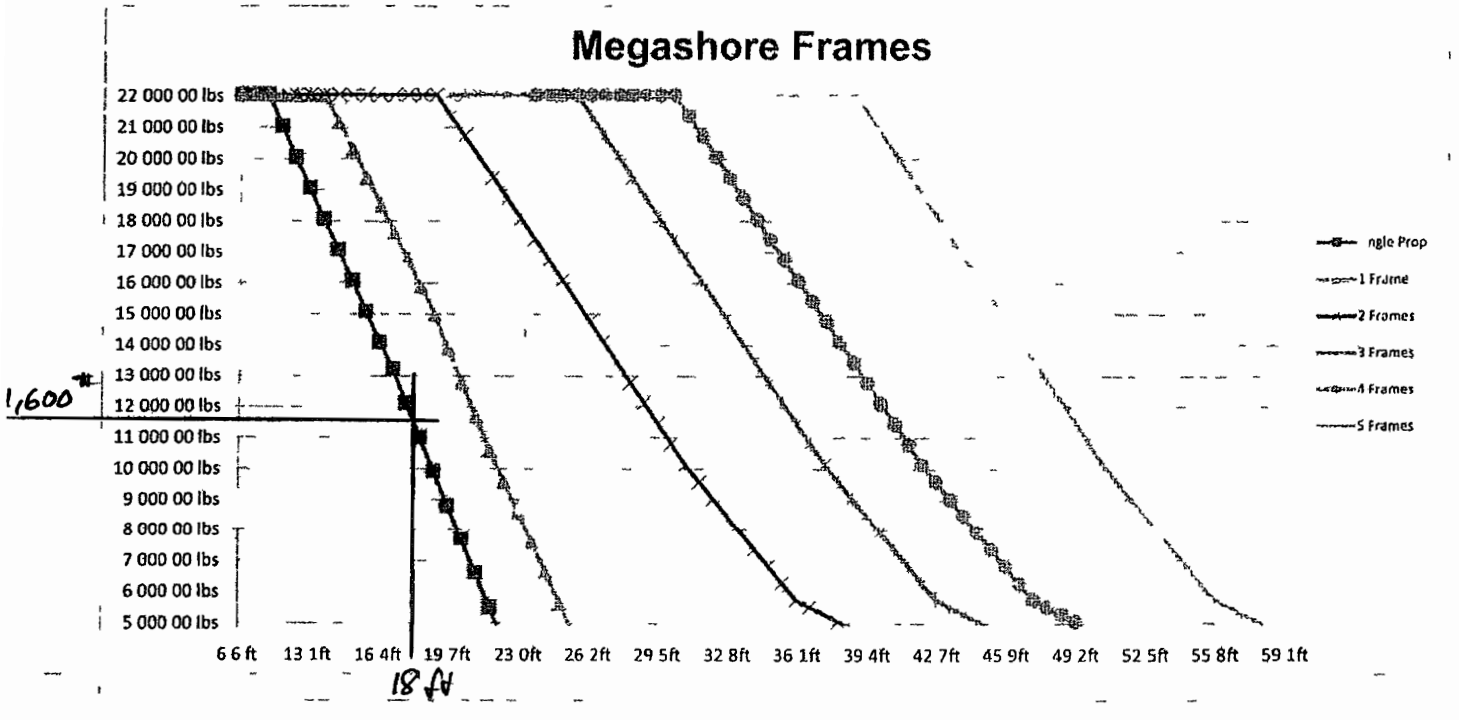
Notes
It is assumed the formwork is restrained from horizontal movement at the formwork level

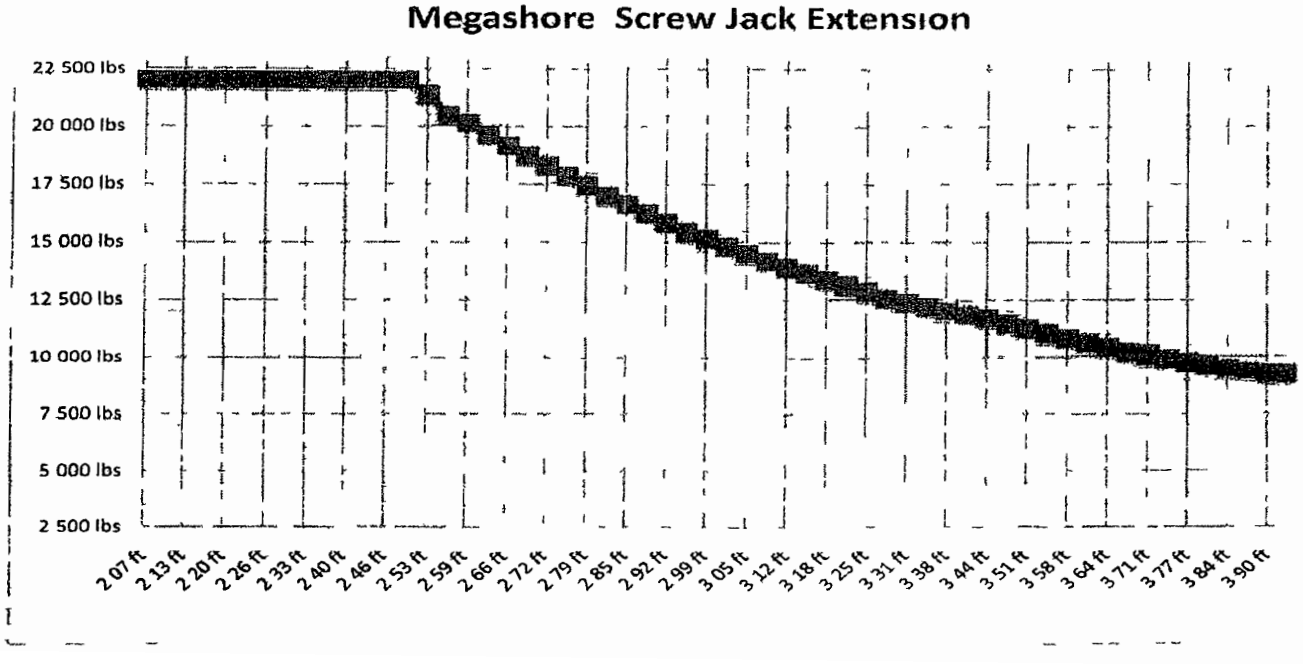
Ledger frames to be fixed in the optimum position

Maximum distance between ledger frames not to exceed 6'-6" between top and bottom chords of ledger frames This rule also applies on floor to soffit heights greater than 36'

On shoring heights over 36 ft refer to Ischebeck Design Services for advice On the higher shoring heights an additional level of frames may be required for ease of erection and stability

Minimum height to base ratio = 4 1





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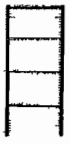
NF3H 2W-L3



NF5H-2W L3

NF6H 2W-L3

Page 1 of 1 Go to page 1

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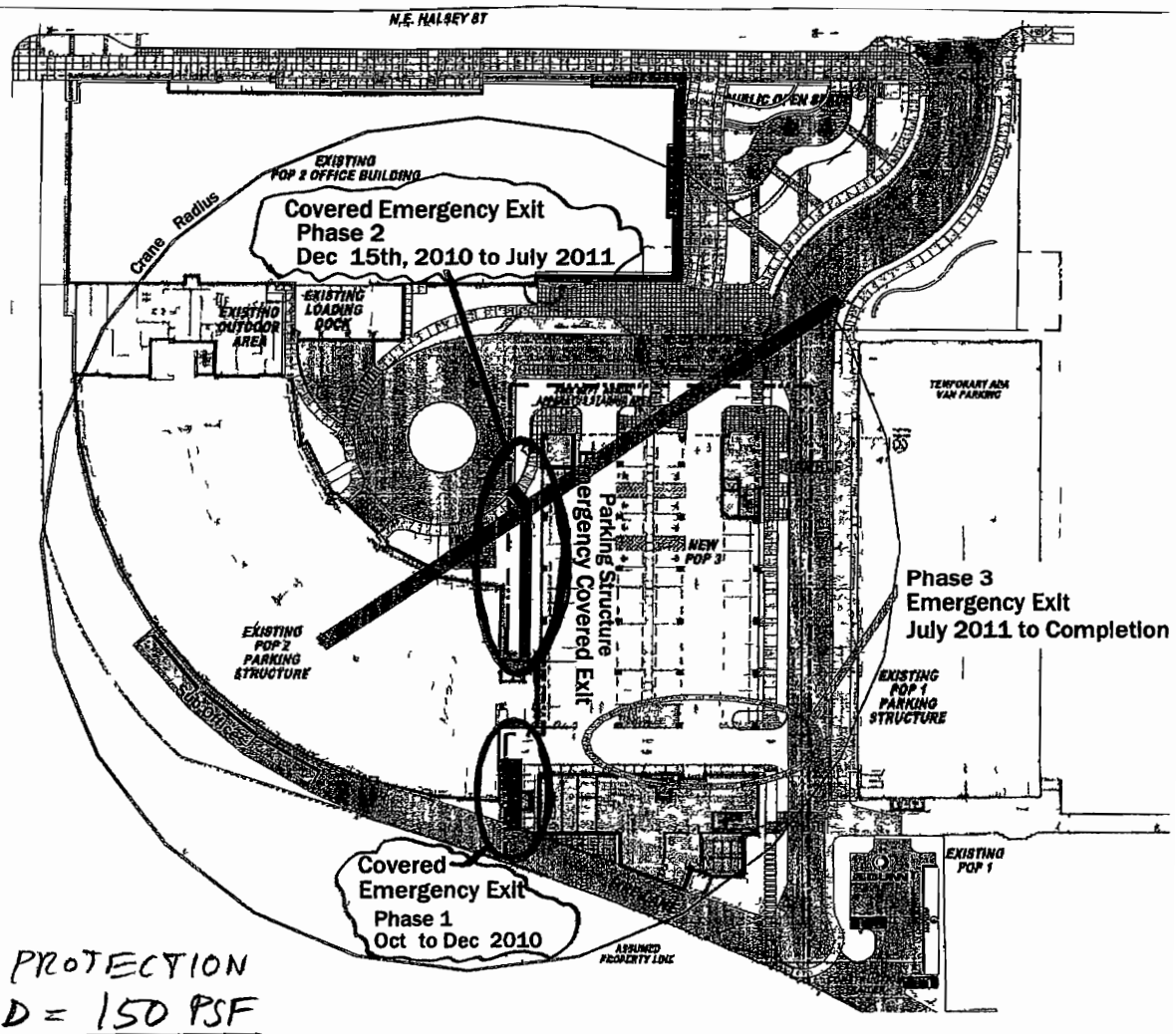
Quality Service Satisfaction

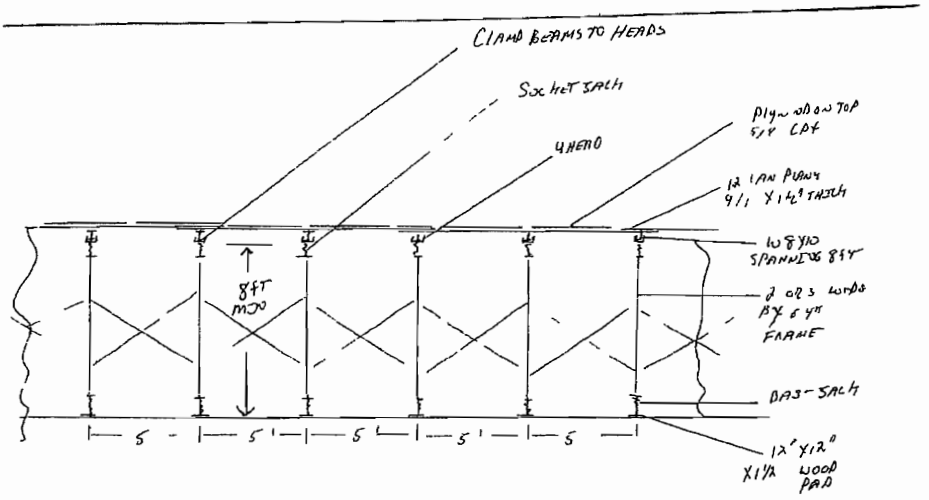
Job Name	Providence POP3 Pedestrian Protection Canopies	Job No	10722	Sheet No	21
Client	Northwest Scaffold Service Inc	Date	Oct 2010	By	mw

TRY:
(2) 2' WIDE x 6'-4" SCAFFOLD FRAMES W/ W8x10 (SPANNING 8') @ 5' O.C W/ 1½" x 9½" MICRO-LAM PLANKING AT COVERED EMERGENCY EXITS (PH 1) AND (PH 2) (SEE PLAN)



AREA PLAN, N.T.S.

⇒ PEDESTRIAN PROTECTION DESIGN LOAD = 150 PSF



TYP SIDE ELEVATION, N.T.S

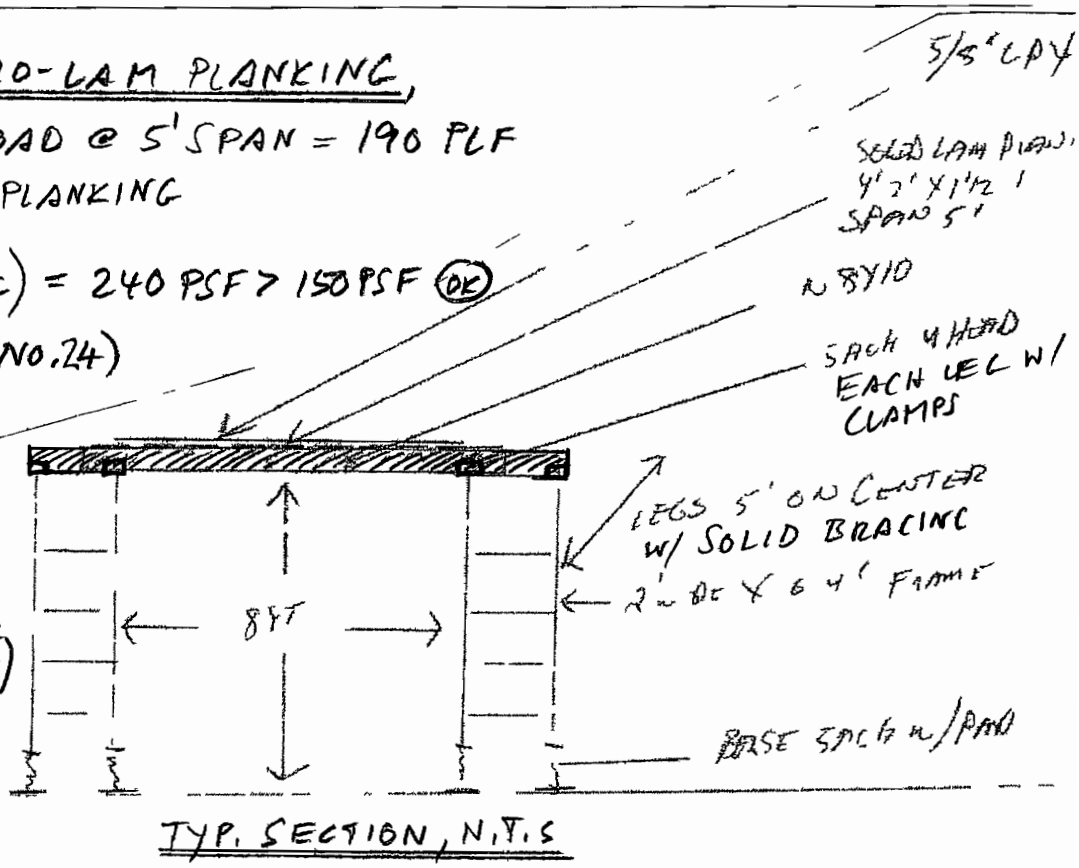
1. CHECK 1½" MICRO-LAM PLANKING,

⇒ ALLOWABLE LOAD @ 5' SPAN = 190 PLF FOR 1½" × 9½" PLANKING

⇒ $\bar{W}_{ALL} = 190\left(\frac{12}{9.5}\right) = 240\ PSF > 150\ PSF$ (OK)

(SEE SHEET NO. 24)

2. CHECK W8×10 @ 5' O.C



TYP. SECTION, N.T.S

⇒ MAX M = $\left[(8+150)(5)+10\right]\left(\frac{8^2}{8}\right)$

↳ S.W.
↳ PROT. LOAD
↳ PLANKING + PLWD

$= 6{,}400\ FT\text{-}\#$

$\hat{=} 6.4^{k}\text{-}FT < 14.5^{k}\text{-}FT = ALL.\ M$ (OK)

(PER AISC MANUAL FOR STEEL CONSTRUCTION, 13th EDITION)

3. CHECK 2'-0" WIDE × 6'-4" NARROW SECTION FRAMES, (↳ S.F. SCAFFOLDING)

⇒ ALLOWABLE LEG LOAD = $3{,}000\left(\frac{4.0}{2.5}\right) = 4{,}800\ \#$

$> 3{,}200\ \# = \underbrace{\left[(8+150)(5)+10\right]\left(\frac{8}{2}\right.}_{\text{SEE ITEM 2}}$

(OK)

⇒ USE: SAFWAY SYSTEMS LS 6QLH OR EQUIVALENT FRAMES @ 5'-0" O.C MAX

(SEE SHEET NO. 25)

4 STABILITY,

a) LONGITUDINAL DIRECTION,

⇒ X-BRACE SOLID BETWEEN FRAMES

b) TRANSVERSE DIRECTION,

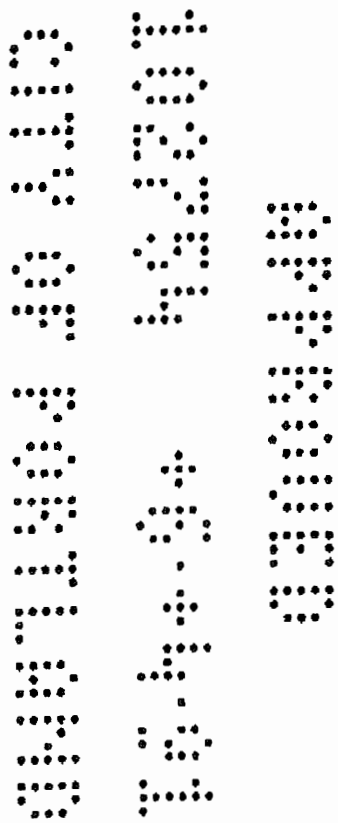
⇒ EXTEND W8x10's TO EXTERIOR LEGS, CONNECT TO ALL LEGS W/ U-JACK & CLAMPS, PROVIDE POSITIVE CONNECTION FROM U-JACKS TO FRAMES

⇒ SEE SHEET NO's 26, 27, 28, 29 FOR BASE JACKS, SCREW JACKS, U-HEADS, AND C-CLAMPS !

SUPERIORITY IN WOOD TECHNOLOGY

MICRO=LAM is a laminated wood plank that's tough, reliable and stiff In fact, in deflection it's 29% stiffer than 2050 dense, select structural Douglas Fir That means important additional stability

MICRO=LAM scaffold planks are available in widths from 10" to 24" and continuous lengths to 80' in three thicknesses (1-1/2", 1 3/4" & 2 1/2") MICRO=LAM scaffold planks are cut from billets in any length you require without a pricing penalty



MICRO=LAM, Scaffold Plank Clear Span Load Table

SIZE 1-1/2"x 9-1/2"

Span Ft	4	5	6	7	8	9	10	11	12
Load Lbs/Ft	300	190	130	100	70	60	50	40	30
Conc Load @ Mid Span Lbs	600	480	400	340	300	260	240	220	200
Uniform Load Deflection In	35	55	80	11	14	18	23	28	32

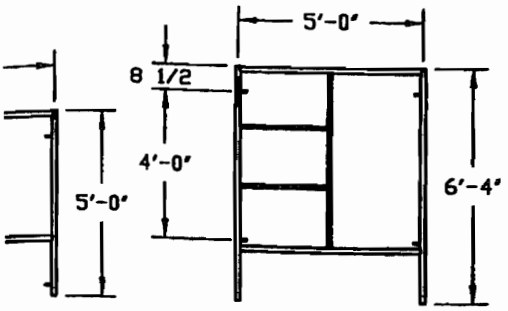
SIZE 1 3/4' x11 3/4"

Span Ft	6	7	8	9	10	11	12	13	14
Load Lbs/Ft	220	160	120	100	80	70	60	50	40
Conc Load @ Mid Span Lbs	670	570	500	440	400	360	330	310	290
Uniform Load Deflection In	68	94	12	15	20	23	28	33	37

SIZE 2 1/2"x11 3/4"

Span Ft	12	13	14	15	16	17	18	19	20
Load Lbs/Ft	110	100	80	70	65	55	50	45	40
Conc Load @ Mid Span Lbs	680	630	580	540	510	480	450	430	410
Uniform Load Deflection In	19	23	26	31	35	39	42	48	54

These tables are for untreated MICRO=LAM



	FM6QLH
	43 LB
	2 200 LB
	2,000 LB

NOTES: 1) MAX ALLOWABLE LEG LOADS ARE BASED UPON TESTS 3 FRAMES HIGH WITH BOTTOM SCREW JACKS EXTENDED 12 INCHES

2) ALLOWABLE UNIFORM LOADS ARE LIMITED TO FRAME TOP LEDGER ONLY

3) TOTAL WEIGHT (BOTH DEAD AND LIVE) MUST NOT EXCEED THESE MAX. ALLOWABLE LOADS

4) THESE COMPONENTS COMPLY WITH FED OSHA REQUIREMENTS WHEN PROPERLY ERECTED

THIS INFORMATION IS SUPPLIED FOR USE BY PERSONS KNOWLEDGEABLE IN SCAFFOLD DESIGN AND QUALIFIED TO EMPLOY GOOD ENGINEERING PRACTICE IN ITS APPLICATION

THIS DATA SHEET IS INTENDED FOR GENERAL INFORMATION ONLY BECAUSE OF THE MANY VARIABLES WHICH AFFECT THE PERFORMANCE OF THIS PRODUCT LINE SOME OF THE INFORMATION MAY NOT APPLY FOR SPECIFIC APPLICATIONS, SEE YOUR SAFWAY DEALER

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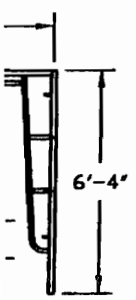
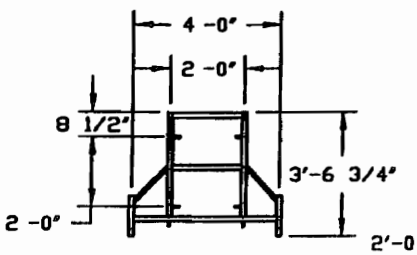
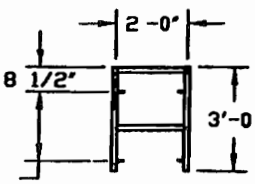
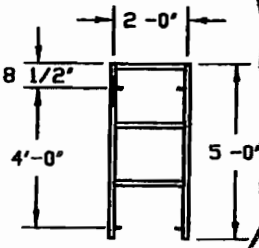
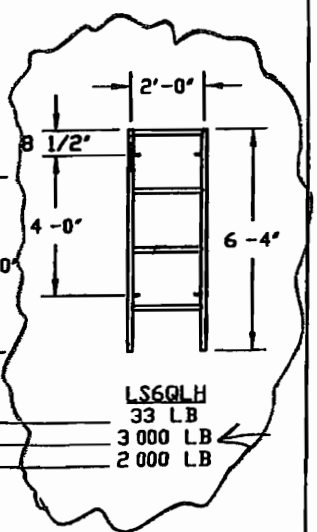
LS SERIES NARROW SECTION FRAMES



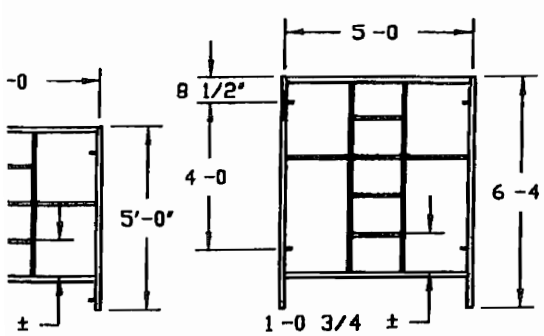








	LSB1QLH	LS3QLH	LS5QLH	LS6QLH
WEIGHT OF FRAME	31 LB	16 LB	25 LB	33 LB
MAX ALLOWED LEG LOAD	3 000 LB	3,000 LB	3 000 LB	3 000 LB
MAX ALLOWED UNIFORM LOAD	N/A	2 000 LB	2 000 LB	2 000 LB



QLH	RT6QLH
LB	51 LB
LB	1 900 LB
LB	1 500 LB

SAFWAY

REV # 01

1/03 orn # 2101

DESCRIPTION: LOAD DATA SHEET STANDARD FRAMES

DISTRIBUTED BY:

SHEET 1 OF 1

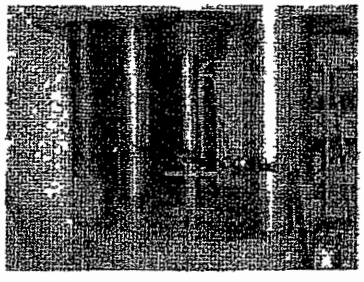
Test Report

IDENTIFICATION SJTP

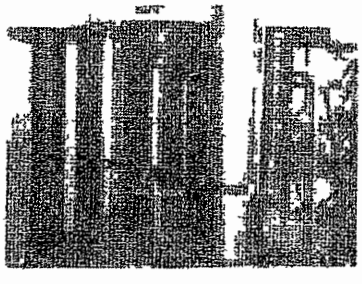
DESCRIPTION GALVANIZED SCREW JACKS WITH 6-1/2" BASE PLATES

LOAD TEST RESULTS

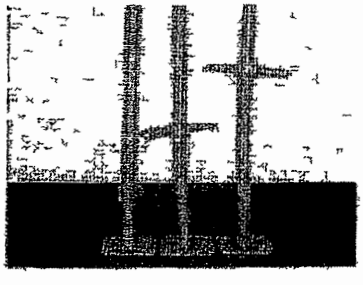
EXTENSION HEIGHT	YIELD LOADS [
6"	56,660
12"	49,595
18"	46,385



Configuration of comprehensive load testing of SJTP screw jacks 6" extension depicted in photograph. Same set up for 12" and 18" extension.



Screw jack at 6" extension at yield load of 16,400 lbs.



Deformation of 6", 12" and 18" extensions after yield.

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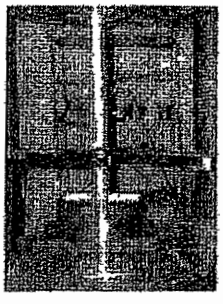
Test Report

IDENTIFICATION SJTS
DESCRIPTION GALVANIZED SCREW JACKS WITH SOCKET

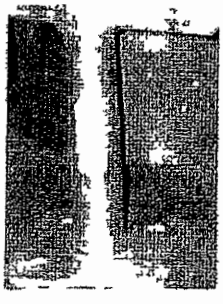
LOAD TEST RESULTS

EXTENSION	MAXIMUM LOAD ACHIEVED (lbs)	SAFETY FACTOR BASED ON YIELD (lbs)	FAILURE MODE
NONE	27 900	6 975	SOCKET TUBE COMPRESSION AND JACK DEFORMATION
MID EXTENSION 13" FROM BASE PLATE TO JACK HANDLE	31 450	7 863	SOCKET TUBE COMPRESSION AND JACK DEFORMATION
MAX EXTENSION 23" FROM BASE PLATE TO JACK HANDLE	31 700	7 925	SOCKET TUBE COMPRESSION AND JACK DEFORMATION

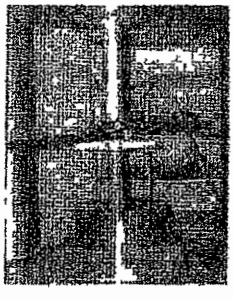


Test configuration with no extension

No extension yield load of 27 900 lbs



Mid extension yield load of 31 450 lbs



Max extension yield load of 31 700 lbs

Lower compressive strength value for jack with no extension is due to direct loading of screw jack handle onto socket tube weld

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Post Shores

(2.375"O.D. Shore, 1.9"O.D. Staff)

(Available Painted or Galvanized)

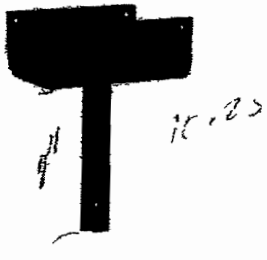
PSV 722(#4)
10'6" TO 16' POST SHORE
WT 60.67 LBS

PSV 721(#3)
8'6" TO 13' POST SHORE
WT 53.45 LBS

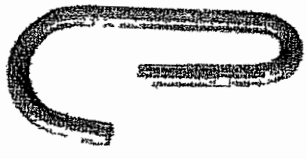
PSV 720(#2)
6'6" TO 11' POST SHORE
WT 45.76 LBS

PSV-719(#1)
5'9" TO 10'3" POST SHORE
WT 39.24 LBS

PSV-718(#0)
3'6" TO 6' POST SHORE
WT 26.49 LBS



PSV 158
5"X7" POST SHORE U HEAD
WT 7.88 LBS



PSV 19
POST SHORE HANDLE
WT 1.41 LBS

PSV 720 18
POST SHORE STAFF W/
BASE PLATE
WT 20.24 LBS

PSV 720 10
POST SHORE STAFF W/
U HEAD
WT 25.08 LBS

PSV 720HDU
6'6" to 11' H.D. POST SHORE
W/ U-HEAD
WT 46.42 LBS

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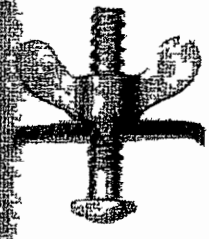
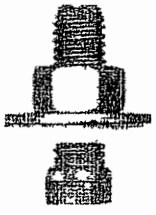
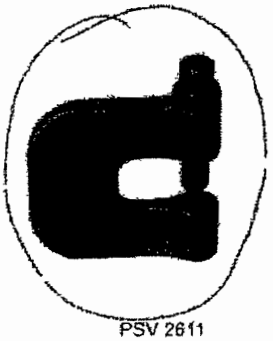


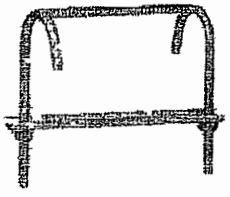
PSV-908A
A-CLAMP ASSEMBLY STEEL
WT 0 5 LBS



PSV 908B
2 A CLAMP ASSEMBLY ALUMINUM
WT 0 3 LBS

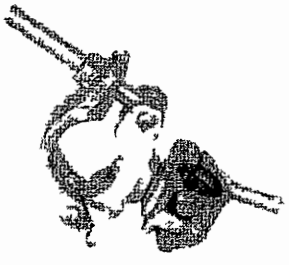


PSV 2611
#2 C-CLAMP
WT 0 38 LBS



PSV 1301
TIMBER BRACING CLAMP
WT 2 71 LBS

PSV-886
BASE FRA
DER STYL
57 73LBS

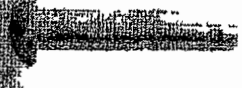


PSV 905
2 1/2"X2" SWIVEL CLAMP
WT 5 35 LBS

PSV 906
2 1/2"X2" RIGHT ANGLE CLAMP
WT 5 22 LBS

PSV 126
NAILING PLATE FOR
10K FRAME LEC
WT 1 19 LBS

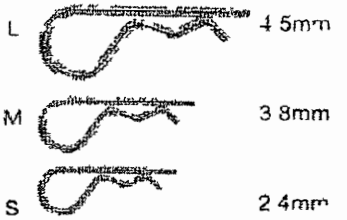
PIN
S



PSV 16
5/8 DRILLED RIVET
WT 0 35 LBS

PSV 129
J PIN
WT 0 55 LBS

3
NNECTOR
LBS



PSV 117L	HITCH PIN WT 0 07 LBS
PSV 117M	HITCH PIN WT 0 05 LBS
PSV 117S	HITCH PIN WT 0 04 LBS

PSV 102
h L SHORING CONNECTOR
WT 0 92 LBS

PSV-102B
UNIVERSAL
SHORING CONNECTOR
WT 1 23 LBS

WE CAN CUSTOMIZE TO MATCH ANY STYLE OR COLOR

WDY Structural • Civil Engineers

STRUCTURAL CALCULATIONS

FOR

PROVIDENCE MEDICAL CENTER POP2
CONSTRUCTION PROTECTION CANOPIES

PORTLAND, OREGON

DECEMBER 22, 2010

☒ NO EXCEPTION NOTED ☐ MAKE CORRECTIONS NOTED
☐ REJECTED ☐ REVISE AND RESUBMIT

THIS REVIEW IS FOR GENERAL CONFORMANCE OF DESIGN LOADS AND METHODOLOGY ONLY. ANY DEVIATION IN PLANS OR SPECIFICATIONS NOT CLEARLY NOTED BY CONTRACTOR OR THEIR ENGINEER HAS NOT BEEN REVIEWED. REVIEW SHALL NOT CONSTITUTE A COMPLETE CHECK OF DETAILED CALCULATIONS OR SERVE TO RELIEVE CONTRACTOR AND THEIR ENGINEER OF CONTRACT RESPONSIBILITY FOR ANY ERROR OR DEVIATION FROM CONTRACT DOCUMENTS AND BUILDING CODE REQUIREMENTS.

AAI ENGINEERING

Date 02/08/11 By Derek Larson

DESIGN PARAMETERS **IBC 2009 with OSSC 2010 Amendments**

PEDESTRIAN PROTECTION LOAD	150 psf

CONTENTS

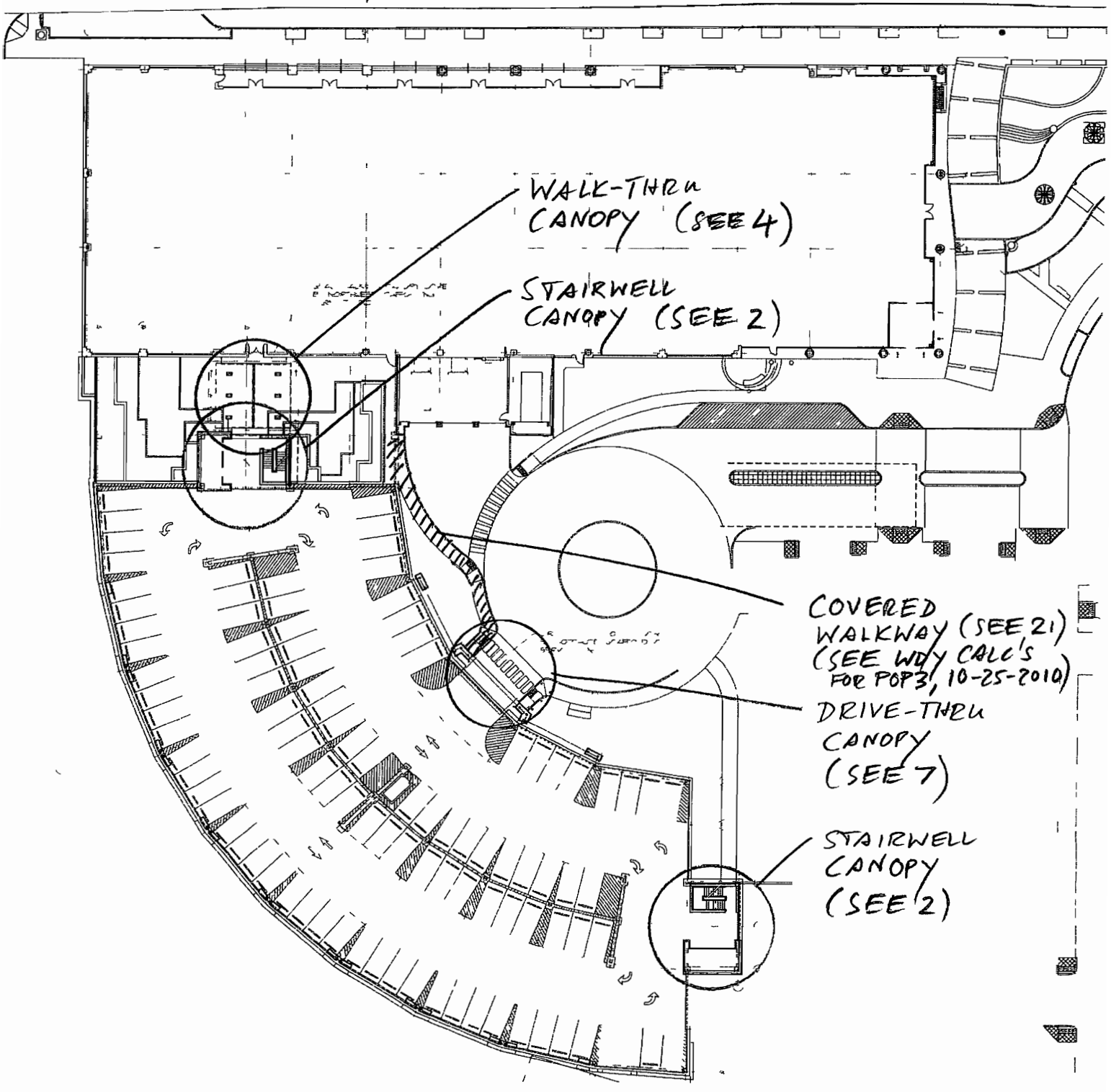
CANOPY DESIGN	1 to 29

6443 SW Beaverton Hillsdale Hwy suite 210 Portland OR 97221 ph 503 203 8111 • fx 503 203 8122 www wdyl com

WDY **Structural • Civil Engineers**

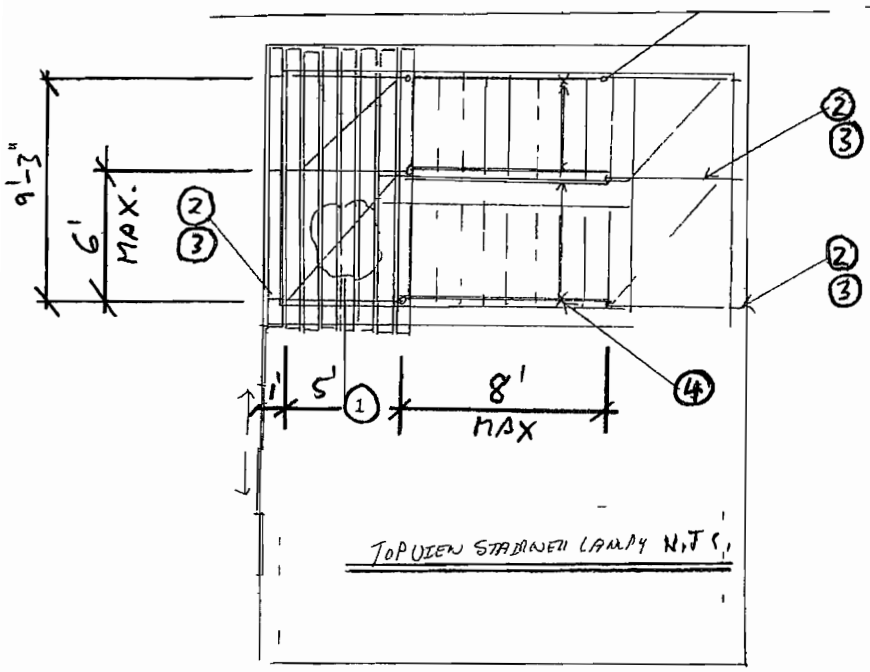
Job Name	**Providence POP2 Construction Protection Canopies**	Job No	**10723**	Sheet No	**1**
Client	**Northwest Scaffold Service Inc**	Date	**Nov 2010**	By	**mw**

PARTIAL SITE PLAN, N.T.S.

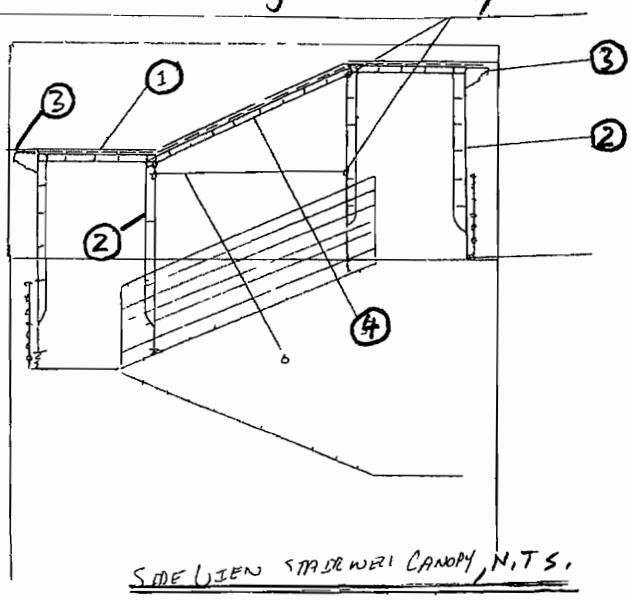


TYP. STAIRWELL CANOPY,

- PEDESTRIAN PROTECTION LL = 150 PSF, (USE S.F. = 1.33 IN CONJUNCTION W/ 150 PSF)



TOP VIEW STAIRWELL CANOPY N.T.S.



SIDE VIEW STAIRWELL CANOPY, N.T.S.

- DESIGN,

(1) $1\frac{1}{2}$" x $9\frac{1}{2}$" MICRO-LAM PLANKING W/ $\frac{5}{8}$" CDX PLYWOOD, MAX SPAN = 6

$\Rightarrow$ MAX. $w = [(4 + 1.8) + (150)] = 155.8$ PSF < 164.2 PSF $= 130$ PLF $\left(\frac{12}{9.5}\right)$

(PLWD → PLANKING)

(SEE 9) (OK)

(2) STEPUP SCAFFOLD FRAME TF6N-5W-L5,

$\Rightarrow$ MAX. LEG LOAD $= 1558\left[\left(\frac{5}{2}\right) + \left(\frac{8}{2}\right)\left(\sim\frac{10}{2}\right)\right] = 5{,}064^{\#}$

$< 14{,}565^{\#} = 4{,}843^{\#}\left(\frac{4.0}{1.33}\right)$ (OK)

(SEE 10-12, TF6N-5W-L3 HAS TUBE OD = 1.625" < 1.69" = TUBE OD FOR TF6N-5W-L5)

TYP. STAIRWELL CANOPY, DESIGN, CONT.,

(3) STEPUP 12" SIDE BRACKET SBCP-12,

$\Rightarrow$ MAX. UNIFORM LOAD = $155.8(1)(6) = 935^{\#}$

$< 1{,}844^{\#} = 613^{\#}\left(\frac{40}{1.33}\right)$ (OK)

(SEE 13,14 SB20-SN IS 20" BRACKET > 12" BRACKET = SBCP-12)

(4) STEPUP PUTLOG TRUSS P-12, MAX. SPAN = 8'

$\Rightarrow$ MAX. $w = 155.8\left(\frac{9.25}{2}\right) = 721$ PLF

$\Rightarrow$ ALL. UNIFORM LOAD FOR P-12 PER STEPUP (SEE 15)

$= 1{,}350^{\#} \triangleq 112.5$ PLF $= \frac{1{,}350}{12}$

$\Rightarrow$ EQIVALENT $M = \frac{112.5(12)^2}{8} = 2{,}025$ FT-#

$\Rightarrow$ EQUIV ALL. w FOR 8' SPAN $= \frac{2{,}025(8)}{8^2} = 253$ PLF

$\Rightarrow$ ADJUST FOR S.F. = 1.33: $253\left(\frac{40}{1.33}\right) = 761$ PLF > 721 PLF

(OK)

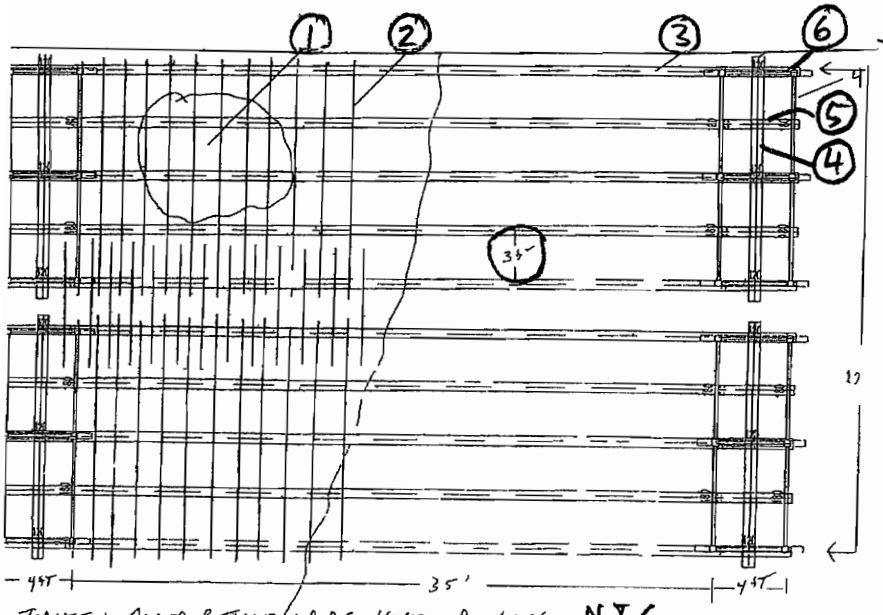
(5) STABILITY,

SCAFFOLD FRAMES ARE CONNECTED TO (E) STAIRWELL RAILING WITH STEPUP CLAMP-S

WALK-THRU CANOPY,

— LOADS : DECKING – PEDESTRIAN PROTECTION LL = 150 PSF

SUPPORT STRUCTURE – 25 PSF SNOW LOAD



TOP VIEW COVER BETWEEN POP 2 OFFICE + PARKING, N.T.S

— SNOW DRIFT:

$P_g = 20$ PSF ; $L_u \geq 11.1'$

$\Rightarrow h_d = 3.2'$, $W_d = 4 \times 3.2 = 12.8'$

$P_d = 3.2(.13 \times 20 + 14) = 53$ PSF

(USE S.F. = 2.5 IN CONJUNCTION W/ SNOW LOAD FOR SUPPORT STRUCTURE)

— DESIGN,

① $\frac{5}{8}$" CDX PLYWOOD, CLEAR SPAN = 16" − 5" = 11"

$\Rightarrow$ MAX. W = 150 PSF = 150 PSF (OK)

(PER APA DESIGN/CONSTRUCTION GUIDE "RESIDENTIAL & COMMERCIAL" – $\frac{5}{8}$" SHEATHING W/ FLOOR JOISTS @ 16" O.C)

② SYMONS ALUMINUM BEAMS (AB) @ 16" O.C., MAX SPAN = 6'

$\Rightarrow$ MAX. $W = \left[\left(1.8 \times \frac{16}{12} + \sim 5\right) + \left(150 \times \frac{16}{12}\right)\right] = 207.4$ PLF

WALK-THRU CANOPY, DESIGN, (2) ALUMINUM BEAMS, CONT

$\Rightarrow$ MAX. $M = \frac{207.4(6)^2}{8} = 933.3$ FT-# $< 11{,}100$ FT-# = ALL. M (OK)

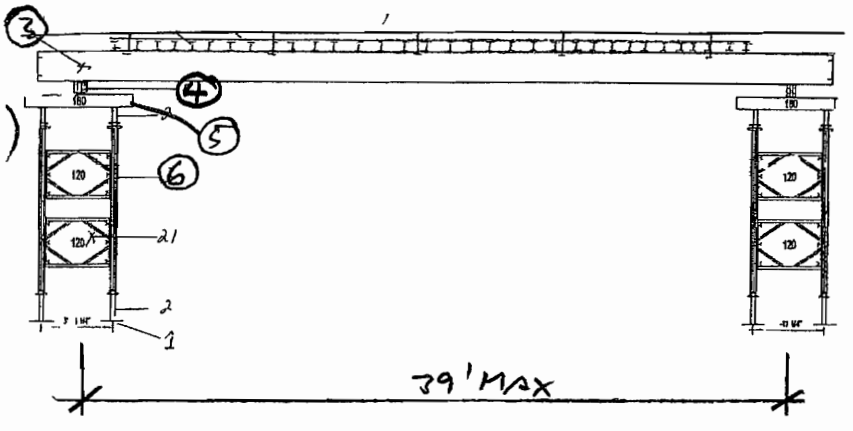
$\Rightarrow$ MAX. $R_{END} = 207.4\left(\frac{6}{2}+\sim 1\right) = 829.6^{\#} < 4{,}100^{\#} = ALL.\ R_{END}$ (OK)

$\Rightarrow$ MAX. $R_{INT} = 207.4(1.25\times 6) = 1{,}556^{\#} < 8{,}200^{\#} = ALL.\ R_{INT}$ (OK)

(SEE 16)

(3) W21x44 @ 3'0.C.

(PROVIDED BY SYMONS, F_y = 36 KSI)



END VIEW CANOPY BETWEEN POP2 OFFICE & PARKING, N.T.S

$\Rightarrow$ MAX $w = \left[(1.8+5)(3) + 44\right] + \left[\left(20+\frac{3}{4}(53)\right)(3)\right] = 244$ PLF

- $(1.8+5)(3)$ → PLYWOOD & ALUMINUM STRINGERS
- 44 → BEAM SELF WEIGHT
- $\left(20+\frac{3}{4}(53)\right)(3)$ → SNOW DRIFT / FLAT ROOF SNOW } > 25PSF

$\Rightarrow$ MAX. $M = \frac{244(39)^2}{8(1{,}000)} = 46.4^{k}\text{-FT} < 163^{k}\text{-FT} = ALL.\ M$ (F_y = 36 KSI) (OK)

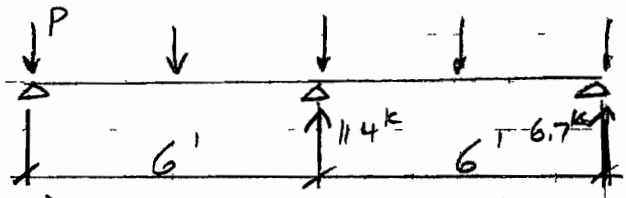
$\Rightarrow$ MAX $R = \frac{244(39)}{2(1{,}000)} = 4.76^{k}$

$\Rightarrow$ SPLICES @ 10' FROM SUPPORT. $M_{10'} \cong 35.4^{k}\text{-FT} < 68.8^{k}\text{-FT}$ (OK)
$V_{10'} \cong 2.3^{k} < 41^{k}$ (SEE 17)

WALK-THRU CANOPY, DESIGN, CONT,

(4.) DBL. TITAN 225 SUPPORT BEAM,

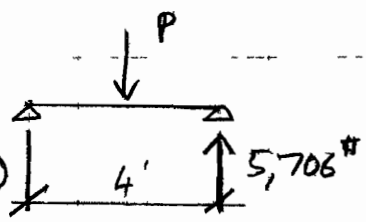
$P = 4.76^{k}$



$\Rightarrow$ MAX $M^{+} = 5,833$ FT-# $< 21,020$
MAX. $M^{-} = 5,419$ FT-# $< 21,020$
MAX. $R_{END} = 6,726^{\#} < 20,000^{\#}$
MAX. $R_{INT} = 11,411^{\#} < 20,000^{\#}$
} (OK) (SEE 18)

(5.) SINGLE TITAN 225 CROSS-BEAM,

$P_{MAX} = 11,411^{\#}$



$\Rightarrow$ MAX. $M^{+} = \frac{11,411(4)}{4} = 11,411$ FT-# $< 21,020$ (OK)

MAX. $R = \frac{11,411}{2} = 5,706^{\#} < 20,000^{\#}$ (OK) (SEE 18)

(6) TITEN ISCHEBECK POST SHORES,

MAX. $H \cong 18'$

MAX. $P \cong 5,706^{\#} < 11,600^{\#}$ (OK) (SEE 19)

(7.) ~5" x 5" BASE PLATE ON 2x12 WOOD PAD,

$$f_{c\perp} = \frac{1}{5^2}\left[\underbrace{5,706}_{\text{SEE ABOVE}} + \underbrace{5,94\left(2\times 1.25\times 6 + \tfrac{1}{2}\times 6\right)}_{\text{TITAN 225 BEAMS}} + \underbrace{\sim 70}_{\text{SHORE}} + \underbrace{\tfrac{1}{2}(2\times 15 + 2\times 23}_{\text{FRAMES}}\right.$$

$= 237$ PSI (OK)

DRIVE-THRU CANOPY,

- LOADS DECKING - PEDESTRIAN PROTECTION LL = 150 PSF

SUPPORT STRUCTURE - 25PSF SNOW LOAD

- SNOW DRIFT

P_G = 20 PSF ; $P_h \approx 13.5'$

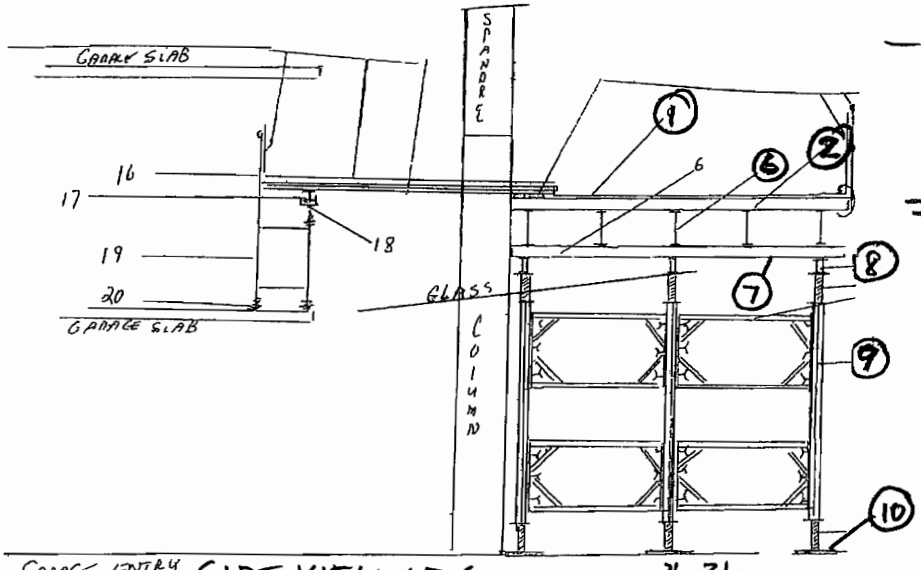
$\Rightarrow \frac{3}{4} h_d = .75 \times .8 = .6'$

$W_d = 4 \times .6 = 2.4' < 12.8'$

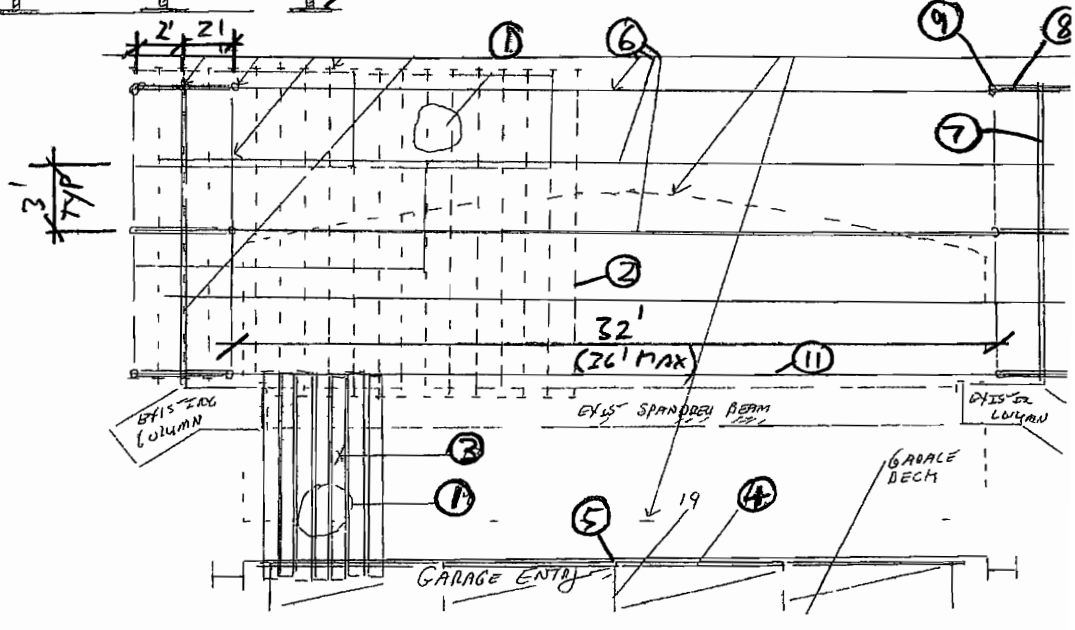
$P_d = 6(.13 \times 20 + 14) = 10PS$

$< 53PS$

$\Rightarrow$ USE SNOWDRIFT PER SHEET L



GARAGE ENTRY SIDE VIEW, N.T.S.



PLAN VIEW GARAGE ENTRY CANOPY, N.T.S

- DESIGN, (1) AND (2), SEE WALK-THRU CANOPY, ITEMS (1) AND (2)

DRIVE-THRU CANOPY, DESIGN, CONT,

(3) DOUBLE $1\frac{1}{2}" \times 9\frac{1}{2}"$ MICRO-LAM PLANKING, MAX. SPAN = 9'

$\Rightarrow$ MAX. $W = [(4\times2+1.8)+(150)] = 159.8$ PSF < 455.9 PSF (OK)
$= 2 \times 60$ PSF $\left(\frac{12}{9.5}\right)\left(\frac{4.0}{1.33}\right)$ (SEE 9)

(4) SYMONS ALUMINUM BEAMS (AB) OR W8x10, MAX. SPAN = 7

$\Rightarrow$ MAX. $W = 159.8\left(\frac{9}{2}\right) = 719.1$ PLF

$\Rightarrow$ MAX. $M = \frac{719.1(7)^2}{8} = 4,404$ FT-# $< 11,100$ FT-# = ALL. M (OK)
$< 16,000$ FT-# (W8x10)

$\Rightarrow$ MAX. $R_{END} = 719.1\left(\frac{7}{2}+\sim1\right) = 3,236^{\#} < 4,100^{\#} =$ ALL. R_{END} (OK)

$\Rightarrow$ MAX. $R_{INT} = 719.1\,(1.25\times7) = 6,292^{\#} < 8,200^{\#} =$ ALL. R_{INT} (OK)
(SEE 16)

(5) STEPUP SCAFFOLD FRAME NF3H-2W-L3

$\Rightarrow$ MAX. LEG LOAD $= 6,292^{\#}$ (SEE ITEM (4))
$< 14,565^{\#} = 4,843^{\#}\left(\frac{4.0}{1.33}\right)$ (OK)
(SEE 10, 11, 20, TF6H-5W-L3
HAS TUBE OD $= 1.675" < 1.69"$ =
TUBE OD FOR NF3H-2W-L3)

(6) THRU (10), SEE WALK-THRU CANOPY, ITEMS (3) THRU (7)

DRIVE-THRU CANOPY, DESIGN, CONT.,

(11) CHECK W21x44 FOR ADDED PLANKING,

SPAN = 36'

$$w \cong \left[44 + \underbrace{(18+5)\left(\tfrac{3}{2}+1\right)}_{\text{PLWD + ALM STRINGERS}} + \underbrace{(4\times 2+1.8)\left(\tfrac{9}{2}+1\right)}_{\text{DBL. PLANKING + PLWD}}\right] + \left[\left(20+\tfrac{1}{2}\left(\tfrac{8.3+2.3}{12.8}\right)(53)\right)6\right]$$

SELFWEIGHT; SNOW DRIFT; FLAT SNOW

$= 367$ PLF

$$\Rightarrow \text{MAX. } M = \frac{367(36)^2}{8(1{,}000)} = 59.5^{k}\text{-FT} < 163^{k}\text{-FT} = \text{ALL.M}$$

($F_y = 36$ KSI) (OK)

$$\Rightarrow \text{MAX. } R = \frac{367(36)}{2(1{,}000)} = 6.61^{k}$$

SPLICE (OK) (SEE SHEET 5)

⇒ CHECK SINGLE TITAN 225 CROSS-BEAM,

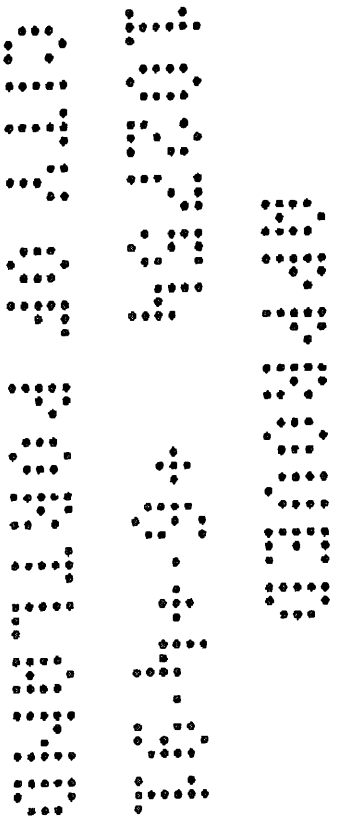
$P \cong 6.7^{k} + (6.61 - 4.76) = 8.6^{k} < 11.4^{k}$ (SEE 6) (OK)

- 6.7^k: SEE SHEET 6, EXT REACTION = 6.7^k
- 6.61^k: SEE ABOVE, $R = 6.61^k$
- 4.76^k: SEE SHEET 6, ITEM (4) $P = 4.76^k$

SUPERIORITY IN WOOD TECHNOLOGY

MICRO=LAM is a laminated wood plank that's tough, reliable and stiff In fact, in deflection it's 29% stiffer than 2050 dense, select structural Douglas Fir That means important additional stability

MICRO=LAM scaffold planks are available in widths from 10" to 24" and continuous lengths to 80' in three thicknesses (1 1/2", 1 3/4" & 2 1/2") MICRO=LAM scaffold planks are cut from billets in any length you require without a pricing penalty



MICRO=LAM, Scaffold Plank Clear Span Load Table

SIZE 1 1/2"x 9 1/2"

Span Ft	4	5	6	7	8	9	10	11	12
Load Lbs/Ft	300	190	130	100	70	60	50	40	30
Conc Load @ Mid Span Lbs	600	480	400	340	300	260	240	220	200
Uniform Load Deflection In	35	55	80	11	14	18	23	28	32

SIZE 1 3/4"x11-3/4"

Span Ft	6	7	8	9	10	11	12	13	14
Load Lbs/Ft	220	160	120	100	80	70	60	50	40
Conc Load @ Mid Span Lbs	670	570	500	440	400	360	330	310	290
Uniform Load Deflection In	68	94	12	15	20	23	28	33	37

SIZE 2-1/2"x11 3/4"

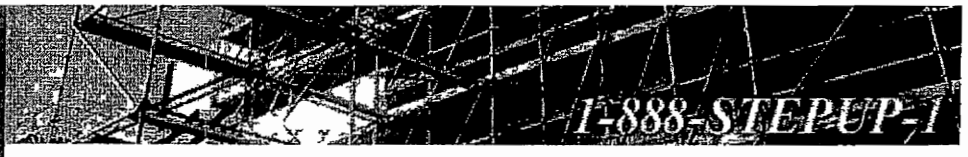
Span Ft	12	13	14	15	16	17	18	19	20
Load Lbs/Ft	110	100	80	70	65	55	50	45	40
Conc Load @ Mid Span Lbs	680	630	580	540	510	480	450	430	410
Uniform Load Deflection In	19	23	26	31	35	39	42	48	54

These tables are for untreated MICRO=LAM

STEPUP

STEPUP SCAFFOLD

The quality source for all your scaffolding needs



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Testing

CHEMICAL ANALYSIS

		Q345 E			AISI 1512	
Element		Result	Min %	Max %	Min %	Max %
C	=	0.13		0.18	0.10	0.16
Mn		1.20	1.00	1.60	1.10	1.40
P	-	[illegible]		0.025	0.000	0.040
S	-	[illegible]		0.025	0.000	0.0[illegible]0

The leg of [illegible] ASTM A29

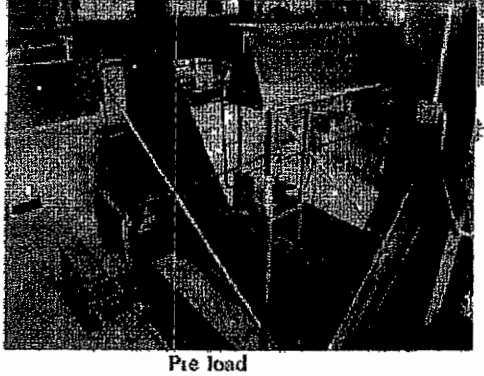
TENSILE TEST RESULTS

Specimen Dimensions OD x Wall in	Area Sq ins	Yield Strength psi 0.2% Offset	Total Load lbs	Tensile Strength psi
1.701 x 0.080	0.455	67 600	35 525	78 100
Requirement	---	60 000 min	---	70 000 min

Tensile [illegible]

SINGLE TIER COMPRESSIVE LOAD TEST

Load at Yield	Load per Leg	Per Leg Load at 4:1 Safety Factor
77 500 lbs	19 375 lbs	4 843 lbs



Pre load

Yield at 77 500 lbs

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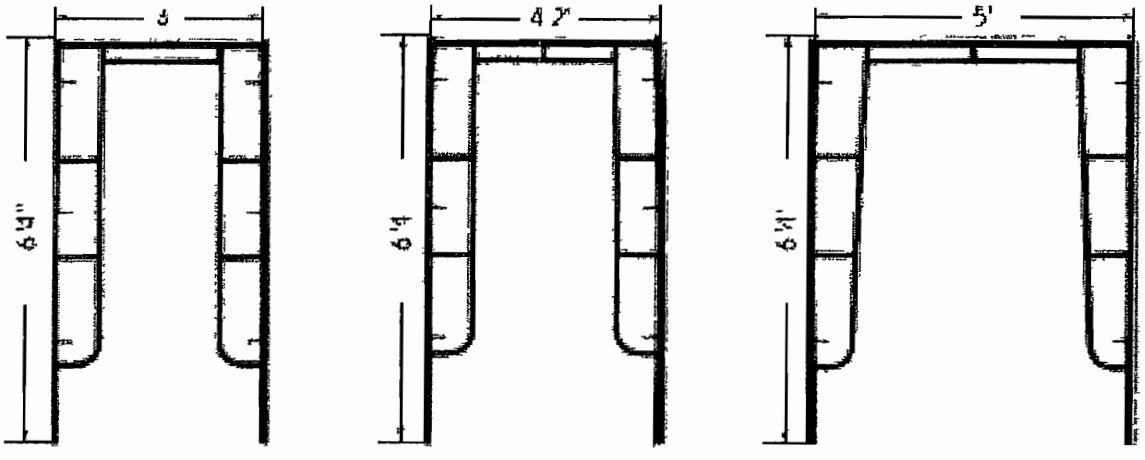
Quality Service Satisfaction

Frames

THROUGH FRAMES

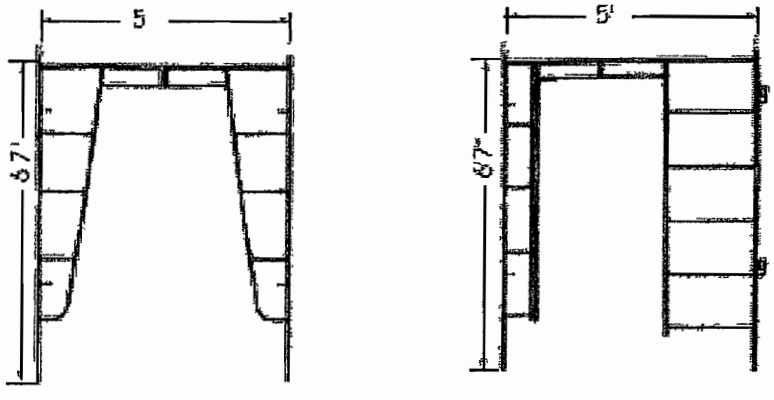
DROP-LOCK THROUGH FRAMES



TF6H-3W-L3 TF6H-4'2W-L3 TF6H-5W-L3

MIGHTY THROUGH FRAMES

- Tube OD 1.625"
- 13 Gauge Tube
- Manganese Steel
- Powder-Coated
- Minimum Yield Strength 50,000 PSI
- Minimum Tensile Strength 65,000 PSI



TF6H-5'-L2-M TF6H-5W-M

THROUGH FRAMES		
DROP-LOCK THROUGH FRAMES		
Item No.	Description	Weight (lbs.)
TF6H-3W-L3	6'4" x 3' Wide Through Frame with Drop-Lock	47.25
TF6H-4'2W-L3	6'4" x 4'2" Wide Through Frame with Drop-Lock	49.50
TF6H-5W-L3	6'4" x 5' Wide Through Frame with Drop-Lock	51.75
MIGHTY THROUGH FRAMES		
Item No.	Description	Weight (lbs.)
TF6H-5W-L2-M	5' x 6'7" Wide Thru Frame with 22" Ladder Mighty Step	46
TF6H-5W-L2-M	5' x 6'7" Wide Thru Frame with 22" Ladder Mighty Step	55

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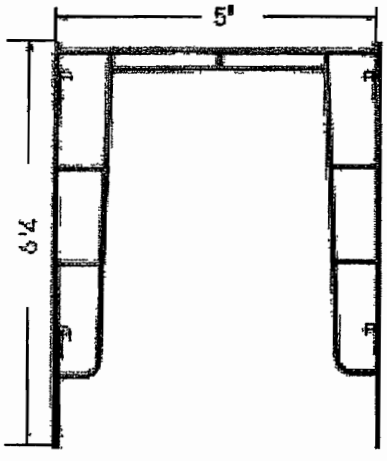
Frames

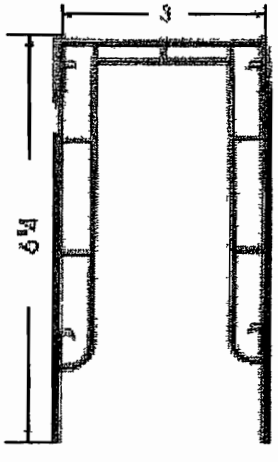
THROUGH FRAMES

V-LOCK THROUGH FRAMES

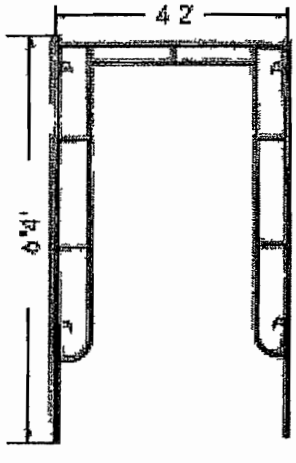


TF5-5W-L5

- Tube OD 1.69"
- 13 Gauge Tube
- Manganese Steel
- Powder Coated
- Minimum Yield Strength 50,000 PSI
- Minimum Tensile Strength 65,000 PSI



TF6H-3W-L5



TF6H-42W-L5

THROUGH FRAMES			
V-LOCK THROUGH FRAMES			
Item No.	Description	Rung Center	Weight (lbs)
TF6H-3W-L5	6'6"H X 3'W with V-Lock	5'	44
TF6H-42W-L5	6'6"H X 4'2"W with V-Lock	5'	46
TF6H-5W-L5	6'6"H X 5'W with V-Lock	5'	49
TF2H-6W-L5	Sidewalk Canopy Frame with V-Lock	2' 4'	57

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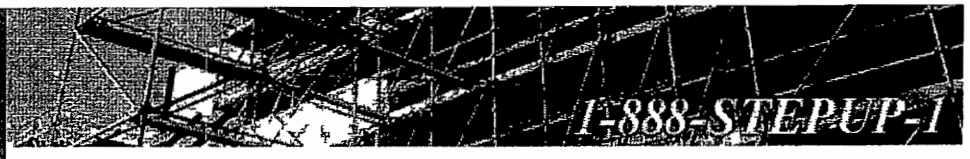

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Testing

CHEMICAL ANALYSIS

			Q215 A,B		Q235 A		AISI 1513	
Element		Result %	Min %	Max %	Min %	Max %	Min %	Max %
C		0.13	0.09	[illegible]	—	0.16	0.10	0.16
Mn	=	1.20	0.25	[illegible]	0.60	50	1.10	1.40
P	=	0.020	—	[illegible]	[illegible]	0.045	0.000	0.040
S		0.016		[illegible]		0.045	0.000	0.050

[illegible]

TENSILE TEST RESULTS

Specimen Dimension OD x Wall	Area [illegible]	[illegible]	[illegible]	[illegible]	% El (2")
1.691 0.09[illegible]	0.453	[illegible]5,700	37,[illegible]22	82,600	17
Requirement	[illegible]	[illegible]	[illegible]	70,000 [illegible]	-

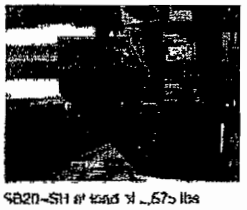
[illegible]

LOAD TEST RESULTS
SB20 SH

Test Sample#	Failure Mode	Yield Load (lbs)	Load with 1.1 Safety Factor (lbs)
1	Deformation of V-Plate (saddle)	2,[illegible]	613
2	Deformation of V-Plate (saddle)	2,575	644

SB20-SH at load of 2,[illegible]50 lbs

[illegible]



SB20-SH at load of 2,575 lbs

Configuration for testing side brackets

Test set-up for SB20-SH type bracket

Several types of scaffold frames were used as the supporting structure.
The type of frame for test purposes had no bearing on resulting loads of side brackets

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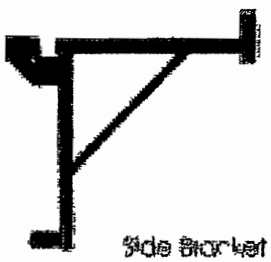
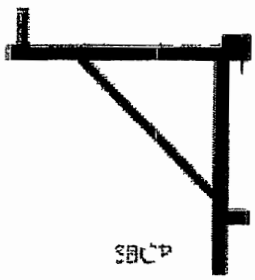
ACCESSORIES

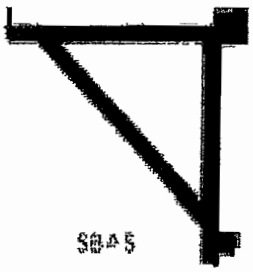
SIDE BRACKETS



Side Bracket

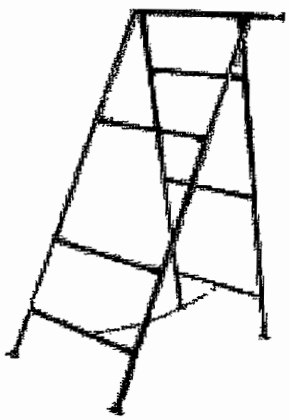


SBCP

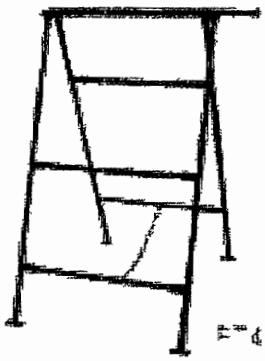


SBAS

FOLDING TRESTLE

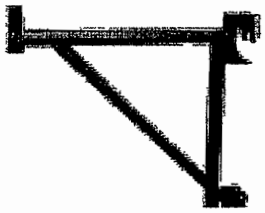


FT6



FT4

END BRACKETS



End Bracket

ACCESSORIES

END BRACKETS

EB20-HH	20" End Bracket with Hook Hanger	13
EB30-HH	30" End Bracket with Hook Hanger	15

SIDE BRACKETS

SB20-SH	20" Side Bracket w/ Saddle Hanger	14
SB20-HH-NY	20" Side Bracket with Hook Hanger, Newyork Style	14
SB20-SH	20" Side Bracket w/Saddle Hanger	10
SB20-HH-H	20" Side Bracket with Hook Hanger, Houston	6.50
SBCP-12	12" Side Bracket with Coupling Pin	5.65
SBCP-20	20" Side Bracket with Coupling Pin	12
SBCP-24	24" Side Bracket with Coupling Pin	15
SBCP-36	36" Side Bracket with Coupling Pin	25
SB20-90	20" Angle Iron Side Bracket with Stud	11
SB20-SH-M	20" Side Bracket, Mighty Style	9.50

FOLDING TRESTLE

FT2	2' Folding Trestle	15
FT3	3' Folding Trestle	19
FT4	4' Folding Trestle	23
FT6	6' Folding Trestle	41
FTE2	2' Folding Trestle Extension	13
FTE3	3' Folding Trestle Extension	17

PUTLOG & ACCESSORIES

PUTLOG

Putlog (P8 P12)

Deep Truss Putlog (P16 P22

PUTLOG ACCESSORIES

PRAH

PPH

PUTLOG ACCESSORIES		
Item No	Description	Weight (lbs)
PPH	Putlog Parallel Hanger	4.30
PRAH	Putlog Right Angle Hanger	5.05

PUTLOG				
Item No	Description	*B (lbs)	*C (lbs)	Weight (lbs)
P8	8' Putlog	1900	1000	28
P12	12' Putlog	[illegible]	[illegible]	31
P16	16' Putlog Deep Truss	2250	[illegible]	54
P22	22' Putlog Deep Truss	1500	[illegible]	98

*B – Uniform Load MAX

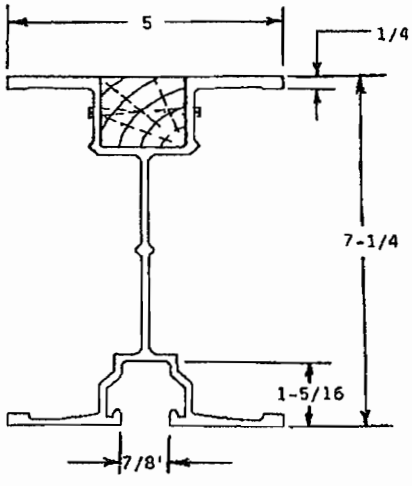
*C – Approved load concentrated in center

ALUMINUM BEAM (AB) PROPERTIES and DESIGN LIMITS

The following Aluminum Beam properties and allowable limits may be used in design calculations for support and loading conditions which are not tabulated

PROPERTIES

Ixx	= 29 20 in (4)
Iyy	= 4 37 in (4)
Rxx	= 2 92 in
Ryy	= 1 13 in
Area	= 3 428 in ²
MOE	= 10 x 10 (6) psi
Alloy/Temper	= 6061-T6



ALLOWABLE LIMITS FOR DESIGN

End Reactions Simple or Multiple Supports without Cantilevers
R = 4100 lbs with 2" min bearing length
R = 5900 lbs with 4" min bearing length

With 6 in Min Cantilever Past Centerline of Support
R = 7400 lbs with 2" min bearing length
R = 7700 lbs when supported by another AB or AS

Intermediate Reactions
R = 9700 lbs with 8" min bearing length (two or three span condition)
R = 8200 lbs with 4" min bearing length (two or three span condition)
R = 9500 lbs with 4" min bearing length (two or three span condition)

Moment
M = 11,100 ft lbs for uniform loading
For AB used as a header (heavy concentrated load), consult a Regional Engineer for lateral stability requirements

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Walnut, CA 91789
PH (909) 594-8201 Ext 14350
FX (909) 594-8253

December 21, 2010

Attn Terry Morrow
Masons Supply

RE W21x44 Roller Deck Beam Splice

Dear Mr Morrow

The purpose of this letter is to state the allowable loads that the butt splice connection in the Symons W21x44 roller deck beam can hold When using (12) ¾" A-325 speed bolts in the splice, and the shear load in the splice is 41 kips or less, the allowable moment in the splice is 68 8 kip-ft The table below shows the allowable moment for a given shear load in the splice The maximum allowable shear load in the splice is 90 kips

Shear (kips)	Allowable Moment (kip-ft)
41	68 8
50	66
60	62
70	57 5
80	52
90	43

¾" A-490 bolts may be substituted for the ¾" A-325 speed bolts in the splice
If you have any questions, please feel free to contact me at (909) 594-8201 Ext 14350

Respectfully,

Kirk J Tatman, P E
Regional Engineer
Symons by Dayton Superior

MEGASHORE beams

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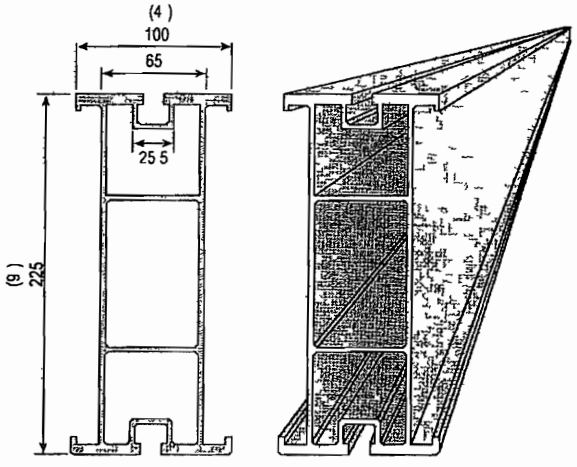
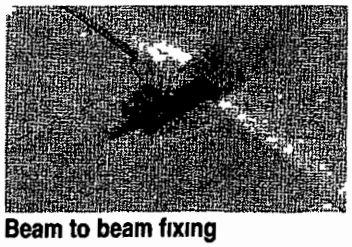
Manufactured from extruded aluminium section,
MEGASHORE beams offer a lightweight, easy to handle solution for decking/soffit applications
All secondary beams are shown color coded in Ischebeck design drawings



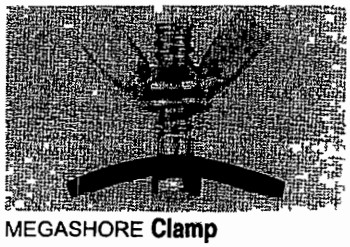
MEGASHORE **225 beam**

	Area	5 057 in²
Ixx	Moment of inertia	54 00 in⁴
Zxx	Section modulus	12 16 in³
E	Modulus of elasticity	9 799 883 lbs/in²
E I	Bending stiffness	31,623 lbs/ft²
W	Weight	5 94 lbs/ft
M	Resistance moment	21 020 lbs ft
S	Allowable shear force	20 000 lbs

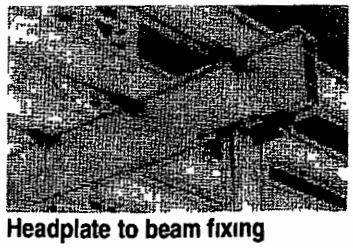
length (m + feet)	length (m +feet)
1 2 = 3-11'	8 0 = 26 3
1 8 = 5-11	9 0 = 29 6
2 4 = 7-11	10 0 = 32 10
3 0 = 9-10"	11 0 = 36 1
3 6 = 11 10'	
4 2 = 13' 9	
4 8 = 15 9	
5 4 = 17 9"	
6 0 = 19 8	
6 4 = 21	
7 2 = 23' 7"	



Beam to beam fixing



MEGASHORE **Clamp**



Headplate to beam fixing

Megashore Loading Graphs

The allowable leg load is determined by the floor to soffit height, the number of ledger frames in height or jack extension

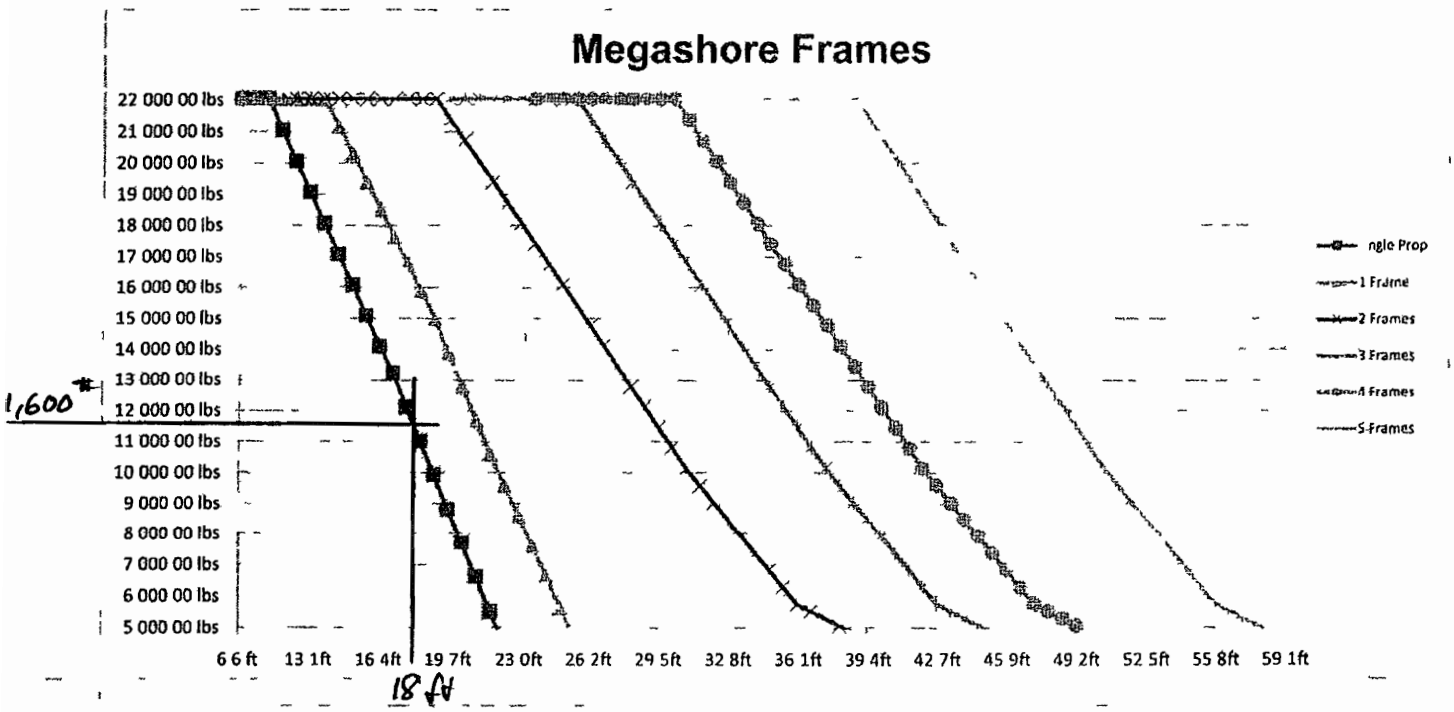
Notes
It is assumed the formwork is restrained from horizontal movement at the formwork level

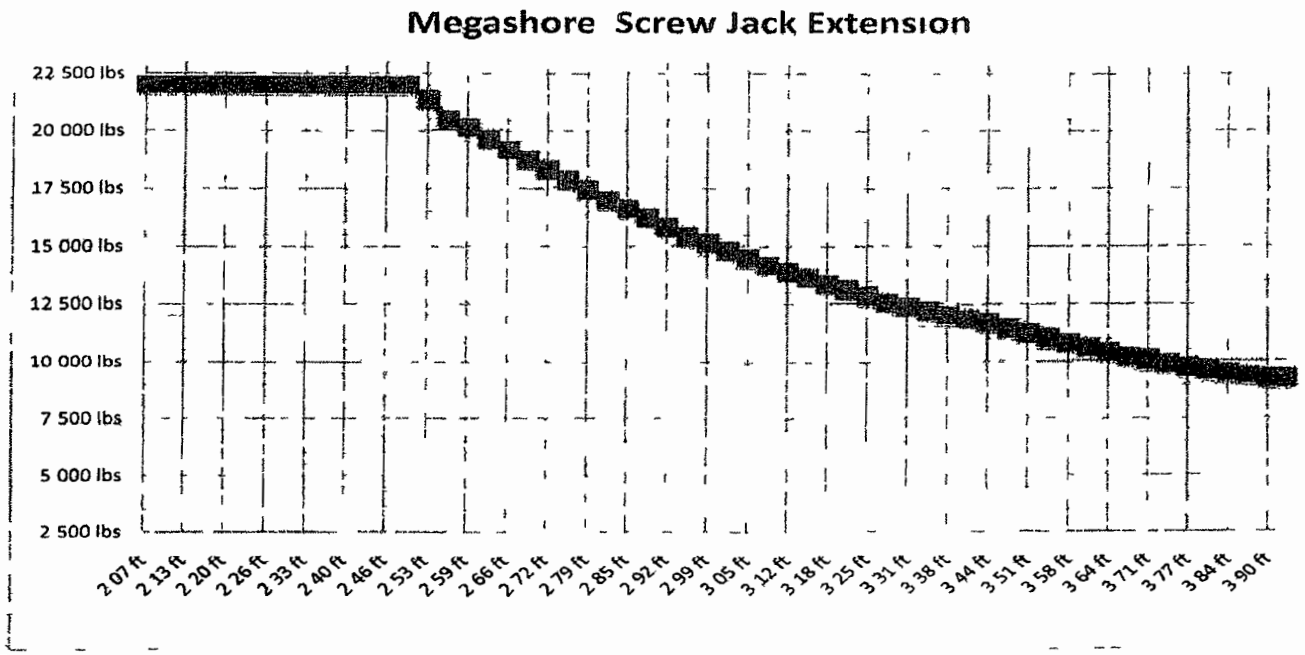
Ledger frames to be fixed in the optimum position

Maximum distance between ledger frames not to exceed 6'-6" between top and bottom chords of ledger frames This rule also applies on floor to soffit heights greater than 36'

On shoring heights over 36 ft refer to Ischebeck Design Services for advice On the higher shoring heights an additional level of frames may be required for ease of erection and stability

Minimum height to base ratio = 4 1





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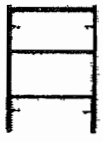
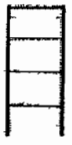
Drop-Lock Narrow Frames

Results **3** items in **1** pages



NF3H 2W-L3



NF5H-2W L3

NF6H-2W-L3

Page 1 of 1 Go to page 1

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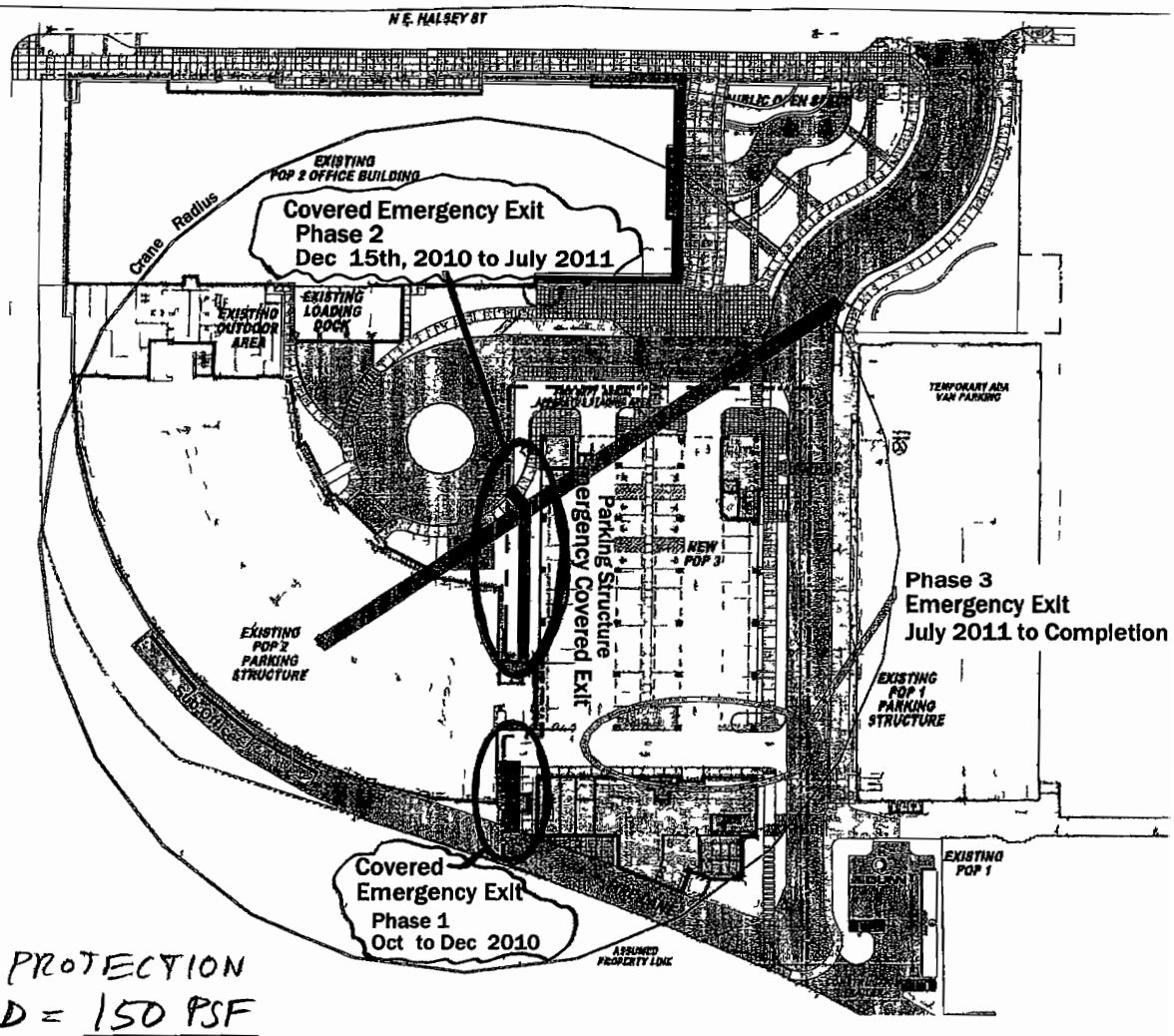
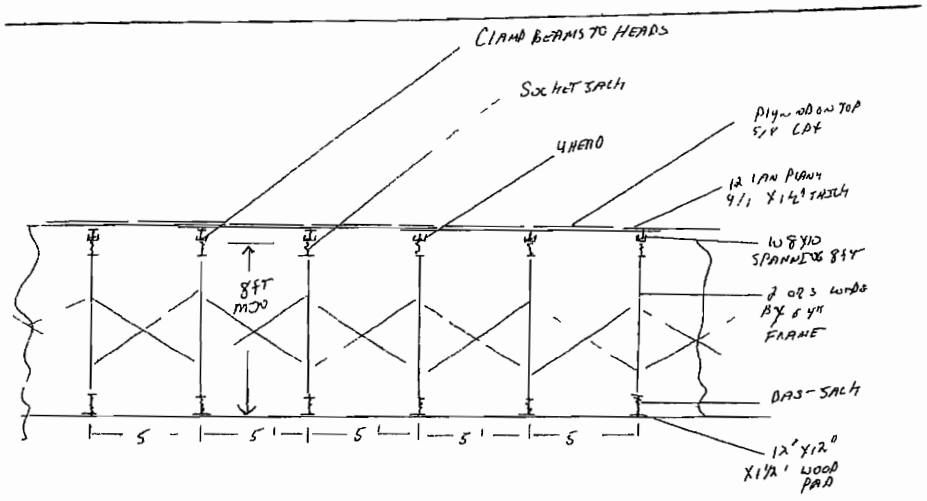
Job Name	Providence POP3 Pedestrian Protection Canopies	Job No	10722	Sheet No	21
Client	Northwest Scaffold Service Inc	Date	Oct 2010	By	mw

TRY:
(2) 2' WIDE x 6'-4" SCAFFOLD FRAMES W/ W8x10 (SPANNING 8') @ 5'0.C W/ 1½" x 9½" MICRO-LAM PLANKING AT COVERED EMERGENCY EXITS (PH1) AND (PH2) (SEE PLAN)



AREA PLAN, N.T.S.

⇒ PEDESTRIAN PROTECTION DESIGN LOAD = 150 PSF



TYP SIDE ELEVATION, N.T.S

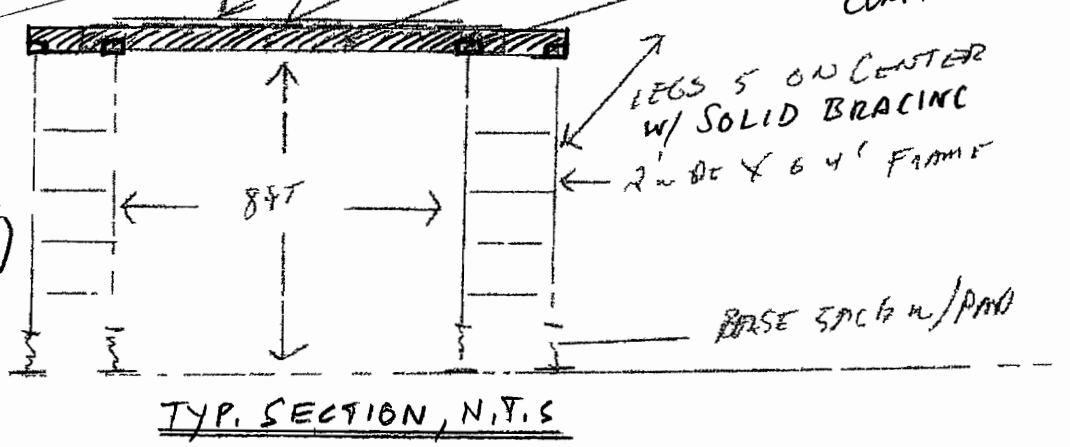
1. CHECK $1\frac{1}{2}$" MICRO-LAM PLANKING,

⇒ ALLOWABLE LOAD @ 5' SPAN = 190 PLF FOR $1\frac{1}{2}$" × $9\frac{1}{2}$" PLANKING

⇒ $\bar{W}_{ALL} = 190\left(\frac{12}{9.5}\right) = 240\ PSF > 150\ PSF$ (OK)

(SEE SHEET NO.24)

2. CHECK W8×10 @ 5'0.C



TYP. SECTION, N.T.S

⇒ MAX M = $\left[(8+150)(5)+10\right]\left(\frac{8^2}{8}\right)$

↳ S.W.
↳ PROF. LOAD
↳ PLANKING + PLWD

$= 6,400\ FT\text{-}\#$

$\hat{=} 6.4^{k}\text{-}FT < 14.5^{k}\text{-}FT = ALL\ M$ (OK)

(PER AISC MANUAL FOR STEEL CONSTRUCTION, 13TH EDITION)

3. CHECK 2'-0" WIDE × 6'-4" NARROW SECTION FRAMES, (↳ S.F SCAFFOLDING)

⇒ ALLOWABLE LEG LOAD = $3,000\left(\frac{4.0}{2.5}\right) = 4,800^{\#}$

$> 3,200^{\#} = \underbrace{\left[(8+150)(5)+10\right]\left(\frac{8}{2}\right.}$ SEE ITEM 2

(OK)

⇒ USE: SAFWAY SYSTEMS LS 6QLH OR EQUIVALENT FRAMES @ 5'0" O.C MAX

(SEE SHEET NO.25)

4 STABILITY,

a) LONGITUDINAL DIRECTION,

⇒ X-BRACE SOLID BETWEEN FRAMES

b) TRANSVERSE DIRECTION,

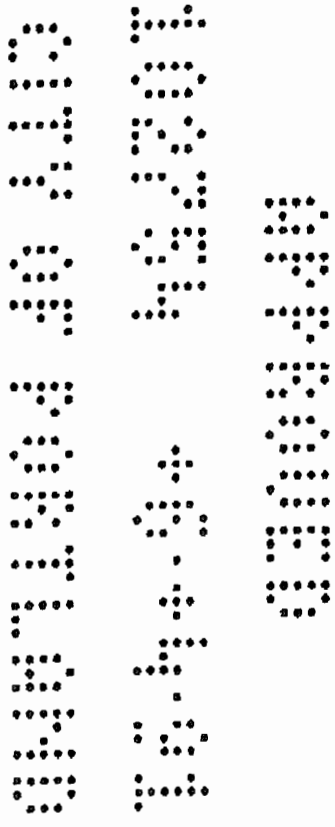
⇒ EXTEND W8x10's TO EXTERIOR LEGS, CONNECT TO ALL LEGS W/ U-JACK & CLAMPS, PROVIDE POSITIVE CONNECTION FROM U-JACKS TO FRAMES

⇒ SEE SHEET NO's 26, 27, 28, 29 FOR BASE JACKS, SCREW JACKS, U-HEADS, AND C-CLAMPS !

SUPERIORITY IN WOOD TECHNOLOGY

MICRO=LAM is a laminated wood plank that's tough, reliable and stiff In fact, in deflection it's 29% stiffer than 2050 dense, select structural Douglas Fir That means important additional stability

MICRO=LAM scaffold planks are available in widths from 10" to 24" and con tinuous lengths to 80' in three thicknesses (1-1/2", 1 3/4" & 2 1/2") MICRO=LAM scaffold planks are cut from billets in any length you require without a pricing penalty



MICRO=LAM, Scaffold Plank Clear Span Load Table

SIZE 1-1/2"x 9 1/2"

Span Ft	4	5	6	7	8	9	10	11	12
Load Lbs/Ft	300	190	130	100	70	60	50	40	30
Conc Load @ Mid Span Lbs	600	480	400	340	300	260	240	220	200
Uniform Load Deflection In	35	55	80	1 1	1 4	1 8	2 3	2 8	3 2

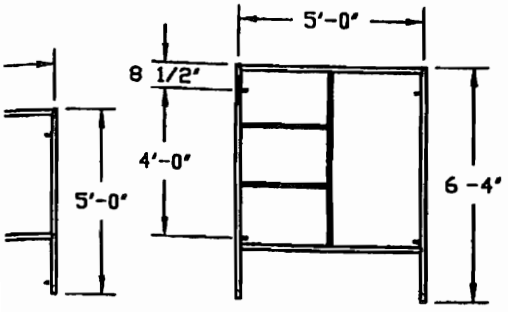
SIZE 1 3/4"x11 3/4"

Span Ft	6	7	8	9	10	11	12	13	14
Load Lbs/Ft	220	160	120	100	80	70	60	50	40
Conc Load @ Mid Span Lbs	670	570	500	440	400	360	330	310	290
Uniform Load Deflection In	68	94	1 2	1 5	2 0	2 3	2 8	3 3	3 7

SIZE 2 1/2"x11 3/4"

Span Ft	12	13	14	15	16	17	18	19	20
Load Lbs/Ft	110	100	80	70	65	55	50	45	40
Conc Load @ Mid Span Lbs	680	630	580	540	510	480	450	430	410
Uniform Load Deflection In	1 9	2 3	2 6	3 1	3 5	3 9	4 2	4 8	5 4

These tables are for untreated MICRO=LAM



	FM6QLH
	43 LB
	2 200 LB
	2,000 LB

NOTES: 1) MAX ALLOWABLE LEG LOADS ARE BASED UPON TESTS 3 FRAMES HIGH WITH BOTTOM SCREW JACKS EXTENDED 12 INCHES

2) ALLOWABLE UNIFORM LOADS ARE LIMITED TO FRAME TOP LEDGER ONLY

3) TOTAL WEIGHT (BOTH DEAD AND LIVE) MUST NOT EXCEED THESE MAX. ALLOWABLE LOADS

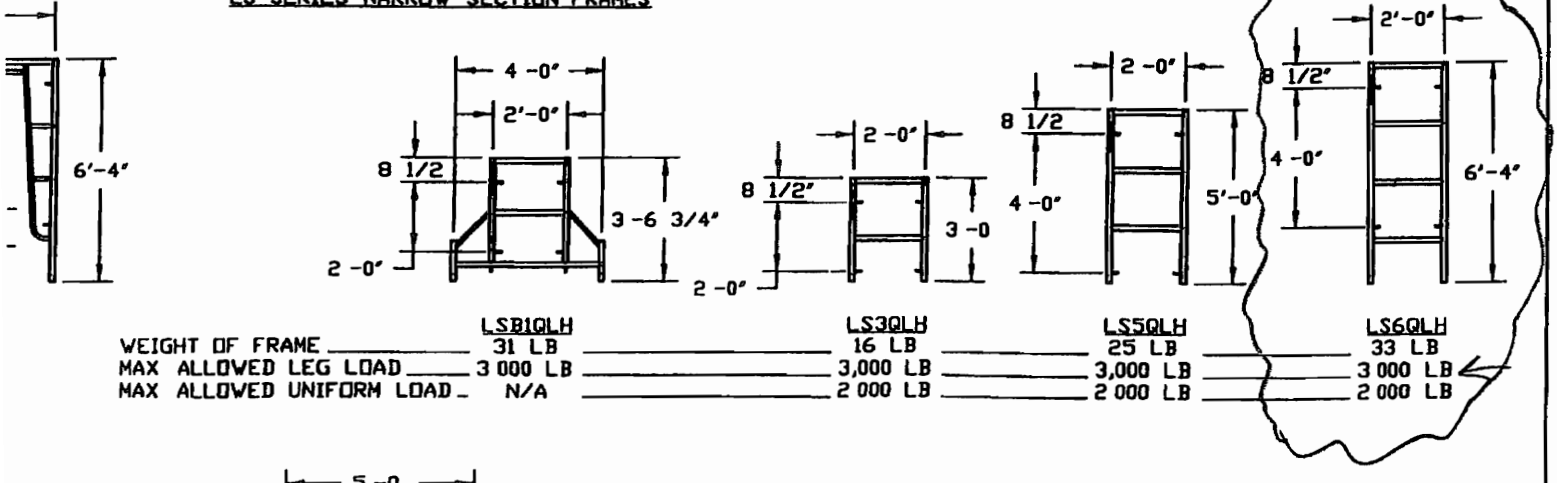
4) THESE COMPONENTS COMPLY WITH FED OSHA REQUIREMENTS WHEN PROPERLY ERECTED

THIS INFORMATION IS SUPPLIED FOR USE BY PERSONS KNOWLEDGEABLE IN SCAFFOLD DESIGN AND QUALIFIED TO EMPLOY GOOD ENGINEERING PRACTICE IN ITS APPLICATION

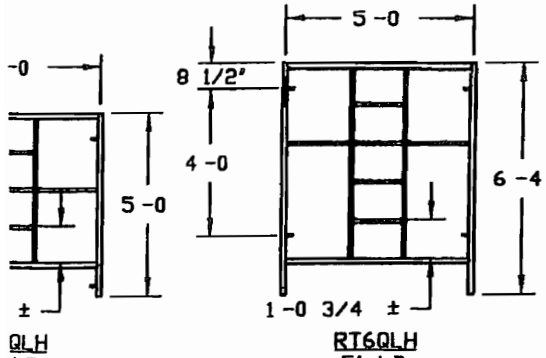
THIS DATA SHEET IS INTENDED FOR GENERAL INFORMATION ONLY BECAUSE OF THE MANY VARIABLES WHICH AFFECT THE PERFORMANCE OF THIS PRODUCT LINE SOME OF THE INFORMATION MAY NOT APPLY FOR SPECIFIC APPLICATIONS, SEE YOUR SAFWAY DEALER

NOTE: SAFWAY PRODUCTS AND EQUIPMENT MUST BE ASSEMBLED AND USED IN ACCORDANCE WITH ALL APPLICABLE FEDERAL, STATE, PROVINCIAL AND LOCAL REGULATIONS COPIES OF COMPLETE SAFETY GUIDELINES APPLICABLE TO THESE AND OTHER PRODUCTS ARE AVAILABLE FROM SAFWAY OR YOUR SAFWAY DEALER, WITHOUT CHARGE

LS SERIES NARROW SECTION FRAMES



	LSB1QLH	LS3QLH	LS5QLH	LS6QLH
WEIGHT OF FRAME	31 LB	16 LB	25 LB	33 LB
MAX ALLOWED LEG LOAD	3 000 LB	3,000 LB	3,000 LB	3 000 LB
MAX ALLOWED UNIFORM LOAD	N/A	2 000 LB	2 000 LB	2 000 LB



QLH	RT6QLH
LB	51 LB
LB	1 900 LB
LB	1,500 LB

SAFWAY

REV # 01

1/03 orn # 2101

DESCRIPTION: LOAD DATA SHEET STANDARD FRAMES

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SHEET 1 OF 1

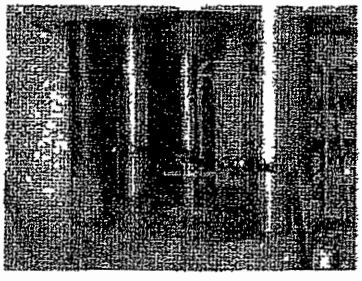
Test Report

IDENTIFICATION SJ1P

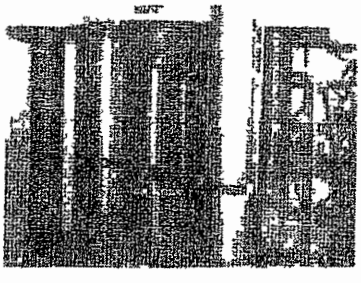
DESCRIPTION GALVANIZED SCREW JACKS WITH 6" 1/2 BASE PLATES

LOAD TEST RESULTS

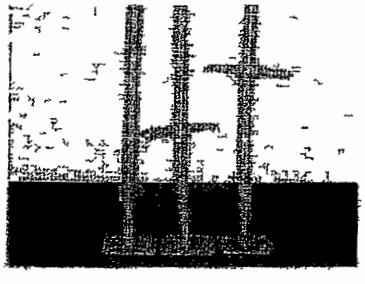
EXTENSION HEIGHT	TEST LOAD (lbs.)
6	56 66
12	49 595
18	46 385



Configuration of compressive load testing of SJ1P screw jacks 6" extension depicted in photograph. Same setup for 12" and 18" extension.



Screw jack at 6" extension at failure load of 56,400 lbs.



Deformation of 6", 12" and 18" extensions after yield.

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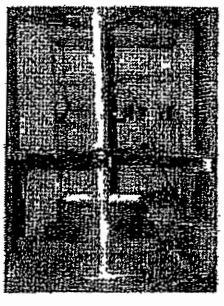
Test Report

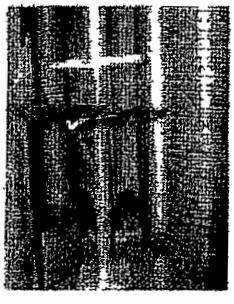
IDENTIFICATION SJ'S
DESCRIPTION GALVANIZED SCREW JACKS WITH SOCKET

LOAD TEST RESULTS

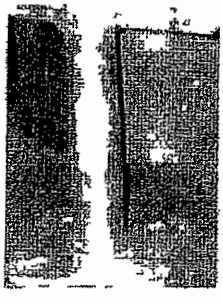
EXTENSION	MAXIMUM LOAD ATYIELD (lbs)	SAFETY FACTOR BASED ON YIELD (lbs)	FAILURE MODE
NONE	27 900	6 975	SOCKET TUBE COMPRESSION AND JACK DEFORMATION
MID EXTENSION 18" FROM BASE PLATE TO JACK HANDLE	31 450	7 863	SOCKET TUBE COMPRESSION AND JACK DEFORMATION
MAX EXTENSION 23" FROM BASE PLATE TO JACK HANDLE	31 700	7 925	SOCKET TUBE COMPRESSION AND JACK DEFORMATION



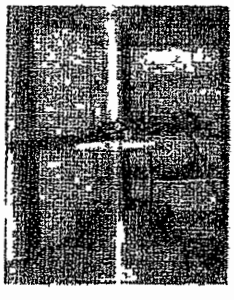
Test configuration with no extension



No extension yield load of 27 900 lbs



Mid extension yield load of 31 450 lbs



Max extension yield load of 31 700 lbs

Lower compressive strength value for jack with no extension is due to direct loading of screw jack handle onto socket tube weld

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(2.375"O.D. Shore, 1.9"O.D. Staff)

(Available Painted or Galvanized)

PSV 722(#4
10'6" TO 16' POST SHORE
WT 60.67 LBS

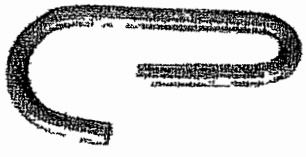
PSV 721(#3)
8'6" TO 13' POST SHORE
WT 53.45 LBS

PSV 720(#2)
6'6" TO 11' POST SHORE
WT 45.76 LBS

PSV-719(#1
5'9" TO 10'3" POST SHORE
WT 39.24 LBS

PSV 718(#0)
3'6" TO 6' POST SHORE
WT 26.49 LBS

PSV 158
5"X7" POST SHORE U HEAD
WT 7.88 LBS



PSV 194
POST SHORE HANDLE
WT 1.41 LBS

PSV 720 1B
POST SHORE STAFF W/
BASE PLATE
WT 20.24 LBS

PSV 720 1U
POST SHORE STAFF W/
U HEAD
WT 25.08 LBS

PSV 720HDU
6'6" to 11'H.D POST SHORE
W/ U-HEAD
WT 46.42 LBS

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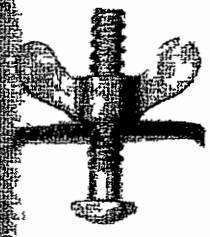
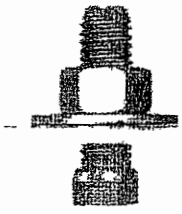
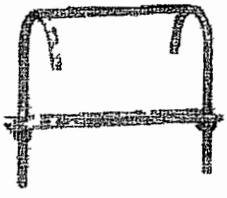


PSV-908A
C-CLAMP ASSEMBLY STEEL
WT 0 51 LBS



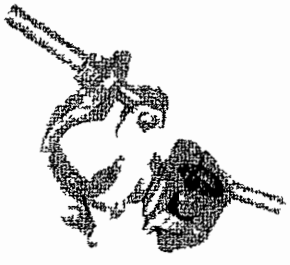
PSV 908B
#2 A CLAMP ASSEMBLY ALUMINUM
WT 0 3 LBS

PSV 2611
#2 C-CLAMP
WT 0 38 LBS



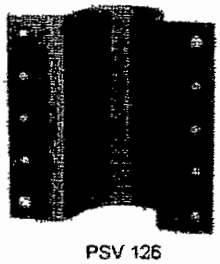
PSV 1301
TIMBER BRACING CLAMP
WT 2 71 LBS

PSV-886
BASE FRA
DER STYL
57 73LBS



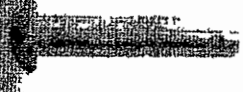
PSV 905
2 1/2"X2" SWIVEL CLAMP
WT 5 35 LBS

PSV 906
2 1/2"X2" RIGHT ANGLE CLAMP
WT 5 22 LBS

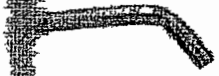


PSV 126
NAILING PLATE FOR
10K FRAME LEG
WT 1 19 LBS

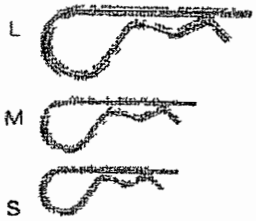
PIN
S



PSV 16
5/8 DRILLED RIVET
WT 0 35 LBS



PSV 129
J PIN
WT 0 55 LBS



PSV 117L	HITCH PIN WT 0 07 LBS
PSV 117M	HITCH PIN WT 0 05 LBS
PSV 117S	HITCH PIN WT 0 04 LBS

PSV 102
H L SHORING CONNECTOR
WT 0 92 LBS

PSV-102B
UNIVERSAL
SHORING CONNECTOR
WT 1 23 LBS

NECTOR
BS

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STRUCTURAL CALCULATIONS

Permit Submittal

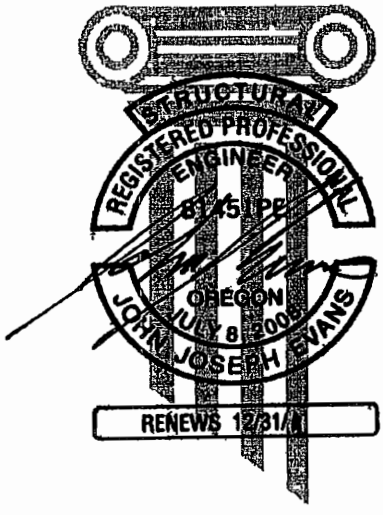
PARKING STRUCTURE II EPANSION
STRUCTURAL STEEL ERECTION BRACING

Portland, OR

For

ALLIANCE INDUSTRIAL GROUP

Date January 21, 2011
Project 2011005



☒ NO EXCEPTION NOTED ☐ MAKE CORRECTIONS NOTED
☐ REJECTED ☐ REVISE AND RESUBMIT

THIS REVIEW IS FOR GENERAL CONFORMANCE OF DESIGN LOADS AND METHODOLOGY ONLY ANY DEVIATION IN PLANS OR SPECIFICATIONS NOT CLEARLY NOTED BY T CONTRACTOR OR THEIR ENGINEER HAS NOT BEEN REVIEW REVIEW SHALL NOT CONSTITUTE A COMPLETE CHECK OF DETAILED CALCULATIONS OR SERVE TO RELIEVE T CONTRACTOR AND THEIR ENGINEER OF CONTRACTU RESPONSIBILITY FOR ANY ERROR OR DEVIATION FROM CONTRACT DOCUMENTS AND BUILDING CODE REQUIREMENTS

AAI ENGINEERING
Date 02/08/11 By Dick Larsen

FACILITIES ENGINEERING

STRUCTURAL ENGINEERING

PILLAR CONSULTING GROUP, INC.

MECHANICAL ENGINEERING

PROJECT MANAGEMENT

1600 SW Western Blvd Suite #290
Corvallis Oregon 97333

WWW PILLAR-INC COM

Phone (541) 752 9202
Fax (541) 752 2587

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Professional Engineering and Project Management

ENGINEERING MEMORANDUM

TO	Andrew Haman, Alliance Industrial Group, Inc		
FROM	John Evans, PE SE	**DATE**	January 21 2011
PROJECT	Parking Structure II Expansion – Structural Steel Erection Bracing, Portland, Oregon		
SUBJECT	Permit Submittal		

We have completed our analysis and design of the structural steel erection bracing plan for the Structural Steel Erection Bracing for the Parking Structure II Expansion in Portland, Oregon Recommendations regarding erection stability bracing, column anchorage, minimum beam connections and other information will be found in the attached 8-1/2" x 11" drawing packet (32 sheets, dated 1/21/11)

We have analyzed and provided recommendations for the following conditions, in accordance with ASCE 37-02 and AISC Design Guide 10

- Stability of columns during erection
- Temporary wire rope x-bracing for lateral wind resistance during steel erection
- Stability of steel frame during deck shakeout and anchorage (loading is limited to the weight of the deck bundles and a maximum of two iron workers within any single area bounded by the building gridlines at one time)

We have not analyzed the building structure for stability or fitness-for-purpose during construction of the elevated concrete floor and roof slabs We recommend that the contractor responsible for that portion of the work retain the services of a qualified engineer to assess the structure during this phase of work Our recommendations are applicable and valid only for erection stability of the steel frame up to and including shakeout of the steel floor deck

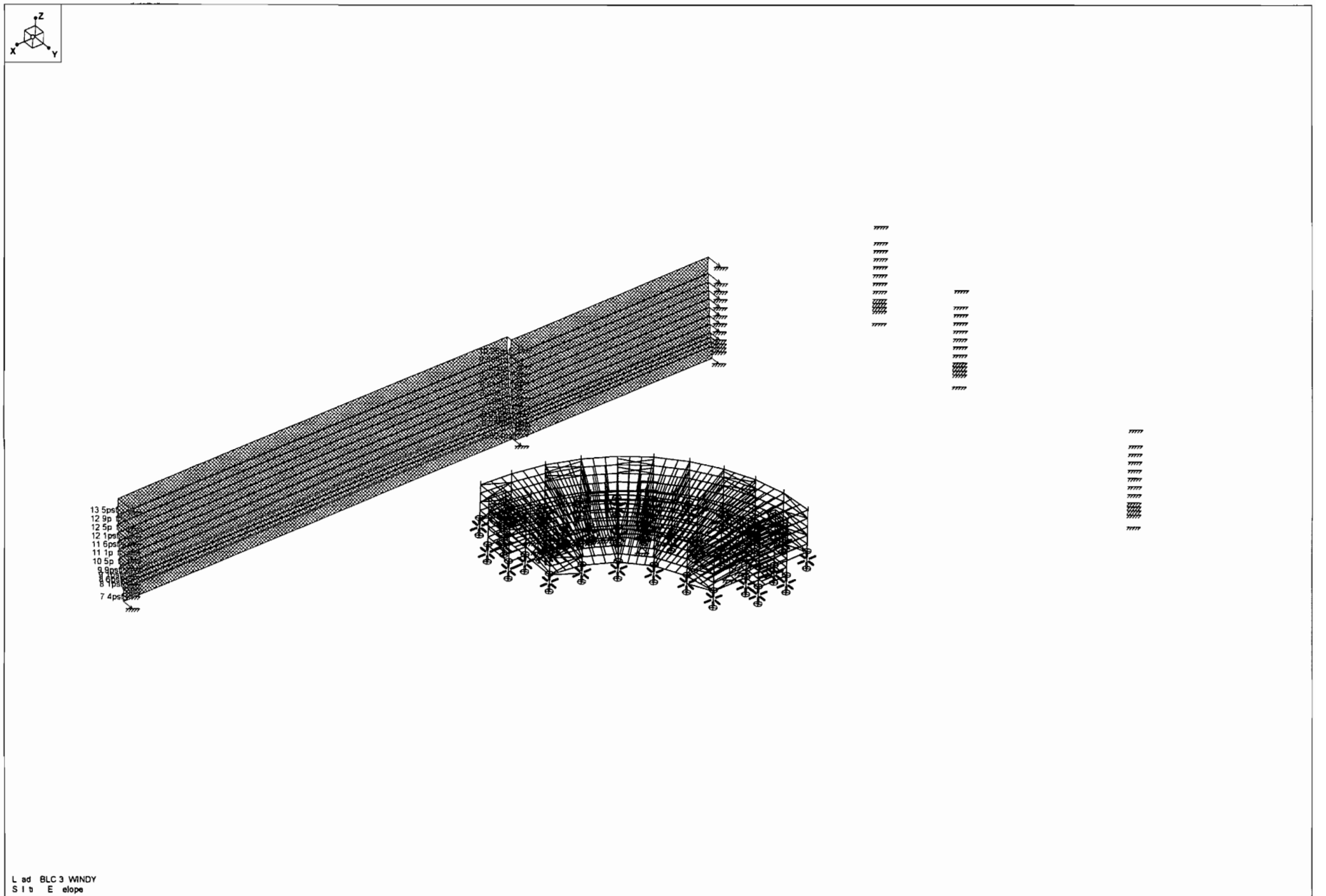
COPY TO			**PROJECT #**	20011005
VIA	☐ Fax	☐ US Mail	☐ Email	☐ N/A

1600 SW Western Blvd Suite 290 ■ Corvallis Oregon 97333 ■ www pillar inc com
Telephone (541) 752 9202 ■ Fax (541) 752 2587

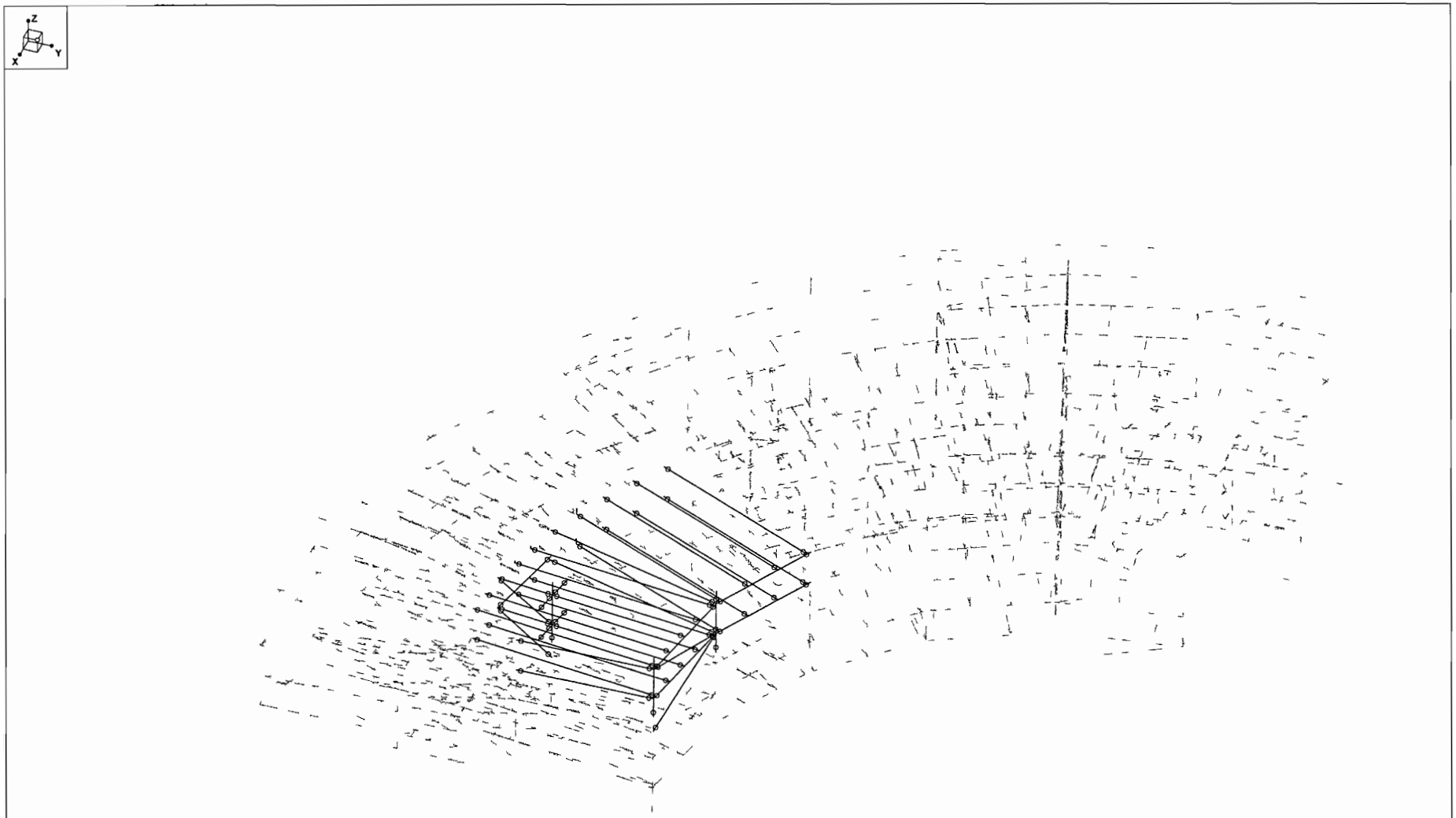
Basis of Design

1. RISA Model created to represent existing building
   a. All existing gravity framing added
   b. All existing braced frames added
   c. Concrete floors modeled as rigid diaphragm
   d. Boundary conditions set at the base of steel structure
2. RISA Model created for each zone of erection (7 zones total)
   a. Each zone was individually added to the previous and analyzed (ie Zone 1 added to existing building and analyzed, Zone 2 added to existing +Zone 1 and analyzed, etc
   b. Column Splices modeled as fixed and splice capacity verified
3. Model Information
   a. Loads
      i. Dead Loads
         1. Self Weight of Structural Framing
         2. Metal Deck
            a. Added as line load on beam
            b. Accounts for worst case bending and shear in member
            c. Drawings indicate deck bundle zones to ensure shear is not exceeded
      ii. Live Loads
         1. (1) 250# man load added to each end of beams
            a. Accounts for shear in beams
            b. Accounts for shear and flexure in girders
      iii. Wind Loads (See wind load calculations)
         1. Modeled as an area load against the face of framing members
         2. Per ASCE 37-02 75% of wind speed was used for construction period less than 6 weeks
         3. Shielding was accounted for per AISC Design Guide 10
      iv. Load Combinations
         1. 0 5 used as arbitrary point in time (APT) factor per ASCE 37 when combining Live and Wind Loads
         2. Wind X and Y combined at 100% / 50%
   b. Cellular Beams (see calculations)
      i. Cellular beams were modeled as solid members
      ii. depth giving an equivalent solid area for wind exposure
      iii. cross sectional area giving an equivalent weight

c Wire Ropes (see calculations)
  i ½" 6x19 IWRC (EIPS) used
    1 Factor of Safety = 3 on Rope Breaking Strength (AISC DG 10)
    2 Factor of Safety = 2 on Rope Breaking x Connection Efficiency
  ii ½" 6x19 Choker Used
    1 Factor of Safety = 2 (See AISC Design Guide 10)
  iii -100 Degree F load added = +/- 600 lb prestress

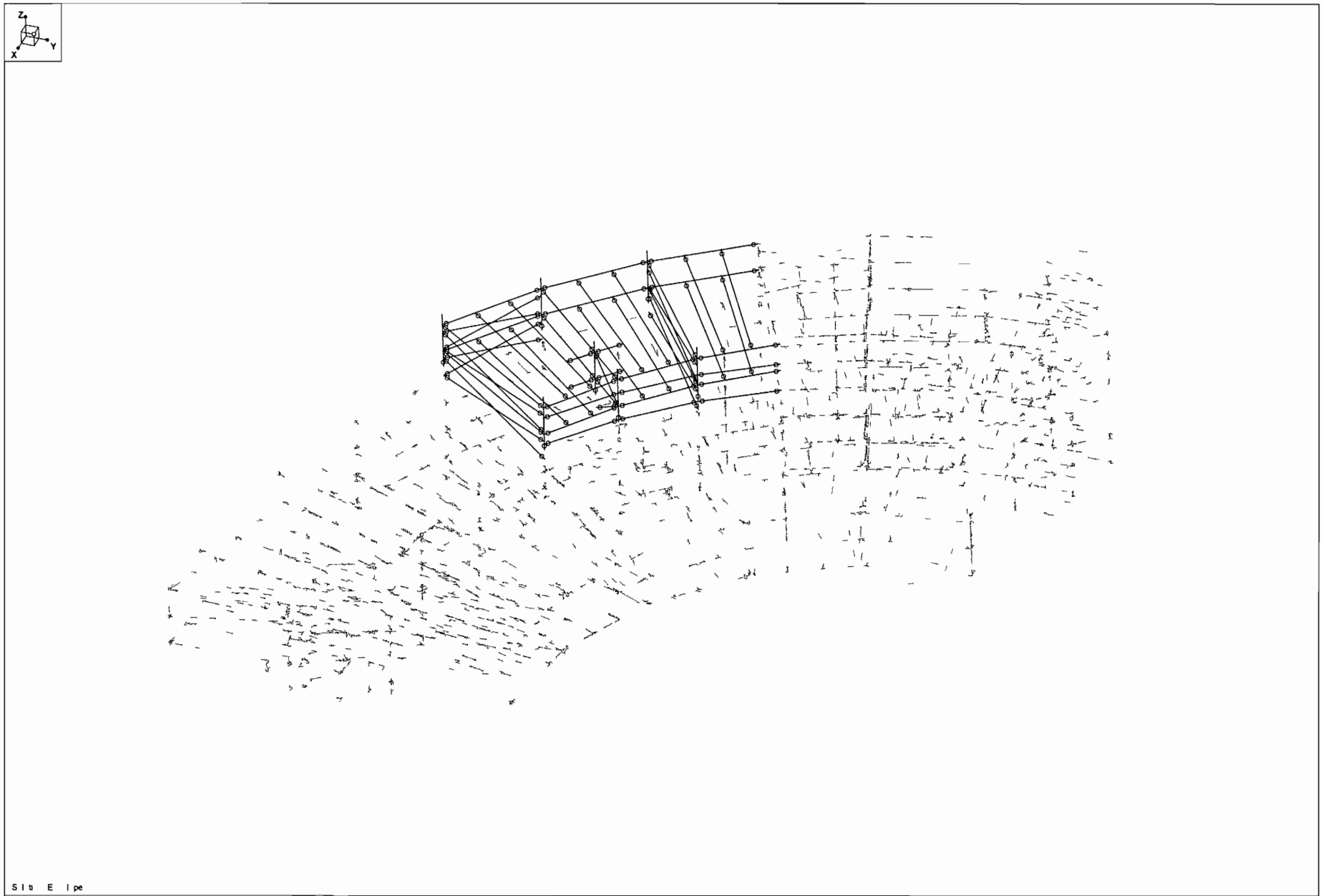


	WIND Y LOADING SHOWN	SK - 1
		Jan 21, 2011 at 4 00 PM
		zone 7 r3d



Solution E I pe

	ZONE 7 SHOWN IN BOLD	SK - 2
		Jan 21 2011 at 3 57 PM
		zone 7 r3d



Slu E lpe

	ZONE 2 SHOWN IN BOLD	SK - 3
		Jan 21, 2011 at 4 09 PM
		zone 2 r3d

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Analysis Summary

Zone 1

Grid 13 Brace Frame – Max Axial = 15 5k
Grid AA Brace Frame N – Max Axial = 13 1k
} < 34k – (2) BOLT BRACE CONN OK

Grid AA Wire Brace – Max Axial = 3 0k — < 8k – (1) 1/2" ϕ OK

Grid 11 Wire Brace – Max Axial = 2 6k
Grid 12 Wire Brace – Max Axial = 2 7k
Grid 14 Wire Brace – Max Axial = 9 3k
Grid 15 Wire Brace – Max Axial = 4 3k
Grid CC Wire Brace – Max Axial = 5 1k
Grid DD Wire Brace – Max Axial = 3 1k
Grid EE Wire Brace – Max Axial = 3 0k
} < 16k – (2) 1/2" ϕ OK

Zone 2

Grid 13 Brace Frame – Max Axial = 11 7k
Grid AA Brace Frame N – Max Axial = 18 7k
Grid 17 Brace Frame – Max Axial = 11 9k
} < 34k – (2) BOLT BRACE CONN OK

Grid AA Wire Brace 1 – Max Axial = 2 6k
Grid AA Wire Brace 2 – Max Axial = 6 8k
Grid 16 Wire Brace – Max Axial = 12 2k
Grid 18 Wire Brace – Max Axial = 10 5k
Grid CC Wire Brace – Max Axial = 4 5k
Grid DD Wire Brace – Max Axial = 5 2k
Grid EE Wire Brace – Max Axial = 3 8k
} < 16k – (2) 1/2" ϕ OK

Zone 3

Grid 13 Brace Frame – Max Axial = 30 1k
Grid AA Brace Frame N – Max Axial = 24 4k
Grid 17 Brace Frame – Max Axial = 24 9k
Grid 21 Brace Frame – Max Axial = 12 6k
} < 34k – (2) BOLT BRACE CONN OK

Grid AA Wire Brace 1 – Max Axial = 2 9k
Grid AA Wire Brace 2 – Max Axial = 6 8k
Grid AA Wire Brace 3 – Max Axial = 8 0k
Grid 19 Wire Brace – Max Axial = 9 5k
Grid 20 Wire Brace – Max Axial = 6 6k
Grid 21 Wire Brace – Max Axial = 12 2k
Grid CC Wire Brace – Max Axial = 4 5k
Grid DD Wire Brace – Max Axial = 5 6k
Grid EE Wire Brace – Max Axial = 4 0
} < 16k – (2) 1/2" ϕ IWRC OK

Zone 4

Grid AA Brace Frame N – Max Axial = 42k
Grid AA Brace Frame E – Max Axial = 41k
} WELD BRACES $\frac{42^k}{3.71^k/in}$ = 12" of 1/4" FILLET = 44.4k

Grid 21 Brace Frame – Max Axial = 8 2k
Grid AA Wire Brace 1 – Max Axial = 2 3k
Grid AA Wire Brace 2 – Max Axial = 3 0k
Grid AA Wire Brace 3 – Max Axial = 2 4k
Grid 22 Wire Brace – Max Axial = 3 3k
} < 16k – (2) 1/2" Φ ok

Grid 23 Wire Brace – Max Axial = 1 1k < 8k – (1) 1/2" Φ ok

Grid CC Wire Brace – Max Axial = 2 4k
Grid DD Wire Brace – Max Axial = 1 9k
Grid EE Wire Brace – Max Axial = 1 5k
} < 16k – (2) 1/2" Φ ok

Zone 5

Grid 13 Brace Frame – Max Axial = 12 0k < 34k – 2 BOLT BRACE CONN ok
Grid AA Brace Frame N – Max Axial = 36 3k – BRACE WELDED @ ZONE 4
Grid GG Brace Frame N – Max Axial = 9 1k – < 34k – 2 BOLT BRACE CONN ok

Grid 11 Wire Brace – Max Axial = 4 2k
Grid 12 Wire Brace – Max Axial = 7 3k
Grid 14 Wire Brace – Max Axial = 6 3k
Grid CC Wire Brace – Max Axial = 6 7k
Grid DD Wire Brace – Max Axial = 8 0k
Grid EE Wire Brace – Max Axial = 6 0k
} < 16k – (2) 1/2" Φ ok

Zone 6

Grid 13 Brace Frame – Max Axial = 12 4k < 34k – (2) BOLT BRACE CONN. ok
Grid AA Brace Frame N – Max Axial = 43k – BRACE WELDED @ ZONE 4
Grid GG Brace Frame N – Max Axial = 25 8k – < 34k – (2) BOLT BRACE CONN ok

Grid 17 Brace Frame – Max Axial = 25 6k
Grid 16 Wire Brace – Max Axial = 2 0k
Grid 18 Wire Brace – Max Axial = 2 2k
Grid CC Wire Brace – Max Axial = 6 9k
Grid DD Wire Brace – Max Axial = 7 4k
Grid EE Wire Brace – Max Axial = 5 9k
} < 16k – (2) BOLT BRACE CONN ok

Zone 7

Grid AA Brace Frame N – Max Axial = 34 4k
Grid AA Brace Frame E – Max Axial = 37 0
} BRACE WELDED @ ZONE 4

Grid GG Brace Frame N – Max Axial = 34 9k
Grid GG Brace Frame E – Max Axial = 37 0
} WELD BRACE

Grid 13 Brace Frame – Max Axial = 12 3k
Grid 17 Brace Frame – Max Axial = 25 3k
Grid 21 Brace Frame – Max Axial = 15 8k
} < 34^k (2) BOLT BRACE CONN ok

Grid AA Wire Brace 1 – Max Axial = 5 2k
Grid AA Wire Brace 2 – Max Axial = 5 9k
Grid AA Wire Brace 3 – Max Axial = 5 5k
Grid 19 Wire Brace – Max Axial = 7 4k
Grid 20 Wire Brace – Max Axial = 7 1k
Grid 22 Wire Brace – Max Axial = 8 9k
Grid CC Wire Brace – Max Axial = 7 1k
Grid EE Wire Brace – Max Axial = 5 3k
} < 16^k (2) 1/2" Φ WIRE ok

CONSTRUCTION WIND LOADS ON STEEL FRAME

$F = q_z G C_f A_f$ <ASCE 7-05, EQ 6-28>

6 WEEK DURATION → 0.75 (95 MPH)

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$

$V = 71$ mph
$K_{zt} = 1.0$
$I = 1.0$
$K_d = 0.85$

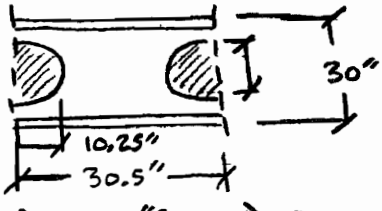
$q_z = 0.00256(1)(1)(85)(71)^2 (K_z)$
$q_z = 10.97 (K_z)$

$G = 0.85$

C_f = ASCE 7-05, FIG 6-22

$C_f = 1.4$ for solid ($w\,h/D \le 7$) (ASCE FIG 6-21)

for cellular beams



($\approx$ 24% WT REDUCTION FROM SOLID SHAPE)

$A_g = 30"(30.5") = 915\ \text{in}^2$
$A_n = A_g - \pi r^2 = 915 - \pi(10.25)^2$
$A_n = 585\ \text{in}^2$
$E = 585/915 = 0.64$

(36% AREA REDUCTION FOR WIND)

$\therefore C_f = 1.6$ per ASCE FIG 6-22

$\frac{1.6}{1.4} = 1.14$ $\quad 1.14(0.64) = 0.73$

$\therefore$ EQUIV. "SOLID" SHAPE WIND LOAD ON CELLULAR BEAMS $= 0.73(A_g)$(WIND PRESSURE)

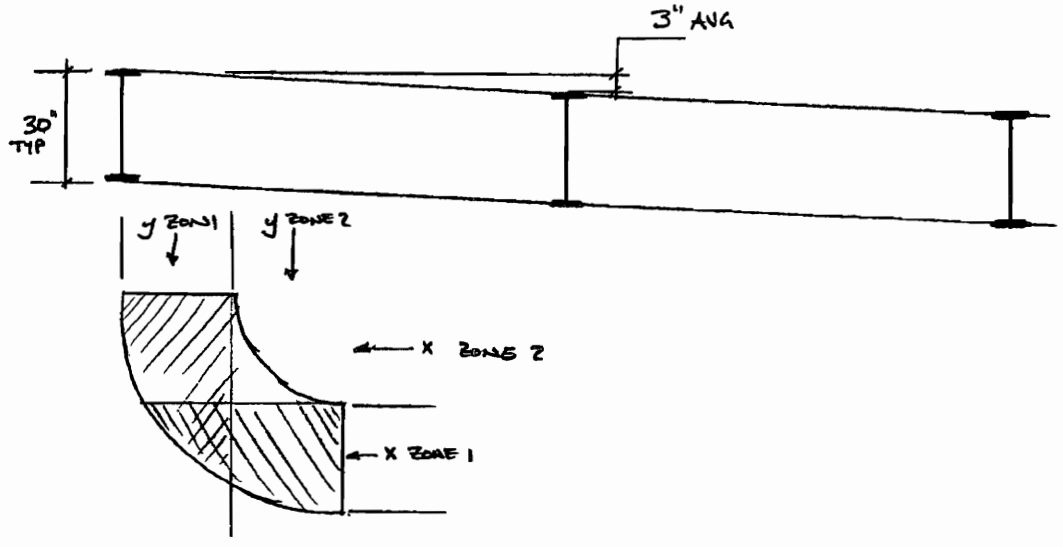
USE $F = 10.97(K_z)(0.85)(1.4)(A_f) = (K_z)(13)(A_f)$ SOLID MEMB.
$= 0.73(K_z)(13)(A_f) = (K_z)(9.5)(A_f)$ CELLULAR MEMB.
(OR SIMPLY 0.73 x SOLID)

SUBJECT	PROJECT#: 200907
	DATE.
	BY JWM
CONSTRUCTION WIND LOADS	SHEET: 1 OF

HT	Kz	SOLID MEMBER WIND LOAD (PSF)
0-15'	0.57	7.4 PSF
15'-20'	0.62	8 1
20'-25'	0.66	8 6
25'-30'	0.70	9.1
30'-40'	0 76	9 9
40'-50'	0.81	10 5
50'-60'	0.85	11.1
60'-70'	0 89	11.6
70'-80'	0.93	12.1
80'-90'	0.96	12.5
90'-100'	0 99	12 9
100'-120'	1.04	13 5

SUBJECT:	PROJECT#
	DATE
	BY
	SHEET: 2 OF

WIND REDUCTION FOR SHIELDING
PER AISC DG 10



X ZONE 2 & y ZONE 2 - NO SHIELDING

y ZONE 1 & X ZONE 1 - SHELDING

$a = 14'$ AVG $\}$ $a/d = 56$
$d = 30"$

SOLIDARITY RATIO = 0.64
PER FIG 32 OF AISC DG 10 - $n = 038$
$n = 8$ (AVERAGE)

$A_B = (1 + 038 + (8-2)(0.38)^2)A = 2.25 A_1$, $\frac{2.25}{8} A_1 = 028 A_1$

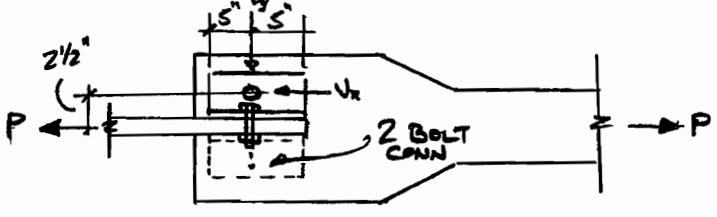
w/ $n = 7$	$\rightarrow A_7 = 2.1 A_x$,	$\frac{2.1}{7} A_1 = 03 A_x$
w/ $n = 6$	$\rightarrow A_6 = 1.96 A_x$	$= 033 A_x$
w/ $n = 5$	$\rightarrow A_5 = 181 A_x$	$= 0.36 A_x$
w/ $n = 4$	$\rightarrow A_4 = 167 A_x$	$= 042 A_x$
w/ $n = 3$	$\rightarrow A_3 = 1.52 A_x$	$= 0.51 A_x$
w/ $n = 2$	$\rightarrow A_2 = 138 A_x$	$= 0.69 A_x$
w/ $n = 1$	$\rightarrow A_1 = 10 A_x$	$= 10 A_x$

AVG DUE TO VERT STEP IN BEAMS = 0.49

USE 49% OF PROJECTED AREA OR $\frac{049}{064}$ = 076% ACTUAL AREA

SUBJECT	PROJECT#:
PARKING	DATE
	BY
	SHEET: OF

PERMANENT BRACE CONNECTION



w/ (1) BOLT CONNECTION (¾" ϕ 490 X)

IF P = 17^k, V_x = 17^k, V_y = 17^k $\left(\frac{2.5"}{5"}\right)$ = 8.5 SRSS = 19^k

NO SHEAR LAG EFFECTS (PER COMMENTARY)

¾" ϕ A325X – CAPACITY = 16.6 x 1.2 = 199^k

BOLT BEARING @ ⅜" ϕ = ⅜ (52.2^k/in) = 19.6^k ✓ OK
w/ 2" EDGE DIST

w/ (2) BOLT CONNECTION ((2) ¾" ϕ A490X)

IF P = 34^k, V_x = $\frac{34^k}{2}$ = 17^k

NO SHEAR LAG EFFECTS (PER COMMENTARY)

¾" ϕ A490X – CAPACITY = 16.6^k x 1.2 = 199^k

BOLT BEARING @ ⅜" ϕ = 17.6^k (SEE ABV) ✓ OK
w/ 2" EDGE DIST

SUBJECT	PROJECT#
POP II PARKING BRACE CONN.	DATE
	BY
	SHEET: OF

WIRE ROPE

- 1/2" IWRC (EIPS)

NOMINAL BREAKING STRENGTH = 26.6^k
T_{ALLOW} = 8.87^k

- 1/2" 6x19 CHOKER
CAPACITY = 1.6T = 3.2^k @ FS = 5 (PER CATALOG)
@ FS = 2 - CAPACITY = 3.2^k (5/2) = 8^k ← GOVERNS

USE 1/2" 6x19 IWRC w/ CHOKER
CAPACITY = 8^k

OR

(2) 1/2" 6x19 IWRC w/ CHOKER
CAPACITY = 16^k

SUBJECT:	PROJECT#
	DATE
	BY
	SHEET OF

Pillar Consulting Group, Inc
1600 SW Western Blvd Suite 290
Corvallis OR 97333
Phone (541) 752 9202
www pillar inc com

PROJECT# 200907
PROJECT POP II
DESIGNER JWM
DESCRIPTION 1/2-INCH DIA 6x19 IWRC EIPS

EFFECTIVE MODULUS & AXIAL STRETCH OF WIRE ROPE AS A FUNCTION OF AXIAL LOAD

Descrption & Assumptions

This template is calculates an effective modulus of elasticity for wire rope of a given diameter, secant modulus, and effective metallic area Preload and constructional stretch are incorporated into the axial deformation calculations The effective modulus is independant of the length but specific magnitude of stretch is not Ref AISC Design Guide 10 (Copyright 2003)

Wire Rope Input Data

Rope Diam	Rope Modulus	Metallic Area	Nom Brkg Str	Length	Preload	Constr Stretch	Descnption
$d_1 = 0\,50$ in	$E_1 = 15000$ ksi	$A_1 = 0\,118$ in^2	$NBS_1 = 26\,6$ kip	$L_1 = 32$ ft	$Pr_1 = 0\,0$ kip	$\%CS_1 = 0\,75\%$	1/2" dia IWRC EIPS 6x19

$FS_1 = 3$ factor of safety against NBS of rope $FS_2 = 2$ factor of safety against (NBS x connection effeciency)

Let $T_1 = 1$ kip, 2 kip NBS_1 Eff = 80% Connection efficiency of wire rope connection

$$\delta_1(L_1,T_1) = \begin{cases} \left[\frac{0\,2\,(NBS_1 - Pr_1)\,L_1}{A_1\,0\,9\,E_1}\right] + \left[\frac{[T_1 - 0\,2\,(NBS_1)]\left[L_1 + \frac{0\,2\,(NBS_1 - Pr_1)\,L_1}{A_1\,0\,9\,E_1}\right]}{A_1\,E_1}\right] + \left[\frac{T_1\,\%CS_1\,L_1}{0\,65\,NBS_1}\right] & \text{if } T_1 > 0\,2\,NBS_1 \\ \frac{T_1\,L_1}{A_1\,0\,9\,E_1} + \frac{T_1\,\%CS_1\,L_1}{0\,65\,NBS_1} & \text{otherwise} \end{cases}$$

$$E_{eff}(L_1,T_1) = \frac{T_1\,L_1}{A_1\,\delta_1(L_1,T_1)} \qquad E_{eff\_20\%NBS} = \frac{0\,2\,NBS_1\,L_1}{A_1\,\delta_1(L_1,0\,2\,NBS_1)} \qquad T_{20\%NBS} = 0\,2\,NBS_1$$

Specific Elongation Sample

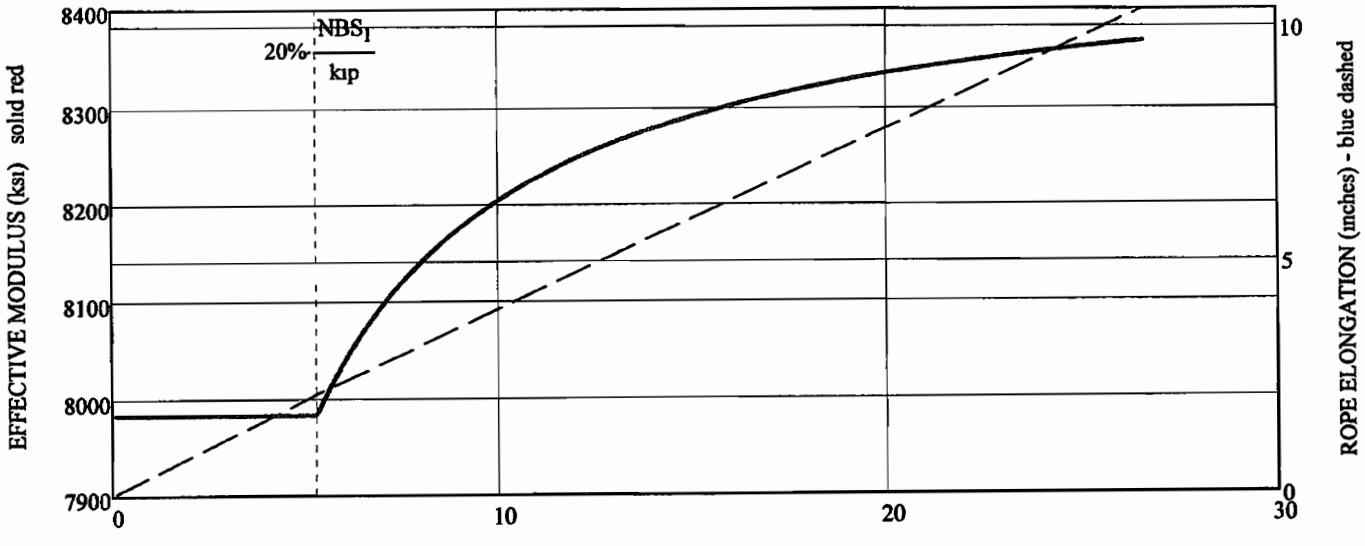
$\delta_1(L_1, 5\text{ kip}) = 2\,038$ in

$$d_{equiv} = \sqrt{\frac{4\,A_1}{\pi}} \qquad T_{allow} = \min\left(\frac{NBS_1}{FS_1}, \frac{NBS_1\,Eff}{FS_2}\right) \qquad f_{yequiv} = \frac{T_{allow}\,1\,67}{A_1}$$

$d_{equiv} = 0\,388$ in $f_{yequiv} = 125\,5$ ksi

$E_{eff\_20\%NBS} = 7983$ ksi $T_{20\%NBS} = 5\,32$ kip $\delta_{20\%NBS} = \delta_1(L_1, 2NBS_1) = 2\,17$ in $\delta_{NBS} = \delta_1(L_1, NBS_1) = 10\,35$ in $T_{allow} = 8\,87$ kip

EFFECTIVE MODULUS OF ELASTICITY



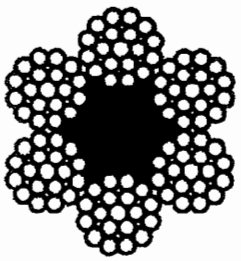
West Coast Wire Rope

WIRE ROPE

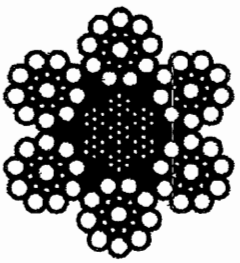
West Coast Wire Rope

GENERAL PURPOSE

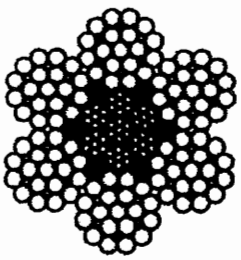
BRIGHT 6X19 CLASS (6x19, 6X21, 6X25, 6X26)



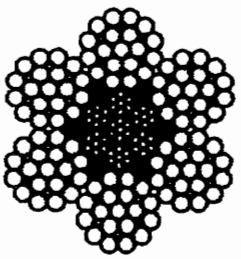
6 x 25 FILLER WIRE
FIBER CORE



6 X 19 SEALE
IWRC



6 X 25 FILLER WIRE
IWRC



6 X 26 IWRC
WARRINGTON SEALE

(IWRC INDEPENDENT WIRE ROPE CORE OR CENTER)
According to Federal Specifications RR-W-410D, preformed right regular lay light to heavy lubrication

Read important warnings and information on pages 6 - 7 and 12 preceding wire rope section

5/16	0 164	4 69	0 18	5 27
7/16	0 32	9 09	0 35	10.2
9/16	0 53	14 09	0 59	16 8
3/4	0 95	26 2	1 04	29 4
1	1 68	46	1 85	51 7
1 1/4	2 63	71	2 89	79 9
1 1/2	3 78	101	4 16	114
1 3/4	5 15	136	5 67	153
2 1/4	8 51	220	9 36	247
2 3/4	-	-	14	361
3 1/2	-	-	22 6	564

NBS / = 266 ℄

NOTE Lang Lay, left lay, alternate lay seale available in some sizes
Heavy lubrication on request

*Listed for comparison only Actual operating loads may vary but should never exceed the recommended design factor or 20% of catalog Breaking Strength

West Coast Wire Rope

SLINGS
WIRE ROPE

West Coast Wire Rope

WORKING LOAD LIMITS
HAND TUCKED SPLICE
FIBER CORE & IWRC

3

1-PART SLINGS
VERTICAL, CHOKER OR VERTICAL BASKET

RATED CAPACITY IN TONS OF 2,000 LBS

RATED CAPACITIES SHOWN APPLY ONLY TO 6X19 AND 6X37 CLASSIFICATION WIRE ROPE

ROPE DIA. (IN)	VERTICAL	CHOKER	BASKET
	EIPS	EIPS	EIPS
5/16	0 83	0 66	1 7
7/16	1 6	1 3	3 2
9/16	2 5	2 1	5 0
3/4	4 3	3 7	8 6
1	7 4	6 4	15
1 1/4	11	9 9	23
1 1/2	16	14	32
1 3/4	22	19	44
2	28	25	56
2 1/4	35	31	70
2 1/2	43	38	86

8K MAX CAP

13K MAX CAP

18.5K MAX @ FS=2

RATED CAPACITIES BASKET HITCH BASED ON D/d RATIO OF 15
RATED CAPACITIES BASED ON PIN DIAMETER NO LARGER THAT NATURAL EYE WIDTH OR LESS THAN THE NOMINAL SLING DIAMETER
RATED CAPACITIES BASED ON DESIGN FACTOR OF 5
HORIZONTAL SLING ANGLES LESS THAN 30 DEGREES SHALL NOT BE USED

DRAG LINES

WIND ∥ TO GRIDS @ TEMPORARY BRACE FRAME

WIND = 11.1 (A_F) @ T5, T6 & T7

COLUMN 11.1 psf (15"/12"/') (2) (10.5') = 291#
GIRDERS 11.1 psf (27"/12) (37.8') (3') = 2832
3123#

BEAMS ARE ∥ TO WIND NO WIND CONTRIBUTION FROM BEAMS

AXIAL STRESS IN BEAM = $\frac{3123^{\#}}{13^{\square\square}\ (TYP)}$ = 240 psi

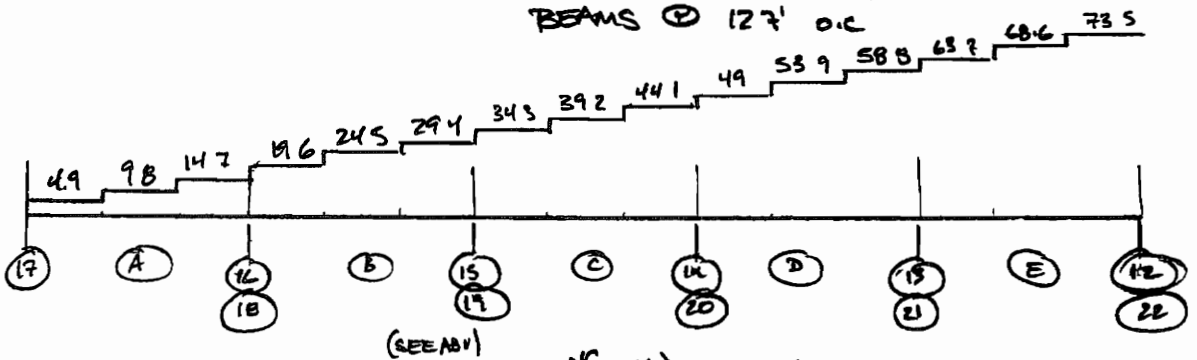
NEGLIGIBLE ✓ ok

GRAVITY = DECK 34 plf (60'/2) + MAN 350# + SELF 625 plf (60'/2) = 3245# (EA END)

(2) 7/8" Φ A325 IN 3/8" MIN PL OK BY INSP

WIND ∥ TO GRID AA
DUE TO STIFFNESS - ASSUME ALL LOAD GOES TO PERMANENT BRACE AT END (CONSERVATIVE)
WIND = 11.1 A_F
WIND RXN @ BM END = 0.76 (0.64) (30") (11.1 psf) (60'/2) = 4.9k
SHIELDING
BM OPENINGS CASTELATED

BEAMS @ 12.7' O.C



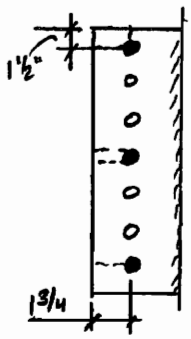
(SEE ABV)
GRAVITY = 3245# + 84 plf (38'/2) = 4.8k ↓

SUBJECT	PROJECT#:
COLLECTORS	DATE:
	BY:
	SHEET: OF

DRAG LINES CONTINUED

WIND ⊥ TO GRID AA CONT.

TYPICAL CONNECTION W27x84



W27x84 BRACED @ 12.5 o.c.

$\frac{kl}{r} = \frac{1(12.5')(12")}{207} = 72.5$

$\frac{F_{cr}}{\Omega} = 20.4^{ksi}$

$P_{max} = 20.4^{ksi}(24.8^{in^2}) = 506^k > 73.5^k$ ✓ ok

W/ NOMINAL GRAVITY LOAD
W27x84 ok

BOLTS - (3) 7/8" Ø A325 N

$\frac{R_n}{\Omega} = (14.4^k)(3) = 43.2^k$ ← GOVERNS

BEARING - @ 1¼" = 27.2^k
@ 2" = 58.3^k } INTERPOLATE → 1¾" = 44.6^k

$(3)(44.6^k)(3/8") = 50^k$

BLOCK SHEAR: @ 3 BOLTS

$A_{gv} = 1.75"(3/8")(4) = 2.626^{in^2}$
$A_{nv} = (1.75" - 13/32")(3/8")(4) = 1.922^{in^2}$
$A_{gt} = 1.5"(3/8")(2) = 1.125^{in^2}$
$A_{nt} = (1.5" - 13/32")(3/8")(2) = 0.773^{in^2}$
$U_{bs} = 1.0$
$F_u = 58^{ksi}$
$F_y = 36^{ksi}$

$\frac{R_n}{\Omega_1} = 111.7^k$, $\frac{R_n}{\Omega_2} = 101.5^k$

SUBJECT	PROJECT#
COLLECTORS CONT	DATE
	BY
	SHEET: OF

DRAG LINES CONT

WIND ⊥ TO GRID AA CONT

GRIDS 15-19 – HORIZ SHR = 34.3^k
VERT SHR = 4.8^k
SRSS = $\sqrt{34.3^2 + 4.8^2}$ = 34.6^k < 50^k

(3) 7/8" ϕ BOLTS OK

GRIDS 12-14 & 20-22
HORIZ SHR = 73.5^k
VERT SHR = 4.8^k
SRSS = $\sqrt{73.5^2 + 4.8^2}$ = 73.7^k

(6) 7/8" ϕ A325N BOLTS

BOLTS – $\frac{R_n}{\Omega}$ = 14.4^k (6) = 86.4^k √OK

BEARING
EDGED DST – (6)(44.6^k)(7/8) = 100^k √OK
SPACING – (6)(42.1^k)(7/8) = 94.7^k √OK
↑ FOR 3" O.C.

BLOCK SHEAR – OK BY INSP
(SEE 8 BOLT COND.)

GRID GG
PER GRID A – (6) BEAMS CAN TAKE WIND LOAD W/ (3) BOLT CONDITION (COUNTING FROM MIDPOINT OF GRID LINE LENGTH
∴ PROVIDE (6) BOLTS @ GRIDS: 13, 14, 21 & 22

SUBJECT.	PROJECT#
COLLECTORS, CONT	DATE.
	BY
	SHEET OF

DRAG LINES CONT

① # GRIDS - MAX BRACE FRAME AXIAL = 30.1^k ② ZONE 3, GRID 1B

BY INSP - IF BRACE FORCE = 30.1^k
THEN COLLECTOR = < 30.1^k

(3) 7/8" ϕ A325 N ok

SUBJECT: COLLECTORS	PROJECT#
	DATE
	BY:
	SHEET: OF

COLUMN SPLICE

WORST CASE: W14x193 CANTILEVERS 21'

WIND = 11.6 PSF
$d = 15.5"$
$t_f = 15\ 3/4"$

$M = 11.6^{psf}(15.75"/12"/1')(21')(21'/2)$

$= 34^{k\,ft}$ ✓ ok

(SEE ATTACHED FOR CAPACITIES)

HSS 8x8 CANTILEVERS 21'

WIND = 11.6 PSF

$M = 11.6^{psf}(8"/12"/1')(21')(21'/2) = 17^{k\,ft}$

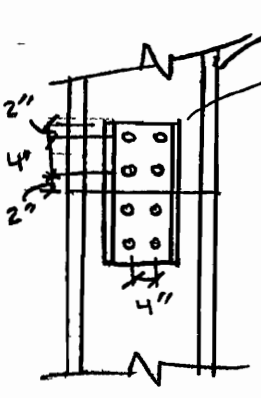
SHEAR ON BOLTS $= \frac{17^{k\,ft}(12"/1')}{16"}$

$= 1275^{\#}$

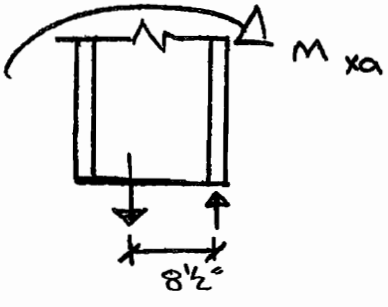
ok BY NSD

SUBJECT	PROJECT#
	DATE:
	BY.
	SHEET: OF

COLUMN SPLICES:



COLUMN STRONG AXIS MOMENT CAPACITY OF SPLICE



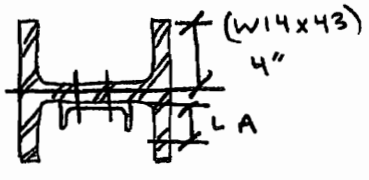
$\frac{R_n}{\Omega}$ BOLTS IN SHEAR = (2)(10.6 K) = 21.2 K

$M_{xa(MAX)}$ = (8½)(21.2) = 180 K·in

$M_{xa\,MAX}$ = 15 K·ft

$\frac{R_n}{\Omega}$ BOLTS IN BRG = (52.2 K/inch)(0.285)(2)
= 29.8 K DOES NOT CONTROL

WEAK AXIS MOMENT CAPACITY OF SPLICE



(MILLED SURFACE)

t_f = 0.53" min

t_w = 0.305" min

$\frac{R_n}{\Omega}$ (BOLTS) = (4)(10.6) = 42.4 K

BRG AREA REQ'D FOR 42.4 K

$\frac{R_n}{\Omega} = \frac{1.8 F_y A_{pb}}{\Omega}$ <AISC 13th J7-1>

A_{pb} req'd = (42.4)(2)/1.8(50) = 0.94 ϕ"

$LA = 4" - \frac{0.305"}{2} - \frac{0.94\,\phi"}{(2)(0.53")} = 2.96"$

204005

SUBJECT: COLUMN SPLICE MOMENT CAP.	PROJECT#: [redacted]
	DATE: 4-1-09
	BY: JWM
	SHEET: 1 OF

$M_{yg\text{-}MAX} = (42.4^K)(2.96'') = 125\tfrac{1}{2}$ K-in

$M_{yg\text{-}MAX} = 10.5$ K-ft

CHECK TENSION CAP OF C9x15

$A_g = 4.41\ in^2$ $F_y = 36$ ksi
$A_n = 3.91\ in^2$ $F_u = 58$ ksi
$t_w = 0.285$ in

$R_n/\Omega = \frac{F_y A_g}{1.6} = \frac{36(4.41)}{1.6} = 99^K$ OK

$R_n/\Omega = \frac{F_u A_e}{\Omega}$ $A_e = UA_n$ $U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = 0.586$ $l = 4''$

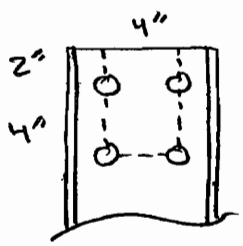
$U = 1 - \frac{.586}{4}$

$U = 0.85$

$= \frac{(58)(3.91)(0.85)}{2.0}$

$R_n/\Omega = 96.8^K$ OK

CHECK BLOCK SHEAR:



$F_y A_{gv} = (36)(3.42\ \#') = 123^K$ ← controls
$F_u A_{nv} = (58)(2.67\ \#') = 155^K$

$U_{bs} = 0.5$ STRONG AXIS moment
1.0 WK AXIS moment

$F_u A_{nt} = (58)(0.89\ \#^2) = 51.7^K$

$\frac{R_n}{\Omega}$ (STRONG AXIS) $= \frac{123 + 0.5(51.7)}{2} = 74.4^K$ (DOES NOT CONTROL)

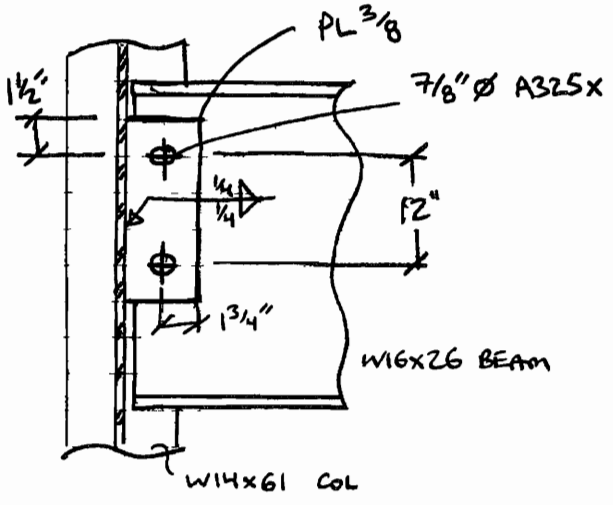
$\frac{R_n}{\Omega}$ (WK AXIS) → DOES NOT control by insp.

∴ MAX ALLOW MOMENT ON COLUMN SPLICE DURING ERECTION: 15 K-ft STRONG-AXIS
10½ K-ft WK-AXIS (8" flg)

SUBJECT	PROJECT#: 2011005
	DATE: 4-1-09
	BY: JWM
COL. SPLICE MOMENT CAP	SHEET: 2 OF

CONTINUOUS WORK PERIOD FRAME CHECK

TYPICAL CONNECTION:



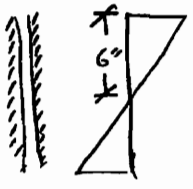
BOLT SHEAR

BOLT R_{nv}/Ω = 18K

M_n/Ω for bolts = (18)(1')

M_n/Ω = 18 K-FT (BOLTS)

FLAG WELD-TO-COL



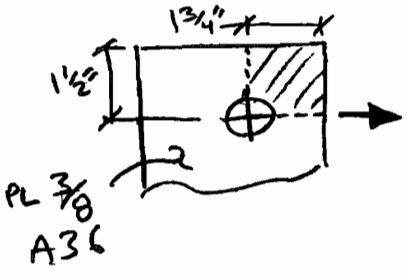
(1.5)(0.928)(4)(2) = 11.14K

33.42K

33.42K

M_n/Ω = 16.7 K-FT (WELDS)

BLOCK SHEAR OF TAB:



$0.6F_yA_{gv} = 0.6(36)(1.75)(0.375) = 14.18^K$ ← controls

$0.6F_uA_{nv} = 0.6(58)(1.25)(0.375) = 16.31^K$

$U_{bs}F_uA_{nt} = 0.5(58)(1)(.375") = 10.875^K$

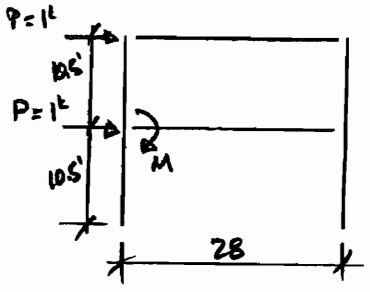
$\Omega = 2.0$ $\frac{R_n}{\Omega} = 12.5^K$

$\frac{M_n}{\Omega} = (12.5\ K)(1')$ = 12.5 K-FT (BLOCK SHEAR)

controls

ALLOW TEMP. MOMENT CAPACITY @ BM-TO-COL CONN ≈ 12.5 K-FT

SUBJECT	PROJECT#.
	DATE
	BY.
	SHEET 1 OF



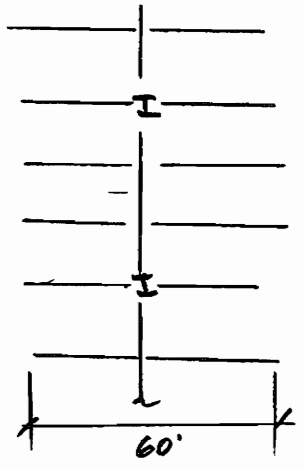
w/ FIXED BASE - $M = 4.9^{k\cdot ft}$

w/ PINNED BASE - $M = 10.6^{k\cdot ft}$ } $15.5^{k\cdot ft}$ AVERAGE

ASSUME PINNED BASE

$$P_{MAX} = \frac{12.5^{k\cdot ft}}{10.6^{k\cdot ft}}(1^k) = 1200^{\#}$$

WIND TRIB TO FRAME



EQUIV SOLID FOR CASTELATED

$0.73(3)(30''/12'')(60') + 2(14''/12'')(10.5) = 341\ \#/\text{TIER}$

$$\frac{1200^{\#}}{341^{\#}} = 3.5^{psf} = P$$

$P = q_z G C_f$, $C_f = 1.4$, $G = 0.85$

$3.5 = q_z (0.85)(1.4)$

$q_z = 3.0^{psf}$

$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$

$V_{MAX} = \sqrt{3.0 / [0.00256(0.96)(1.0)(0.85)(1.0)]}$

$= 38$ MPH

NO BRACING REQ'D FOR GUSTS < 38 MPH

USE 30 MPH

SUBJECT	PROJECT#:
	DATE:
	BY:
	SHEET: OF

CELLULAR BEAMS

CREATE SECTION W/ SIM WT. & WIND EXPOSURE AS CELLULAR BEAM

CELL BEAM	WT	d	0.75d (EQUIV SOLID AREA)	AREA
LB30x57/68	62.5	30.6	22.3	183#
LB30x57/62	59.5	30.5	22.3	175#
LB30x57	57	30.5	22.3	168#
LB30x50/57	53.5	30.4	22.2	157#
LB30x44/50	47	30.2	22.0	138#
LB21x22/26	24	20.2	14.7	71#
LB18x22	22	17.8	13.0	65#

SUBJECT: PARKING STRUCT CELLULAR BEAMS	PROJECT# 201005
	DATE.
	BY
	SHEET: OF

SHORING @ CC.5/12, DD 3/12, CC 5/22, DD.3/22

DL: FRAMING - 5 psf
DECKING - 3 psf
MISC - 2 psf
10 psf

LL: CONST - 25 psf

TRIB AREA = 12' x 21½ = 126 ft²

LOAD = (10 psf + 25 psf)(126 ft²) = 4410#

SPECIFY 5k SHORING LOAD

SUBJECT	PROJECT#:
	DATE
	BY
	SHEET OF

METAL DECK INSTALL & BRACE REMOVAL

// TO LETTER GRID

GRID AA & GG - WIND TRANSFERS DIRECTLY TO GIRDER
∴ NO DIAPHRAM FORCES

GRID DD - FRAME WILL LIKELY TAKE ALL FORCES
BUT; SAY DIAPH CANTILEVERS OFF GRIDS AA & GG

OVER (1) BAY, WIND HITS (6) BEAMS

$0.49\,(0.73)(11.1^{psf})(30''/12''/')(60'/2)(6) = 1786^{\#}/BAY$
SHIELDING EQUIV DEPTH

$\frac{1786^{\#}}{18'} = 100^{plf} < 137^{plf}$ ✓ ok

// TO # GRIDS

$11.1^{psf}(27''/12''/')(2)(37.67'(\frac{4}{2})) = 3763^{\#}$ TO FRAME LINE

$\frac{3763^{\#}}{60'} = 63^{plf} < 137^{plf}$ ✓ ok

3W x 20 GA DECK w/
1 PDF @ EA EDGE @ EA SUPPORT
& BUTTON PUNCH @ 36' O.C ok

GRID 17 BF -
BEAM AREA = $(27''/12 \times 37.67' \times 4 + 21''/12 \times 28.3' \times 4) = 537^{\#}/TIER$
COL AREA = $(10.5')(15''/12''/')(3.5) = 46^{\#}/TIER$
$583^{\#}$
$11.1^{psf}(583^{\#})(2) = 129^{k}/BRACE$ - TEMP CONN OK
2 TIERS

GRID AA BF
SAME @ GG
BEAM AREA = $(0.73(30'/12)(60')(16)) = 1752^{\#}/TIER$
COL AREA = $(10.5)(15''/12''/')(3.5)(2) = 92^{\#}$
$1844^{\#}$
$11.1(1844^{\#})(2) = 41^{k}/BRACE$ - 12" OF 1/4" FILLET OK = 44^{k}
2 TIERS

SUBJECT:	PROJECT#.
	DATE:
	BY:
	SHEET: OF

1 PDF @ Ea Edge @ Ea Support, Button Punch @ 36" oc

Connection Variables

t	0 047	inches	Deck
l	42 67	feet	Deck
h	3	inches	Deck
w	36	inches	Deck
np	3		# Purlins
s	36	inches	BP Spacing
#P	2		# Pins
x1	16	inches	Pin Distance to Center of Deck
x2	16	inches	Pin Distance to Center of Deck
x3	0	inches	Pin Distance to Center of Deck
x4	0	inches	Pin Distance to Center of Deck
A	1		Usually, if not, its Conservative

Deck Variables

d	12	inches	flute pitch
e	2 25	inches	1/2 lower flute flat
w	3 35	inches	diag dim
f	4 5	inches	upper flute flat
I	1 203	in^4/ft	

Safety Factors

Ωc	2 35	ULT->ASD	FS Connections
Ωc	2	ULT->ASD	FS Deck

Solved Values

Qf	2329 132	lbs	pin
Qs	530 16	lbs	BP
αs	0 23		
lv	10 67	feet	span
λ	0 38		
$\alpha 1$, $\alpha 1$	0 89		
ne, ns	14 00		# BP in a deck piece
B	7 14		
N	0 33	per foot	# pins per foot
s	15 7	inches	developed flute width

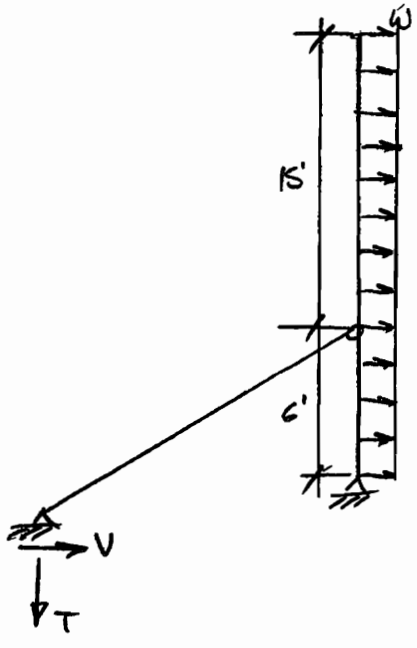
Connections

Sne	1006 8	plf
Snl	322 4	plf
Snc	348 2	plf

Deck

Snb	3096 4	plf

Governing	322 4	plf	ULT
ASD	137 2	plf	ASD



DECK = 3" w/ 3½"
NWT CONC (3000#)

$w = 11.6^{psf}(15/12"/') = 14.5^{plf}$

$V = 533^{\#}$
$T = 267^{\#}$

3/8" FORGED EYE BOLT
IN 3/8" TITEN HD ROD COUPLE

PER SIMPSON CATALOG
3/8" ϕ x 2 3/4" EMBED
ALLOW $V = 1060^{\#}$
ALLOW $T = 640^{\#}$ } FS = 40

$\frac{533}{1060} + \frac{267}{640} = 0.92$ ✓ ok
$FS = 4/0.92 = 4.34$ ✓ ok

FORCE IN ROPE =
$\sqrt{533^2 + 267^2} = 600^{\#}$

(1) ½" ϕ ok

w/ BRACE @ 1:1
$V = 533^{\#}$
$T = 533^{\#}$

$\frac{533}{1060} + \frac{533}{640} = 1.34$

$FS = \frac{4}{1.34} = 3.0$ ✓ ok

FORCE IN BRACE = $\sqrt{2}(533^{\#})$
$= 754^{\#}$

SPECIFY $2000^{\#}$

SUBJECT:	PROJECT#:
	DATE
	BY
	SHEET OF

WDY Structural • Civil Engineers

STRUCTURAL CALCULATIONS

FOR

PROVIDENCE MEDICAL CENTER POP2
CONSTRUCTION PROTECTION CANOPIES

PORTLAND, OREGON

DECEMBER 22, 2010
(REVISED FEBRUARY 21, 2011 – REVISED PAGES ONLY!)

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STRUCTURAL
REGISTERED PROFESSIONAL
ENGINEER
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MEIKO WINNIG
EXPIRES 06-30-2012

☒ NO EXCEPTION NOTED ☐ MAKE CORRECTIONS NOTED
☐ REJECTED ☐ REVISE AND RESUBMIT

THIS REVIEW IS FOR GENERAL CONFORMANCE OF DESIGN LOADS AND METHODOLOGY ONLY ANY DEVIATION FROM PLANS OR SPECIFICATIONS NOT CLEARLY NOTED BY THE CONTRACTOR OR THEIR ENGINEER HAS NOT BEEN REVIEWED REVIEW SHALL NOT CONSTITUTE A COMPLETE CHECK OF DETAILED CALCULATIONS OR SERVE TO RELIEVE THE CONTRACTOR AND THEIR ENGINEER OF CONTRACTUAL RESPONSIBILITY FOR ANY ERROR OR DEVIATION FROM CONTRACT DOCUMENTS AND BUILDING CODE REQUIREMENTS

AAI ENGINEERING

Date 02/23/2011 By [signature]

DESIGN PARAMETERS **IBC 2009 with OSSC 2010 Amendments**

PEDESTRIAN PROTECTION LOAD	150 psf

CONTENTS

CANOPY DESIGN 6 7 8a 8b

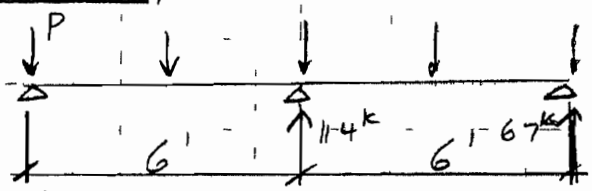
6443 SW Beaverton Hillsdale Hwy suite 210 Portland OR 97221 ph 503 203 8111 fx 503 203 8122 www wdyi com

16-199336 DMO 01 CO

WALK-THRU CANOPY, DESIGN, CONT,

(4.) DBL. TITAN 225 SUPPORT BEAM,

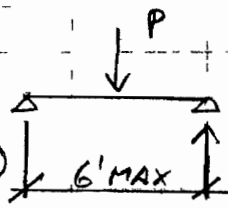
$P = 4.76^{k}$



$\Rightarrow$ MAX $M^{+} = 5,873$ FT-# $< 21,020$
MAX. $M^{-} = 5,419$ FT-# $< 21,020$
MAX. $R_{END} = 6,726^{\#} < 20,000^{\#}$
MAX. $R_{INT} = 11,411^{\#} < 20,000^{\#}$
} (OK) (SEE 18)

(5.) SINGLE TITAN 225 CROSS-BEAM,

$P_{MAX} = 11,411^{\#}$



$\Rightarrow$ MAX. $M^{+} = \frac{11,411(6)}{4} = 17,117$ FT-# $< 21,020$ (OK) △1

MAX. $R = \frac{11,411}{2} = 5,706^{\#} < 20,000^{\#}$ (OK) (SEE 18)

(6.) TITEN ISCHEBECK POST SHORES,

MAX. $H \cong 18'$

MAX. $P \cong 5,706^{\#} < 11,600^{\#}$ (OK) (SEE 19)

↳ (ASSUME NO LEDGER FRAMES!) △1

(7.) ~5"x5" BASE PLATE ON 2x12 WOOD PAD,

$$f_{c\perp} = \frac{1}{5^2}\left[\underbrace{5,706}_{\text{SEE ABOVE}} + \underbrace{5.94\left(2\times 1.25\times 6 + \tfrac{1}{2}\times 6\right)}_{\text{TITAN 225 BEAMS}} + \underbrace{\sim 70}_{\text{SHORE}} + \underbrace{\tfrac{1}{2}\left(2\times 15 + 2\times 73\right)}_{\text{FRAMES}}\right]$$

$= 237$ PSI (OK)

DRIVE-THRU CANOPY,

- LOADS DECKING - PEDESTRIAN PROTECTION LL = 150 PSF
SUPPORT STRUCTURE - 25PSF SNOW LOAD
- SNOW DRIFT

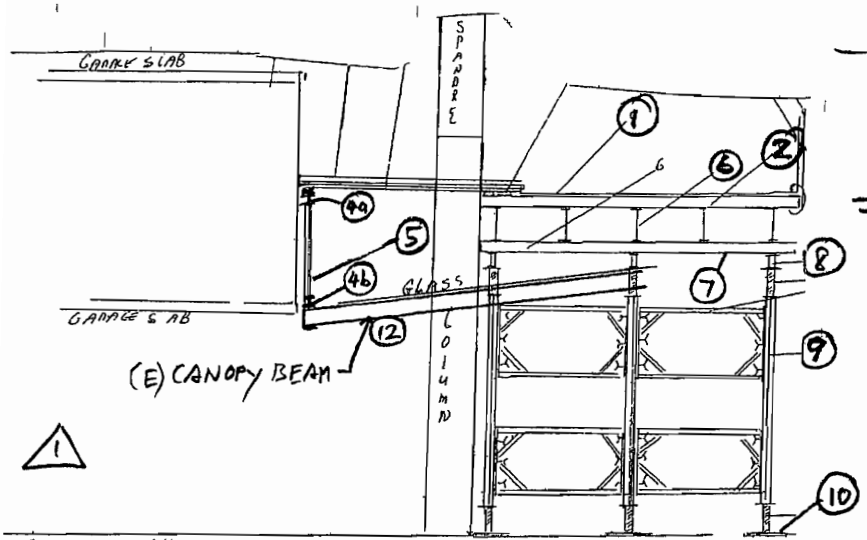
$P_f = 20$ PSF ; $P_u \doteq 135'$

$\Rightarrow \frac{3}{4} h_d = 75 \times .8 = 1.6'$

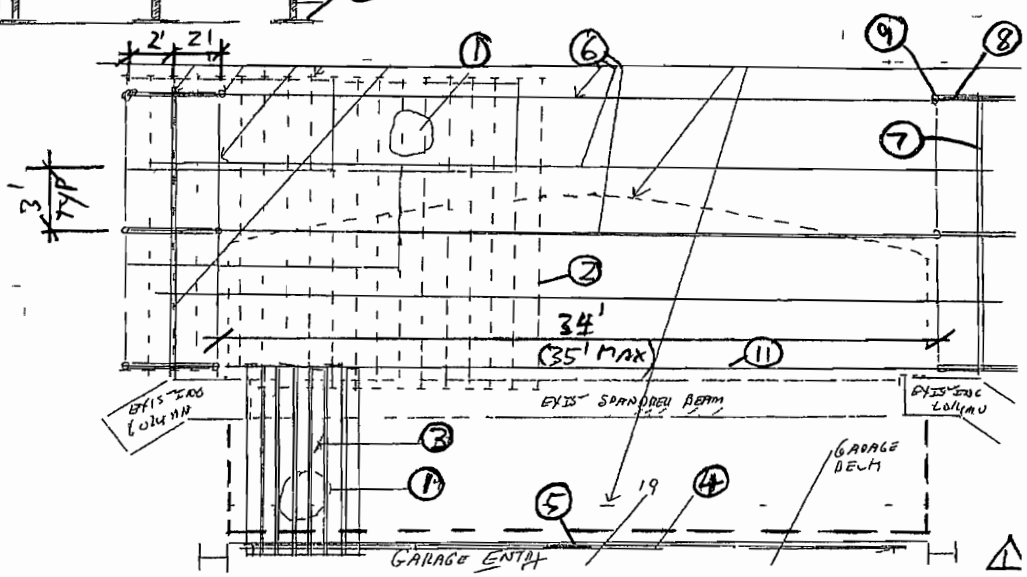
$W_d = 4 \times 1.6 = 2.4' < 12.8'$

$P_d = 6(.13 \times 20 + 14) = 40$ PSF < 53PSF

$\Rightarrow$ USE SNOWDRIFT PER SHEET 4



GARAGE ENTRY SIDE VIEW, N.T.S.



PLAN VIEW GARAGE ENTRY CANOPY, N.T.S

- DESIGN, (1) AND (2), SEE WALK-THRU CANOPY, ITEMS (1) AND (2)

DRIVE-THRU CANOPY, DESIGN, (CONT.)

(3) DOUBLE $1\frac{1}{2}$" × $9\frac{1}{2}$" MICRO-LAM PLANKING, MAX. SPAN = 9'

⇒ MAX. $W = [(4\times2+1.8)+(150)] = 159.8$ PSF < 455 PSF (OK)
$= 2\times60\text{ PSF}\left(\frac{12}{9.5}\right)\left(\frac{4.0}{1.33}\right)$ (SEE 9)

(4a) SYMONS ALUMINUM BEAMS (AB) OR W8×10, MAX. SPAN = 7'

⇒ MAX. $W = 159.8\left(\frac{9}{2}\right) = 719.1$ PLF

⇒ MAX $M = \frac{719.1(7)^2}{8} = 4{,}404$ FT-# < 11,100 FT-# = ALL. M (OK)
< 16,000 FT-# (W8×10)

⇒ MAX. $R_{END} = 719.1\left(\frac{7}{2}+1\right) = 3{,}236^{\#} < 4{,}100^{\#} = \text{ALL. } R_{END}$ (OK)

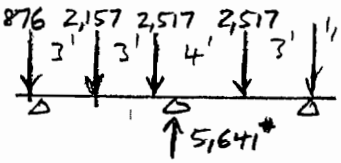
⇒ MAX. $R_{INT} = 719.1\,(1.25\times7) = 6{,}292^{\#} < 8{,}200^{\#} = \text{ALL. } R_{INT}$ (OK)
(SEE 16)

(5) STEPUP SCAFFOLD FRAMES TF6N-3W-L3 OR TF-6N-3W-L3

⇒ MAX. LEG LOAD = $6{,}292^{\#}$ (SEE ITEM (4)) △1
$< 14{,}565^{\#} = 4{,}843^{\#}\left(\frac{4.0}{1.33}\right)$ (OK) (SEE 10, 11)

(4b) W8×10×14', MAX SPAN = 6'

⇒ MAX $M^- = 3{,}602$ FT-# < 16,000 FT-#
MAX. $M^+ = 3{,}560$ FT-# < 3,602 (OK)



$\Sigma = 11{,}505^{\#}$
$= (719.1\text{ PLF})\left(\frac{32}{2}\right)$
(OK)

△1

(6) THRU (10), SEE WALK-THRU CANOPY, ITEMS (3) THRU (7)

△ Feb 2011

DRIVE-THRU CANOPY, DESIGN, CONT.,

(11) CHECK W21x44 FOR ADDED PLANKING,

SPAN = 36'

$$w \cong \Big[\underbrace{44}_{\text{SELFWEIGHT}} + \underbrace{(1.8+5)\left(\tfrac{3}{2}+1\right)}_{\text{PLWD + ALU STRINGERS}} + \underbrace{(4\times2+1.8)\left(\tfrac{9}{2}+1\right)}_{\text{DBL PLANKING + PLWD}}\Big] + \Big[\underbrace{\left(20+\tfrac{1}{2}\left(\tfrac{8.3+2.3}{12.8}\right)(53)\right)}_{\text{FLAT SNOW}}\underbrace{6}_{\text{SNOW DRIFT}}\Big]$$

$= 367$ PLF

$$\Rightarrow \text{MAX. } M = \frac{367(36)^2}{8(1{,}000)} = 59.5^k\text{-FT} < 163^k\text{-FT} = \text{ALL. } M$$

($F_y = 36$ KSI) OK

$$\Rightarrow \text{MAX. } R = \frac{367(36)}{2(1{,}000)} = 6.61^k$$

SPLICE OK (SEE SHEET 5)

⇒ CHECK SINGLE TITAN 225 CROSS-BEAM,

$$P \cong 6.7^k + (6.61 - 4.76) = \underline{8.6^k} < 11.4^k \text{ (SEE 6)}$$ OK

↳ SEE SHEET 6, ITEM (4) $P = 4.76^k$

→ SEE ABOVE, $R = 6.61^k$

→ SEE SHEET 6, EXT REACTION $= 6.7^k$

△

(12) CHECK (E) GLASS CANOPY SUPPORT BEAMS FOR (N) PLANKING,

⇒ ADDED DEAD LOAD TO EACH (E) GLASS CANOPY SUPPORT BEAM FROM (N) PLANKING ⇒ $\sim 335^{\#}$

THIS INCREASE IS NEGLIGIBLE BECAUSE IT IS APPLIED CLOSE TO THE 5-BOLTED CONNECTION TO THE PARKING SLAB EDGE SUPPORT BEAM!

recycle at fond

REVIEWED BY JE DUNN NORTHWEST INC
REVIEWED BY Ann Young
DATE 02/11/11
SUBMITTAL # 051250.8

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FEB 09 2011
JE DUNN CONSTRUCTION CO

WILDCAT™

BUCKLING-RESTRAINED BRACES

Brace & Gusset Calculations

Providence Office Park Parking Structure 2
Portland, Oregon

Structural Engineer: Equilibrium Engineers
Steel Fabricator: L&M Industrial Fabrication

SUBMITTAL REVIEW
☒ NO EXCEPTIONS TAKEN ☐ REVISE AND RESUBMIT
☐ MAKE CORRECTIONS NOTED ☐ SUBMIT SPECIFIED ITEM
CHECKED ONLY FOR GENERAL CONFORMANCE WITH THE DESIGN CONCEPT OF THE PROJECT AND GENERAL COMPLIANCE WITH THE INFORMATION IN THE CONTRACT DOCUMENTS. ANY ACTION SHOWN ABOVE, IS SUBJECT TO THE REQUIREMENTS OF THE PLANS AND SPECIFICATIONS. CONTRACTOR IS RESPONSIBLE FOR DIMENSIONS WHICH SHALL BE CONFIRMED AND CORRELATED AT THE JOB SITE. FABRICATION PROCESSES AND TECHNIQUES OF CONSTRUCTION, COORDINATION OF HIS WORK WITH THAT OF ALL OTHER TRADES AND THE SATISFACTORY PERFORMANCE OF HIS WORK.
Equilibrium Engineers LLC
BY
DATE 2/15/11

Exp 6/30/11

February 4, 2011

10-199336 DFS 01 CO

Performance Criteria

Testing of the Wildcat™ braces conforms to ANSI/AISC 341, Appendix T All tests performed were subassemblage tests except the WC150 Br 2 test

The 9 in^2 – 11 5 in^2/module braces are qualified by the following tests per AISC 341

- WC500Br1 as a brace test (allowable range 305 9 k/module - 917 7 k/module, 7 67 in^2/module ≤ A_{sc}/module = 23in^2/module)
- WC500Br2 as a subassemblage test (allowable range < 459k/module, A_{sc} ≤ 11 5in^2/module) Selected qualification tests for this brace are WC500 Br1 & WC500 Br2, allowable strain is brace is ε = 1 41% > 1 30% (maximum brace strain in this project)

The 5 5 in^2 - 8 5 in^2/module braces are qualified by the following tests per AISC 341

- WC250Br1 as a brace test (allowable range 152 95 k/module - 458 85 k/module, 3 83 in^2/module ≤ A_{sc}/module = 11 5 in^2/module)
- WC250Br2 as a brace test (allowable range 152 95 k/module - 458 85 k/module, 3 83 in^2/module ≤ A_{sc}/module = 11 5 in^2/module)
- WC780 as 2-WC390 subassemblage (allowable range < 389k/module, A_{sc} = 9 76 in^2/module) Selected qualification tests for this brace are WC250 Br2 & WC780 as 2-WC390, allowable strain is brace is ε = 1 89% > 1 30% (maximum brace strain in this project)

See Appendix A for the stiffness analysis and further explanation thereof All braces for this project fall within the allowed interpolation methods allowed by ANSI/AISC 341

Physical Requirements

The brace has been designed to meet the load/stiffness requirement given in the contract documents The area of steel core (A_{sc}) was designed to equal the P_u/ϕ / $F_{y\ act}$ Where $P_u = \rho^*(P_{u\ max} + P_{u\ ortho})$ taken from detail 4/PSS3 3 and $F_{y\ act}$ is the actual yielding force for each specific heat as determined by the material test reports The stiffness requirement was met by having the brace be ±10% K_{model} where $K_{model} = A^*E/L_{wp\text{-}wp}$ where E = 46 4*10^6 psi per note 1 on 4/PSS3 3

Contents

1 Appendix A provides the stiffness and overstrength factor determination
2 Appendix B includes all of the gusset connection calculations, brace design calculations, brace frame elevations, and the gusset details and schedule Appendix D shows which limit states and portions of the connection that Star Seismic has checked ~~Star Seismic has not designed the beam/column connection, but has provided the UFM loads that must be combined with gravity and drag loads for the EOR to complete the design of this connection.~~
3 Appendix C includes the steel core testing data
4 Appendix D included as a detailed hand calculations to demonstrate the design methodology used in the calculations included in Appendix B
5 Appendix E is included as an expansion to the qualification test results It includes summaries of brace loads and strains

STAR SEISMIC

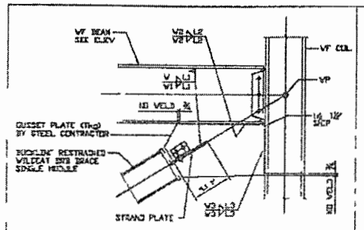
WILDCAT™

BUCKLING-RESTRAINED BRACES

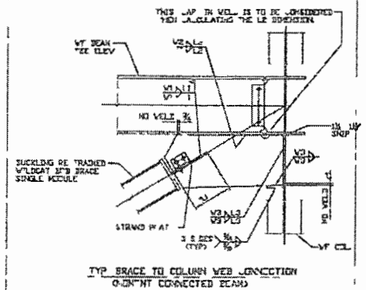
Appendix A
Stiffness Analysis

Star Seismic, LLC 6300 N Sagewood Dr Suite H #511 Park City UT 84098 (435)940-9222

MICRO

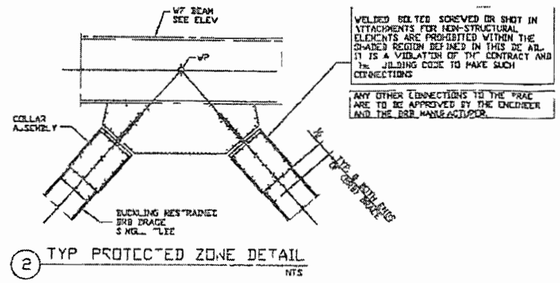


VP STRAP TO COLUMN FLANGE CONNECTION (MOMENT CONNECTED BEAM)

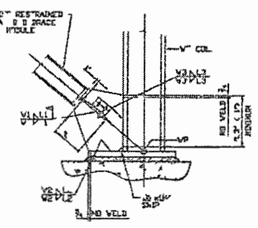


TYP BRACE TO COLUMN WEB CONNECTION (MOMENT CONNECTED BEAM)

1 SINGLE TUBE / MOMENT BEAM NTS



2 TYP PROTECTED ZONE DETAIL NTS



3 TYP BRACE TO COLUMN CONN NTS

4 TYP CHEVRON BRACE CONN (SINGLE TUBE) NTS

ELEVATION VIEW (SINGLE TUBE)

5 TYPICAL SHIM INSERTION NTS

WILDCAT GUSSET CONNECTION SCHEDULE

STAR BRACE#	BRACE END	STAR PCMK	ENGS DESIG	LOCATION	ELEV	GUSSET PL TH (IN)	BRACE/GUSSET WELD # WELDS	W1	L1 (IN)	BEAM WELD W2	L2(IN)	COLUMN WELD W3	L3(IN)
1	Lower	WC361a	BRB3	21/DD EE	4 5	3/4	4	1/2	16	5/16	36	5/16	18
1	Upper	WC361a	BRB3	21/DD EE	4 5	3/4	4	1/2	16	3/8	30	5/16	16
2	Lower	WC319a	BRB2	21/DD EE	5-6	3/4	4	1/2	15	5/16	38	5/16	18
2	Upper	WC319a	BRB2	21/DD-EE	5-6	3/4	4	1/2	15	5/16	36	5/16	16
3	Lower	WC361a	BRB3	21/CC DD	4 5	3/4	4	1/2	16	5/16	38	5/16	18
3	Upper	WC361a	BRB3	21/CC DD	4 5	3/4	4	1/2	16	3/8	30	5/16	16
4	Lower	WC319a	BRB2	21/CC DD	5-6	3/4	4	1/2	15	5/16	36	5/16	18
4	Upper	WC319a	BRB2	21/CC-DD	5-6	3/4	4	1/2	15	5/16	32	5/16	16
5	Lower	WC361b	BRB3	17/DD EE	4 5	3/4	4	1/2	16	5/16	35	5/16	19
5	Upper	WC361b	BRB3	17/DD EE	4 5	3/4	4	1/2	16	3/8	29	5/16	17
6	Lower	WC319a	BRB2	17/DD EE	5-6	3/4	4	1/2	15	5/16	33	5/16	18
6	Upper	WC319a	BRB2	17/DD EE	5 6	3/4	4	1/2	15	5/16	32	5/16	16
7	Lower	WC361a	BRB3	17/CC DD	R4 R5	3/4	4	1/2	16	5/16	36	5/16	18
7	Upper	WC361a	BRB3	17/CC-DD	R4 R5	3/4	4	1/2	16	3/8	30	5/16	16
8	Lower	WC319b	BRB2	17/CC DD	R5-R6	3/4	4	1/2	15	5/16	32	5/16	18
8	Upper	WC319b	BRB2	17/CC-DD	R5-R6	3/4	4	1/2	15	5/16	32	5/16	16
9	Lower	WC319a	BRB2	13/DD EE	4 5	3/4	4	1/2	15	5/16	33	5/16	18
9	Upper	WC319a	BRB2	13/DD EE	4-5	3/4	4	1/2	15	5/16	32	5/16	16
10	Lower	WC282a	BRB1	13/DD EE	5-6	5/8	4	1/2	15	5/16	32	5/16	18
10	Upper	WC282a	BRB1	13/DD EE	5-6	5/8	4	1/2	15	5/16	29	5/16	16
11	Lower	WC319a	BRB2	13/CC DD	4-5	3/4	4	1/2	15	5/16	33	5/16	18
1	Upper	WC319a	BRB2	13/CC-DD	4 5	3/4	4	1/2	15	5/16	32	5/16	16
12	Lower	WC282a	BRB1	13/CC DD	5-6	5/8	4	1/2	15	5/16	32	5/16	18
12	Upper	WC282a	BRB1	13/CC-DD	5-6	5/8	4	1/2	15	5/16	29	5/16	16
13	Lower	WC433a	BRB6	AA/21 22	4 5	1	4	1/2	21	5/16	79	5/16	16
13	Upper	WC433a	BRB6	AA/21 22	4-5	1	4	1/2	21	5/16	73	5/16	14
14	Lower	WC393b	BRB5	AA/21 22	5-6	1	4	1/2	19	5/16	78	5/16	16
14	Upper	WC393b	BRB5	AA/21 22	5-6	1	4	1/2	19	5/16	71	5/16	14
15	Lower	WC433a	BRB6	GG/14 12	4 5	1	4	1/2	21	5/16	77	5/16	16
15	Upper	WC433a	BRB6	GG/14 12	4-5	1	4	1/2	21	5/16	72	5/16	14
16	Lower	WC393a	BRB5	GG/14 12	5-6	1	4	1/2	19	5/16	76	5/16	15
16	Upper	WC393a	BRB5	GG/14 12	5 6	1	4	1/2	19	5/16	69	5/16	13
17	Lower	WC393b	BRB5	AA/12 13	4-5	1	4	1/2	19	5/16	79	5/16	16
17	Upper	WC393b	BRB5	AA/12 13	4 5	1	4	1/2	19	5/16	73	5/16	14
18	Lower	WC355a	BRB4	AA/12 13	5-6	1	4	1/2	17	5/16	78	5/16	16
18	Upper	WC355a	BRB4	AA/12 13	5-6	1	4	1/2	17	5/16	71	5/16	14
19	Lower	WC433a	BRB6	GG/22 20	4 5	1	4	1/2	21	5/16	78	5/16	16
19	Upper	WC433a	BRB6	GG/22 20	4 5	1	4	1/2	21	5/16	72	5/16	14
20	Lower	WC393a	BRB5	GG/22 20	5-6	1	4	1/2	19	5/16	77	5/16	16
20	Upper	WC393a	BRB5	GG/22 20	5-6	1	4	1/2	19	5/16	71	5/16	14

6 BRB CONNECTION SCHEDULE (WELDED CONNECTIONS) NTS

NOTE
1. ALL GUSSET PLATES ARE TO BE A-36 GR50

REGISTERED PROFESSIONAL ENGINEER
6/30/11

SUBMITTAL REVIEW

☒ NO EXCEPTIONS TAKEN ☐ REVISE AND RESUBMIT
☐ MAKE CORRECTIONS NOTED ☐ SUBMIT SPECIFIED ITEM

CHECKING IS ONLY FOR GENERAL CONFORMANCE WITH THE DESIGN CONCEPT OF THE PROJECT AND GENERAL COMPLIANCE WITH THE INFORMATION IN THE CONTRACT DOCUMENTS. ANY ACTION SHOWN ABOVE, IS SUBJECT TO THE REQUIREMENTS OF THE PLANS AND SPECIFICATIONS. CONTRACTOR IS RESPONSIBLE FOR: DIMENSIONS WHICH SHALL BE FABRICATED AND CORRELATED AT THE JOB SITE; FABRICATION PROCESSES AND TECHNIQUES OF CONSTRUCTION; COORDINATION OF HIS WORK WITH THAT OF ALL OTHER TRADES, AND THE SATISFACTORY PERFORMANCE OF HIS WORK.

Equilibrium Engineers LLC

BY _____ DATE 3/9/11

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3	8/15 11 BP	REVISED PC MK TO MATCH ELEV LAYOUT
2	2/04/11 BP	REVISED FOR APPROVAL
1	1/31/11 SNJ	FOR APPROVAL

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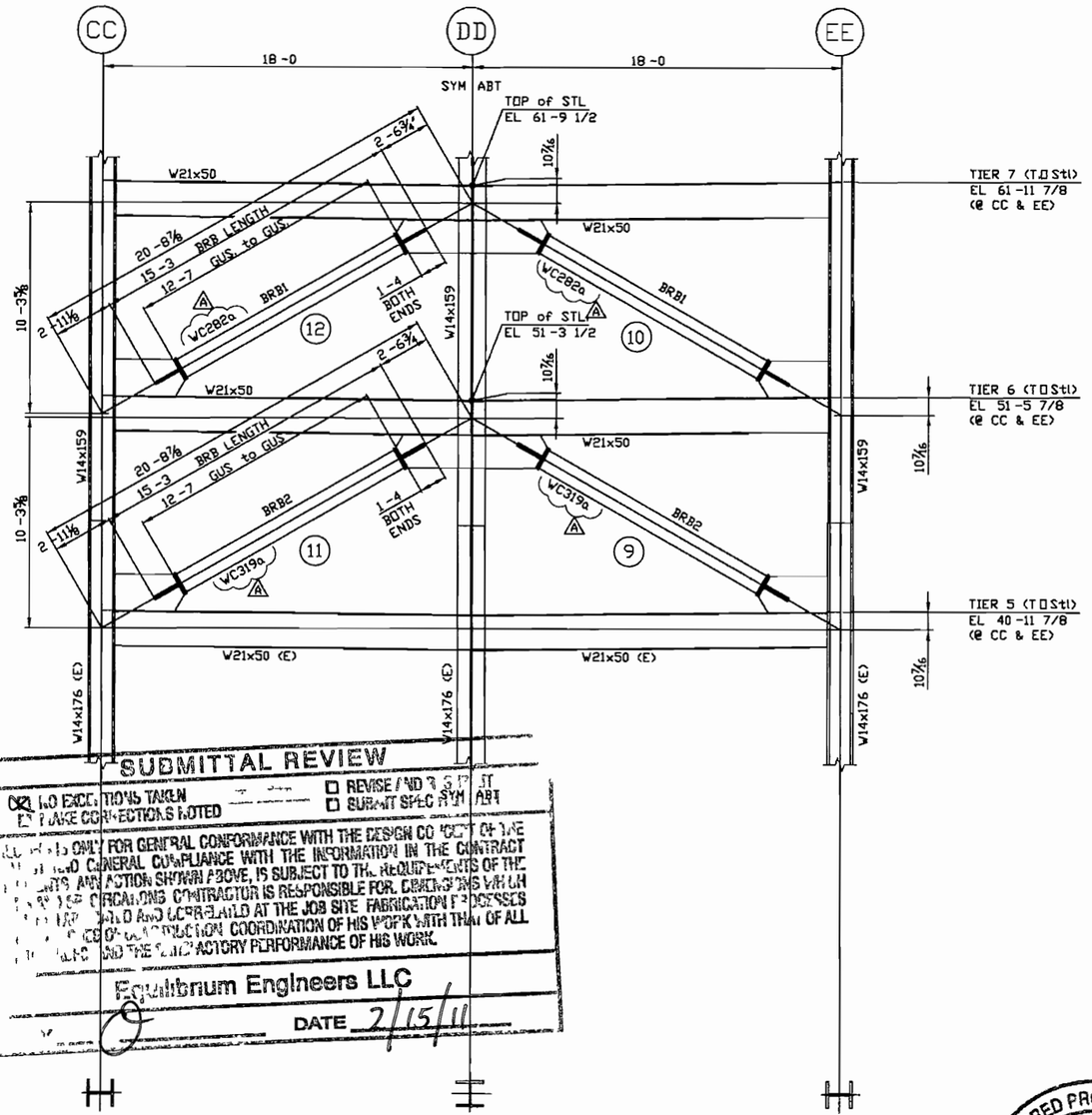
109 N. SAGEWOOD DR SUITE H 9511
PARK CITY, UT 84098
PHONE: 1-435-940-9022
FAX 1-435-655-6572

NAME: WILDCAT™ GUSSET DETAILS AND SCHEDULE
PROJECT: POP P SKING STRUCTURE & EXPANSION
PROJECT NO: 110187 DATE 1/27/11 PAGE WC-01

10-199336 DFS 01 CO



LINE 13

REF: SECT 3/PSS31

STEEL DETAILER NOTE
ALL GUSSET to GUSSET DIMENSIONS MUST BE HELD IF POSSIBLE. PLEASE NOTIFY STAR SEISMIC IMMEDIATELY IF NOT POSSIBLE

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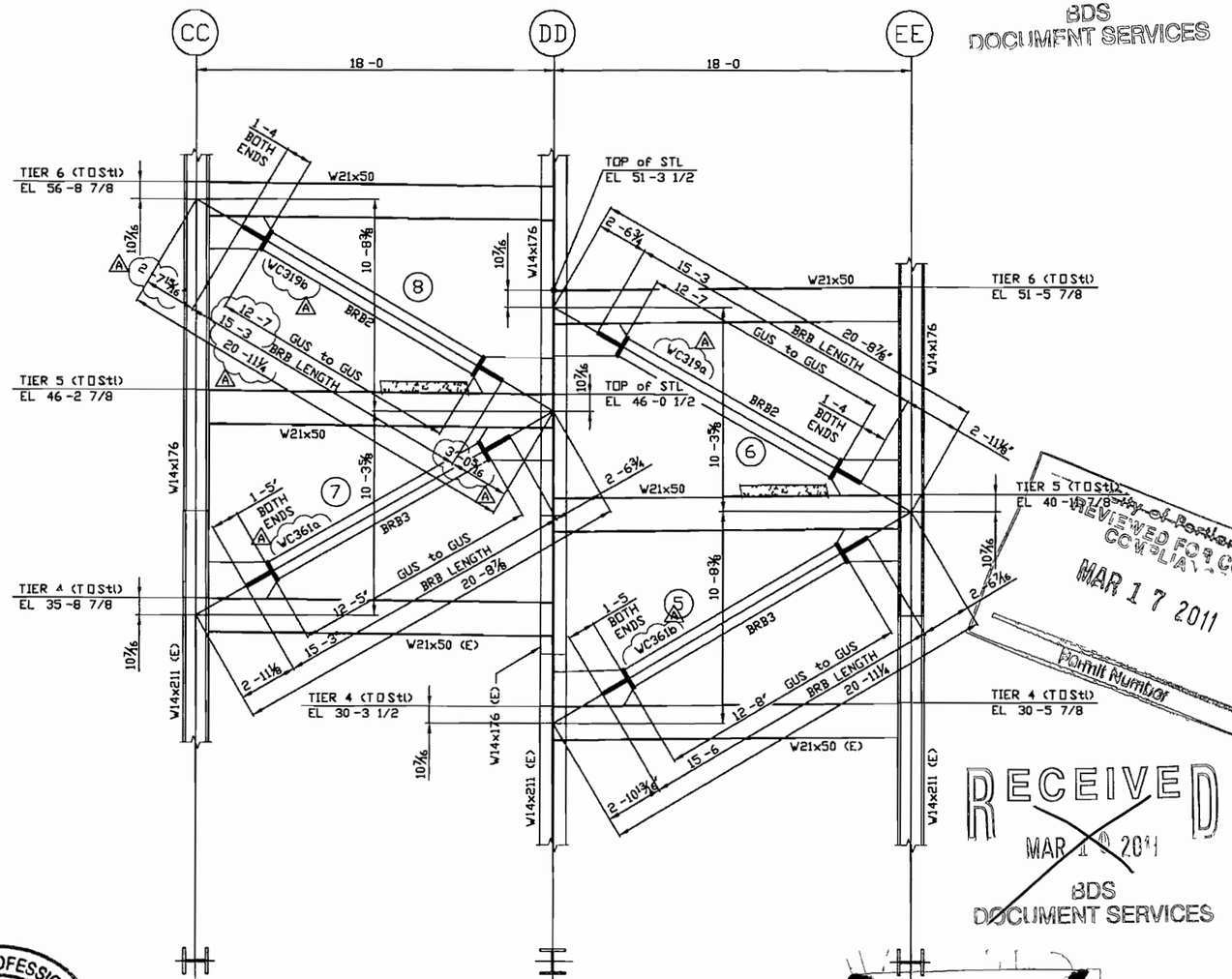

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REGISTERED PROFESSIONAL ENGINEER
16430
OREGON
MAR 18 1999
ARGAN N JOHNSON JR
6/30/11

LINE 17

REF: SECT 2/PSS31

STEEL DETAILER NOTE
ALL GUSSET to GUSSET DIMENSIONS MUST BE HELD IF POSSIBLE. PLEASE NOTIFY STAR SEISMIC IMMEDIATELY IF NOT POSSIBLE

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
A	02/04/2011 BP	REVISED MK @ S AND SOME DIMENSIONS PER LATEST DESIGN
		ISSUE FOR APPROVAL
—	01/31/2011 SJ	ORIGINAL ISSUE / FOR REVIEW ONLY

SUBJECT Braced Frame @ Lines 13 & 17
PROJECT PROVIDENCE PARKING STRUCTURE
LOCATION PORTLAND OREGON
CUSTOMER
SCALE NTS
DRAWN BY BP DATE 01/28/2011

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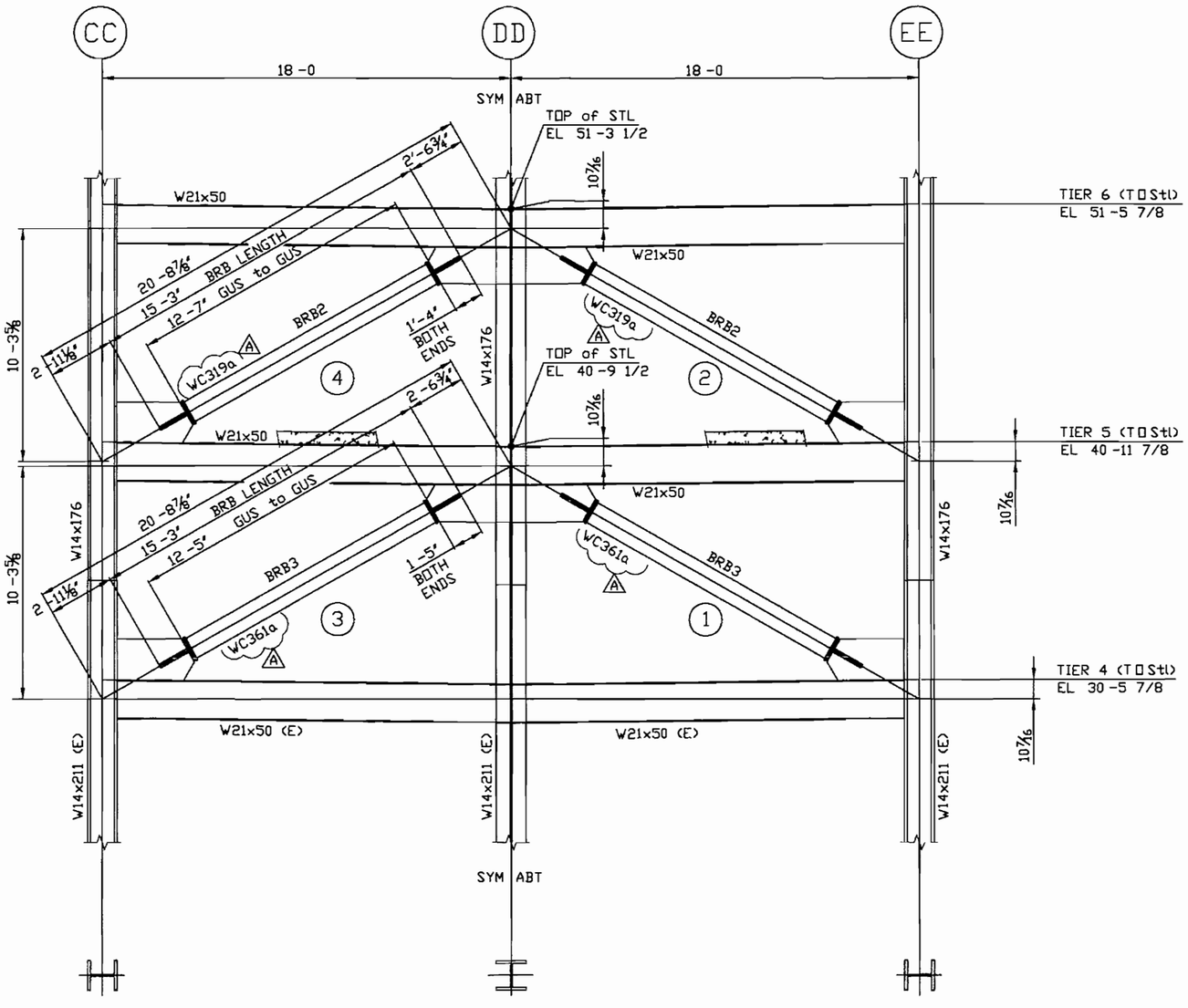
JOB NO 110127 PG NO SS-01



LINE 21

REF: SECT 1/PSS31

STEEL DETAILER NOTE:
ALL GUSSET to GUSSET DIMENSIONS MUST BE HELD IF POSSIBLE PLEASE NOTIFY STAR SEISMIC IMMEDIATELY IF NOT POSSIBLE

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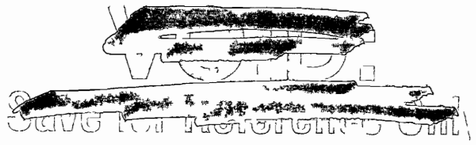

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16430
OREGON
MAR 16 1993
ARGAN N JOHNSON JR
exp 6/30/11

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REVIEW FOR GENERAL CONFORMANCE WITH THE DESIGN CONCEPT OF THE PROJECT AND GENERAL COMPLIANCE WITH THE INFORMATION GIVEN IN THE CONTRACT DOCUMENTS. CONTRACTOR'S OWN ABOVE, IS SUBJECT TO THE REQUIREMENTS OF THE CONTRACT DOCUMENTS. CONTRACTOR'S OWN PORTION IS RESPONSIBLE FOR DIMENSIONS WHICH ARE TO BE CONFIRMED AND CORRELATED AT THE JOB SITE, FABRICATION PROCESSES AND TECHNIQUES OF CONSTRUCTION, COORDINATION OF HIS WORK WITH THAT OF ALL OTHER TRADES AND THE SATISFACTORY PERFORMANCE OF HIS WORK.

Equilibrium Engineers LLC

DATE 2/15/11

△	—		SUBJECT Braced Frame @ Line 21
△	—		PROJECT PROVIDENCE PARKING STRUCTURE
			LOCATION PORTLAND OREGON
			CUSTOMER
			SCALE NTS
			DRAWN BY BP DATE 01/28/2011
A	02/04/2011 BP	REVISED MK #'S AND SOME DIMENSIONS PER LATEST DESIGN	
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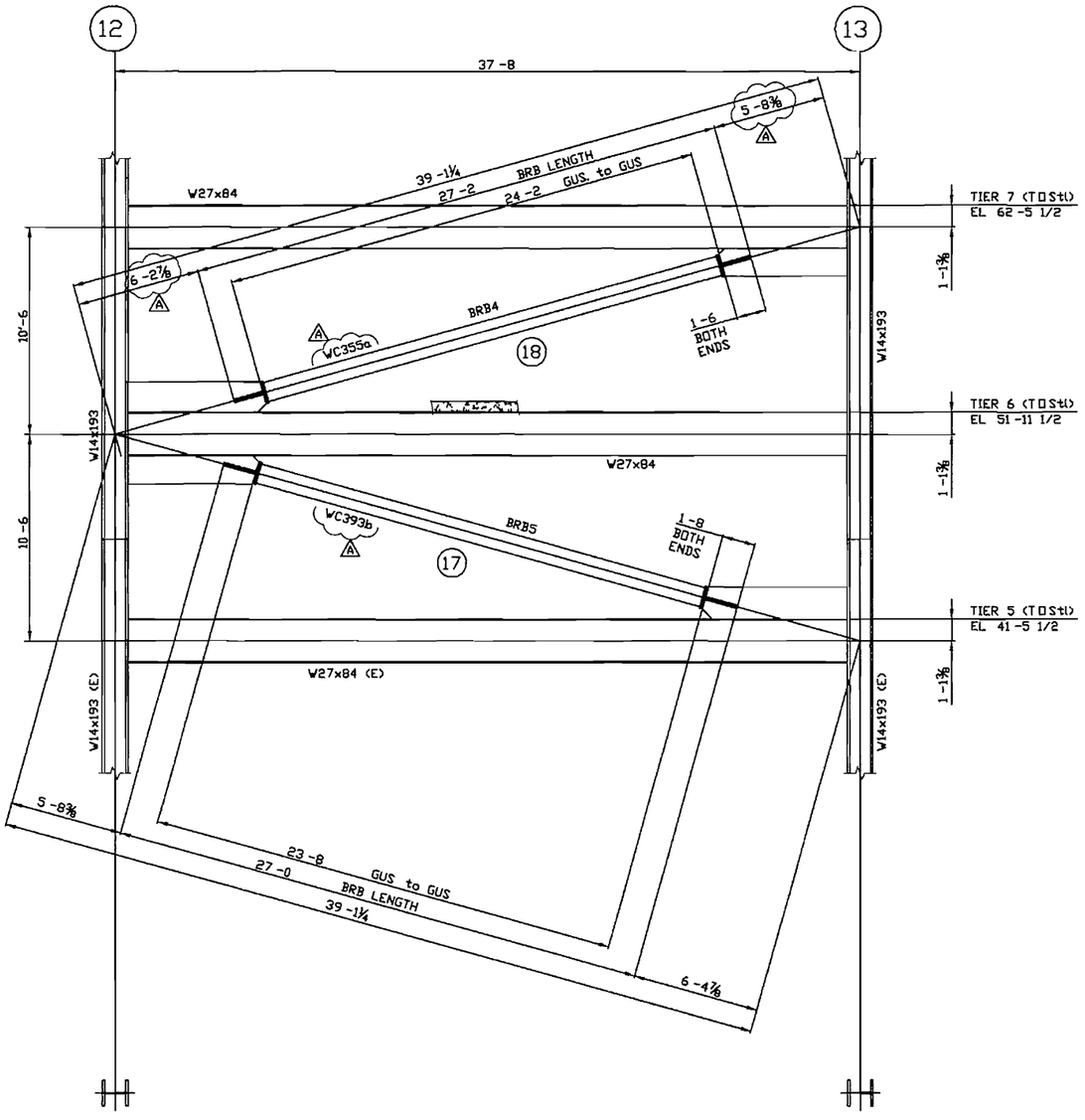
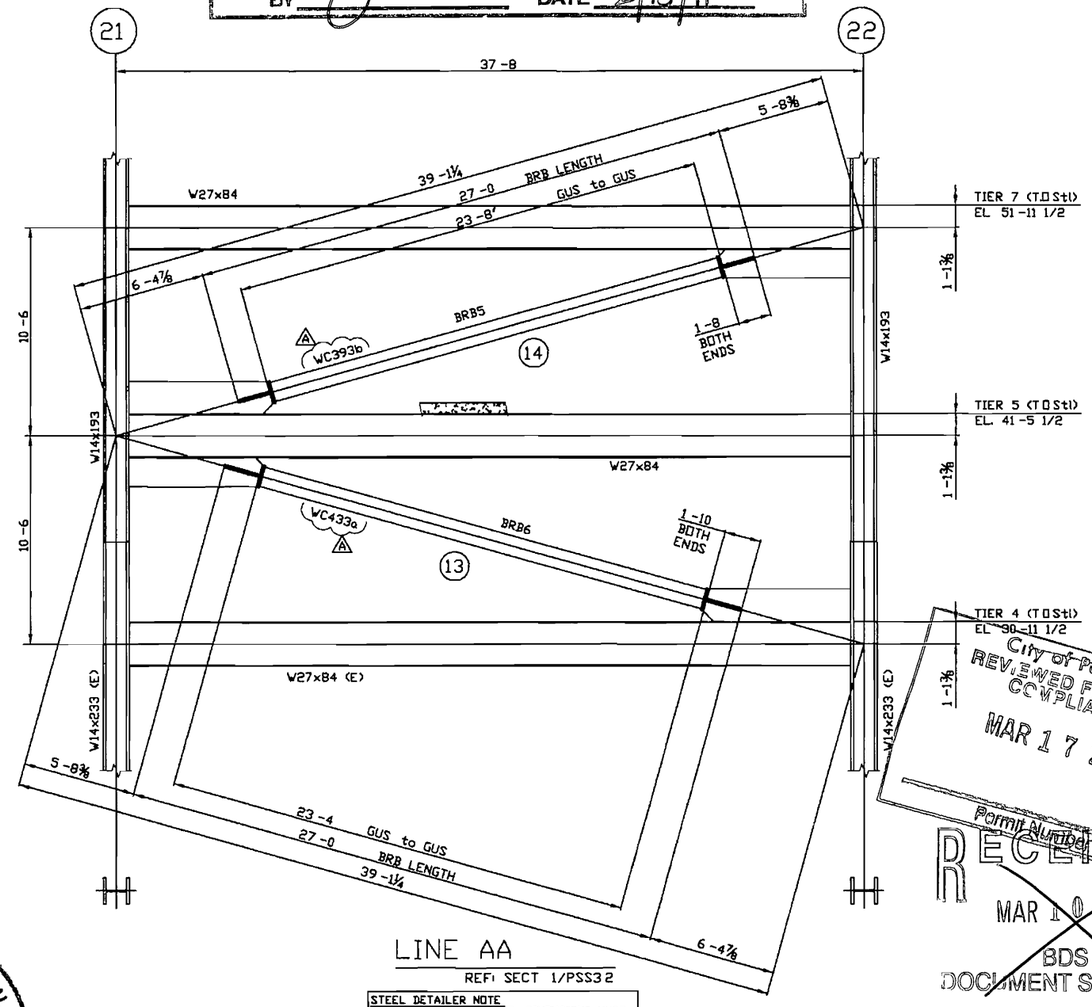
JOB NO 10127 PG NO SS 02



LINE AA

REF: SECT 2/PSS32

STEEL DETAILER NOTE
ALL GUSSET to GUSSET DIMENSIONS MUST BE HELD IF POSSIBLE PLEASE NOTIFY STAR SEISMIC IMMEDIATELY IF NOT POSSIBLE



LINE AA

REF: SECT 1/PSS32

STEEL DETAILER NOTE
ALL GUSSET to GUSSET DIMENSIONS MUST BE HELD IF POSSIBLE PLEASE NOTIFY STAR SEISMIC IMMEDIATELY IF NOT POSSIBLE

SUBMITTAL REVIEW

☒ [illegible] TAKEN ☐ REVISE AND [illegible]
☐ [illegible] NOTED ☐ SUBMIT SPECIFIED ITEM

REVIEW FOR GENERAL CONFORMANCE WITH THE DESIGN CONCEPT OF THE PROJECT AND COMPLIANCE WITH THE INFORMATION IN THE CONTRACT DOCUMENTS. ANY ACTION SHOWN ABOVE IS SUBJECT TO THE REQUIREMENTS OF THE CONTRACT DOCUMENTS. CONTRACTOR IS RESPONSIBLE FOR: CONFIRMING AND CORRELATING AT THE JOB SITE; FABRICATION PROCESSES AND TECHNIQUES OF CONSTRUCTION; COORDINATION OF HIS WORK WITH THAT OF ALL OTHER TRADES AND THE SATISFACTORY PERFORMANCE OF HIS WORK.

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OREGON
MAR 18 1993
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SUBJECT Braced Frame @ Lines AA (2 PLC'S)
PROJECT PROVIDENCE PARKING STRUCTURE
LOCATION PORTLAND OREGON
CUSTOMER
SCALE NTS
DRAWN BY BP DATE 01/28/2011

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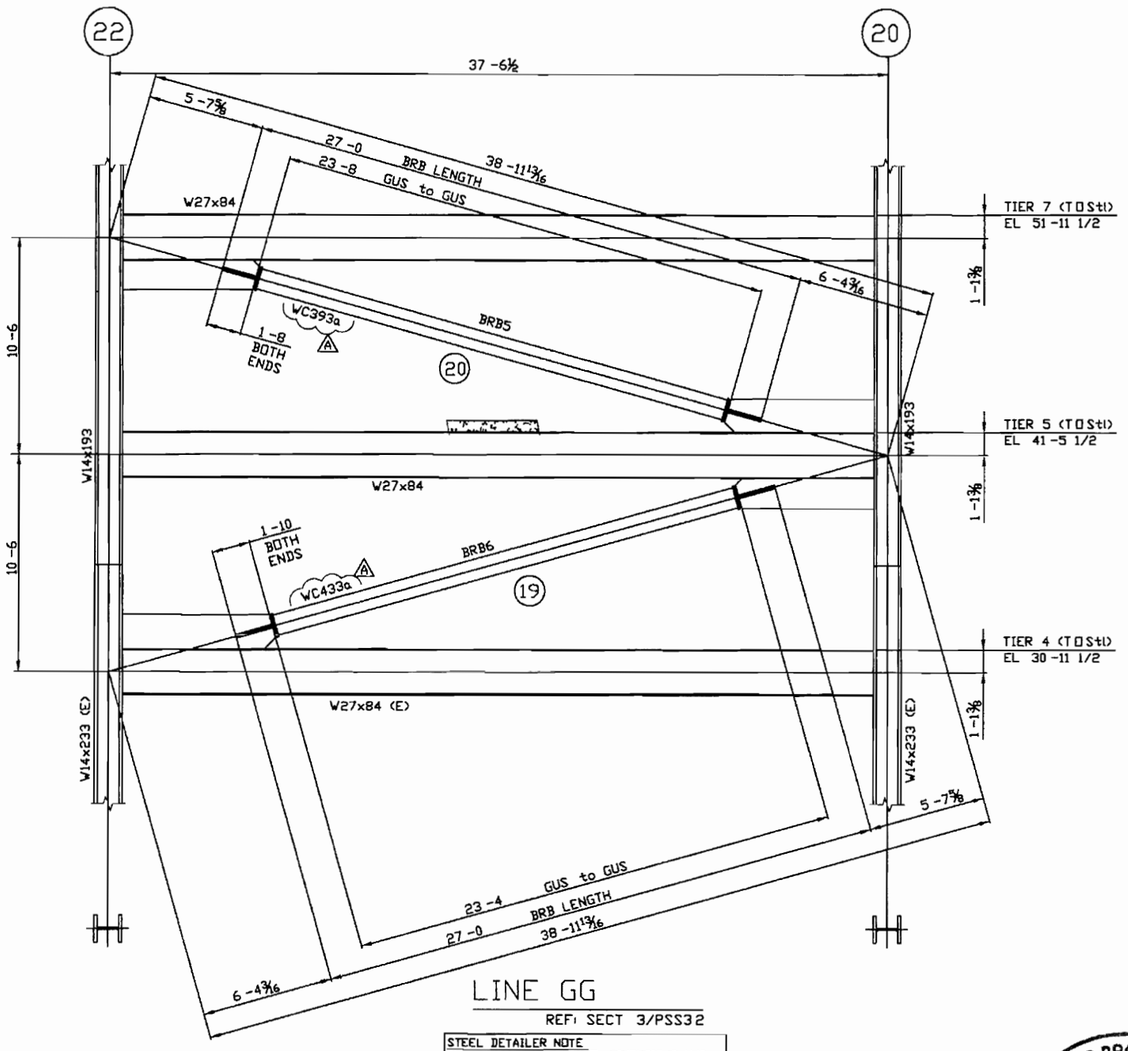
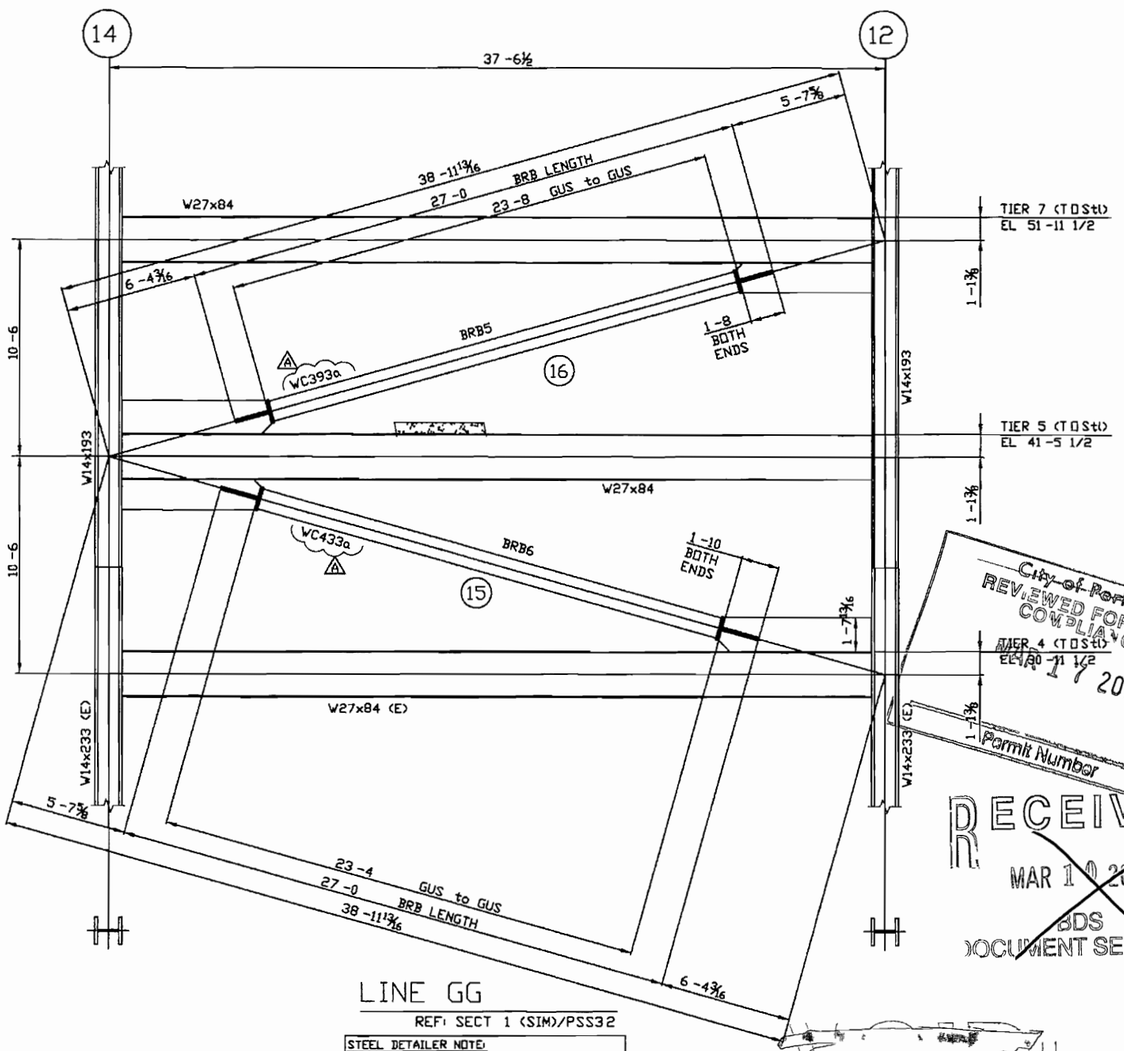


LINE GG

REF: SECT 3/PSS32

STEEL DETAILER NOTE
ALL GUSSET to GUSSET DIMENSIONS MUST BE HELD IF POSSIBLE PLEASE NOTIFY STAR SEISMIC IMMEDIATELY IF NOT POSSIBLE



LINE GG

REF: SECT 1 (SIM)/PSS32

STEEL DETAILER NOTE:
ALL GUSSET to GUSSET DIMENSIONS MUST BE HELD IF POSSIBLE PLEASE NOTIFY STAR SEISMIC IMMEDIATELY IF NOT POSSIBLE

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☐ REVISE AND
☐ SUBMIT SPECIFIED ITEM
☐ MAKE CORRECTIONS NOTED

[illegible] ONLY FOR GENERAL CONFORMANCE WITH THE DESIGN CONCEPT OF THE [illegible] AND GENERAL COMPLIANCE WITH THE INFORMATION IN THE CONTRACT [illegible] AND SPECIFICATIONS. CONTRACTOR IS RESPONSIBLE FOR DIMENSIONS [illegible] AND CORRELATED AT THE JOB SITE. FABRICATION PROCESSES [illegible] OF CONSTRUCTION. COORDINATION OF HIS WORK WITH THAT OF ALL [illegible] THE SATISFACTORY PERFORMANCE OF HIS WORK.

Equilibrium Engineers LLC

DATE 2/15/11

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16430
OREGON
MAR 16 1993
ARGAN N JOHNSON JR
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		ISSUE FOR APPROVAL
△	01/31/2011 SJ	ORIGINAL ISSUE / FOR REVIEW ONLY

SUBJECT Braced Frame @ Lines GG (2 PLC'S)
PROJECT PROVIDENCE PARKING STRUCTURE
LOCATION PORTLAND OREGON
CUSTOMER
SCALE NTS
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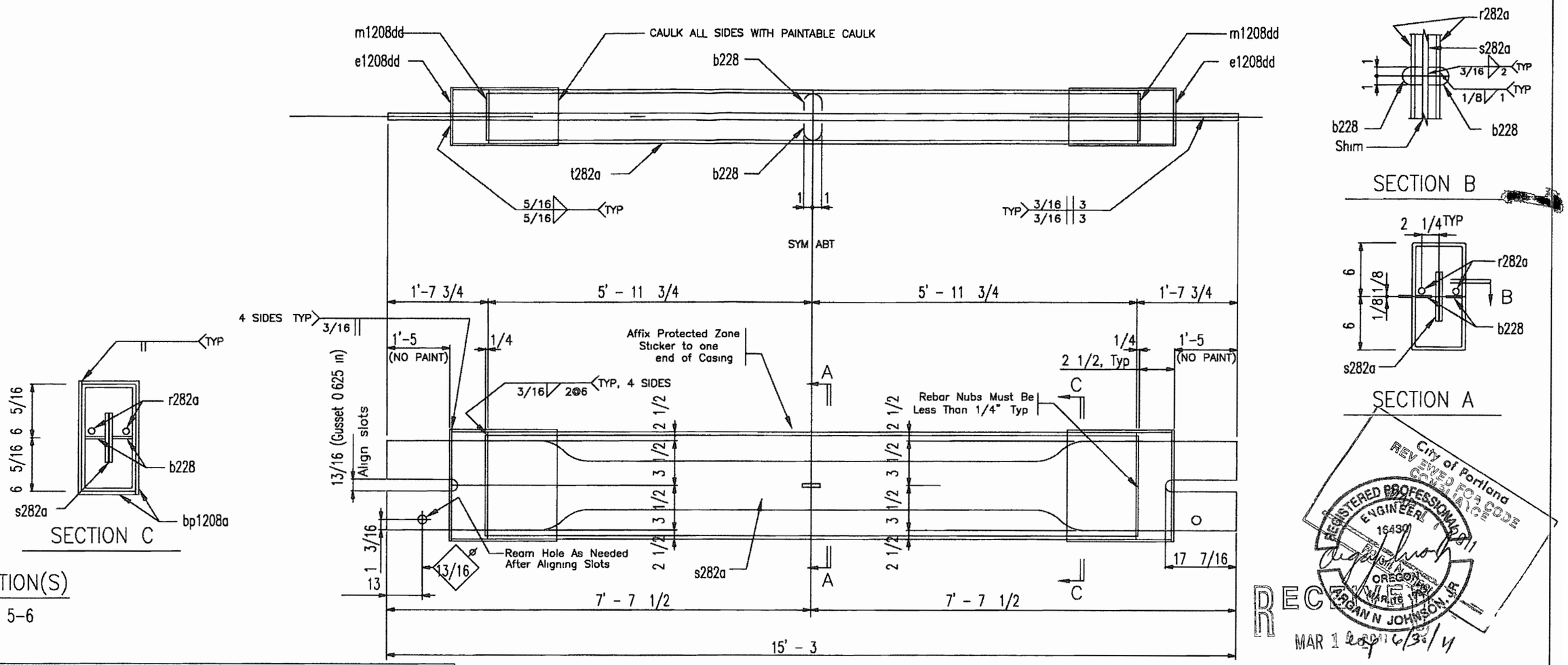
JOB NO 110127 PG NO SS 04



BRACE LOCATION(S)

13/DD-EE at el 5-6

2 BRACE WC282a

City of Portland
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REGISTERED PROFESSIONAL ENGINEER
16430
OREGON
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BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
2	Brace	15	3	WC282a	BRB1
2	HSS12X8X1/4	11	11 1/2	t282a	A500-Gr B
8	PL 1/4 x 24	1	8 1/4	bp1208a	A36
4	FB 3/4x 7	15	3	s282a	A36-LY
4	PL 1/4x2	0	2 7/8	b228	A36
4	PL 1/4 x 7 1/2	0	11 1/2	m1208dd	A36
4	#4 rebar	12	5	r282a	
4	PL 1/2 x 8 1/8	1	0 1/8	e1208dd	A36

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
△		
△		
△		
A	2/3/2011 SNJ	Approval

SHOP PLACE PIECE MARK ON LEFT HAND END OF MEMBER AS DETAILED
ERECT MEMBER WITH PIECE MARK TO END SHOWN ON PLACING PLAN

MATERIAL A36 & AS NOTED
SHOP PAINT None
APPROX BRACE WT 1905 EA
SURFACE PREPARATION -
APPROX CONCR 0.246 YRDS EA

SUBJECT WC282a
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication
ZONE SS BRB NO 10 12
ERECTION DWG ENG DWG PSS3 1
DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

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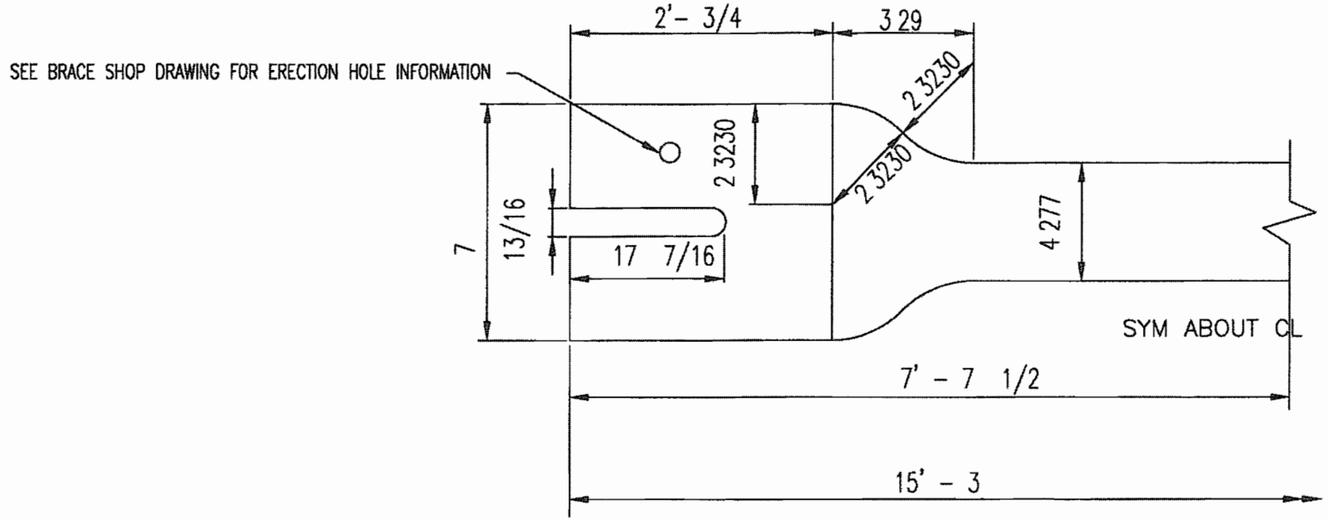
PARK CITY, UT 84098
www starseismic net
PHONE 1-435-940-9222
FAX 1-435-655-0073

JOB NO 110127 DWG NO WC282a



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16430
OREGON
MAR 16, 1993
ARGAN N JOHNSON, JR.
exp 6/30
City of Portland
REVIEWED FOR CODE COMPLIANCE
MAR 17 2011
Limit Number

Lysc=133 5 in

Total Brace Asc= 6 42 in^2

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
4	STRAND PLATES			s282a	
4	FB 3/4x 7	15	3	s282a	A36-LY Heat#120-0237

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
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4 STRAND PLATES s282a

SHOP NOTE DEBURR ALL CUT EDGES

SUBJECT s282a
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication

DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

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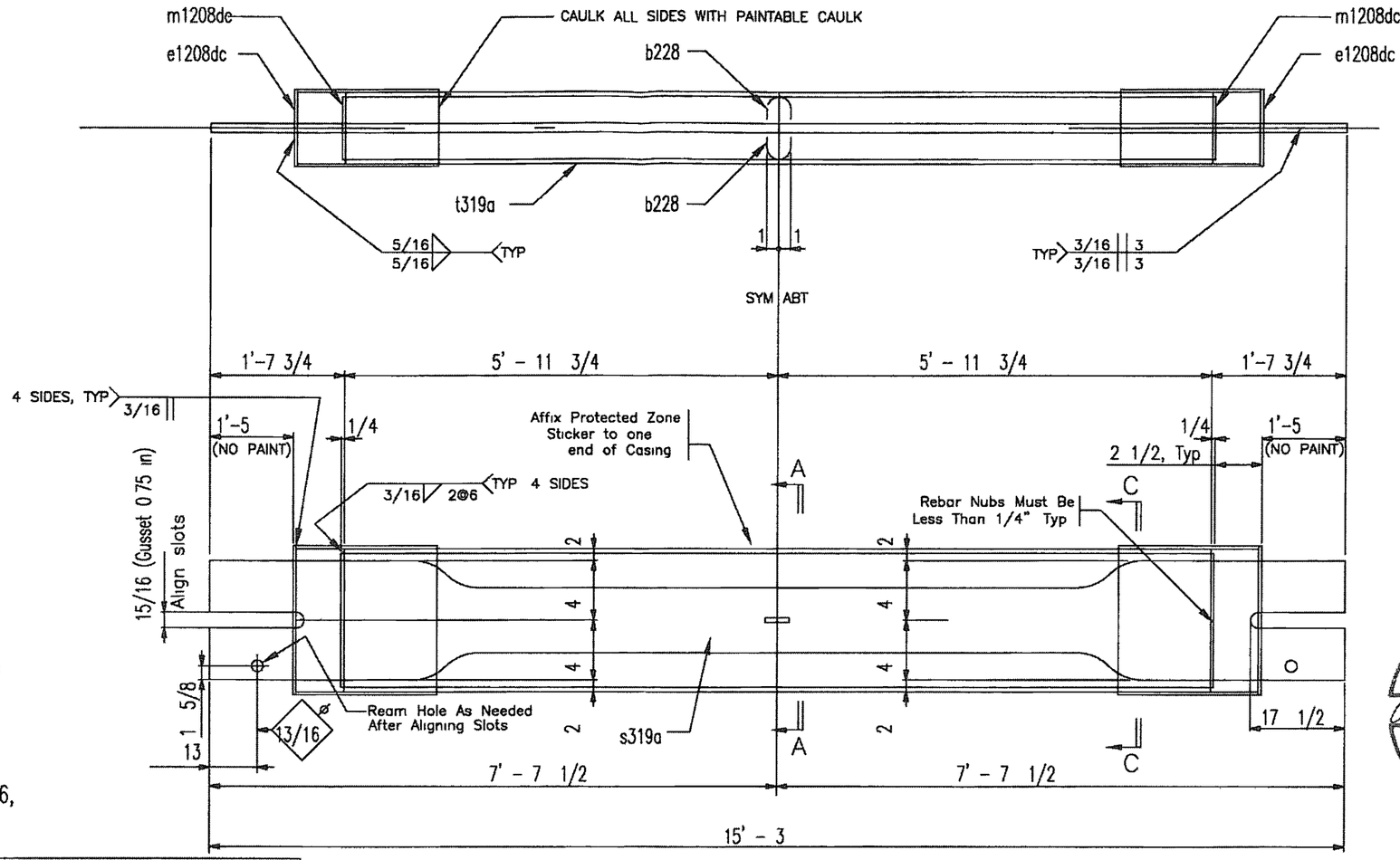
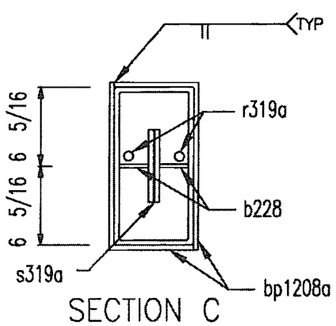
JOB NO 110127 DWG NO s282a



5 BRACE WC319a



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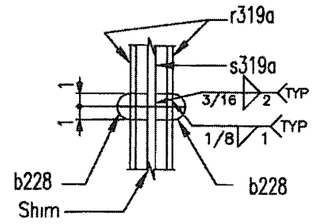

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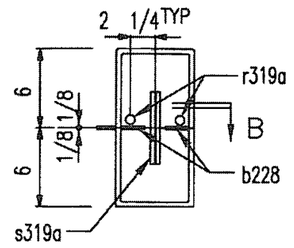

SECTION A

BRACE LOCATION(S)

17 & 21/DD-EE at el 5-6, 21/CC-DD at el 5-6,
13/CC-DD & DD-EE at el 4-5

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
5	Brace	15	3	WC319a	BRB2
5	HSS12X8X1/4	11	11 1/2	t319a	A500-Gr B
20	PL 1/4 x 24	1	8 1/4	bp1208a	A36
10	FB 3/4x 8	15	3	s319a	A36-LY
10	PL 1/4x2	0	2 7/8	b228	A36
10	PL 1/4 x 7 1/2	0	11 1/2	m1208dc	A36
10	#4 rebar	12	5	r319a	
10	PL 1/2 x 8 1/8	1	0 1/8	e1208dc	A36

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
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△		
△		
△ A	2/3/2011 SNJ	Approval

SHOP PLACE PIECE MARK ON LEFT HAND END OF MEMBER AS DETAILED
ERECT MEMBER WITH PIECE MARK TO END SHOWN ON PLACING PLAN

MATERIAL A36 & AS NOTED
SHOP PAINT None
APPROX BRACE WT 1951 EA
SURFACE PREPARATION -
APPROX CONCR 0.243 YRDS EA

SUBJECT WC319a
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication
ZONE SS BRB NO 2 4 6 9 11
ERECTION DWG ENG DWG PSS3 1
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www starseismic net
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FAX 1-435-655-0073

JOB NO 110127 DWG NO WC319a

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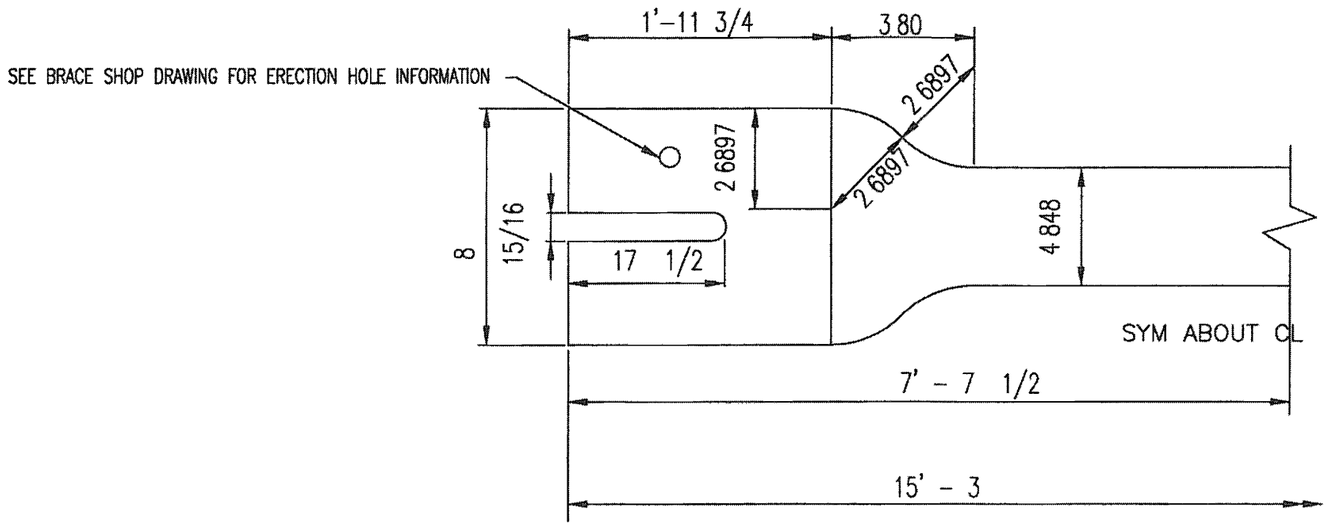

Lysc=135 5 in

Total Brace Asc= 7 27 in^2

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
10	STRAND PLATES			s319a	
10	FB 3/4x 8	15	3	s319a	A36-LY Heat#120-0237

10 STRAND PLATES s319a

SHOP NOTE DEBURR ALL CUT EDGES

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
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SUBJECT s319a

PROJECT POP Parking Structure 2 Expansion

LOCATION Portland, OR

CUSTOMER L&M Industrial Fabrication

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FAX 1-435-655-0073

JOB NO 110127 DWG NO s319a

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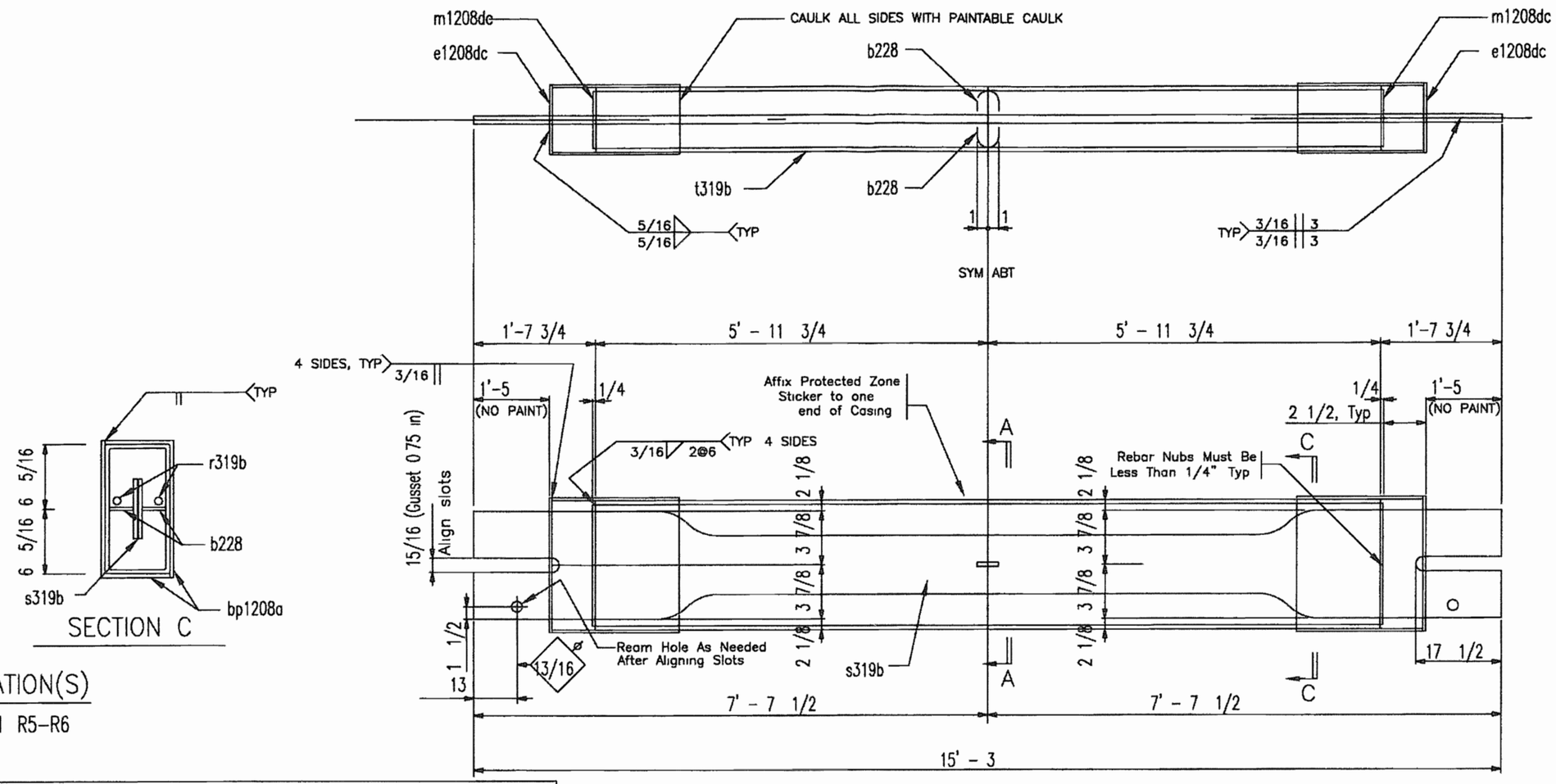
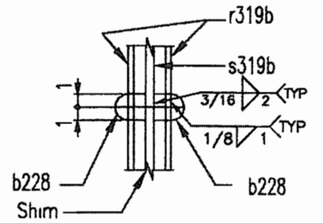


SECTION C



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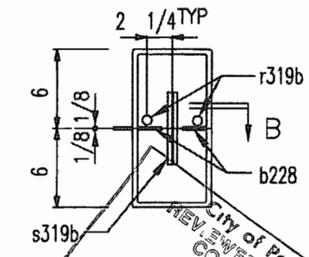

SECTION A

BRACE LOCATION(S)

17/CC-DD at el R5-R6

1 BRACE WC319b

City of Portland
REVIEWED FOR CODE COMPLIANCE
MAR 17 2011
Permit Number

REGISTERED PROFESSIONAL ENGINEER
16430
OREGON
MAR 16 1993
ARGAN N. JOHNSON, JR.
EXP 16/30/11

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BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
1	Brace	15	3	WC319b	BRB2
1	HSS12X8X1/4	11	11 1/2	t319b	A500-Gr B
4	PL 1/4 x 24	1	8 1/4	bp1208a	A36
2	FB 3/4x 7 3/4	15	3	s319b	A36-LY
2	PL 1/4x2	0	2 7/8	b228	A36
2	PL 1/4 x 7 1/2	0	11 1/2	m1208dc	A36
2	#4 rebar	12	5	r319b	
2	PL 1/2 x 8 1/8	1	0 1/8	e1208dc	A36

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
△		
△		
△		
△ A	2/3/2011 SNJ	Approval

SHOP PLACE PIECE MARK ON LEFT HAND END OF MEMBER AS DETAILED
ERECT MEMBER WITH PIECE MARK TO END SHOWN ON PLACING PLAN

MATERIAL A36 & AS NOTED
SHOP PAINT None
APPROX BRACE WT 1943 EA
SURFACE PREPARATION -
APPROX CONCR 0.243 YRDS EA

SUBJECT WC319b
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication
ZONE SS BRB NO 8
ERECTION DWG ENG DWG PSS3.1
DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

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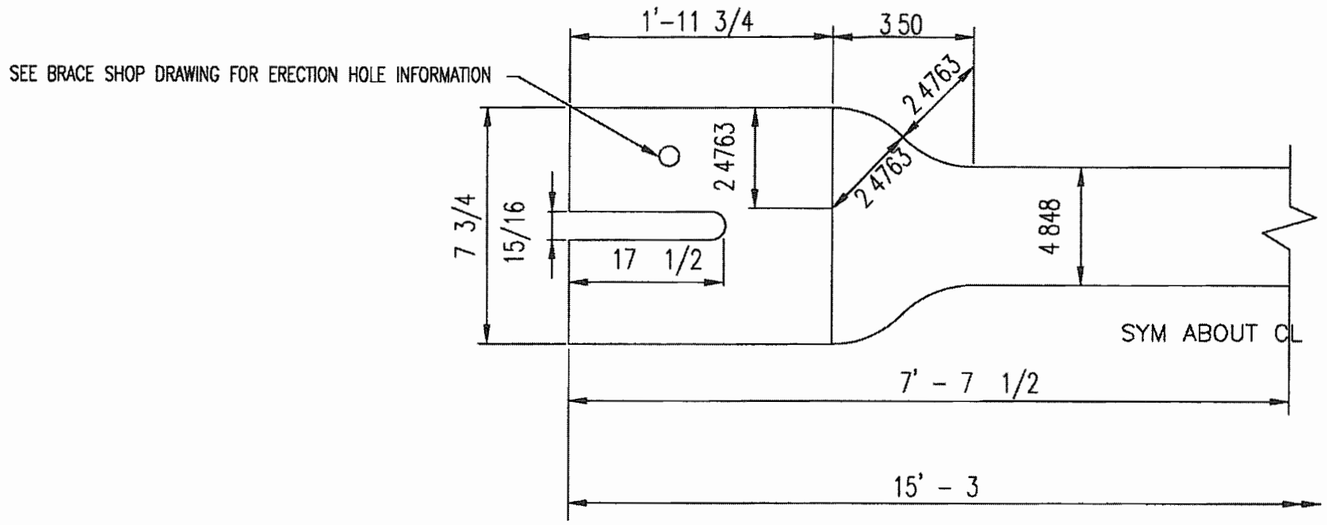
JOB NO 110127 DWG NO WC319b



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City of Portland
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DOCUMENT REVIEWED FOR CODE COMPLIANCE
MAR 17 2011
REGISTERED PROFESSIONAL ENGINEER
16430
OREGON
MAR 18 1993
ARGAN N JOHNSON, JR
exp 6/13/11

Lysc=135 5 in

Total Brace Asc= 7 27 in^2

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
2	STRAND PLATES			s319b	
2	FB 3/4x 7 3/4	15	3	s319b	A36-LY Heat#120-0237

2 STRAND PLATES s319b

SHOP NOTE DEBURR ALL CUT EDGES

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
A	2/3/2011 SNJ	Approval

SUBJECT s319b
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication

DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

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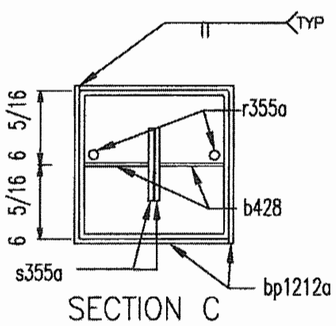
FAX 1-435-655-0073

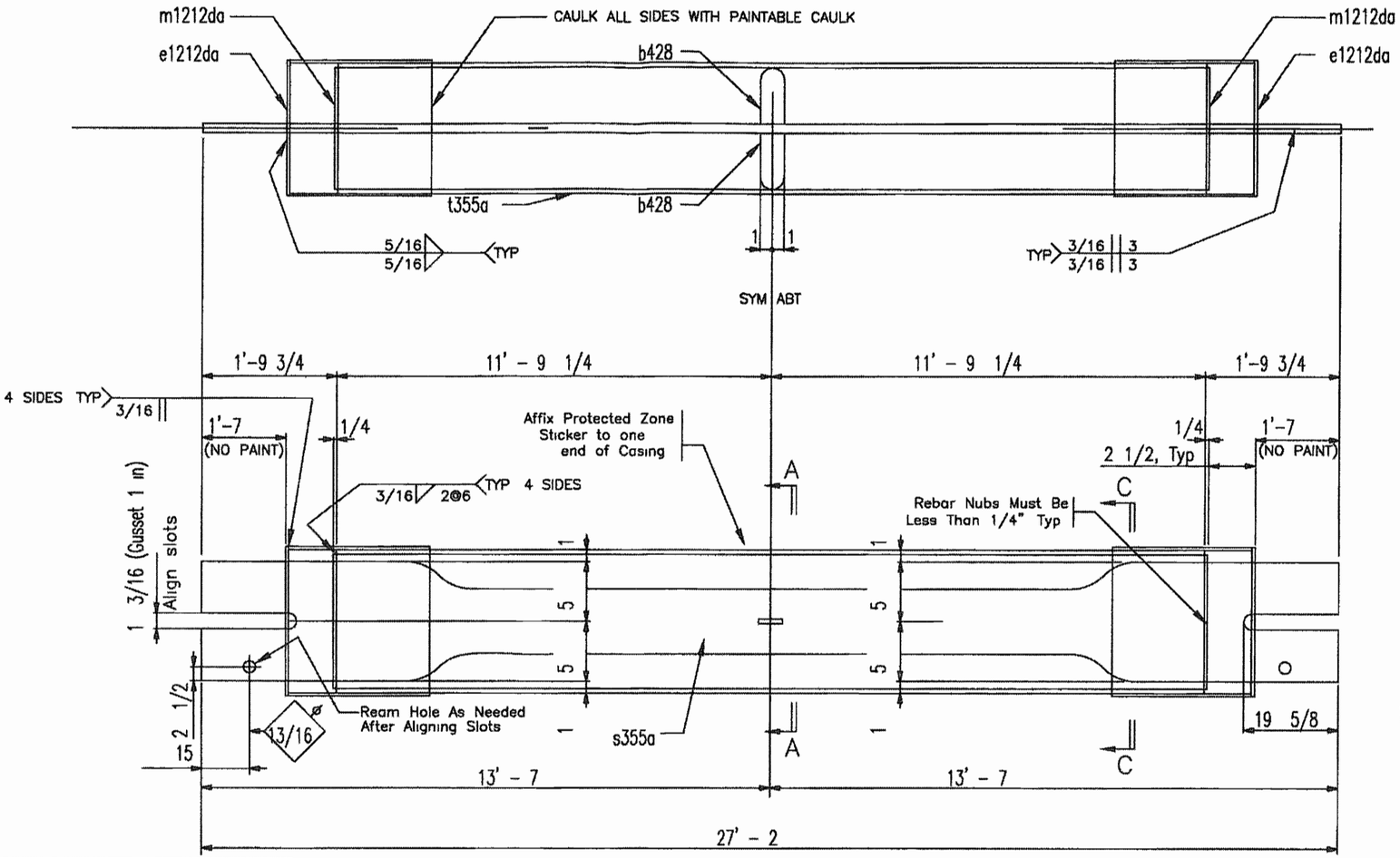
JOB NO 110127 DWG NO s319b

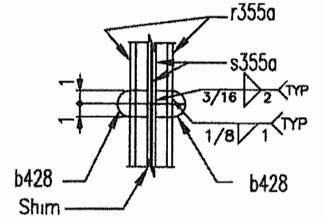


SECTION C

BRACE LOCATION(S)

AA/12-13 at el 5-6





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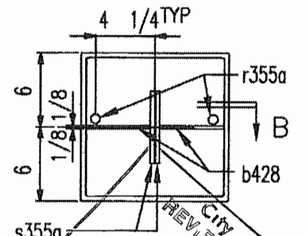

SECTION A

City of Portland
REVIEWED FOR CODE COMPLIANCE
APR 17 2011
PERMIT NUMBER

REGISTERED PROFESSIONAL ENGINEER
16430
OREGON
MAR 18 1993
ARGAN N JOHNSON, JR
exp 6/30/MAR 1 0 2011

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DOCUMENT SERVICES

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
1	Brace	27	2	WC355a	BRB4
1	HSS12X12X1/4	23	6 1/2	t355a	A500-Gr B
4	PL 1/4 x 24	2	0 1/4	bp1212a	A36
2	FB 3/4x 10	27	2	s355a	A36-LY
2	PL 1/4x2	0	4 7/8	b428	A36
2	PL 1/4 x 11 1/2	0	11 1/2	m1212da	A36
2	#4 rebar	24	0	r355a	
2	PL 1/2 x 12 1/8	1	0 1/8	e1212da	A36

1 BRACE WC355a

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
A	2/3/2011 SNJ	Approval

SHOP PLACE PIECE MARK ON LEFT HAND END OF MEMBER AS DETAILED
ERECT MEMBER WITH PIECE MARK TO END SHOWN ON PLACING PLAN

MATERIAL A36 & AS NOTED
APPROX BRACE WT 5004 EA
APPROX CONCR 0.752 YRDS EA
SHOP PAINT None
SURFACE PREPARATION -

SUBJECT WC355a
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication
ZONE SS BRB NO 18
ERECTION DWG ENG DWG PSS3 2
DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

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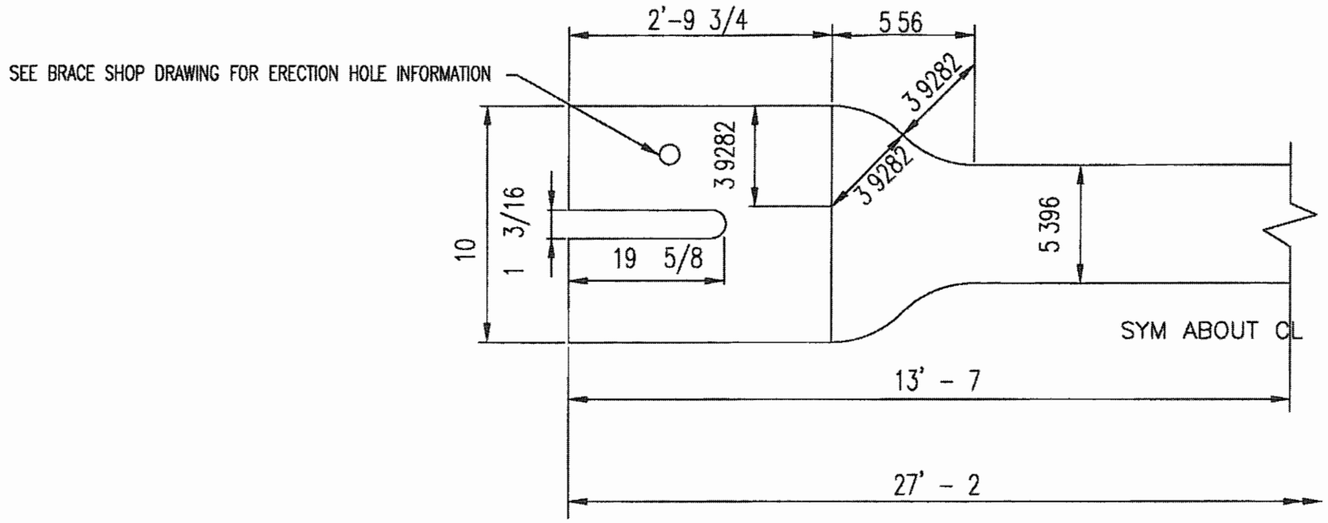
JOB NO 110127 DWG NO WC355a



City of Portland
REVIEWED FOR CODE COMPLIANCE
Permit Number
RECEIVED
MAR 1 2 2011
BDS
DOCUMENT SERVICES

Lysc=258 5 in

Total Brace Asc= 8 09 in^2

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
2	STRAND PLATES			s355a	
2	FB 3/4x 10	27	2	s355a	A36-LY Heat#120-0237

2 STRAND PLATES s355a

SHOP NOTE DEBURR ALL CUT EDGES

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
A	2/3/2011 SNJ	Approval

SUBJECT s355a
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication

DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

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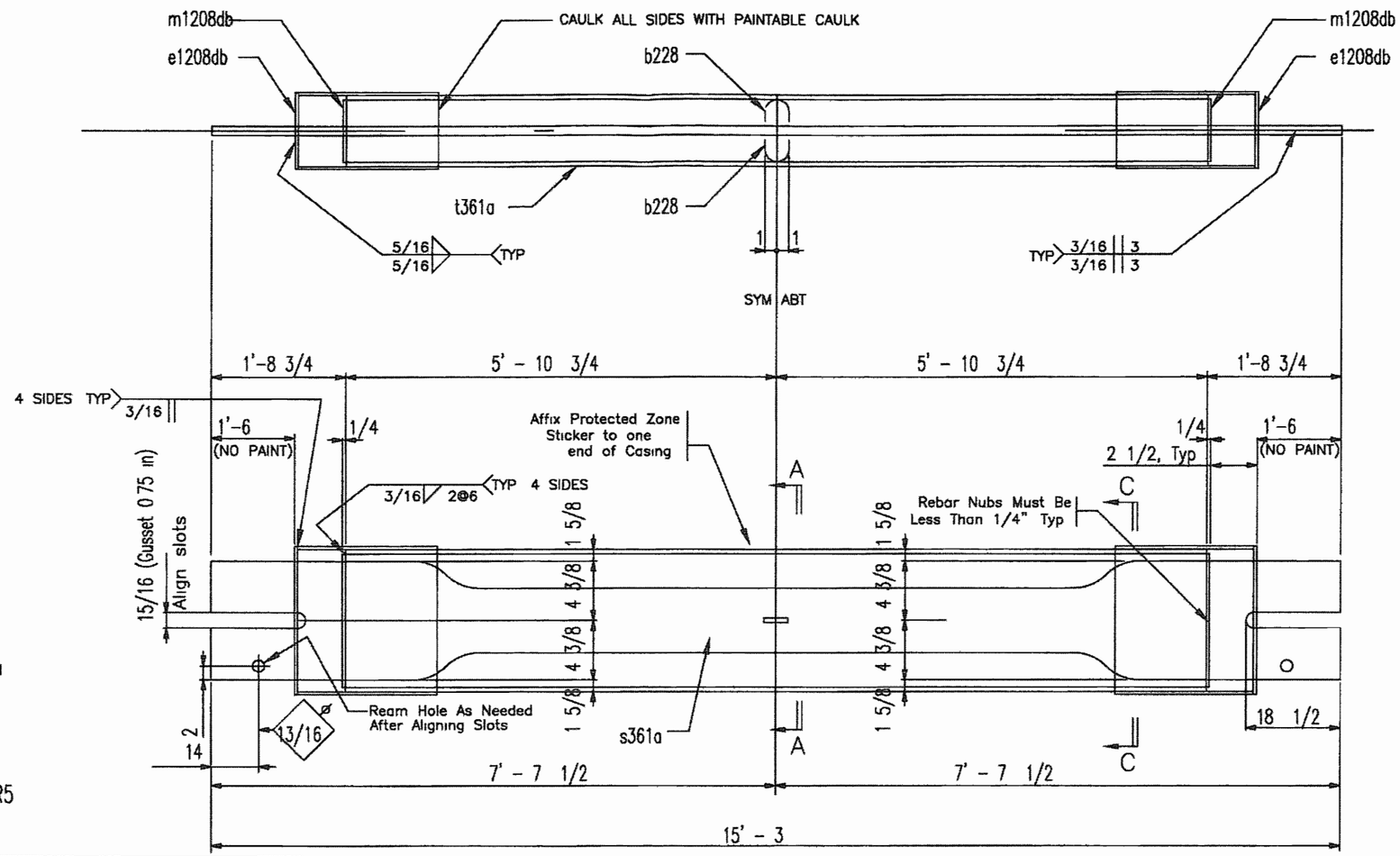
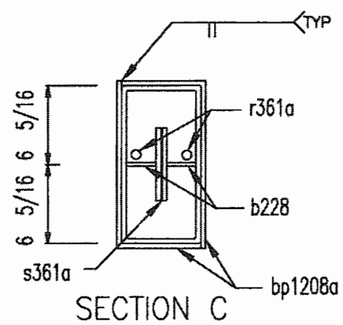
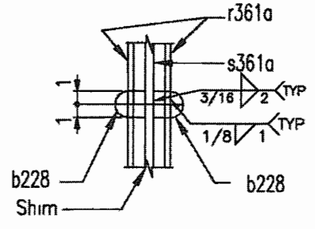
JOB NO 110127 DWG NO s355a

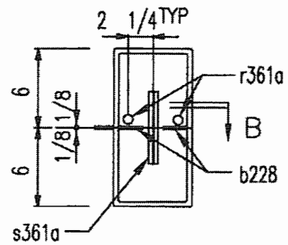




SECTION C



SECTION B



SECTION A

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MAR 1 0 2011

REGISTERED PROFESSIONAL ENGINEER 16430 OREGON MAR 16 1999 ARGAN N JOHNSON JR
exp 6/30/11

City of Portland
REVIEWED FOR CODE COMPLIANCE
MAR 17 2011
Permit Number

BRACE LOCATION(S)

21/CC-DD & DD-EE at el 4-5, 17/CC-DD R4-R5

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
3	Brace	15	3	WC361a	BRB3
3	HSS12X8X1/4	11	9 1/2	t361a	A500-Gr B
12	PL 1/4 x 24	1	8 1/4	bp1208a	A36
6	FB 3/4x 8 3/4	15	3	s361a	A36-LY
6	PL 1/4x2	0	2 7/8	b228	A36
6	PL 1/4 x 7 1/2	0	11 1/2	m1208db	A36
6	#4 rebar	12	3	r361a	
6	PL 1/2 x 8 1/8	1	0 1/8	e1208db	A36

3 BRACE WC361a

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
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A	2/3/2011 SNJ	Approval

SHOP PLACE PIECE MARK ON LEFT HAND END OF MEMBER AS DETAILED
ERECT MEMBER WITH PIECE MARK TO END SHOWN ON PLACING PLAN

MATERIAL A36 & AS NOTED
SHOP PAINT None
APPROX BRACE WT 1975 EA
SURFACE PREPARATION -
APPROX CONCR 0.237 YRDS EA

SUBJECT WC361a
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication
ZONE SS BRB NO 1 3 7
ERECTION DWG ENG DWG PSS3 1
DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

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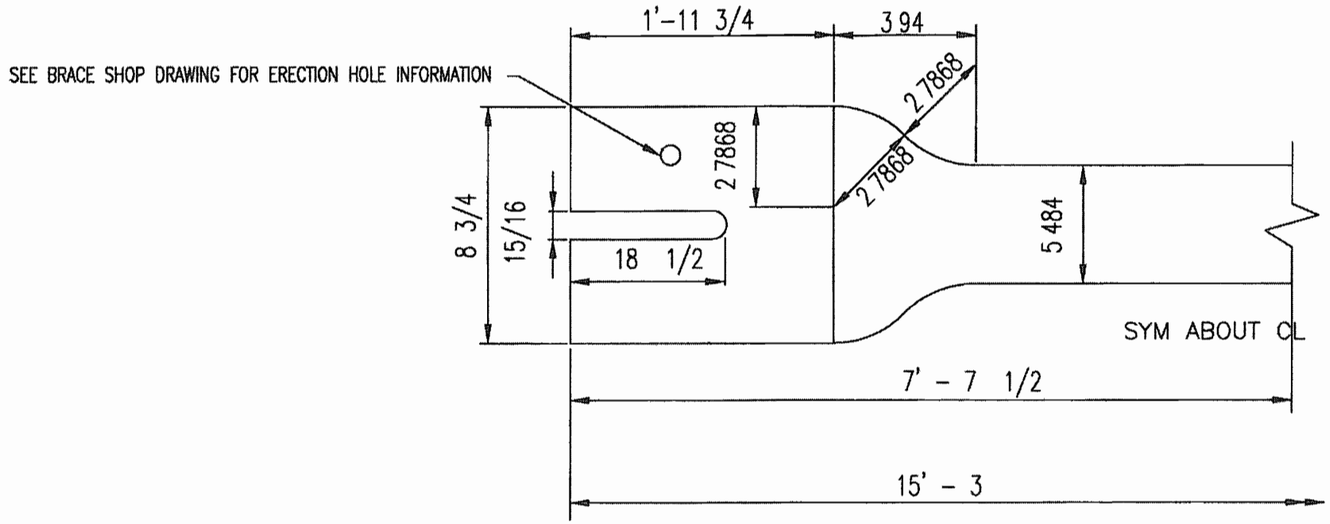
FAX 1-435-655-0073

JOB NO 110127 DWG NO WC361a



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City of Portland
REVIEWED FOR
CODE COMPLIANCE
MAR 17 2011
Permit Number

REGISTERED PROFESSIONAL ENGINEER
16439
OREGON
MAR 16 1993
ARGAN N JOHNSON JR
exp 6/30/12

Lysc=135 5 in

Total Brace Asc= 8 23 in^2

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
6	STRAND PLATES			s361a	
6	FB 3/4x 8 3/4	15	3	s361a	A36-LY Heat#120-0237

6 STRAND PLATES s361a

SHOP NOTE DEBURR ALL CUT EDGES

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
A	2/3/2011 SNJ	Approval

SUBJECT s361a
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication

DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

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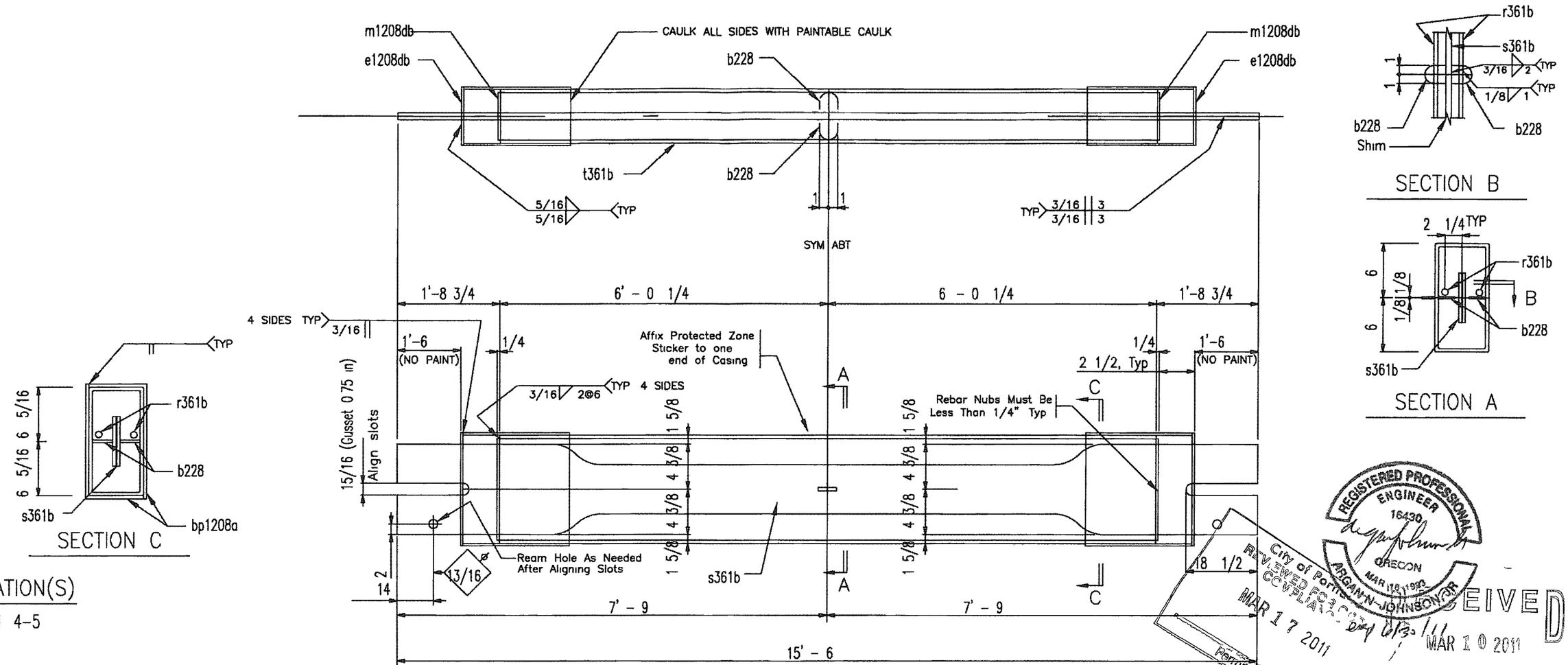
JOB NO 110127 DWG NO s361a



BRACE LOCATION(S)

17/DD-EE at el 4-5

1 BRACE WC361b

REGISTERED PROFESSIONAL ENGINEER 16430 OREGON MAR 18 1993 ARGANN-JOHNSON, JR

City of Portland REVIEWED FOR CODE COMPLIANCE MAR 17 2011 Permit Number

RECEIVED MAR 10 2011 BDS DOCUMENT SERVICES

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
1	Brace	15	6	WC361b	BRB3
1	HSS12X8X1/4	12	0 1/2	t361b	A500-Gr B
4	PL 1/4 x 24	1	8 1/4	bp1208a	A36
2	FB 3/4x 8 3/4	15	6	s361b	A36-LY
2	PL 1/4x2	0	2 7/8	b228	A36
2	PL 1/4 x 7 1/2	0	11 1/2	m1208db	A36
2	#4 rebar	12	6	r361b	
2	PL 1/2 x 8 1/8	1	0 1/8	e1208db	A36

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
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△ A	2/3/2011 SNJ	Approval

SHOP PLACE PIECE MARK ON LEFT HAND END OF MEMBER AS DETAILED
ERECT MEMBER WITH PIECE MARK TO END SHOWN ON PLACING PLAN

MATERIAL A36 & AS NOTED
SHOP PAINT None
APPROX BRACE WT 2011 EA
SURFACE PREPARATION -
APPROX CONCR 0.242 YRDS EA

SUBJECT WC361b
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication
ZONE SS BRB NO 5
ERECTION DWG ENG DWG PSS3 1
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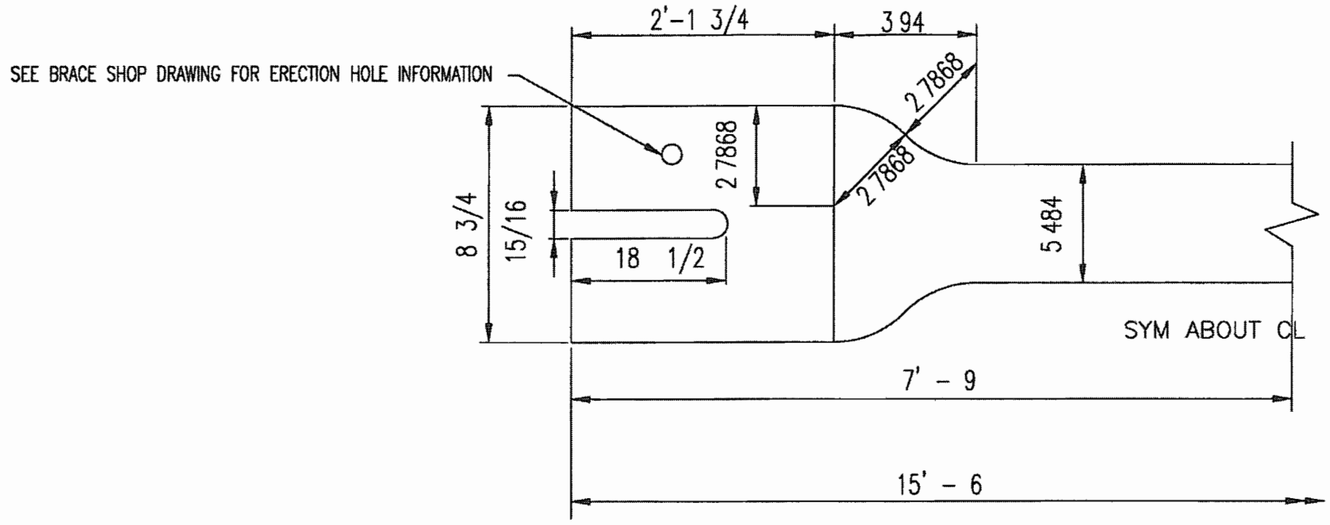
JOB NO 110127 DWG NO WC361b



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Permit Number

REGISTERED PROFESSIONAL ENGINEER
16480
OREGON
MAR 18 1993
ARGAN N JOHNSON JR
exp 6/30/11

Lysc=134 5 in

Total Brace Asc= 8 23 in^2

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
2	STRAND PLATES			s361b	
2	FB 3/4x 8 3/4	15	6	s361b	A36-LY Heat#120-0237

2 STRAND PLATES s361b

SHOP NOTE DEBURR ALL CUT EDGES

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
A	2/3/2011 SNJ	Approval

SUBJECT s361b
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication

DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

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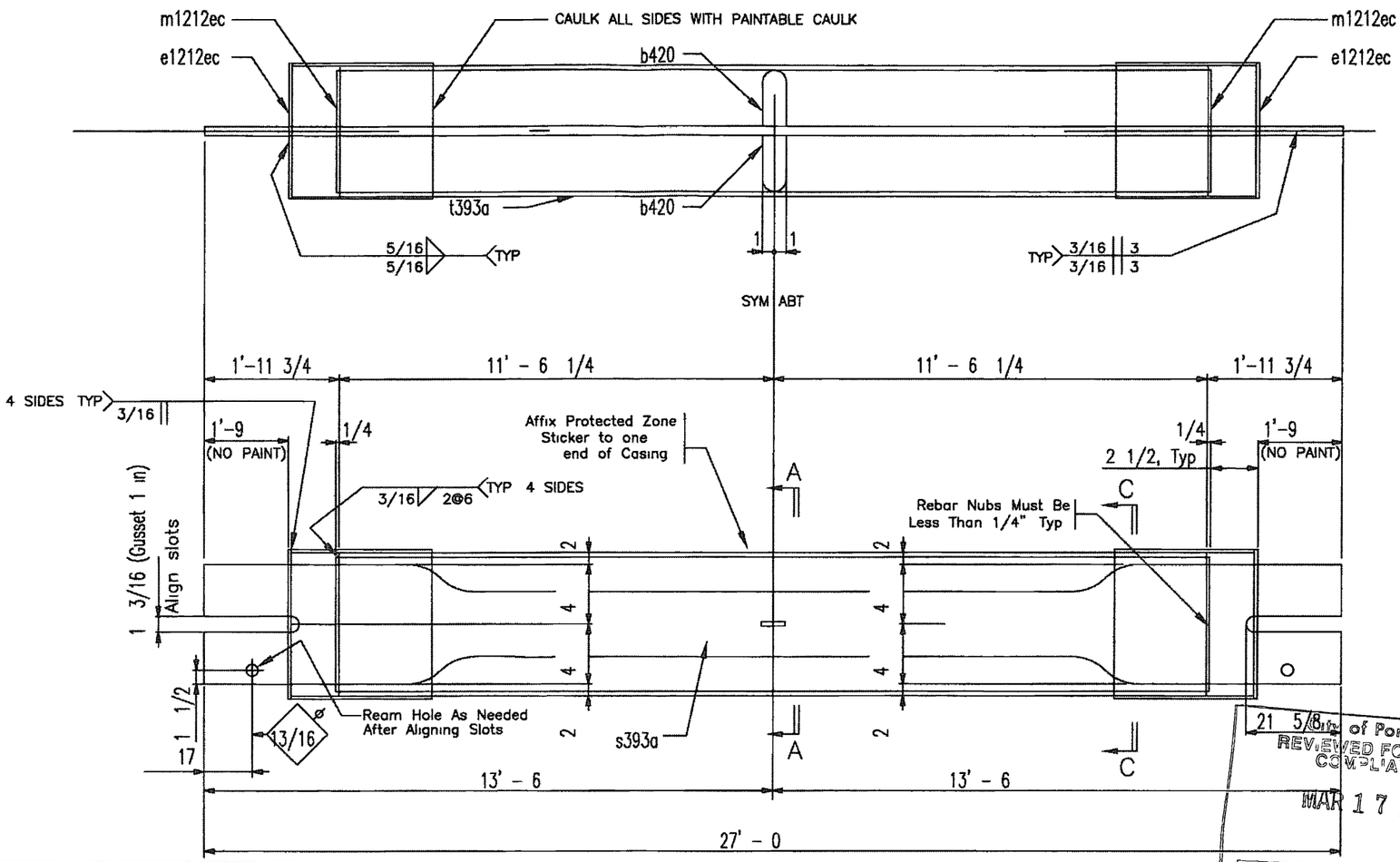
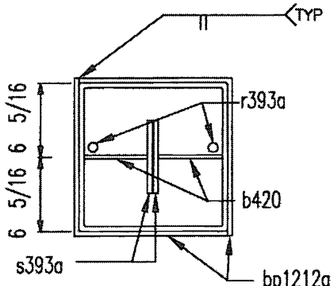
JOB NO 110127 DWG NO s361b





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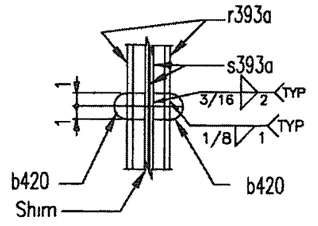
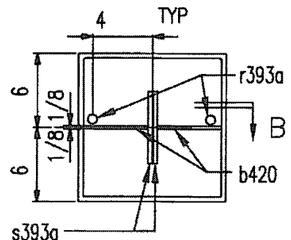


SECTION B



SECTION A

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MAR 10 2011
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REGISTERED PROFESSIONAL ENGINEER
16430
OREGON
MAR 15 1993
JORGAN N JOHNSON JR.
exp 6/30/11

21 5/8 of Portland
REVIEWED FOR CODE COMPLIANCE
MAR 17 2011
Permit Number

BRACE LOCATION(S)

GG/14-12 & GG/22-20 at el 5-6

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
2	Brace	27	0	WC393a	BRB5
2	HSS12X12X1/4	23	0 1/2	t393a	A500-Gr B
8	PL 1/4 x 24	2	0 1/4	bp1212a	A36
4	FB 1 x 8	27	0	s393a	A36-LY
4	PL 1/4x2	0	4 5/8	b420	A36
4	PL 1/4 x 11 1/2	0	11 1/2	m1212ec	A36
4	#4 rebar	23	6	r393a	
4	PL 1/2 x 12 1/8	1	0 1/8	e1212ec	A36

2 BRACE WC393a

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
△	—	
△	—	
△	—	
A	2/3/2011 SNJ	Approval

SHOP PLACE PIECE MARK ON LEFT HAND END OF MEMBER AS DETAILED
ERECT MEMBER WITH PIECE MARK TO END SHOWN ON PLACING PLAN

MATERIAL A36 & AS NOTED
SHOP PAINT None
APPROX BRACE WT 5013 EA
SURFACE PREPARATION -
APPROX CONCR 0.727 YRDS EA

SUBJECT WC393a
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication
ZONE SS BRB NO 16 20
ERECTION DWG ENG DWG PSS3 2
DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

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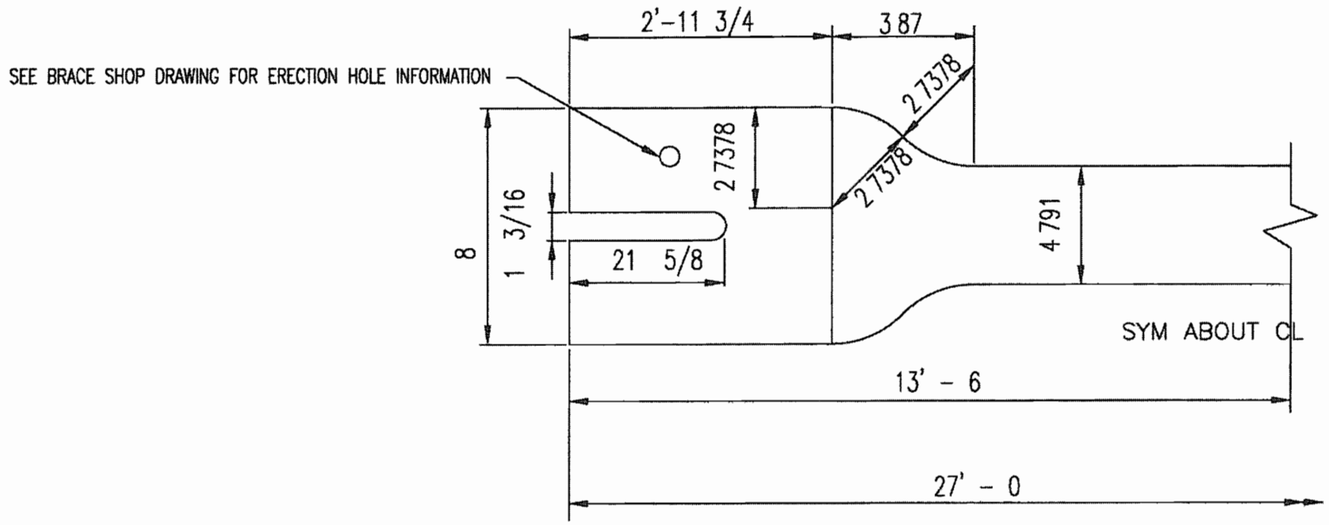
FAX 1-435-655-0073

JOB NO 110127 DWG NO WC393a



RECEIVED
MAR 1 0 2011
BDS
DOCUMENT SERVICES

City of Portland
REVIEWED FOR CODE COMPLIANCE
MAR 17 2011
Permit Number

REGISTERED PROFESSIONAL ENGINEER
16439
OREGON
MAR 16 1991
ARGAN N JOHNSON JR.
exp 6/30/11

Lysc=252 5 in

Total Brace Asc= 9 58 in^2

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
4	STRAND PLATES			s393a	
4	FB 1 x 8	27	0	s393a	A36-LY Heat#110-0242

4 STRAND PLATES s393a

SHOP NOTE DEBURR ALL CUT EDGES

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
△	—	
△	—	
△	—	
△	—	
△	—	
△	—	
A	2/3/2011 SNJ	Approval

SUBJECT s393a
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication

DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

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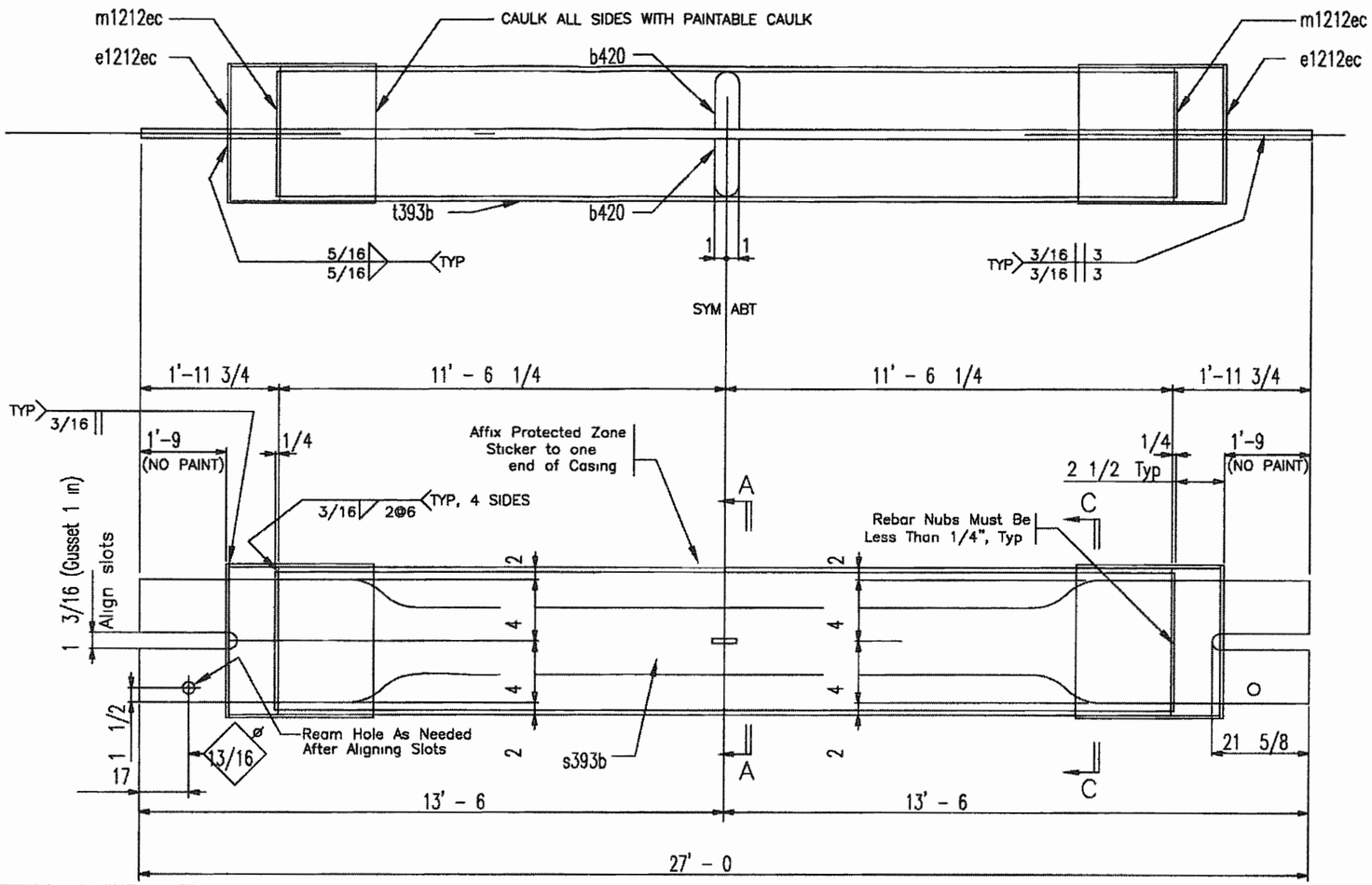
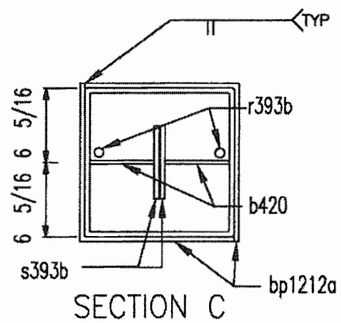
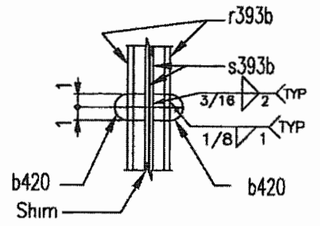
JOB NO 110127 DWG NO s393a

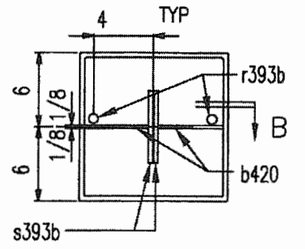




SECTION C



SECTION B



RECEIVED
MAR 10 2011
BDS
ENT SERVICES

REGISTERED PROFESSIONAL ENGINEER
16432
OREGON
MAR 16 1982
ARGANN JOHNSON
of Portland
REVIEWED FOR CODE COMPLIANCE
MAR 17 2011
Permit Number

BRACE LOCATION(S)

AA/21-22 at el 5-6, AA/12-13 at el 4-5

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
2	Brace	27	0	WC393b	BRB5
2	HSS12X12X1/4	23	0 1/2	t393b	A500-Gr B
8	PL 1/4 x 24	2	0 1/4	bp1212a	A36
4	FB 1 x 8	27	0	s393b	A36-LY
4	PL 1/4x2	0	4 5/8	b420	A36
4	PL 1/4 x 11 1/2	0	11 1/2	m1212ec	A36
4	#4 rebar	23	6	r393b	
4	PL 1/2 x 12 1/8	1	0 1/8	e1212ec	A36

2 BRACE WC393b

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
A	2/3/2011 SNJ	Approval

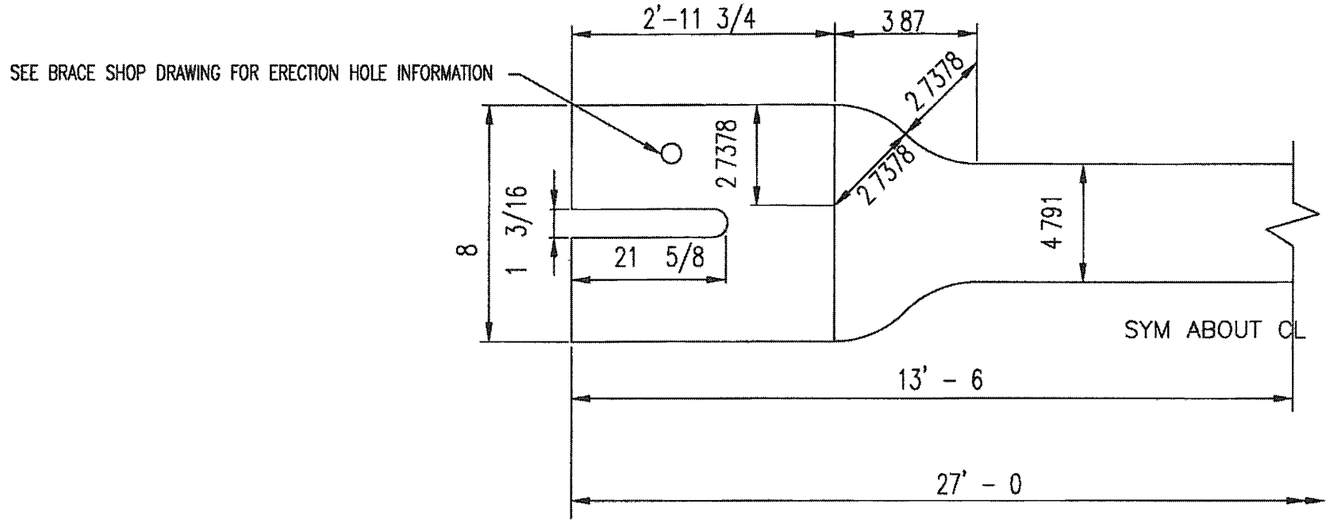
SHOP PLACE PIECE MARK ON LEFT HAND END OF MEMBER AS DETAILED
ERECT MEMBER WITH PIECE MARK TO END SHOWN ON PLACING PLAN

MATERIAL A36 & AS NOTED
SHOP PAINT None
APPROX BRACE WT 5013 EA
SURFACE PREPARATION -
APPROX CONCR 0.727 YRDS EA

SUBJECT WC393b
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication
ZONE SS BRB NO 14 17
ERECTION DWG ENG DWG PSS32
DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

STAR SEISMIC™
6300 N SAGEWOOD DR
SUITE H #511
PARK CITY, UT 84098
www starseismic net
PHONE 1-435-940-9222
FAX 1-435-655-0073

JOB NO 110127 DWG NO WC393b



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City of Portland
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MAR 17 2011
Permit Number

REGISTERED PROFESSIONAL ENGINEER
16430
OREGON
MAR 16 1993
ARGAN N JOHNSON JR
exp 6/30/11

Lysc=252 5 in

Total Brace Asc= 9 58 in^2

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
4	STRAND PLATES			s393b	
4	FB 1 x 8	27	0	s393b	A36-LY Heat#110-0242

4 STRAND PLATES s393b

SHOP NOTE DEBURR ALL CUT EDGES

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
A	2/3/2011 SNJ	Approval

SUBJECT s393b
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication

DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

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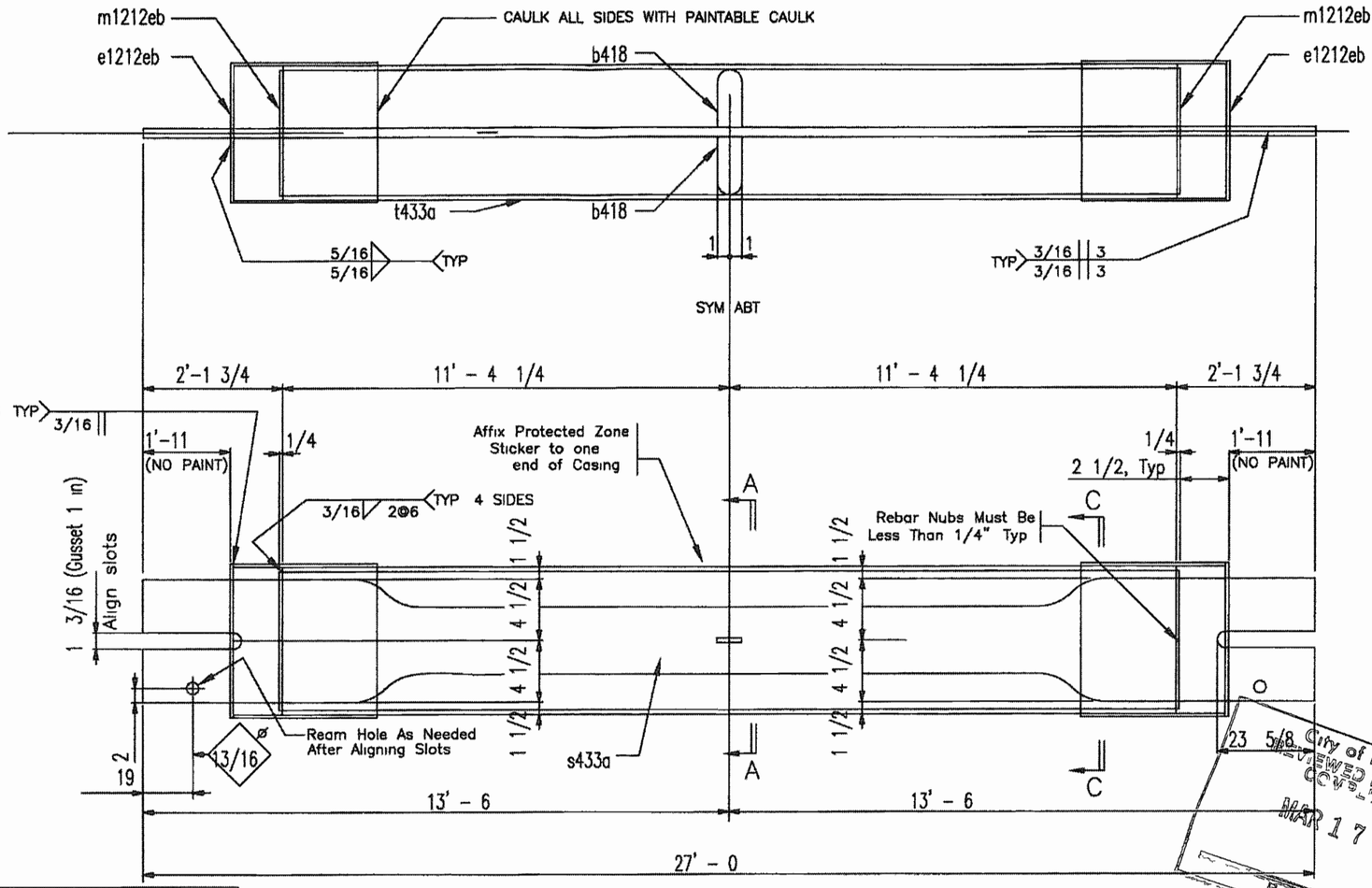
6300 N SAGEWOOD DR
SUITE H #511
PARK CITY, UT 84098
www starseismic net
PHONE 1-435-940-9222
FAX 1-435-655-0073

JOB NO 110127 DWG NO s393b



BRACE LOCATION(S)

AA/21-22 & GG/14-12 & GG/22-20 at el 4-5

BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
3	Brace	27	0	WC433a	BRB6
3	HSS12X12X5/16	22	8 1/2	t433a	A500-Gr B
12	PL 1/4 x 24	2	0 1/4	bp1212a	A36
6	FB 1 x 9	27	0	s433a	A36-LY
6	PL 1/4x2	0	4 9/16	b418	A36
6	PL 1/4 x 11 1/2	0	11 1/2	m1212eb	A36
6	#4 rebar	23	2	r433a	
6	PL 1/2 x 12 1/8	1	0 1/8	e1212eb	A36

3 BRACE WC433a

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
	—	
	—	
	—	
A	2/3/2011 SNJ	Approval

SHOP PLACE PIECE MARK ON LEFT HAND END OF MEMBER AS DETAILED
ERECT MEMBER WITH PIECE MARK TO END SHOWN ON PLACING PLAN

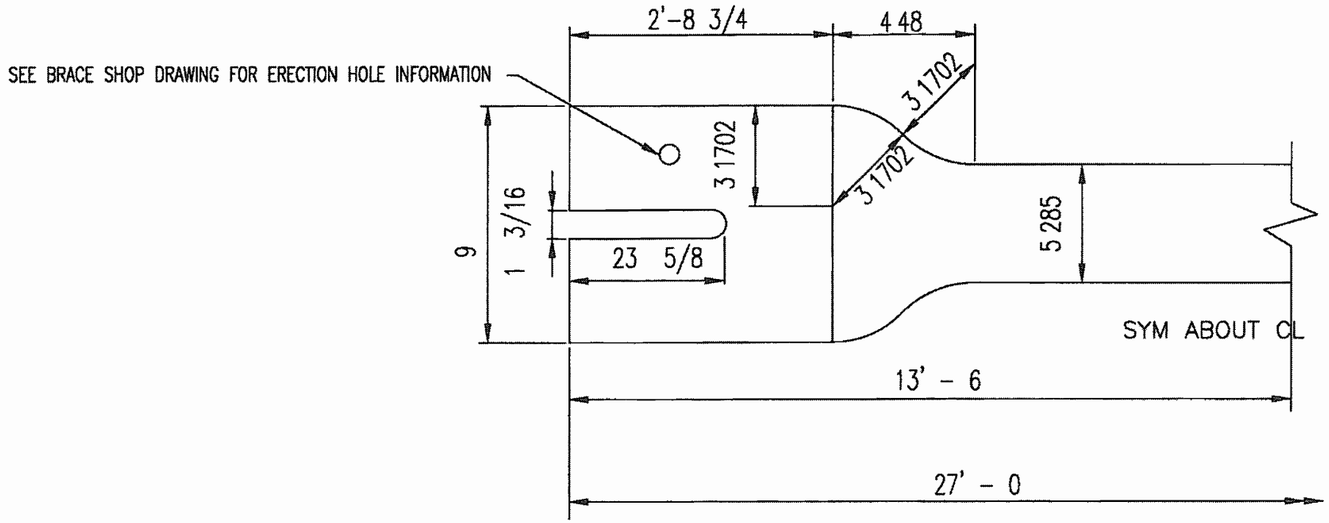
MATERIAL A36 & AS NOTED
APPROX BRACE WT 5201 EA
APPROX CONCR 0.694 YRDS EA
SHOP PAINT None
SURFACE PREPARATION -

SUBJECT WC433a
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication
ZONE SS BRB NO 13 15 19
ERECTION DWG ENG DWG PSS3 2
DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

STAR SEISMIC™
6300 N SAGEWOOD DR
SUITE H #511
PARK CITY, UT 84098
www starseismic net
PHONE 1-435-940-9222
FAX 1-435-655-0073

JOB NO 110127 DWG NO WC433a

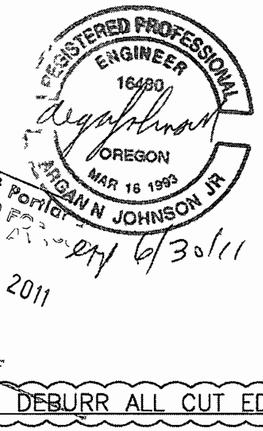
RECEIVED
MAR 10 2011
BDS
DEVELOPMENT SERVICES
REGISTERED PROFESSIONAL ENGINEER 16430 OREGON MAR 19 1993 MEGAN N JOHNSON, JR
REVIEWED FOR CODE COMPLIANCE
MAR 17 2011
exp 6/30/11
Permit Number



Lysc=258 5 in

Total Brace Asc= 10 57 in^2

RECEIVED
MAR 1 0 2011
BDS
DOCUMENT SERVICES



BILL OF MATERIAL

NO PCS	DESCRIPTION	LENGTH FT	LENGTH IN	MARK	REMARKS
6	STRAND PLATES			s433a	
6	FB 1 x 9	27	0	s433a	A36-LY Heat#110-0242

6 STRAND PLATES s433a

SHOP NOTE DEBURR ALL CUT EDGES

NO	DATE AND CHECKED BY	DESCRIPTION OF REVISION
A	2/3/2011 SNJ	Approval

SUBJECT s433a
PROJECT POP Parking Structure 2 Expansion
LOCATION Portland, OR
CUSTOMER L&M Industrial Fabrication

DRAWN BY SNJ DATE 2/2/2011 CHECKED BY BP

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6300 N SAGEWOOD DR
SUITE H #511
PARK CITY, UT 84098
www starseismic net
PHONE 1-435-940-9222
FAX 1-435-655-0073

JOB NO 110127 DWG NO s433a

Design Criteria. 2010 IBC
95 MPH (3 Second Gust)
Exposure B
I_W=1 00

Window System KAWNEER

Storefront 450-VG- Offset (1-¾" x 4-½")

450-VG-019	I_X= 2 985 in^4	S_X= 1 244 in^3
450-SSG-005	I_X= 2 445 in^4	S_X= 1 352 in^3
450-VG-012	I_X= 3 074 in^4	S_X= 1 192 in^3

Strength Criteria: F_b = 15152 psi for 6063 T6 Aluminum

Deflection Criteria $\Delta_{allow} = l/175$ Overall Mullion Deflection

Required Moment of Inertia (I_x) Equation $I_{rqd} = \frac{21wl^3}{640000}$

Equation Assumptions
- Uniform Distributed Load
- Deflection $\Delta = l/175$
- $E_{AL} = 10x10^6$psi
- l In Terms of Feet

RECEIVED
MAY 0 6 REC'D
JE DUNN CONSTRUCTION CO.

Equilibrium Engineers LLC
DATE 5/23/11
ALL CONSTRUCTION COORDINATION OF HIS WORK WITH THAT OF ALL
AND THE SATISFACTORY PERFORMANCE OF HIS WORK

REGISTERED PROFESSIONAL ENGINEER
18346PE
OREGON
MARCH 19, 1982
PEDER R GOLBERG

RECEIVED
MAY 25 2011
BDS
DOCUMENT SERVICES

REVIEWED BY JE DUNN NORTHWEST INC
REVIEWED BY CD
DATE 5/12/11
SUBMITTAL # 08413.3R

James G Pierson, Inc
Consulting Structural Engineers
320 S W Stark, Suite 535 Portland, Oregon 97204
Tel (503) 226-1286 Fax (503) 226 3130

Project: PROVIDENCE PARKING STRUCTURE 2	Job no
Location: 4400 NE HALSEY PDX, OR	Date
Client: COWER GLASS	Sheet no: 1

CULVER GLASS

2011

PROVIDENCE
PARKING STRUCTURE 2

JON R JURGENS & ASSOCIATES, INC
15455 NW GREENBRIAR PARKWAY - SUITE 260
BEAVERTON, OREGON 97006
PHONE 503-690-1779

CULVER GLASS
2619 NW INDUSTRIAL BLDG B SUITE #3
PORTLAND, OREGON 97210
PHONE 503-226-2520
FAX 503-226-0972

ARCH SET – 12/13/10

FIELD VERIFY DIMENSIONS

DATE	ACTION

CULVER GLASS

PROVIDENCE HOSPITAL PARKING STRUCTURE 2
4401 NE HALEY ST
PORTLAND OREGON 97213

10

INDEX

1.0	COVER SHEET
1.1	INDEX, GENERAL NOTES & SCHEDULES
2.0	FLOOR PLAN
2.1	FLOOR PLAN
2.2	FLOOR PLAN
4.0	ELEVATIONS
4.1	DETAILS
4.2	DETAILS
4.3	DETAILS
4.4	DETAILS
4.5	DETAILS
4.6	DETAILS

PRODUCT INFORMATION

) EXTERIOR PA G

) RAM O WHEE VIC OF SE CLED

TWI TITANIU ALSPAR WI 9C2053

CLASS. SEL SCHEDUL

DOOR SE CHEDUL

JOHN SE LAN DOW CORNING BUILDING SEAL

GENERAL NOTES

AWING VIEWED FROM TERIOR

2) GLAZING CONTRACTOR REL VE ALL DIMENSIONS ROUGH OPENING

CULVER WILL ASSUME NO RESPONSIBILITY FOR ERRORS RESU TH FROM THE USE OF THESE DR GS BY THEIR TRADES.

) GENERAL ON OR SHALL BE RESPONSIBL OR THE, OR ION THE OCATION OF ASER PIPING ANY POSSIBLE INTERFERENCE TH WINDOW MCHORING YSTEM

5) TUMINOU ABR /ZINC CHROM TE OR AN DUAL SEPAR BON TERIAL IS REQUIRED WHEN ALUMINUM OMES IN CO WITH STEEL OR ANY OTHER DISSIMILAR METALS.

6) — WE ME AL BU JOIN OVERLA AND TERSE ION RE EE ED PPROPRI TE CAULKING

7) ALL PERMANE E TCHERS TO AL STUD ARE ASE ON IMMUM OF GA

) ALL PERIMETER ASTENERS ICK OR CUU ARE BASED UPON CUED AN /OR ORDU FILLE SON UNITS.

) ALL AILS OWN WITH WORK FROM OTH ER TR ES OR REFERENCE ONL

TEMPERED GLASS REQUIREMENTS

DOR EC
EHAP
RE TION 406
AR NDRAPI

CLAZING AN INDIVIDUAL ED OR OPERABL ANCL ADJACENT DOOR WHERE THE REA EST XPOSED EDGE OF GLAZIN WITHIN 4—INCH ARC OF THER VERTIC EDGE OF THE DOOR CLOSED POSITION AD WHEP THE BOTTOM XPOSED EDGE OF THE CL ZING LESS THAIR WICHE ABOVE TH ALKIN SURF CE.

EXCEPTIONS:
REL WHERE THERE AN TERVENING WAL OR THER PERMANEN ARRIER BETWEEN THE DOR AD GLAZIN WHERE THRO GH THE DOOR CLOSED OP TORAGE AREA FEE OR ESS DEPTH GLAZING APPLICATION AL COMPL TH OR TEM GLAZIN IN ALLS PERPENDICULAR THE LANE OF DOOR IN CL SED POSITION, THER THAN THE LL TOWARDS WHICH TH DOOR SWINGS WHEN OP ICD, ONE AND TWO-FAMIL DWELLINGS OR WITHIN MELLING UNITS GROU

CLAZING AN INDIVIDUAL XED OR OPERABLE ANCL TI IAN THOSE DC TIONS ESCRIBED RECEDING ITEM WHICH EET ALL OF THE OLL WING CONDITION

EXPOSED AREA OF AN INDIVIDUAL ANC GREATER TH OUARE FEET
EXPOSED ITTOM EDGE LESS THAN INCHES ABOV TH FLOOR,
EXPOSED TOP EDG REATER THAN 36 WCHE ABOVE THE FLOOR AND
ON OR ONE WALK SURFACE(S) THI 36 INCHES ORIZON ALL OF THE PLAN OF GLAZING

EXCEPTIO: APE CLA OR ND CLI-ICD OR BHE OLLOWIN IMS ALLA ONS.
PROTECTIVE BAR IMD OR MO IDENT AP ELE OF THIS AMBING ORIZON AL LOAD OF 50 LB/FT THOU ON CTAND THE GLAS IMS ALLED OF THE ACCESSIBLE SIRES OF THE GLAZING 34 INCHES 38 CHES ABOVE THE OUTBOARD AME MOULATING GLASS UNITS OR MUL GL ING WHERE THE BOTTOM EXPOSED EDGE OF THE GLASS IS 25 FTE OR ORE BOVE ANY GRADE OOR WAL ING SURFACE OR NEP OR ON AL OR SLOPED (WITHIN 45 DEGREE OF HOR ON AL SURFACE ADJACE THE GLAS EXTEND

ABBREVIATIONS

BP	ARCHITE TU AL REFERENCE
BALO	DIMENSION ON
	LIGH OPENIN
	DOOR OPENING
	ONC AL FINIU
	INTERIOR OPENING
	EMPERED GLASS
	SPUNWIRED GLAS
	WIRE GLASS
	MIRROR GL SS
	BOTTOM OF LIGH
	TOP OF MULLON
	TOP OF STEEL
	CENTE LINE
	THEW
	WINDOW CLIP WELDED TO STEEL
	DEAD DAD CLIP WELDED TO TEEL
	—CL CC TCHING TO TEEL
	—CLIP CONNECTING TO CONCRETE
	CLIP CONNECTING TO MC AL FRAMING
	CL CCH ECTING TO WOOD FRAMING
	—CLIP CONNECTIN TEEL
	CL CON CTIN MONRYE
	CL CONN_CTING MC AL MING
	CL CO CTIN WOOD FRAMING
	PERIMETER GLASS OCK —CLIP
	ADDE STEEL
QH	CEILWER
	OPPOSITE
SPL	SPLICE ON
PWL	ANCL

EGEND

LEE ION NUMBER
SHEET NUMBER
KEY PL ELEV TION BUBBLE

DET NUMBER
SHEET NUMBER
DETAIL UBBLE

GLASS SCHEDULE

TYP	
	CARIMA GLASS
	TEMPERED WHEN REQ

FIELD VERIFY DIMENSIONS!

REVISIC	UMBER	DATE	ACTION

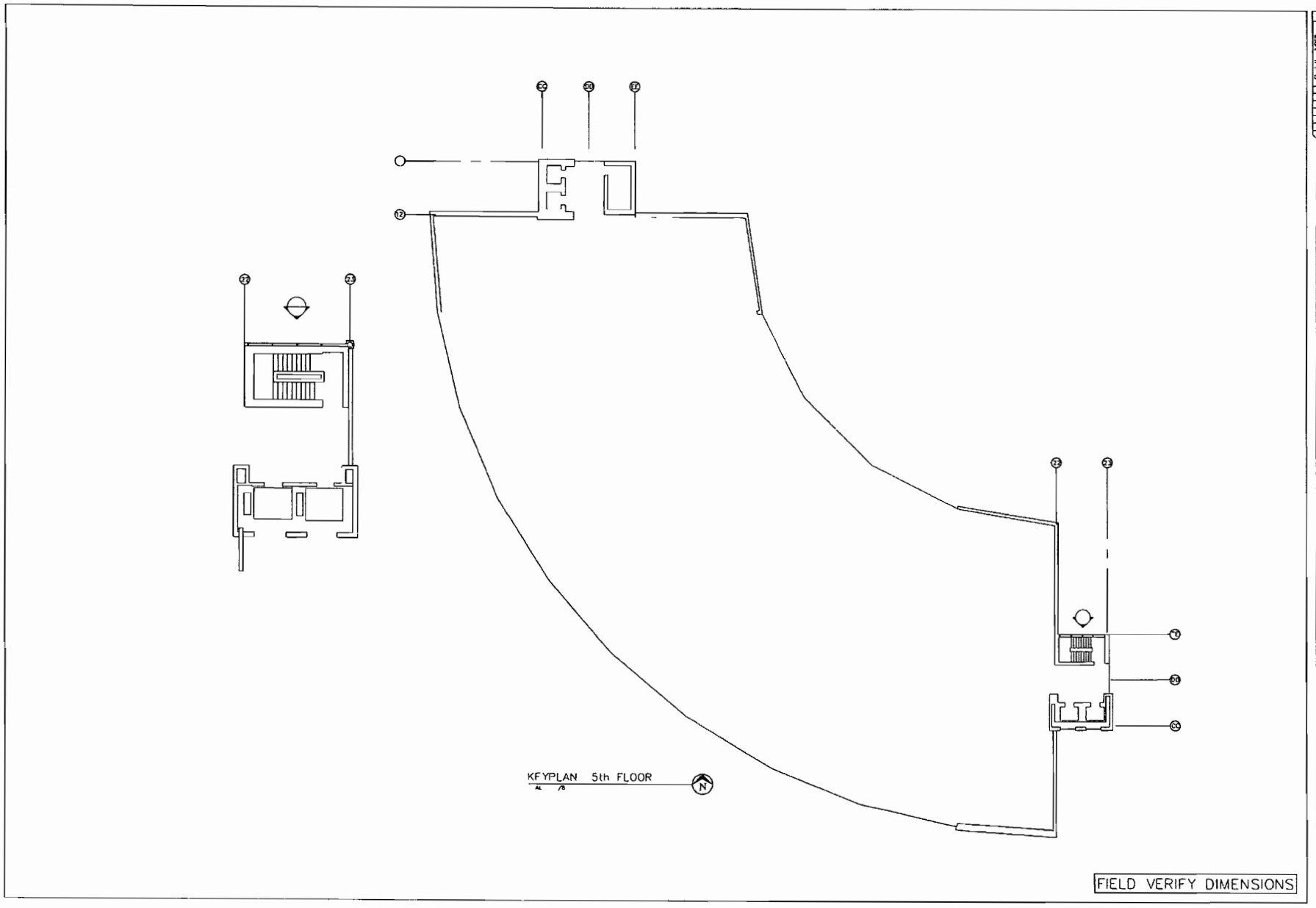
TE LOC

CULVER GLASS
45 9 N W 1 ON TRI L T BLDG 8 TE 1
PORT D, ONLCOM 97...
PHONE 05-256-25.0
X 45-5-97

PROVIDENCE HOSPI I AL PARKING STRUCTURE 2
4 NE HALSEY ST
PORTL ND CLCM 9721

11

3



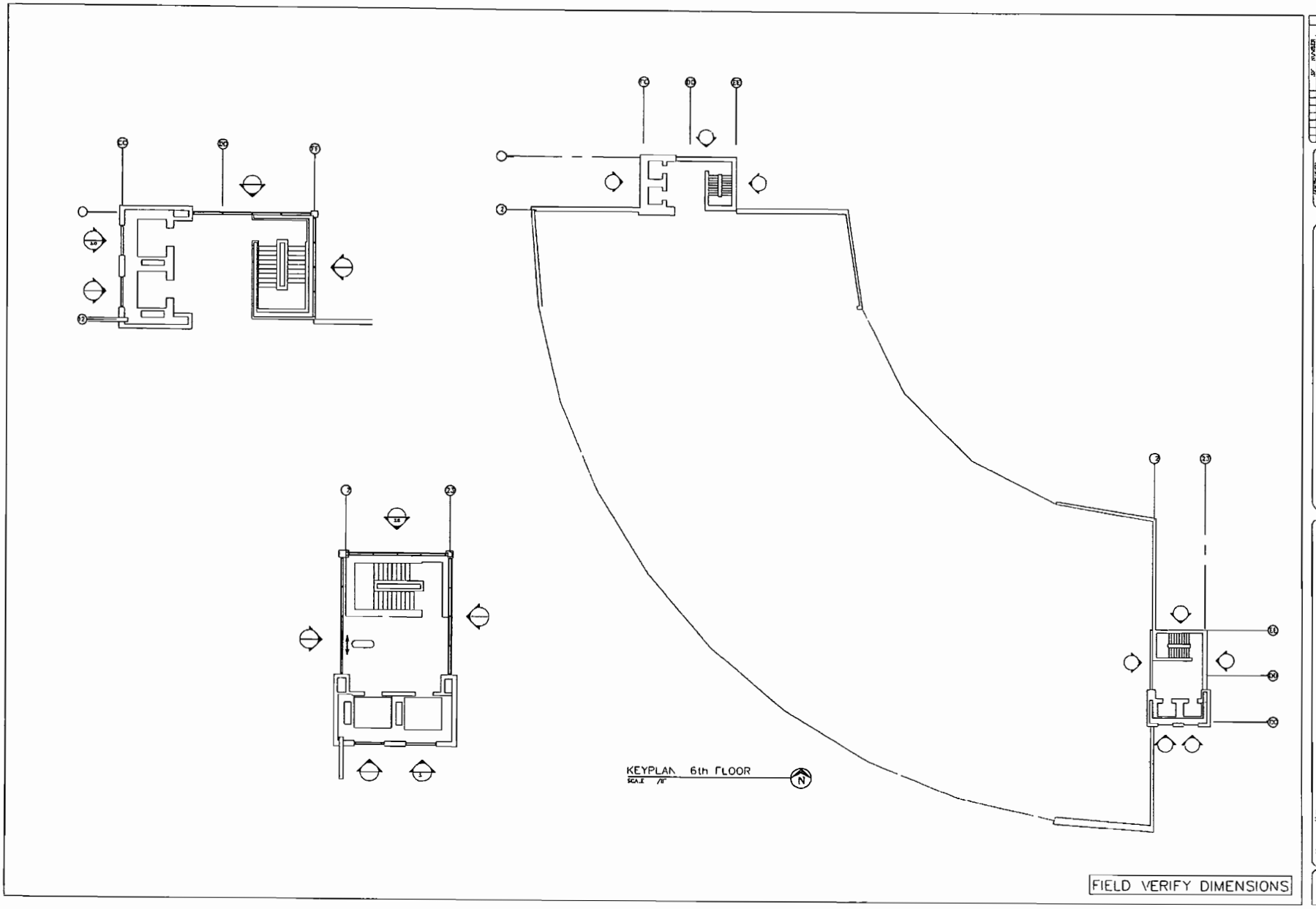
DATE
ACTION

CULVER GLASS

PROVIDENCE HOSPITAL PARKING STRUCTURE 2

20

4



DATE	ACTION

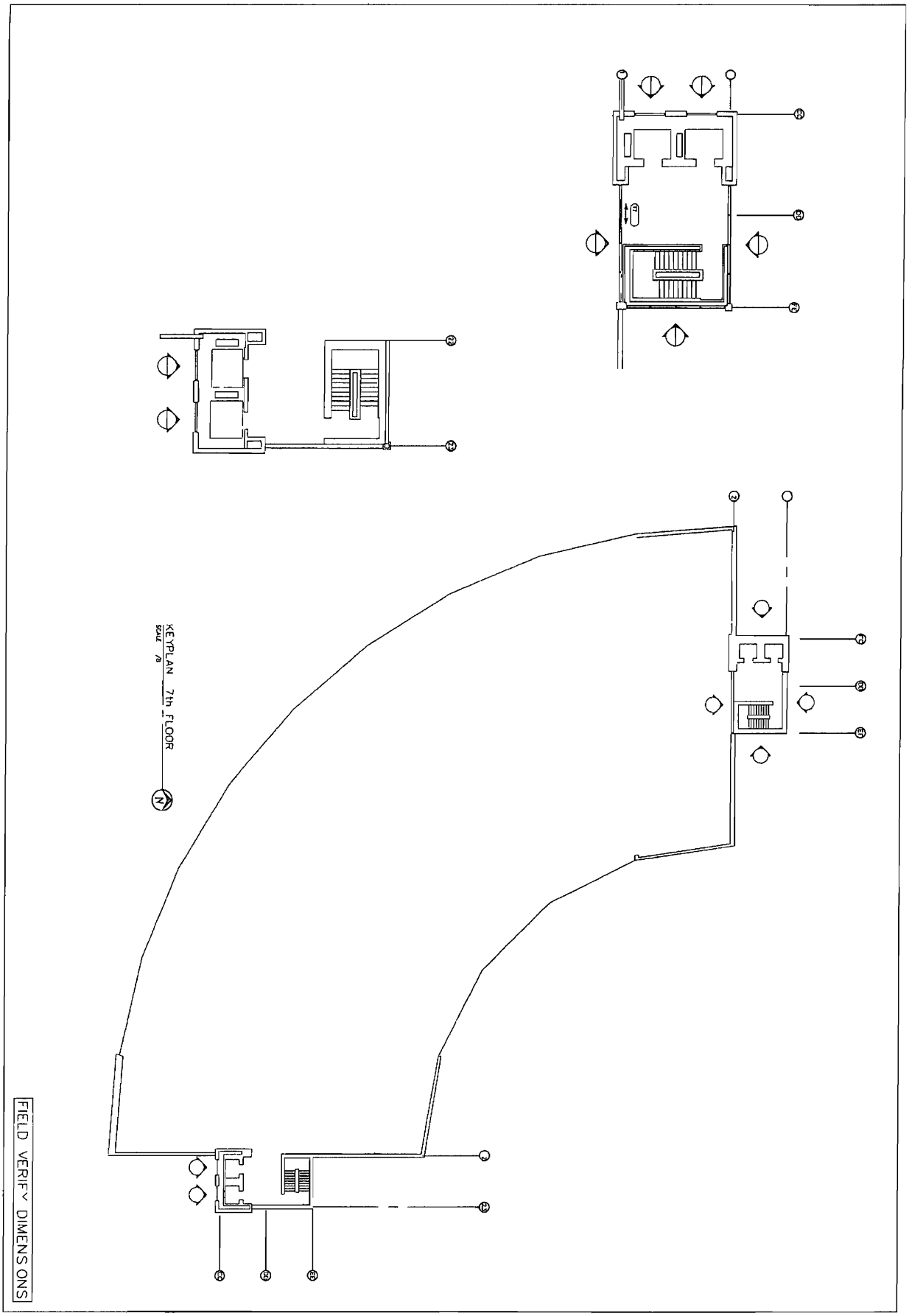
CULVER GLASS
INDUSTRIAL ST BLDG
PH NE 503-6 25
F X 13-26

> PROVIDENCE HOSPITAL PARKING STRUCTURE 2
NE H LSEY ST
PORTL ND REGON 3721

21

9

9



FIELD VERIFY DIMENSIONS

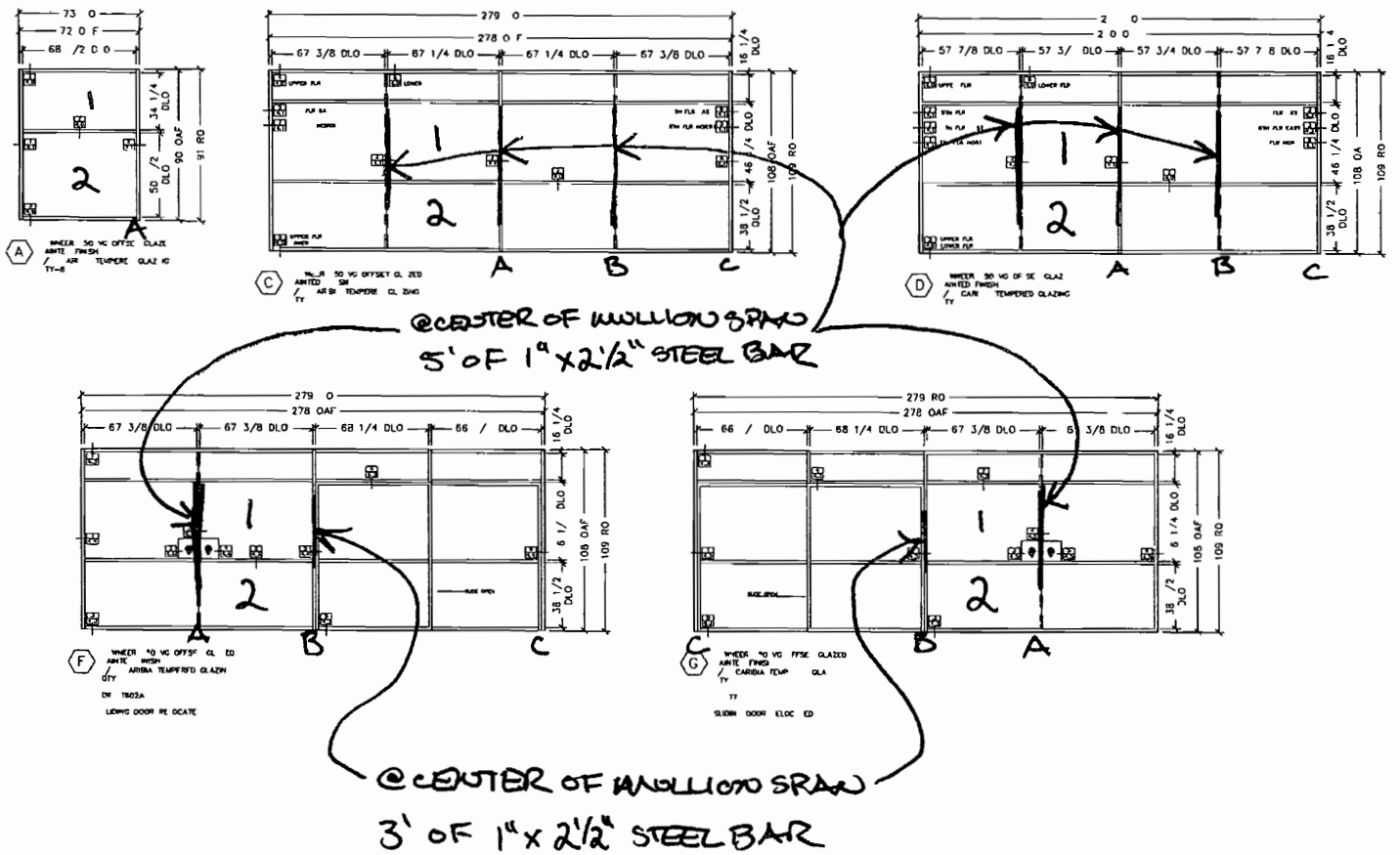
22

PROVIDENCE HOSPITAL PARKING STRUCTURE 2
NE H. LSEY ST
PORTL ND OI EGON 97

CULVER GLASS
DUSTRI L BLDG B TE
PO TL D OREGON 9
PHONE
FAX

DATE
ACTION

22



FIELD VERIFY DIMENSIONS

DATE

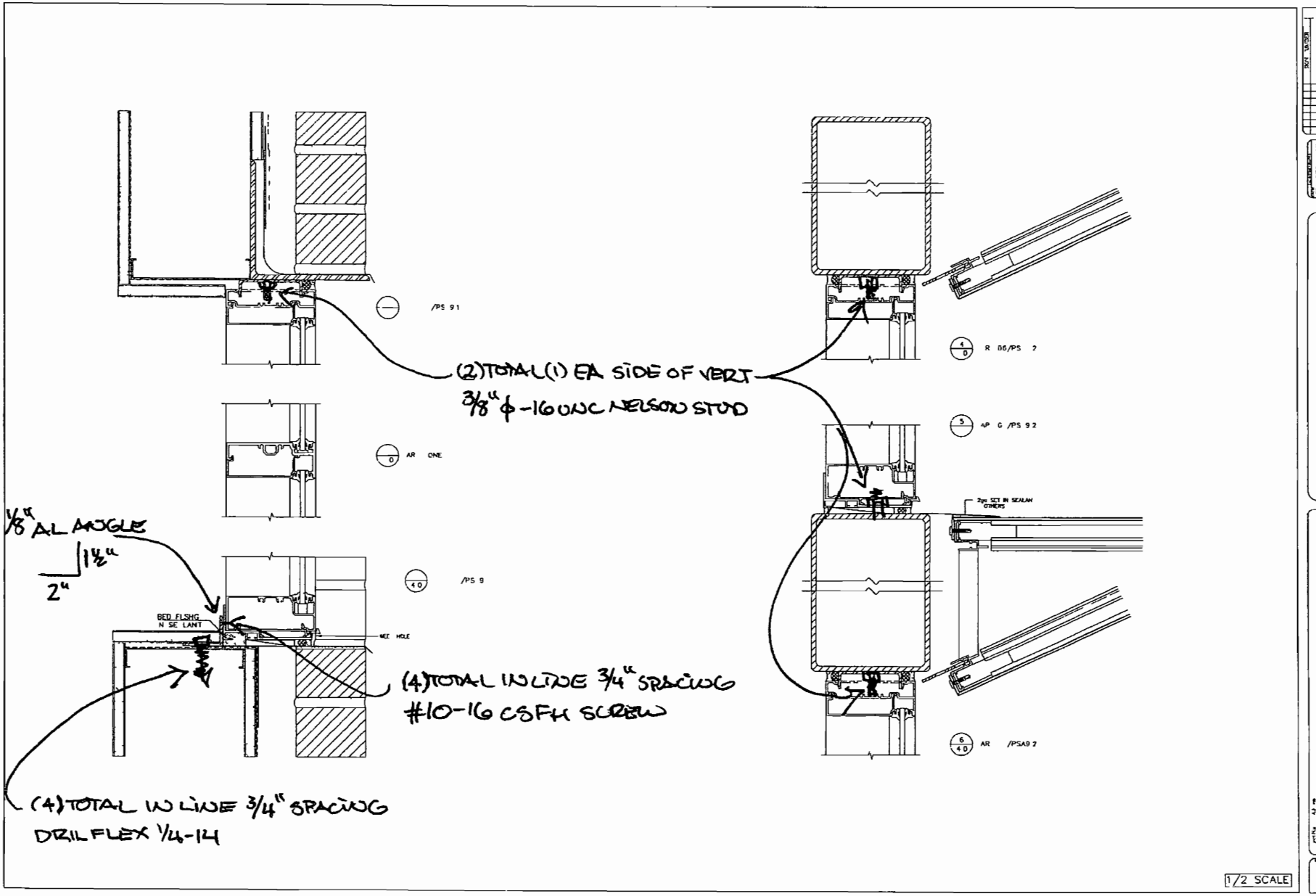
ACTION

CULVER GLASS

> PROVIDENCE HOSPITAL PARKING STRUCTURE 2
NE H LSEY ST
PORTLA OREGON

30

7



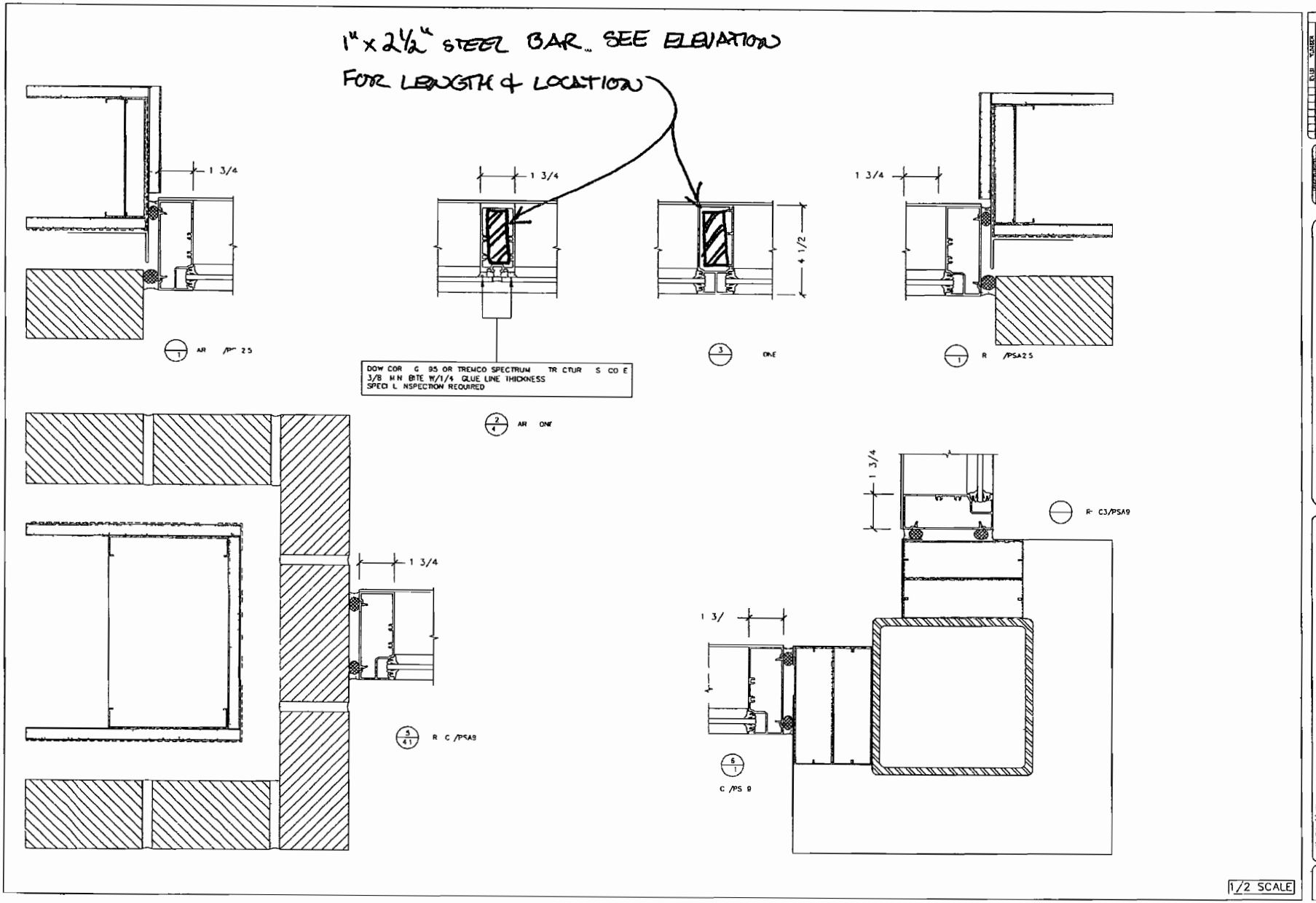
DATE | ACTION

CULVER GLASS

PROVIDENCE HOSPITAL PARKING STRUCTURE 2

40

8

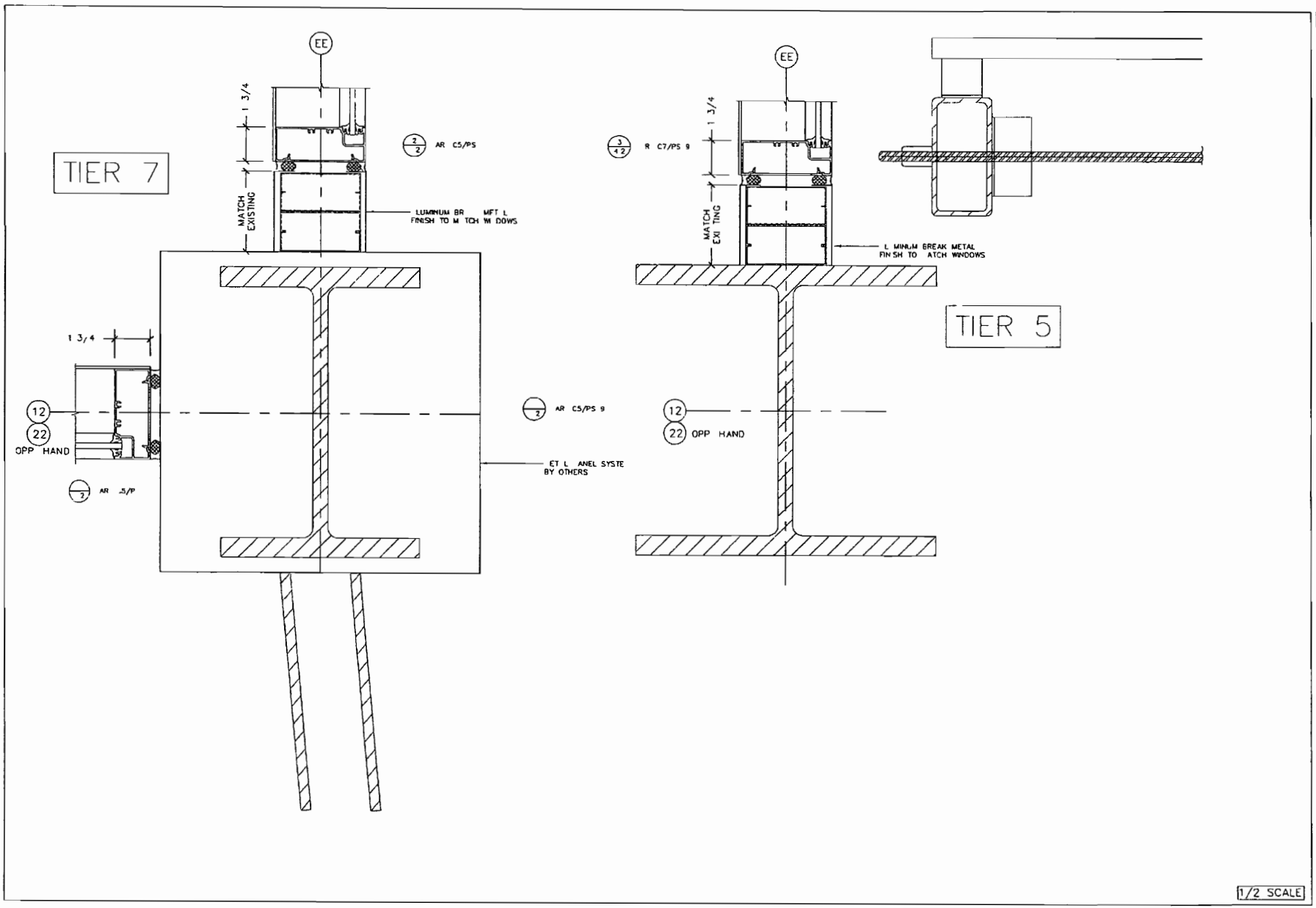


DATE | ACTION

CULVER GLASS

PROVIDI NCE HOSPITAL P ARKING STRUCTURE 2

41

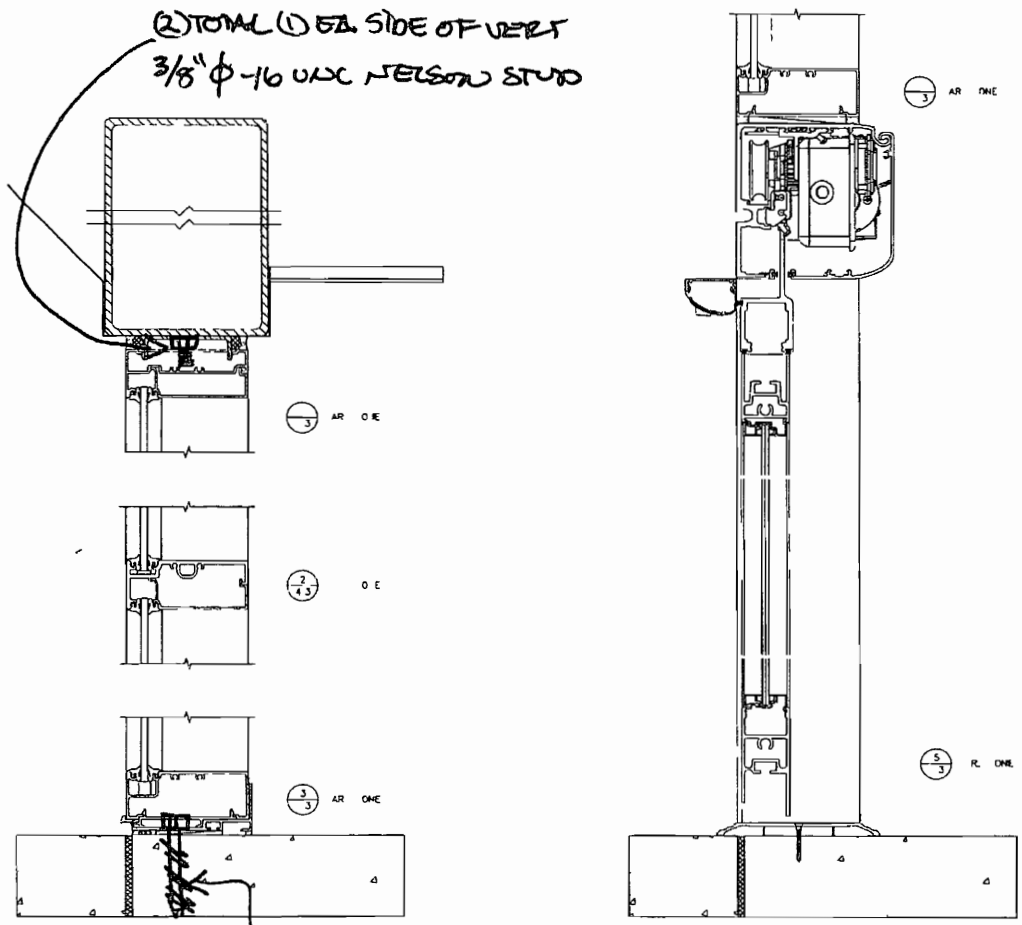


DATE	ACTION

CULVER GLASS

PROVIDENCE HOSPITAL PARKING STRUCTURE 2

42



SPECIAL INSPECTION REQUIRED

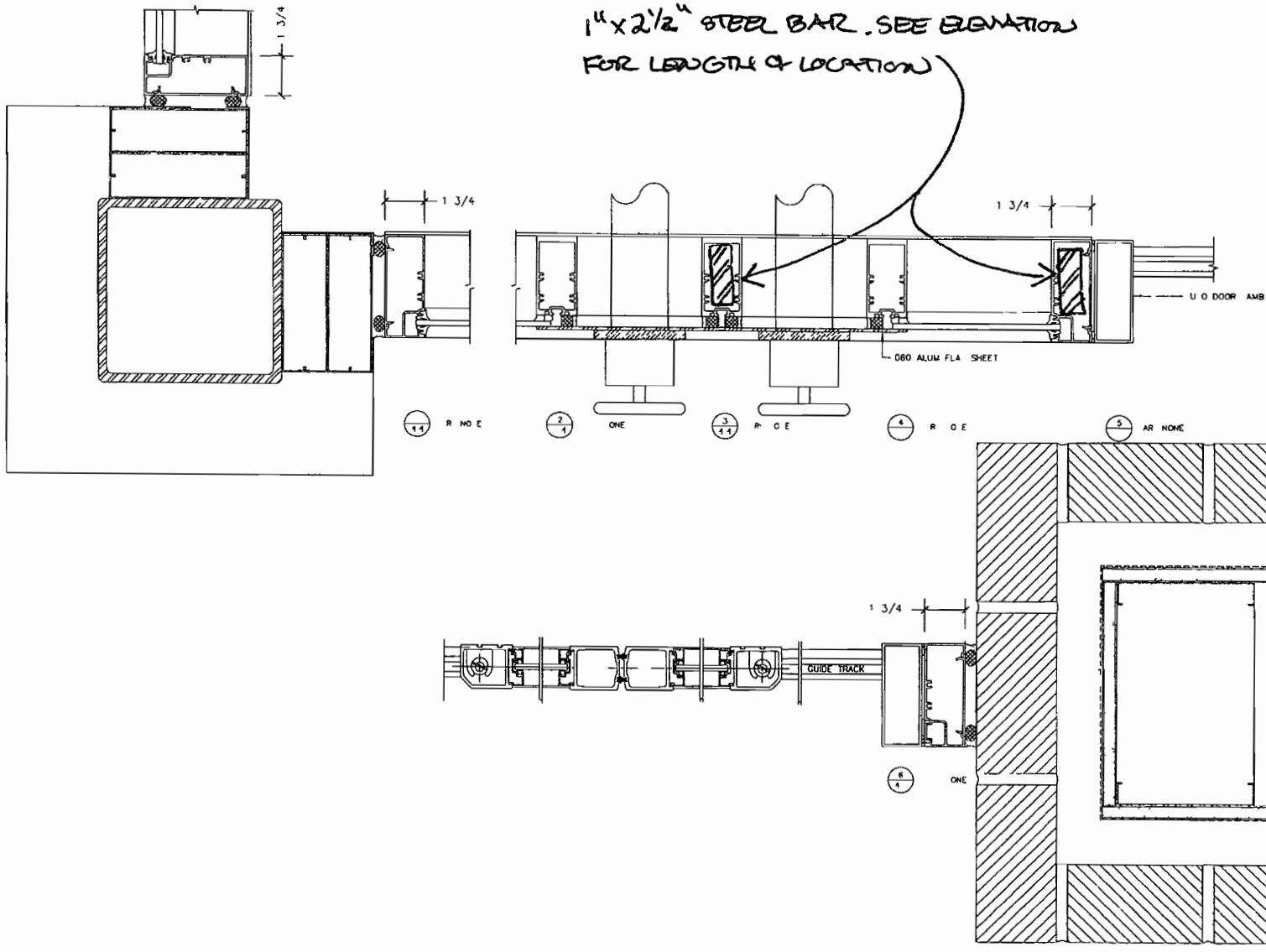
1/2 SCALE

DATE	ACTION

CULVER GLASS

> PROVIDENCE HOSPITAL PARKING STRUCTURE 2

43



DATE | ACTION

CULVER GLASS

PROVIDENCE HOSPITAL PARKING STRUCTURE 2

44

DATE
ACTION

CULVER GLASS

PROVIDENCE HOSPITAL PARKING STRUCTURE 2

45

HORIZONTAL WIND LOAD DEFLECTION OF FRAMING MEMBERS

The deflection of curtainwall and storefront framing members, Δ, is typically limited to the overall span $L_{[overall]}$ (inches) divided by 175 When it becomes difficult to meet this criteria through conventional means, or if it becomes costly to meet this criteria because of the need for additional reinforcing, then the deflection of the overall span may exceed $L_{[overall]}/175$

Section 2403 3 of the International Building Code states, *"To be considered firmly supported, the framing members for each individual pane of glass shall be designed so the deflection of the edge of the glass perpendicular to the glass pane shall not exceed $L_{[edge]}/175$ of the glass edge length or ¾" (19 1 mm), whichever is less, when subjected to the larger of the positive or negative load where loads are combined as specified in Section 1605 "*

The purpose of this deflection limitation is to keep the curvature of the edges of glass panels within limits that coincide with predictable glass behavior It is very common for a window frame to be made up of several lites of glass over the height of the window mullions, separated by horizontal muntins framed into the mullions In this scenario, the overall deflection of the mullions could be greater than $L_{[overall]}/175$, but still not exceed $L_{[edge]}/175$, or ¾", for the edges of the lites Obviously, in the case where there is a single lite over the mullion span, the overall mullion span would need to meet these criteria

It is a common misconception that the overall span of a window mullion cannot exceed this $L_{[overall]}/175$ or ¾" criteria This is generally the criteria specified by many manufacturers and designers, which is partly why this misconception is so common ASTM E 1300 is a standard used to determine maximum uniform pressures and deflections on glass panes with various support conditions (this standard is referenced in OSSC 2404 1) The use of this standard is subjected to a list of assumed conditions, one of these being 5 2 4, which states *"The glass edge support system is sufficiently stiff to limit lateral deflection of the supported glass edges to no more than L/175 of their lengths "* The mullion's overall deflection does not affect the performance of glass panels evaluated under E 1300, as long as 2403 3 is satisfied

In cases where the overall mullion deflection Δ does exceed $L_{[overall]}/175$, a more accurate loading and deflection diagram will be generated for the mullion, and from this, the relative deflection over the length of the most critical glass panel(s) will be determined In many cases, $L_{[edge]}$ for the most critical pane is much less than $L_{[overall]}$, and in these instances, the glass edge deflection will be noted as acceptable by inspection Other conditions will also be evaluated as necessary, such as overall stability in the window frames due to deflection and proximity of window frames to adjacent building components

Wind Pressures for Components and Cladding per 2009 IBC 1609 6
Wind Load Determination - Alternate All-Heights Method, Modifies ASCE 7-05 Method 2

Project Providence Office Park Parking Garage Addition

Location Portland, OR

Client Culver Glass

Input

Enter Basic Wind Speed mph	94 5
Enter Exposure	B
Enter Height Above Ground, feet	80.0
Enter Importance Factor, I_w	1.00
Enter K_{zt} per ASCE Fig 6-4	1
Enter Tributary Area to Element, ft^2	

Output

Pressure $q_z = 0.00256V^2$ psf	22 9	(IR 16-7 Eq 1)
K_z	0 927	(ASCE 7-05 Table 6-3)
Pressure Coefficients, C_{net}		
	Zone 4	Zone 5
Wall, Positive	0 92	0 92
Wall, Negative	-0 92	-1 68
Parapet, Positive	2 87	3 64
Parapet, Negative	-1 68	-2 45
Net Pressures, $P_{net} = q_z K_z C_{net} I_w K_{zt}$, lbs/ft^2		
	Zone 4	**Zone 5**
Wall, Positive	19 5	19 5
Wall, Negative	**-19 5**	**-35 6**
Parapet, Positive	60 6	77 2
Parapet, Negative	-35 6	-51 9

Alternate all heights meted can be used on regularly shaped buildings which meet all of the following conditions

1 The building is less than 75 feet in height with a height to least width ratio of 4 or less OR a building structure that has a fundamental frequency greater or equal to 1 hertz (period of 1 second)

2 The buidling is not sensitive to dynamic affects

3 The building is not located on a site for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration

VERTICAL MULLION REINFORCING VERIFICATION

	Mullion	Ix (in^4)	Sx (in^3)
A	450 VG 019	2 985	1 244
B	450 SSG 005	2 445	1 352
C	450 VG 012	3 074	1 192
D			
E			
F			
G			

94 5 mph Exposure B I = 1 00

Zone 4 Wind Pressure 19 5 psf

Zone 5 Wind Pressure 35 6 psf

$$I_{rq\ d} = \frac{21\,wl^{3}}{640000}$$

James G Pierson, Inc — Since 1946 — STRUCTURAL ENGINEERS

	Windows	Span Length (in)	Trib Width (in)	Wind Press (psf)	Wind Load (#/in)	Mullion Type	I Rq'd (in4)	f_b (psi)	Reinf Required	Wind Rxn Design (lbs)	Allow Δ (in)	Comments on Reinforcement
Sheet 3 0	A A	91	36 5	35 6	9 02	A	1 549	7 508	No	411	0 52	
	C-A	109	69	34 6	16 58	C	4 892	20 656	YES!!	930	0 62	5 ft of 1 x 2 1/2 Steel Bar
	C B	109	69 0625	34 6	16 59	B	4 897	18 228	YES!!	931	0 62	5 ft of 1 x 2 1/2 Steel Bar
	C C	109	35 4375	35 6	8 76	A	2 585	10 459	No	477	0 62	
	D A	109	59 5	34 9	14 42	C	4 255	17 967	YES!!	802	0 62	5 ft of 1 x 2 1/2 Steel Bar
	D B	109	59 5625	34 9	14 44	B	4 260	15 857	YES!!	803	0 62	5 ft of 1 x 2 1/2 Steel Bar
	D C	109	30 6875	35 6	7 59	A	2 239	9 057	No	413	0 62	
	F A	109	69 125	19 3	9 26	B	2 734	10 177	YES!!	931	0 62	5 ft of 1 x 2 1/2 Steel Bar
	F-B	109	103 5625	19 1	13 74	A	4 053	16 399	YES!!	1395	0 62	3 ft of 1 x 2 1/2 Steel Bar
	F C	109	69 875	19 5	9 46	A	2 792	11 296	No	941	0 62	
	G-A	109	69 125	19 3	9 26	B	2 734	10 177	YES!!	931	0 62	5 ft of 1 x 2 1/2 Steel Bar
	G B	109	103 5625	19 1	13 74	A	4 053	16 399	YES!!	1395	0 62	3 ft of 1 x 2 1/2 Steel Bar
	G C	109	69 875	19 5	9 46	A	2 792	11 296	No	941	0 62	

POP II PG Addition

Cbeam 2005
James G Pierson Inc
3/16/2011 14 50 File C-A D-A

By Aaron Rudis

Beam Results						
Max Span Deflection	=	0 5109	(Span	2	@	30 00)
Max Positive Moment(1)		17162 #	(Span	1	@	24 50)
Max Positive Moment(2)	=	24623 #	(Span	2	@	30 00)

Member Information

Span	L(in)	I(in 4)	S(in 3)	E(psi)	Reinf I	Reinf E
1	24 500	3 074	1 192	1 0e+007		
2	60 000	3 074	1 192	1 0e+007	1 302	2 9e+007
3	24 500	3 074	1 192	1 0e+007		

Distributed Load Information

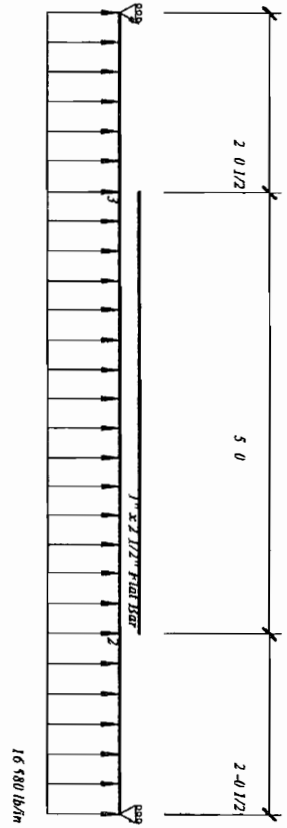
Span	W1(#/in)	W2(#/in)	X1(in)	X2(in)
1	16 580	16 580	0 000	24 500
2	16 580	16 580	0 000	60 000
3	16 580	16 580	0 000	24 500

Joints Free to Displace

Free Joints 2 3

Support Reactions

Joint	Pounds
1	904
4	904



Maximum distributed load value shown only see distributed load table for detailed information

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21

POP II PG Addition

Cbeam 2005
James G Pierson Inc
3/16/2011 14 50 File C-A D-A

Span No 1

	0 00L	0 10L	0 20L	0 30L	0 40L	0 50L	0 60L	0 70L	0 80L	0 90L	1 00L
Location	0 00	2 45	4 90	7 35	9 80	12 25	14 70	17 15	19 60	22 05	24 50
Shear	903 61	862 99	822 37	781 75	741 13	700 51	659 88	619 26	578 64	538 02	497 40
Moment	0 0	2164 1	4228 6	6193 7	8059 2	9825 2	11491 7	13058 6	14526 1	15894 0	17162 4
Defl	0 0000	0 0421	0 0837	0 1245	0 1641	0 2021	0 2383	0 2721	0 3035	0 3319	0 3573
Stress	0 0	1815 5	3547 5	5196 0	6761 1	8242 6	9640 7	10955 2	12186 3	13333 9	14398 0

Span No 2

	0 00L	0 10L	0 20L	0 30L	0 40L	0 50L	0 60L	0 70L	0 80L	0 90L	1 00L
Location	0 00	6 00	12 00	18 00	24 00	30 00	36 00	42 00	48 00	54 00	60 00
Shear	497 40	397 92	298 44	198 96	99 48	0 00	99 48	198 96	298 44	397 92	497 40
Moment	17162 4	19848 3	21937 4	23429 6	24324 9	24623 4	24324 9	23429 6	21937 4	19848 3	17162 4
Defl	0 3573	0 4107	0 4538	0 4853	0 5045	0 5109	0 5045	0 4853	0 4538	0 4107	0 3573
Stress	14398 0	16651 3	18403 9	19655 7	20406 8	20657 2	20406 8	19655 7	18403 9	16651 3	14398 0

Span No 3

	0 00L	0 10L	0 20L	0 30L	0 40L	0 50L	0 60L	0 70L	0 80L	0 90L	1 00L
Location	0 00	2 45	4 90	7 35	9 80	12 25	14 70	17 15	19 60	22 05	24 50
Shear	497 40	538 02	578 64	619 26	659 88	700 50	741 13	781 75	822 37	862 99	903 61
Moment	17162 4	15894 0	14526 1	13058 6	11491 7	9825 2	8059 2	6193 7	4228 6	2164 1	0 0
Defl	0 3573	0 3319	0 3035	0 2721	0 2383	0 2021	0 1641	0 1245	0 0837	0 0421	0 0000
Stress	14398 0	13333 9	12186 3	10955 2	9640 7	8242 6	6761 1	5196 0	3547 5	1815 5	0 0

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POP II PG Addition

Cbeam 2005
James G Pierson Inc
3/16/2011 14 49 File C-B D-B F-A G-A

By Aaron Rudis

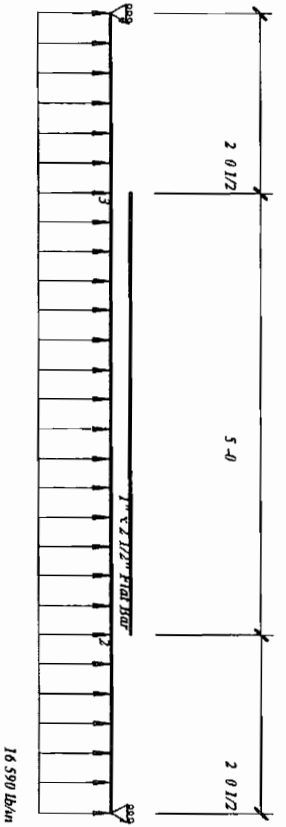
Beam Results							
Max	Span Deflection	-	0 5816	(Span	2	@	30 00)
Max	Positive Moment(1)	-	17173 #	(Span	1	@	24 50)
Max	Positive Moment(2)		24638 #	(Span	2	@	30 00)

Member Information						
Span	L(in)	I(in 4)	S(in 3)	E(psi)	Reinf I	Reinf E
1	24 500	2 445	1 352	1 0e+007		
2	60 000	2 445	1 352	1 0e+007	1 302	2 9e+007
3	24 500	2 445	1 352	1 0e+007		

Distributed Load Information				
Span	W1(#/in)	W2(#/in)	X1(in)	X2(in)
1	16 590	16 590	0 000	24 500
2	16 590	16 590	0 000	60 000
3	16 590	16 590	0 000	24 500

Joints Free to Displace
Free Joints 2 3

Support Reactions	
Joint	Pounds
1	904
4	904



Maximum distributed load value shown only see distributed load table for detailed information

POP II PG Addition

Cbeam 2005
James G Pierson Inc
3/16/2011 14 49 File C-B D-B F-A G-A

Span No 1

	0 00L	0 10L	0 20L	0 30L	0 40L	0 50L	0 60L	0 70L	0 80L	0 90L	1 00L
Location	0 00	2 45	4 90	7 35	9 80	12 25	14 70	17 15	19 60	22 05	24 50
Shear	904 16	863 51	822 86	782 22	741 57	700 93	660 28	619 64	578 99	538 35	497 70
Moment	0 0	2165 4	4231 2	6197 4	8064 1	9831 1	11498 6	13066 5	14534 8	15903 6	17172 7
Defl	0 0000	0 0492	0 0979	0 1455	0 1916	0 2357	0 2775	0 3164	0 3521	0 3842	0 4124
Stress	0 0	1601 6	3129 6	4583 9	5964 5	7271 5	8504 9	9664 6	10750 6	11763 0	12701 7

Span No 2

	0 00L	0 10L	0 20L	0 30L	0 40L	0 50L	0 60L	0 70L	0 80L	0 90L	1 00L
Location	0 00	6 00	12 00	18 00	24 00	30 00	36 00	42 00	48 00	54 00	60 00
Shear	497 70	398 16	298 62	199 08	99 54	0 00	99 54	199 08	298 62	398 16	497 70
Moment	17172 7	19860 3	21950 6	23443 7	24339 6	24638 2	24339 6	23443 7	21950 6	19860 3	17172 7
Defl	0 4124	0 4713	0 5187	0 5534	0 5745	0 5816	0 5745	0 5534	0 5187	0 4713	0 4124
Stress	12701 7	14689 6	16235 7	17340 0	18002 7	18223 5	18002 7	17340 0	16235 7	14689 6	12701 7

Span No 3

	0 00L	0 10L	0 20L	0 30L	0 40L	0 50L	0 60L	0 70L	0 80L	0 90L	1 00L
Location	0 00	2 45	4 90	7 35	9 80	12 25	14 70	17 15	19 60	22 05	24 50
Shear	497 70	538 35	578 99	619 64	660 28	700 93	741 57	782 22	822 86	863 51	904 16
Moment	17172 7	15903 6	14534 8	13066 5	11498 6	9831 1	8064 1	6197 4	4231 2	2165 4	0 0
Defl	0 4124	0 3842	0 3521	0 3164	0 2775	0 2357	0 1916	0 1455	0 0979	0 0492	0 0000
Stress	12701 7	11763 0	10750 6	9664 6	8504 9	7271 5	5964 5	4583 9	3129 6	1601 6	0 0

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Email software@mcalsoft com

POP II PG Addition

Cbeam 2005

James G Pierson Inc

3/16/2011 14 49 File F-B G-B

By Aaron Rudis

Beam Results						
Max Span Deflection	=	0 5436	(Span	2	@	18 00)
Max Positive Moment(1)	=	18180 #	(Span	1	@	36 50)
Max Positive Moment(2)	=	20406 #	(Span	2	@	18 00)

Member Information

Span	L(in)	I(in 4)	S(in 3)	E(psi)	Reinf I	Reinf E
1	36 500	2 985	1 244	1 0e+007		
2	36 000	2 985	1 244	1 0e+007	1 302	2 9e+007
3	36 500	2 985	1 244	1 0e+007		

Distributed Load Information

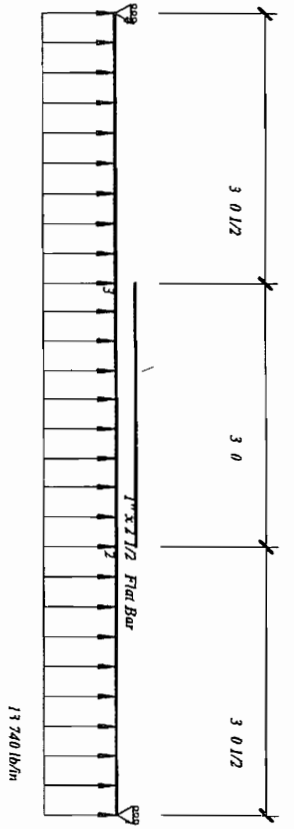
Span	W1(#/in)	W2(#/in)	X1(in)	X2(in)
1	13 740	13 740	0 000	36 500
2	13 740	13 740	0 000	36 000
3	13 740	13 740	0 000	36 500

Joints Free to Displace

Free Joints 2 3

Support Reactions

Joint	Pounds
1	749
4	749



Maximum distributed load value shown only see distributed load table for detailed information

POP II PG Addition

Cbeam 2005

James G Pierson Inc

3/16/2011 14 49 File F-B G-B

Span No 1

	0 00L	0 10L	0 20L	0 30L	0 40L	0 50L	0 60L	0 70L	0 80L	0 90L	1 00L
Location	0 00	3 65	7 30	10 95	14 60	18 25	21 90	25 55	29 20	32 85	36 50
Shear	748 83	698 68	648 53	598 38	548 23	498 07	447 92	397 77	347 62	297 47	247 32
Moment	0 0	2641 7	5100 4	7376 0	9468 5	11378 0	13104 5	14647 9	16008 2	17185 5	18179 7
Defl	0 0000	0 0663	0 1314	0 1942	0 2538	0 3091	0 3594	0 4038	0 4417	0 4725	0 4956
Stress	0 0	2123 6	4100 0	5929 2	7611 3	9146 3	10534 1	11774 8	12868 3	13814 7	14613 9

Span No 2

	0 00L	0 10L	0 20L	0 30L	0 40L	0 50L	0 60L	0 70L	0 80L	0 90L	1 00L
Location	0 00	3 60	7 20	10 80	14 40	18 00	21 60	25 20	28 80	32 40	36 00
Shear	247 32	197 86	148 39	98 93	49 46	0 00	49 46	98 93	148 39	197 86	247 32
Moment	18179 7	18981 1	19604 3	20049 5	20316 6	20405 6	20316 6	20049 5	19604 3	18981 1	18179 7
Defl	0 4956	0 5127	0 5261	0 5358	0 5416	0 5436	0 5416	0 5358	0 5261	0 5127	0 4956
Stress	14613 9	15258 1	15759 1	16116 9	16331 7	16403 2	16331 7	16116 9	15759 1	15258 1	14613 9

Span No 3

	0 00L	0 10L	0 20L	0 30L	0 40L	0 50L	0 60L	0 70L	0 80L	0 90L	1 00L
Location	0 00	3 65	7 30	10 95	14 60	18 25	21 90	25 55	29 20	32 85	36 50
Shear	247 32	297 47	347 62	397 77	447 92	498 07	548 23	598 38	648 53	698 68	748 83
Moment	18179 7	17185 5	16008 2	14647 9	13104 5	11378 0	9468 5	7376 0	5100 4	2641 7	0 0
Defl	0 4956	0 4725	0 4417	0 4038	0 3594	0 3091	0 2538	0 1942	0 1314	0 0663	0 0000
Stress	14613 9	13814 7	12868 3	11774 8	10534 1	9146 3	7611 3	5929 2	4100 0	2123 6	0 0

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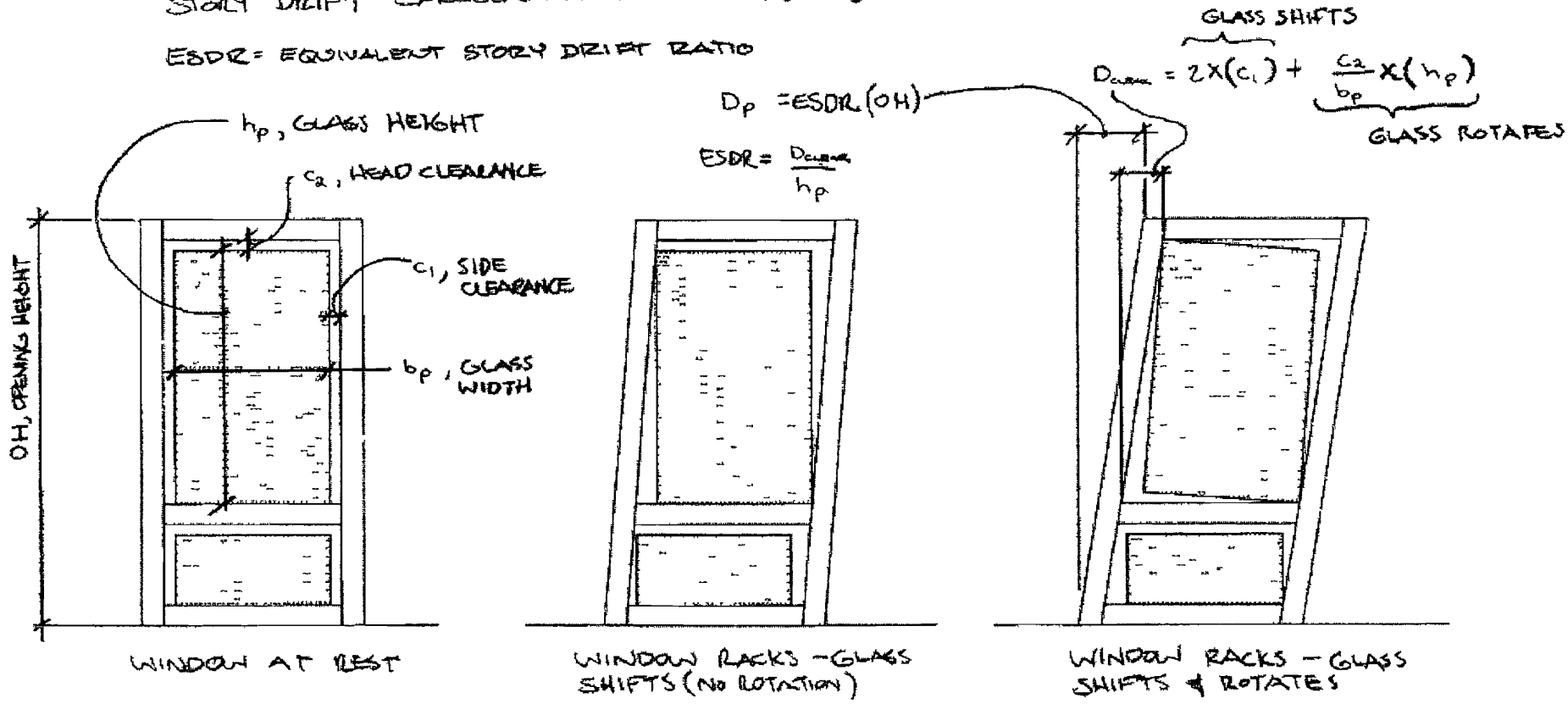
Story Drift Calculations For Windows per ASCE 7-05 13 9 5

D_p based on (information provided or calculated)

	Window	Expected Drift (in)	Story Height (in)	Side Clearance C_1 (in)	Head Clearance C_2 (in)	Glass Width b_p (in)	Glass Height h_p (in)	D_{clear} (in)	D_p (in)	$D_{clear} > D_p$?
Sheet 3 0	A-1	1 25	126	0 4375	0 4375	69 25	35	1 10	0 35	OK
	A-2	1 25	126	0 4375	0 4375	69 25	51 25	1 20	0 51	OK
	C-1	1 25	126	0 4375	0 4375	68 375	47	1 18	0 47	OK
	C-2	1 25	126	0 4375	0 4375	68 375	39 25	1 13	0 39	OK
	D-1	1 25	126	0 4375	0 4375	58 875	47	1 22	0 47	OK
	D-2	1 25	126	0 4375	0 4375	58 875	39 25	1 17	0 39	OK
	F-1	1 25	126	0 4375	0 4375	68 5	47	1 18	0 47	OK
	F-2	1 25	126	0 4375	0 4375	68 5	39 25	1 13	0 39	OK
	G-1	1 25	126	0 4375	0 4375	68 5	47	1 18	0 47	OK
	G-2	1 25	126	0 4375	0 4375	68 5	39 25	1 13	0 39	OK

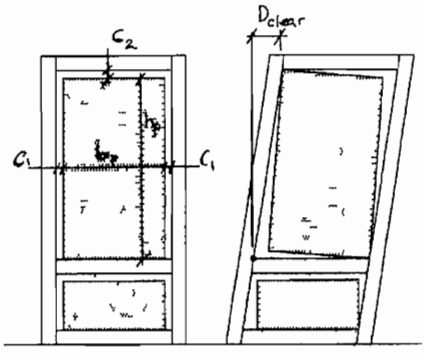
STORY DRIFT CALCULATION FOR WINDOWS

ESDR = EQUIVALENT STORY DRIFT RATIO

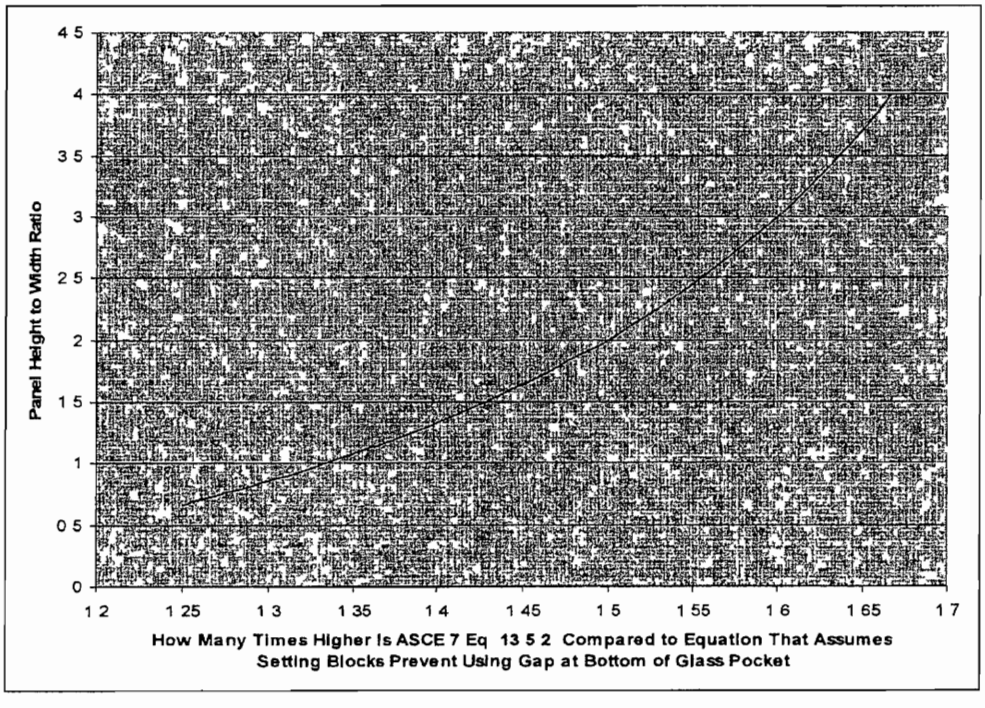


Drift Analysis – Modified to Account for Presence of Setting Blocks in Glazing Pocket

$$D_{clear} = 2c_1 + \frac{c_2}{b_p}\left(h_p\right)$$



This drift analysis differs slightly with ASCE 13 5 9 in the form of the equation for D_{clear} which is described in Sec 13 5 9 1 Exception 1 (see Eq 13 5-2) If the ASCE form of the equation is expanded into two terms the term which accounts for glass rotation gives twice the amount of the equation above The only way that this type of rotation could occur is if the glass was to pivot about a point at the center of the panel and the limit of rotation was defined by the clearance between the frame and glass at not only the top of the panel (denoted as c_2) but also at the bottom This however is not realistic because the clearance at the bottom is taken up by nylon setting blocks at either end of the panel which prevents this movement from happening Therefore the rotational component of the lateral panel movement should only be a function of the top clearance which is how the above equation is written Using the ASCE s form of the equation gives a larger drift value usually at least 25% greater Since these values commonly differ by at least this much it is not necessary to compare it to 1 25 times the inelastic drift per Equation 13 5 2 Below is a graph showing how many times larger the panel drift is that the ASCE 7 Equation 13 5-2 gives compared to the equation above This relationship varies with the panel aspect ratio



James G Pierson, Inc

Consulting Structural Engineers

320 S W Stark Suite 535 Portland, Oregon 97204

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Project	Job no
Location	Date
Client	Sheet no 21

HEAD & SILL CONN TO STEEL

DETAIL 1/4 ○, 4/4.0, 5/4 ○, 6/4 ○, 1/4.3 RXS = 1395 lb

USE 3/8"ϕ-16 UNC NELSON STUD

V_{ALLOW} = 2128 lb (2) TOTAL @ VERT V_{ACT} = 1395/2 = 698 lb
T_{ALLOW} = 2325 lb (1) EA. SIDE T_{ACT} = 1395/2(.5) = 299 lb
2/3(1.5)

IE $(698/2128)^2 + (299/2325)^2$ = 12 < 1.0 ok

SILL CONNECTION TO STEEL STUDS (ASSUME 18 GA)

DETAIL 3/4 ○ RXS = 411 lb

USE DRIL FLEX 1/4-14 + #10-16 OSFH SCREWS

CONN OF SILL TO ANGLE

T_{ALLOW} = 239 lb USE (4) SCREWS T_{ACT} = 411/4 = 103 lb ok
3/4" SPACING

CONN OF ANGLE TO SS

V_{ALLOW} = 925 lb (4) TOTAL IN LINE V_{ACT} = 411/4 = 103#
T_{ALLOW} = 274 lb 3/4" SPACING T_{ACT} = 411/4(.25) - 192#
2/3(1)

IE = $(103/925)^2 + (192/274)^2$ = 0.5 < 1.0 ok

SILL CONN. TO CONCRETE (3000 PSI)

DETAIL 3/4.3 RXS = 1398 lb

USE SIMPSON TITEN HD 1/2"ϕ 4" EMB (3) @ EA. VERT SPECIAL INSP. REQUIRED

V_{ACT} = 1398(1.6) = 2232 lb } SEE CALC OUTPUT
T_{ACT} = 1395(1.6)(.25)/(2/3(2)) = 419 lb }

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Project	Job no
Location	Date
Client	Sheet no 22

Anchor Calculations

Anchor Selector (Version 4 5 1 0)

Job Name Sill Connection Revised

Date/Time 5/5/2011 2 07 33 PM

Calculation Summary ACI 318 Appendix D For Cracked Concrete per ACI 318 08

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
1/2 Titen HD	N/A	1	4	1

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	3000 0	1 20

Condition	Thickness (in)	Suppl Edge Reinforcement
A tension and shear	12	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb ft)	M_{uy} (lb ft)
140	0	744	0	0

e_x(in)	e_y(in)	Mod/high seismic	Apply entire shear @ front row
0	0	No	No

Individual Anchor Tension Loads

N_{ua1} (lb)
140 00

e_{Nx}(in)	e_{Ny}(in)
0 00	0 00

Individual Anchor Shear Loads

V_{ua1} (lb)
744 00

e_{Vx}(in)	e_{Vy}(in)
0 00	0 00

Tension Strengths

Steel (Φ = 0 65)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
20130	13084 50	140 00	0 0107

Concrete Breakout (Φ = 0 75)

N_{cb}(lb)	ΦN_{cb}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{cb}$
2734 09	2050 57	140 00	0 0683

Pullout (Φ = 0 65)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{pn}$
0 00	0 00	140 00	Infinity

Side Face Blowout does not apply

Shear Strengths

Steel (Φ = 0 60)

V_{sa}(lb)	ΦV_{sa} (lb)	V_{ua}(lb)	$V_{ua}/\Phi V_{sa}$
7455	4473 00	744 00	0 1663

Concrete Breakout (case 1) (Φ = 0 75)

V_{cbx}(lb)	ΦV_{cbx}(lb)	V_{uax}(lb)	$V_{uax}/\Phi V_{cbx}$
4485 58	3364 18	0 00	0 0000

V_{cby}(lb)	ΦV_{cby}(lb)	V_{uay}(lb)	$V_{uay}/\Phi V_{cby}$	$V_{ua}/\Phi V_{cb}$
1077 02	807 76	744 00	0 9211	0 9211

Concrete Breakout (case 2) does not apply to single anchor layout

Concrete Breakout (case 3) (Φ = 0 75)

c_{x1} edge

V_{cby}(lb)	ΦV_{cby}(lb)	V_{uay}(lb)	$V_{uay}/\Phi V_{cby}$
12062 06	9046 55	744 00	0 0822

c_{y1} edge

V_{cbx}(lb)	ΦV_{cbx}(lb)	V_{uax}(lb)	$V_{uax}/\Phi V_{cbx}$
21053 78	15790 34	0 00	0 0000

c_{x2} edge

V_{cby}(lb)	ΦV_{cby}(lb)	V_{uay}(lb)	$V_{uay}/\Phi V_{cby}$
12062 06	9046 55	744 00	0 0822

c_{y2} edge

V_{cbx}(lb)	ΦV_{cbx}(lb)	V_{uax}(lb)	$V_{uax}/\Phi V_{cbx}$	$V_{ua}/\Phi V_{cb}$
2154 03	1615 52	0 00	0 0000	0 0822

Pryout (Φ = 0 70)

V_{cp}(lb)	ΦV_{cp}(lb)	V_{uax}(lb)	$V_{uax}/\Phi V_{cp}$
5468 18	3827 73	0	0 0000

V_{cp}(lb)	ΦV_{cp}(lb)	V_{uay}(lb)	$V_{uay}/\Phi V_{cp}$	$V_{ua}/\Phi V_{cp}$
5468 18	3827 73	744	0 1944	0 1944

Interaction check

T Max(0 07) <= 0 2 and V Max(0 92) <= 1 0 [Sec D 7 2]

Interaction check PASS

Use 1/2" diameter Titen HD anchor(s) with 4 in embedment

REVIEWED BY JE DUNN NORTHWEST INC
REVIEWLD BY Ann Young
DATE 3/30/11
SUBMITTAL # 055100.4

kpff Consulting Engineers

March 3 2011

SUBMITTAL REVIEW
☐ NO EXCEPTIONS TAKEN ☐ MAKE CORRECTIONS NOTED ☐ REVISE AND RESUBMIT ☐ SUBMIT SPECIFIED ITEM
RECEIVED FOR GENERAL CONFORMANCE WITH THE DESIGN CONCEPT OF THE PROJECT AND GENERAL COMPLIANCE WITH THE INFORMATION IN THE CONTRACT DOCUMENTS. ANY ACTION SHOWN IS SUBJECT TO THE REQUIREMENTS OF THE CONTRACT DOCUMENTS. CONTRACTOR IS RESPONSIBLE FOR DIMENSIONS WHICH SHALL BE CONFIRMED AND CORRELATED AT THE JOB SITE. FABRICATION PROCESSES AND TECHNIQUES OF CONSTRUCTION, COORDINATION OF HIS WORK WITH THAT OF ALL OTHER TRADES AND THE SATISFACTORY PERFORMANCE OF HIS WORK.
Equilibrium Engineers LLC
DATE 4/4/11

Milo White
L & M Industrial Fabrication
31975 Rolland Dr
Tangent, OR 97389

RE P ovidence Office Park Parking Structure 2 Expansion
Stair Towers 3 and 4 Structural Calculations

Dear Milo

Attached please find calculation sheets 01 through 09, dated February 17 2011 which verify the structural adequacy of stair tower 3 and 4 foi the Providence Office Park Parking Structure 2 Expansion, as shown on drawing sheets E-1 through E-5 dated March 3, 2011 Design is based on the requirements of the 2010 Oregon Structural Specialty Code based on the 2009 International Building Code

If you have any questions or need further information, please call me

Sincerely,

Jesse C Nash

JCN kw
21063 01/calcs 03 03 11 docx

STRUCTURAL REGISTERED PROFESSIONAL ENGINEER 15773 OREGON JAN 21 1992 JERALD L ABDIE
EXPIRES 12-31

10-199 336-DR-Hs

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Seattle Tacoma Portland Eugene San Francisco Sacramento Los Angeles Irvine San Diego Phoenix Denver St Louis New York

kpff Consulting Engineers Portland Oregon	Project PROVIDENCE OFFICE STAIR TOWER 3 & 4	By JCN	Sheet No 01/09
	Location PORTLAND, OR	Date 02/17/11	
	Client L & M FAB,	Revised	Job No
	STAIR CALCULATIONS	Date	

LOADS:

DEAD

PAN TREADS & RISERS	3 psf
2" CONC TREADS	25 psf
HSS FRAMING	12 psf
	40 psf
LANDING CONC (4")	50 psf
- 10 ga pk	6 psf
- FRAMING	14 psf
	70 psf

LIVE: 100 psf

SEISMIC

$$F_p = \frac{0.4 a_p S_{DS} W_p}{(R_p / I_p)} (1 + 2 z/h)$$

MAX: $1.6 S_{DS} I_p W_p$

MIN: $0.3 S_{DS} I_p W_p$

S_{DS}: 0.72

a_p 1.0 } ASCE 7 TABLE 13.5.1 OTHER RIGID COMPONENTS LOW DEFORMABILITY

R_p 1.5 }

I_p 1.5

z/h : 1.0

$F_p = 0.864 W_p$

MAX 1.73 W_p ✓

MIN 0.32 W_p ✓

DESIGN

INNER STRINGERS

MAX SPAN = ~~13'-0"~~ 8'-8"

TRIB. 2'-0"

{ AFTER CALCS WERE COMPLETE I REALIZED I USED THE WRONG SPAN, BUT CALCS ARE CONSERVITIVE

$W_U = 1.2(2\ 40 psf) + 1.6(2 \cdot 100 psf) = 416 plf$

$W_{ALL} = 2\ 40 psf + 2\ 100 psf = 280 plf$

$M_V = wl^2/8 = ~~8.8~~ kft = 3.9 kft$

$\phi M_n = 69.5 kft$ ✓ (HSS 12x2x1/4 AISC TABLE 3-12)

$\Delta_{TOTAL} = 5wl^4/384EI = 0.071" = L/2197$ ✓ OK

$R_U = ~~2.7 k~~ = 1.8k$

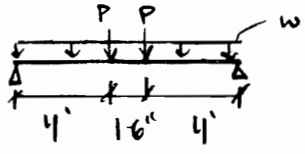
$R_{ALL} = ~~1.8 k~~ = 1.2k$

USE HSS 12x2x1/4 INNER STRINGERS

DESIGN CONT

INTERMEDIATE LANDING FLIGHT HEADER

$P_U = 27\,k$, $P_{ALL} = 1.8\,k$ } REACTION FROM INNER STRINGER



$W_U = 225'(1.6\ 100\,psf + 1.2\ 70\,psf) = 549\,plf$

$W_{ALL} = 225(100\,psf + 70\,psf) = 383\,plf$

$R_U = 5.3\,k$

$R_{ALL} = 3.6\,k$

$M_U = wl^2/8 + Pa = 6.2\,kft + 10.8\,kft = 17.0\,kft$

$\Delta_{ALL} = L/240 = 0.475"$ BUT LIMIT TO $L/360 = 0.316"$ $\therefore I_{REQD} = 20\,in^4$

TRY HSS $6\times4\times3/8$ $I = 28.3\,in^4$ $\phi M_n = 41\,kft > 17\,kft$ ✓OK

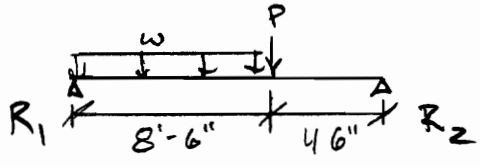
$\Delta_{TOTAL} = 5wl^4/384EI + Pa/24EI(3l^2 - 4a^2) = 0.086" + 0.131" = 0.217"$ ✓OK

USE HSS $6\times4\times3/8$ AT INTERMEDIATE LANDING HEADER

OUTER STRINGER (FROM INT LANDING TO FLIGHT HEADER)

$P_U = 5.3\,k$, $P_{ALL} = 3.6\,k$ } REACTION FROM INT LANDING FLIGHT HEADER

$W_U = 416\,plf$, $W_{ALL} = 280\,plf$ } SEE PG 1



$M_U = 20.8\,kft$ – SEE ATTACHED

$\phi M_n = 69.5\,kft$ ✓OK (HSS $12\times2\times1/4$ AISC TABLE 3-12)

$\Delta_{TOTAL} = 0.151" = L/1033$ ✓OK

$R_{1U} = 4.2k$ $R_{2U} = 4.6k$

$R_{1ALL} = 2.9k$ $R_{2ALL} = 3.1k$

USE HSS $12\times2\times1/4$ OUTER STRINGER

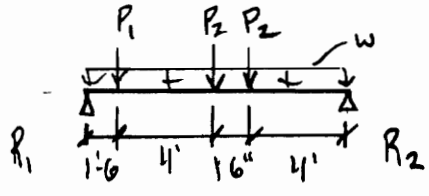
FLOOR LEVEL FLIGHT HEADER

$P_{1U} = 4.2k$, $P_{1ALL} = 2.9k$ } FROM OUTER STRINGER

$P_{2U} = 2.7k$, $P_{2ALL} = 1.8k$ } FROM INNER STRINGER

$W_U = 549\,plf$

$W_{ALL} = 383\,plf$



03/09

Title
Dsgnr
Description

Job #
Date 7 19AM 16 FEB 11

Scope

Single Span Beam Analysis

Description Outer Stringer (from int landing to flight header)

General Information

Center Span	13 00 ft	Moment of Inertia	86 900 in4
Left Cantilever	ft	Elastic Modulus	29 000 ksi
Right Cantilever	ft	Beam End Fixity	Pin Pin

Point Loads

Magnitude	5 300 k	k	k	k	k
Location	8 500 ft	ft	ft	ft	ft

Trapezoidal Loads

Magnitude @ Left	0 416 k/ft	k/ft	k/ft	k/ft
Magnitude @ Right	0 416 k/ft	k/ft	k/ft	k/ft
Dist To Left Side	ft	ft	ft	ft
Dist To Right Side	8 500 ft	ft	ft	ft

Query Values

Center Location	0 000 ft	**Left Cant**	0 000 ft	**Right Cant**	0 000 ft
Moment	0 00 k ft		0 00 k ft		0 00 k ft
Shear	4 21 k		0 00 k		0 00 k
Deflection	0 00000 in		0 00000 in		0 00000 in

Summary

Moments

Max + @ Center	20 79 k ft	at	8 49 ft
Max @ Center	0 00 k ft	at	0 00 ft
@ Left Cant	0 00 k ft		
@ Right Cant	0 00 k ft		

Maximum = 20 79 k ft

Shears

@ Left	4 21 k
@ Right	4 62 k
Maximum	4 62 k

Reactions

@ Left	4 21 k
@ Right	4 62 k

Deflections

@ Center	0 224 in	at	6 77 ft
@ Left Cant	0 000 in	at	0 00 ft
@ Right Cant	0 000 in	at	0 00 ft

DESIGN CONT

FLOOR LEVEL FLIGHT HEADER (CONT)

M_U = 24.3 kft - SEE ATTACHED

TRY HSS 6x4x1/2 $\quad \phi M_n$ = 50.5 kft ✓OK

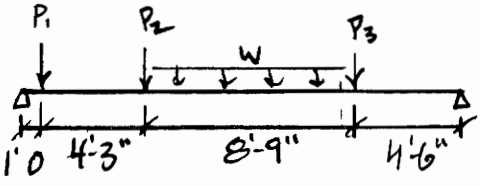
Δ_{TOTAL} = 0.350" = L/377 ✓OK

R_{1U} = 9.0K $\quad R_{2U}$ = 6.7K

R_{1ALL} = 6.2K $\quad R_{2ALL}$ = 4.6K

USE HSS 6x4x1/2 AT FLOOR LEVEL FLIGHT HEADER

OUTER STRINGER (FULL RUN)



P_{1U} = 11'·549 plf/2 = 3.0K

P_{1ALL} = 11'·383 plf/2 = 2.1K

P_{2U} = 6.7K } FROM FLOOR LEVEL FLIGHT HEADER

P_{2ALL} = 4.6K }

P_{3U} = 5.3K } FROM INT LANDING FLIGHT HEADER

P_{3ALL} = 3.6K }

W_U = 416 plf

W_{ALL} = 280 plf

M_U = 44.4 kft - SEE ATTACHED

ϕM_n = 69.5 kft ✓ (HSS 12x2x1/4 AISC TABLE 3-12)

Δ_{TOTAL} = 0.765" = L/290 > L/240 ✓OK

CALC Δ_{LIVE} - ASSUME 40psf DEAD + 100 psf LIVE

$\therefore \Delta_{LIVE} = \Delta_{TOTAL} \cdot \frac{100}{140}$ = 0.546" = L/407 > L/360 ✓OK

USE HSS 12x2x1/4 OUTER STRINGER

LANDING

10 GA R 2' CONT SPAN $\quad W_U = 1.2(70psf) + 1.6(100psf) = 244$ plf $\quad M_U = Wl^2/12 = 976$ in#

$Z = bt^2/4 = 12"(0.1345")^2/4 = 0.0543\ in^3 \quad \phi M_n = 0.9 \cdot 36ksi \cdot 0.0543\ in^3 = 1759$ in# ✓OK

Δ_D = 0.063" = L/379 FROM ROARK'S FORMULAS FOR STRESS & STRAIN R FIXED ON 4 SIDES

L 3x3x1/4 SUPPORT

SPAN = 4.5' $\quad M_U = (2')(244\ psf)(4.5)^2/8 = 1.3$ kft $\quad \phi M_n$ = 2.1 kft > 1.3 kft ✓OK

(SEE ATTACHED)

05/09

Title
Dsgnr
Description

Job #
Date 7 32AM 16 FEB 11

Scope

Single Span Beam Analysis

Description Floor Level Flight Header

General Information

Center Span	11 00 ft	Moment of Inertia	34 000 in4
Left Cantilever	ft	Elastic Modulus	29 000 ksi
Right Cantilever	ft	Beam End Fixity	Pin Pin

Uniform Loads

On Center Span		On Left Cantilever		On Right Cantilever	
# 1	0 549 k/ft	# 1	k/ft	# 1	k/ft

Point Loads

Magnitude	4 200 k	2 700 k	2 700 k	k	k
Location	1 500 ft	5 500 ft	7 000 ft	ft	ft

Query Values

Center Location	0 000 ft	Left Cant	0 000 ft	Right Cant	0 000 ft
Moment	0 00 k ft		0 00 k ft		0 00 k ft
Shear	8 98 k		0 00 k		0 00 k
Deflection	0 00000 in		0 00000 in		0 00000 in

Summary

Moments				Shears		Reactions	
Max + @ Center	24 27 k ft	at	5 51 ft	@ Left	8 98 k	@ Left	8 98 k
Max @ Center	0 00 k ft	at	11 00 ft	@ Right	6 66 k	@ Right	6 66 k
@ Left Cant	0 00 k ft			Maximum	8 98 k		
@ Right Cant	0 00 k ft			**Deflections**			
				@ Center	0 514 in	at	5 49 ft
Maximum =	**24 27** k ft			@ Left Cant	0 000 in	at	0 00 ft
				@ Right Cant	0 000 in	at	0 00 ft

06/09

Rev 580003
User KW-0600870, Ver 5 8 0 1 Nov 2006
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Single Span Beam Analysis

Description Outer Stringer (full run)

General Information

Center Span	18 50 ft	Moment of Inertia	86 900 in4
Left Cantilever	ft	Elastic Modulus	29 000 ksi
Right Cantilever	ft	Beam End Fixity	Pin Pin

Point Loads

Magnitude	3 000 k	6 700 k	5 300 k	k	k
Location	1 000 ft	5 250 ft	14 000 ft	ft	ft

Trapezoidal Loads

Magnitude @ Left	0 416 k/ft	k/ft	k/ft	k/ft
Magnitude @ Right	0 416 k/ft	k/ft	k/ft	k/ft
Dist To Left Side	5 250 ft	ft	ft	ft
Dist To Right Side	14 000 ft	ft	ft	ft

Query Values

Center Location	0 000 ft	**Left Cant**	0 000 ft	**Right Cant**	0 000 ft
Moment	0 00 k ft		0 00 k ft		0 00 k ft
Shear	10 67 k		0 00 k		0 00 k
Deflection	0 00000 in		0 00000 in		0 00000 in

Summary

Moments				**Shears**		**Reactions**	
Max + @ Center	44 41 k ft	at	7 60 ft	@ Left	10 67 k	@ Left	10 67 k
Max @ Center	0 00 k ft	at	18 50 ft	@ Right	7 97 k	@ Right	7 97 k
@ Left Cant	0 00 k ft			Maximum	10 67 k		
@ Right Cant	0 00 k ft			**Deflections**			
				@ Center	1 123 in	at	9 12 ft
Maximum =	**44 41** k ft			@ Left Cant	0 000 in	at	0 00 ft
				@ Right Cant	0 000 in	at	0 00 ft

07/09

Bending Capacity of Angle L 3x3x1/4

CONSTANTS

$F_y = 36000 \frac{lb}{in^2}$ $E = 29000000 \frac{lb}{in^2}$ $\phi = 0.9$ $\Omega = 1.67$

INPUT

$t = 0.25\ in$ $C_b = 1.0$ $Z = 1.02\ in^3$

$b = 3\ in$ $L = 4.5\ ft$ $S = 0.569\ in^3$

CALCULATIONS

YEILDING

$M_y = F_y S$ $M_y = 1707\ lb\ ft$

LTB

Bending about geometric axes of equal leg angle

$$M_{ec} = \frac{0.66 E b^4 t C_b}{L^2}\left[\sqrt{1 + 0.78\left(\frac{L t}{b^2}\right)^2} - 1\right] \quad \text{(toe in compression)}$$

$M_{ec} = 7308\ lb\ ft$

$$M_{et} = \frac{0.66 E b^4 t C_b}{L^2}\left[\sqrt{1 + 0.78\left(\frac{L t}{b^2}\right)^2} + 1\right] \quad \text{(toe in tension)}$$

$M_{et} = 29461\ lb\ ft$

$$M_{ltb} = if\left[M_{et} > 0.8M_y, \left(1.92 - 1.17\sqrt{\frac{0.8M_y}{M_{et}}}\right)0.8M_y, \left(0.92 - \frac{0.17M_{et}}{0.8M_y}\right)M_{et}\right]$$

$M_{ltb} = 2278\ lb\ ft$

LLB

Check slendeness

$\lambda = \frac{b}{t}$ $\lambda = 12$ $\lambda_r = 0.91\sqrt{\frac{E}{F_y}}$ $\lambda_p = 0.54\sqrt{\frac{E}{F_y}}$

$\lambda_{check} = if(\lambda < \lambda_p, "Compact", if(\lambda < \lambda_r, "Non\text{-}compact", "Slender"))$

$\lambda_{check} = "Compact"$

$$M_{llb} = if\left[\lambda < \lambda_p, 10000000000\ lb\ ft, if\left[\lambda < \lambda_r, F_y\, 0.8S\left[2.43 - 1.72\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}}\right], \frac{0.8\, S\, 0.71\, E}{\left(\frac{b}{t}\right)^2}\right]\right]$$

$M_{llb} = 10000000000\ lb\ ft$

Nominal Moment Strength

$$M = \begin{pmatrix} 1.5M_y \\ M_{ltb} \\ M_{llb} \end{pmatrix} \quad M_n = min(M) \quad M_n = 2278\ lb\ ft$$

$\frac{M_n}{\Omega} = 1364\ lb\ ft$ $\phi M_n = 2050\ lb\ ft$

DESIGN CONT

CONNECTIONS:

WELDS.

$R_{U\,MAX} = 10.7\,K$

REQD 3/16 FILLET = $10.7k / 0.75 \cdot 0.6 \cdot 70ks \cdot 0.707 \cdot 0.18" = 2.7"$

ALL WELDED CONN ARE > 2.7" LONG ✓OK

SHEAR TABS

$R_{U\,MAX} = 10.7\,K$

REQD 1/4" PL = $10.7K / 0.75 \cdot 0.6 \cdot 36ks \cdot 0.25" = 2.6"$

ALL SHEAR TABS ARE > 2.6" LONG ✓OK

BOLTS

$R_{U\,MAX} = 10.7\,K$

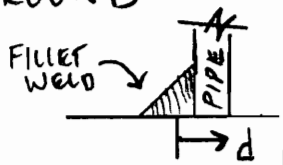
$\phi r_n = 7.95\,K$ (3/4"Ø A307 BOLTS FROM TABLE 7-1)

PROVIDE MIN (2) 3/4"Ø A307 BOLTS

HANDRAIL DESIGN

ASSUME 1 1/4"Ø X-STRONG PIPE W/ 1/4" FILLET WELD ALL AROUND

M_{ALL} 1 1/4"Ø X-S = 0.686 kft — AISC TABLE 3-15



WELD CAPACITY = $M = S_W\, t_w\, \sigma_{ALL}$

$S_W = \pi d^2/4$ $\qquad d = 1.66" + \frac{2(1/4")}{3} = 1.83\,in$

$S_W = \pi(1.83in)^2/4 = 2.63\,in^2$

$\therefore M_{ALL\,WELD} = (2.63\,in^2)(1/4)(0.3 \cdot 0.707 \cdot 70\,ksi) = 9.76\,k\text{-}in = 0.813\,kft > 0.686\,kft$

PIPE WILL CONTROL DESIGN

DETERMINE MAX SPACING.

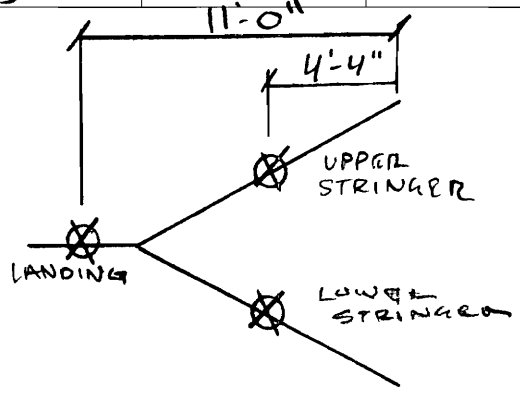
$M = wl^2/36" = 0.686\,kft$ $\qquad w = 50\,plf$

$l_{MAX} = 54" = 4'\text{-}6"$ $\qquad 50plf \cdot 225\# > 200\#$ ✓OK

USE 1 1/4"Ø X-STRONG PIPE @ 4'-6" MAX W/ 1/4" FILLET WELD ALL AROUND

kpff Consulting Engineers Portland Oregon	Project PROVIDENCE STAIR TOWER 3 & 4	By JCN	Sheet No 09/09
	Location PORTLAND OR	Date	
	Client L & M FAB	Revised	Job No
	STAIR CALCULATIONS	Date	

SEISMIC DESIGN



WEIGHTS

LANDING : 45' · 9.5' 70psf = 30K

STRINGERS : 4' · 8.67 · 40psf = 1.4K

FLOOR LANDING 4' · 11' 70psf = 3.1K

DISTRIBUTE LOADS:

LANDING · (0.864)(3.0K) = 2.6K (HALF GOES UP & HALF GOES DOWN)

STRINGERS · (0.864)(1.4K) = 1.2K

M = (4.33')(1.2K) + (11')(2.6K/2) = 19.5 kft

RESOLVE COUPLE BETWEEN STRINGERS

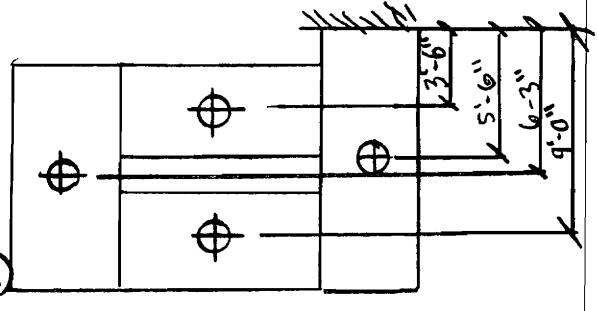
T/C = 19.5 kft/4' = 4.9 K

REQD 3/16 WELD = 4.9K/0.75 0.6 · 70ksi · 0.707 0.18" = 1.22"

PROVIDE MIN 2" OF 3/16" FILLET @ STRINGER CONN.

CONNECTION TO STRUCTURE

DISTRIBUTE LOADS



LANDING = 2.6K

STRINGERS = 1.2K

FLOOR LANDING = (0.864)(3.1K) = 2.7K

M = (3.5')(1.2K) + (9')(1.2K) + (6.25')(2.6K) + (5.5)(2.7K)

M = 46.1 K

T/C = 46.1K/4' = 11.5K

REQD 3/16" FILLET = 11.5K/0.75 0.6 70ksi 0.707 0.18" = 2.88"

PROVIDE MIN 3" OF 3/16" FILLET AT STRINGER TO STRUCTURE CONN

J.R.MacKenzie, P.E., S.E.

Civil and Structural Engineer
Phone 604 999 2457

Project Providence Office Park II Addition Location Portland Oregon	Project No 11 55 VicWest 25 28 98900	Page No 1 of 76
Subject Wall Panels	Prepared By J R MacKenzie Design/Check/Review Design	Date 24 May 2011

Codes Oregon Structural Specialty Code 2007 and ASCE 7-05
Aluminum Design Manual 2005, The Aluminum Association

Material
Cladding 4 mm Alpolic, 3105-H14 aluminum, F_{tu}=22 ksi, F_{ty}=18 ksi
Studs 18 ga at 16" c/c

Occupancy Category II [Table 1-1]

Live Loads
Snow p_g=10 psf (ground snow load load) [Snow Load Analysis for Oregon 2007],
p_f=0 7*C_e*C_t*I_S*p_g=10 1 psf (Roof Snow Load), Exposure C_e=1 2 (Exposure B, sheltered) [T 7-2], C_t=1 2 (unheated structure) [T 7-3], Importance I_S=1 0, but not less than 25 0 psf [OSSC-2007 1608 1], therefore p_f=25 0 psf,
except Canopy Eyebrows 51 6 psf (because of snow drifting)
V=94 5 mph, Exposure B, I_w=1 0, z=25 to 65 feet, h=75 feet (more than 60 feet, use Method 2 [Section 6 5, ASCE 7])
GC_p=0 9,-0 9 in Zone 4, 0 9, -1 8 in Zone 5 [Figure 6-17]
K_z=2 01*$(z/z_g)^{2/\alpha}$=0 70 at 30 feet, 0 91 at 75 feet [Table 6-3]
α=7 0, z_g=1200 for Exposure B [Table 6-2]
K_{zt}=1 0 [Section 6 5 7 2]
K_d=0 85 [Table 6-4]
q_z=0 00256*K_z* K_{zt}* K_d *V^2*I_w=13 6 psf at 30 feet, 17 7 psf at 75 feet [Equation 6-15]
q_i=0 0 (exposed only to exterior pressures, building wall takes internal pressures)
p_i=q_z*GC_p=12 2 psf at 30 feet, 15 9 psf at 75 feet on windward faces [Sect 6 5 12 4 2]
p_i=q_h*GC_p= -15 9 psf at Zone 4, -31 8 psf at Zone 5 on leeward faces [Sect 6 5 12 4 2]
p=-31 8 psf (upwards) for Canopy Eyebrows
Seismic S_s=0 97, F_a=1 11, S_{DS}=2*F_a*S_s/3=0 71
I_p=1 0 [Section 13 1 3]
panel a_p=1 0, R_p=2 5, connections a_p=1 25, R_p=1 0 [Table 13 5-1]
F_p=0 4*a_p*S_{DS}*W_p*(1+2*z/h)/(R_p/I_p) (assume z=h) [Equation 13 3-1]
=0 34*W_p panel, 1 06*W_p connector
F_p=1 6*S_{DS}* I_p*W_p=1 14*W_p maximum [Equation 13 3-2]
F_p=0 3*S_{DS}* I_p*W_p=0 21*W_p minimum [Equation 13 3-3]
panel dead is 3 psf, so F_p=1 0 psf panel, 3 4 psf connector, so wind governs

Deflections
Panel L/60

Index

Page	*Item*
1 to 10	Excel Worksheets, wall panels 30 to 75 feet above grade, Panels A to I
1 to 10	Excel Worksheets, canopy panels 30 to 65 feet above grade, Panel J to K

Date Sealed 24 May 2011

SUBMITTAL REVIEW
☒ NO EXCEPTIONS TAKEN ☐ REVISE AND RESUBMIT
☐ MAKE CORRECTIONS NOTED ☐ SUBMIT SPECIFIED ITEM
CHECKING IS ONLY FOR GENERAL CONFORMANCE WITH THE DESIGN CONCEPT OF THE PROJECT AND GENERAL COMPLIANCE WITH THE INFORMATION IN THE CONTRACT DOCUMENTS. ANY ACTION SHOWN ABOVE, IS SUBJECT TO THE REQUIREMENTS OF THE PLANS AND SPECIFICATIONS. CONTRACTOR IS RESPONSIBLE FOR DIMENSIONS WHICH SHALL BE FABRICATED AND CORRELATED AT THE JOB SITE; FABRICATION PROCESSES AND TECHNIQUES OF CONSTRUCTION; COORDINATION OF HIS WORK WITH THAT OF ALL OTHER TRADES AND THE SATISFACTORY PERFORMANCE OF HIS WORK.
Equilibrium Engineers LLC
DATE 6/2/11

10-19936-DFS-05.06

STRUCTURAL REGISTERED PROFESSIONAL ENGINEER 16444 OREGON JULY 11 2000 JAMES R MACKENZIE
EXPIRES 31 Dec 2012

J.R.MacKenzie, P.E., S.E.

Civil and Structural Engineer
Phone 604 999 2457

Project Providence Office Park ~~II~~ - Addition Location Portland Oregon	Project No ~~11-37~~ VicWest 25-28 - 98420	Page No 1 of 76
Subject Wall Panels	Prepared By J R MacKenzie Design/Check/Review Design	Date ~~4 April 2011~~

Codes — Oregon Structural Specialty Code 2007 and ASCE 7-05
Aluminum Design Manual 2005, The Aluminum Association

Material
- Cladding — 4 mm Alpolic, 3105-H14 aluminum F_{tu}=22 ksi F_{ty}=18 ksi
- Studs — 18 ga at 16 ' c/c

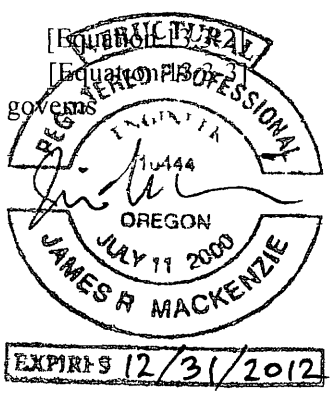
Occupancy — Category II [Table 1 1]

Live Loads
- Snow — p =10 psf (ground snow load load) [Snow Load Analysis for Oregon 2007],
  p_f=0 7*C_e*C_t*I_S*p =10 1 psf (Roof Snow Load), Exposure C_e=1 2 (Exposure B, sheltered) [T 7 2], C_t=1 2 (unheated structure) [T 7-3], Importance I_S=1 0 but not less than 25 0 psf [OSSC 2007 1608 1] therefore p_f=25 0 psf
  except Canopy Eyebrows 51 6 psf (because of snow drifting)
- Wind — V=94 5 mph Exposure B, I_w=1 0 z=25 to 65 feet h=75 feet (more than 60 feet use Method 2 [Section 6 5 ASCE 7])
  GC_p=0 9 -0 9 in Zone 4, 0 9, -1 8 in Zone 5 [Figure 6 17]
  K_z=2 01*(z/z)$^{2/\alpha}$=0 70 at 30 feet 0.91 at 75 feet [Table 6 3]
  α=7 0 z_t=1200 for Exposure B [Table 6 2]
  K_{zt}=1 0 [Section 6 5 7 2]
  K_d=0 85 [Table 6 4]
  q_z=0 00256*K_z* K_{zt}* K_d *V^2*I_w=13 6 psf at 30 feet 17,0 psf at 65 feet [Equation 6-15]
  q_h=0 0 (exposed only to exterior pressures, building wall takes internal pressures)
  p=q_z*GC_p=12 2 psf at 30 feet 15.9 psf at 75 feet on windward faces [Sect 6 5 12 4 2]
  p=q_h*GC_p= 15.9 psf at Zone 4 -31.8 psf at Zone 5 on leeward faces [Sect 6 5 12 4 2]
  p= 31.8 psf (upwards) for Canopy Eyebrows
- Seismic — S_s=0 97 F_a=1 11, S_{DS}=2*F_a*S_s/3=0 71
  I_p=1 0 [Section 13 1 3]
  panel a_p=1 0 R_p=2 5 connections a_p=1 25, R_p=1 0 [Table 13 5-1]
  F_p=0 4*a_p*S_{DS}*W_p*(1+2*z/h)/(R_p/I_p) (assume z=h) [Equation 13 3-1]
  =0 34*W_p panel 1 06*W_p connector
  F_p=1 6*S_{DS}* I_p*W_p=1 14*W_p maximum [Equation 13-3]
  F_p=0 3*S_{DS}* I_p*W_p=0 21*W_p minimum [Equation 13-3]
  panel dead is 3 psf, so F_p=1 0 psf panel, 3 4 psf connector so wind governs

Deflections
- Panel — L/60

Index

Page	*Item*
~~1 to 10~~	~~Excel Worksheets, 0 to 30 feet above grade, Panels A to L~~
1 to 10	Excel Worksheets, 30 to 75 feet above grade, Panel A to
1 to 10	Excel Worksheets, 65 feet above grade, Roof Panels A to



Date Sealed 4 April 2011

TOMLAND CONSULTING	PROJECT	DATE	MAY 13 11
976 THERMAL DRIVE	PROVIDENCE OFFICE PARK II	REV #	
COQUITLAM B C V3J 6S1	- ADDITION	R DATE	
CELL 604 728 4032	PORTLAND OREGON	PROJ #	25 28-98900
EMAIL thomp1@shaw ca		PAGE #	1 of 6

DESIGN CRITERIA OSSC 2007
ASCE 7-05
ALUM DESIGN MANUAL - 2005

DESIGN LOADS ROOF SNOW LOAD IS PSF + DRIFT

SEISMIC ASCE 7 05 SECTION 11 8
OCCUPANCY CAT B OR II
SITE CLASS D
SEISMIC DESIGN CAT - D
IMPORTANCE I = 1 C Ss 0 97 Fa = 1 11
Sds 0 71g
S1 0 39g
Fv 1 74
Sd1 - 0 38g

WIND BASIC WIND SPEED V. 95 MPH
EXPOSURE B
IMPORTANCE Iw 1 C
Z - 24491 - 170 33 = 74 58
TO 170 33 - 0'
Z > 60' USE METHOD 2

TOMLAND CONSULTING	PROJECT	DATE	MAY 18/11
976 THERMAL DRIVE	PROVIDENCE CFFICE PARK II	REV #	
COQUITLAM B C V3J 6S1	- 10017TH	R DATE	
CELL 604 728 4032	PORTLAND OREGON	PROJ #	2528-9890a
EMAIL thomp1@shaw ca		PAGE #	2 of 6

6 5 11 2 2 COMPONENTS + CLADDING-

	Zone 4	Zone 5
GCp INWARD FOR 10 SQ FT PANEL	+ 0 9	+ C 9
GCp OUTWARDS FOR 10 SQ FT PANEL	- 0 9	- 1 8

$$q_z = 0.00256\ K_z \cdot K_{zt} \cdot K_d\ V^2\ I$$

	K_h (ASH)	K_z
< 30'0	0 70	$2.01\ (z/z_g)^{2/\alpha}$
		$= 2.01\left(\frac{30}{1200}\right)^{2/9.0} = 2.01(0.025)^{0.2557}$
		= 0 7006
< 75'0	0.91	$= 1.01\left(\frac{75}{1200}\right)^{0.2557}$
		= 0.9102

TOMLAND CONSULTING
976 THERMAL DRIVE
COQUITLAM B C V3J 6S1
CELL 604 728 4032
EMAIL thomp1@shaw ca

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At 30'-0" $q_z = 0.00256 \times 0.7000 \times 100 \times 0.85 \times 97.5^2 \times 10$

$= 13.61$ PSF

At 64'-0" $q_z = 0.00256 \times 0.9102 \times 100 \times 0.85 \times 97.5^2 \times 10$

$= 17.68$ PSF

$P_{INWARD} = q((-C_p) - q_i((-C_{pi})$

WALL PANELS ARE INSTALLED WITH STEEL STUDS AND GWB WALLS BY OTHERS $- q_i((-C_{pi}) - 0$

		Zone 4	Zone 5
At 30'-0"	P_{INWARD}	$13.61 \times 0.9 =$ 12.25 PSF	$13.61 \times 0.9 =$ 12.25 PSF
	$P_{OUTWARD}$	13.61×-0.9 $= -12.25$ PSF	13.61×1.5 $= -24.50$ PSF
At 75'-0"	P_{INWARD}	$17.68 \times 0.9 =$ 15.91 PSF	$17.68 \times 0.9 =$ 15.91 PSF
	$P_{OUTWARD}$	17.68×-0.9 $= -15.91$ PSF	17.68×1.5 $= -31.82$ PSF

TOMLAND CONSULTING	PROJECT	DATE	MAY 18 / 11
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CELL 604 728 4032	PORTLAND OREGON	PROJ #	25-28 98900
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SNOW LOAD ON EYE BROWS

$\Delta h = 11 + 24 + 24 - \frac{119}{12} = 9.91'$

$l_u = 42 + 36 + 42 \quad 120'$

$P_f = 0.7\, C_c\, C_t\, I\, p_g$

$P_f = 0.7 \times 1.2 \times 1.2 \times 1.0 \times 1.0 = 10.1\ PSF$

BUT NO LESS THAN 25 PSF

$P_d = 0.43 \sqrt[3]{l_u} \sqrt{P_f + 10} - 15$

$= 0.43 \times 120^{.55} \times 21^{.45} - 15$

$= 29.78'$ SAY 30

$\gamma = 0.13 \times 10 + 1.4 \quad 15.3\ PCF$

HI LOW SNOW LOAD $3 \times 15.3 = \underline{45.9\ PSF}$

USE DRIFTING LOADING - $\underline{51.0\ PSF}$ (WINDWARD)

$38.5\ PSF$ (LEEWARD)

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976 THERMAL DRIVE
COQUITLAM B C V3J 6S1
CELL 604 728 4032
EMAIL thomp1@shaw ca

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3.0 DESIGN CRITERIA

3.1 WIND DESIGN LOADS

> 30'-0 TO 75'-0

WALL PANEL + SOFFIT TO ^ ELEVATION

BASIC WIND DESIGN LOADS	ZONE 4 kPa	ZONE 4 PSF	ZONE 5 kPa	ZONE 5 PSF
P INWARD	-	15.91	-	15.91
P OUTWARD	-	15.91	-	31.82

RAIN SCREEN FACTOR = 75
MAX PANEL DEFLECTION = L/60
MAX FRAME DEFLECTION = L/180

SNOW LOAD ON EYE BROWS

SNOW LOAD 51.6 PSF DOWNWARD

WIND LOAD 31.82 PSF OUTWARD

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976 THERMAL DRIVE
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BASIC WIND SPEED	94 5 MPH
OCCUPANCY CATEGORY	B OR II
WIND IMPORTANCE FACTOR	1 0
WIND EXPOSURE	B
ENCLOSURE CLASSIFICATION	ENCLOSED
~~G Cpi INWARD~~	
~~OUTWARD~~	

WIND DESIGN LOADS BASED ON ASCE 7 05 CHAPTER 6
MINIMUM DESIGN LOADS FOR BUILDINGS AND OTHER STRUCTURES

SEISMIC IMPORTANCE FACTOR Ip	1 0
OCCUPANCY CATEGORY	B OR II
SITE CLASS	D
MAPPED SPECTRAL RESPONSE ACCELERATIONS	
Ss	0 97
S1	0 53
COMPONENT RESPONSE MODIFICATION FACTOR	
Rp	2 5
COMPONENT AMPLIFICATION FACTOR	
ap	1 0

SEISMIC DESIGN LOADS BASED ON ASCE 7 05 CHAPTER 13
MINIMUM DESIGN LOADS FOR BUILDINGS AND OTHER STRUCTURES "

GOVERNING LOAD CRITERIA IS **WIND LOADING**

TOMLAND CONSULTING	PROJECT	DATE	MAY 18/11
976 THERMAL DRIVE	PROVIDENCE OFFICE PARK II	REV #	
COQUITLAM B C V3J 6S1	- ADDITION	R DATE	
CELL 604 728 4032	PLATTLAND ORCHARD	PROJ #	25-28-98900
EMAIL thomp1@shaw ca		PAGE #	7 of

WALL PANELS

PANEL	LENGTH	HEIGHT	# OF STIFF	TYPE OF STIFF	# OF CLIPS	# SCREW/CLIP
A	12	59 5	0	10 10 10	5	1 11 3
B	73 5	59 5	1		9	
C	73 5	23 5	0		7	
D	73 5	41 5	1		8	
E	102 CY	39.31	2		10	
F	50 25	39 31	2		13	
G	102 68	18	0		9	
H	103 75	36	2		10	
I	16	54 5	0		5	

EYE BROW PANELS

J	138 15	35 315	3	12	
K	117 25	35 315	2	11	
L					
M					

TOMLAND CONSULTING				PAGE 1 of 10
ARCHITECTURAL PANEL DESIGN CALCULATIONS FOR MERCURY METALS APOLIC PANELS				
PROJECT	PROVIDENCE OFFICE PARK II - Addition		DATE	May 19 / 11
LOCATION	PORTLAND OREGON		PROJ #	25 28 98900
PANELS	PE CORE - ROOF PANELS FROM 30 TO 65 FEET ELEVATION		REV #	0

DESIGN CRITERIA

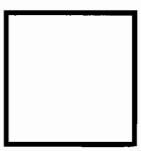
CANADA				USA		
CODE BCBC 2006				CODE OSSR 2007		
		KPA	PSF	BASE WIND SPEED	94 5	MPH
BASE WIND PRESS	1/10	0	0 0	BASE PRESSURE	13 61	PSF
	1/50	0	0 0	EXPOSURE	B	
				IMPORTANCE	1	
PANEL ELEV MIN	0	M // FT	0 0	PANEL ELEV MIN	30	FT
PANEL ELEV MAX	0	M // FT	0 0	PANEL ELEV MAX	75	FT

CALCULATED MAXIMUM WIND PRESSURES

PRESSURE	CAN SPEC (Pa)		CAN FACT (Pf)		USA SPEC (Pa)		USA FACT (Pf)		USE FOR CALC Pa	USE FOR CALC Pf
DIRECTION	Kpa	PSF	Kpa	PSF	PSF	Kpa	PSF	Kpa	PSF	PSF
INWARD	0 00	0 0	0 000	0 0	51 60	2 47	82 6	3 95	51 6	82 6
OUTWARD	0 00	0 0	0 000	0 0	31 82	1 52	50 9	2 44	31 8	50 9

CAN PHI FACTOR FOR DEFLECTION	0 75	RAIN SCREEN PANEL FACTOR	0 75
MAXIMUM DEFLECTION OF PANEL	60	USA PHI FACTOR FOR WIND DEFLECTION	0 70
MAXIMUM DEFLECTION OF FRAME	180		

PANEL DESIGNATION

PANEL SHAPE	EDGE CONDIT N [CODE]	LOADING CRITERIA	MAXIMUM STRESS ksi	MAXIMUM DEFLECTION in
SQUARE [1]  b / b	SIMPLE [1]	Bending	0 2874 (Wb/ts^2)	4 43 (Wb*b^2 / t^3)*10^ 6
		Membrane at center	6 (Wm/(b ts))^ 6666	0 013*(Wm b^2 / t)^ 3333
		Summary	Wm = 39571 10^ 10 (b^4/t^8) Wb^3	
		Chk Δ max		0443 Pa b^4/(Eap Tap^3)/12^2
RECTANGULAR [2] b / a WHERE $\alpha = b / a$ a > b	SIMPLE [1]	Bending σb max at center	0 75 (Wb b)/(a ts^2*(1+1 61 α^3))	14 (Wb b^3)/(a t^3) 10^ 6
		σa at center	Wb b*(0 225+ 382α^2 320α^3)/ ts^2	
		Membrane	8 (Wm/(a ts))^ 6666	0 016 ((Wm b^3)/(a t))^ 3333
		Summary	Wm = 6 66992 10^ 10 (b^6/(a^2 t^8)) Wb^3	
		Chk Δ max		γ Pa b^4/(Eap Tap^3)/12^2

THEORY

1) W total = W bending + W membrane
2) Deflection in bending = Deflection in membrane

ARCHITECTURAL PANEL DESIGN CALCULATIONS FOR MERCURY METALS APOLIC PANELS

PROJECT	PROVIDENCE OFFICE PARK II - Addition	DATE	May 19 / 11
LOCATION	PORTLAND OREGON	PROJ #	25 28 98900
PANELS	PE CORE - ROOF PANELS FROM 30 TO 65 FEET ELEVATION	REV #	0

ALPOLIC PROPERTIES

STRUCTURAL		4	mm
TENSILE STRENGTH		6913	psi
TENSILE YIELD STRENGTH		6429	psi
MIN YEILD STRENGTH		18000	psi
PUNCHING SHEAR RESIS	Max load	1920	lb
	1 diam	4025	psi
FLATWISE TENSILE		1806	psi
FLATWISE SHEAR		1225	psi
MIN BOND STRENGTH		33 6	in lb/in
WEIGHT		1 12	psf

MISCELLANEOUS

STRUCTURAL		VALUE	UNIT
COEF OF EXPANSION		1310000	in/in/deg F
PANEL FLEXURAL MODULUS (E)		5770000	psi
ALUM FLEXURAL MODULUS (E)		10000000	psi
ALUM MOMENT OF INTERIA (I)		0 000189	in^4/in
ALUM SECTION MODULUS (S)		0 00241	in^3/in
FLEXURAL STIFFNESS (EI)		1140	lb in^2/in
ALUM THICKNESS	ta =	0 02	in
APPARENT PANEL THK	ts =	0 1204	in
PANEL THICKNESS	t =	0 1575	in

PANEL CRITERIA

PANEL NUMBER	LENGTH a	HEIGHT b	a / b	PANEL SHAPE Code #	EDGE COND Code #	Wf + FACTOR / WORK	Wf FACTOR / WORK	Mf = Pf*b^2 / 144 x 8	σf = Mf / Sap	Reinforcem t Req d
	in	in				lb	lb	lb in	PSI	17100
J	138 75	35 315		2	1	2107 0	1299 3	67 0	27726 2	YES
OR J	138 75	35 315	3 9			1316 9	812 1			
K	117 25	35 315		2	1	1780 5	1098 0	67 0	27726 2	YES
OR K	117 25	35 315	3 3			1112 8	686 2			
L	0	0		#DIV/0!	1	0 0	0 0	0 0	0 0	NO
OR L	0	0	#DIV/0!			0 0	0 0			
M	0	0		#DIV/0!	1	0 0	0 0	0 0	0 0	NO
OR M	0	0	#DIV/0!			0 0	0 0			
N	0	0		#DIV/0!	1	0 0	0 0	0 0	0 0	NO
OR N	0	0	#DIV/0!			0 0	0 0			
O	0	0		#DIV/0!	1	0 0	0 0	0 0	0 0	NO
OR O	0	0	#DIV/0!			0 0	0 0			
P	0	0		#DIV/0!	1	0 0	0 0	0 0	0 0	NO
OR P	0	0	#DIV/0!			0 0	0 0			
Q	0	0		#DIV/0!	1	0 0	0 0	0 0	0 0	NO
OR Q	0	0	#DIV/0!			0 0	0 0			
R	0	0		#DIV/0!	1	0 0	0 0	0 0	0 0	NO
OR R	0	0	#DIV/0!			0 0	0 0			

PANEL REINFORCEMENT

PANEL NUMBER	REIN POINTS TOP / BOT	REINFORCED PANEL LENGTH a	REINFORCED PANEL HEIGHT b	a / b	PANEL SHAPE Code #	EDGE COND Code #	Wf + FACTORED	Wf FACTORED	α = b/a	Δmax = γ w b^4 / Ea Ta^3 144
	1/2//1/3//1/4	in	in	*			lb	lb		in
J	4	34 7	35 3	1 0	1	1	526 7	324 8	1 0181	1 0237
K	3	39 1	35 3	1 1	1	1	593 5	366 0	0 9036	1 3129
L	0	0 0	0 0	#DIV/0!	#DIV/0!	1	0 0	0 0	#DIV/0!	#DIV/0!
M	0	0 0	0 0	#DIV/0!	#DIV/0!	1	0 0	0 0	#DIV/0!	#DIV/0!
N	0	0 0	0 0	#DIV/0!	#DIV/0!	1	0 0	0 0	#DIV/0!	#DIV/0!
O	0	0 0	0 0	#DIV/0!	#DIV/0!	1	0 0	0 0	#DIV/0!	#DIV/0!
P	0	0 0	0 0	#DIV/0!	#DIV/0!	1	0 0	0 0	#DIV/0!	#DIV/0!
Q	0	0 0	0 0	#DIV/0!	#DIV/0!	1	0 0	0 0	#DIV/0!	#DIV/0!
R	0	0 0	0 0	#DIV/0!	#DIV/0!	1	0 0	0 0	#DIV/0!	#DIV/0!

ARCHITECTURAL PANEL DESIGN CALCULATIONS FOR MERCURY METALS APOLIC PANELS

PROJECT	PROVIDENCE OFFICE PARK II - Addition	DATE	May 19 / 11
LOCATION	PORTLAND OREGON	PROJ #	25 28 98900
PANELS	PE CORE - ROOF PANELS FROM 30 TO 65 FEET ELEVATION	REV #	0

REINFORCED PANEL BENDING & MEMBRANE LOADS AND DEFLECTIONS

PANEL NUMBER	Cy	C	1/C	Wb fact lb	Wm fact lb	Wt fact lb	DEFb fact in	DEFm fact in	DEF+/	DEF CHECK
J+	0 001364	0 00087843	1138 4	79 794	446 298	526 092	0 328	0 328	0 00	OK
J	0 001364	0 00087843	1138 4	66 470	257 979	324 449	0 273	0 273	0 00	OK
K+	0 001374	0 00122143	818 7	75 114	517 649	592 763	0 347	0 347	0 00	OK
K	0 001374	0 00122143	818 7	62 816	302 748	365 564	0 290	0 290	0 00	OK
L+	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
L	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
M+	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
M	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
N+	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
N	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
O+	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
O	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
P+	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
P	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Q+	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Q	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
R+	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
R	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

REINFORCED PANEL DESIGN - STRESS

PANEL #	MAX CENTER STRESS			MAX LOAD	MAX LOAD	Fy	PANEL	Ft	PANEL	FRAME
	BENDING fb fact	MEMBRANE fm fact	TOTAL ft fact	parallel to b	parallel to a	BENDING STRESS	CENTER CHECK	TENSILE STRESS	TENSION CHECK	REACTION Rf avg
	psi	psi	psi	lb / in	lb / in	psi		psi		lb / in
J+	1580 9	133 4	1714	16 1	16 1	17100	OK	6429	OK	7 67
J	1316 9	92 6	1410	11 2	11 2	17100	OK	6429	OK	4 73
K+	1488 2	147 3	1635	17 7	15 8	17100	OK	6429	OK	7 67
K	1244 5	103 0	1348	12 4	11 0	17100	OK	6429	OK	4 73
L+	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	17100	#DIV/0!	6429	#DIV/0!	#DIV/0!
L	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	17100	#DIV/0!	6429	#DIV/0!	#DIV/0!
M+	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	17100	#DIV/0!	6429	#DIV/0!	#DIV/0!
M	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	17100	#DIV/0!	6429	#DIV/0!	#DIV/0!
N+	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	17100	#DIV/0!	6429	#DIV/0!	#DIV/0!
N	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	17100	#DIV/0!	6429	#DIV/0!	#DIV/0!
O+	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	17100	#DIV/0!	6429	#DIV/0!	#DIV/0!
O	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	17100	#DIV/0!	6429	#DIV/0!	#DIV/0!
P+	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	17100	#DIV/0!	6429	#DIV/0!	#DIV/0!
P	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	17100	#DIV/0!	6429	#DIV/0!	#DIV/0!
Q+	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	17100	#DIV/0!	6429	#DIV/0!	#DIV/0!
Q	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	17100	#DIV/0!	6429	#DIV/0!	#DIV/0!
R+	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	17100	#DIV/0!	6429	#DIV/0!	#DIV/0!
R	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	17100	#DIV/0!	6429	#DIV/0!	#DIV/0!

ARCHITECTURAL PANEL DESIGN CALCULATIONS FOR MERCURY METALS APOLIC PANELS

PROJECT	PROVIDENCE OFFICE PARK II - Addition	DATE	May 19 / 11
LOCATION	PORTLAND OREGON	PROJ #	25 28 98900
PANELS	PE CORE - ROOF PANELS FROM 30 TO 65 FEET ELEVATION	REV #	0

REINFORCED PANEL DESIGN - DEFLECTION

PANEL #	DEFaj fact in	DEFaj spec in	LENGTH a	DEFLECT RATIO	HEIGHT b	DEFLECT RATIO	DEF CRITERIA	LENGTH a OK	LENGTH b OK	REINF PANEL ?
J+	0 328	0 1435	34 7	241 7	35 3	246 1	60	1	1	NO
J	0 273	0 1195	34 7	290 2	35 3	295 5	60	1	1	NO
K+	0 347	0 1519	39 1	257 4	35 3	232 5	60	1	1	NO
K	0 290	0 1270	39 1	307 7	35 3	278 1	60	1	1	NO
L+	#DIV/0!	#DIV/0!	0 0	#DIV/0!	0 0	#DIV/0!	60	#DIV/0!	#DIV/0!	#DIV/0!
L	#DIV/0!	#DIV/0!	0 0	#DIV/0!	0 0	#DIV/0!	60	#DIV/0!	#DIV/0!	#DIV/0!
M+	#DIV/0!	#DIV/0!	0 0	#DIV/0!	0 0	#DIV/0!	60	#DIV/0!	#DIV/0!	#DIV/0!
M	#DIV/0!	#DIV/0!	0 0	#DIV/0!	0 0	#DIV/0!	60	#DIV/0!	#DIV/0!	#DIV/0!
N+	#DIV/0!	#DIV/0!	0 0	#DIV/0!	0 0	#DIV/0!	60	#DIV/0!	#DIV/0!	#DIV/0!
N	#DIV/0!	#DIV/0!	0 0	#DIV/0!	0 0	#DIV/0!	60	#DIV/0!	#DIV/0!	#DIV/0!
O+	#DIV/0!	#DIV/0!	0 0	#DIV/0!	0 0	#DIV/0!	60	#DIV/0!	#DIV/0!	#DIV/0!
O	#DIV/0!	#DIV/0!	0 0	#DIV/0!	0 0	#DIV/0!	60	#DIV/0!	#DIV/0!	#DIV/0!
P+	#DIV/0!	#DIV/0!	0 0	#DIV/0!	0 0	#DIV/0!	60	#DIV/0!	#DIV/0!	#DIV/0!
P	#DIV/0!	#DIV/0!	0 0	#DIV/0!	0 0	#DIV/0!	60	#DIV/0!	#DIV/0!	#DIV/0!
Q+	#DIV/0!	#DIV/0!	0 0	#DIV/0!	0 0	#DIV/0!	60	#DIV/0!	#DIV/0!	#DIV/0!
Q	#DIV/0!	#DIV/0!	0 0	#DIV/0!	0 0	#DIV/0!	60	#DIV/0!	#DIV/0!	#DIV/0!
R+	#DIV/0!	#DIV/0!	0 0	#DIV/0!	0 0	#DIV/0!	60	#DIV/0!	#DIV/0!	#DIV/0!
R	#DIV/0!	#DIV/0!	0 0	#DIV/0!	0 0	#DIV/0!	60	#DIV/0!	#DIV/0!	#DIV/0!

FRAME RAIL ANAYLSIS

PANEL	PANEL ORIENT Vert = 1 Horz = 2	HEAD RAIL IN in	SIDE RAIL IN in	IN PLANE HEAD RAIL LOAD lb / in	IN PLANE SIDE RAIL LOAD lb / in	NO OF TOP RAIL CLIPS	TOP RAIL SPAN in	BOT RAIL SPAN in	SUPPORT SIDE RAIL SPAN in	NO OF SIDE RAIL CLIPS
J	2	138 75	35 315	16 1	16 1	10	15 0	34 7	16 0	2
K	2	117 25	35 315	17 7	15 8	9	14 2	39 1	16 0	2
L	1	0	0	#DIV/0!	#DIV/0!	1	#DIV/0!	0 0	16 0	1
M	1	0	0	#DIV/0!	#DIV/0!	1	#DIV/0!	0 0	16 0	1
N	1	0	0	#DIV/0!	#DIV/0!	1	#DIV/0!	0 0	16 0	1
O	1	0	0	#DIV/0!	#DIV/0!	1	#DIV/0!	0 0	16 0	1
P	1	0	0	#DIV/0!	#DIV/0!	1	#DIV/0!	0 0	16 0	1
Q	1	0	0	#DIV/0!	#DIV/0!	1	#DIV/0!	0 0	16 0	1
R	1	0	0	#DIV/0!	#DIV/0!	1	#DIV/0!	0 0	16 0	1

ARCHITECTURAL PANEL DESIGN CALCULATIONS FOR MERCURY METALS APOLIC PANELS

PROJECT	PROVIDENCE OFFICE PARK II - Addition	DATE	May 19 / 11
LOCATION	PORTLAND OREGON	PROJ #	25-28 98900
PANELS	PE CORE - ROOF PANELS FROM 30 TO 65 FEET ELEVATION	REV #	0

CHECK REINFORCEMENT "T STRESS & DEFLECTION

PANEL #	Sxx t&b #	REINFORCED PANEL SPAN in	REINFORCED PANEL SPACING IN	LOAD lb / in	MOMENT lb in	Sxx t&b in^3	BENDING STRESS	STRESS CRITERIA <33250psi	DEF spec in	DEFL/L	DEFL CRITRIA 60
J+	5	35 3	34 7	14 9	2325 3	0 5	4651	OK	0 066	534 4	OK
J		35 3	34 7	9 2	1433 9	0 2	7170	OK	0 041	851 3	OK
K+	5	35 3	39 1	16 8	2619 9	0 5	5240	OK	0 074	524 9	OK
K		35 3	39 1	10 4	1615 6	0 2	8078	OK	0 046	851 3	OK
L+	0	0 0	0 0	0 0	0 0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
L		0 0	0 0	0 0	0 0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
M+	0	0 0	0 0	0 0	0 0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
M		0 0	0 0	0 0	0 0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
N+	0	0 0	0 0	0 0	0 0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
N		0 0	0 0	0 0	0 0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
O+	0	0 0	0 0	0 0	0 0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
O		0 0	0 0	0 0	0 0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
P+	0	0 0	0 0	0 0	0 0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
P		0 0	0 0	0 0	0 0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Q+	0	0 0	0 0	0 0	0 0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Q		0 0	0 0	0 0	0 0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
R+	0	0 0	0 0	0 0	0 0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
R		0 0	0 0	0 0	0 0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

CHECK REINFORCED EXTERIOR FRAME STRESS (OUTWARD & INWARD)

PANEL #	TOP&BOT RAIL LENGTH in	REACTION LOADS IN / OUT lb / in	TOP&BOT RAIL MOMENT lb in	RAIL Sxx TOP Sxx BOT in^3	TOP&BOT RAIL STRESS psi	TOP&BOT RAIL <21000 psi	SIDERAILS LENGTH in	SIDERAILS MOMENT lb in	SIDERAILS STRESS psi	SIDERAILS <21000 psi
J+	15 0	7 67	214 9	0 135	2082 2	OK	16 0	245 4	2377 9	OK
J	15 0	4 73	132 5	0 1032	1284 0	OK	16 0	151 3	1466 4	OK
K+	14 2	7 67	192 1	0 135	1861 4	OK	16 0	245 4	2377 9	OK
K	14 2	4 73	118 5	0 1032	1147 9	OK	16 0	151 3	1466 4	OK
L+	#DIV/0!	#DIV/0!	#DIV/0!	0 135	#DIV/0!	#DIV/0!	16 0	#DIV/0!	#DIV/0!	#DIV/0!
L	#DIV/0!	#DIV/0!	#DIV/0!	0 1032	#DIV/0!	#DIV/0!	16 0	#DIV/0!	#DIV/0!	#DIV/0!
M+	#DIV/0!	#DIV/0!	#DIV/0!	0 135	#DIV/0!	#DIV/0!	16 0	#DIV/0!	#DIV/0!	#DIV/0!
M	#DIV/0!	#DIV/0!	#DIV/0!	0 1032	#DIV/0!	#DIV/0!	16 0	#DIV/0!	#DIV/0!	#DIV/0!
N+	#DIV/0!	#DIV/0!	#DIV/0!	0 135	#DIV/0!	#DIV/0!	16 0	#DIV/0!	#DIV/0!	#DIV/0!
N	#DIV/0!	#DIV/0!	#DIV/0!	0 1032	#DIV/0!	#DIV/0!	16 0	#DIV/0!	#DIV/0!	#DIV/0!
O+	#DIV/0!	#DIV/0!	#DIV/0!	0 135	#DIV/0!	#DIV/0!	16 0	#DIV/0!	#DIV/0!	#DIV/0!
O	#DIV/0!	#DIV/0!	#DIV/0!	0 1032	#DIV/0!	#DIV/0!	16 0	#DIV/0!	#DIV/0!	#DIV/0!
P+	#DIV/0!	#DIV/0!	#DIV/0!	0 135	#DIV/0!	#DIV/0!	16 0	#DIV/0!	#DIV/0!	#DIV/0!
P	#DIV/0!	#DIV/0!	#DIV/0!	0 1032	#DIV/0!	#DIV/0!	16 0	#DIV/0!	#DIV/0!	#DIV/0!
Q+	#DIV/0!	#DIV/0!	#DIV/0!	0 135	#DIV/0!	#DIV/0!	16 0	#DIV/0!	#DIV/0!	#DIV/0!
Q	#DIV/0!	#DIV/0!	#DIV/0!	0 1032	#DIV/0!	#DIV/0!	16 0	#DIV/0!	#DIV/0!	#DIV/0!
R+	#DIV/0!	#DIV/0!	#DIV/0!	0 135	#DIV/0!	#DIV/0!	16 0	#DIV/0!	#DIV/0!	#DIV/0!
R	#DIV/0!	#DIV/0!	#DIV/0!	0 1032	#DIV/0!	#DIV/0!	16 0	#DIV/0!	#DIV/0!	#DIV/0!

ARCHITECTURAL PANEL DESIGN CALCULATIONS FOR MERCURY METALS APOLIC PANELS

PROJECT	PROVIDENCE OFFICE PARK II - Addition	DATE	May 19 / 11
LOCATION	PORTLAND OREGON	PROJ #	25 28 98900
PANELS	PE CORE - ROOF PANELS FROM 30 TO 65 FEET ELEVATION	REV #	0

CHECK REINFORCED EXTERIOR FRAME DEFLECTION (OUTWARD & INWARD)

PANEL #	REACTION LOADS IN / OUT	RAIL Ixx 1	TOP&BOT DEFLECT	TOP&BOT DEFL/L	TOP&BOT DEFL CRITRIA	SIDERAIL DEFLECT	SIDERAIL DEFL/L	SIDERAIL DEFL CRITRIA
	lb / in	in^4	in	180 / L	L/180	in	180	L/180
J+	7 67	0 06087	0 0082	21836	OK	0 0108	16743	OK
J	4 73	0 06087	0 0051	35410	OK	0 0066	27151	OK
K+	7 67	0 06087	0 0066	27323	OK	0 0108	16743	OK
K	4 73	0 06087	0 0041	44307	OK	0 0066	27151	OK
L+	#DIV/0!	0 06087	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
L	#DIV/0!	0 06087	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
M+	#DIV/0!	0 06087	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
M	#DIV/0!	0 06087	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
N+	#DIV/0!	0 06087	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
N	#DIV/0!	0 06087	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
O+	#DIV/0!	0 06087	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
O	#DIV/0!	0 06087	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
P+	#DIV/0!	0 06087	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
P	#DIV/0!	0 06087	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Q+	#DIV/0!	0 06087	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Q	#DIV/0!	0 06087	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
R+	#DIV/0!	0 06087	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
R	#DIV/0!	0 06087	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

FINAL PANEL SUMMARY

PANEL #	HEAD & BOTTOM RAIL LENGTH	NO OF CLIPS ON EACH	HEAD & BOTTOM CLIP SPACING	SIDERAIL LENGTH	SIDERAIL CLIPS	SIDERAIL CLIP SPACING	RAIL SECTION	PANEL REINFORCE @ POINTS	REINFORCE SECTION	PANEL ORIENT'N
	in	#	in	in	#	in		1/2 1/3 1/4		
J	138 8	10	15 0	35 3	2	16 0	1	4	5	HORIZ
K	117 3	9	14 2	35 3	2	16 0	1	3	5	HORIZ
L	0 0	1	#DIV/0!	0 0	1	16 0	1	0	0	VERTICAL
M	0 0	1	#DIV/0!	0 0	1	16 0	1	0	0	VERTICAL
N	0 0	1	#DIV/0!	0 0	1	16 0	1	0	0	VERTICAL
O	0 0	1	#DIV/0!	0 0	1	16 0	1	0	0	VERTICAL
P	0 0	1	#DIV/0!	0 0	1	16 0	1	0	0	VERTICAL
Q	0 0	1	#DIV/0!	0 0	1	16 0	1	0	0	VERTICAL
R	0 0	1	#DIV/0!	0 0	1	16 0	1	0	0	VERTICAL

ARCHITECTURAL PANEL DESIGN CALCULATIONS FOR MERCURY METALS APOLIC PANELS

PROJECT	PROVIDENCE OFFICE PARK II - Addition	DATE	May 19 / 11
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PANELS	PE CORE - ROOF PANELS FROM 30 TO 65 FEET ELEVATION	REV #	0

PANEL ATTACHMENT TO SUBGIRTS / STUDS

PANEL	TYPE EXT 1 INT 2	VJ SGIRT SPACING IN	SUBGIRT THK IN	INTERIOR SCREW POSITION	SCREW# TOP&BOT	TOTAL SCREW # / PANEL	PULLOUT /SCREW LB	ALLOW PULLOUT 10	FASTEN OK	Pf SUBGIRT LB/FT
J	2	16	0 048	2 4	19 4	10 9	74 5	141 3	OK	67 9
K	2	16	0 048	2 4	16 6	9 5	72 2	141 3	OK	67 9
L	1	16	0 048	0	2	1 3	0 0	141 3	OK	67 9
M	2	16	0 048	0	2	1 0	0 0	141 3	OK	67 9
N	1	16	0 048	0	2	1 3	0 0	141 3	OK	67 9
O	1	16	0 048	0	2	1 3	0 0	141 3	OK	67 9
P	1	16	0 048	0	2	1 3	0 0	141 3	OK	67 9
Q	1	16	0 048	0	2	1 3	0 0	141 3	OK	67 9
R	2	16	0 048	0	2	1 0	0 0	141 3	OK	67 9

SUBGIRT ATTACHMENT TO STEEL STUDS

PANEL	V STUD SPACING IN	STUD THK IN	T SCREWS SUBGIRT TO STUDS	ALLOW Z SCREW LD LB	ALLOW PULLOUT 10	FASTENER PULLOUT OK	TOP SUBGIRT SHEAR LB / L FT	ALLOW SHEAR 10	FASTENER SHEAR OK
J	16	0	14 5	56 0	#N/A	#N/A	6 6	#N/A	#N/A
K	16	0	13 1	52 4	#N/A	#N/A	6 6	#N/A	#N/A
L	16	0	1	0 0	#N/A	#N/A	0 0	#N/A	#N/A
M	16	0	1	0 0	#N/A	#N/A	0 0	#N/A	#N/A
N	16	0	1	0 0	#N/A	#N/A	0 0	#N/A	#N/A
O	16	0	1	0 0	#N/A	#N/A	0 0	#N/A	#N/A
P	16	0	1	0 0	#N/A	#N/A	0 0	#N/A	#N/A
Q	16	0	1	0 0	#N/A	#N/A	0 0	#N/A	#N/A
R	16	0	1	0 0	#N/A	#N/A	0 0	#N/A	#N/A

ARCHITECTURAL PANEL DESIGN CALCULATIONS FOR MERCURY METALS APOLIC PANELS

PROJECT	PROVIDENCE OFFICE PARK II - Addition	DATE	May 19 / 11
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PANELS	PE CORE - ROOF PANELS FROM 30 TO 65 FEET ELEVATION	REV #	0

PRIMARY SUBGIRT ATTACHMENT TO CONCRETE WALLS

PANEL	INTERIOR SCREW POSITION	SCREW# TOP&BOT	PULLOUT /SCREW LB	SUBGIRT THK IN	SUBGIRT T&B PIN SPACING	PULLOUT /T&B PIN LB	ALLOW PULLOUT 25x1 25	TOP FASTENER OK	ALLOW SHEAR 25x1 25	TOP FASTENER OK
J	2 4	19 4	74 5	0	0	0 0	225 0	OK	299 0	OK
K	2 4	16 6	72 2	0	0	0 0	225 0	OK	299 0	OK
L	0 0	2	0 0	0	0	#DIV/0!	225 0	#DIV/0!	299 0	OK
M	0 0	2	0 0	0	0	#DIV/0!	225 0	#DIV/0!	299 0	OK
N	0 0	2	0 0	0	0	#DIV/0!	225 0	#DIV/0!	299 0	OK
O	0 0	2	0 0	0	0	#DIV/0!	225 0	#DIV/0!	299 0	OK
P	0 0	2	0 0	0	16	#DIV/0!	225 0	#DIV/0!	299 0	OK
Q	0 0	2	0 0	0	0	#DIV/0!	225 0	#DIV/0!	299 0	OK
R	0 0	2	0 0	0	0	#DIV/0!	225 0	#DIV/0!	299 0	OK

CHECK SUBGIRT STRESS AND DEFLECTION

PANEL	SUBGIRT SHAPE Z = 1 HAT = 2	SUBGIRT DEPTH d IN	SUBGIRT FLANGE b IN	SUBGIRT THK t IN	SUBGIRT Sxx IN^3	MAX M WIND LOAD LB IN	SUBGIRT WIND STRESS PSI	SUBGIRT BENDING Syy IN^3/LFT	DL / L FT LB /FT	SUBGIRT BENDING STRESS PSI
J	1	0	0	0 048	#DIV/0!	1629 2	#DIV/0!	0 0046	3 73	810 2
K	1	0	0	0 048	#DIV/0!	1629 2	#DIV/0!	0 0046	3 73	810 2
L	1	0	0	0 048	#DIV/0!	1629 2	#DIV/0!	0 0046	3 73	810 2
M	1	0	0	0 048	#DIV/0!	1629 2	#DIV/0!	0 0046	3 73	810 2
N	1	0	0	0 048	#DIV/0!	1629 2	#DIV/0!	0 0046	3 73	810 2
O	1	0	0	0 048	#DIV/0!	1629 2	#DIV/0!	0 0046	3 73	810 2
P	1	0	0	0 048	#DIV/0!	1629 2	#DIV/0!	0 0046	3 73	810 2
Q	1	0	0	0 048	#DIV/0!	1629 2	#DIV/0!	0 0046	3 73	810 2
R	1	0	0	0 048	#DIV/0!	1629 2	#DIV/0!	0 0046	3 73	810 2

ARCHITECTURAL PANEL DESIGN CALCULATIONS FOR MERCURY METALS APOLIC PANELS

PROJECT	PROVIDENCE OFFICE PARK II - Addition	DATE May 19 / 11
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PANELS	PE CORE - ROOF PANELS FROM 30 TO 65 FEET ELEVATION	REV # 0

SCREW PULLOUT CHART					SHEAR CHART		
SCREW	SUBGIRT THK	PULLOUT ULT	SAFTEY FACTOR	PULLOUT CALC	SHEAR ULT	SAFTEY FACTOR	SHEAR CALC
14	0 019	268	4	67 0	511	4	127 8
14	0 024	329	4	82 3	849	4	212 3
14	0 03	428	4	107 0	885	4	221 3
14	0 036	562	4	140 5	1244	4	311 0
14	0 048	800	4	200 0	1764	4	441 0
14	0 06	1151	4	287 8	2100	4	525 0
14	0 075	1152	4	288 0	2584	4	646 0
14	0 105	1858	4	464 5	2650	4	662 5
12	0 019	209	4	52 3	432	4	108 0
12	0 024	261	4	65 3	703	4	175 8
12	0 03	338	4	84 5	753	4	188 3
12	0 036	390	4	97 5	1018	4	254 5
12	0 048	649	4	162 3	1452	4	363 0
12	0 06	908	4	227 0	1758	4	439 5
12	0 075	1259	4	314 8	2138	4	534 5
12	0 105	1949	4	487 3	2048	4	512 0
10	0 019	148	4	37 0	312	4	78 0
10	0 024	241	4	60 3	478	4	119 5
10	0 03	311	4	77 8	589	4	147 3
10	0 036	357	4	89 3	830	4	207 5
10	0 048	565	4	141 3	1206	4	301 5
10	0 06	826	4	206 5	1268	4	317 0
10	0 075	1111	4	277 8	1552	4	388 0
10	0 105	1796	4	449 0		4	
8	0 019	156	4	39 0	294	4	73 5
8	0 024	193	4	48 3	496	4	124 0
8	0 03	265	4	66 3	560	4	140 0
8	0 036	298	4	74 5	740	4	185 0
8	0 048	491	4	122 8	1060	4	265 0
8	0 06	703	4	175 8	1078	4	269 5
8	0 075	959	4	239 8		4	
8	0 105	1558	4	389 5		4	

PINBOLT PULLOUT CHART							
PIN	CONC F c (KSI)	PULLOUT ULT (LB)	SAFTEY FACTOR	PULLOUT CALC (IB)	SHEAR ULT	SAFTEY FACTOR	SHEAR CALC
1875x 875	3 625	342	4	85 5	674	4	168 5
25x1	3 625	837	4	209 3	1196	4	299 0
25x1 25	3 625	900	4	225 0	1196	4	299 0
25x1 5	3 625	1035	4	258 8	1204	4	301 0

ARCHITECTURAL PANEL DESIGN CALCULATIONS FOR MERCURY METALS APOLIC PANELS

PROJECT	PROVIDENCE OFFICE PARK II - Addition	DATE May 19 / 11
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PANELS	PE CORE - ROOF PANELS FROM 30 TO 65 FEET ELEVATION	REV # 0

FOR VALUES OF β & γ FOR RECTANGULAR PLATES

a / b	EDGES SUPPORTED			EDGES FIXED			STRESS RATIOS
	β	γ	ε	β1	β2	ε	b / a
1	0 2874	0 0444	0 4200	0 3078	0 1386	0 0138	1 00
1 1	0 3318	0 0530	0 4375	0 3486	0 1590	0 0164	0 89
1 2	0 3762	0 0616	0 4550	0 3834	0 1794	0 0188	0 80
1 3	0 4158	0 0697	0 4665	0 4122	0 1944	0 0209	0 72
1 4	0 4530	0 0770	0 4780	0 4356	0 2094	0 0226	0 67
1 5	0 4542	0 0843	0 4845	0 4252	0 2190	0 0240	0 61
1 6	0 5172	0 0906	0 4910	0 4680	0 2286	0 0251	0 57
1 7	0 5448	0 0964	0 4950	0 4794	0 2346	0 0260	0 53
1 8	0 5688	0 1017	0 4990	0 4872	0 2406	0 0267	0 50
1 9	0 5910	0 1064	0 5010	0 4932	0 2439	0 0272	0 47
2 0	0 6102	0 1110	0 5030	0 4974	0 2472	0 0277	0 45
3 0	0 7134	0 1335	0 5050	0 5000	0 2500	0 0284	0 33
4 0	0 7410	0 1400	0 5020	0 5000	0 2500	0 0284	0 31
5 0	0 7476	0 1417	0 5010	0 5000	0 2500	0 0284	0 30
infinity	0 7500	0 1421	0 5000	0 5000	0 2500	0 0284	0 30

VALUES FOR T REINFORCEMENT				SHAPE
TYPE	Sxxt in^3	Sxxb in^3	Ixx in^4	
1	0 2019	0 0317	0 02826	T 7/8x7/8X 1
2	0 2869	0 0487	0 0482	T 1x1x 125
3	0 3921	0 1137	0 09096	SQ 875x 8751x 1
4	0 4077	0 1331	0 1036	SQ 875x 875x 125
5	0 5	0 2	0 2	SQ 1 x 1 x 125
6	0 5219	0 1943	0 1461	R 1 5x 875x 125
7	0 6075	0 2432	0 1992	R 2x 875x 125
8	1 2202	0 4831	0 5735	R 2x1 5x 125

VALUES FOR TOP BOTTOM & SIDE RAILS						
TYPE	Syyext in^3	Syyint in^3	Iyy in^4	Sxxext in^3	Sxxint in^3	Ixx in^4
1	0 14410	0 11880	0 10358	0 13500	0 10320	0 06087
2	0 1042	0 0846	0 06221	0 0948	0 097	0 05289
3	0 0317	0 0278	0 02359	0 0633	0 0836	0 0484

ALPOLIC THICKNESS & MODULUS								
Tap	Tap	Tpe	ts	Eap	Eap Tap^3	Sap	Tap^3 Tpe^3	I
mm	in	in	in	PSI	1bf in	in^3/in	in^3	in^4/in
3	0 1181	0 0781	0 0996	7110000	11700	0 00165	0 0011708	9 7569E 05
4	0 1575	0 1175	0 1204	5770000	22500	0 00242	0 0022848	0 0001904
6	0 2362	0 1962	0 1543	4220000	55600	0 00397	0 0056251	0 00046876

TOMLAND CONSULTING					PAGE 1 of 10
ARCHITECTURAL PANEL DESIGN CALCULATIONS FOR MERCURY METALS APOLIC PANELS					
PROJECT	PROVIDENCE OFFICE PARK II - Addition			DATE	May 19/ 11
LOCATION	PORTLAND OREGON			PROJ #	25 28 98900
PANELS	PE CORE - PANELS FROM 30 TO 75 FEET ELEVATION			REV #	0

DESIGN CRITERIA

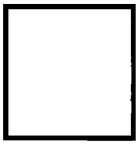
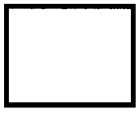
CANADA				USA			
CODE	BCBC 2006			CODE	OSSR 2007		
		KPA	PSF	BASE WIND SPEED	94 5	MPH	
BASE WIND PRESS	1/10	0	0 0	BASE PRESSURE	13 61	PSF	
	1/50	0	0 0	EXPOSURE	B	*	
				IMPORTANCE	1		
PANEL ELEV MIN	0	M // FT	0 0	PANEL ELEV MIN	30	FT	
PANEL ELEV MAX	0	M // FT	0 0	PANEL ELEV MAX	75	FT	

CALCULATED MAXIMUM WIND PRESSURES

PRESSURE	CAN SPEC (Pa)		CAN FACT (Pf)		USA SPEC (Pa)		USA FACT (Pf)		USE FOR CALC Pa	USE FOR CALC Pf
DIRECTION	Kpa	PSF	Kpa	PSF	PSF	Kpa	PSF	Kpa	PSF	PSF
INWARD	0 00	0 0	0 000	0 0	15 91	0 76	25 5	1 22	15 9	25 5
OUTWARD	0 00	0 0	0 000	0 0	31 82	1 52	50 9	2 44	31 8	50 9

CAN PHI FACTOR FOR DEFLECTION	0 75	RAIN SCREEN PANEL FACTOR	0 75
MAXIMUM DEFLECTION OF PANEL	60	USA PHI FACTOR FOR WIND DEFLECTION	0 70
MAXIMUM DEFLECTION OF FRAME	180		

PANEL DESIGNATION

PANEL SHAPE	EDGE CONDIT N [CODE]	LOADING CRITERIA	MAXIMUM STRESS ksi	MAXIMUM DEFLECTION in
SQUARE [1]  b / b	SIMPLE [1]	Bending	0 2874 (Wb/ts^2)	4 43 (Wb b^2 / t^3)*10^ 6
		Membrane at center	6*(Wm/(b ts))^ 6666	0 013 (Wm b^2 / t)^ 3333
		Summary	Wm = 39571*10^ 10 (b^4/t^8)*Wb^3	
		Chk Δ max		0443 Pa b^4/(Eap*Tap^3)/12^2
RECTANGULAR [2]  b / a WHERE $\alpha = b / a$ a > b	SIMPLE [1]	Bending σb max at center	0 75 (Wb b)/(a ts^2 (1+1 61 α^3))	14 (Wb*b^3)/(a t^3) 10^ 6
		σa at center	Wb b (0 225+ 382α^2 320α^3)/ ts^2	
		Membrane	8 (Wm/(a ts))^ 6666	0 016 ((Wm b^3)/(a t))^ 3333
		Summary	Wm = 6 66992 10^ 10 (b^6/(a^2 t^8))*Wb^3	
		Chk Δ max		γ Pa b^4/(Eap Tap^3)/12^2

THEORY		
	1)	W total = W bending + W membrane
	2)	Deflection in bending = Deflection in membrane