shawn pierce engineering

Project Number:	**09.9-2/2**
Project Description:	**Melfred Borzall Expansion/Remodel**
Project Location:	**2712 Airpark Drive, Santa Maria, CA**
Client:	**Madjedi Design Management**
Date:	**April 28, 2010**

This calculation set is reflective of an expansion and remodel at the noted project location. All existing information has been provided to this firm by others and not verified by this firm. Only the portions of the expansion and remodel addressed in this calculation set have been designed by this firm.

This calc set replaces the calc set previously provided by this firm dated February 8, 2010

The signing engineer is only responsible for building systems reviewed directly by him, as outlined within these structural calculations.

The engineer shall not be responsible for errors and omissions in the project not in conformance with these calculations and the 2007 CBC.

The engineer accepts no responsibility for field inspections during construction, or for the method or form of construction.

It is the responsibility of the contractor to point out any points of conflict to the engineer that will not allow for the structure to be built as per the specs and drawings.

This structure has been designed only for the loads within these calculations. Any additional loads or discrepancies shall be brought to the engineer's immediate attention.

It is the goal of the lateral calculations contained herein to provide the building with sufficient strength to resist a minor earthquake without damage, a moderate earthquake without structural damage, and a major earthquake without collapse. An "earthquake proof" building is neither feasible nor required by the Code.

These calculations are valid only for the above referenced project and location. Any use of these calculations on any other project with the written permission of the engineer is strictly prohibited and the engineer accepts no responsibility for any other projects associated with these calculations other than the above referenced project.

phone: 805.801.9385 | 277 N. Beechnut Ave., Nipomo, CA 93444 | e-mail: pierce.engineering@gmail.com

SHEAR WALL SCHEDULE

MARK	SHEATHING[8,9]	NAILING[1,2,7]		RIM BOARD/BLOCKING TRANSFER TO DOUBLE TOP PLATE[1,2,3]	SOLE PLATE CONNECTION TO RIM BOARD/BLOCKING[10,12]			5/8" DIA. ANCHOR BOLTS W/ 3"SQx0.229" PLATE WASHERS[6], UNO		CAPACITY	
		EDGE	FIELD		SOLE PLATE[11]	CONNECTOR	RIM BOARD OR BLK'G (MIN)	SILL PLATE (MIN)	ANCHOR BOLT SPACING	SEISMIC (plf)	WIND (plf)
1	3/8" CDX PLYWOOD (24/0), APPLIED TO ONE SIDE, BLOCK ALL EDGES	8^d AT 6"o.c.	8^d AT 12"o.c.	RBC (ROOF BOUNDARY CLIP) AT 24"o.c.; A35 AT 24"o.c.; OR LTP4 AT 24"o.c.	2x	16^d AT 6"o.c.[1,2]	2x DF OR 1 3/4" LSL	2x PT DF	60"o.c.	260	365
2	3/8" CDX PLYWOOD (24/0), APPLIED TO ONE SIDE, BLOCK ALL EDGES	8^d AT 4"o.c.	8^d AT 12"o.c.	RBC (ROOF BOUNDARY CLIP) AT 24"o.c.; A35 AT 24"o.c.; OR LTP4 AT 24"o.c.	2x	16^d AT 4"o.c.[1,2] STAGGERED	2x DF OR 1 3/4" LSL	2x PT DF	45"o.c.	350	530
3	3/8" CDX PLYWOOD (24/0), APPLIED TO ONE SIDE, BLOCK ALL EDGES	8^d AT 3"o.c.	8^d AT 12"o.c.	RBC (ROOF BOUNDARY CLIP) AT 24"o.c.; A35 AT 24"o.c.; OR LTP4 AT 24"o.c.	2x	SDS25412 SCREWS AT 12"o.c.	3x DF OR 3 1/2" LSL	2x PT DF 3x PT DF	15"o.c. 30"o.c.	490	685
4	3/8" CDX PLYWOOD (24/0), APPLIED TO ONE SIDE, BLOCK ALL EDGES	8^d AT 2"o.c.	8^d AT 12"o.c.	RBC (ROOF BOUNDARY CLIP) AT 24"o.c.; A35 AT 24"o.c.; OR LTP4 AT 24"o.c.	2x	SDS25412 SCREWS AT 9"o.c.	3x DF OR 3 1/2" LSL	2x PT DF 3x PT DF	12"o.c. 24"o.c.	600	895
5	3/8" CDX PLYWOOD (24/0), APPLIED TO BOTH SIDES, BLOCK ALL EDGES[4,5]	8^d AT 4"o.c.	8^d AT 12"o.c.	RBC (ROOF BOUNDARY CLIP) AT 24"o.c.; A35 AT 24"o.c.; OR LTP4 AT 24"o.c.	3x	SDS25600 SCREWS AT 8"o.c.	3x DF OR 3 1/2" LSL	3x PT DF	20"o.c.	760	1060
6	3/8" CDX PLYWOOD (24/0), APPLIED TO BOTH SIDES, BLOCK ALL EDGES[4,5]	8^d AT 3"o.c.	8^d AT 12"o.c.	RBC (ROOF BOUNDARY CLIP) AT 24"o.c.; A35 AT 24"o.c.; OR LTP4 AT 24"o.c.	3x	SDS25600 SCREWS AT 6"o.c.	3x DF OR 3 1/2" LSL	3x PT DF	16"o.c.	980	1370

NOTES TO SHEAR WALL SCHEDULE

1) ALL NAILS TO BE COMMON:
   8^d (0.131" x 2 1/2"), 10^d (0.148" x 3"), 16^d (0.162" x 3 1/2")
2) GALVANIZED NAILS SHALL BE HOT DIPPED OR TUMBLED
3) RIM BOARD/BLOCKING TRANSFERS NOTED SHALL BE FULL LENTH OF WALL LINE, NOT JUST OVER THE SHEAR WALL
4) OFFSET VERTICAL JOINTS BY ONE STUD SPACE (MIN) ON OPPOSITE FACE OF WALL, PANEL END STUDS TO BE 3x (MIN)
5) FOUNDATION SILL PLATES AND ALL FRAMING MEMBERS AT ADJOINING PANEL EDGES SHALL BE 3" NOMINAL OR WIDER AND NAILS SHALL BE STAGGERED IN ALL CASES
6) THE HOLE IN THE PLATE WASHER IS PERMITTED TO BE DIAGONALLY SLOTTED WITH A WIDTH OF 13/16" AND A LENGTH OF 1 3/4", PROVIDED A STANDARD CUT WASHER IS PLACED BETWEEN THE PLATE WASHER AND THE NUT
7) ALL NAILING SHALL BE FINISHED OFF BY HAND HAMMERING TO ENSURE NAILS ARE DRIVEN FLUSH TO FACE OF PLWOOD. OVERDRIVEN NAILING MAY REQUIRE ENTIRE PANEL TO BE REMOVED AND REPLACED UPON OBSERVATION
8) 3/8" APA-RATED OSB SHEATHING (SPAN RATING = 24/0) MAY SUBSTITUTE CDX PLYWOOD
9) PANELS MAY BE INSTALLED EITHER HORIZONTALLY OR VERTICALLY, PROVIDED THE WALL FRAMING IS SPACED NO GREATER THAN 16"o.c.
10) MAXIMUM FLOOR SHEATHING THICKNESS IS 1"
11) PANEL EDGES SHALL NOT ADJOIN AT SOLE PLATE
12) NAILING ON NARROW FACE SHALL EXCEED SPACING SHOWN BELOW

NAIL SIZE	CLOSEST ON CENTER SPACING PER ROW			
	1 3/4" TIMBERSTRAND LSL	3 1/2" TIMBERSTRAND LSL	MICROLLAM LVL	PARALLAM PSL
8^d (0.131" x 2 1/2")	3"	3"	3"	3"
10^d (0.148" x 3")	4"	3"	4"	4"
16^d (0.162" x 3 1/2")	6"	3 1/2"	8"	6"

* IF MORE THAN ONE ROW OF NAILS IS USED, THE ROWS MUST BE OFFSET AT LEAST 1/2" AND STAGGERED

FRAMING ANALYSIS

277 N. Beechnut Ave
Nipomo, CA 93444
Phone No.: (805) 801-9385
Email: pierce.engineering@gmail.com

Building Unit Weights

Material	Main Floor (psf)	Int. Walls (psf)	Ceiling (psf)
Plywood			
Floor (1-1/8" Plywood)	3.5	---	---
Walls (3/8" Plywood)	---	1.2	---
Framing Members			
Floor (18" I-Joists (Max) at 16"o.c.)	3.2	---	---
Walls (2x6 at 16"o.c.)	---	4.0	---
Ceiling (2x6 at 16"o.c., where applicable)	---	---	4.0
Ceiling			
Dropped Panel Ceiling	2.0	---	---
5/8" Gyp Board	---	---	2.8
Finishes			
Carpet or 1/4" Tile	2.0	---	---
1/2" Gyp Board	---	4.4	---
Insulation	1.0	0.3	0.3
Mechanical/Electrical/Plumbing	1.0	0.1	0.2
Miscellaneous	1.3	---	0.2
Dead Loads	14.0	10.0	7.5
Live Load	50.0	---	10.0

* All Dead Loads have been derived from either the ASCE 7-05 "Minimum Design Dead Loads" Table C3-1, or per the manufacturer specs
* All Live Loads have been derived from the ASCE 7-05 "Minimum Uniformly Distributed Live Loads..." Table 4-1
* Live Load has been reduced for the Roof Framing per ASCE 7-05, Section 4.9

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZAL EXPANSION 09.7-2/2
SHEET NO. ____ OF ____
CALCULATED BY ____ DATE 4/10
CHECKED BY ____ DATE ____
SCALE ____

SOFFIT FRAMING & DESIGN

✓ OUT-OF-PLANE LOADING AT EXTERIOR FACE (SHOP FACE)

SOFFIT HEIGHT ≤ 5'-0"
PURLIN SPACING = 5'-4"
OUT-OF-PLANE LOADING = 5 psf

LOAD TO EA. PURLIN = 5 psf (5/2)(5'-4") = 70#/PURLIN

USING #10 'TAPFAST' SCREWS & ASSUMING THE FOLLOWING
(E) PURLINS ARE MIN 20GA
#10 SCREW DIAMETER = 0.190
USE 0.164"Ø WOOD SCREW
MIN EMBED = 8 (0.190) = 1.5"
Z = 89#/SCREW

USE (2) #10 TAPFAST SCREWS AT PURLIN TO DBL TOP R

✓ SOFFIT SUPPORT BM (OPTION) ℓ = 22'-0"

	WALL
DL	7 psf
LL	—
TRIB	5'

R_D = 385 + SELF WT.

USE: DBL 2x12 DF #2 (MIN)
SEE ENERCALC RESULTS

✓ SOFFIT SUPPORT BM (OPTION) ℓ = 17'-6"

	WALL
DL	7 psf
LL	—
TRIB	3'3"

R_D = 200 + SELF WT.

USE: DBL 2x8 DF #2 (MIN)
SEE ENERCALC RESULTS

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Multiple Simple Beam Design

Description : Soffit Support Beams

Wood Beam Design : Soffit Support Beam - L = 22'

BEAM Size : **2-2x10, Sawn, Fully Unbraced**
Using Allowable Stress Design with 2006 IBC & ASCE 7-05 Load Combinations, Major Axis Ben

Calculations per IBC 2006, CBC 2007, 2005 NDS

Wood Species : Douglas Fir - Larch
Wood Grade : No.2

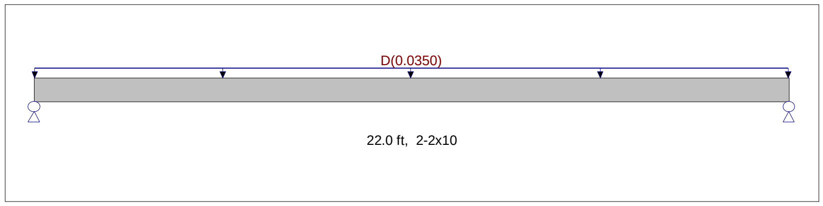
Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	32.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.0070 k/ft, Trib= 5.0 ft

Design Summary

Max fb/Fb Ratio = **0.727** : 1
fb : Actual : 699.29 psi at 11.000 ft in Span # 1
Fb : Allowable : 961.53 psi
Load Comb : +D
Max fv/FvRatio = **0.127** : 1
fv : Actual : 22.87 psi at 0.000 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +D



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.45						
Right Support	0.45						

Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.690 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	382

Description : Soffit Support Beams

Wood Beam Design : Soffit Beam - L = 17'-6"

BEAM Size : **2-2x8, Sawn, Fully Unbraced**
Using Allowable Stress Design with 2006 IBC & ASCE 7-05 Load Combinations, Major Axis Ben

Calculations per IBC 2006, CBC 2007, 2005 NDS

Wood Species : Douglas Fir - Larch
Wood Grade : No.2

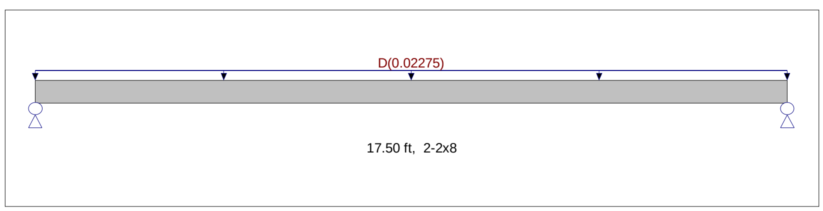
Fb - Tension	900 psi	Fc - Prll	1350 psi	Fv	180 psi	Ebend- xx	1600 ksi	Density	32.21 pcf
Fb - Compr	900 psi	Fc - Perp	625 psi	Ft	575 psi	Eminbend - xx	580 ksi		

Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.0070 k/ft, Trib= 3.250 ft

Design Summary

Max fb/Fb Ratio = **0.455** : 1
fb : Actual : 482.69 psi at 8.750 ft in Span # 1
Fb : Allowable : 1,061.73 psi
Load Comb : +D
Max fv/FvRatio = **0.086** : 1
fv : Actual : 15.55 psi at 0.000 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +D



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.24						
Right Support	0.24						

Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.384 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	546

FRAMING ANALYSIS

Office Expansion

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZAL EXPANSION 09.9-2/2
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 11/09
CHECKED BY ______ DATE ______
SCALE OFFICE EXPANSION

TYP. FLOOR JOISTS ℓ_{max} = 23'-0"

	FLOOR
DL	14psf
LL	50psf (OFFICE)

USE: 18" RED-I65 @ 16"o.c. w/ MIU 2.56/18 HGRS WHERE APPLICABLE

SEE 'TJ-BEAM' RESULTS

FB-1 TYP. FLOOR HDR @ OFFICE ℓ = 4'-6"

	FLOOR	WALL
DL	14psf	9psf
LL	50psf	—
TRIB	11'-6"	12.5'

R_D = 620#
R_L = 1295#
R_{TL} = 1915#

USE: 6x10 DF#2 (MIN) w/ 2x6 DF#2 TRIMMER (MIN)

SEE ENERCALC RESULTS

FB-2 FLOOR HDR ℓ = 9'-6" - LINE (4.9)

	FLOOR	WALL
DL	14psf	9psf
LL	50psf	—
TRIB	11'-6"	12.5'

R_D = 1300#
R_L = 2735#
R_{TL} = 4035#

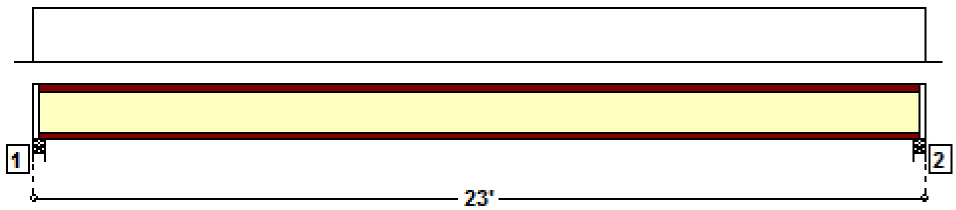
USE: TRIPLE 1 3/4" x 11 7/8" 1.9E LVL w/ 4x6 DF#2 POSTS @ EA. END & TYP. CONTINUOUS FTG

SEE ENERCALC RESULTS

Typical Floor Joist = 18" Red-I65 at 16" o.c.

18" TJI®/L65 @ 16" o/c

THIS PRODUCT MEETS OR EXCEEDS THE SET DESIGN CONTROLS FOR THE APPLICATION AND LOADS LISTED



Product Diagram is Conceptual.

LOADS:

Analysis is for a Joist Member.

Primary Load Group - Office Bldgs - Offices (psf): 50.0 Live at 100 % duration, 14.0 Dead, 20.0 Partition

SUPPORTS:

		Input Width	Bearing Length	Vertical Reactions (lbs) Live/Dead/Uplift/Total	Detail	Other
1	Stud wall	3.50"	2.25"	767 / 521 / 0 / 1288	End, Rim	1 Ply 1 1/4" x 18" 1.3E TimberStrand® LSL
2	Stud wall	3.50"	2.25"	767 / 521 / 0 / 1288	End, Rim	1 Ply 1 1/4" x 18" 1.3E TimberStrand® LSL

DESIGN CONTROLS:

	Maximum	Design	Control	Result	Location
Shear (lbs)	1265	-1255	2535	Passed (50%)	Rt. end Span 1 under Floor loading
Vertical Reaction (lbs)	1265	1265	1535	Passed (82%)	Bearing 2 under Floor loading
Moment (Ft-Lbs)	7140	7140	10380	Passed (69%)	MID Span 1 under Floor loading
Live Load Defl (in)		0.323	0.452	Passed (L/839)	MID Span 1 under Floor loading
Total Load Defl (in)		0.543	1.129	Passed (L/499)	MID Span 1 under Floor loading
TJPro		47	45	Passed	Span 1

-Deflection Criteria: STANDARD(LL:L/600,TL:L/240).

-Deflection analysis is based on composite action with single layer of 23/32" Panels (24" Span Rating) GLUED & NAILED wood decking.

-Bracing(Lu): All compression edges (top and bottom) must be braced at 4' 5" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.

-2000 lbs concentrated load requirements for standard non-residential floors have been considered for reaction and shear.

TJ-Pro RATING SYSTEM

-The TJ-Pro Rating System value provides additional floor performance information and is based on a GLUED & NAILED 23/32" Panels (24" Span Rating) decking. The controlling span is supported by walls. Additional considerations for this rating include: Ceiling - Suspended Ceiling. A structural analysis of the deck has not been performed by the program. Comparison Value: 2.2

ADDITIONAL NOTES:

-IMPORTANT! The analysis presented is output from software developed by iLevel®. Allowable product values shown are in accordance with current iLevel® materials and code accepted design values. The specific product application, input design loads and stated dimensions have been provided by others (____________________________), have not been checked for conformance with the design drawings of the building, and have not been reviewed by iLevel® Engineering.

-THIS ANALYSIS FOR iLevel® PRODUCTS ONLY! PRODUCT SUBSTITUTION VOIDS THIS ANALYSIS.

-Allowable Stress Design methodology was used for Building Code IBC analyzing the iLevel® Custom product listed above.

PROJECT INFORMATION:

Borzall Expansion
Santa Maria, CA

OPERATOR INFORMATION:

Michael Hayley
RedBuilt, LLC
3189 East Highway 46
Paso Robles, CA 93446
Phone : (805) 227-6633
Fax : (805) 227-6633
mhayley@redbuilt.com

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Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Wood Beam Design

Description : FB-1 - Typ Floor Header at Office

Material Properties

Calculations per IBC 2006, CBC 2007, 2005 NDS

Analysis Method : Allowable Stress Design
Load Combination 2006 IBC & ASCE 7-05

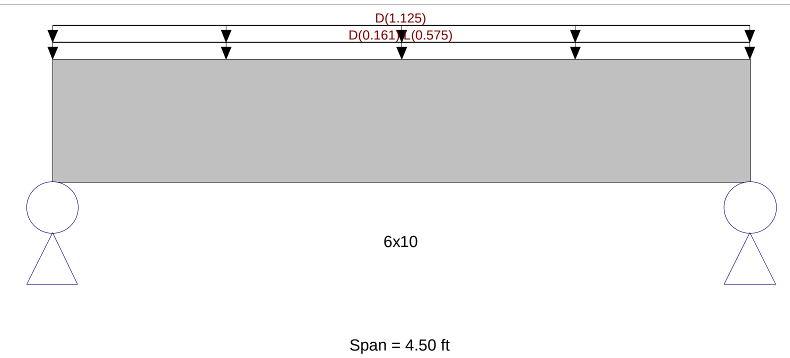
Wood Species : Douglas Fir - Larch
Wood Grade : No.2

Beam Bracing : Completely Unbraced

Fb - Tension	875.0	psi
Fb - Compr	875.0	psi
Fc - Prll	600.0	psi
Fc - Perp	625.0	psi
Fv	170.0	psi
Ft	425.0	psi

E : Modulus of Elasticity

Ebend- xx	1,300.0	ksi
Eminbend - xx	470.0	ksi
Density	32.210	pcf



Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.0140, L = 0.050 ksf, Tributary Width = 11.50 ft
Uniform Load : D = 0.090 ksf, Tributary Width = 12.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.787**: 1	Maximum Shear Stress Ratio	=	**0.463** : 1
Section used for this span		**6x10**	Section used for this span		**6x10**
fb : Actual	=	687.58 psi	fv : Actual	=	78.63 psi
FB : Allowable	=	873.83 psi	Fv : Allowable	=	170.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	2.250 ft	Location of maximum on span	=	3.713 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.010 in	Ratio =	5158
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.034 in	Ratio =	1583
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios			Summary of Moment Values			Summary of Shear Values		
Segment Length	Span #	M	V	C_d	Mactual	fb-design	Fb-allow	Vactual	fv-design	Fv-allow
+D										
Length = 4.50 ft	1	0.545	0.320	1.000	3.28	476.46	873.83	1.90	54.48	170.00
+D+L+H										
Length = 4.50 ft	1	0.787	0.463	1.000	4.74	687.58	873.83	2.74	78.63	170.00
+D+Lr+H										
Length = 4.50 ft	1	0.545	0.320	1.000	3.28	476.46	873.83	1.90	54.48	170.00
+D+0.750Lr+0.750L+H										
Length = 4.50 ft	1	0.726	0.427	1.000	4.38	634.80	873.83	2.53	72.59	170.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+Lr	1	0.0341	2.273		0.0000	0.000

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Printed: 17 NOV 2009, 1:46PM

Wood Beam Design

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ENERCALC, INC. 1983-2009, Ver: 6.0.24, N:40336

Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : FB-2 - Floor Header at Office - L=9'-6"

Material Properties

Calculations per IBC 2006, CBC 2007, 2005 NDS

Analysis Method : Allowable Stress Design
Load Combination 2006 IBC & ASCE 7-05

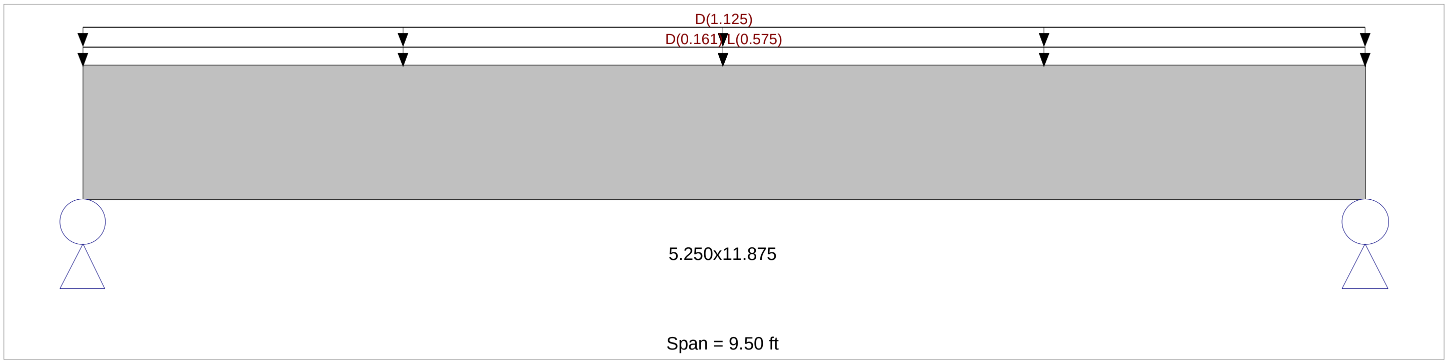
Wood Species : iLevel Truss Joist
Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Completely Unbraced

Fb - Tension	2,600.0	psi
Fb - Compr	2,600.0	psi
Fc - Prll	2,510.0	psi
Fc - Perp	750.0	psi
Fv	285.0	psi
Ft	1,555.0	psi

E : Modulus of Elasticity

Ebend- xx	1,900.0	ksi
Eminbend - xx	1,900.0	ksi
Density	32.210	pcf



Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.0140, L = 0.050 ksf, Tributary Width = 11.50 ft
Uniform Load : D = 0.090 ksf, Tributary Width = 12.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.796 : 1	Maximum Shear Stress Ratio	=	0.601 : 1
Section used for this span		5.250x11.875	Section used for this span		5.250x11.875
fb : Actual	=	2,057.08 psi	fv : Actual	=	171.42 psi
FB : Allowable	=	2,584.11 psi	Fv : Allowable	=	285.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	4.750 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.076 in	Ratio =	1493
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.249 in	Ratio =	458
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios			Summary of Moment Values			Summary of Shear Values		
Segment Length	Span #	M	V	C_d	Mactual	fb-design	Fb-allow	Vactual	fv-design	Fv-allow
+D										
Length = 9.50 ft	1	0.552	0.417	1.000	14.67	1,426.23	2,584.11	4.94	118.85	285.00
+D+L+H										
Length = 9.50 ft	1	0.796	0.601	1.000	21.15	2,057.08	2,584.11	7.12	171.42	285.00
+D+Lr+H										
Length = 9.50 ft	1	0.552	0.417	1.000	14.67	1,426.23	2,584.11	4.94	118.85	285.00
+D+0.750Lr+0.750L+H										
Length = 9.50 ft	1	0.735	0.555	1.000	19.53	1,899.37	2,584.11	6.58	158.28	285.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+Lr	1	0.2488	4.798		0.0000	0.000

Wood Column

Description : FB-2 - Support Post

General Information

Code Ref : 2006 IBC, ANSI / AF&PA NDS-2005

Analysis Method : Allowable Stress Design
End Fixities : Top & Bottom Pinned
Overall Column Height 12.50 ft
(Used for non-slender calculations)
Wood Species Douglas Fir - Larch
Wood Grade No.2

Fb - Tension	900.0 psi	Fv	180.0 psi
Fb - Compr	900.0 psi	Ft	575.0 psi
Fc - Prll	1,350.0 psi	Density	32.210 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Load Combination 2006 IBC & ASCE 7-05

Wood Section Name 4x6
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	3.50 in
Exact Depth	5.250 in
Area	19.25 in^2
Ix	266.93 in^4
Iy	68.78 in^4

Allowable Stress Modification Factors

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

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 12.5ft, K = 1.0
Y-Y (depth) axis :Unbraced Length for Y-Y Axis buckling = 12.5 ft, K = 1.0

Applied Loads

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

Column self weight included : 53.823 lbs * Dead Load Factor
AXIAL LOADS . . .
FB-2 Reaction: Axial Load at 12.50 ft, D = 1.30, L = 2.735 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.8894** : 1
Load Combination	+D+L+H
Governing NDS Formla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are . . .	
Applied Axial	4.089 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	250.20 psi
PASS Maximum Shear Stress Ratio =	**0.0** : 1
Load Combination	+D+0.750Lr+0.750L+H
Location of max.above base	12.50 ft
Applied Design Shear	0.0 psi
Allowable Shear	180.0 psi

Maximum SERVICE Lateral Load Reactions . .

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

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension
Cf or Cv : Size based factors	1.000	1.000	

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D	0.2945	PASS	0.0 ft	0.0	PASS	12.50 ft
+D+L+H	0.8894	PASS	0.0 ft	0.0	PASS	12.50 ft
+D+Lr+H	0.2945	PASS	0.0 ft	0.0	PASS	12.50 ft
+D+0.750Lr+0.750L+H	0.7407	PASS	0.0 ft	0.0	PASS	12.50 ft

Maximum Reactions - Unfactored

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top
D Only				
L Only				

Maximum Deflections for Load Combinations - Unfactored Loads

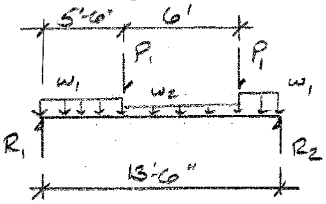
Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZALL EXPANSION 09.9.2/2
SHEET NO.
OF
CALCULATED BY
DATE 11/09
CHECKED BY
DATE
SCALE OFFICE EXPANSION

FB-3 FLOOR HDR l = 13'-6" - LINE (N.9)

	FLOOR	WALL
DL	14 psf	9 psf
LL	50 psf	—
TRIB	11'-6"/3'	12'-6"



P_{1D} = 940#
P_{1L} = 1980#
FB-6

USE: 5 1/4" x 14" 2.0E PSL (MIN)
w/ 6x6 DF#2 POST @ EA. END
& TYP. CONT. FTG

SEE ENERCALC RESULTS

R_{1D} = 2390# R_{2D} = 2690#
R_{1L} = 4400# R_{2L} = 4770#
R_{1TL} = 6790# R_{2TL} = 7460#

FB-4 FLOOR BEAM l = 9'-6" LINE (O)

	FLOOR
DL	14 psf
LL	50 psf
TRIB	14'

R_D = 990#
R_L = 3330#
R_{TL} = 4320#

USE: 5 1/4" x 11 1/4" 2.0E PSL (MIN)
w/ 4x6 DF#2 POSTS
& TYP. CONT. FTG

SEE ENERCALC RESULTS

D PRODUCT 207

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Printed: 6 JAN 2010, 8:01AM

Wood Beam Design

Y:\@Shawn Pierce Engineering\09_Projects\Borzal Addition_02\Engineering\Design\melford borzall expansion.ec6
ENERCALC, INC. 1983-2009, Ver: 6.1.01, N:40336

Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : FB-3 - Floor Header at Office - L=13'-6"

Material Properties

Calculations per IBC 2006, CBC 2007, 2005 NDS

Analysis Method : Allowable Stress Design
Load Combination 2006 IBC & ASCE 7-05

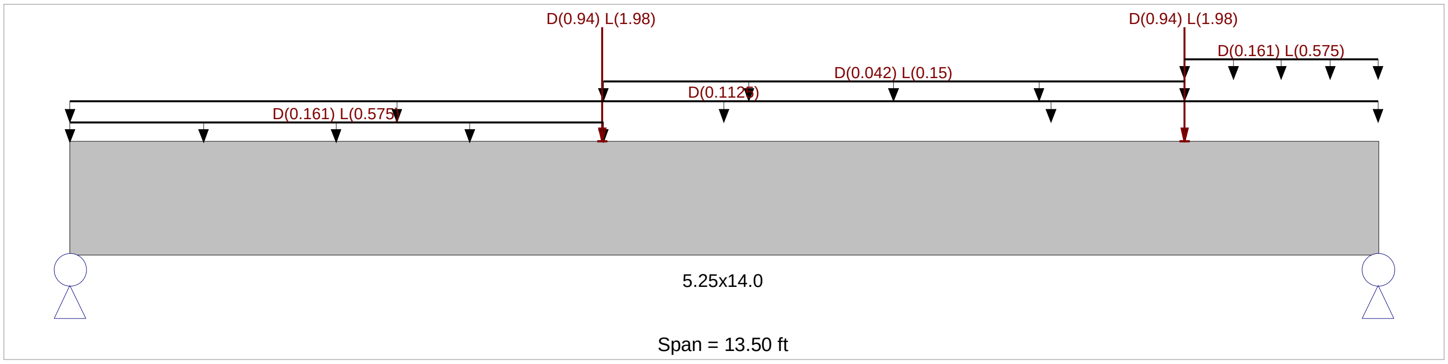
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

Beam Bracing : Completely Unbraced

Fb - Tension	2,900.0	psi
Fb - Compr	2,900.0	psi
Fc - Prll	2,900.0	psi
Fc - Perp	750.0	psi
Fv	290.0	psi
Ft	2,025.0	psi

E : Modulus of Elasticity

Ebend- xx	2,000.0	ksi
Eminbend - xx	2,000.0	ksi
Density	32.210	pcf



Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.0140, L = 0.050 ksf, Extent = 0.0 -->> 5.50 ft, Tributary Width = 11.50 ft
Uniform Load : D = 0.0090 ksf, Tributary Width = 12.50 ft
Uniform Load : D = 0.0140, L = 0.050 ksf, Extent = 5.50 -->> 11.50 ft, Tributary Width = 3.0 ft
Uniform Load : D = 0.0140, L = 0.050 ksf, Extent = 11.50 -->> 13.50 ft, Tributary Width = 11.50 ft
Point Load : D = 0.940, L = 1.980 k @ 5.50 ft, (FB-6)
Point Load : D = 0.940, L = 1.980 k @ 11.50 ft, (FB-6)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.592** 1	Maximum Shear Stress Ratio	=	**0.455** : 1
Section used for this span		**5.25x14.0**	Section used for this span		**5.25x14.0**
fb : Actual	=	1,696.44 psi	fv : Actual	=	132.00 psi
FB : Allowable	=	2,865.66 psi	Fv : Allowable	=	290.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	5.535 ft	Location of maximum on span	=	12.353 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.203 in	Ratio =	797
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.320 in	Ratio =	505
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios			Summary of Moment Values			Summary of Shear Values		
Segment Length	Span #	M	V	C_d	Mactual	fb-design	Fb-allow	Vactual	fv-design	Fv-allow
+D										
Length = 13.50 ft	1	0.214	0.166	1.000	8.75	612.15	2,865.66	2.36	48.13	290.00
+D+L+H										
Length = 13.50 ft	1	0.592	0.455	1.000	24.24	1,696.44	2,865.66	6.47	132.00	290.00
+D+Lr+H										
Length = 13.50 ft	1	0.214	0.166	1.000	8.75	612.15	2,865.66	2.36	48.13	290.00
+D+0.750Lr+0.750L+H										
Length = 13.50 ft	1	0.497	0.383	1.000	20.37	1,425.37	2,865.66	5.44	111.03	290.00

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Printed: 6 JAN 2010, 8:01AM

Wood Beam Design

Y:\@Shawn Pierce Engineering\09_Projects\Borzal Addition_02\Engineering\Design\melford borzall expansion.ec6
ENERCALC, INC. 1983-2009, Ver: 6.1.01, N:40336

Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : FB-3 - Floor Header at Office - L=13'-6"

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+Lr	1	0.3203	6.683		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	6.792	7.460
D Only	2.389	2.691
L Only	4.403	4.769
D+L+S	6.792	7.460
D+L+Lr	6.792	7.460

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Printed: 6 JAN 2010, 8:04AM

Wood Column

Y:\@Shawn Pierce Engineering\09_Projects\Borzal Addition_02\Engineering\Design\melford borzall expansion.ec6
ENERCALC, INC. 1983-2009, Ver: 6.1.01, N:40336

Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : FB-3 - Support Post

General Information

Code Ref : 2006 IBC, ANSI / AF&PA NDS-2005

Analysis Method : Allowable Stress Design
End Fixities: Top & Bottom Pinned
Overall Column Height: 12.50 ft
(Used for non-slender calculations)
Wood Species: Douglas Fir - Larch
Wood Grade: No.2

Fb - Tension	750.0 psi	Fv	170.0 psi
Fb - Compr	750.0 psi	Ft	475.0 psi
Fc - Prll	700.0 psi	Density	32.210 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,300.0	1,300.0	1,300.0 ksi
Minimum	470.0	470.0	

Load Combination 2006 IBC & ASCE 7-05

Wood Section Name: **6x6**
Wood Grading/Manuf.: Graded Lumber
Wood Member Type: Sawn

Exact Width: **5.50** in
Exact Depth: **5.50** in
Area: 30.250 in^2
Ix: 76.255 in^4
Iy: 76.255 in^4

Allowable Stress Modification Factors

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

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 12.5ft, K = 1.0
Y-Y (depth) axis :Unbraced Length for Y-Y Axis buckling = 12.5 ft, K = 1.0

Applied Loads

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

Column self weight included : 84.579 lbs * Dead Load Factor
AXIAL LOADS . . .
FB-2 Reaction: Axial Load at 12.50 ft, D = 2.690, L = 4.770 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.6091** : 1
Load Combination: +D+L+H
Governing NDS Formla: Comp Only, fc/Fc'
Location of max.above base: 0.0 ft
At maximum location values are . . .
Applied Axial: 7.545 k
Applied Mx: 0.0 k-ft
Applied My: 0.0 k-ft
Fc : Allowable: 409.44 psi

PASS Maximum Shear Stress Ratio = **0.0** : 1
Load Combination: +D+0.750Lr+0.750L+H
Location of max.above base: 12.50 ft
Applied Design Shear: 0.0 psi
Allowable Shear: 170.0 psi

Maximum SERVICE Lateral Load Reactions . .
Top along Y-Y 0.0 k Bottom along Y-Y 0.0 k
Top along X-X 0.0 k Bottom along X-X 0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension
Cf or Cv : Size based factors	1.000	1.000	

Load Combination Results

	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
Load Combination	Stress Ratio	Status	Location	Stress Ratio	Status	Location
+D	0.2240	PASS	0.0 ft	0.0	PASS	12.50 ft
+D+L+H	0.6091	PASS	0.0 ft	0.0	PASS	12.50 ft
+D+Lr+H	0.2240	PASS	0.0 ft	0.0	PASS	12.50 ft
+D+0.750Lr+0.750L+H	0.5129	PASS	0.0 ft	0.0	PASS	12.50 ft

Maximum Reactions - Unfactored

Note: Only non-zero reactions are listed.

	X-X Axis Reaction		Y-Y Axis Reaction	
Load Combination	@ Base	@ Top	@ Base	@ Top
D Only				
L Only				

Maximum Deflections for Load Combinations - Unfactored Loads

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

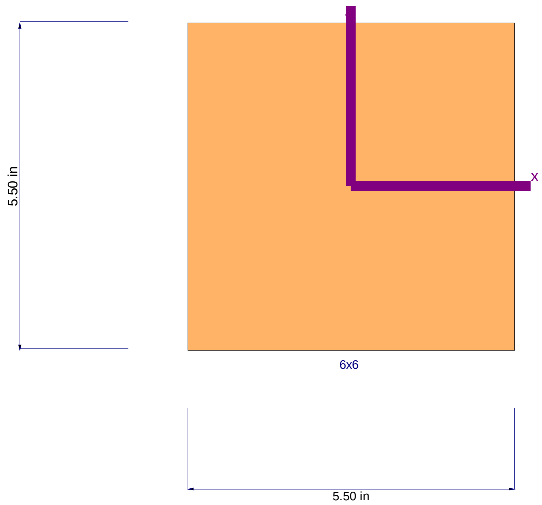
Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

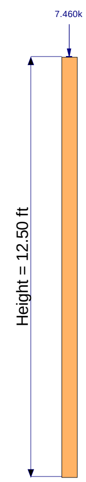
Wood Column

Y:\@Shawn Pierce Engineering\09_Projects\Borzal Addition_02\Engineering\Design\melford borzall expansion.ec6
ENERCALC, INC. 1983-2009, Ver: 6.1.01, N:40336
Lic. # : KW-06008640
License Owner : PIERCE ENGINEERING

Description : FB-3 - Support Post

Sketches





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

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Printed: 17 NOV 2009, 1:49PM

Wood Beam Design

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ENERCALC, INC. 1983-2009, Ver: 6.0.24, N:40336

Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : FB-4 - Floor Beam - Line O

Material Properties

Calculations per IBC 2006, CBC 2007, 2005 NDS

Analysis Method : Allowable Stress Design
Load Combination 2006 IBC & ASCE 7-05

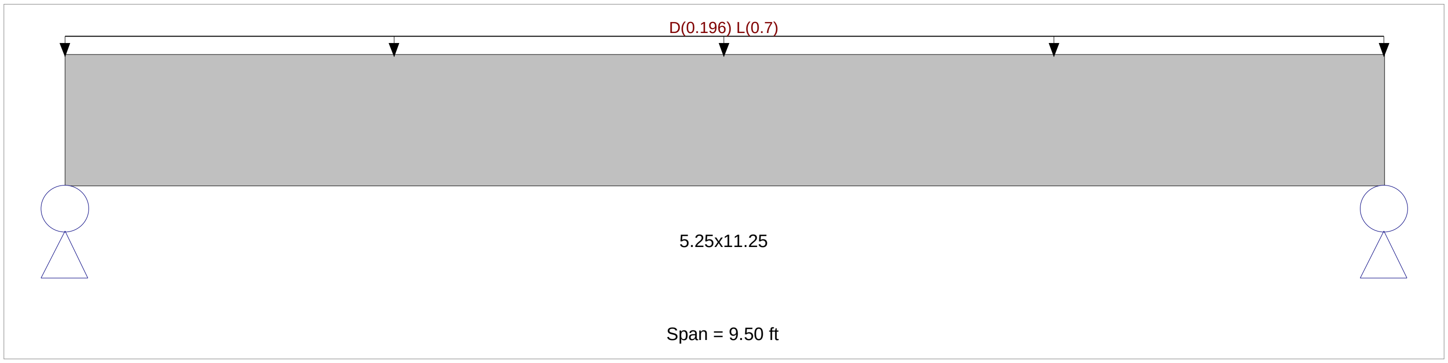
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

Beam Bracing : Beam is Fully Braced against lateral-torsion buckling

Fb - Tension	2,900.0	psi
Fb - Compr	2,900.0	psi
Fc - Prll	2,900.0	psi
Fc - Perp	750.0	psi
Fv	290.0	psi
Ft	2,025.0	psi

E : Modulus of Elasticity

Ebend- xx	2,000.0	ksi
Eminbend - xx	2,000.0	ksi
Density	32.210	pcf



Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.0140, L = 0.050 ksf, Tributary Width = 14.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.383** : 1	Maximum Shear Stress Ratio	=	**0.306** : 1
Section used for this span		**5.25x11.25**	Section used for this span		**5.25x11.25**
fb : Actual	=	1,111.45 psi	fv : Actual	=	88.84 psi
FB : Allowable	=	2,900.00 psi	Fv : Allowable	=	290.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	4.750 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.104 in	Ratio =	1098
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.135 in	Ratio =	845
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios			Summary of Moment Values			Summary of Shear Values		
Segment Length	Span #	M	V	C_d	Mactual	fb-design	Fb-allow	Vactual	fv-design	Fv-allow
+D										
Length = 9.50 ft	1	0.088	0.070	1.000	2.36	255.75	2,900.00	0.80	20.44	290.00
+D+L+H										
Length = 9.50 ft	1	0.383	0.306	1.000	10.26	1,111.45	2,900.00	3.50	88.84	290.00
+D+Lr+H										
Length = 9.50 ft	1	0.088	0.070	1.000	2.36	255.75	2,900.00	0.80	20.44	290.00
+D+0.750Lr+0.750L+H										
Length = 9.50 ft	1	0.309	0.247	1.000	8.28	897.52	2,900.00	2.82	71.74	290.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+Lr	1	0.1348	4.798		0.0000	0.000

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZALL EXPANSION 09.9-2/2
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 12/09
CHECKED BY ______ DATE ______
SCALE OFFICE EXPANSION

FB-5 HEAD - OUT BM @ HVAC UNIT SPACE $\ell = 6'0"$

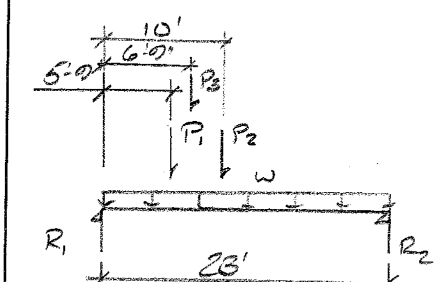
	FLOOR
DL	14 psf
LL	50 psf
TRIB MAX	9'

$R_D = 400^{\#}$
$R_L = 1350^{\#}$
$R_T = 1750^{\#}$

USE: 1 3/4" x 18" 1.9E LVL (MIN) w/ MIU1.81/18 HNGRS

SEE ENERCALC RESULTS

FB-6- BM TO SUPPORT HEAD OUT @ HVAC UNIT SPACE $\ell = 23'0"$



$R_{1D} = 940^{\#}$ $R_{2D} = 680^{\#}$
$R_{1L} = 1980^{\#}$ $R_{2L} = 1500^{\#}$
$R_{1T} = 2920^{\#}$ $R_{2T} = 2180^{\#}$

	FLOOR
DL	14 psf
LL	50 psf
TRIB	1.33'

$P_{1D} = 14\,psf\,(6/2)(5') = 210^{\#}$
$P_{1L} = 40\,psf\,(6/2)(5') = 600^{\#}$

$\left.\begin{array}{l} P_{2D} = 400^{\#} \\ P_{2L} = 1350^{\#} \end{array}\right\}$ FB-5 R

$P_{3D} = 250^{\#}$ (ASSUMED WT)

USE: DBL 1 3/4" x 18" 1.9E LVL (MIN) w/ DBL 2x STUDS @ EA. END

SEE ENERCALC RESULTS

D PRODUCT 207

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Wood Beam Design

Description : FB-5 - Head-out Beam at HVAC Unit Space

Material Properties

Calculations per IBC 2006, CBC 2007, 2005 NDS

Analysis Method : Allowable Stress Design
Load Combination 2006 IBC & ASCE 7-05

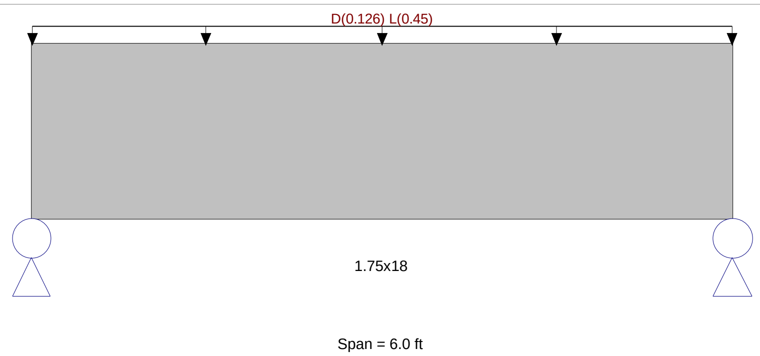
Wood Species : iLevel Truss Joist
Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Beam is Fully Braced against lateral-torsion buckling

Fb - Tension	2600	psi
Fb - Compr	2600	psi
Fc - Prll	2510	psi
Fc - Perp	750	psi
Fv	285	psi
Ft	1555	psi

E : Modulus of Elasticity

Ebend- xx	1900	ksi
Eminbend - xx	965.71	ksi
Density	32.21	pcf



Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.0140, L = 0.050 ksf, Tributary Width = 9.0 ft, (Floor)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.128 : 1	Maximum Shear Stress Ratio	=	0.149 : 1
Section used for this span		**1.75x18**	Section used for this span		**1.75x18**
fb : Actual	=	333.17 psi	fv : Actual	=	42.48 psi
FB : Allowable	=	2,600.00 psi	Fv : Allowable	=	285.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	3.000 ft	Location of maximum on span	=	4.530 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.008 in	Ratio =	8796	
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0	<480
Max Downward Total Deflection	0.011 in	Ratio =	6789	
Max Upward Total Deflection	0.000 in	Ratio =	0	<360

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios			Summary of Moment Values			Summary of Shear Values		
Segment Length	Span #	M	V	C_d	Mactual	fb-design	Fb-allow	Vactual	fv-design	Fv-allow
+D										
Length = 6.0 ft	1	0.029	0.034	1.000	0.60	76.03	2,600.00	0.20	9.69	285.00
+D+L+H										
Length = 6.0 ft	1	0.128	0.149	1.000	2.62	333.17	2,600.00	0.89	42.48	285.00
+D+Lr+H										
Length = 6.0 ft	1	0.029	0.034	1.000	0.60	76.03	2,600.00	0.20	9.69	285.00
+D+0.750Lr+0.750L+H										
Length = 6.0 ft	1	0.103	0.120	1.000	2.12	268.88	2,600.00	0.72	34.28	285.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+Lr	1	0.0106	3.030		0.0000	0.000

Wood Beam Design

Description : FB-5 - Head-out Beam at HVAC Unit Space

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.749	1.749
D Only	0.399	0.399
L Only	1.350	1.350
D+L+S	1.749	1.749
D+L+Lr	1.749	1.749

Wood Beam Design

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Description : FB-6 - Beam to Support Head out at HVAC Unit Space

Material Properties

Calculations per IBC 2006, CBC 2007, 2005 NDS

Analysis Method : Allowable Stress Design
Load Combination 2006 IBC & ASCE 7-05

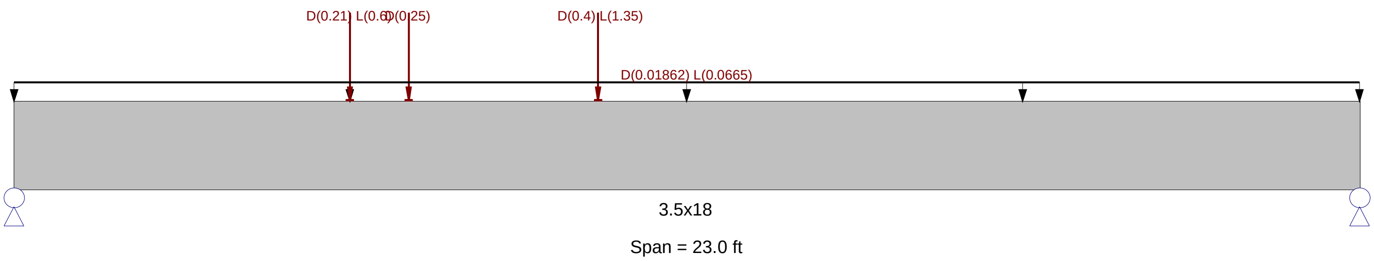
Wood Species : iLevel Truss Joist
Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Beam is Fully Braced against lateral-torsion buckling

Fb - Tension	2600	psi
Fb - Compr	2600	psi
Fc - Prll	2510	psi
Fc - Perp	750	psi
Fv	285	psi
Ft	1555	psi

E : Modulus of Elasticity

Ebend- xx	1900	ksi
Eminbend - xx	965.71	ksi
Density	32.21	pcf



Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.0140, L = 0.050 ksf, Tributary Width = 1.330 ft, (Floor)
Point Load : D = 0.210, L = 0.60 k @ 5.750 ft, (P1)
Point Load : D = 0.40, L = 1.350 k @ 10.0 ft, (P2)
Point Load : D = 0.250 k @ 6.750 ft, (P3 - Mechanical Uni)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.486: 1	Maximum Shear Stress Ratio	=	0.231 : 1
Section used for this span		3.5x18	Section used for this span		3.5x18
fb : Actual	=	1,264.89 psi	fv : Actual	=	65.85 psi
FB : Allowable	=	2,600.00 psi	Fv : Allowable	=	285.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	10.005 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.368 in	Ratio =	750	
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0	<480
Max Downward Total Deflection	0.532 in	Ratio =	518	
Max Upward Total Deflection	0.000 in	Ratio =	0	<360

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios			Summary of Moment Values			Summary of Shear Values		
Segment Length	Span #	M	V	C_d	Mactual	fb-design	Fb-allow	Vactual	fv-design	Fv-allow
+D										
Length = 23.0 ft	1	0.147	0.074	1.000	6.02	382.36	2,600.00	0.89	21.13	285.00
+D+L+H										
Length = 23.0 ft	1	0.486	0.231	1.000	19.92	1,264.89	2,600.00	2.77	65.85	285.00
+D+Lr+H										
Length = 23.0 ft	1	0.147	0.074	1.000	6.02	382.36	2,600.00	0.89	21.13	285.00
+D+0.750Lr+0.750L+H										
Length = 23.0 ft	1	0.402	0.192	1.000	16.45	1,044.25	2,600.00	2.30	54.67	285.00

Wood Beam Design

Description : FB-6 - Beam to Support Head out at HVAC Unit Space

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+Lr	1	0.5319	11.155		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.914	2.178
D Only	0.936	0.676
L Only	1.978	1.502
D+L+S	2.914	2.178
D+L+Lr	2.914	2.178

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277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

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CHECKED BY DATE
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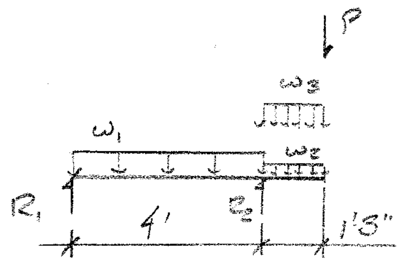
FB-7 - BM TO SUPPORT FLOOR CANTILEVER - LINE (9.4) B/W (3)-(4) $\ell = 5'3"$



	FLOOR	WALL
DL	14 psf	9 psf
LL	50 psf	—
TRIB	11'6"/4'	12'

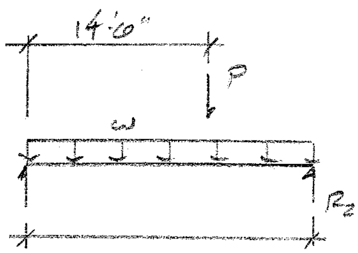
$P_D = 9\,psf(12')(4') = 435^{\#}$
$P_L = \phi$

$R_{1D} = 325^{\#}$ $R_{2D} = 1150^{\#}$
$R_{1L} = 1150^{\#}$ $R_{2L} = 1440^{\#}$
$R_{1T} = 1475^{\#}$ $R_{2T} = 2590^{\#}$

USE $1\frac{3}{4}" \times 18"$ 1.55E LSL TIMBERSTRAND (MIN) w/ U14 (MIN) HANGER & 4x POST AT WALL

SEE ENERCALC RESULTS

FB-8 - BM TO SUPPORT R_1 OF FB-10 $\ell = 23'0"$



	FLOOR
DL	14 psf
LL	40 psf
TRIB	1.33'

$P_D = 325^{\#}$
$P_L = 1150^{\#}$ } FB-10 R_1

$R_{1D} = 330^{\#}$ $R_{2D} = 420^{\#}$
$R_{1L} = 1190^{\#}$ $R_{2L} = 1490^{\#}$
$R_{1T} = 1520^{\#}$ $R_{2T} = 1910^{\#}$

USE: DBL $1\frac{3}{4}" \times 18"$ 1.55E LSL TIMBERSTRAND (MIN)

SEE ENERCALC RESULTS

FB-9 BM UNDER WALL $\ell = 7'6"$

	FLOOR	WALL
DL	14 psf	9 psf
LL	50 psf	—
TRIB	0.67'	12'

$R_D = 470^{\#}$
$R_L = 130^{\#}$
$R_{TL} = 600^{\#}$

USE: $1\frac{3}{4}" \times 18"$ 1.55E LSL TIMBERSTRAND w/ U14 (MIN)

SEE ENERCALC RESULTS

D PRODUCT 207

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Multiple Simple Beam Design

Description : FB-7, 8, 9

Wood Beam Design : FB-7 - Beam to Support Floor Cantilever - Line O.4 Between 3 & 4

BEAM Size : **1.750x18.000, Gang-Lam LVL, Fully Unbraced** — **Calculations per IBC 2006, CBC 2007, 2005 NDS**
Using Allowable Stress Design with 2006 IBC & ASCE 7-05 Load Combinations, Major Axis Ben

Wood Species : iLevel Truss Joist — Wood Grade : TimberStrand LSL 1.55E

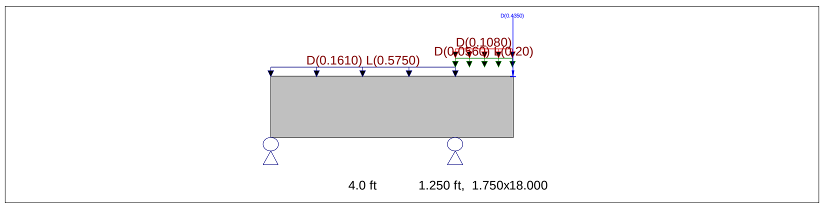
Fb - Tension	2,325.0 psi	Fc - Prll	2,050.0 psi	Fv	310.0 psi	Ebend- xx	1,550.0 ksi	Density	32.210 pcf
Fb - Compr	2,325.0 psi	Fc - Perp	800.0 psi	Ft	1,070.0 psi	Eminbend - xx	787.82 ksi		

Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.0140, L = 0.050 k/ft, 0.0 ft to 4.0 ft, Trib= 11.50 ft
Unif Load: D = 0.0140, L = 0.050 k/ft, 4.0 to 5.250 ft, Trib= 4.0 ft
Unif Load: D = 0.0090 k/ft, 4.0 to 5.250 ft, Trib= 12.0 ft
Point: D = 0.4350 k @ 5.250 ft

Design Summary

Max fb/Fb Ratio = **0.069** : 1
fb : Actual : 148.15 psi at 1.780 ft in Span # 1
Fb : Allowable : 2,136.88 psi
Load Comb : +D+L+H, LL Comb Run (L*)
Max fv/FvRatio = **0.260** : 1
fv : Actual : 80.69 psi at 4.000 ft in Span # 1
Fv : Allowable : 310.00 psi
Load Comb : +D+L+H, LL Comb Run (LL)



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.17	1.11					
Right Support	1.15	1.44					

Max Deflections

Downward L+Lr+S	0.003 in	Downward Total	0.003 in
Upward L+Lr+S	-0.003 in	Upward Total	-0.003 in
Live Load Defl Ratio	11940	Total Defl Ratio	11940

Description : FB-7, 8, 9

Wood Beam Design : FB-8 - Beam To Support R1 of FB-10

BEAM Size : **3.500x18.000, Gang-Lam LVL, Fully Unbraced** — **Calculations per IBC 2006, CBC 2007, 2005 NDS**
Using Allowable Stress Design with 2006 IBC & ASCE 7-05 Load Combinations, Major Axis Ben

Wood Species : iLevel Truss Joist — Wood Grade : TimberStrand LSL 1.55E

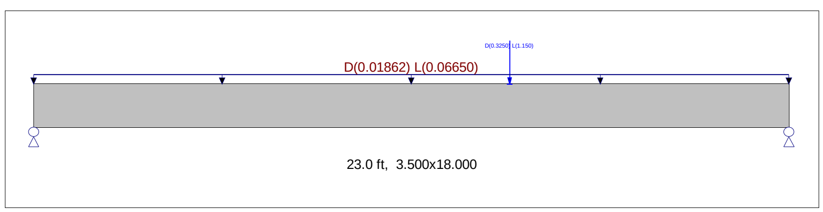
Fb - Tension	2,325.0 psi	Fc - Prll	2,050.0 psi	Fv	310.0 psi	Ebend- xx	1,550.0 ksi	Density	32.210 pcf
Fb - Compr	2,325.0 psi	Fc - Perp	800.0 psi	Ft	1,070.0 psi	Eminbend - xx	787.82 ksi		

Applied Loads

Unif Load: D = 0.0140, L = 0.050 k/ft, Trib= 1.330 ft
Point: D = 0.3250, L = 1.150 k @ 14.50 ft

Design Summary

Max fb/Fb Ratio = **0.439** : 1
fb : Actual : 834.71 psi at 14.490 ft in Span # 1
Fb : Allowable : 1,902.49 psi
Load Comb : +D+L+H
Max fv/FvRatio = **0.137** : 1
fv : Actual : 42.49 psi at 21.543 ft in Span # 1
Fv : Allowable : 310.00 psi
Load Comb : +D+L+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.33	1.19					
Right Support	0.42	1.49					

Max Deflections

Downward L+Lr+S	0.334 in	Downward Total	0.428 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	825	Total Defl Ratio	644

Description : FB-7, 8, 9

Wood Beam Design : FB-9 - Beam under Wall

BEAM Size : **1.750x18.000, Gang-Lam LVL, Fully Unbraced** — **Calculations per IBC 2006, CBC 2007, 2005 NDS**
Using Allowable Stress Design with 2006 IBC & ASCE 7-05 Load Combinations, Major Axis Ben

Wood Species : iLevel Truss Joist — Wood Grade : TimberStrand LSL 1.55E

Fb - Tension	2,325.0 psi	Fc - Prll	2,050.0 psi	Fv	310.0 psi	Ebend- xx	1,550.0 ksi	Density	32.210 pcf
Fb - Compr	2,325.0 psi	Fc - Perp	800.0 psi	Ft	1,070.0 psi	Eminbend - xx	787.82 ksi		

Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.0140, L = 0.050 k/ft, Trib= 0.670 ft
Unif Load: D = 0.0090 k/ft, Trib= 12.0 ft

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:
Project Notes :

Job # 09.9-2/2

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Multiple Simple Beam Design

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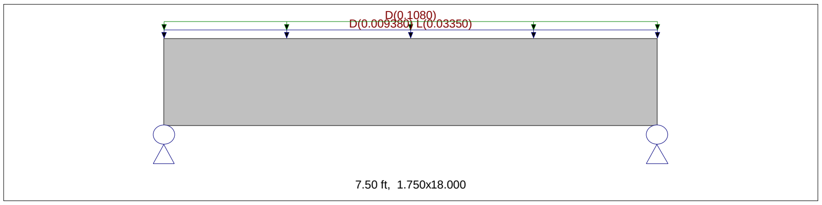
Design Summary

Max fb/Fb Ratio = **0.088** : 1
fb : Actual : 141.01 psi at 3.750 ft in Span # 1
Fb : Allowable : 1,607.08 psi
Load Comb : +D+L+H

Max fv/FvRatio = **0.055** : 1
fv : Actual : 17.11 psi at 6.025 ft in Span # 1
Fv : Allowable : 310.00 psi
Load Comb : +D+L+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.47	0.13					
Right Support	0.47	0.13					

Max Deflections

Downward L+Lr+S	0.002 in	Downward Total	0.009 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	49484	Total Defl Ratio	10496

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Nipomo, CA 93444
(805) 801-9385

JOB: BORZALL EXPANSION 09.9.2/2
SHEET NO.: OF:
CALCULATED BY: [signature] DATE: 2/10
CHECKED BY: DATE:
SCALE:

LANDING FLOOR JOIST ℓ= 4'-0"

NOT USED

	LANDING	
DL	14 psf	R_D = 40#
LL	100 psf	R_L = 270#
TRIB	1.33'	R_T = 310#

USE 2×6 DF#2 @ 16" o.c. w/ LUS26 HGRS IF APPLICABLE

SEE ENERCALC RESULTS

D PRODUCT 207

Multiple Simple Beam Design

Wood Beam Design : Landing Floor Joists

BEAM Size : **2x6, Sawn, Fully Unbraced**

Calculations per IBC 2006, CBC 2007, 2005 NDS

Using Allowable Stress Design with 2006 IBC & ASCE 7-05 Load Combinations, Major Axis Ben

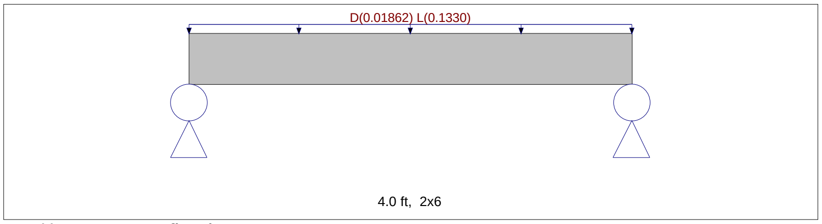
Wood Species : Douglas Fir - Larch   Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	32.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.0140, L = 0.10 k/ft, Trib= 1.330 ft

Design Summary



Max fb/Fb Ratio = **0.416** : 1
- fb : Actual : 481.17 psi at 2.000 ft in Span # 1
- Fb : Allowable : 1,156.14 psi
- Load Comb : +D+L+H

Max fv/FvRatio = **0.237** : 1
- fv : Actual : 42.64 psi at 0.000 ft in Span # 1
- Fv : Allowable : 180.00 psi
- Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.04	0.27					
Right Support	0.04	0.27					

Max Deflections

Downward L+Lr+S	0.023 in	Downward Total	0.026 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	2073	Total Defl Ratio	1819

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(805) 801-9385

JOB BORZAL EXPANSION 09.9-2/2
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 4/10
CHECKED BY ______ DATE ______
SCALE STEEL STAIR FRM'G

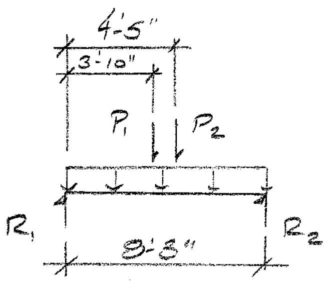
STAIR STRINGER $\ell_{MAX} = 13'0"$

	STAIR
DL	5 psf
LL	100 psf
TRIB	2'

$R_D = 140^{\#}$
$R_L = 2750^{\#}$
$R_T = 2890^{\#}$

USE: HSS 10x2x 1/8" (MIN)
SEE ENERCALC RESULTS

FB-13 - STAIR LANDING BM @ STRINGERS $\ell_{MAX} = 8'3"$



	STAIR LANDING
DL	5 psf
LL	100 psf
TRIB	2'

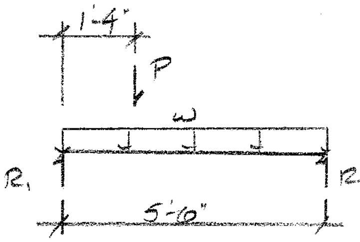
$P_{1D} = 5 psf (2')(4'\text{-}8\tfrac{1}{2}) = 25^{\#}$
$P_{1L} = 100 psf (2')(4'\text{-}8\tfrac{1}{2}) = 470^{\#}$

$P_{2D} = 140^{\#}$
$P_{2L} = 2750^{\#}$ } STAIR STRINGER ABOVE

USE HSS 8x2x 1/8" (MIN)
SEE ENERCALC RESULTS

$R_{1D} = 1500^{\#}$ $R_{2D} = 1600^{\#}$
$R_{1L} = 2350^{\#}$ $R_{2L} = 2520^{\#}$
$R_{1T} = 2500^{\#}$ $R_{2T} = 4120^{\#}$

FB-14 - TOP STAIR LANDING BEAM $\ell = 5'6"$ (MAX)



	STAIR LANDING
DL	5 psf
LL	100 psf
TRIB	4'-6" (MAX)

$P_D = 25^{\#}$
$P_L = 470^{\#}$ } SEE ABOVE

USE: HSS 8x2x 1/8" (MIN)
SEE ENERCALC RESULTS

$R_{1D} = 80^{\#}$ $R_{2D} = 70^{\#}$
$R_L = 1590^{\#}$ $R_{2L} = 1350^{\#}$
$R_{1T} = 1670^{\#}$ $R_{2T} = 1420^{\#}$

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

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Multiple Simple Beam Design

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Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : Stair Stringer, FB-13, 14

Steel Beam Design : Stair Stringer

STEEL Section : **HSS10X2X1/8, Fully Unbraced**
Using Allowable Stress Design with 2006 IBC & ASCE 7-05 Load Combinations, Major Axis Bending

Calculations per IBC 2006, CBC 2007, 13th AISC
Fy = 46.0 ksi E = 29,000.0 ksi

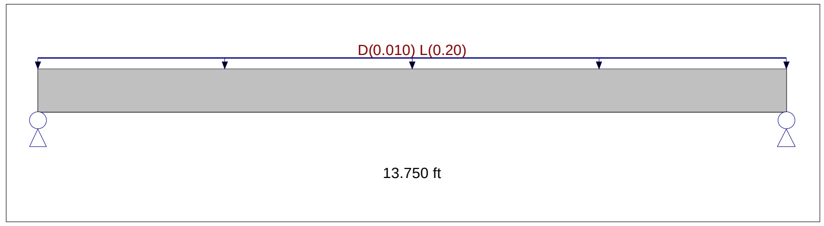
Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.0050, L = 0.10 k/ft, Trib= 2.0 ft

Design Summary

Max fb/Fb Ratio = **0.318** : 1
Mu : Applied 5.196 k-ft at 6.875 ft in Span # 1
Mn / Omega : Allow 16.339 k-ft
Load Comb : +D+L+H

Max fv/FvRatio = **0.059** : 1
Vu : Applied 1.511 k at 0.000 ft in Span # 1
Vn / Omega : Allow 25.443 k
Load Comb : +D+L+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.14	1.38					
Right Support	0.14	1.38					

Max Deflections			
Downward L+Lr+S	0.196 in	Downward Total	0.215 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	843	Total Defl Ratio	767

Description : Stair Stringer, FB-13, 14

Steel Beam Design : FB-13 - Stair Landing Beam @ Stringers

STEEL Section : **HSS8X2X1/8, Fully Unbraced**
Using Allowable Stress Design with 2006 IBC & ASCE 7-05 Load Combinations, Major Axis Bending

Calculations per IBC 2006, CBC 2007, 13th AISC
Fy = 46.0 ksi E = 29,000.0 ksi

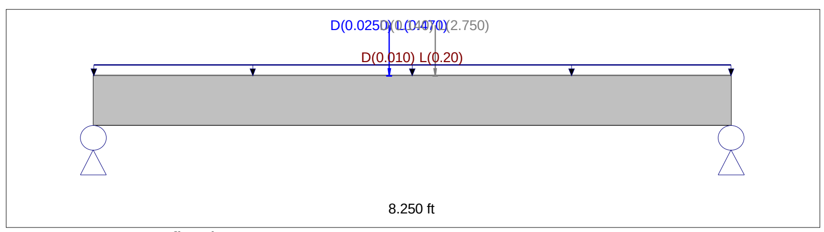
Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.0050, L = 0.10 k/ft, Trib= 2.0 ft
Point: D = 0.0250, L = 0.470 k @ 3.830 ft
Point: D = 0.140, L = 2.750 k @ 4.420 ft

Design Summary

Max fb/Fb Ratio = **0.737** : 1
Mu : Applied 8.643 k-ft at 4.428 ft in Span # 1
Mn / Omega : Allow 11.730 k-ft
Load Comb : +D+L+H

Max fv/FvRatio = **0.097** : 1
Vu : Applied 2.678 k at 8.250 ft in Span # 1
Vn / Omega : Allow 27.469 k
Load Comb : +D+L+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.15	2.35					
Right Support	0.16	2.52					

Max Deflections			
Downward L+Lr+S	0.189 in	Downward Total	0.200 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	524	Total Defl Ratio	494

Description : Stair Stringer, FB-13, 14

Steel Beam Design : --None--

STEEL Section : **HSS8X2X1/8, Fully Unbraced**
Using Allowable Stress Design with 2006 IBC & ASCE 7-05 Load Combinations, Major Axis Bending

Calculations per IBC 2006, CBC 2007, 13th AISC
Fy = 46.0 ksi E = 29,000.0 ksi

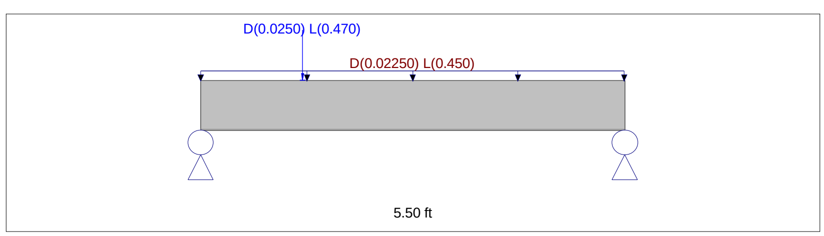
Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.0050, L = 0.10 k/ft, Trib= 4.50 ft
Point: D = 0.0250, L = 0.470 k @ 1.330 ft

Design Summary

Max fb/Fb Ratio = **0.184** : 1
Mu : Applied 2.162 k-ft at 2.493 ft in Span # 1
Mn / Omega : Allow 11.730 k-ft
Load Comb : +D+L+H

Max fv/FvRatio = **0.062** : 1
Vu : Applied 1.697 k at 0.000 ft in Span # 1
Vn / Omega : Allow 27.469 k
Load Comb : +D+L+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.10	1.59					
Right Support	0.09	1.35					

Max Deflections			
Downward L+Lr+S	0.025 in	Downward Total	0.026 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	2679	Total Defl Ratio	2514

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZA ERRATION 09.9-2/2

SHEET NO. ______ OF ______

CALCULATED BY ______ DATE 4/10

CHECKED BY ______ DATE ______

SCALE STEEL STAIR FRMG

FB-15 - TOP STAIR LANDING BEAM $\ell = 9'-3''$

STAIR LANDING

DL	5psf	$R_D = 100^{\#}$
LL	100psf	$R_L = 1275^{\#}$
TRIB	5'-6 1/2	R_{TL} 1375#

USE: HSS 8×2×1/8" (MIN)

SEE ENERCALC RESULTS

STEEL POST @ STAIR FRMG

OPTION #1 $h = 9'-0''$

$P_D = 140^{\#}$ (STRINGER) + $1600^{\#}$ (FB-13) = $1740^{\#}$

$P_L = 2750^{\#}$ (STRINGER) + $2520^{\#}$ (FB-13) = $5270^{\#}$

OPTION #2 $h = 12'-0''$

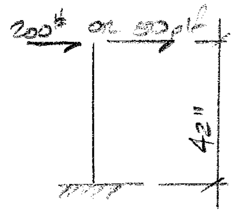
$P_D = 5$psf$(2')(4'\text{-}8\tfrac{1}{2})$ (SHORT STRINGER) + $70^{\#}$ (FB-14) = $100^{\#}$

$P_L = 100$psf$(2')(4'\text{-}8\tfrac{1}{2})$ (SHORT STRINGER) + $1350^{\#}$ (FB-14) = $1820^{\#}$

USE: HSS 3×1/8" COLUMN

SEE ENERCALC RESULTS

HANDRAIL SUPPORT



$M_{MAX} = 200^{\#}(42'') = 8400^{\#''}$

or

$M_{MAX} = 50plf(4') = 200^{\#}(42) = 8400^{\#''}$

$T_{MAX} = 8400^{\#''}/2'' = 4200^{\#}$

WELD REQ'D $= 4200^{\#}/900^{\#/\text{in}}/_{1/16}(2'') = 2.33/16$

USE: 3/16" FILLET WELD ALL AROUND

12.5

D PRODUCT 207

Multiple Simple Beam Design

Description : FB-15

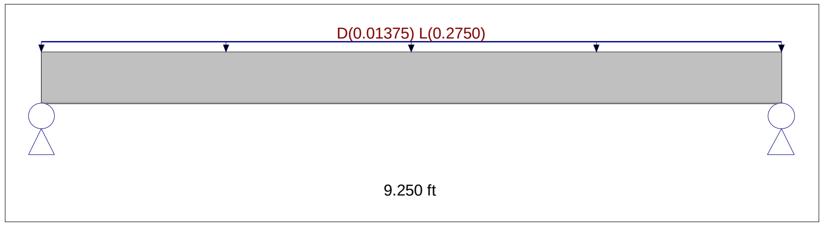
Steel Beam Design : FB-15 - Top Stair Landing Beam

STEEL Section : **HSS8X2X1/8, Fully Unbraced** **Calculations per IBC 2006, CBC 2007, 13th AISC**
Using Allowable Stress Design with 2006 IBC & ASCE 7-05 Load Combinations, Major Axis Bending Fy = 46.0 ksi E = 29,000.0 ksi

Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.0050, L = 0.10 k/ft, Trib= 2.750 ft

Design Summary



Max fb/Fb Ratio =	**0.271** : 1		
Mu : Applied	3.175 k-ft	at	4.625 ft in Span # 1
Mn / Omega : Allow	11.730 k-ft		
Load Comb :	+D+L+H		
Max fv/FvRatio =	**0.050** : 1		
Vu : Applied	1.373 k	at	9.250 ft in Span # 1
Vn / Omega : Allow	27.469 k		
Load Comb :	+D+L+H		

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.10	1.27					
Right Support	0.10	1.27					

Max Deflections			
Downward L+Lr+S	0.100 in	Downward Total	0.108 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	1109	Total Defl Ratio	1027

Steel Column

Description : Steel Post Supporting Steel Stair Framing - Option #2

General Information

Code Ref : 2006 IBC, AISC Manual 13th Edition

Steel Section Name : **HSS 3X0.125**
Analysis Method : 2006 IBC & ASCE 7-05
Steel Stress Grade , A-500, Grade B, Fy = 42 ksi, Carbon Stee
Fy : Steel Yield 42.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi
Load Combination : Allowable Stress

Overall Column Height 12.0 ft
Top & Bottom Fixity Top & Bottom Pinned

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 12ft, K = 1.0
Y-Y (depth) axis :Unbraced Length for Y-Y Axis buckling = 12 ft, K = 1.0

Applied Loads

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

Column self weight included : 46.101 lbs * Dead Load Factor
AXIAL LOADS . . .
Option #1: Axial Load at 12.0 ft, D = 1.60, L = 5.270 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.8753** : 1
Load Combination +D+L+H
Location of max.above base 0.0 ft
At maximum location values are . . .
Pu : Axial 6.916 k
Pn / Omega : Allowable 7.901 k
Mu-x : Applied 0.0 k-ft
Mn-x / Omega : Allowable 2.022 k-ft
Mu-y : Applied 0.0 k-ft
Mn-y / Omega : Allowable 2.022 k-ft

PASS Maximum Shear Stress Ratio = **0.0** : 1
Load Combination
Location of max.above base 0.0 ft
At maximum location values are . . .
Vu : Applied 0.0 k
Vn / Omega : Allowable 0.0 k

Maximum SERVICE Load Reactions . .
Top along X-X 0.0 k
Bottom along X-X 0.0 k
Top along Y-Y 0.0 k
Bottom along Y-Y 0.0 k

Maximum SERVICE Load Deflections . . .
Along Y-Y 0.0 in at 0.0ft above base
for load combination :
Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D	0.208	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+L+H	0.875	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+Lr+H	0.208	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+S+H	0.208	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750Lr+0.750L+H	0.709	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750L+0.750S+H	0.709	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+W+H	0.208	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.70E+H	0.208	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750Lr+0.750L+0.750W+H	0.709	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750L+0.750S+0.750W+H	0.709	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750Lr+0.750L+0.5250E+H	0.709	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750L+0.750S+0.5250E+H	0.709	PASS	0.00 ft	0.000	PASS	0.00 ft
+0.60D+W+H	0.125	PASS	0.00 ft	0.000	PASS	0.00 ft
+0.60D+0.70E+H	0.125	PASS	0.00 ft	0.000	PASS	0.00 ft

Maximum Reactions - Unfactored

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top
D Only				
L Only				

Steel Column

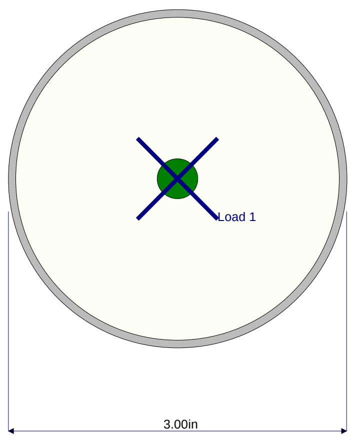
Description : Steel Post Supporting Steel Stair Framing - Option #2

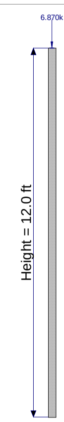
Maximum Deflections for Load Combinations - Unfactored Loads

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Steel Section Properties : HSS 3X0.125

Depth	=	3.000 in	I xx	=	1.09 in^4	J	=	2.190 in^4
Web Thick	=	0.000 in	S xx	=	0.73 in^3			
Flange Width	=	3.000 in	R xx	=	1.020 in			
Flange Thick	=	0.125 in						
Area	=	1.050 in^2	I yy	=	1.090 in^4			
Weight	=	3.842 plf	S yy	=	0.730 in^3			
			R yy	=	1.020 in			
Ycg	=	0.000 in						





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

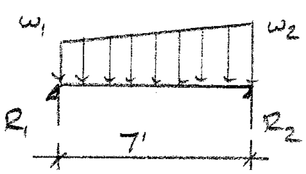
FRAMING ANALYSIS

Existing Office Remodel

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB Borrmall Expansion 09.9-2/2
SHEET NO. ____ OF ____
CALCULATED BY ____ DATE 11/09
REVISED ~~CHECKED~~ BY ____ DATE 4/10
SCALE Office Remodel

FB-10 Exterior Door Hdr ℓ = 7'-0" Approx intersection of lines (1.5) & (7.7)



	Floor	Ceiling Frmg	
DL	14 psf	16 psf	(Stucco wall & frmg)
LL	50 psf	—	
TRIB	17/2 & 22/2	5'-6/2 & 10/2	

R_{1D} = 690# R_{2D} = 770#
R_{1L} = 940# R_{2L} = 1780#
R_{1T} = 1630# R_{2T} = 2550#

Partition load = 15 psf
TRIB = 17/2 & 22/2

USE: 5 1/4" x 9 1/4" 2.0E PSL w/
2x trimrs (min) or (E)
MBHA5.50/9.25 Hgr to masonry wall
& typ. cont. ftg

See Enercalc results

FB-11 Floor Beam @ Hallway ℓ = 5'-6"

	Floor
DL	14 psf
LL	50 psf
TRIB	17/2

USE 6x8 DF#2 (min) w/
2x trimrs (min) & (E) ftg

See Enercalc results

R_D = 350#
R_L = 1170#
R_T = 1520#

Partition load = 15 psf
TRIB = 17/2

FB-12 Not used

D PRODUCT 207

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Printed: 7 APR 2010, 2:41PM

Wood Beam Design

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ENERCALC, INC. 1983-2010, Ver: 6.1.03, N:40336

Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : FB-10 - Exterior Foor Header - Approxmiate Intersection of Lines 1.5 & M.7 - Revised 4/7/10

Material Properties

Calculations per IBC 2006, CBC 2007, 2005 NDS

Analysis Method : Allowable Stress Design
Load Combination 2006 IBC & ASCE 7-05

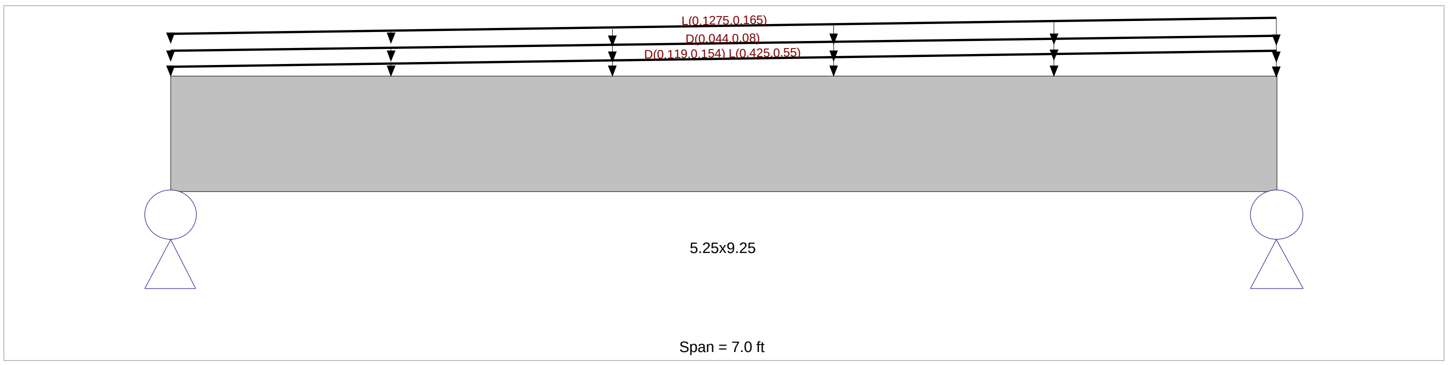
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

Beam Bracing : Completely Unbraced

Fb - Tension	2,900.0	psi
Fb - Compr	2,900.0	psi
Fc - Prll	2,900.0	psi
Fc - Perp	750.0	psi
Fv	290.0	psi
Ft	2,025.0	psi

E : Modulus of Elasticity

Ebend- xx	2,000.0	ksi
Eminbend - xx	2,000.0	ksi
Density	32.210	pcf



Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Varying Uniform Load : D(S,E) = 0.1190->0.1540, L(S,E) = 0.4250->0.550 k/ft, Extent = 0.0 -->> 7.0 ft, Trib Width = 1.0 ft
Varying Uniform Load : D(S,E) = 0.0440->0.080 k/ft, Extent = 0.0 -->> 7.0 ft, Trib Width = 1.0 ft
Varying Uniform Load : L(S,E) = 0.1275->0.1650 k/ft, Extent = 0.0 -->> 7.0 ft, Trib Width = 1.0 ft, (Partition Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.287**: 1	Maximum Shear Stress Ratio	=	**0.251** : 1
Section used for this span		**5.25x9.25**	Section used for this span		**5.25x9.25**
fb : Actual	=	828.15 psi	fv : Actual	=	72.83 psi
FB : Allowable	=	2,889.66 psi	Fv : Allowable	=	290.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	3.570 ft	Location of maximum on span	=	6.230 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.050 in	Ratio =	1685
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.066 in	Ratio =	1267
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios			Summary of Moment Values			Summary of Shear Values		
Segment Length	Span #	M	V	C_d	Mactual	fb-design	Fb-allow	Vactual	fv-design	Fv-allow
+D										
Length = 7.0 ft	1	0.071	0.063	1.000	1.28	205.70	2,889.66	0.59	18.18	290.00
+D+L+H										
Length = 7.0 ft	1	0.287	0.251	1.000	5.17	828.15	2,889.66	2.36	72.83	290.00
+D+Lr+H										
Length = 7.0 ft	1	0.071	0.063	1.000	1.28	205.70	2,889.66	0.59	18.18	290.00
+D+S+H										
Length = 7.0 ft	1	0.071	0.063	1.000	1.28	205.70	2,889.66	0.59	18.18	290.00
+D+0.750Lr+0.750L+H										
Length = 7.0 ft	1	0.233	0.204	1.000	4.20	672.53	2,889.66	1.92	59.17	290.00
+D+0.750L+0.750S+H										
Length = 7.0 ft	1	0.233	0.204	1.000	4.20	672.53	2,889.66	1.92	59.17	290.00
+D+W+H										

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Wood Beam Design

Description : FB-10 - Exterior Foor Header - Approxmiate Intersection of Lines 1.5 & M.7 - Revised 4/7/10

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	Summary of Moment Values Mactual	fb-design	Fb-allow	Summary of Shear Values Vactual	fv-design	Fv-allow
Length = 7.0 ft	1	0.071	0.063	1.000	1.28	205.70	2,889.66	0.59	18.18	290.00
+D+0.70E+H										
Length = 7.0 ft	1	0.071	0.063	1.000	1.28	205.70	2,889.66	0.59	18.18	290.00
+D+0.750Lr+0.750L+0.750W+H										
Length = 7.0 ft	1	0.233	0.204	1.000	4.20	672.53	2,889.66	1.92	59.17	290.00
+D+0.750L+0.750S+0.750W+H										
Length = 7.0 ft	1	0.233	0.204	1.000	4.20	672.53	2,889.66	1.92	59.17	290.00
+D+0.750Lr+0.750L+0.5250E+H										
Length = 7.0 ft	1	0.233	0.204	1.000	4.20	672.53	2,889.66	1.92	59.17	290.00
+D+0.750L+0.750S+0.5250E+H										
Length = 7.0 ft	1	0.233	0.204	1.000	4.20	672.53	2,889.66	1.92	59.17	290.00
+0.60D+W+H										
Length = 7.0 ft	1	0.043	0.038	1.000	0.77	123.42	2,889.66	0.35	10.91	290.00
+0.60D+0.70E+H										
Length = 7.0 ft	1	0.043	0.038	1.000	0.77	123.42	2,889.66	0.35	10.91	290.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+Lr	1	0.0663	3.535		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.815	3.087
D Only	0.691	0.774
L Only	2.123	2.313
D+L+S	2.815	3.087
D+L+Lr	2.815	3.087

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Printed: 7 APR 2010, 2:44PM

Wood Beam Design

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ENERCALC, INC. 1983-2010, Ver: 6.1.03, N:40336

Lic. # : KW-06008640 License Owner : PIERCE ENGINEERING

Description : FB-11 - Floor Beam o/ Hallway - Revised 4/7/10

Material Properties

Calculations per IBC 2006, CBC 2007, 2005 NDS

Analysis Method : Allowable Stress Design
Load Combination 2006 IBC & ASCE 7-05

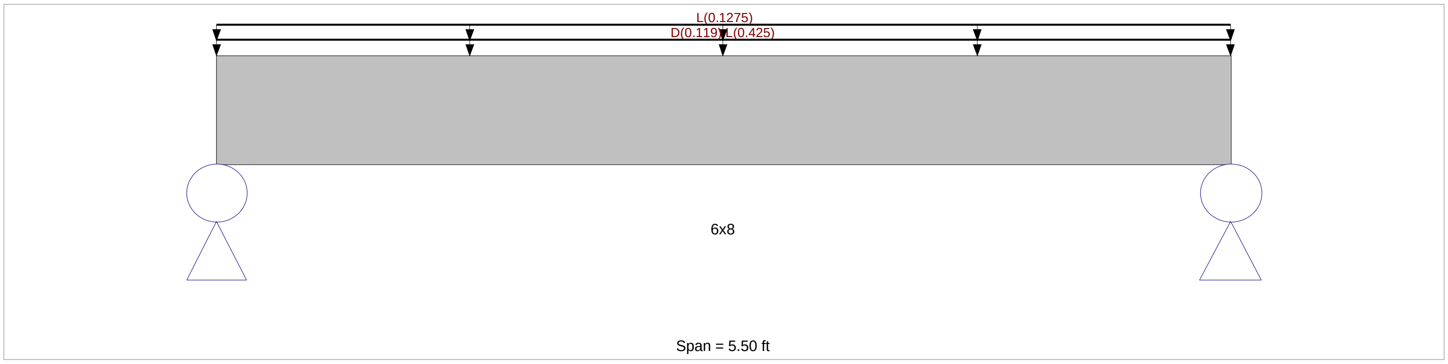
Wood Species : Douglas Fir - Larch
Wood Grade : No.2

Beam Bracing : Completely Unbraced

Fb - Tension	875.0	psi
Fb - Compr	875.0	psi
Fc - Prll	600.0	psi
Fc - Perp	625.0	psi
Fv	170.0	psi
Ft	425.0	psi

E : Modulus of Elasticity

Ebend- xx	1,300.0	ksi
Eminbend - xx	470.0	ksi
Density	32.210	pcf



Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.0140, L = 0.050 ksf, Tributary Width = 8.50 ft
Uniform Load : L = 0.0150 ksf, Tributary Width = 8.50 ft, (Partition Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.685** : 1	Maximum Shear Stress Ratio	=	**0.312** : 1
Section used for this span		**6x8**	Section used for this span		**6x8**
fb : Actual	=	599.04 psi	fv : Actual	=	53.10 psi
FB : Allowable	=	873.88 psi	Fv : Allowable	=	170.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	2.750 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.046 in	Ratio =	1446
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.056 in	Ratio =	1174
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios			Summary of Moment Values			Summary of Shear Values		
Segment Length	Span #	M	V	C_d	Mactual	fb-design	Fb-allow	Vactual	fv-design	Fv-allow
+D										
Length = 5.50 ft	1	0.129	0.059	1.000	0.48	112.84	873.88	0.28	10.00	170.00
+D+L+H										
Length = 5.50 ft	1	0.685	0.312	1.000	2.57	599.04	873.88	1.46	53.10	170.00
+D+Lr+H										
Length = 5.50 ft	1	0.129	0.059	1.000	0.48	112.84	873.88	0.28	10.00	170.00
+D+S+H										
Length = 5.50 ft	1	0.129	0.059	1.000	0.48	112.84	873.88	0.28	10.00	170.00
+D+0.750Lr+0.750L+H										
Length = 5.50 ft	1	0.546	0.249	1.000	2.05	477.49	873.88	1.16	42.32	170.00
+D+0.750L+0.750S+H										
Length = 5.50 ft	1	0.546	0.249	1.000	2.05	477.49	873.88	1.16	42.32	170.00
+D+W+H										
Length = 5.50 ft	1	0.129	0.059	1.000	0.48	112.84	873.88	0.28	10.00	170.00

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
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Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Printed: 7 APR 2010, 2:44PM

Wood Beam Design

Z:\@Shawn Pierce Engineering\09_Projects\Borzal Addition_02\Engineering\Design\melford borzall expansion.ec6
ENERCALC, INC. 1983-2010, Ver: 6.1.03, N:40336

Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : FB-11 - Floor Beam o/ Hallway - Revised 4/7/10

Load Combination		Max Stress Ratios			Summary of Moment Values			Summary of Shear Values		
Segment Length	Span #	M	V	C_d	Mactual	fb-design	Fb-allow	Vactual	fv-design	Fv-allow
+D+0.70E+H										
Length = 5.50 ft	1	0.129	0.059	1.000	0.48	112.84	873.88	0.28	10.00	170.00
+D+0.750Lr+0.750L+0.750W+H										
Length = 5.50 ft	1	0.546	0.249	1.000	2.05	477.49	873.88	1.16	42.32	170.00
+D+0.750L+0.750S+0.750W+H										
Length = 5.50 ft	1	0.546	0.249	1.000	2.05	477.49	873.88	1.16	42.32	170.00
+D+0.750Lr+0.750L+0.5250E+H										
Length = 5.50 ft	1	0.546	0.249	1.000	2.05	477.49	873.88	1.16	42.32	170.00
+D+0.750L+0.750S+0.5250E+H										
Length = 5.50 ft	1	0.546	0.249	1.000	2.05	477.49	873.88	1.16	42.32	170.00
+0.60D+W+H										
Length = 5.50 ft	1	0.077	0.035	1.000	0.29	67.70	873.88	0.17	6.00	170.00
+0.60D+0.70E+H										
Length = 5.50 ft	1	0.077	0.035	1.000	0.29	67.70	873.88	0.17	6.00	170.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+Lr	1	0.0562	2.778		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.872	1.872
D Only	0.353	0.353
L Only	1.519	1.519
D+L+S	1.872	1.872
D+L+Lr	1.872	1.872

FRAMING ANALYSIS

Conference Room Expansion

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZALL EXPANSION 09.9-2/2
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 11/09
CHECKED BY ______ DATE ______
SCALE CONFERENCE ROOM EXTENSION

✓ FLOOR JOISTS ℓ = 11'-3"

	FLOOR
DL	14 psf
LL	100 psf

R_D = 160#
R_L = 1125#
R_{TL} = 1285#

USE: 11 7/8" TJI 210 (MIN) @ 24" o.c.
w/ IUS2.1/11 (w/ WEB STIFFENERS)
OR ITS 2.06/11.88 HANGERS

SEE 'FORTE' RESULTS

✓ FLOOR BM - LINE F.5 & J.5 ℓ = 10'-6"

	FLOOR	UPPER WALL
DL	14 psf	9 psf
LL	100 psf	—
TRIB	11'-3 1/2	13'

R_D = 1030#
R_L = 2955#
R_{TL} = 3985#

USE: 5 1/4" x 11 7/8" 2.0E PSL (MIN)
w/ HGLTV 5.511 x OSL (R) (MIN)
OR CCL (R) COLUMN CAP

SEE 'FORTE' RESULTS

✓ FLOOR BM. - MIDSPAN ℓ = 10'-6"

	FLOOR
DL	14 psf
LL	100 psf
TRIB	11'-3"

R_D = 830#
R_L = 5910#
R_{TL} = 6740#

USE: 7" x 11 7/8" 2.0E PSL (MIN)
w/ GLTV4 11.88-2 HGR(S)

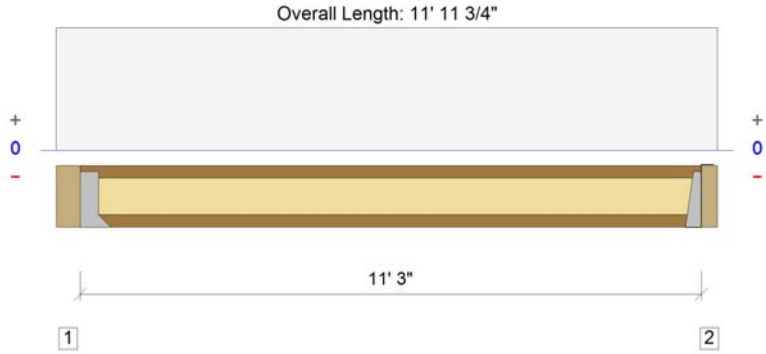
SEE 'FORTE' RESULTS

D PRODUCT 207

Floor Framing, Conference Room Floor Joists

1 PIECE(S) 11 7/8" TJI® 210 @ 24" OC PASSED

All Dimensions are Horizontal - Drawing is Conceptual



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

Design Results	Actual @ Location	Allowed	Result	LDF
Member Reaction (lbs)	1294 @ 5 1/4"	1460	Passed (89%)	100%
Shear (lbs)	1294 @ 5 1/4"	1655	Passed (78%)	100%
Moment (Ft-lbs)	3639 @ 6' 3/4"	3795	Passed (96%)	100%
Live Load Defl. (in)	0.233 @ 6' 3/4"	0.281	Passed (L/578)	- -
Total Load Defl. (in)	0.268 @ 6' 3/4"	0.563	Passed (L/503)	- -
TJ Pro Rating	54	40	Passed	- -

- Deflection criteria: LL (L/480) and TL (L/240).
- Design results assume a fully braced condition where sheathing is properly nailed to all compression edges at the top of the joist and that the compression edges at the bottom of the joist are properly braced to provide lateral stability.
- Bracing (Lu): All compression edges (top and bottom) must be braced at 3' 4 1/16" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- A structural analysis of the deck has not been performed.
- 2000 lbs concentrated load requirements for standard non-residential floors have been considered for reaction and shear.
- Deflection analysis is based on composite action with a single layer of 23/32" iLevel® Edge Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: 1/2" Gypsum ceiling, bridging or blocking.

Supports	Total Bearing	Available Bearing	Required Bearing	Support Reactions (lbs) Dead/Floor/Roof/Snow	Accessories
1 - 11 7/8" Beam - Doug Fir	5.25"	Hanger	- -	182 / 1213 / 0 / 0	None
2 - 11 7/8" Beam - Doug Fir	3.50"	Hanger	- -	178 / 1183 / 0 / 0	None

Loads	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Roof Live (non snow: 1.25)	Snow (1.15)	Comments
1 - Uniform(PSF)	0 to 11' 11 3/4"	24"	15.0	100.0	0.0	0.0	Assembly - Moveable Seating and Lobbies

Connector: Simpson Strong-Tie Connectors

Support	Model	Top Nails	Face Nails	Member Nails	Accessories
1 - Face Mount Hanger	Connector not found	N/A	N/A	N/A	
2 - Top Mount Hanger	Connector not found	N/A	N/A	N/A	

Operator Information	Job Notes	Member Notes
Shawn Pierce Shawn Pierce Engineering (805) 801-9385 pierce.engineering@gmail.com		

Notes

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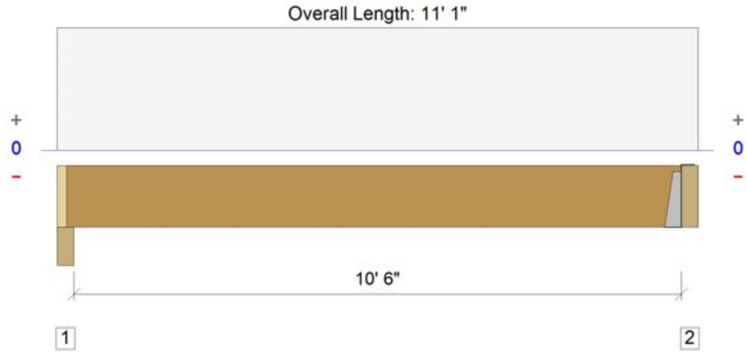
- THIS ANALYSIS FOR iLEVEL® PRODUCTS ONLY! PRODUCT SUBSTITUTION VOIDS THIS ANALYSIS.
- Refer to current iLevel® application guides and literature on how to install this product and its accessories.

1 PIECE(S) 5 1/4" x 11 7/8" 2.0E Parallam® PSL PASSED

All Dimensions are Horizontal - Drawing is Conceptual



Member Type : Flush Beam | Building Use : Commercial | Building Code : IBC | Design Methodology : ASD | System : Floor

Design Results

Design Results	Actual @ Location	Allowed	Result	LDF
Member Reaction (lbs)	4178 @ 2"	5742	Passed (73%)	- -
Shear (lbs)	3386 @ 9' 9 5/8"	12053	Passed (28%)	100%
Moment (Ft-lbs)	11054 @ 5' 5 3/4"	29854	Passed (37%)	100%
Live Load Defl. (in)	0.125 @ 5' 5 3/4"	0.266	Passed (L/999+)	- -
Total Load Defl. (in)	0.174 @ 5' 5 3/4"	0.531	Passed (L/734)	- -

- Deflection criteria: LL (L/480) and TL (L/240).
- Design results assume a fully braced condition where all compression edges (top and bottom) are properly braced to provide lateral stability.
- Bracing (Lu): All compression edges (top and bottom) must be braced at 19' 4 1/8" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- 2000 lbs concentrated load requirements for standard non-residential floors have been considered for reaction and shear.

Supports

Supports	Total Bearing	Available Bearing	Required Bearing	Support Reactions (lbs) Dead/Floor/Roof/Snow	Accessories
1 - Beam - Doug Fir	3.50"	1.75"	1.50"	1207 / 3082 / 0 / 0	1 3/4" Rim Board
2 - 11 7/8" Beam - Doug Fir	3.50"	Hanger	Hanger	1232 / 3152 / 0 / 0	None

Loads

Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non snow: 1.25)	Snow (1.15)	Comments
1 - Uniform(PSF)	0 to 11' 1"	5' 7 1/2"	15.0	100.0	0.0	0.0	Assembly - Moveable Seating and Lobbies
2 - Uniform(PLF)	0 to 11' 1"	N/A	117.0	0.0	0.0	0.0	

Connector: Simpson Strong-Tie Connectors

Support	Model	Top Nails	Face Nails	Member Nails	Accessories
2 - Top Mount Hanger	Connector not found	N/A	N/A	N/A	

Operator Information | Job Notes | Member Notes

Shawn Pierce
Shawn Pierce Engineering
(805) 801-9385
pierce.engineering@gmail.com

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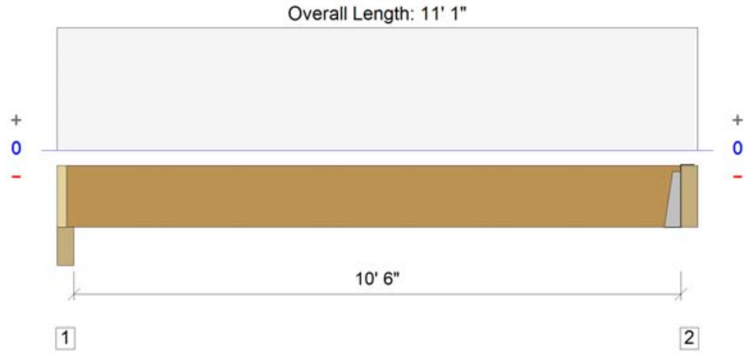
- Rim board and squash blocks are NOT designed by this software and loads on these accessories are assumed to bypass the member being designed. Loads at the end of an overhang, even if located on the closure material, are used to design the member.
- Refer to current iLevel® application guides and literature on how to install this product and its accessories.

1 PIECE(S) 7" x 11 7/8" 2.0E Parallam® PSL PASSED

All Dimensions are Horizontal - Drawing is Conceptual



Member Type : Flush Beam | Building Use : Commercial | Building Code : IBC | Design Methodology : ASD | System : Floor

Design Results

Design Results	Actual @ Location	Allowed	Result	LDF
Member Reaction (lbs)	7039 @ 2"	7656	Passed (92%)	- -
Shear (lbs)	5705 @ 9' 9 5/8"	16071	Passed (36%)	100%
Moment (Ft-lbs)	18624 @ 5' 5 3/4"	39805	Passed (47%)	100%
Live Load Defl. (in)	0.187 @ 5' 5 3/4"	0.266	Passed (L/681)	- -
Total Load Defl. (in)	0.220 @ 5' 5 3/4"	0.531	Passed (L/581)	- -

- Deflection criteria: LL (L/480) and TL (L/240).
- Design results assume a fully braced condition where all compression edges (top and bottom) are properly braced to provide lateral stability.
- Bracing (Lu): All compression edges (top and bottom) must be braced at 34' 4 5/8" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- 2000 lbs concentrated load requirements for standard non-residential floors have been considered for reaction and shear.

Supports

Supports	Total Bearing	Available Bearing	Required Bearing	Support Reactions (lbs) Dead/Floor/Roof/Snow	Accessories
1 - Beam - Doug Fir	3.50"	1.75"	1.61"	1063 / 6164 / 0 / 0	1 3/4" Rim Board
2 - 11 7/8" Beam - Doug Fir	3.50"	Hanger	Hanger	1084 / 6305 / 0 / 0	None

Loads

Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non snow: 1.25)	Snow (1.15)	Comments
1 - Uniform(PSF)	0 to 11' 1"	11' 3"	15.0	100.0	0.0	0.0	Assembly - Moveable Seating and Lobbies

Connector: Simpson Strong-Tie Connectors

Support	Model	Top Nails	Face Nails	Member Nails	Accessories
2 - Top Mount Hanger	HGLTV7.12X Depth12	6-16d common	12-16d common	6-16d common	

Operator Information	Job Notes	Member Notes
Shawn Pierce Shawn Pierce Engineering (805) 801-9385 pierce.engineering@gmail.com		

Notes

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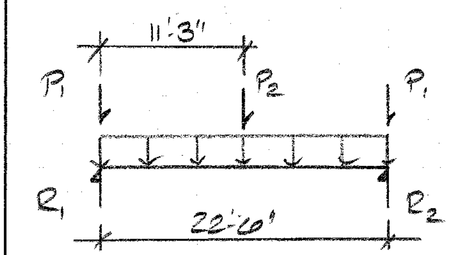
Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZALL EXPANSION 09.9-2/2
SHEET NO. ____ OF ____
CALCULATED BY ____ DATE 11/09
CHECKED BY ____ DATE ____
SCALE CONFERENCE ROOM EXTENSION

FLOOR BM - LINE 3 ℓ = 22'-6"



	FLOOR	UPPER WALL
DL	14 psf	9 psf
LL	100 psf	—
TRIB	1'	13'

P_{1_D} = 1030# P_{2_D} = 830#
P_{1_L} = 2955# P_{2_L} = 5910#

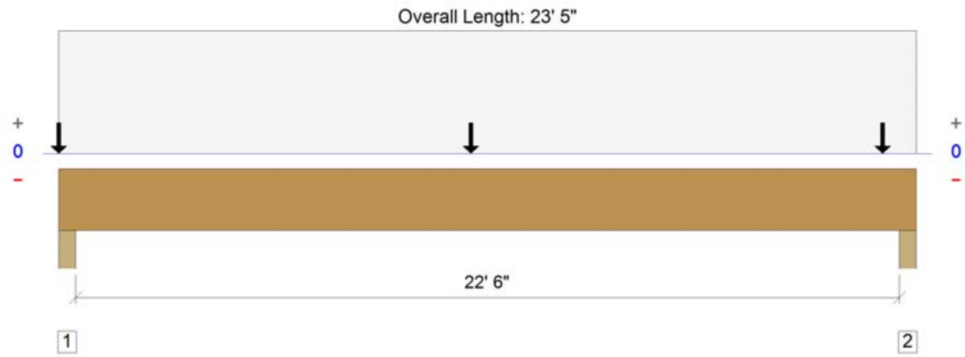
$R_{1D} = R_{2D}$ = 2920#
$R_{1L} = R_{2L}$ = 7035#
$R_{1R} = R_{2R}$ = 9955#

USE: 7"x 18" 2OE PSL OR
W12x 35 A36 STEEL BM

POSTS REQUIRED TO SUPPORT 'FLOOR BM - LINE 3' SEE 'SEISMIC RESTRAINT AT LINE 3'

1 PIECE(S) 7" x 18" 2.0E Parallam® PSL PASSED

All Dimensions are Horizontal - Drawing is Conceptual



Member Type : Flush Beam | Building Use : Commercial | Building Code : IBC | Design Methodology : ASD | System : Floor

Design Results	Actual @ Location	Allowed	Result	LDF
Member Reaction (lbs)	9401 @ 4"	28875	Passed (33%)	- -
Shear (lbs)	5856 @ 21' 5 1/2"	24360	Passed (24%)	100%
Moment (Ft-lbs)	49363 @ 11' 3"	87330	Passed (57%)	100%
Live Load Defl. (in)	0.507 @ 11' 3"	0.569	Passed (L/539)	- -
Total Load Defl. (in)	0.619 @ 11' 3"	1.138	Passed (L/441)	- -

- Deflection criteria: LL (L/480) and TL (L/240).
- Design results assume a fully braced condition where all compression edges (top and bottom) are properly braced to provide lateral stability.
- Bracing (Lu): All compression edges (top and bottom) must be braced at 23' 5" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- 2000 lbs concentrated load requirements for standard non-residential floors have been considered for reaction and shear.

Supports	Total Bearing	Available Bearing	Required Bearing	Support Reactions (lbs) Dead/Floor/Roof/Snow	Accessories
1 - Column - Doug Fir	5.50"	5.50"	1.79"	2126 / 7276 / 0 / 0	None
2 - Column - Doug Fir	5.50"	5.50"	1.70"	2039 / 6886 / 0 / 0	None

Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non snow: 1.25)	Snow (1.15)	Comments
1 - Uniform(PSF)	0 to 23' 5"	1'	15.0	100.0	0.0	0.0	Assembly - Moveable Seating and Lobbies
2 - Point(lb)	0	N/A	1030	2955	0	0	Beam on J.5
3 - Point(lb)	11' 3"	N/A	830	5910	0	0	Beam at Midspan
4 - Point(lb)	22' 6"	N/A	1030	2955	0	0	Beam on F.5

Operator Information

Shawn Pierce
Shawn Pierce Engineering
(805) 801-9385
pierce.engineering@gmail.com

Job Notes

Member Notes

Notes

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FOUNDATION ANALYSIS

Foundation Analysis

General Soil Information used for design of Foundation in this section:

1. Minimum required soil bearing pressure for DL + LL = **2000** psf.
2. Minimum footing requirements for stud walls shall be per Table 1805.4.2 of 2007 California Building Code, unless a soils investigation requires, or these structural calculations, requires otherwise.
3. This firm recommends that on building sites exhibiting characteristics of instability, a soils investigation be prepared, unless waived by the local building review agency. Deviation from design values used in these calculations shall be brought to this firms attention immediately.
4. Soils Report prepared by:

GSI Soils, Inc.			
141 Suburban Road - Suite D-1			
San Luis Obispo, CA			
Report No.:	98-138	Date:	11/23/09

FOUNDATION ANALYSIS

Metal Building Foundation

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BOZZALL EXPANSION 09.9.2/2

SHEET NO. ______ OF ______

CALCULATED BY ______ DATE 2/10

CHECKED BY ______ DATE ______

SCALE ______

VERTICAL DESIGN

FRAMES ON LINES 5, 6, 7, 8, 9, & 10 AT LINE W

MAX COMPRESSIVE LOAD (ASD) → 16.1k (ASCE EQ 3)
MAX UPLIFT LOAD (STRENGTH) → 9.4k (ACI EQ. 9-6)

PAD FOOTING REQUIRED FOR MAX COMPRESSIVE LOAD:

USE: 8'-0" SQ × 18" DP (MIN) W/
(2)-#5 TOP & BTM

SEE ENERCALC RESULTS

ANCHOR BOLTS REQ'D FOR VERTICAL UPLIFT

MAX UPLIFT LOAD (STRENGTH) → 9.4k (ACI EQ. 9-6)
MAX SHEAR LOAD AT MAX UPLIFT → 7.7k (TOWARDS SLAB EDGE)
WIND LOADING

MAX UPLIFT LOAD (STRENGTH) → Ø ← SEISMIC LOADING
MAX SHEAR LOAD (STRENGTH) → 5.4k (TOWARDS SLAB) ACI EQ 9-7
→ 0.6k (TOWARDS SLAB EDGE)

USE: (4) 3/4"Ø F1554 GRADE 36 ANCHOR BOLTS W/ A MIN EMBED OF 6" & #3 HAIRPINS AT EA. PAIR OF ANCHOR BOLTS (SEE HAIRPIN DESIGN IN THIS CALC SET)

SEE 'SIMPSON ANCHOR DESIGNER FOR ACI318' RESULTS

D PRODUCT 207

General Footing Design

Description : Pad Footing at Lines 5,6,7,8,9,&10 - Line W

General Information

Calculations per IBC 2006, CBC 2007, ACI 318-05

Material Properties

f'c : Concrete 28 day strength	=	2.50	ksi
Fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	2,850.0	ksi
Concrete Density	=	145.0	pcf
Φ Values Flexure	=	0.90	
Shear	=	0.850	

Analysis Settings

Min Steel % Bending Reinf.	=	0.00140	
Min Allow % Temp Reinf.	=	0.00180	
Min. Overturning Safety Factor	=	1.50	: 1
Min. Sliding Safety Factor	=	1.50	: 1
AutoCalc Footing Weight as DL	:	No	
AutoCalc Pedestal Weight as DL	:	No	

Soil Design Values

Allowable Soil Bearing	=	2.0	ksf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	350.0	pcf
Soil/Concrete Friction Coeff.	=	0.350	

Increases based on footing Depth

Reference Depth below Surface	=	0.0	ft
Allow. Pressure Increase per foot of depth	=	0.0	ksf
when base footing is below	=	0.0	ft

Increases based on footing Width

Allow. Pressure Increase per foot of width	=	0.0	ksf
when footing is wider than	=	0.0	ft

Dimensions

Width along X-X Axis	=	3.0	ft
Length along Z-Z Axi	=	3.0	ft
Footing Thicknes	=	18.0	in

Load location offset from footing center...

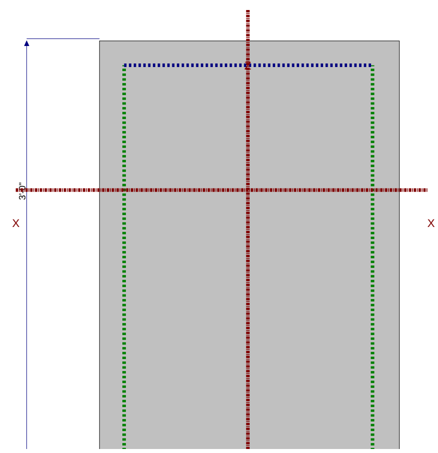
ex : Along X-X Axis	=	0	in
ez : Along Z-Z Axis	=	0	in

Pedestal dimensions...

px : Along X-X Axis	=	0.0	in
pz : Along Z-Z Axis	=	0.0	in
Height	=	0.0	in

Rebar Centerline to Edge of Concrete..

at Bottom of footing	=	3.313	in



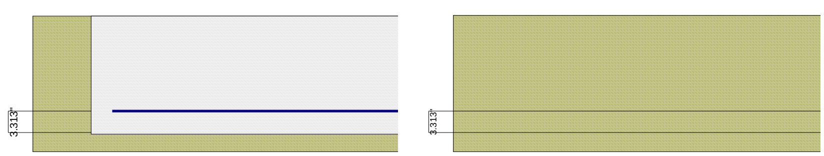
Reinforcing

Bars along X-X Axis

Number of Bars	=		2.0
Reinforcing Bar Size	=	#	5

Bars along Z-Z Axis

Number of Bars	=		2.0
Reinforcing Bar Size	=	#	5



Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	8.20	7.90	0.0	0.0	0.50	2.50	0.0	k
OB : Overburden	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0	ksf
M-xx	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
M-zz	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
V-x	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0	k
V-z	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0	k

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Printed: 1 FEB 2010, 5:29AM

General Footing Design

Z:\@Shawn Pierce Engineering\09_Projects\Borzal Addition_02\Engineering\Design\melford borzall expansion.ec6
ENERCALC, INC. 1983-2009, Ver: 6.1.01, N:40336

Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : Pad Footing at Lines 5,6,7,8,9,&10 - Line W

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8945	Soil Bearing	1.789 ksf	2.0 ksf	+D+Lr+H
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.2129	Z Flexure (+X)	2.859 k-ft	13.433 k-ft	+1.20D+1.60Lr+0.80W
PASS	0.2129	Z Flexure (-X)	2.859 k-ft	13.433 k-ft	+1.20D+1.60Lr+0.80W
PASS	0.2129	X Flexure (+Z)	2.859 k-ft	13.433 k-ft	+1.20D+1.60Lr+0.80W
PASS	0.2129	X Flexure (-Z)	2.859 k-ft	13.433 k-ft	+1.20D+1.60Lr+0.80W
PASS	0.04752	1-way Shear (+X)	4.039 psi	85.0 psi	+1.20D+1.60Lr+0.80W
PASS	0.04752	1-way Shear (-X)	4.039 psi	85.0 psi	+1.20D+1.60Lr+0.80W
PASS	0.04752	1-way Shear (+Z)	4.039 psi	85.0 psi	+1.20D+1.60Lr+0.80W
PASS	0.04752	1-way Shear (-Z)	4.039 psi	85.0 psi	+1.20D+1.60Lr+0.80W
PASS	0.1293	2-way Punching	21.987 psi	170.0 psi	+1.20D+1.60Lr+0.80W

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc	Zecc	Actual Soil Bearing Stress +Z	+Z	-X	-X	Actual / Allowable Ratio
X-X, +D	2.0	n/a	0.0	0.9111	0.9111	n/a	n/a	0.456
X-X, +D+L+H	2.0	n/a	0.0	0.9111	0.9111	n/a	n/a	0.456
X-X, +D+Lr+H	2.0	n/a	0.0	1.789	1.789	n/a	n/a	0.895
X-X, +D+0.750Lr+0.750L+H	2.0	n/a	0.0	1.569	1.569	n/a	n/a	0.785
X-X, +D+W+H	2.0	n/a	0.0	0.9667	0.9667	n/a	n/a	0.483
X-X, +D+0.70E+H	2.0	n/a	0.0	1.106	1.106	n/a	n/a	0.553
X-X, +D+0.750Lr+0.750L+0.750W+H	2.0	n/a	0.0	1.611	1.611	n/a	n/a	0.806
X-X, +D+0.750L+0.750S+0.750W+H	2.0	n/a	0.0	0.9528	0.9528	n/a	n/a	0.476
X-X, +D+0.750Lr+0.750L+0.5250E+H	2.0	n/a	0.0	1.715	1.715	n/a	n/a	0.858
X-X, +D+0.750L+0.750S+0.5250E+H	2.0	n/a	0.0	1.057	1.057	n/a	n/a	0.529
X-X, +0.60D+W+H	2.0	n/a	0.0	0.6022	0.6022	n/a	n/a	0.301
X-X, +0.60D+0.70E+H	2.0	n/a	0.0	0.7411	0.7411	n/a	n/a	0.371
Z-Z, +D	2.0	0.0	n/a	n/a	n/a	0.9111	0.9111	0.456
Z-Z, +D+L+H	2.0	0.0	n/a	n/a	n/a	0.9111	0.9111	0.456
Z-Z, +D+Lr+H	2.0	0.0	n/a	n/a	n/a	1.789	1.789	0.895
Z-Z, +D+0.750Lr+0.750L+H	2.0	0.0	n/a	n/a	n/a	1.569	1.569	0.785
Z-Z, +D+W+H	2.0	0.0	n/a	n/a	n/a	0.9667	0.9667	0.483
Z-Z, +D+0.70E+H	2.0	0.0	n/a	n/a	n/a	1.106	1.106	0.553
Z-Z, +D+0.750Lr+0.750L+0.750W+H	2.0	0.0	n/a	n/a	n/a	1.611	1.611	0.806
Z-Z, +D+0.750L+0.750S+0.750W+H	2.0	0.0	n/a	n/a	n/a	0.9528	0.9528	0.476
Z-Z, +D+0.750Lr+0.750L+0.5250E+H	2.0	0.0	n/a	n/a	n/a	1.715	1.715	0.858
Z-Z, +D+0.750L+0.750S+0.5250E+H	2.0	0.0	n/a	n/a	n/a	1.057	1.057	0.529
Z-Z, +0.60D+W+H	2.0	0.0	n/a	n/a	n/a	0.6022	0.6022	0.301
Z-Z, +0.60D+0.70E+H	2.0	0.0	n/a	n/a	n/a	0.7411	0.7411	0.371

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status

Footing Has NO Overturning

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Sliding SafetyRatio	Status

Footing Has NO Sliding

Footing Flexure

Footing Flexure Load Combination...	Mu	Which Side ?	Tension @ Bot. or Top ?	As Req'd	Gvrn. As	Actual As	Phi*Mn	Status
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK

General Footing Design

Description : Pad Footing at Lines 5,6,7,8,9,&10 - Line W

Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK

General Footing Design

Description : Pad Footing at Lines 5,6,7,8,9,&10 - Line W

Footing Flexure

Footing Flexure Load Combination...	Mu	Which Side ?	Tension @ Bot. or Top ?	As Req'd	Gvrn. As	Actual As	Phi*Mn	Status
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK
Z-Z, +0.90D+E+1.60H	1.235 k-ft	+X	Bottom	0.02 in2/ft	Bending	0.21 in2/ft	13.433 k-ft	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Phi*Vn / Vu	Status
+0.90D+E+1.60H	1.744 psi	1.744 psi	1.744 psi	1.744 psi	1.744 psi	85.0 psi	0.02052	OK
+0.90D+E+1.60H	1.744 psi	1.744 psi	1.744 psi	1.744 psi	1.744 psi	85.0 psi	0.02052	OK
+0.90D+E+1.60H	1.744 psi	1.744 psi	1.744 psi	1.744 psi	1.744 psi	85.0 psi	0.02052	OK
+0.90D+E+1.60H	1.744 psi	1.744 psi	1.744 psi	1.744 psi	1.744 psi	85.0 psi	0.02052	OK
+0.90D+E+1.60H	1.744 psi	1.744 psi	1.744 psi	1.744 psi	1.744 psi	85.0 psi	0.02052	OK
+0.90D+E+1.60H	1.744 psi	1.744 psi	1.744 psi	1.744 psi	1.744 psi	85.0 psi	0.02052	OK
+0.90D+E+1.60H	1.744 psi	1.744 psi	1.744 psi	1.744 psi	1.744 psi	85.0 psi	0.02052	OK
+0.90D+E+1.60H	1.744 psi	1.744 psi	1.744 psi	1.744 psi	1.744 psi	85.0 psi	0.02052	OK
+0.90D+E+1.60H	1.744 psi	1.744 psi	1.744 psi	1.744 psi	1.744 psi	85.0 psi	0.02052	OK
+0.90D+E+1.60H	1.744 psi	1.744 psi	1.744 psi	1.744 psi	1.744 psi	85.0 psi	0.02052	OK

Punching Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+0.90D+E+1.60H	9.494 psi	170.0psi	0.05585	OK
+0.90D+E+1.60H	9.494 psi	170.0psi	0.05585	OK
+0.90D+E+1.60H	9.494 psi	170.0psi	0.05585	OK
+0.90D+E+1.60H	9.494 psi	170.0psi	0.05585	OK
+0.90D+E+1.60H	9.494 psi	170.0psi	0.05585	OK
+0.90D+E+1.60H	9.494 psi	170.0psi	0.05585	OK
+0.90D+E+1.60H	9.494 psi	170.0psi	0.05585	OK
+0.90D+E+1.60H	9.494 psi	170.0psi	0.05585	OK
+0.90D+E+1.60H	9.494 psi	170.0psi	0.05585	OK

General Footing Design

Description : Pad Footing at Lines 5,6,7,8,9,&10 - Line W

Punching Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+0.90D+E+1.60H	9.494 psi	170.0 psi	0.05585	OK

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 5-10 @ W   Date/Time : 2/1/2010 5:58:33 AM

1) Input

Calculation Method : ACI 318 Appendix D For Cracked Concrete

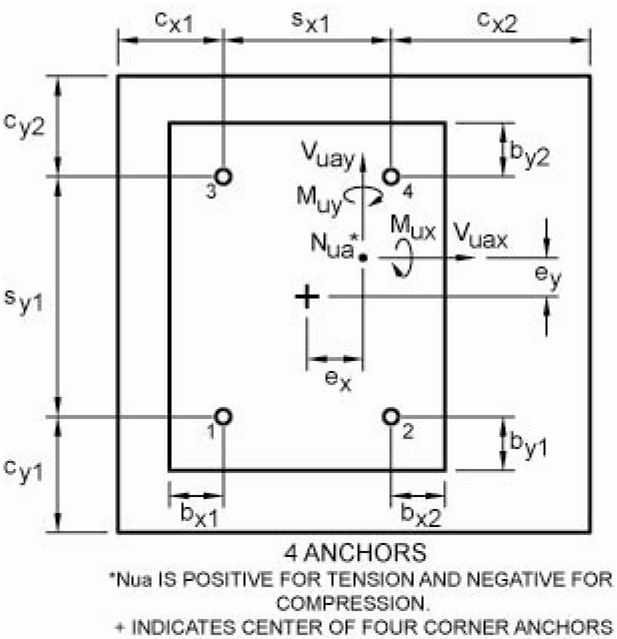
Calculation Type : Analysis

a) Layout

Anchor : 3/4" Heavy Hex Bolt   Number of Anchors : 4

Steel Grade: F1554 GR. 36   Embedment Depth : 6 in

Built-up Grout Pads : No



Anchor Layout Dimensions :

c_{x1} : 16.5 in

c_{x2} : 16.5 in

c_{y1} : 11.5 in

c_{y2} : 21.5 in

Dimensions shown reflect condition if column were eccentric on pad footing. Actual condition will have pad footing centered on column location

b_{x1} : 2.5 in

b_{x2} : 2.5 in

b_{y1} : 3.5 in

b_{y2} : 4.5 in

s_{x1} : 3 in

s_{y1} : 3 in

Warning: Edge distance(s) and/or spacing(s) entered are not in compliance with minimum 6 times the anchor diameter requirements for torqued bolts as detailed in ACI 318 Section D.8.1 and D.8.2. User is responsible for complying with minimum cover requirements in ACI 318.

b) Base Material

Concrete : Normal weight

f'_c : 2500.0 psi

Cracked Concrete : Yes

$\Psi_{c,V}$: 1.00

Condition : A tension and shear

ϕF_p : 1381.3 psi

Thickness, h : 18 in

Supplementary edge reinforcement : No

c) Factored Loads

Load factor source : ACI 318 Section 9.2

N_{ua} : 9400 lb

V_{uax} : 0 lb

V_{uay} : -7700 lb

M_{ux} : 0 lb*ft

M_{uy} : 0 lb*ft

e_x : 0 in

e_y : 0 in

Moderate/high seismic risk or intermediate/high design category : No

Apply entire shear load at front row for breakout : No

d) Anchor Parameters

Anchor Model = HB75

d_o = 0.75 in

Category = N/A

h_{ef} = 5.25 in

h_{min} = 6.75 in

c_{ac} = 7.875 in

c_{min} = [minimum required by ACI 318 Section D8.2]

s_{min} = 3 in

Ductile = Yes

2) Tension Force on Each Individual Anchor

Anchor #1 N_{ua1} = 2350.00 lb

Anchor #2 N_{ua2} = 2350.00 lb

Anchor #3 N_{ua3} = 2350.00 lb

Anchor #4 N_{ua4} = 2350.00 lb

Sum of Anchor Tension ΣN_{ua} = 9400.00 lb

a_x = 0.00 in

a_y = 0.00 in

e'_{Nx} = 0.00 in

e'_{Ny} = 0.00 in

3) Shear Force on Each Individual Anchor

Resultant shear forces in each anchor:

Anchor #1 V_{ua1} = 1925.00 lb (V_{ua1x} = 0.00 lb , V_{ua1y} = -1925.00 lb)

Anchor #2 V_{ua2} = 1925.00 lb (V_{ua2x} = 0.00 lb , V_{ua2y} = -1925.00 lb)

Anchor #3 V_{ua3} = 1925.00 lb (V_{ua3x} = 0.00 lb , V_{ua3y} = -1925.00 lb)

Anchor #4 V_{ua4} = 1925.00 lb (V_{ua4x} = 0.00 lb , V_{ua4y} = -1925.00 lb)

Sum of Anchor Shear ΣV_{uax} = 0.00 lb, ΣV_{uay} = -7700.00 lb

e'_{Vx} = 0.00 in

e'_{Vy} = 0.00 in

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

$N_{sa} = nA_{se}f_{uta}$ [Eq. D-3]

Number of anchors acting in tension, n = 4

N_{sa} = 19370 lb (for each individual anchor)

ϕ = 0.75 [D.4.4]

ϕN_{sa} = 14527.50 lb (for each individual anchor)

5) Concrete Breakout Strength of Anchor Group in Tension [Sec. D.5.2]

$N_{cbg} = A_{Nc}/A_{Nco}\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ [Eq. D-5]

Number of influencing edges = 0

h_{ef} = 5.25 in

A_{Nco} = 248.06 in^2 [Eq. D-6]

A_{Nc} = 351.56 in^2

$\Psi_{ec,Nx}$ = 1.0000 [Eq. D-9]

$\Psi_{ec,Ny}$ = 1.0000 [Eq. D-9]

$\Psi_{ec,N}$ = 1.0000 (Combination of x-axis & y-axis eccentricity factors.)

$\Psi_{ed,N}$ = 1.0000 [Eq. D-10 or D-11]

Note: Cracking shall be controlled per D.5.2.6

$\Psi_{c,N}$ = 1.0000 [Sec. D.5.2.6]

$\Psi_{cp,N}$ = 1.0000 [Eq. D-12 or D-13]

$N_b = k_c\sqrt{f'_c}\,h_{ef}^{1.5}$ = 14435.11 lb [Eq. D-7]

k_c = 24 [Sec. D.5.2.6]

N_{cbg} = 20457.93 lb [Eq. D-5]

ϕ = 0.75 [D.4.4]

ϕN_{cbg} = 15343.45 lb (for the anchor group)

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

$N_p = 8A_{brg}f'_c$ [Eq. D-15]

$A_{brg} = 0.9110\ in^2$

$N_{pn} = \Psi_{c,p}N_p$ [Eq. D-14]

$\Psi_{c,p} = 1.0$ [D.5.3.6]

N_{pn} = 18220.00 lb

$\phi = 0.70$ [D.4.4]

$\phi\ N_{pn}$ = 12754.00 lb (for each individual anchor)

7) Side Face Blowout of Anchor in Tension [Sec. D.5.4]

Concrete side face blowout strength is only calculated for headed anchors in tension close to an edge, $c_{a1} < 0.4h_{ef}$. Not applicable in this case.

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

$V_{sa} = n0.6A_{se}f_{uta}$ [Eq. D-20]

V_{sa} = 11625.00 lb (for each individual anchor)

$\phi = 0.65$ [D.4.4]

$\phi\ V_{sa}$ = 7556.25 lb (for each individual anchor)

9) Concrete Breakout Strength of Anchor Group in Shear [Sec D.6.2]

Concrete breakout strength has not been evaluated against applied shear load(s) per user option. Refer to Section D.4.2.1 of ACI 318 for conditions where calculations of the concrete breakout strength may not be required.

10) Concrete Pryout Strength of Anchor Group in Shear [Sec. D.6.3]

$V_{cpg} = k_{cp}N_{cbg}$ [Eq. D-30]

$k_{cp} = 2$ [Sec. D.6.3.1]

e_{Nx} = 0.00 in (Applied shear load eccentricity relative to anchor group c.g.)

e_{Ny} = 0.00 in (Applied shear load eccentricity relative to anchor group c.g.)

$\Psi_{ec,Nx}$ = 1.0000 [Eq. D-9] (Calulated using applied shear load eccentricity)

$\Psi_{ec,Ny}$ = 1.0000 [Eq. D-9] (Calulated using applied shear load eccentricity)

$\Psi_{ec,N'}$ = 1.0000 (Combination of x-axis & y-axis eccentricity factors)

$N_{cbg} = (A_{Nca}/A_{Nc})(\Psi_{ec,N'}/\Psi_{ec,N})N_{cbg}$

N_{cbg} = 20457.93 lb (from Section (5) of calculations)

$A_{Nc} = 351.56\ in^2$ (from Section (5) of calculations)

A_{Nca} = 351.56 in^2 (considering all anchors)

$\Psi_{ec,N}$ = 1.0000 (from Section(5) of calculations)

N_{cbg} = 20457.93 lb (considering all anchors)

V_{cpg} = 40915.85 lb

ϕ = 0.70 [D.4.4]

ϕV_{cpg} = 28641.10 lb (for the anchor group)

11) Check Demand/Capacity Ratios [Sec. D.7]

Tension

- Steel : 0.1618
- Breakout : 0.6126
- Pullout : 0.1843
- Sideface Blowout : N/A

Shear

- Steel : 0.2548
- Breakout : N/A
- Pryout : 0.2688

T.Max(0.61) + V.Max(0.27) = 0.88 <= 1.2 [Sec D.7.3]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 6 in. embedment

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 5-10 @ W - EQ+ Date/Time : 2/1/2010 5:55:11 AM

1) Input

Calculation Method : ACI 318 Appendix D For Cracked Concrete

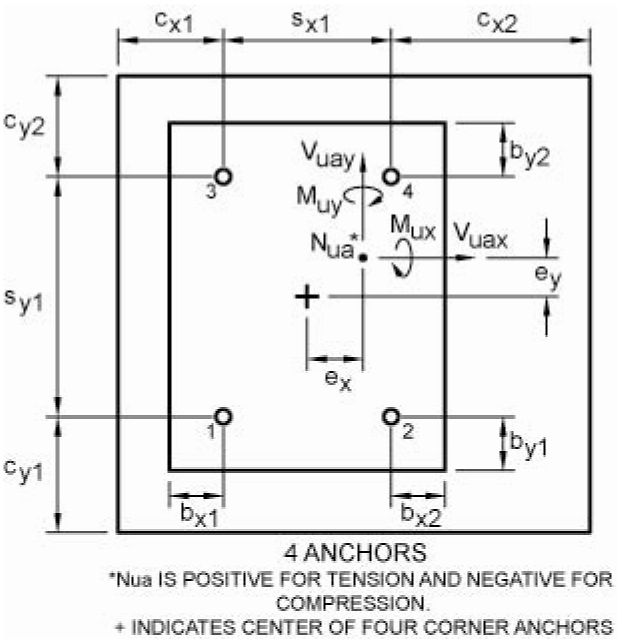
Calculation Type : Analysis

a) Layout

Anchor : 3/4" Heavy Hex Bolt Number of Anchors : 4

Steel Grade: F1554 GR. 36 Embedment Depth : 6 in

Built-up Grout Pads : No



Anchor Layout Dimensions :

c_{x1} : 16.5 in

c_{x2} : 16.5 in

c_{y1} : 11.5 in

c_{y2} : 21.5 in

b_{x1} : 2.5 in

b_{x2} : 2.5 in

b_{y1} : 3.5 in

b_{y2} : 4.5 in

s_{x1} : 3 in

s_{y1} : 3 in

Warning: Edge distance(s) and/or spacing(s) entered are not in compliance with minimum 6 times the anchor diameter requirements for torqued bolts as detailed in ACI 318 Section D.8.1 and D.8.2. User is responsible for complying with minimum cover requirements in ACI 318.

b) Base Material

Concrete : Normal weight

f'_c : 2500.0 psi

Cracked Concrete : Yes

$\Psi_{c,V}$: 1.00

Condition : A tension and shear

ϕF_p : 1381.3 psi

Thickness, h : 18 in

Supplementary edge reinforcement : No

c) Factored Loads

Load factor source : ACI 318 Section 9.2

N_{ua} : 0 lb

V_{uax} : 0 lb

V_{uay} : 5400 lb

M_{ux} : 0 lb*ft

M_{uy} : 0 lb*ft

e_x : 0 in

e_y : 0 in

Moderate/high seismic risk or intermediate/high design category : Yes

Apply entire shear load at front row for breakout : No

d) Anchor Parameters

Anchor Model = HB75

d_o = 0.75 in

Category = N/A

h_{ef} = 5.25 in

h_{min} = 6.75 in

c_{ac} = 7.875 in

c_{min} = [minimum required by ACI 318 Section D8.2]

s_{min} = 3 in

Ductile = Yes

2) Tension Force on Each Individual Anchor

Anchor #1 N_{ua1} = 0.00 lb

Anchor #2 N_{ua2} = 0.00 lb

Anchor #3 N_{ua3} = 0.00 lb

Anchor #4 N_{ua4} = 0.00 lb

Sum of Anchor Tension ΣN_{ua} = 0.00 lb

a_x = 0.00 in

a_y = 0.00 in

e'_{Nx} = 0.00 in

e'_{Ny} = 0.00 in

3) Shear Force on Each Individual Anchor

Resultant shear forces in each anchor:

Anchor #1 V_{ua1} = 1350.00 lb (V_{ua1x} = 0.00 lb , V_{ua1y} = 1350.00 lb)

Anchor #2 V_{ua2} = 1350.00 lb (V_{ua2x} = 0.00 lb , V_{ua2y} = 1350.00 lb)

Anchor #3 V_{ua3} = 1350.00 lb (V_{ua3x} = 0.00 lb , V_{ua3y} = 1350.00 lb)

Anchor #4 V_{ua4} = 1350.00 lb (V_{ua4x} = 0.00 lb , V_{ua4y} = 1350.00 lb)

Sum of Anchor Shear ΣV_{uax} = 0.00 lb, ΣV_{uay} = 5400.00 lb

e'_{Vx} = 0.00 in

e'_{Vy} = 0.00 in

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

$N_{sa} = nA_{se}f_{uta}$ [Eq. D-3]

Number of anchors acting in tension, n = 0

N_{sa} = 19370 lb (for each individual anchor)

19370#/2.5 = 7748#/anchor > 5400# applied to anchor group Anchor Group exceeds 2.5 factor for non-ductile connection.

ϕ = 0.75 [D.4.4]

ϕN_{sa} = 14527.50 lb (for each individual anchor)

5) Concrete Breakout Strength of Anchor Group in Tension [Sec. D.5.2]

$N_{cbg} = A_{Nc}/A_{Nco}\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ [Eq. D-5]

Number of influencing edges = 0

h_{ef} = 5.25 in

A_{Nco} = 248.06 in^2 [Eq. D-6]

A_{Nc} = 351.56 in^2

$\Psi_{ec,Nx}$ = 1.0000 [Eq. D-9]

$\Psi_{ec,Ny}$ = 1.0000 [Eq. D-9]

$\Psi_{ec,N}$ = 1.0000 (Combination of x-axis & y-axis eccentricity factors.)

$\Psi_{ed,N}$ = 1.0000 [Eq. D-10 or D-11]

Note: Cracking shall be controlled per D.5.2.6

$\Psi_{c,N}$ = 1.0000 [Sec. D.5.2.6]

$\Psi_{cp,N}$ = 1.0000 [Eq. D-12 or D-13]

$N_b = k_c\sqrt{f'_c}\,h_{ef}^{1.5}$ = 14435.11 lb [Eq. D-7]

k_c = 24 [Sec. D.5.2.6]

N_{cbg} = 20457.93 lb [Eq. D-5]

ϕ = 0.75 [D.4.4]

ϕ_{seis} = 0.75

ϕN_{cbg} = 11507.58 lb (for the anchor group)

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

$N_p = 8A_{brg}f'_c$ [Eq. D-15]

A_{brg} = 0.9110 in^2

$N_{pn} = \Psi_{c,p}N_p$ [Eq. D-14]

$\Psi_{c,p}$ = 1.0 [D.5.3.6]

N_{pn} = 18220.00 lb

ϕ = 0.70 [D.4.4]

ϕ_{seis} = 0.75

$\phi N_{pn} = \phi N_{eq}$ = 9565.50 lb (for each individual anchor)

7) Side Face Blowout of Anchor in Tension [Sec. D.5.4]

Concrete side face blowout strength is only calculated for headed anchors in tension close to an edge, $c_{a1} < 0.4h_{ef}$. Not applicable in this case.

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

V_{eq} = 11625.00 lb (for each individual anchor)

ϕ = 0.65 [D.4.4]

ϕV_{eq} = 7556.25 lb (for each individual anchor)

9) Concrete Breakout Strength of Anchor Group in Shear [Sec D.6.2]

Concrete breakout strength has not been evaluated against applied shear load(s) per user option. Refer to Section D.4.2.1 of ACI 318 for conditions where calculations of the concrete breakout strength may not be required.

10) Concrete Pryout Strength of Anchor Group in Shear [Sec. D.6.3]

$V_{cpg} = k_{cp}N_{cbg}$ [Eq. D-30]

k_{cp} = 2 [Sec. D.6.3.1]

e_{Nx} = 0.00 in (Applied shear load eccentricity relative to anchor group c.g.)

e_{Ny} = 0.00 in (Applied shear load eccentricity relative to anchor group c.g.)

$\Psi_{ec,Nx}$ = 1.0000 [Eq. D-9] (Calulated using applied shear load eccentricity)

$\Psi_{ec,Ny}$ = 1.0000 [Eq. D-9] (Calulated using applied shear load eccentricity)

$\Psi_{ec,N'}$ = 1.0000 (Combination of x-axis & y-axis eccentricity factors)

$N_{cbg} = (A_{Nca}/A_{Nc})(\Psi_{ec,N'}/\Psi_{ec,N})N_{cbg}$

N_{cbg} = 20457.93 lb (from Section (5) of calculations)

A_{Nc} = 351.56 in^2 (from Section (5) of calculations)

A_{Nca} = 351.56 in^2 (considering all anchors)

$\Psi_{ec,N}$ = 1.0000 (from Section(5) of calculations)

N_{cbg} = 20457.93 lb (considering all anchors)

V_{cpg} = 40915.85 lb

ϕ = 0.70 [D.4.4]

ϕ_{seis} = 0.75

ϕV_{cpg} = 21480.82 lb (for the anchor group)

11) Check Demand/Capacity Ratios [Sec. D.7]

Tension

- Steel : 0.0000
- Breakout : 0.0000
- Pullout : 0.0000
- Sideface Blowout : N/A

Shear

- Steel : 0.1787
- Breakout : N/A
- Pryout : 0.2514

T.Max(0) <= 0.2 and V.Max(0.25) <= 1.0 [Sec D.7.2]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 6 in. embedment

BRITTLE FAILURE GOVERNS: Governing anchor failure mode is brittle failure. Per 2006 IBC Section 1908.1.16, anchors shall be governed by a ductile steel element in structures assigned to Seismic Design Category C, D, E, or F. Alternatively the minimum design strength of the anchor(s) shall be at least 2.5 times the factored forces or the anchor attachment to the structure shall undergo ductile yielding at a load level less than the design strength of the anchor(s). Designer must exercise own judgement to determine if this design is suitable.

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 5-10 @ W - EQ-

Date/Time : 2/1/2010 5:54:16 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
3/4" Heavy Hex Bolt	F1554 GR. 36	4	6	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
A tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
0	0	-600	0	0

e_x(in)	e_y(in)	Mod/high seismic	Apply entire shear @ front row
0	0	Yes	No

Individual Anchor Tension Loads

N_{ua1} (lb)	N_{ua2} (lb)	N_{ua3} (lb)	N_{ua4} (lb)
0.00	0.00	0.00	0.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)	V_{ua2} (lb)	V_{ua3} (lb)	V_{ua4} (lb)
150.00	150.00	150.00	150.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
19370	14527.50	0.00	0.0000

Concrete Breakout (Φ = 0.75 , Φ_{seis} = 0.75)

N_{cbg}(lb)	ΦN_{cbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua}/\Phi N_{cbg}$
20457.93	11507.58	0.00	0.0000

Pullout (Φ = 0.70 , Φ_{seis} = 0.75)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	$N_{ua} / \Phi N_{pn}$
18220.00	9565.50	0.00	0.0000

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.65)

V_{eq}(lb)	ΦV_{eq}(lb)	V_{ua}(lb)	$V_{ua} / \Phi V_{eq}$
11625	7556.25	150.00	0.0199

Concrete Breakout has not been evaluated per user option.

Pryout (Φ = 0.70 , Φ_{seis} = 0.75)

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uax}(lb)	$\Sigma V_{uax} / \Phi V_{cpg}$
40915.85	21480.82	0	0.0000

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uay}(lb)	$\Sigma V_{uay} / \Phi V_{cpg}$	$\Sigma V_{ua} / \Phi V_{cpg}$
40915.85	21480.82	-600	0.0279	0.0279

Interaction check

V.Max(0.03) <= 0.2 and T.Max(0) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 6 in. embedment

BRITTLE FAILURE GOVERNS: Governing anchor failure mode is brittle failure. Per 2006 IBC Section 1908.1.16, anchors shall be governed by a ductile steel element in structures assigned to Seismic Design Category C, D, E, or F. Alternatively the minimum design strength of the anchor(s) shall be at least 2.5 times the factored forces or the anchor attachment to the structure shall undergo ductile yielding at a load level less than the design strength of the anchor(s). Designer must exercise own judgement to determine if this design is suitable.

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BOZZAL EXPANSION 09.9-2/2
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 2/10
CHECKED BY ______ DATE ______
SCALE ______

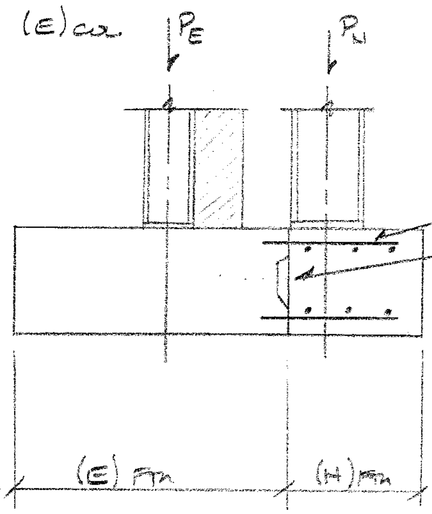
VERTICAL DESIGN (CONT.)

FRAMES ON LINES 5, 6, 7, 8, 9, & 10 AT LINE A

MAX COMPRESSIVE LOAD (ASD) ⟶ 15.8k (ASCE EQ 3)
└ (N) STRUCTURE

MAX COMPRESSIVE LOAD FROM (E) METAL BLDG
D = 2.83k
C = 2.08k
L = 8.31k
ASCE EQ 3 = 13.22k

USE: 4'3" x 5'9" x 18" DP PAD FTG (TOTAL) W/ #3 BARS, EA. WAY, TOP & BTM



(N) REINFORCING INTO FACE OF (E) FTG
SHEAR KEY IF REQ'D

$M_{MAX} = 8.446$ k ft

$V_{MAX} = 15.8$ k

USE: (3)-#5 BARS EPOXIED INTO TOP & BTM OF (E) FTG W/ SIMPSON 'SET' EPOXY & 9" x 51" x 1½" DEEP SHEAR KEY
MIN EMBED OF EPOXY DOWELS = 6½" (5" + 1½" DEPTH OF KEY)

SEE 'SIMPSON ANCHOR DESIGNER FOR ACI 318' FOR EPOXY DESIGN

SHEAR KEY

$\phi V_n = \phi\left[\frac{4}{3}\sqrt{f'_c}\, b_w h\right]$

$\phi = 0.75$

$15.8k(1.7) = 0.75\left[\frac{4}{3}\sqrt{f'_c}\, b_w h\right]$

$540 in^2 = b_w h$

IF $b_w = 51"$

$h = 10.5"$ ⟸ USE 9" W/ COMBINATION OF EPOXY DOWELS

USE: 9" x 51" SHEAR KEY 1½" DEEP

PRODUCT 207

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Printed: 1 FEB 2010, 11:23AM

Combined Footing Design

Z:\@Shawn Pierce Engineering\09_Projects\Borzal Addition_02\Engineering\Design\melford borzall expansion.ec6
ENERCALC, INC. 1983-2009, Ver: 6.1.01, N:40336

Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : Combining (E) Footing and (N) Footing to resist vertical Loads - Line A

General Information

Material Properties

f'c : Concrete 28 day strength	2.50	ksi
Fy : Rebar Yield	60	ksi
Ec : Concrete Elastic Modulus	2,850.0	ksi
Concrete Density	145	pcf
ϕ : Phi Values — Flexure :	0.9	
Shear :	0.85	

Analysis/Design Settings

Calculate footing weight as dead load ?	Yes
Calculate Pedestal weight as dead load ?	No
Min Steel % Bending Reinf (based on 'd')	0.0014
Min Allow % Temp Reinf (based on thick)	0.0018
Min. Overturning Safety Factor	1.5 : 1
Min. Sliding Safety Factor	1.5 : 1

Soil Information

Allowable Soil Bearing	2	ksf
Increase Bearing By Footing Weight	No	
Soil Passive Sliding Resistance	350.0	pcf
(Uses entry for "Footing base depth below soil surface" for force)		
Coefficient of Soil/Concrete Friction	0.350	

Soil Bearing Increase

Footing base depth below soil surface		ft
Increases based on footing Depth		
Allow. Pressure Increase per foot		ksf
when base footing is below		ft
Increases based on footing Width . . .		
Increase per foot of width		ksf
when length -or- width is greater than		ft
Maximum Allowed Bearing Pressure	10	ksf
(zero is no limit)		
Adjusted Allowable Soil Bearing		ksf

(Allowable Soil Bearing adjusted for footing weight and depth & width increases as specified by user.)

Dimensions & Reinforcing

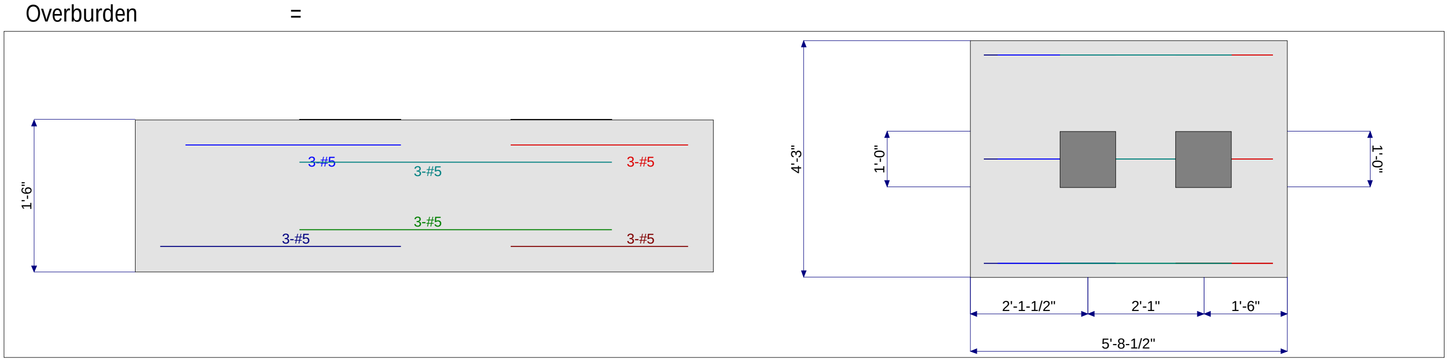
Distance Left of Column #1	=	2.125	ft
Between Columns	=	2.083	ft
Distance Right of Column #2	=	1.50	ft
Total Footing Length	=	5.708	ft
Footing Width	=	4.250	ft
Footing Thickness	=	18	in
Rebar Center to Concrete Edge @ Top	=	3	in
Rebar Center to Concrete Edge @ Bottom	=	3	in

Pedestal dimensions...		Col #1	Col #2	
Sq. Dim.	=	12	12	in
Height	=			in

	Count	Size #	As Actual	As Req'd
Bars left of Col #1				
Bottom Bars	3	5	0.930	1.652 in^2
Top Bars	3	5	0.930	0.0 in^2
Bars Btwn Cols				
Bottom Bars	3	5	0.930	1.652 in^2
Top Bars	3	5	0.930	0.0 in^2
Bars Right of Col #2				
Bottom Bars	3	5	0.930	1.652 in^2
Top Bars	3	5	0.930	0.0 in^2

Applied Loads

		DL	Lr	L	S	W	E	H	
Applied @ Left Column									
Axial Load Downward	=	4.910	8.310						k
Moment (+CCW)	=								k-ft
Shear (+X)	=								k
Applied @ Right Column									
Axial Load Downward	=	7.90	7.90						k
Moment (+CCW)	=								k-ft
Shear (+X)	=								k
Overburden	=								



Combined Footing Design

Description : Combining (E) Footing and (N) Footing to resist vertical Loads - Line A

DESIGN SUMMARY

Design OK

	Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.9607	Soil Bearing	1.921 ksf	2.0 ksf	D+Lr
PASS	No OTM	Overturning	0.0 k-ft	0.0 k-ft	No OTM
PASS	No Sliding	Sliding	0.0 k	8.004 k	No Sliding
PASS	No Uplift	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.08085	1-way Shear - Col #1	6.873 psi	85.0 psi	+1.20D+1.60Lr+0.50L
PASS	0.05229	1-way Shear - Col #2	4.445 psi	85.0 psi	+1.40D
PASS	0.03803	2-way Punching - Col #1	6.465 psi	170.0 psi	+1.20D+1.60Lr+0.50L
PASS	0.02880	2-way Punching - Col #2	4.896 psi	170.0 psi	+1.20D+1.60Lr+0.50L
PASS	No Bending	Flexure - Left of Col #1 - Top	0.0 k-ft	0.0 k-ft	N/A
PASS	0.05179	Flexure - Left of Col #1 - Bottom	3.195 k-ft	61.698 k-ft	+1.40D
PASS	No Bending	Flexure - Between Cols - Top	0.0 k-ft	0.0 k-ft	N/A
PASS	0.1347	Flexure - Between Cols - Bottom	8.314 k-ft	61.698 k-ft	+1.20D+1.60Lr+0.50L
PASS	No Bending	Flexure - Right of Col #2 - Top	0.0 k-ft	0.0 k-ft	N/A
PASS	0.1369	Flexure - Right of Col #2 - Bottom	8.446 k-ft	61.698 k-ft	+1.20D+1.60Lr+0.50L

Soil Bearing

Load Combination...	Total Bearing	Eccentricity from Ftg CL	Actual Soil Bearing Stress @ Left Edge	Actual Soil Bearing Stress @ Right Edge	Allowable	Actual / Allow Ratio
+D	18.09 k	0.394 in	0.44 ksf	1.05 ksf	2.00 ksf	0.526
+D+L+H	18.09 k	0.394 in	0.44 ksf	1.05 ksf	2.00 ksf	0.526
+D+Lr+H	34.30 k	0.343 in	0.91 ksf	1.92 ksf	2.00 ksf	0.961
+D+0.750Lr+0.750L+H	30.24 k	0.350 in	0.79 ksf	1.70 ksf	2.00 ksf	0.852

Overturning Stability

Load Combination...	Moments about Left Edge k-ft Overturning	Resisting	Ratio	Moments about Right Edge k-ft Overturning	Resisting	Ratio
D	0.00	0.00	999.000	0.00	0.00	999.000
D+Lr	0.00	0.00	999.000	0.00	0.00	999.000

Sliding Stability

Load Combination...	Sliding Force	Resisting Force	Sliding SafetyRatio
D	0.00 k	8.00 k	999
D+Lr	0.00 k	13.68 k	999

Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu	Distance from left	Tension Side	As Req'd	Governed by	Actual As	Phi*Mn	Mu / PhiMn
D+Lr	0.000	0.000	0	0.000	0	0.000	0.000	0.000
D+Lr	0.000	0.019	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.001	0.038	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.002	0.057	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.004	0.076	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.006	0.095	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.009	0.114	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.012	0.133	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.016	0.152	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.020	0.171	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.024	0.190	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.030	0.209	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.035	0.228	Bottom	1.652	Min. Percent	0.930	61.698	0.001
D+Lr	0.042	0.247	Bottom	1.652	Min. Percent	0.930	61.698	0.001
D+Lr	0.048	0.266	Bottom	1.652	Min. Percent	0.930	61.698	0.001
D+Lr	0.056	0.285	Bottom	1.652	Min. Percent	0.930	61.698	0.001
D+Lr	0.064	0.304	Bottom	1.652	Min. Percent	0.930	61.698	0.001
D+Lr	0.072	0.323	Bottom	1.652	Min. Percent	0.930	61.698	0.001
D+Lr	0.081	0.342	Bottom	1.652	Min. Percent	0.930	61.698	0.001
D+Lr	0.090	0.362	Bottom	1.652	Min. Percent	0.930	61.698	0.001
D+Lr	0.101	0.381	Bottom	1.652	Min. Percent	0.930	61.698	0.002
D+Lr	0.111	0.400	Bottom	1.652	Min. Percent	0.930	61.698	0.002
D+Lr	0.122	0.419	Bottom	1.652	Min. Percent	0.930	61.698	0.002

Combined Footing Design

Description : Combining (E) Footing and (N) Footing to resist vertical Loads - Line A

Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu	Distance from left	Tension Side	As Req'd	Governed by	Actual As	Phi*Mn	Mu / PhiMn
D+Lr	0.134	0.438	Bottom	1.652	Min. Percent	0.930	61.698	0.002
D+Lr	0.146	0.457	Bottom	1.652	Min. Percent	0.930	61.698	0.002
D+Lr	0.159	0.476	Bottom	1.652	Min. Percent	0.930	61.698	0.003
D+Lr	0.173	0.495	Bottom	1.652	Min. Percent	0.930	61.698	0.003
D+Lr	0.187	0.514	Bottom	1.652	Min. Percent	0.930	61.698	0.003
D+Lr	0.202	0.533	Bottom	1.652	Min. Percent	0.930	61.698	0.003
D+Lr	0.217	0.552	Bottom	1.652	Min. Percent	0.930	61.698	0.004
D+Lr	0.233	0.571	Bottom	1.652	Min. Percent	0.930	61.698	0.004
D+Lr	0.249	0.590	Bottom	1.652	Min. Percent	0.930	61.698	0.004
D+Lr	0.266	0.609	Bottom	1.652	Min. Percent	0.930	61.698	0.004
D+Lr	0.284	0.628	Bottom	1.652	Min. Percent	0.930	61.698	0.005
D+Lr	0.302	0.647	Bottom	1.652	Min. Percent	0.930	61.698	0.005
D+Lr	0.321	0.666	Bottom	1.652	Min. Percent	0.930	61.698	0.005
D+Lr	0.341	0.685	Bottom	1.652	Min. Percent	0.930	61.698	0.006
D+Lr	0.361	0.704	Bottom	1.652	Min. Percent	0.930	61.698	0.006
D+Lr	0.382	0.723	Bottom	1.652	Min. Percent	0.930	61.698	0.006
D+Lr	0.404	0.742	Bottom	1.652	Min. Percent	0.930	61.698	0.007
D+Lr	0.426	0.761	Bottom	1.652	Min. Percent	0.930	61.698	0.007
D+Lr	0.449	0.780	Bottom	1.652	Min. Percent	0.930	61.698	0.007
D+Lr	0.472	0.799	Bottom	1.652	Min. Percent	0.930	61.698	0.008
D+Lr	0.496	0.818	Bottom	1.652	Min. Percent	0.930	61.698	0.008
D+Lr	0.521	0.837	Bottom	1.652	Min. Percent	0.930	61.698	0.008
D+Lr	0.546	0.856	Bottom	1.652	Min. Percent	0.930	61.698	0.009
D+Lr	0.572	0.875	Bottom	1.652	Min. Percent	0.930	61.698	0.009
D+Lr	0.599	0.894	Bottom	1.652	Min. Percent	0.930	61.698	0.010
D+Lr	0.627	0.913	Bottom	1.652	Min. Percent	0.930	61.698	0.010
D+Lr	0.655	0.932	Bottom	1.652	Min. Percent	0.930	61.698	0.011
D+Lr	0.684	0.951	Bottom	1.652	Min. Percent	0.930	61.698	0.011
D+Lr	0.713	0.970	Bottom	1.652	Min. Percent	0.930	61.698	0.012
D+Lr	0.743	0.989	Bottom	1.652	Min. Percent	0.930	61.698	0.012
D+Lr	0.774	1.008	Bottom	1.652	Min. Percent	0.930	61.698	0.013
D+Lr	0.806	1.027	Bottom	1.652	Min. Percent	0.930	61.698	0.013
D+Lr	0.838	1.046	Bottom	1.652	Min. Percent	0.930	61.698	0.014
D+Lr	0.872	1.065	Bottom	1.652	Min. Percent	0.930	61.698	0.014
D+Lr	0.905	1.085	Bottom	1.652	Min. Percent	0.930	61.698	0.015
D+Lr	0.940	1.104	Bottom	1.652	Min. Percent	0.930	61.698	0.015
D+Lr	0.975	1.123	Bottom	1.652	Min. Percent	0.930	61.698	0.016
D+Lr	1.011	1.142	Bottom	1.652	Min. Percent	0.930	61.698	0.016
D+Lr	1.048	1.161	Bottom	1.652	Min. Percent	0.930	61.698	0.017
D+Lr	1.085	1.180	Bottom	1.652	Min. Percent	0.930	61.698	0.018
D+Lr	1.124	1.199	Bottom	1.652	Min. Percent	0.930	61.698	0.018
D+Lr	1.162	1.218	Bottom	1.652	Min. Percent	0.930	61.698	0.019
D+Lr	1.202	1.237	Bottom	1.652	Min. Percent	0.930	61.698	0.019
D+Lr	1.243	1.256	Bottom	1.652	Min. Percent	0.930	61.698	0.020
D+Lr	1.284	1.275	Bottom	1.652	Min. Percent	0.930	61.698	0.021
D+Lr	1.326	1.294	Bottom	1.652	Min. Percent	0.930	61.698	0.021
D+Lr	1.369	1.313	Bottom	1.652	Min. Percent	0.930	61.698	0.022
D+Lr	1.412	1.332	Bottom	1.652	Min. Percent	0.930	61.698	0.023
D+Lr	1.457	1.351	Bottom	1.652	Min. Percent	0.930	61.698	0.024
D+Lr	1.502	1.370	Bottom	1.652	Min. Percent	0.930	61.698	0.024
D+Lr	1.548	1.389	Bottom	1.652	Min. Percent	0.930	61.698	0.025
D+Lr	1.595	1.408	Bottom	1.652	Min. Percent	0.930	61.698	0.026
D+Lr	1.642	1.427	Bottom	1.652	Min. Percent	0.930	61.698	0.027
D+Lr	1.690	1.446	Bottom	1.652	Min. Percent	0.930	61.698	0.027
D+Lr	1.740	1.465	Bottom	1.652	Min. Percent	0.930	61.698	0.028
D+Lr	1.790	1.484	Bottom	1.652	Min. Percent	0.930	61.698	0.029
D+Lr	1.840	1.503	Bottom	1.652	Min. Percent	0.930	61.698	0.030
D+Lr	1.892	1.522	Bottom	1.652	Min. Percent	0.930	61.698	0.031
D+Lr	1.944	1.541	Bottom	1.652	Min. Percent	0.930	61.698	0.032
D+Lr	1.998	1.560	Bottom	1.652	Min. Percent	0.930	61.698	0.032
D+Lr	2.052	1.579	Bottom	1.652	Min. Percent	0.930	61.698	0.033
D+Lr	2.107	1.598	Bottom	1.652	Min. Percent	0.930	61.698	0.034
D+Lr	2.162	1.617	Bottom	1.652	Min. Percent	0.930	61.698	0.035
D+Lr	2.219	1.636	Bottom	1.652	Min. Percent	0.930	61.698	0.036

Description : Combining (E) Footing and (N) Footing to resist vertical Loads - Line A

Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu	Distance from left	Tension Side	As Req'd	Governed by	Actual As	Phi*Mn	Mu / PhiMn
D+Lr	2.273	1.655	Bottom	1.652	Min. Percent	0.930	61.698	0.037
D+Lr	2.327	1.674	Bottom	1.652	Min. Percent	0.930	61.698	0.038
D+Lr	2.378	1.693	Bottom	1.652	Min. Percent	0.930	61.698	0.039
D+Lr	2.428	1.712	Bottom	1.652	Min. Percent	0.930	61.698	0.039
D+Lr	2.476	1.731	Bottom	1.652	Min. Percent	0.930	61.698	0.040
D+Lr	2.523	1.750	Bottom	1.652	Min. Percent	0.930	61.698	0.041
D+Lr	2.568	1.769	Bottom	1.652	Min. Percent	0.930	61.698	0.042
D+Lr	2.611	1.789	Bottom	1.652	Min. Percent	0.930	61.698	0.042
D+Lr	2.653	1.808	Bottom	1.652	Min. Percent	0.930	61.698	0.043
D+Lr	2.694	1.827	Bottom	1.652	Min. Percent	0.930	61.698	0.044
D+Lr	2.732	1.846	Bottom	1.652	Min. Percent	0.930	61.698	0.044
D+Lr	2.769	1.865	Bottom	1.652	Min. Percent	0.930	61.698	0.045
D+Lr	2.805	1.884	Bottom	1.652	Min. Percent	0.930	61.698	0.045
D+Lr	2.839	1.903	Bottom	1.652	Min. Percent	0.930	61.698	0.046
D+Lr	2.871	1.922	Bottom	1.652	Min. Percent	0.930	61.698	0.047
D+Lr	2.902	1.941	Bottom	1.652	Min. Percent	0.930	61.698	0.047
D+Lr	2.931	1.960	Bottom	1.652	Min. Percent	0.930	61.698	0.048
D+Lr	2.959	1.979	Bottom	1.652	Min. Percent	0.930	61.698	0.048
D+Lr	2.985	1.998	Bottom	1.652	Min. Percent	0.930	61.698	0.048
D+Lr	3.009	2.017	Bottom	1.652	Min. Percent	0.930	61.698	0.049
D+Lr	3.033	2.036	Bottom	1.652	Min. Percent	0.930	61.698	0.049
D+Lr	3.054	2.055	Bottom	1.652	Min. Percent	0.930	61.698	0.049
D+Lr	3.074	2.074	Bottom	1.652	Min. Percent	0.930	61.698	0.050
D+Lr	3.092	2.093	Bottom	1.652	Min. Percent	0.930	61.698	0.050
D+Lr	3.109	2.112	Bottom	1.652	Min. Percent	0.930	61.698	0.050
D+Lr	3.125	2.131	Bottom	1.652	Min. Percent	0.930	61.698	0.051
D+Lr	3.138	2.150	Bottom	1.652	Min. Percent	0.930	61.698	0.051
D+Lr	3.151	2.169	Bottom	1.652	Min. Percent	0.930	61.698	0.051
D+Lr	3.161	2.188	Bottom	1.652	Min. Percent	0.930	61.698	0.051
D+Lr	3.171	2.207	Bottom	1.652	Min. Percent	0.930	61.698	0.051
D+Lr	3.179	2.226	Bottom	1.652	Min. Percent	0.930	61.698	0.052
D+Lr	3.185	2.245	Bottom	1.652	Min. Percent	0.930	61.698	0.052
D+Lr	3.190	2.264	Bottom	1.652	Min. Percent	0.930	61.698	0.052
D+Lr	3.193	2.283	Bottom	1.652	Min. Percent	0.930	61.698	0.052
D+Lr	3.195	2.302	Bottom	1.652	Min. Percent	0.930	61.698	0.052
D+Lr	3.195	2.321	Bottom	1.652	Min. Percent	0.930	61.698	0.052
D+Lr	3.194	2.340	Bottom	1.652	Min. Percent	0.930	61.698	0.052
D+Lr	3.192	2.359	Bottom	1.652	Min. Percent	0.930	61.698	0.052
D+Lr	3.187	2.378	Bottom	1.652	Min. Percent	0.930	61.698	0.052
D+Lr	3.182	2.397	Bottom	1.652	Min. Percent	0.930	61.698	0.052
D+Lr	3.175	2.416	Bottom	1.652	Min. Percent	0.930	61.698	0.051
D+Lr	3.167	2.435	Bottom	1.652	Min. Percent	0.930	61.698	0.051
D+Lr	3.157	2.454	Bottom	1.652	Min. Percent	0.930	61.698	0.051
D+Lr	3.145	2.473	Bottom	1.652	Min. Percent	0.930	61.698	0.051
D+Lr	3.133	2.492	Bottom	1.652	Min. Percent	0.930	61.698	0.051
D+Lr	3.118	2.512	Bottom	1.652	Min. Percent	0.930	61.698	0.051
D+Lr	3.103	2.531	Bottom	1.652	Min. Percent	0.930	61.698	0.050
D+Lr	3.086	2.550	Bottom	1.652	Min. Percent	0.930	61.698	0.050
D+Lr	3.067	2.569	Bottom	1.652	Min. Percent	0.930	61.698	0.050
D+Lr	3.047	2.588	Bottom	1.652	Min. Percent	0.930	61.698	0.049
D+Lr	3.026	2.607	Bottom	1.652	Min. Percent	0.930	61.698	0.049
D+Lr	8.314	2.626	Bottom	1.652	Min. Percent	0.930	61.698	0.135
D+Lr	8.232	2.645	Bottom	1.652	Min. Percent	0.930	61.698	0.133
D+Lr	8.153	2.664	Bottom	1.652	Min. Percent	0.930	61.698	0.132
D+Lr	8.077	2.683	Bottom	1.652	Min. Percent	0.930	61.698	0.131
D+Lr	8.003	2.702	Bottom	1.652	Min. Percent	0.930	61.698	0.130
D+Lr	7.931	2.721	Bottom	1.652	Min. Percent	0.930	61.698	0.129
D+Lr	7.862	2.740	Bottom	1.652	Min. Percent	0.930	61.698	0.127
D+Lr	7.796	2.759	Bottom	1.652	Min. Percent	0.930	61.698	0.126
D+Lr	7.733	2.778	Bottom	1.652	Min. Percent	0.930	61.698	0.125
D+Lr	7.672	2.797	Bottom	1.652	Min. Percent	0.930	61.698	0.124
D+Lr	7.613	2.816	Bottom	1.652	Min. Percent	0.930	61.698	0.123
D+Lr	7.557	2.835	Bottom	1.652	Min. Percent	0.930	61.698	0.122
D+Lr	7.504	2.854	Bottom	1.652	Min. Percent	0.930	61.698	0.122

Description : Combining (E) Footing and (N) Footing to resist vertical Loads - Line A

Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu	Distance from left	Tension Side	As Req'd	Governed by	Actual As	Phi*Mn	Mu / PhiMn
D+Lr	7.453	2.873	Bottom	1.652	Min. Percent	0.930	61.698	0.121
D+Lr	7.405	2.892	Bottom	1.652	Min. Percent	0.930	61.698	0.120
D+Lr	7.360	2.911	Bottom	1.652	Min. Percent	0.930	61.698	0.119
D+Lr	7.317	2.930	Bottom	1.652	Min. Percent	0.930	61.698	0.119
D+Lr	7.277	2.949	Bottom	1.652	Min. Percent	0.930	61.698	0.118
D+Lr	7.240	2.968	Bottom	1.652	Min. Percent	0.930	61.698	0.117
D+Lr	7.205	2.987	Bottom	1.652	Min. Percent	0.930	61.698	0.117
D+Lr	7.173	3.006	Bottom	1.652	Min. Percent	0.930	61.698	0.116
D+Lr	7.144	3.025	Bottom	1.652	Min. Percent	0.930	61.698	0.116
D+Lr	7.117	3.044	Bottom	1.652	Min. Percent	0.930	61.698	0.115
D+Lr	7.093	3.063	Bottom	1.652	Min. Percent	0.930	61.698	0.115
D+Lr	7.072	3.082	Bottom	1.652	Min. Percent	0.930	61.698	0.115
D+Lr	7.053	3.101	Bottom	1.652	Min. Percent	0.930	61.698	0.114
D+Lr	7.037	3.120	Bottom	1.652	Min. Percent	0.930	61.698	0.114
D+Lr	7.024	3.139	Bottom	1.652	Min. Percent	0.930	61.698	0.114
D+Lr	7.013	3.158	Bottom	1.652	Min. Percent	0.930	61.698	0.114
D+Lr	7.006	3.177	Bottom	1.652	Min. Percent	0.930	61.698	0.114
D+Lr	7.001	3.196	Bottom	1.652	Min. Percent	0.930	61.698	0.113
D+Lr	6.999	3.216	Bottom	1.652	Min. Percent	0.930	61.698	0.113
D+Lr	6.999	3.235	Bottom	1.652	Min. Percent	0.930	61.698	0.113
D+Lr	7.002	3.254	Bottom	1.652	Min. Percent	0.930	61.698	0.113
D+Lr	7.008	3.273	Bottom	1.652	Min. Percent	0.930	61.698	0.114
D+Lr	7.017	3.292	Bottom	1.652	Min. Percent	0.930	61.698	0.114
D+Lr	7.029	3.311	Bottom	1.652	Min. Percent	0.930	61.698	0.114
D+Lr	7.043	3.330	Bottom	1.652	Min. Percent	0.930	61.698	0.114
D+Lr	7.060	3.349	Bottom	1.652	Min. Percent	0.930	61.698	0.114
D+Lr	7.080	3.368	Bottom	1.652	Min. Percent	0.930	61.698	0.115
D+Lr	7.103	3.387	Bottom	1.652	Min. Percent	0.930	61.698	0.115
D+Lr	7.129	3.406	Bottom	1.652	Min. Percent	0.930	61.698	0.116
D+Lr	7.157	3.425	Bottom	1.652	Min. Percent	0.930	61.698	0.116
D+Lr	7.189	3.444	Bottom	1.652	Min. Percent	0.930	61.698	0.117
D+Lr	7.223	3.463	Bottom	1.652	Min. Percent	0.930	61.698	0.117
D+Lr	7.260	3.482	Bottom	1.652	Min. Percent	0.930	61.698	0.118
D+Lr	7.299	3.501	Bottom	1.652	Min. Percent	0.930	61.698	0.118
D+Lr	7.342	3.520	Bottom	1.652	Min. Percent	0.930	61.698	0.119
D+Lr	7.388	3.539	Bottom	1.652	Min. Percent	0.930	61.698	0.120
D+Lr	7.436	3.558	Bottom	1.652	Min. Percent	0.930	61.698	0.121
D+Lr	7.487	3.577	Bottom	1.652	Min. Percent	0.930	61.698	0.121
D+Lr	7.542	3.596	Bottom	1.652	Min. Percent	0.930	61.698	0.122
D+Lr	7.599	3.615	Bottom	1.652	Min. Percent	0.930	61.698	0.123
D+Lr	7.659	3.634	Bottom	1.652	Min. Percent	0.930	61.698	0.124
D+Lr	7.722	3.653	Bottom	1.652	Min. Percent	0.930	61.698	0.125
D+Lr	7.788	3.672	Bottom	1.652	Min. Percent	0.930	61.698	0.126
D+Lr	7.856	3.691	Bottom	1.652	Min. Percent	0.930	61.698	0.127
D+Lr	7.928	3.710	Bottom	1.652	Min. Percent	0.930	61.698	0.128
D+Lr	7.998	3.729	Bottom	1.652	Min. Percent	0.930	61.698	0.130
D+Lr	8.062	3.748	Bottom	1.652	Min. Percent	0.930	61.698	0.131
D+Lr	8.122	3.767	Bottom	1.652	Min. Percent	0.930	61.698	0.132
D+Lr	8.176	3.786	Bottom	1.652	Min. Percent	0.930	61.698	0.133
D+Lr	8.226	3.805	Bottom	1.652	Min. Percent	0.930	61.698	0.133
D+Lr	8.270	3.824	Bottom	1.652	Min. Percent	0.930	61.698	0.134
D+Lr	8.310	3.843	Bottom	1.652	Min. Percent	0.930	61.698	0.135
D+Lr	8.344	3.862	Bottom	1.652	Min. Percent	0.930	61.698	0.135
D+Lr	8.374	3.881	Bottom	1.652	Min. Percent	0.930	61.698	0.136
D+Lr	8.398	3.900	Bottom	1.652	Min. Percent	0.930	61.698	0.136
D+Lr	8.417	3.919	Bottom	1.652	Min. Percent	0.930	61.698	0.136
D+Lr	8.432	3.939	Bottom	1.652	Min. Percent	0.930	61.698	0.137
D+Lr	8.441	3.958	Bottom	1.652	Min. Percent	0.930	61.698	0.137
D+Lr	8.446	3.977	Bottom	1.652	Min. Percent	0.930	61.698	0.137
D+Lr	8.445	3.996	Bottom	1.652	Min. Percent	0.930	61.698	0.137
D+Lr	8.440	4.015	Bottom	1.652	Min. Percent	0.930	61.698	0.137
D+Lr	8.429	4.034	Bottom	1.652	Min. Percent	0.930	61.698	0.137
D+Lr	8.414	4.053	Bottom	1.652	Min. Percent	0.930	61.698	0.136
D+Lr	8.394	4.072	Bottom	1.652	Min. Percent	0.930	61.698	0.136

Combined Footing Design

Description : Combining (E) Footing and (N) Footing to resist vertical Loads - Line A

Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu	Distance from left	Tension Side	As Req'd	Governed by	Actual As	Phi*Mn	Mu / PhiMn
D+Lr	8.368	4.091	Bottom	1.652	Min. Percent	0.930	61.698	0.136
D+Lr	8.338	4.110	Bottom	1.652	Min. Percent	0.930	61.698	0.135
D+Lr	8.303	4.129	Bottom	1.652	Min. Percent	0.930	61.698	0.135
D+Lr	8.263	4.148	Bottom	1.652	Min. Percent	0.930	61.698	0.134
D+Lr	8.218	4.167	Bottom	1.652	Min. Percent	0.930	61.698	0.133
D+Lr	8.168	4.186	Bottom	1.652	Min. Percent	0.930	61.698	0.132
D+Lr	8.114	4.205	Bottom	1.652	Min. Percent	0.930	61.698	0.132
D+Lr	8.054	4.224	Bottom	1.652	Min. Percent	0.930	61.698	0.131
D+Lr	7.989	4.243	Bottom	1.652	Min. Percent	0.930	61.698	0.129
D+Lr	7.920	4.262	Bottom	1.652	Min. Percent	0.930	61.698	0.128
D+Lr	7.846	4.281	Bottom	1.652	Min. Percent	0.930	61.698	0.127
D+Lr	7.767	4.300	Bottom	1.652	Min. Percent	0.930	61.698	0.126
D+Lr	7.683	4.319	Bottom	1.652	Min. Percent	0.930	61.698	0.125
D+Lr	7.594	4.338	Bottom	1.652	Min. Percent	0.930	61.698	0.123
D+Lr	7.501	4.357	Bottom	1.652	Min. Percent	0.930	61.698	0.122
D+Lr	7.402	4.376	Bottom	1.652	Min. Percent	0.930	61.698	0.120
D+Lr	7.299	4.395	Bottom	1.652	Min. Percent	0.930	61.698	0.118
D+Lr	7.191	4.414	Bottom	1.652	Min. Percent	0.930	61.698	0.117
D+Lr	7.078	4.433	Bottom	1.652	Min. Percent	0.930	61.698	0.115
D+Lr	6.960	4.452	Bottom	1.652	Min. Percent	0.930	61.698	0.113
D+Lr	6.838	4.471	Bottom	1.652	Min. Percent	0.930	61.698	0.111
D+Lr	6.711	4.490	Bottom	1.652	Min. Percent	0.930	61.698	0.109
D+Lr	6.579	4.509	Bottom	1.652	Min. Percent	0.930	61.698	0.107
D+Lr	6.442	4.528	Bottom	1.652	Min. Percent	0.930	61.698	0.104
D+Lr	6.300	4.547	Bottom	1.652	Min. Percent	0.930	61.698	0.102
D+Lr	6.154	4.566	Bottom	1.652	Min. Percent	0.930	61.698	0.100
D+Lr	6.003	4.585	Bottom	1.652	Min. Percent	0.930	61.698	0.097
D+Lr	5.847	4.604	Bottom	1.652	Min. Percent	0.930	61.698	0.095
D+Lr	5.687	4.623	Bottom	1.652	Min. Percent	0.930	61.698	0.092
D+Lr	5.521	4.643	Bottom	1.652	Min. Percent	0.930	61.698	0.089
D+Lr	5.351	4.662	Bottom	1.652	Min. Percent	0.930	61.698	0.087
D+Lr	5.177	4.681	Bottom	1.652	Min. Percent	0.930	61.698	0.084
D+Lr	4.997	4.700	Bottom	1.652	Min. Percent	0.930	61.698	0.081
D+Lr	4.815	4.719	Bottom	1.652	Min. Percent	0.930	61.698	0.078
D+Lr	4.634	4.738	Bottom	1.652	Min. Percent	0.930	61.698	0.075
D+Lr	4.457	4.757	Bottom	1.652	Min. Percent	0.930	61.698	0.072
D+Lr	4.284	4.776	Bottom	1.652	Min. Percent	0.930	61.698	0.069
D+Lr	4.113	4.795	Bottom	1.652	Min. Percent	0.930	61.698	0.067
D+Lr	3.946	4.814	Bottom	1.652	Min. Percent	0.930	61.698	0.064
D+Lr	3.783	4.833	Bottom	1.652	Min. Percent	0.930	61.698	0.061
D+Lr	3.622	4.852	Bottom	1.652	Min. Percent	0.930	61.698	0.059
D+Lr	3.465	4.871	Bottom	1.652	Min. Percent	0.930	61.698	0.056
D+Lr	3.312	4.890	Bottom	1.652	Min. Percent	0.930	61.698	0.054
D+Lr	3.162	4.909	Bottom	1.652	Min. Percent	0.930	61.698	0.051
D+Lr	3.015	4.928	Bottom	1.652	Min. Percent	0.930	61.698	0.049
D+Lr	2.872	4.947	Bottom	1.652	Min. Percent	0.930	61.698	0.047
D+Lr	2.732	4.966	Bottom	1.652	Min. Percent	0.930	61.698	0.044
D+Lr	2.595	4.985	Bottom	1.652	Min. Percent	0.930	61.698	0.042
D+Lr	2.462	5.004	Bottom	1.652	Min. Percent	0.930	61.698	0.040
D+Lr	2.332	5.023	Bottom	1.652	Min. Percent	0.930	61.698	0.038
D+Lr	2.206	5.042	Bottom	1.652	Min. Percent	0.930	61.698	0.036
D+Lr	2.083	5.061	Bottom	1.652	Min. Percent	0.930	61.698	0.034
D+Lr	1.963	5.080	Bottom	1.652	Min. Percent	0.930	61.698	0.032
D+Lr	1.847	5.099	Bottom	1.652	Min. Percent	0.930	61.698	0.030
D+Lr	1.735	5.118	Bottom	1.652	Min. Percent	0.930	61.698	0.028
D+Lr	1.626	5.137	Bottom	1.652	Min. Percent	0.930	61.698	0.026
D+Lr	1.520	5.156	Bottom	1.652	Min. Percent	0.930	61.698	0.025
D+Lr	1.418	5.175	Bottom	1.652	Min. Percent	0.930	61.698	0.023
D+Lr	1.320	5.194	Bottom	1.652	Min. Percent	0.930	61.698	0.021
D+Lr	1.224	5.213	Bottom	1.652	Min. Percent	0.930	61.698	0.020
D+Lr	1.133	5.232	Bottom	1.652	Min. Percent	0.930	61.698	0.018
D+Lr	1.045	5.251	Bottom	1.652	Min. Percent	0.930	61.698	0.017
D+Lr	0.960	5.270	Bottom	1.652	Min. Percent	0.930	61.698	0.016
D+Lr	0.879	5.289	Bottom	1.652	Min. Percent	0.930	61.698	0.014

Combined Footing Design

Description : Combining (E) Footing and (N) Footing to resist vertical Loads - Line A

Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu	Distance from left	Tension Side	As Req'd	Governed by	Actual As	Phi*Mn	Mu / PhiMn
D+Lr	0.801	5.308	Bottom	1.652	Min. Percent	0.930	61.698	0.013
D+Lr	0.727	5.327	Bottom	1.652	Min. Percent	0.930	61.698	0.012
D+Lr	0.657	5.346	Bottom	1.652	Min. Percent	0.930	61.698	0.011
D+Lr	0.590	5.366	Bottom	1.652	Min. Percent	0.930	61.698	0.010
D+Lr	0.527	5.385	Bottom	1.652	Min. Percent	0.930	61.698	0.009
D+Lr	0.467	5.404	Bottom	1.652	Min. Percent	0.930	61.698	0.008
D+Lr	0.410	5.423	Bottom	1.652	Min. Percent	0.930	61.698	0.007
D+Lr	0.358	5.442	Bottom	1.652	Min. Percent	0.930	61.698	0.006
D+Lr	0.309	5.461	Bottom	1.652	Min. Percent	0.930	61.698	0.005
D+Lr	0.263	5.480	Bottom	1.652	Min. Percent	0.930	61.698	0.004
D+Lr	0.221	5.499	Bottom	1.652	Min. Percent	0.930	61.698	0.004
D+Lr	0.183	5.518	Bottom	1.652	Min. Percent	0.930	61.698	0.003
D+Lr	0.148	5.537	Bottom	1.652	Min. Percent	0.930	61.698	0.002
D+Lr	0.117	5.556	Bottom	1.652	Min. Percent	0.930	61.698	0.002
D+Lr	0.090	5.575	Bottom	1.652	Min. Percent	0.930	61.698	0.001
D+Lr	0.066	5.594	Bottom	1.652	Min. Percent	0.930	61.698	0.001
D+Lr	0.046	5.613	Bottom	1.652	Min. Percent	0.930	61.698	0.001
D+Lr	0.029	5.632	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.017	5.651	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.007	5.670	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.002	5.689	Bottom	1.652	Min. Percent	0.930	61.698	0.000
D+Lr	0.000	5.708	0	0.000	0	0.000	0.000	0.000

	One Way Shear			Punching Shear		
Load Combination...	Phi Vn	vu @ Col #1	vu @ Col #2	Phi Vn	vu @ Col #1	vu @ Col #2
+1.40D	85.00 psi	3.99 psi	4.44 psi	170.00 psi	1.56 psi	0.58 psi
+1.20D+0.50Lr+1.60L+1.60H	85.00 psi	4.50 psi	3.72 psi	170.00 psi	2.94 psi	1.87 psi
+1.20D+1.60Lr+0.50L	85.00 psi	6.87 psi	3.51 psi	170.00 psi	6.46 psi	4.90 psi

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzall - Pad Ftg Retrofit   Date/Time : 2/1/2010 11:53:29 AM

1) Input

Calculation Method : ACI 318 Appendix D For Cracked Concrete

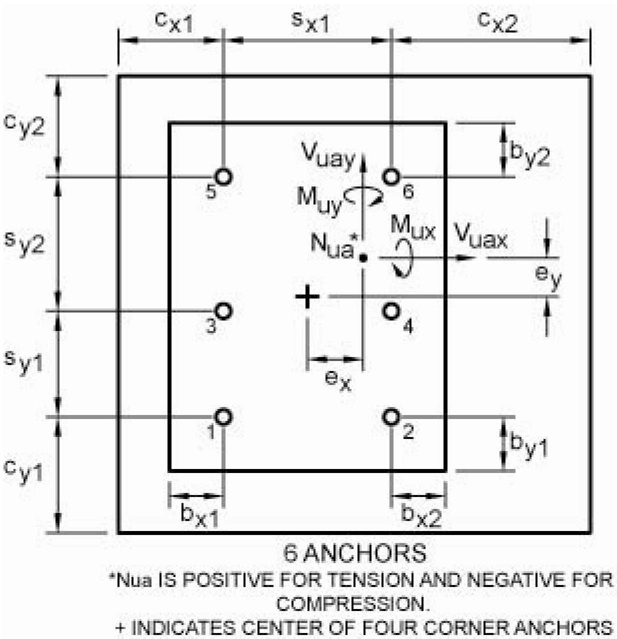
Calculation Type : Analysis

a) Layout

Anchor : 5/8" SET-XP   Number of Anchors : 6

Steel Grade: A307 GR. C   Embedment Depth : 5 in

Built-up Grout Pads : No



Anchor Layout Dimensions :

c_{x1} : 3.3125 in

c_{x2} : 3.3125 in

c_{y1} : 3.3125 in

c_{y2} : 3.3125 in

b_{x1} : 1 in

b_{x2} : 1 in

b_{y1} : 1 in

b_{y2} : 1 in

s_{x1} : 17.375 in

s_{y1} : 22.18 in

s_{y2} : 22.128 in

b) Base Material

Concrete : Normal weight

f'_c : 2500.0 psi

Cracked Concrete : Yes

$\Psi_{c,V}$: 1.00

Condition : B tension and shear

ϕF_p : 1381.3 psi

Thickness, h : 24 in
Supplementary edge reinforcement : No
Hole Condition : Dry Concrete
Inspection : Continuous
Temperature Range : 1 (Maximum 110 ^oF short term and 75 ^oF long term temp.)

c) Factored Loads

Load factor source : ACI 318 Section 9.2

N_{ua} : 0 lb

V_{uax} : 0 lb

V_{uay} : 0 lb

M_{ux} : 0 lb*ft

M_{uy} : 8446 lb*ft

e_x : 0 in

e_y : 0 in

Moderate/high seismic risk or intermediate/high design category : No
Anchor w/ sustained tension : No
Anchors only resist wind and/or seismic loads : No
Apply entire shear load at front row for breakout : No

d) Anchor Parameters

From C-SAS-2009:

Anchor Model = SETXP	d_o = 0.625 in
Category = 1	h_{ef} = 5 in
h_{min} = 8.125 in	c_{ac} = 15 in
c_{min} = 1.75 in	s_{min} = 3 in

Ductile = Yes

2) Tension Force on Each Individual Anchor

Anchor #1 N_{ua1} = 1881.00 lb

Anchor #2 N_{ua2} = 17.27 lb

Anchor #3 N_{ua3} = 1881.00 lb

Anchor #4 N_{ua4} = 17.27 lb

Anchor #5 N_{ua5} = 1881.00 lb

Anchor #6 N_{ua6} = 17.27 lb

Sum of Anchor Tension ΣN_{ua} = 5694.80 lb

a_x = 18.54 in

a_y = 0.00 in

e'_{Nx} = 8.53 in

e'_{Ny} = 0.00 in

3) Shear Force on Each Individual Anchor

Resultant shear forces in each anchor:

Anchor #1 V_{ua1} = 0.00 lb (V_{ua1x} = 0.00 lb , V_{ua1y} = 0.00 lb)

Anchor #2 V_{ua2} = 0.00 lb (V_{ua2x} = 0.00 lb , V_{ua2y} = 0.00 lb)

Anchor #3 V_{ua3} = 0.00 lb (V_{ua3x} = 0.00 lb , V_{ua3y} = 0.00 lb)

Anchor #4 V_{ua4} = 0.00 lb (V_{ua4x} = 0.00 lb , V_{ua4y} = 0.00 lb)

Anchor #5 V_{ua5} = 0.00 lb (V_{ua5x} = 0.00 lb , V_{ua5y} = 0.00 lb)

Anchor #6 V_{ua6} = 0.00 lb (V_{ua6x} = 0.00 lb , V_{ua6y} = 0.00 lb)

Sum of Anchor Shear ΣV_{uax} = 0.00 lb, ΣV_{uay} = 0.00 lb

e'_{Vx} = 0.00 in

e'_{Vy} = 0.00 in

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

$N_{sa} = nA_{se}f_{uta}$ [Eq. D-3]

Number of anchors acting in tension, n = 6

N_{sa} = 13110 lb (for each individual anchor) [C-SAS-2009]

ϕ = 0.75 [D.4.4]

ϕN_{sa} = 9832.50 lb (for each individual anchor)

5) Concrete Breakout Strength of Anchor Group in Tension [Sec. D.5.2]

$N_{cbg} = A_{Nc}/A_{Nco}\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ [Eq. D-5]

Number of influencing edges = 4

h_{ef} = 5 in

A_{Nco} = 225.00 in^2 [Eq. D-6]

A_{Nc} = 792.02 in^2

$\Psi_{ec,Nx}$ = 0.4679 [Eq. D-9]

$\Psi_{ec,Ny}$ = 1.0000 [Eq. D-9]

$\Psi_{ec,N}$ = 0.4679 (Combination of x-axis & y-axis eccentricity factors.)

Smallest edge distance, $c_{a,min}$ = 3.31 in

$\Psi_{ed,N}$ = 0.8325 [Eq. D-10 or D-11]

Note: Cracking shall be controlled per D.5.2.6

$\Psi_{c,N}$ = 1.0000 [Sec. D.5.2.6]

$\Psi_{cp,N}$ = 1.0000 [Eq. D-12 or D-13]

$N_b = k_c \sqrt{f'_c}\, h_{ef}^{1.5}$ = 9503.29 lb [Eq. D-7]

k_c = 17 [Sec. D.5.2.6]

N_{cbg} = 13030.25 lb [Eq. D-5]

ϕ = 0.65 [D.4.4]

ϕN_{cbg} = 8469.66 lb (for the anchor group)

6) Adhesive Strength of Anchor in Tension [Sec. D.5.3 (AC308 Sec.3.3)]

$\tau_{k,cr}$ = 718 psi [C-SAS-2009]

$\tau_{k,max,cr} = k_{cr} \sqrt{h_{ef}} \sqrt{f'_c} / (\pi\, d_o)$ [Eq. D-16i]

k_{cr} = 17 [C-SAS-2009]

h_{ef} (unadjusted) = 5 in

$\tau_{k,max,cr}$ = 968.00 psi

$N_{ao} = \tau_{k,cr} \pi\, d_o h_{ef}$ = 7048.95 lb [Eq. D-16f]

$\tau_{k,uncr}$ = 2263.00 psi for use in [Eq. D-16d]

$s_{cr,Na} = \min[20 d_o \sqrt{(\tau_{k,uncr}/1450)}\ , 3h_{ef}]$ = 15.000 in [Eq. D-16d]

$c_{cr,Na} = s_{cr,Na}/2$ = 7.500 in [Eq. D-16e]

$N_{ag} = A_{Na}/A_{Nao} \Psi_{ed,Na} \Psi_{g,Na} \Psi_{ec,Na} \Psi_{p,Na} N_{ao}$ [Eq. D-16b]

A_{Nao} = 225.00 in^2 [Eq. D-16c]

A_{Na} = 792.02 in^2

$\Psi_{ec,Nax} = 1/(1+2e'_{Nx}/s_{cr,Na})$ = 0.4679 [Eq. D-16j]

$\Psi_{ec,Nay} = 1/(1+2e'_{Ny}/s_{cr,Na})$ = 1.0000 [Eq. D-16j]

$\Psi_{ec,Na}$ = 0.4679 (Combination of x-axis and y-axis eccentricity factors.)

Smallest edge distance, $c_{a,min}$ = 3.31 in

$\Psi_{ed,Na} = \min[0.7+0.3c_{a,min}/c_{cr,Na}\ , 1.0]$ = 0.8325 [Eq. D-16m]

$\Psi_{p,Na}$ = 1.0000 [Sec. D.5.3.14]

$\Psi_{g,Na} = \Psi_{g,Nao}+(s/s_{cr,Na})^{0.5}(1-\Psi_{g,Nao})$ [Eq. D-16g]

s = 15.000 in (largest spacing limited by $s_{cr,Na}$)

$\Psi_{g,Nao} = \max\{\sqrt{n} - [(\sqrt{n} - 1)(\tau_{k,cr}/\tau_{k,max,cr})^{1.5}], 1.0\}$ [Eq. D-16h]

$\Psi_{g,Nao} = 1.5235$

$\Psi_{g,Na} = 1.0000$

N_{ag} = 9665.03 lb [Eq. D-16b]

$\phi = 0.65$ [C-SAS-2009]

ϕN_{ag} = 6282.27 lb (for the anchor group)

7) Side Face Blowout of Anchor in Tension [Sec. D.5.4]

Concrete side face blowout strength is only calculated for headed anchors in tension close to an edge, $c_{a1} < 0.4h_{ef}$. Not applicable in this case.

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

$V_{sa} = n0.6A_{se}f_{uta}$ [Eq. D-20]

V_{sa} = 7865.00 lb (for each individual anchor) [C-SAS-2009]

$\phi = 0.65$ [D.4.4]

ϕV_{sa} = 5112.25 lb (for each individual anchor)

9) Concrete Breakout Strength of Anchor Group in Shear [Sec D.6.2]

Case 1: Anchor(s) closest to edge checked against sum of anchor shear loads at the edge
In x-direction...

$V_{cbgx} = A_{vcx}/A_{vcox}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-22]

c_{a1} = 3.31 in

A_{vcx} = 131.67 in^2

A_{vcox} = 49.38 in^2 [Eq. D-23]

$\Psi_{ec,V} = 1.0000$ [Eq. D-26]

$\Psi_{ed,V} = 0.9000$ [Eq. D-27 or D-28]

$\Psi_{c,V} = 1.0000$ [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{bx} = 2528.48 lb

V_{cbgx} = 6068.36 lb [Eq. D-22]

$\phi = 0.70$

ϕV_{cbgx} = 4247.85 lb (for the anchor group)

In y-direction...

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 3.31 in

A_{vcy} = 82.29 in^2

A_{vcoy} = 49.38 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 0.9000 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{by} = 2528.48 lb

V_{cbgy} = 3792.72 lb [Eq. D-22]

ϕ = 0.70

ϕV_{cbgy} = 2654.91 lb (for the anchor group)

Case 2: Anchor(s) furthest from edge checked against total shear load
In x-direction...

$V_{cbgx} = A_{vcx}/A_{vcox}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-22]

c_{a1} = 16.00 in (adjusted for edges per D.6.2.4)

A_{vcx} = 1222.39 in^2

A_{vcox} = 1152.00 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 0.7414 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{bx} = 26841.45 lb

V_{cbgx} = 21116.42 lb [Eq. D-22]

ϕ = 0.70

ϕV_{cbgx} = 14781.49 lb (for the entire anchor group)

In y-direction...

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 16.00 in (adjusted for edges per D.6.2.4)

A_{vcy} = 576.00 in^2

A_{vcoy} = 1152.00 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 0.7414 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{by} = 26841.45 lb

V_{cbgy} = 9950.21 lb [Eq. D-22]

ϕ = 0.70

ϕV_{cbgy} = 6965.15 lb (for the entire anchor group)

Case 3: Anchor(s) closest to edge checked for parallel to edge condition

Check anchors at c_{x1} edge

$V_{cbgx} = A_{vcx}/A_{vcox}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-22]

c_{a1} = 3.31 in

A_{vcx} = 131.67 in^2

A_{vcox} = 49.38 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{bx} = 2528.48 lb

V_{cbgx} = 6742.62 lb [Eq. D-22]

$V_{cbgy} = 2 * V_{cbgx}$ [Sec. D.6.2.1(c)]

V_{cbgy} = 13485.24 lb

ϕ = 0.70

ϕV_{cbgy} = 9439.67 lb (for the anchor group)

Check anchors at c_{y1} edge

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 3.31 in

A_{vcy} = 82.29 in^2

A_{vcoy} = 49.38 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{\ d_o}\sqrt{\ f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{by} = 2528.48 lb

V_{cbgy} = 4214.14 lb [Eq. D-22]

$V_{cbgx} = 2 * V_{cbgy}$ [Sec. D.6.2.1(c)]

V_{cbgx} = 8428.27 lb

ϕ = 0.70

ϕV_{cbgx} = 5899.79 lb (for the anchor group)

Check anchors at c_{x2} edge

$V_{cbgx} = A_{vcx}/A_{vcox}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-22]

c_{a1} = 3.31 in

A_{vcx} = 131.67 in^2

A_{vcox} = 49.38 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Eq. D-27 or D-28] [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{\ d_o}\sqrt{\ f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{bx} = 2528.48 lb

V_{cbgx} = 6742.62 lb [Eq. D-22]

$V_{cbgy} = 2 * V_{cbgx}$ [Sec. D.6.2.1(c)]

V_{cbgy} = 13485.24 lb

ϕ = 0.70

ϕV_{cbgy} = 9439.67 lb (for the anchor group)

Check anchors at c_{y2} edge

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 3.31 in

A_{vcy} = 82.29 in^2

A_{vcoy} = 49.38 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{by} = 2528.48 lb

V_{cbgy} = 4214.14 lb [Eq. D-22]

$V_{cbgx} = 2 * V_{cbgy}$ [Sec. D.6.2.1(c)]

V_{cbgx} = 8428.27 lb

ϕ = 0.70

ϕV_{cbgx} = 5899.79 lb (for the anchor group)

10) Concrete Pryout Strength of Anchor Group in Shear [Sec. D.6.3]

$V_{cpg} = \min[k_{cp}N_{ag}, k_{cp}N_{cbg}]$ [Eq. D-30b]

k_{cp} = 2 [Sec. D.6.3.2]

e_{Nx} = 0.00 in (Applied shear load eccentricity relative to anchor group c.g.)

e_{Ny} = 0.01 in (Applied shear load eccentricity relative to anchor group c.g.)

$\Psi_{ec,Nx}$ = 1.0000 [Eq. D-9] (Calulated using applied shear load eccentricity)

$\Psi_{ec,Ny}$ = 0.9988 [Eq. D-9] (Calulated using applied shear load eccentricity)

$\Psi_{ec,N'}$ = 0.9988 (Combination of x-axis & y-axis eccentricity factors)

$N_{ag} = (A_{Naa}/A_{Na})(\Psi_{ec,N'}/\Psi_{ec,Na})N_{ag}$

N_{ag} = 9665.03 lb (from Section (6) of calculations)

A_{Na} = 792.02 in^2 (from Section (6) of calculations)

A_{Naa} = 792.02 in^2 (considering all anchors)

$\Psi_{ec,Na}$ = 0.4679 (from Section(6) of calculations)

N_{ag} = 20632.80 lb (considering all anchors)

$N_{cbg} = (A_{Nca}/A_{Nc})(\Psi_{ec,N'}/\Psi_{ec,N})N_{cbg}$

N_{cbg} = 13030.25 lb (from Section (5) of calculations)

A_{Nc} = 792.02 in^2 (from Section (5) of calculations)

A_{Nca} = 792.02 in^2 (considering all anchors)

$\Psi_{ec,N}$ = 0.4679 (from Section(5) of calculations)

N_{cbg} = 27816.84 lb (considering all anchors)

V_{cpg} = 41265.61 lb

ϕ = 0.70 [D.4.4]

ϕV_{cpg} = 28885.93 lb (for the anchor group)

11) Check Demand/Capacity Ratios [Sec. D.7]

Tension

- Steel : 0.1913
- Breakout : 0.6724
- Adhesive : 0.9065
- Sideface Blowout : N/A

Shear

- Steel : 0.0000
- Breakout (case 1) : 0.0000
- Breakout (case 2) : 0.0000
- Breakout (case 3) : 0.0000
- Pryout : 0.0000

V.Max(0) <= 0.2 and T.Max(0.91) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 5/8" diameter A307 GR. C SET-XP anchor(s) with 5 in. embedment

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZALL EXPANSION 09.09-2/2
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 2/10
CHECKED BY ______ DATE ______
SCALE ______

VERTICAL DESIGN (CONT.)

FRAMES ON LINES 5,6,7,8,9 & 10 AT LINE A (CONT.)

ANCHOR BOLTS REQ'D FOR VERTICAL UPLIFT

MAX UPLIFT LOAD (STRENGTH) ⟶ 8.1k (ACI EQ. 9-6)
MAX SHEAR LOAD AT MAX UPLIFT ⟶ 2.9k (TOWARDS SLAB)
WIND
MAX SHEAR LOAD TOWARDS SLAB EDGE ⟶ 5.3k w/ 7.1k COMPRESSION LOAD

MAX UPLIFT LOAD (STRENGTH) ⟶ ϕ (ACI EQ 9-7)
MAX SHEAR LOAD (STRENGTH) ⟶ 5.2k (TOWARDS SLAB) (ACI EQ 9-7)
SEISMIC ⟶ 0.4k (TOWARDS SLAB EDGE) (ACI EQ 9-7)

USE: (4) 3/4" ϕ F1554 GRADE 36 ANCHOR BOLTS w/ A MIN EMBED OF 6" & #3 HAIRPINS AT EA PAIR OF ANCHOR BOLTS (SEE HAIRPIN DESIGN IN THIS CALC SET)

SEE 'SIMPSON ANCHOR DESIGNER FOR ACI 318' RESULTS

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 5-10 @ A (W) Date/Time : 2/4/2010 4:43:37 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
3/4" Heavy Hex Bolt	F1554 GR. 36	4	6	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
A tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
8100	2900	0	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)	N_{ua2} (lb)	N_{ua3} (lb)	N_{ua4} (lb)
2025.00	2025.00	2025.00	2025.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)	V_{ua2} (lb)	V_{ua3} (lb)	V_{ua4} (lb)
725.00	725.00	725.00	725.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
19370	14527.50	2025.00	0.1394

Concrete Breakout (Φ = 0.75)

N_{cbg}(lb)	ΦN_{cbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua}/\Phi N_{cbg}$
13070.34	9802.76	8100.00	0.8263

Pullout (Φ = 0.70)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	$N_{ua} / \Phi N_{pn}$
18220.00	12754.00	2025.00	0.1588

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.65)

V_{sa}(lb)	ΦV_{sa} (lb)	V_{ua}(lb)	$V_{ua} / \Phi V_{sa}$
11625	7556.25	725.00	0.0959

Concrete Breakout has not been evaluated per user option.

Pryout (Φ = 0.70)

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uax}(lb)	$\Sigma V_{uax} / \Phi V_{cpg}$
26140.68	18298.48	2900	0.1585

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uay}(lb)	$\Sigma V_{uay} / \Phi V_{cpg}$	$\Sigma V_{ua} / \Phi V_{cpg}$
26140.68	18298.48	0	0.0000	0.1585

Interaction check

V.Max(0.16) <= 0.2 and T.Max(0.83) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 6 in. embedment

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 5-10 @ A (W) Date/Time : 2/4/2010 4:47:24 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
3/4" Heavy Hex Bolt	F1554 GR. 36	4	6	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
A tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
-7100	-5300	0	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)	N_{ua2} (lb)	N_{ua3} (lb)	N_{ua4} (lb)
0.00	0.00	0.00	0.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)	V_{ua2} (lb)	V_{ua3} (lb)	V_{ua4} (lb)
1325.00	1325.00	1325.00	1325.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua} / \Phi N_{sa}$
19370	14527.50	0.00	0.0000

Concrete Breakout (Φ = 0.75)

N_{cbg}(lb)	ΦN_{cbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua} / \Phi N_{cbg}$
13070.34	9802.76	0.00	0.0000

Pullout (Φ = 0.70)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	N_{ua} /ΦN_{pn}
18220.00	12754.00	0.00	0.0000

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.65)

V_{sa}(lb)	ΦV_{sa} (lb)	V_{ua}(lb)	V_{ua} /ΦV_{sa}
11625	7556.25	1325.00	0.1754

Concrete Breakout has not been evaluated per user option.

Pryout (Φ = 0.70)

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uax}(lb)	ΣV_{uax} /ΦV_{cpg}
26140.68	18298.48	-5300	0.2896

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uay}(lb)	ΣV_{uay} /ΦV_{cpg}	ΣV_{ua} /ΦV_{cpg}
26140.68	18298.48	0	0.0000	0.2896

Interaction check

T.Max(0) <= 0.2 and V.Max(0.29) <= 1.0 [Sec D.7.2]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 6 in. embedment

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 5-10 @ A (EQ)   Date/Time : 2/4/2010 4:48:57 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
3/4" Heavy Hex Bolt	F1554 GR. 36	4	6	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
A tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
0	5200	0	0	0

e_x(in)	e_y(in)	Mod/high seismic	Apply entire shear @ front row
0	0	Yes	No

Individual Anchor Tension Loads

N_{ua1} (lb)	N_{ua2} (lb)	N_{ua3} (lb)	N_{ua4} (lb)
0.00	0.00	0.00	0.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)	V_{ua2} (lb)	V_{ua3} (lb)	V_{ua4} (lb)
1300.00	1300.00	1300.00	1300.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

19370>2.5x(5200)

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
19370	14527.50	0.00	0.0000

Concrete Breakout (Φ = 0.75 , Φ_{seis} = 0.75)

N_{cbg}(lb)	ΦN_{cbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua}/\Phi N_{cbg}$
13070.34	7352.07	0.00	0.0000

Pullout (Φ = 0.70 , Φ_{seis} = 0.75)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{pn}$
18220.00	9565.50	0.00	0.0000

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.65)

V_{eq}(lb)	ΦV_{eq}(lb)	V_{ua}(lb)	$V_{ua}/\Phi V_{eq}$
11625	7556.25	1300.00	0.1720

Concrete Breakout has not been evaluated per user option.

Pryout (Φ = 0.70 , Φ_{seis} = 0.75)

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uax}(lb)	$\Sigma V_{uax}/\Phi V_{cpg}$
26140.68	13723.86	5200	0.3789

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uay}(lb)	$\Sigma V_{uay}/\Phi V_{cpg}$	$\Sigma V_{ua}/\Phi V_{cpg}$
26140.68	13723.86	0	0.0000	0.3789

Interaction check

T.Max(0) <= 0.2 and V.Max(0.38) <= 1.0 [Sec D.7.2]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 6 in. embedment

BRITTLE FAILURE GOVERNS: Governing anchor failure mode is brittle failure. Per 2006 IBC Section 1908.1.16, anchors shall be governed by a ductile steel element in structures assigned to Seismic Design Category C, D, E, or F. Alternatively the minimum design strength of the anchor(s) shall be at least 2.5 times the factored forces or the anchor attachment to the structure shall undergo ductile yielding at a load level less than the design strength of the anchor(s). Designer must exercise own judgement to determine if this design is suitable.

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 5-10 @ A (EQ)　　　Date/Time : 2/4/2010 4:51:31 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
3/4" Heavy Hex Bolt	F1554 GR. 36	4	6	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
A tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
0	-400	0	0	0

e_x(in)	e_y(in)	Mod/high seismic	Apply entire shear @ front row
0	0	Yes	No

Individual Anchor Tension Loads

N_{ua1} (lb)	N_{ua2} (lb)	N_{ua3} (lb)	N_{ua4} (lb)
0.00	0.00	0.00	0.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)	V_{ua2} (lb)	V_{ua3} (lb)	V_{ua4} (lb)
100.00	100.00	100.00	100.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

19370>>>2.5x400

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
19370	14527.50	0.00	0.0000

Concrete Breakout (Φ = 0.75 , Φ_{seis} = 0.75)

N_{cbg}(lb)	ΦN_{cbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua}/\Phi N_{cbg}$
13070.34	7352.07	0.00	0.0000

Pullout (Φ = 0.70 , Φ_{seis} = 0.75)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	N_{ua} /ΦN_{pn}
18220.00	9565.50	0.00	0.0000

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.65)

V_{eq}(lb)	ΦV_{eq}(lb)	V_{ua}(lb)	V_{ua} /ΦV_{eq}
11625	7556.25	100.00	0.0132

Concrete Breakout has not been evaluated per user option.

Pryout (Φ = 0.70 , Φ_{seis} = 0.75)

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uax}(lb)	ΣV_{uax} /ΦV_{cpg}
26140.68	13723.86	-400	0.0291

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uay}(lb)	ΣV_{uay} /ΦV_{cpg}	ΣV_{ua} /ΦV_{cpg}
26140.68	13723.86	0	0.0000	0.0291

Interaction check

V.Max(0.03) <= 0.2 and T.Max(0) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 6 in. embedment

BRITTLE FAILURE GOVERNS: Governing anchor failure mode is brittle failure. Per 2006 IBC Section 1908.1.16, anchors shall be governed by a ductile steel element in structures assigned to Seismic Design Category C, D, E, or F. Alternatively the minimum design strength of the anchor(s) shall be at least 2.5 times the factored forces or the anchor attachment to the structure shall undergo ductile yielding at a load level less than the design strength of the anchor(s). Designer must exercise own judgement to determine if this design is suitable.

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZALL EXPANSION 09.9-2/2
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 2/10
CHECKED BY ______ DATE ______
SCALE ______

VERTICAL DESIGN (CONT.)

FRAMES ON LINES 4 & 11 AT LINE W

MAX COMPRESSIVE LOAD (ASD) ⟶ 9.9 k (ASCE EQ. 3)
PAD FOOTING REQUIRED FOR MAX COMPRESSIVE LOAD:

USE: 2'3" SQ × 18" DP (MIN) w/
(2)-#5 TOP & BTM

SEE ENERCALC RESULTS

ANCHOR BOLTS REQ'D FOR VERTICAL UPLIFT

MAX UPLIFT LOAD (STRENGTH) ⟶ 9.6 k (ACI EQ. 9-6)
MAX SHEAR LOAD AT MAX UP ⟶ 4.6 k (TOWARDS SLAB)
WIND LOADING
MAX SHEAR LOAD TOWARDS SLAB EDGE → 6.4k w/ 4.7k COMPRESSIVE LOAD

MAX UPLIFT LOAD (STRENGTH) ⟶ 0 (ACI EQ 9-7)
MAX SHEAR LOAD (STRENGTH) ⟶ 3.3 k (TOWARDS SLAB) (ACI EQ 9-7)
SEISMIC ⟶ 0.3 k (TOWARDS SLAB EDGE) (ACI EQ 9-7)
SEE DESIGN FOR ANCHORS ON LINES 5-10 @ W

USE: (4) 3/4"⌀ F1554 GRADE 36 ANCHOR BOLTS w/ A MIN EMBED OF 6" & #3 HAIRPINS AT EA. PAIR OF ANCHOR BOLTS (SEE HAIRPIN DESIGN IN THIS CALC SET.)

SEE 'SIMPSON ANCHOR DESIGNER FOR ACI 318' RESULTS

General Footing Design

Description : Pad Footing at Lines 4 & 11 - Line W

General Information

Calculations per IBC 2006, CBC 2007, ACI 318-05

Material Properties

f'c : Concrete 28 day strength	=	2.50 ksi
Fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	2,850.0 ksi
Concrete Density	=	145.0 pcf
Φ Values Flexure	=	0.90
Shear	=	0.850

Analysis Settings

Min Steel % Bending Reinf.	=	0.00140
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.50 : 1
Min. Sliding Safety Factor	=	1.50 : 1
AutoCalc Footing Weight as DL	:	No
AutoCalc Pedestal Weight as DL	:	No

Soil Design Values

Allowable Soil Bearing	=	2.0 ksf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	350.0 pcf
Soil/Concrete Friction Coeff.	=	0.350

Increases based on footing Depth

Reference Depth below Surface	=	0.0 ft
Allow. Pressure Increase per foot of depth	=	0.0 ksf
when base footing is below	=	0.0 ft

Increases based on footing Width

Allow. Pressure Increase per foot of width	=	0.0 ksf
when footing is wider than	=	0.0 ft

Dimensions

Width along X-X Axis	=	2.250 ft
Length along Z-Z Axi	=	2.250 ft
Footing Thicknes	=	18.0 in

Load location offset from footing center...

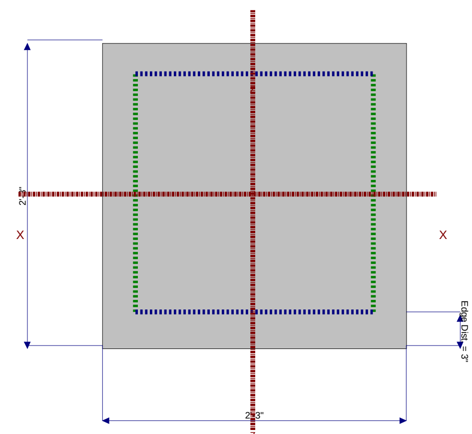
ex : Along X-X Axis	=	0 in
ez : Along Z-Z Axis	=	0 in

Pedestal dimensions...

px : Along X-X Axis	=	0.0 in
pz : Along Z-Z Axis	=	0.0 in
Height	=	0.0 in

Rebar Centerline to Edge of Concrete..

at Bottom of footing	=	3.313 in



Reinforcing

Bars along X-X Axis

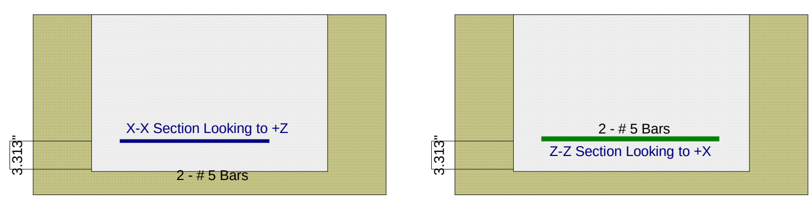
Number of Bars	=	2.0
Reinforcing Bar Size	=	# 5

Bars along Z-Z Axis

Number of Bars	=	2.0
Reinforcing Bar Size	=	# 5

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a



Applied Loads

		D	Lr	L	S	W	E	H
P : Column Load	=	5.20	4.70	0.0	0.0	0.0	1.90	0.0 k
OB : Overburden	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0 ksf
M-xx	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0 k-ft
M-zz	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0 k-ft
V-x	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0 k
V-z	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0 k

General Footing Design

Description : Pad Footing at Lines 4 & 11 - Line W

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.9780	Soil Bearing	1.956 ksf	2.0 ksf	+D+Lr+H
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.09656	Z Flexure (+X)	1.720 k-ft	17.810 k-ft	+1.20D+1.60Lr+0.50L
PASS	0.09656	Z Flexure (-X)	1.720 k-ft	17.810 k-ft	+1.20D+1.60Lr+0.50L
PASS	0.09656	X Flexure (+Z)	1.720 k-ft	17.810 k-ft	+1.20D+1.60Lr+0.50L
PASS	0.09656	X Flexure (-Z)	1.720 k-ft	17.810 k-ft	+1.20D+1.60Lr+0.50L
PASS	n/a	1-way Shear (+X)	0.0 psi	85.0 psi	n/a
PASS	0.0	1-way Shear (-X)	0.0 psi	0.0 psi	n/a
PASS	n/a	1-way Shear (+Z)	0.0 psi	85.0 psi	n/a
PASS	n/a	1-way Shear (-Z)	0.0 psi	85.0 psi	n/a
PASS	n/a	2-way Punching	11.182 psi	85.0 psi	+1.20D+1.60Lr+0.50L

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc	Zecc	Actual Soil Bearing Stress +Z	+Z	-X	-X	Actual / Allowable Ratio
X-X, +D	2.0	n/a	0.0	1.027	1.027	n/a	n/a	0.514
X-X, +D+L+H	2.0	n/a	0.0	1.027	1.027	n/a	n/a	0.514
X-X, +D+Lr+H	2.0	n/a	0.0	1.956	1.956	n/a	n/a	0.978
X-X, +D+0.750Lr+0.750L+H	2.0	n/a	0.0	1.723	1.723	n/a	n/a	0.862
X-X, +D+0.70E+H	2.0	n/a	0.0	1.290	1.290	n/a	n/a	0.645
X-X, +D+0.750Lr+0.750L+0.5250E+H	2.0	n/a	0.0	1.920	1.920	n/a	n/a	0.960
X-X, +D+0.750L+0.750S+0.5250E+H	2.0	n/a	0.0	1.224	1.224	n/a	n/a	0.612
X-X, +0.60D+0.70E+H	2.0	n/a	0.0	0.8790	0.8790	n/a	n/a	0.440
Z-Z, +D	2.0	0.0	n/a	n/a	n/a	1.027	1.027	0.514
Z-Z, +D+L+H	2.0	0.0	n/a	n/a	n/a	1.027	1.027	0.514
Z-Z, +D+Lr+H	2.0	0.0	n/a	n/a	n/a	1.956	1.956	0.978
Z-Z, +D+0.750Lr+0.750L+H	2.0	0.0	n/a	n/a	n/a	1.723	1.723	0.862
Z-Z, +D+0.70E+H	2.0	0.0	n/a	n/a	n/a	1.290	1.290	0.645
Z-Z, +D+0.750Lr+0.750L+0.5250E+H	2.0	0.0	n/a	n/a	n/a	1.920	1.920	0.960
Z-Z, +D+0.750L+0.750S+0.5250E+H	2.0	0.0	n/a	n/a	n/a	1.224	1.224	0.612
Z-Z, +0.60D+0.70E+H	2.0	0.0	n/a	n/a	n/a	0.8790	0.8790	0.440

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status

Footing Has NO Overturning

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Sliding SafetyRatio	Status

Footing Has NO Sliding

Footing Flexure

Footing Flexure Load Combination...	Mu	Which Side ?	Tension @ Bot. or Top ?	As Req'd	Gvrn. As	Actual As	Phi*Mn	Status
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK

General Footing Design

Description : Pad Footing at Lines 4 & 11 - Line W

Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

General Footing Design

Description : Pad Footing at Lines 4 & 11 - Line W

Footing Flexure

Footing Flexure Load Combination...	Mu	Which Side ?	Tension @ Bot. or Top ?	As Req'd	Gvrn. As	Actual As	Phi*Mn	Status
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.8224 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Phi*Vn / Vu	Status
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK

Punching Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+0.90D+E+1.60H	5.347 psi	170.0 psi	0.03145	OK
+0.90D+E+1.60H	5.347 psi	170.0 psi	0.03145	OK
+0.90D+E+1.60H	5.347 psi	170.0 psi	0.03145	OK
+0.90D+E+1.60H	5.347 psi	170.0 psi	0.03145	OK
+0.90D+E+1.60H	5.347 psi	170.0 psi	0.03145	OK

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 4 & 11 @ W Date/Time : 2/1/2010 6:19:05 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
3/4" Heavy Hex Bolt	F1554 GR. 36	4	6	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
A tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
9600	0	4600	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)	N_{ua2} (lb)	N_{ua3} (lb)	N_{ua4} (lb)
2400.00	2400.00	2400.00	2400.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)	V_{ua2} (lb)	V_{ua3} (lb)	V_{ua4} (lb)
1150.00	1150.00	1150.00	1150.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
19370	14527.50	2400.00	0.1652

Concrete Breakout (Φ = 0.75)

N_{cbg}(lb)	ΦN_{cbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua}/\Phi N_{cbg}$
20457.93	15343.45	9600.00	0.6257

Pullout ($\Phi = 0.70$)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	$N_{ua} / \Phi N_{pn}$
18220.00	12754.00	2400.00	0.1882

Side-Face Blowout does not apply

Shear Strengths

Steel ($\Phi = 0.65$)

V_{sa}(lb)	ΦV_{sa} (lb)	V_{ua}(lb)	$V_{ua} / \Phi V_{sa}$
11625	7556.25	1150.00	0.1522

Concrete Breakout has not been evaluated per user option.

Pryout ($\Phi = 0.70$)

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uax}(lb)	$\Sigma V_{uax} / \Phi V_{cpg}$
40915.85	28641.10	0	0.0000

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uay}(lb)	$\Sigma V_{uay} / \Phi V_{cpg}$	$\Sigma V_{ua} / \Phi V_{cpg}$
40915.85	28641.10	4600	0.1606	0.1606

Interaction check

V.Max(0.16) <= 0.2 and T.Max(0.63) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 6 in. embedment

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 4 & 11 @ W

Date/Time : 2/1/2010 6:20:38 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
3/4" Heavy Hex Bolt	F1554 GR. 36	4	6	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
A tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
-4700	0	-6400	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)	N_{ua2} (lb)	N_{ua3} (lb)	N_{ua4} (lb)
0.00	0.00	0.00	0.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)	V_{ua2} (lb)	V_{ua3} (lb)	V_{ua4} (lb)
1600.00	1600.00	1600.00	1600.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua} / \Phi N_{sa}$
19370	14527.50	0.00	0.0000

Concrete Breakout (Φ = 0.75)

N_{cbg}(lb)	ΦN_{cbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua} / \Phi N_{cbg}$
20457.93	15343.45	0.00	0.0000

Pullout (Φ = 0.70)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	$N_{ua} / \Phi N_{pn}$
18220.00	12754.00	0.00	0.0000

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.65)

V_{sa}(lb)	ΦV_{sa} (lb)	V_{ua}(lb)	$V_{ua} / \Phi V_{sa}$
11625	7556.25	1600.00	0.2117

Concrete Breakout has not been evaluated per user option.

Pryout (Φ = 0.70)

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uax}(lb)	$\Sigma V_{uax} / \Phi V_{cpg}$
40915.85	28641.10	0	0.0000

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uay}(lb)	$\Sigma V_{uay} / \Phi V_{cpg}$	$\Sigma V_{ua} / \Phi V_{cpg}$
40915.85	28641.10	-6400	0.2235	0.2235

Interaction check

T.Max(0) <= 0.2 and V.Max(0.22) <= 1.0 [Sec D.7.2]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 6 in. embedment

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZALL EXPANSION 09.09-2/2

SHEET NO. ______ OF ______

CALCULATED BY ______ DATE 2/10

CHECKED BY ______ DATE ______

SCALE ______

VERTICAL DESIGN (CONT.)

FRAMES ON LINES 4 & 11 AT LINE A

MAX COMPRESSIVE LOAD (ASD) → 9.9k (ASCE EQ 3)
└ NEW STRUCTURE

MAX COMPRESSIVE LOAD FROM (E) METAL BLDG

D = 0.55 k

C = 0.29 k

L = 1.16 k

ASCE EQ 3 = 1.99 k

(E) FTG, 24"W x 18" DP w/ (2)-#5 T&B,
OK FOR (N) & (E) POINT LOADS

SEE FOLLOWING CALC.

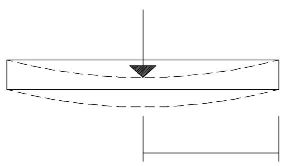
D PRODUCT 207

shawn pierce engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB	Borzall Expansion - 09.9-2/2		
SHEET NO.		OF	
CALCULATED BY	sdp	DATE	2/10
CHECKED BY		DATE	
SCALE			

Max Point Load @ (E) Footing

*Assume Cantilever Footing



L' = Length at which footing eliminates downward load

*Weight of Ftg. approx. replacing the weight of soil removed

Equations

$$M_u = \phi M_n = \phi A_s f_y \left(d - a/2\right)$$

$$\text{where ...} \quad a = \frac{A_s f_y}{.85 f'_c b} \qquad \phi = 0.9$$

$$M_{Max\ (Fixed-Guided)} = \frac{1.7\omega_{DL} L^2}{3}$$

$$\text{so...} \quad L' = \sqrt{\frac{3M_u}{1.7\omega_{DL}}} \rightarrow P_{max} = 2\omega_{DL} L'$$

$$\text{Let } F_{b(allow)} = F_b - \frac{\omega_{DL(actual)}}{b_{ftg}}$$

$$V_c = \frac{F_b L'}{2 b_{ftg} d_{ftg}} \qquad V_u = 2\phi\sqrt{f'_c}$$

$$\text{finally}: \quad P_{max} = F_{b(allow)}(2L')b_{ftg}$$

Input

Footing Information

width	24	in
height	18	in
f'_c	2500	psi
F_b	2000	psf (per soils report or CBC min.)

Reinforcing Steel

(2) #5 T&B

☐ Two Story

	ω_{DL} (psf)	ω_{LL} (psf)	Trib (ft)
$\omega_{DL\ Roof}$	78	0	5
$\omega_{DL\ Wall}$	0	0	0
$\omega_{DL\ 2nd\ Flr.}$	0	0	0

Calcs

width	24	in
depth	15	in
f'_c	2.5	ksi
f_y	60	ksi
A_s	0.62	in^2
A_{Ftg}	3.00	ft^3/ft
F_b	2000	psf

Dead Loads

Roof	390	plf
Wall	0	plf
2nd Flr	0	plf
Ftg.	n/a	plf
Slab	50	plf
Sum	*440*	plf

Calcs (cont.)

a	0.729	in
M_u	40.832	k-ft
L'	12.80	ft
$F_{b(allow)}$	1780	psf
V_c	63.3	psi
V_u	85.0	psi

Results

P_{max}	30,600	lbs

SHEAR GOVERNED

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZALL EXPANSION 09.9-2/2
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 2/10
CHECKED BY ______ DATE ______
SCALE ______

VERTICAL DESIGN (CONT.)

FRAMES ON LINES 4 & 11 AT LINE A

ANCHOR BOLTS REQ'D FOR VERTICAL UPLIFT

MAX UPLIFT LOAD (STRENGTH) ⟶ 8.1k (ACI EQ. 9-6)
MAX SHEAR LOAD AT MAX UPLIFT ⟶ 4.4k (TOWARDS SLAB EDGE)
WIND

MAX UPLIFT LOAD (STRENGTH) ⟶ ϕ (ACI EQ. 9-7)
MAX SHEAR LOAD (STRENGTH) ⟶ 3.1k (TOWARDS SLAB) (ACI EQ 9-7)
⟶ .4k (TOWARDS SLAB EDGE) (ACI EQ 9-7)
SEISMIC

USE: (4) 3/4"ϕ F1554 GRADE 36 ANCHOR BOLTS w/ A MIN EMBED OF 7" & #3 HAIRPINS AT EA. PAIR OF ANCHOR BOLTS (SEE HAIRPIN DESIGN IN THIS CALC SET)

SEE SIMPSON ANCHOR DESIGNER FOR ACI 318 RESULTS

D PRODUCT 207

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 4-11 @ A (W)   Date/Time : 2/4/2010 5:14:24 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
3/4" Heavy Hex Bolt	F1554 GR. 36	4	7	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
A tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
8100	-4400	0	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)	N_{ua2} (lb)	N_{ua3} (lb)	N_{ua4} (lb)
2025.00	2025.00	2025.00	2025.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)	V_{ua2} (lb)	V_{ua3} (lb)	V_{ua4} (lb)
1100.00	1100.00	1100.00	1100.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua} / \Phi N_{sa}$
19370	14527.50	2025.00	0.1394

Concrete Breakout (Φ = 0.75)

N_{cbg}(lb)	ΦN_{cbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua} / \Phi N_{cbg}$
14952.98	11214.74	8100.00	0.7223

Pullout (Φ = 0.70)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	N_{ua} /ΦN_{pn}
18220.00	12754.00	2025.00	0.1588

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.65)

V_{sa}(lb)	ΦV_{sa} (lb)	V_{ua}(lb)	V_{ua} /ΦV_{sa}
11625	7556.25	1100.00	0.1456

Concrete Breakout has not been evaluated per user option.

Pryout (Φ = 0.70)

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uax}(lb)	ΣV_{uax} /ΦV_{cpg}
29905.96	20934.17	-4400	0.2102

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uay}(lb)	ΣV_{uay} /ΦV_{cpg}	ΣV_{ua} /ΦV_{cpg}
29905.96	20934.17	0	0.0000	0.2102

Interaction check

T.Max(0.72) + V.Max(0.21) = 0.93 <= 1.2 [Sec D.7.3]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 7 in. embedment

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 4-11 @ A (EQ-) Date/Time : 2/4/2010 5:16:18 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
3/4" Heavy Hex Bolt	F1554 GR. 36	4	7	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
A tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
0	3100	0	0	0

e_x(in)	e_y(in)	Mod/high seismic	Apply entire shear @ front row
0	0	Yes	No

Individual Anchor Tension Loads

N_{ua1} (lb)	N_{ua2} (lb)	N_{ua3} (lb)	N_{ua4} (lb)
0.00	0.00	0.00	0.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)	V_{ua2} (lb)	V_{ua3} (lb)	V_{ua4} (lb)
775.00	775.00	775.00	775.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
19370	14527.50	0.00	0.0000

Concrete Breakout (Φ = 0.75 , Φ_{seis} = 0.75)

N_{cbg}(lb)	ΦN_{cbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua}/\Phi N_{cbg}$
14952.98	8411.05	0.00	0.0000

Pullout (Φ = 0.70 , Φ_{seis} = 0.75)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	N_{ua} /ΦN_{pn}
18220.00	9565.50	0.00	0.0000

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.65)

V_{eq}(lb)	ΦV_{eq}(lb)	V_{ua}(lb)	V_{ua} /ΦV_{eq}
11625	7556.25	775.00	0.1026

Concrete Breakout has not been evaluated per user option.

Pryout (Φ = 0.70 , Φ_{seis} = 0.75)

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uax}(lb)	ΣV_{uax} /ΦV_{cpg}
29905.96	15700.63	3100	0.1974

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uay}(lb)	ΣV_{uay} /ΦV_{cpg}	ΣV_{ua} /ΦV_{cpg}
29905.96	15700.63	0	0.0000	0.1974

Interaction check

V.Max(0.2) <= 0.2 and T.Max(0) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 7 in. embedment

BRITTLE FAILURE GOVERNS: Governing anchor failure mode is brittle failure. Per 2006 IBC Section 1908.1.16, anchors shall be governed by a ductile steel element in structures assigned to Seismic Design Category C, D, E, or F. Alternatively the minimum design strength of the anchor(s) shall be at least 2.5 times the factored forces or the anchor attachment to the structure shall undergo ductile yielding at a load level less than the design strength of the anchor(s). Designer must exercise own judgement to determine if this design is suitable.

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BOREAL EXPANSION 09.9.2/2
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 2/10
CHECKED BY ______ DATE ______
SCALE ______

VERTICAL DESIGN (CONT.)

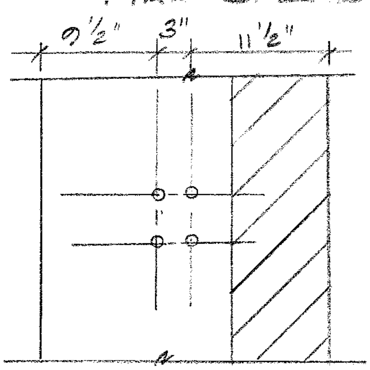
FRAMES ON LINES 4 & 11 AT LINES X & Y

MAX COMPRESSIVE LOADS (ASD) ⟶ MIN

USE: 24" x 18" DP CONT. FTG (MIN)
W/ (2)-#5 TOP & BTM

ANCHOR BOLTS REQ'D FOR VERTICAL UPLIFT

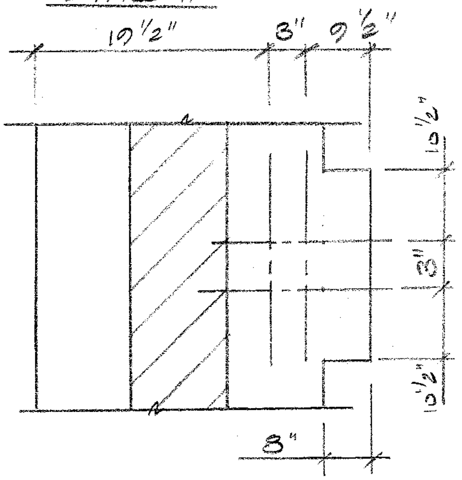
MAX UPLIFT FORCE (STRENGTH) ⟶ NO UPLIFT FORCE
MAX SHEAR LOAD (STRENGTH) ⟶ 2.8k (1.6) = 4.5k
TOWARD SLAB EDGE (WORST CASE)
WIND FORCE, NO SEISMIC FORCES



USE: (4)-3/4"ϕ F1554 GRADE 36
ANCHOR BOLTS W/ A MIN EMBED
OF 6"

SEE 'SIMPSON ANCHOR DESIGNER FOR ACI 318' RESULTS

LINE 11



D PRODUCT 207

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZALL EXPANSION 09.9-2/2
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 2/10
CHECKED BY ______ DATE ______
SCALE ______

VERTICAL DESIGN (CONT.)

FRAMES ON LINES 2-10.1 AT LINE P

MAX COMPRESSIVE LOAD (ASD) → 10.9k (ASCE EQ. 6a)
PAD FOOTING REQUIRED | USE: 2'-3" SQ × 18" DP (MIN) W/ (2)-#5 TOP & BTM
SEE ENERCALC RESULTS

ANCHOR BOLTS REQ'D FOR VERTICAL UPLIFT

MAX UPLIFT FORCE (STRENGTH) → 15.4k (ACI EQ 9-6)
MAX SHEAR LOAD AT MAX UPLIFT → 7.6k (TOWARDS SLAB)
WIND
MAX SHEAR LOAD TOWARDS SLAB EDGE → 9.0k W/ 12.0k COMPRESSIVE LOAD

MAX UPLIFT FORCE (SEIS) (STRENGTH) → 0 (ACI EQ 9-7)
MAX SHEAR LOAD (SEIS) (STRENGTH) → 2.0k (TOWARDS SLAB)
→ 1.4k (TOWARDS SLAB EDGE)
FOR LOADINGS TOWARDS SLAB SEE DESIGN FOR ANCHORS ON LINES 5'-10 & 4

USE: (4) 3/4"Ø F1554-GRADE 36 ANCHOR BOLTS W/ A MIN EMBED OF 9" & #3 HAIRPINS AT EA. PAIR OF ANCHOR BOLTS (SEE HAIRPIN DESIGN IN THIS CALC SET.)

SEE 'SIMPSON ANCHOR DESIGNER FOR ACI 318' RESULTS

NOTE: FRAMES ON LINES 1.5 & 11 @ LINE P HAVE SIMILAR LOADS. PAD FOOTINGS & ANCHORAGE NOTED ABOVE WILL BE USED AT THESE LOCATIONS AS WELL.

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

General Footing Design

Description : Pad Footing at Lines 2-10.1 - Line P

General Information

Calculations per IBC 2006, CBC 2007, ACI 318-05

Material Properties

f'c : Concrete 28 day strength	=	2.50	ksi
Fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	2,850.0	ksi
Concrete Density	=	145.0	pcf
Φ Values Flexure	=	0.90	
Shear	=	0.850	

Analysis Settings

Min Steel % Bending Reinf.	=	0.00140	
Min Allow % Temp Reinf.	=	0.00180	
Min. Overturning Safety Factor	=	1.50	: 1
Min. Sliding Safety Factor	=	1.50	: 1
AutoCalc Footing Weight as DL	:	No	
AutoCalc Pedestal Weight as DL	:	No	

Soil Design Values

Allowable Soil Bearing	=	2.660	ksf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	350.0	pcf
Soil/Concrete Friction Coeff.	=	0.350	

Increases based on footing Depth

Reference Depth below Surface	=	0.0	ft
Allow. Pressure Increase per foot of depth	=	0.0	ksf
when base footing is below	=	0.0	ft

Increases based on footing Width

Allow. Pressure Increase per foot of width	=	0.0	ksf
when footing is wider than	=	0.0	ft

Dimensions

Width along X-X Axis	=	2.250	ft
Length along Z-Z Axi	=	2.250	ft
Footing Thicknes	=	18.0	in

Load location offset from footing center...

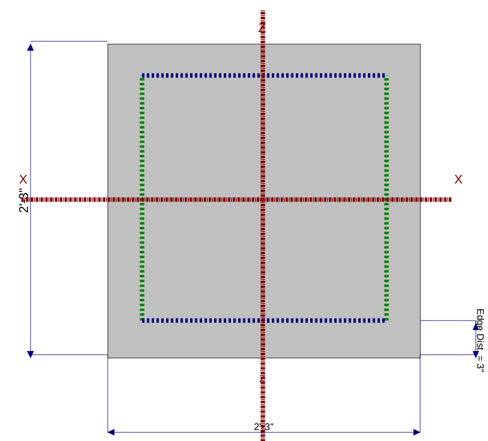
ex : Along X-X Axis	=	0	in
ez : Along Z-Z Axis	=	0	in

Pedestal dimensions...

px : Along X-X Axis	=	0.0	in
pz : Along Z-Z Axis	=	0.0	in
Height	=	0.0	in

Rebar Centerline to Edge of Concrete..

at Bottom of footing	=	3.313	in



Reinforcing

Bars along X-X Axis

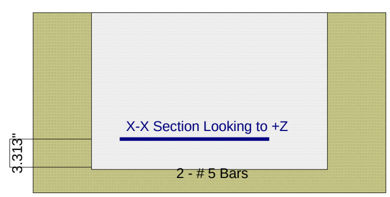
Number of Bars	=	2.0
Reinforcing Bar Size	=	# 5

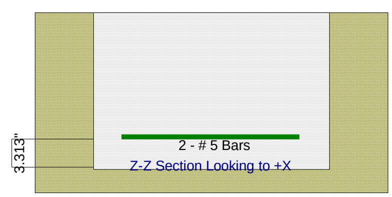
Bars along Z-Z Axis

Number of Bars	=	2.0
Reinforcing Bar Size	=	# 5

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	4.10	3.80	0.0	0.0	5.20	3.70	0.0	k
OB : Overburden	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0	ksf
M-xx	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
M-zz	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
V-x	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0	k
V-z	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0	k

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Printed: 1 FEB 2010, 6:32AM

General Footing Design

Z:\@Shawn Pierce Engineering\09_Projects\Borzal Addition_02\Engineering\Design\melford borzall expansion.ec6
ENERCALC, INC. 1983-2009, Ver: 6.1.01, N:40336

Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : Pad Footing at Lines 2-10.1 - Line P

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8056	Soil Bearing	2.143 ksf	2.660 ksf	+D+0.750Lr+0.750L+0.
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.1064	Z Flexure (+X)	1.895 k-ft	17.810 k-ft	+1.20D+1.60Lr+0.80W
PASS	0.1064	Z Flexure (-X)	1.895 k-ft	17.810 k-ft	+1.20D+1.60Lr+0.80W
PASS	0.1064	X Flexure (+Z)	1.895 k-ft	17.810 k-ft	+1.20D+1.60Lr+0.80W
PASS	0.1064	X Flexure (-Z)	1.895 k-ft	17.810 k-ft	+1.20D+1.60Lr+0.80W
PASS	n/a	1-way Shear (+X)	0.0 psi	85.0 psi	n/a
PASS	0.0	1-way Shear (-X)	0.0 psi	0.0 psi	n/a
PASS	n/a	1-way Shear (+Z)	0.0 psi	85.0 psi	n/a
PASS	n/a	1-way Shear (-Z)	0.0 psi	85.0 psi	n/a
PASS	n/a	2-way Punching	12.319 psi	85.0 psi	+1.20D+1.60Lr+0.80W

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc	Zecc	Actual Soil Bearing Stress +Z	+Z	-X	-X	Actual / Allowable Ratio
X-X, +D	2.660	n/a	0.0	0.8099	0.8099	n/a	n/a	0.305
X-X, +D+L+H	2.660	n/a	0.0	0.8099	0.8099	n/a	n/a	0.305
X-X, +D+Lr+H	2.660	n/a	0.0	1.560	1.560	n/a	n/a	0.587
X-X, +D+0.750Lr+0.750L+H	2.660	n/a	0.0	1.373	1.373	n/a	n/a	0.516
X-X, +D+W+H	2.660	n/a	0.0	1.837	1.837	n/a	n/a	0.691
X-X, +D+0.70E+H	2.660	n/a	0.0	1.321	1.321	n/a	n/a	0.497
X-X, +D+0.750Lr+0.750L+0.750W+H	2.660	n/a	0.0	2.143	2.143	n/a	n/a	0.806
X-X, +D+0.750L+0.750S+0.750W+H	2.660	n/a	0.0	1.580	1.580	n/a	n/a	0.594
X-X, +D+0.750Lr+0.750L+0.5250E+H	2.660	n/a	0.0	1.757	1.757	n/a	n/a	0.661
X-X, +D+0.750L+0.750S+0.5250E+H	2.660	n/a	0.0	1.194	1.194	n/a	n/a	0.449
X-X, +0.60D+W+H	2.660	n/a	0.0	1.513	1.513	n/a	n/a	0.569
X-X, +0.60D+0.70E+H	2.660	n/a	0.0	0.9975	0.9975	n/a	n/a	0.375
Z-Z, +D	2.660	0.0	n/a	n/a	n/a	0.8099	0.8099	0.305
Z-Z, +D+L+H	2.660	0.0	n/a	n/a	n/a	0.8099	0.8099	0.305
Z-Z, +D+Lr+H	2.660	0.0	n/a	n/a	n/a	1.560	1.560	0.587
Z-Z, +D+0.750Lr+0.750L+H	2.660	0.0	n/a	n/a	n/a	1.373	1.373	0.516
Z-Z, +D+W+H	2.660	0.0	n/a	n/a	n/a	1.837	1.837	0.691
Z-Z, +D+0.70E+H	2.660	0.0	n/a	n/a	n/a	1.321	1.321	0.497
Z-Z, +D+0.750Lr+0.750L+0.750W+H	2.660	0.0	n/a	n/a	n/a	2.143	2.143	0.806
Z-Z, +D+0.750L+0.750S+0.750W+H	2.660	0.0	n/a	n/a	n/a	1.580	1.580	0.594
Z-Z, +D+0.750Lr+0.750L+0.5250E+H	2.660	0.0	n/a	n/a	n/a	1.757	1.757	0.661
Z-Z, +D+0.750L+0.750S+0.5250E+H	2.660	0.0	n/a	n/a	n/a	1.194	1.194	0.449
Z-Z, +0.60D+W+H	2.660	0.0	n/a	n/a	n/a	1.513	1.513	0.569
Z-Z, +0.60D+0.70E+H	2.660	0.0	n/a	n/a	n/a	0.9975	0.9975	0.375

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status

Footing Has NO Overturning

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Sliding SafetyRatio	Status

Footing Has NO Sliding

Footing Flexure

Footing Flexure Load Combination...	Mu	Which Side ?	Tension @ Bot. or Top ?	As Req'd	Gvrn. As	Actual As	Phi*Mn	Status
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK

Description : Pad Footing at Lines 2-10.1 - Line P

Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK

General Footing Design

Description : Pad Footing at Lines 2-10.1 - Line P

Footing Flexure

Footing Flexure Load Combination...	Mu	Which Side ?	Tension @ Bot. or Top ?	As Req'd	Gvrn. As	Actual As	Phi*Mn	Status
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK
Z-Z, +0.90D+E+1.60H	0.9236 k-ft	+X	Bottom	0.01 in2/ft	Bending	0.28 in2/ft	17.810 k-ft	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Phi*Vn / Vu	Status
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK
+0.90D+E+1.60H	0.0 psi	0.0 psi	0.0 psi	0.0 psi	0.0 psi	85.0 psi	0.0	OK

Punching Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+0.90D+E+1.60H	6.005 psi	170.0 psi	0.03533	OK
+0.90D+E+1.60H	6.005 psi	170.0 psi	0.03533	OK
+0.90D+E+1.60H	6.005 psi	170.0 psi	0.03533	OK
+0.90D+E+1.60H	6.005 psi	170.0 psi	0.03533	OK
+0.90D+E+1.60H	6.005 psi	170.0 psi	0.03533	OK
+0.90D+E+1.60H	6.005 psi	170.0 psi	0.03533	OK
+0.90D+E+1.60H	6.005 psi	170.0 psi	0.03533	OK
+0.90D+E+1.60H	6.005 psi	170.0 psi	0.03533	OK
+0.90D+E+1.60H	6.005 psi	170.0 psi	0.03533	OK

General Footing Design

Description : Pad Footing at Lines 2-10.1 - Line P

Punching Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+0.90D+E+1.60H	6.005 psi	170.0psi	0.03533	OK

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 2-10.1 @ P - Wind

Date/Time : 2/1/2010 7:02:25 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
3/4" Heavy Hex Bolt	F1554 GR. 36	4	9	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
A tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
15400	0	7600	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)	N_{ua2} (lb)	N_{ua3} (lb)	N_{ua4} (lb)
3850.00	3850.00	3850.00	3850.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)	V_{ua2} (lb)	V_{ua3} (lb)	V_{ua4} (lb)
1900.00	1900.00	1900.00	1900.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua} / \Phi N_{sa}$
19370	14527.50	3850.00	0.2650

Concrete Breakout (Φ = 0.75)

N_{cbg}(lb)	ΦN_{cbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua} / \Phi N_{cbg}$
34365.39	25774.04	15400.00	0.5975

Pullout (Φ = 0.70)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	$N_{ua} / \Phi N_{pn}$
18220.00	12754.00	3850.00	0.3019

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.65)

V_{sa}(lb)	ΦV_{sa} (lb)	V_{ua}(lb)	$V_{ua} / \Phi V_{sa}$
11625	7556.25	1900.00	0.2514

Concrete Breakout has not been evaluated per user option.

Pryout (Φ = 0.70)

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uax}(lb)	$\Sigma V_{uax} / \Phi V_{cpg}$
68730.78	48111.55	0	0.0000

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uay}(lb)	$\Sigma V_{uay} / \Phi V_{cpg}$	$\Sigma V_{ua} / \Phi V_{cpg}$
68730.78	48111.55	7600	0.1580	0.1580

Interaction check

T.Max(0.60) + V.Max(0.25) = 0.85 <= 1.2 [Sec D.7.3]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 9 in. embedment

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 2-10.1 @ P - Wind Date/Time : 2/1/2010 7:03:51 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
3/4" Heavy Hex Bolt	F1554 GR. 36	4	9	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
A tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
-12000	0	-9000	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)	N_{ua2} (lb)	N_{ua3} (lb)	N_{ua4} (lb)
0.00	0.00	0.00	0.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)	V_{ua2} (lb)	V_{ua3} (lb)	V_{ua4} (lb)
2250.00	2250.00	2250.00	2250.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua} / \Phi N_{sa}$
19370	14527.50	0.00	0.0000

Concrete Breakout (Φ = 0.75)

N_{cbg}(lb)	ΦN_{cbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua} / \Phi N_{cbg}$
34365.39	25774.04	0.00	0.0000

Pullout (Φ = 0.70)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	N_{ua} /ΦN_{pn}
18220.00	12754.00	0.00	0.0000

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.65)

V_{sa}(lb)	ΦV_{sa} (lb)	V_{ua}(lb)	V_{ua} /ΦV_{sa}
11625	7556.25	2250.00	0.2978

Concrete Breakout has not been evaluated per user option.

Pryout (Φ = 0.70)

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uax}(lb)	ΣV_{uax} /ΦV_{cpg}
68730.78	48111.55	0	0.0000

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uay}(lb)	ΣV_{uay} /ΦV_{cpg}	ΣV_{ua} /ΦV_{cpg}
68730.78	48111.55	-9000	0.1871	0.1871

Interaction check

T.Max(0) <= 0.2 and V.Max(0.3) <= 1.0 [Sec D.7.2]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 9 in. embedment

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 2-10.1 @ P - E-　　　　Date/Time : 2/1/2010 7:05:43 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
3/4" Heavy Hex Bolt	F1554 GR. 36	4	9	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
A tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
0	0	-1400	0	0

e_x(in)	e_y(in)	Mod/high seismic	Apply entire shear @ front row
0	0	Yes	No

Individual Anchor Tension Loads

N_{ua1} (lb)	N_{ua2} (lb)	N_{ua3} (lb)	N_{ua4} (lb)
0.00	0.00	0.00	0.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)	V_{ua2} (lb)	V_{ua3} (lb)	V_{ua4} (lb)
350.00	350.00	350.00	350.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua} / \Phi N_{sa}$
19370	14527.50	0.00	0.0000

Concrete Breakout (Φ = 0.75 , Φ_{seis} = 0.75)

N_{cbg}(lb)	ΦN_{cbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua} / \Phi N_{cbg}$
34365.39	19330.53	0.00	0.0000

Pullout (Φ = 0.70 , Φ_{seis} = 0.75)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	N_{ua} /ΦN_{pn}
18220.00	9565.50	0.00	0.0000

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.65)

V_{eq}(lb)	ΦV_{eq}(lb)	V_{ua}(lb)	V_{ua} /ΦV_{eq}
11625	7556.25	350.00	0.0463

Concrete Breakout has not been evaluated per user option.

Pryout (Φ = 0.70 , Φ_{seis} = 0.75)

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uax}(lb)	ΣV_{uax} /ΦV_{cpg}
68730.78	36083.66	0	0.0000

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uay}(lb)	ΣV_{uay} /ΦV_{cpg}	ΣV_{ua} /ΦV_{cpg}
68730.78	36083.66	-1400	0.0388	0.0388

Interaction check

V.Max(0.05) <= 0.2 and T.Max(0) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 9 in. embedment

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZALI EXPANSION 09,9-2/2
SHEET NO. OF
CALCULATED BY DATE 2/10
CHECKED BY DATE
SCALE

VERTICAL DESIGN (CONT.)

FRAMES ON LINES 2-10.1 AT LINE N

MAX COMPRESSIVE LOAD (ASD) → 11.7k (ASCE EQ.3)
(N) STRUCTURE

MAX COMPRESSIVE LOAD FROM (E) METAL BLDG

D = 2.83k
C = 2.08k
L = 8.31k

ASCE EQ 3 = 13.22k

SEE 'FRAMES ON LINES 5, 6, 7, 8, 9 & 10 AT LINE A' FOR PAD FOOTING TO USE & RETROFIT REQUIRED

ANCHOR BOLTS REQ'D FOR VERTICAL UPLIFT

MAX UPLIFT LOAD (STRENGTH) → 13.7k (ACI EQ. 9-6)
MAX SHEAR LOAD AT MAX UPLIFT → 6.2k (TOWARDS SLAB)
WIND
MAX SHEAR LOAD TOWARDS SLAB EDGE → 5.8k W/ 15.7k COMP. LOAD

MAX UPLIFT FORCE (SEIS) (STRENGTH) → ∅ (ACI EQ 9-7)
MAX SHEAR LOAD (SEIS) (STRENGTH) → 2.0k (TOWARDS SLAB)
→ 1.4k (TOWARDS SLAB EDGE)

USE: (4) 3/4"∅ F1554 GRADE 36 ANCHOR BOLTS W/ A MIN EMBED OF 12" & #3 HAIRPINS AT EA PAIR OF ANCHOR BOLTS (SEE HAIRPIN DESIGN IN THIS CALC SET)

SEE 'SIMPSON ANCHOR DESIGNER FOR ACI 318' RESULTS

Seismic Design OK by Inspection

D PRODUCT 207

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 2-10.1 @ N (W)

Date/Time : 2/4/2010 5:37:59 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
		4		

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	NaN	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
A tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
13700	6200	0	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)	N_{ua2} (lb)	N_{ua3} (lb)	N_{ua4} (lb)
3425.00	3425.00	3425.00	3425.00

e'_{Nx}(in)	e'_{Ny}(in)
NaN	NaN

Individual Anchor Shear Loads

V_{ua1} (lb)	V_{ua2} (lb)	V_{ua3} (lb)	V_{ua4} (lb)
1550.00	1550.00	1550.00	1550.00

e'_{Vx}(in)	e'_{Vy}(in)
NaN	NaN

Tension Strengths

Steel (Φ = NaN)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua} / \Phi N_{sa}$
	NaN	3425.00	NaN

Concrete Breakout (Φ = NaN)

N_{cbg}(lb)	ΦN_{cbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua} / \Phi N_{cbg}$
NaN	NaN	13700.00	NaN

Pullout does not apply

Side-Face Blowout (Φ = NaN)

N_{sbg}(lb)	ΦN_{sbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua} / \Phi N_{sbg}$
30542.82	22907.12	6850.00	0.2990

Shear Strengths

Steel (Φ = NaN)

V_{sa}(lb)	ΦV_{sa} (lb)	V_{ua}(lb)	$V_{ua} / \Phi V_{sa}$
	NaN	1550.00	NaN

Concrete Breakout has not been evaluated per user option.

Pryout (Φ = NaN)

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uax}(lb)	$\Sigma V_{uax} / \Phi V_{cpg}$
NaN	NaN	6200	

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uay}(lb)	$\Sigma V_{uay} / \Phi V_{cpg}$	$\Sigma V_{ua} / \Phi V_{cpg}$
NaN	NaN	0		NaN

Interaction check

Interaction check:

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Borzal - Line 2-10.1 @ N (W) Date/Time : 2/4/2010 5:40:01 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
3/4" Heavy Hex Bolt	F1554 GR. 36	4	12	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
A tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
-15700	-5800	0	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)	N_{ua2} (lb)	N_{ua3} (lb)	N_{ua4} (lb)
0.00	0.00	0.00	0.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)	V_{ua2} (lb)	V_{ua3} (lb)	V_{ua4} (lb)
1450.00	1450.00	1450.00	1450.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
19370	14527.50	0.00	0.0000

Concrete Breakout (Φ = 0.75)

N_{cbg}(lb)	ΦN_{cbg}(lb)	ΣN_{ua}(lb)	$\Sigma N_{ua}/\Phi N_{cbg}$
19446.45	14584.84	0.00	0.0000

Pullout (Φ = 0.70)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{pn}$
18220.00	12754.00	0.00	0.0000

Side-Face Blowout (Φ = 0.75)

N_{sb}(lb)	ΦN_{sb}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sb}$
0.00	0.00	0.00	N/A

Shear Strengths

Steel (Φ = 0.65)

V_{sa}(lb)	ΦV_{sa} (lb)	V_{ua}(lb)	$V_{ua}/\Phi V_{sa}$
11625	7556.25	1450.00	0.1919

Concrete Breakout has not been evaluated per user option.

Pryout (Φ = 0.70)

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uax}(lb)	$\Sigma V_{uax}/\Phi V_{cpg}$
38892.90	27225.03	-5800	0.2130

V_{cpg}(lb)	ΦV_{cpg}(lb)	ΣV_{uay}(lb)	$\Sigma V_{uay}/\Phi V_{cpg}$	$\Sigma V_{ua}/\Phi V_{cpg}$
38892.90	27225.03	0	0.0000	0.2130

Interaction check

T.Max(0) <= 0.2 and V.Max(0.21) <= 1.0 [Sec D.7.2]

Interaction check: PASS

Use 3/4" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 12 in. embedment

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277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BOREAL EXPANSION 09.9-2/2
SHEET NO. ____ OF ____
CALCULATED BY ____ DATE 2/10
CHECKED BY ____ DATE ____
SCALE ____

VERTICAL DESIGN (CONT.)

FRAMES ON LINES 1.6 & 11 AT LINE P

MAX COMPRESSIVE LOADS (ASD) → 7.4k (ASCE EQ. 6a)

USE: 24"x18"DP CONT. FTG (MIN)
w/ (2) #5 TOP & BTM

SEE 'EXISTING FOOTING' CALC.

ANCHOR BOLTS REQ'D FOR VERTICAL UPLIFT

MAX UPLIFT FORCE (STRENGTH) → 6.2k (ACI EQ 9-6)
MAX SHEAR LOAD AT MAX UPLIFT → 4.0k (TOWARDS SLAB)
WIND
MAX SHEAR LOAD TOWARDS SLAB EDGE → 4.6k w/ 8.0k COMP. LOAD

MAX UPLIFT FORCE (SEIS) (STRENGTH) → ϕ (ACI EQ 9-7)
MAX SHEAR LOAD (SEIS) (STRENGTH) → 1.2k (TOWARDS SLAB)
→ 0.8k (TOWARDS SLAB EDGE)
FOR LOADING TOWARDS SLAB SEE DESIGN FOR ANCHORS ON LINE 2-10.1 @ P

USE: (4) 3/4" ϕ F1554 GRADE 36 ANCHOR BOLTS w/ A MIN EMBED OF 6" & #3 HAIRPINS AT EA. PAIR OF ANCHOR BOLTS (SEE HAIRPIN DESIGN IN THIS CALC SET.)

SEE DESIGN FOR 'FRAMES ON LINES 4 & 11 AT LINE W'

D PRODUCT 207

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277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORBALL EXPANSION 099-2/2
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 2/10
CHECKED BY ______ DATE ______
SCALE ______

VERTICAL DESIGN (CONT.)

FRAMES ON LINES 1.5 & 11 AT LINE N

MAX COMPRESSIVE LOAD (ASD) ⟶ 11.9k (ASCE EQ 3)
└ NEW STRUCTURE

MAX COMPRESSIVE LOAD FROM (E) METAL BLDG

D = 0.55k
C = 0.29k
L = 1.16k

ASCE EQ 3 = 1.99k

(E) FTG, 24"W × 18"DP W/ (2)-#5 T&B,
OK FOR (N) & (E) POINT LOADS

SEE (E) FTG CALC.

ANCHOR BOLTS REQ'D FOR VERTICAL UPLIFT

MAX UPLIFT LOAD (STRENGTH) ⟶ 5.0k (ACI EQ 9-6)
MAX SHEAR AT MAX UPLIFT ⟶ 3.5k (TOWARDS SLAB)

MAX SHEAR LOAD TOWARD SLAB EDGE ⟶ 3.2k W/10.4k COMP. LOAD

MAX UPLIFT FORCE (SES) (STRENGTH) ⟶ ∅ (ACI EQ 9-6)
MAX SHEAR LOAD (SES) (STRENGTH) ⟶ 1.4k (TOWARDS SLAB)
⟶ 1.0k (TOWARDS SLAB EDGE)

USE: (4) 3/4"∅ F1554 GRADE 36 ANCHOR BOLTS W/ A MIN EMBED OF 7" & #3 HAIRPINS AT EA. PAIR OF ANCHOR BOLTS (SEE HAIRPIN DESIGN IN THIS CALC SET)

SEE ANCHOR BOLT DESIGN FOR LINES 4 & 11 AT LINE A

D PRODUCT 207

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277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB	Borzall Expansion - 09.9-2/2		
SHEET NO.		OF	
CALCULATED BY	sdp	DATE	2/10
CHECKED BY		DATE	
SCALE			

Standard Continuous Perimeter Footing - Vertical Loading

Check Bending Capacity

b= 12 in h = 24 in d = 20.69 in

Reinforcing (2) - #5 Top & Btm F_y = 60000 psi

f'_c = 2500 psi

ρ = 0.0025

$$M_u = \frac{0.9bd^2 f_y \rho}{12}\left(1 - \frac{\rho f_y}{1.7 f'_c}\right) = 55683 \text{ lb-ft}$$

$$V_u = \left(\frac{2\sqrt{f'_c}\,bd}{2}\right) = 12413 \text{ lbs}$$

Vertical Foundation Design provided by Pad Footings previously designed

Check Actual Conditions

$P_{Down\ DL}$ = 0.1 lbs (ASD) $P_{Down\ LL}$ = 0.1 lbs (ASD) $P_{Down\ TL}$ = 0.1 lbs (

Soil Bearing Pressure 2000 psf

L' = Required Length = P_{Down}/(Soil Bearing Pressure*b) = 0.00 ft

L = L'/2 = 0.00 ft

$P_{Down\ U}$ = 1.4*(DL + CLL) + 1.7(LL) = 0.31 lbs

w_U = $P_{Down\ U}$/L' = 6200 plf

$M_U = w_U L^2/3$ = 1E-06 lb-ft < 55683 lb-ft

$V_U = w_U L$ = 0.155 lbs < 12413 lbs

Check Uplift

P_{Up} = 7800 lbs Strength Design

$P_{Up\ U}$ = P_{Up}*.7 = 5460 lbs

0.7 = MBNA Factor

Determine Dead Load Resistance

Footing: 150pcf*b*h = 300 plf

Slab: 150pcf*t*4' = 200 plf t = 4 in

$w_{U\ DL}$ = 1.4*(Ftg + Slab) = 700 plf

Req'd L = $P_{Up\ U}/w_{U\ DL}$ = 7.8 ft

Max L Shear = $V_U/w_{U\ DL}$ = 17.73 ft

Max L Moment = $\sqrt{\dfrac{M_U * 3}{w_{UDL}}}$ = 15.45 ft

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277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB Borzall Expansion - 09.9-2/2
SHEET NO. ______ OF ______
CALCULATED BY sdp DATE 2/10
CHECKED BY ______ DATE ______
SCALE ______

Standard Continuous Perimeter Footing - Thrust Loading

Check Bending Capacity

b= 12 in h = 24 in d = 8.688 in

Reinforcing (2) - #5 Top & Btm F_y = 60000 psi

f'_c = 2500 psi

ρ = 0.0030

$M_u = \frac{0.9hd^2 f_y \rho}{12}\left(1-\frac{\rho f_y}{1.7f'_c}\right)$ = 23221 lb-ft

$V_u = \left(\frac{2\sqrt{f'_c}\,hd}{2}\right)$ = 10425 lbs

Check Passive Resistance

Passive Pressure = 350 psf

Allowed Ultimate Passive Pressure = 700 plf

L_{Max} Shear = V_u/Allowed Ult Passive Press = 14.89 ft

L_{Max} Mnt = $\sqrt{\frac{M_U * 2 * 3}{1.7 * w_{Passive}}}$ = 10.82 ft CONTROLS

Total Passive Resistance = L_{Max}*All Ult Pass Press = 7574 lbs

Slab Tension = t*3t*5*√f'$_c$ = 12000 lbs t = 4 in

Total Thrust Resistance = 19574 lbs

ASD Thrust Resistance = (Total Thrust)/1.7 = 11514 lbs

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277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

Check Thrust

Concrete Resistance for 4 in Slab

V_{thrust} Outward

Check Rebar: (1) - #3 Hairpin

Tension of Steel = $0.9(A_b)f_y$ = 7920 lbs **CONTROLS**

Check Concrete Tensile Failure at end of Hairpin:

$T_{Slab} = 1.6\sqrt{f'_c}bd$ = 38400 lbs

f'_c = 2500 psi W at end of hairpin 10 ft

Check Concrete Tensile Failure at end of Base plate:

Assume no Hairpin

b_{Plate} = 10 in

$T_{Slab} = 1.6\sqrt{f'_c}bd$ = 3200 lbs

V_{thrust} Inward

Check Concrete Compressive Failure at face of base plate:

Adjacent Slab will brace for buckling

$C_{Slab} = 0.25f'_c bd$ = 25000 lbs

$C_{Slab\ design} = C_{slab}$/Factor of Safety = 12500 lbs

Factor of Safety = 2

FOUNDATION ANALYSIS

Office Expansion

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZANI EXPANSION 09.9-2/2

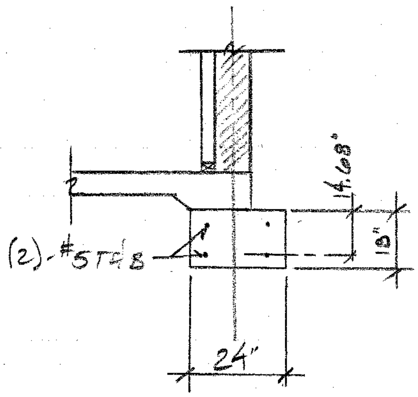
SHEET NO. ______ OF ______

CALCULATED BY ______ DATE 11/09

CHECKED BY ______ DATE ______

SCALE OFFICE EXPANSION

TYP. FOOTING - LINE (P)



UNIFORM LOADS ON FTG

			DL	LL
WAINSCOTTING	78psf (5'4")	=	420plf	
FLOOR	14psf (11'-6")	=	160plf	
	50psf (11'-6")	=		575plf
WALL	16psf (25') (ASSUMED STUCCO)	=	400plf	
			985plf	575plf

MAX LOAD COMBO D+L = 1560plf

F_b = 2000psf (2') = 4000plf

ADD'L CAPACITY = 4000 - 1560 = 2440plf

MAX POINT LOAD (FB-3)

P_D = 1.71k

P_L = 3.595k

TYP. FOOTING OK FOR ADD'L LOADING FROM POINT LOAD

SEE ENERCALC RESULTS

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Printed: 17 NOV 2009, 1:52PM

Concrete Beam Design

Z:\@Shawn Pierce Engineering\09_Projects\Borzal Addition_02\Engineering\Design\melford borzall expansion.ec6
ENERCALC, INC. 1983-2009, Ver: 6.0.24, N:40336

Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : Typ Footing Aloing Line P - Max Point Load

Material Properties

Calculations per IBC 2006, CBC 2007, ACI 318-05

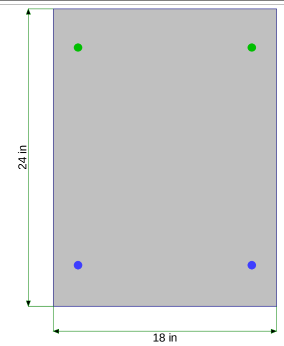
f'c = 2.50 ksi
fr = $f'c^{1/2}$ * 7.50 = 375.0 psi
ψ Density = 145.0 pcf
Elastic Modulus = 2,850.0 ksi
Load Combination 2006 IBC & ASCE 7-05

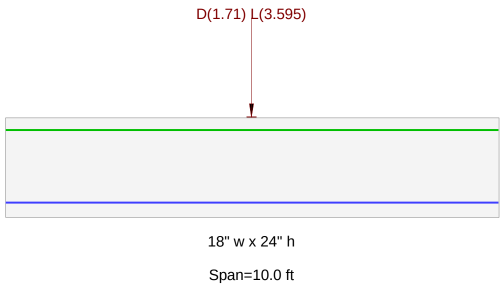
φ Phi Values Flexure : 0.90
Shear : 0.750
β_1 = 0.850

Fy - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi

Fy - Stirrups = 40.0 ksi
E - Stirrups = 29,000.0 ksi
Stirrup Bar Size # # 3
Num of Bars Crossing Inclined Crack = 2

Beam is supported on an elastic foundation.





Cross Section & Reinforcing Details

Rectangular Section, Width = 18.0 in, Height = 24.0 in
Span #1 Reinforcing....
2-#5 at 3.313 in from Bottom, from 0.0 to 13.50 ft in this span
2-#5 at 3.125 in from Top, from 0.0 to 13.50 ft in this span

Applied Loads

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

Load for Span Number 1
Point Load : D = 1.710, L = 3.595 k @ 5.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.155** : 1
Section used for this span **Typical Section**
Mu : Applied 9.331 k-ft
Mn * Phi : Allowable 60.210 k-ft
Load Combination +1.20D+0.50Lr+1.60L+1.60
Location of maximum on span 5.077 ft
Span # where maximum occurs Span # 1

Maximum Deflection
Max Downward L+Lr+S Deflection 0.000 in
Max Upward L+Lr+S Deflection 0.000 in
Max Downward Total Deflection 0.017 in
Max Upward Total Deflection 0.005 in

Maximum Soil Pressure = **0.364** ksf at 5.00 ft

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	0.039	0.039
D Only	0.013	0.013
D+L	0.039	0.039

Shear Stirrup Requirements

Maximum Forces & Stresses for Load Combinations

Load Combination Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	5.077	9.33	60.21	0.15
+1.40D					
Span # 1	1	5.077	2.86	60.21	0.05
+1.20D+0.50Lr+1.60L+1.60H					
Span # 1	1	5.077	9.33	60.21	0.15
+1.20D+1.60Lr+0.50L					
Span # 1	1	5.077	4.60	60.21	0.08

FOUNDATION ANALYSIS

Conference Room Expansion

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZALL EXPANSION 09.09-2/2

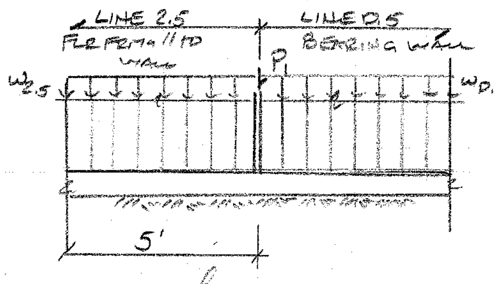
SHEET NO. ______ OF ______

CALCULATED BY ______ DATE 11/09

CHECKED BY ______ DATE ______

SCALE CONFERENCE ROOM EXTENSION

✓ (E) FOOTING FOR ADD'L POINT LOAD INTERSECTION OF LINES (2.5) & (F.5)



	FLOOR	WALL
DL	14 psf	9 psf
LL	100 psf	—
TRIB	1' / 11.25/2	12'-6"/25'

$w_{2.5_D}$ = 130 plf $w_{F.5_D}$ = 305 plf

$w_{2.5_L}$ = 100 plf $w_{F.5_L}$ = 565 plf

P_{1D} = 160#

P_{1L} = 1125#

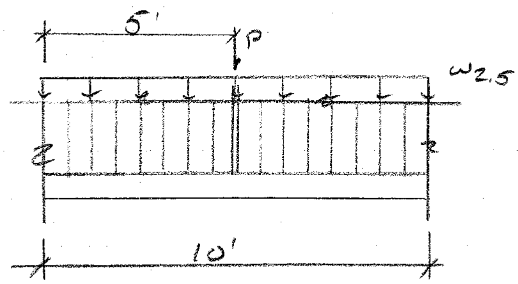
F_b = 2000 psf

SUBGRADE MODULUS = 150 psi/in

(E) 12"W x 18" DP CONTINUOUS FTG w/ (1)-#5 TOP & BTM

SEE ENERCALC RESULTS

✓ (E) FOOTING FOR ADD'L POINT LOAD INTERSECTION OF LINES (G.5) & (2.5)



$w_{2.5_D}$ = 130 plf
$w_{2.5_L}$ = 100 plf } SEE ABOVE

P_{1D} = 830#

P_{1L} = 5910#

F_b = 2000 psf

SUBGRADE MODULUS = 150 psi/in

(E) 12"W x 18" DP CONTINUOUS FTG w/ (1)-#5 TOP & BTM

SEE ENERCALC RESULTS

D PRODUCT 207

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

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Concrete Beam Design

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ENERCALC, INC. 1983-2009, Ver: 6.0.24, N:40336

Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : (E) Footing for Additional Point Load - Intersection of Lines 2.5 & D.5

Material Properties

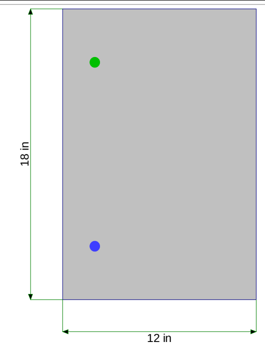
Calculations per IBC 2006, CBC 2007, ACI 318-05

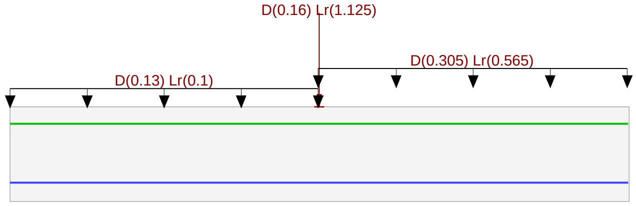
f'c = 2.50 ksi
fr = f'c$^{1/2}$ * 7.50 = 375.0 psi
ψ Density = 145.0 pcf
Elastic Modulus = 2,850.0 ksi
Load Combination 2006 IBC & ASCE 7-05

φ Phi Values Flexure : 0.90
Shear : 0.750
β_1 = 0.850

Fy - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi
Fy - Stirrups = 40.0 ksi
E - Stirrups = 29,000.0 ksi
Stirrup Bar Size # # 3
Num of Bars Crossing Inclined Crack = 2

Beam is supported on an elastic foundation.





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 18.0 in
Span #1 Reinforcing....
1-#5 at 3.313 in from Bottom, from 0.0 to 15.0 ft in this span
1-#5 at 3.313 in from Top, from 0.0 to 15.0 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Point Load : D = 0.160, Lr = 1.125 k @ 5.0 ft
Uniform Load : D = 0.130, Lr = 0.10 k/ft, Extent = 0.0 -->> 5.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.3050, Lr = 0.5650 k/ft, Extent = 5.0 -->> 10.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.103** : 1
Section used for this span **Typical Section**
Mu : Applied 2.384 k-ft
Mn * Phi : Allowable 23.075 k-ft
Load Combination +1.20D+1.60Lr+0.50L
Location of maximum on span 5.077 ft
Span # where maximum occurs Span # 1

Maximum Deflection
Max Downward L+Lr+S Deflection 0.000 in
Max Upward L+Lr+S Deflection 0.000 in
Max Downward Total Deflection 0.063 in
Max Upward Total Deflection 0.015 in

Maximum Soil Pressure = **1.361** ksf at 10.00 ft

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	0.031	0.105
D Only	0.025	0.045
D+Lr	0.031	0.105

Shear Stirrup Requirements

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	5.077	2.38	23.07	0.10
+1.40D					
Span # 1	1	5.385	0.28	23.07	0.01
+1.20D+0.50Lr+1.60L+1.60H					
Span # 1	1	5.077	0.91	23.07	0.04

Concrete Beam Design

Description : (E) Footing for Additional Point Load - Intersection of Lines 2.5 & D.5

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
+1.20D+1.60Lr+0.50L					
Span # 1	1	5.077	2.38	23.07	0.10

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
Span 1	1	0.0630	10.000		0.0000	0.000

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

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Concrete Beam Design

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ENERCALC, INC. 1983-2009, Ver: 6.0.24, N:40336

Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : (E) Footing for Additional Point Load - Intersection of Lines 2.5 & G.5

Material Properties

Calculations per IBC 2006, CBC 2007, ACI 318-05

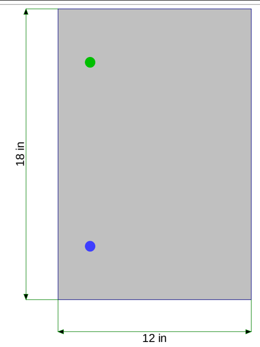
f'c = 2.50 ksi
fr = $f'c^{1/2}$ * 7.50 = 375.0 psi
ψ Density = 145.0 pcf
Elastic Modulus = 2,850.0 ksi
Load Combination 2006 IBC & ASCE 7-05

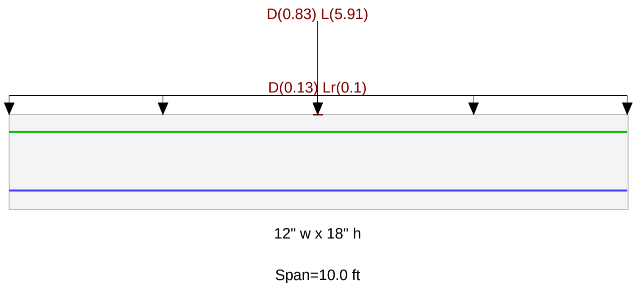
ϕ Phi Values Flexure : 0.90
Shear : 0.750
β_1 = 0.850

Fy - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi

Fy - Stirrups = 40.0 ksi
E - Stirrups = 29,000.0 ksi
Stirrup Bar Size # # 3
Num of Bars Crossing Inclined Crack = 2

Beam is supported on an elastic foundation,





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 18.0 in
Span #1 Reinforcing....
1-#5 at 3.313 in from Bottom, from 0.0 to 15.0 ft in this span
1-#5 at 3.313 in from Top, from 0.0 to 15.0 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Point Load : D = 0.830, L = 5.910 k @ 5.0 ft
Uniform Load : D = 0.130, Lr = 0.10 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.532**: 1
Section used for this span **Typical Section**
Mu : Applied 12.275 k-ft
Mn * Phi : Allowable 23.075 k-ft
Load Combination +1.20D+0.50Lr+1.60L+1.60
Location of maximum on span 4.923 ft
Span # where maximum occurs Span # 1

Maximum Deflection
Max Downward L+Lr+S Deflection 0.000 in
Max Upward L+Lr+S Deflection 0.000 in
Max Downward Total Deflection 0.054 in
Max Upward Total Deflection 0.020 in

Maximum Soil Pressure = **1.167** ksf at 5.00 ft

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	0.073	0.073
D Only	0.032	0.032
D+Lr	0.040	0.040
D+L	0.073	0.073

Shear Stirrup Requirements

Maximum Forces & Stresses for Load Combinations

Load Combination Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	4.923	12.28	23.07	0.53
+1.40D					
Span # 1	1	4.923	1.36	23.07	0.06
+1.20D+0.50Lr+1.60L+1.60H					
Span # 1	1	4.923	12.28	23.07	0.53

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Concrete Beam Design

Description : (E) Footing for Additional Point Load - Intersection of Lines 2.5 & G.5

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
+1.20D+1.60Lr+0.50L					
Span # 1	1	4.923	4.64	23.07	0.20

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
Span 1	1	0.0540	5.000		0.0000	0.000

Concrete Beam Design

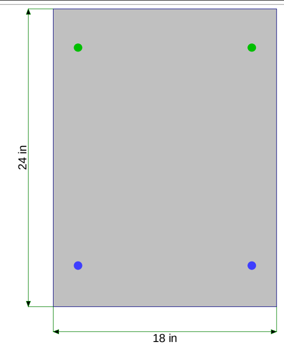
Description : Continuous footing under new column

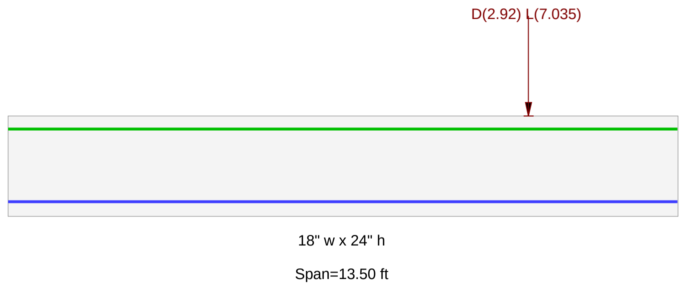
Material Properties

Calculations per IBC 2006, CBC 2007, ACI 318-05

f'c	=	2.50 ksi	φ Phi Values	Flexure :		0.90
fr = $f'c^{1/2}$ * 7.50	=	375.0 psi		Shear :		0.750
ψ Density	=	145.0 pcf	β_1		=	0.850
Elastic Modulus	=	2,850.0 ksi				
Load Combination		2006 IBC & ASCE 7-05				
Fy - Main Rebar	=	60.0 ksi	Fy - Stirrups		=	40.0 ksi
E - Main Rebar	=	29,000.0 ksi	E - Stirrups		=	29,000.0 ksi
			Stirrup Bar Size #		#	3
Num of Bars Crossing Inclined Crack					=	2

Beam is supported on an elastic foundation,





Cross Section & Reinforcing Details

Rectangular Section, Width = 18.0 in, Height = 24.0 in

Span #1 Reinforcing....

2-#5 at 3.313 in from Bottom, from 0.0 to 13.50 ft in this span

2-#5 at 3.125 in from Top, from 0.0 to 13.50 ft in this span

Applied Loads

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

Load for Span Number 1

Point Load : D = 2.920, L = 7.035 k @ 10.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.216** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	12.985	k-ft
Mn * Phi : Allowable	60.210	k-ft
Load Combination	+1.20D+0.50Lr+1.60L+1.60	
Location of maximum on span	10.488	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection		
Max Downward L+Lr+S Deflection	0.000	in
Max Upward L+Lr+S Deflection	0.000	in
Max Downward Total Deflection	0.067	in
Max Upward Total Deflection	-0.035	in

Maximum Soil Pressure = **1.455** ksf at 13.50 ft

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	-0.001	0.227
D Only	-0.000	0.066
D+L	-0.001	0.227

Shear Stirrup Requirements

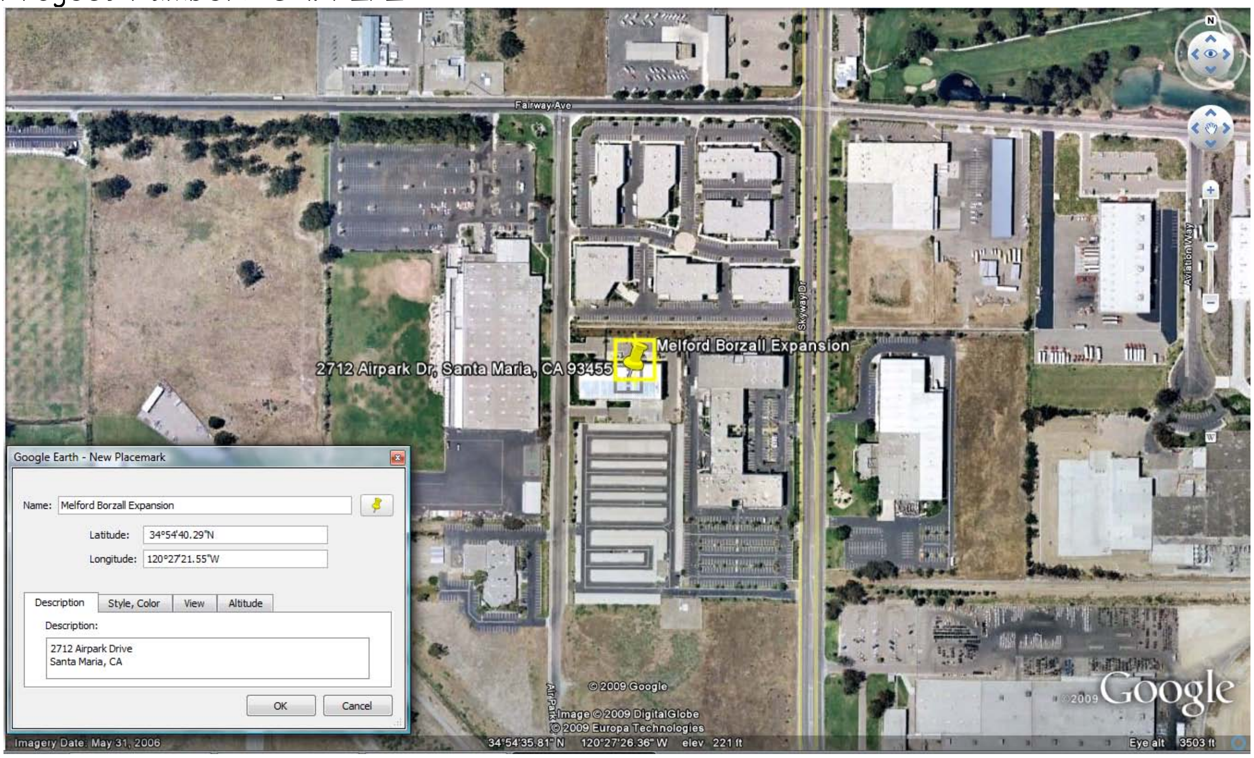
Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	10.488	12.98	60.21	0.22
+1.40D					
Span # 1	1	10.488	3.60	60.21	0.06
+1.20D+0.50Lr+1.60L+1.60H					
Span # 1	1	10.488	12.98	60.21	0.22
+1.20D+1.60Lr+0.50L					
Span # 1	1	10.488	6.18	60.21	0.10

shawn pierce engineering

LATERAL ANALYSIS

Melford Borzall Expansion
Project Number: 09.9-2/2



Latitude: 34°54'40.29" *or* 34.91°   Longitude: -120°27'21.55" *or* -120.46°

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 34.91
Longitude = -120.46
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.009999999776482582 deg grid spacing

Period (sec)	Sa (g)
0.2	1.266 (Ss, Site Class B)
1.0	0.434 (S1, Site Class B)

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 34.91
Longitude = -120.46
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.566

Period (sec)	Sa (g)
0.2	1.266 (SMs, Site Class D)
1.0	0.680 (SM1, Site Class D)

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 34.91
Longitude = -120.46
Design Spectral Response Accelerations SDs and SD1
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.0 ,Fv = 1.566

Period (sec)	Sa (g)
0.2	0.844 (SDs, Site Class D)
1.0	0.453 (SD1, Site Class D)

ASCE 7-05 Seismic Factor Determination

Description : Melford Borzall Expansion

Occupancy Category

Calculations per IBC 2006 & ASCE 7-05

Occupancy Category of Building or Other Structure : "II" : All Buildings and other structures except those listed as Category I, III, and IV — *ACSE 7-05, Page 3, Table 1-1*

Occupancy Importance Factor = 1 — *ACSE 7-05, Page 116, Table 11.5-1*

USER DEFINED Ground Motion

ASCE 7-05 9.4.1.1

Max. Ground Motions, 5% Damping :

S_S = 1.266 g, 0.2 sec response
S_1 = 0.4340 g, 1.0 sec response

Site Class, Site Coeff. and Design Category

Site Classification "D" : Shear Wave Velocity 600 to 1,200 ft/sec = D — *ASCE 7-05 Table 20.3-1*

Site Coefficients Fa & Fv — *ASCE 7-05 Table 11.4-1 & 11.4-2*
(using straight-line interpolation from table values)
Fa = 1.00
Fv = 1.57

Maximum Considered Eartquake Acceleration — *ASCE 7-05 Table 11.4-3*
S_{MS} = Fa * Ss = 1.266
S_{M1} = Fv * S1 = 0.680

Design Spectral Acceleration — *ASCE 7-05 Table 11.4-4*
$S_{DS} = S_{MS} * 2/3$ = 0.844
$S_{D1} = S_{M1} * 2/3$ = 0.453

Seismic Design Category = D *(SD1 is most severe)* — *ASCE 7-05 Table 11.6-1*

Resisting System

ASCE 7-05 Table 12.2-1

Basic Seismic Force Resisting System . . . Bearing Wall Systems
Light-framed walls sheathed w/wood structural panels rated for shear resistance or steel sheets.

Response Modification Coefficient " R " = 6.50
System Overstrength Factor " Wo " = 3.00
Deflection Amplification Factor " Cd " = 4.00

NOTE! See ASCE 7-05 for all applicable footnotes.

Building height Limits :

Category "A & B" Limit:	No Limit
Category "C" Limit:	No Limit
Category "D" Limit:	Limit = 65
Category "E" Limit:	Limit = 65
Category "F" Limit:	Limit = 65

Redundancy Factor

ASCE 7-05 Section 12.3.4

Seismic Design Category of D, E, or F therefore Redundancy Factor " p " = 1.3

Lateral Force Procedure

ASCE 7-05 Section 12.8

Equivalent Lateral Force Procedure
The "Equivalent Lateral Force Procedure" is being used according to the provisions of ASCE 7-05 12.8

Determine Building Period

Use ASCE 12.8-7

Structure Type for Building Period Calculation : All Other Structural Systems

" Ct " value = 0.020
" x " value = 0.75
" hn " : Height from base to highest level = 12.330 ft

" Ta " Approximate fundemental period using Eq. 12.8-7 : Ta = Ct * (hn ^ x) = 0.132 sec
"TL" : Long-period transition period per ASCE 7-05 Maps 22-15 -> 22-20 12.000 sec

Building Period " Ta " Calculated from Approximate Method selected = 0.132 sec

" Cs " Response Coefficient

ASCE 7-05 Section 12.8.1.1

S_{DS} : Short Period Design Spectral Response	= 0.844	From Eq. 12.8-2, Preliminary Cs	=	0.130
" R " : Response Modification Factor	= 6.50	From Eq. 12.8-3 & 12.8-4 , Cs need not exceed	=	0.530
" I " : Occupancy Importance Factor	= 1	From Eq. 12.8-5 & 12.8-6, Cs not be less than	=	0.010

User has selected ASCE 12.8.1.3 : Regular structure,
Less than 5 Stories and with T <<= 0.5 sec, SO Ss <= 1.5 for Cs calculation

Cs : Seismic Response Coefficient = S_{DS} / (R/I) * 0.70 = 0.0909

ASCE 7-05 Seismic Factor Determination

Description : Melford Borzall Expansion

Seismic Base Shear

Calculated for Allowable Stress Design Load Combinations *ASCE 7-05 Section 12.8.3*

Cs = 0.0909 from 12.8.1.1

W (see Sum Wi below) = 0.00 k

Vertical Distribution of Seismic Forces

Seismic Base Shear V = Cs * W = 0.00 k

" k " : hx exponent based on Ta = 1.00

Table of building Weights by Floor Level...

Level #	Wi : Weight	Hi : Height	(Wi * Hi) ^k	Cvx	Fx=Cvx * V	Sum Story Shear	Sum Story Moment

Sum Wi = 0.00 k Sum Wi * Hi = 0.00 k-ft Total Base Shear = 0.00 k
Base Moment = 0.0 k-ft

Diaphragm Forces : Seismic Design Category "D", "E" & "F"

ASCE 7-05 9.5.2.6.4.4

Level #	Wi	Fi	Sum Fi	Sum Wi	Fpx

Wpx . . . . Weight at level of diaphragm and other structure elements attached to it.
Fi . . . . Design Lateral Force applied at the level.
Sum Fi . . . . Sum of "Lat. Force" of current level plus all levels above
MIN Req'd Force @ Level . . . . $0.20 * S_{DS} * I * Wpx$
MAX Req'd Force @ Level . . . . $0.40 * S_{DS} * I * Wpx$
Fpx : Design Force @ Level . . . . Wpx * SUM(x->n) Fi / SUM(x->n) wi, x = Current level, n = Top Level

Wall Anchorage

Concrete & Masonry Wall Normal Force : Minimum Force per ACSE 7-05 12.11.1

Minimum Factor : 0.40 * SDS * Importance * Weight = 0.3376 * Weight

Concrete & Masonry Wall Anchorage : Seismic Design Category "C" & "D" per ACSE 7-05 12.11.2.1

Actual Wall Weight Tributary to Anchor = lbs/ lin. ft

Fp : Anchorage Design Force . . .

Rigid Diaphragm Design Force = 0.40 * SDS * I * Trib. Weight = 0.00 lbs / foot

Flexible Diaphragm Design Force = 0.80 * SDS * I * Trib. Weight = 0.00 lbs / foot

Combination of Load Effects

ASCE 7-05 12.4.2.3

Load Description	D Dead Load	Qe Seismic Load			E H & V Load Effect
	0.000	0.000	E = p * Qe + 0.20 * SDS * D	=	0.000
	0.000	0.000	E = p * Qe + 0.20 * SDS * D	=	0.000
	0.000	0.000	E = p * Qe + 0.20 * SDS * D	=	0.000
	0.000	0.000	E = p * Qe + 0.20 * SDS * D	=	0.000
	0.000	0.000	E = p * Qe + 0.20 * SDS * D	=	0.000
	0.000	0.000	E = p * Qe + 0.20 * SDS * D	=	0.000
	0.000	0.000	E = p * Qe + 0.20 * SDS * D	=	0.000
	0.000	0.000	E = p * Qe + 0.20 * SDS * D	=	0.000

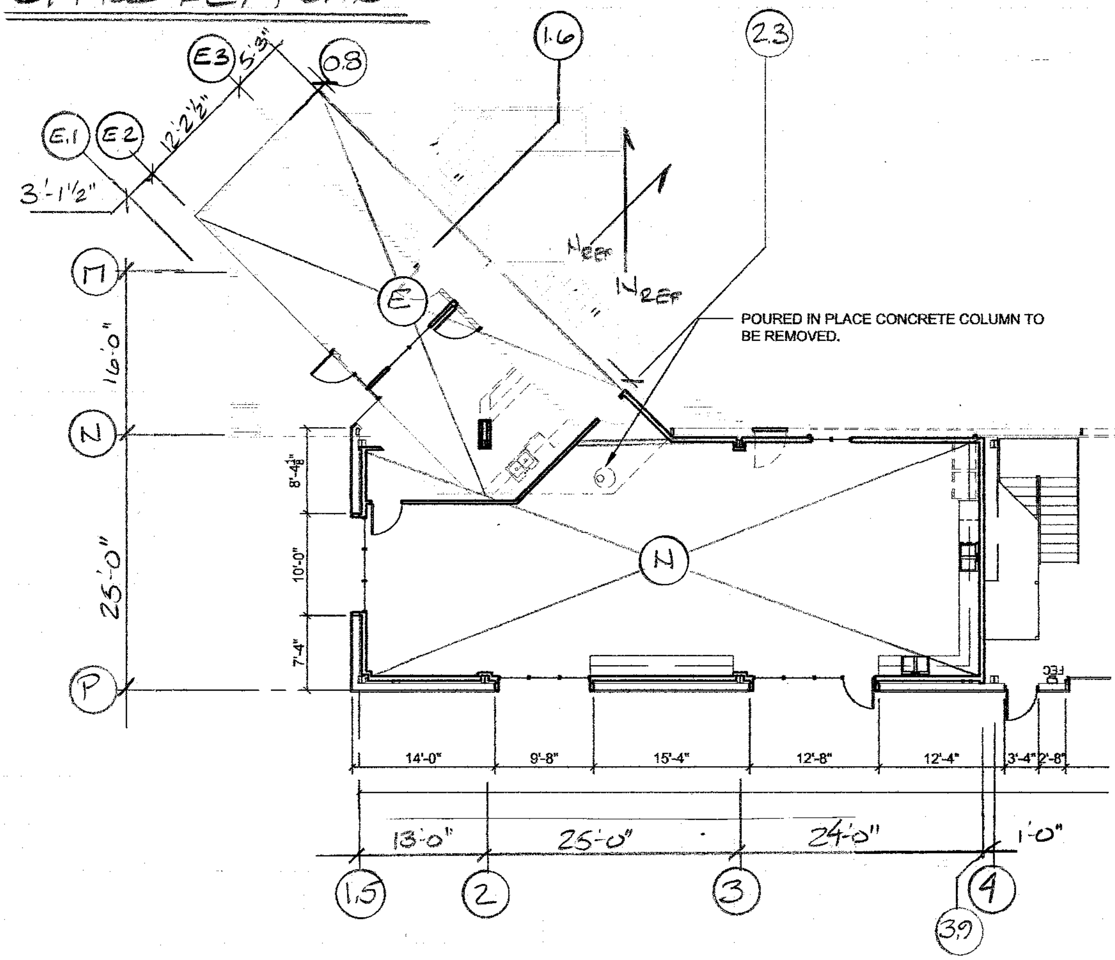
LATERAL ANALYSIS

Office Expansion

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BOREAL EXPANSION 07.7-2/2
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 1/9/8
CHECKED BY ______ DATE ______
SCALE ______

OFFICE KEY PLAN



DIAPHRAGM LOADS

E*
N/S: 14psf + 12.33' [16psf (0.6) + 9psf (2.0)] 1/17'-6" = 33.5plf
E/W: 14psf + 12.33' [16psf (Ø) + 9psf (2.5) + 78plf (1.0)] 1/42.167' = 43.4plf

N
N/S: 14psf + 12.33' [9psf (2.0)] 1/25' = 22.9plf
E/W: 14psf + 12.33' [9psf (2.0)] 1/62' = 17.6plf

* NO CHANGES TO (E) DIAPHRAGM OR LATERAL RESTRAINT SYSTEM

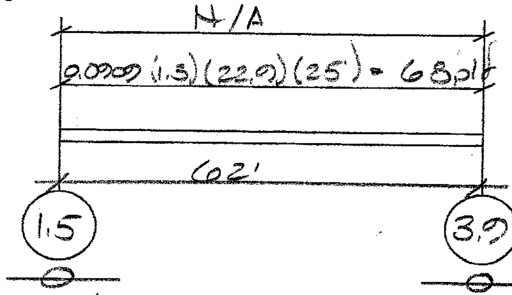
D PRODUCT 207

DIAPHRAGM: H

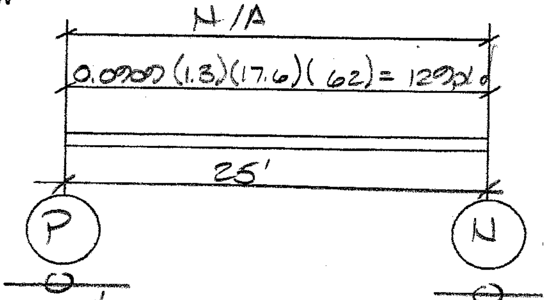
RF

FLR

N/S



E/W



WIND

SEISMIC

N/S				E/W		
0		0	WIND	0		0
2110#		2110#	SEISMIC	1615#		1615#
	85plf		V		26plf	
		—	STRUT	650#		1615#
800#			CHORD			
	1310#		COMBINED		165#	
			WIND			
			SEISMIC			
			V			
			STRUT			

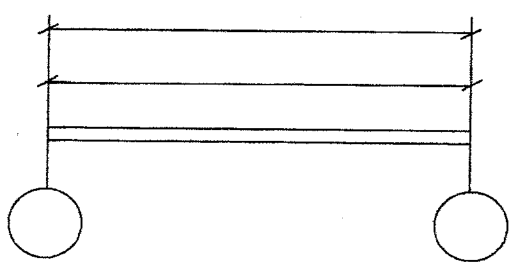
DIAPHRAGM:

RF

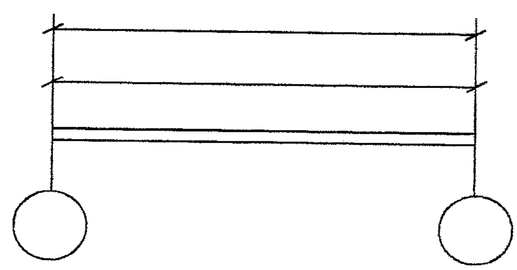
FLR

NOT USED

N/S



E/W



WIND

SEISMIC

WIND

SEISMIC

V

STRUT

CHORD

COMBINED

WIND

SEISMIC

V

STRUT

WIND = 0.6D

SEISMIC = $(0.60 - 0.14S_{DS})D = (0.6 - 0.14(0.844)) = 0.48D$

$V = P/L_{TOT}$	M_{OT} (% OF WALL) (P) (HT) =	M_{RES} $W_{DL} L^2/2$	P_{UP} $M_{OT} - M_{RES} / L$ =	USE HOLDOWN
	NOT USED			
	NOT USED			
	NOT USED			
1/S (1.5) 2110# / 11'-6" 184 plf	5.5/11.5 (2110)(11') = 11100 #ft	155(0.48)(5.5)²/2 = 1125 #ft	2000#	HDU2 TO FDN
1/S (3.9) 2110# / 13' 163 plf	2110#(11') = 23210 #ft	155(0.48)(13)²/2 = 6285 #ft	1355#	HDU2 TO FDN
1/S (P) 1615# / 13'-6" 120 plf	1615(11) = 17765 #ft	260(0.48)(13.5)²/2 = 11370 #ft	405#	HDU2 TO FDN
1/S (N) 1615# / 10' 162 plf	1615(11) = 17765 #ft	260(0.48)(10)²/2 = 6240 #ft	1215#	HDU2 TO FDN
	NOT USED			
	NOT USED			

Shawn Pierce Engineering

277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BOREAL EXPANSION 09.9-2/2

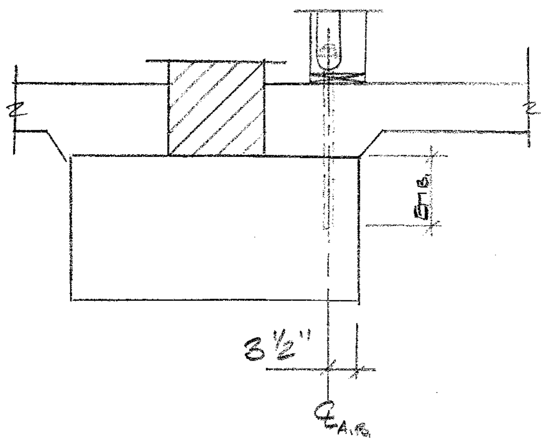
SHEET NO. ______ OF ______

CALCULATED BY ______ DATE 2/10

CHECKED BY ______ DATE ______

SCALE ______

HOLDOWN INTO (E) FOOTING LINE (N)



$P_{up} = 1215^{\#}$ (SEISMIC)

$P_u = 1215/0.7 = 1740^{\#}$

USE: 5/8" Ø A307 ALL THREAD ROD EPOXIED INTO (E) FTG W/ SIMPSON 'SET-XP' EPOXY W/ EMB = 6" (MIN) & 3 1/2" MIN EDGE DISTANCE

SEE 'SIMPSON ANCHOR DESIGNER FOR 318' RESULTS

TYPICAL HOLDOWN BOLT INSTALLATION IN (N) FOOTING

FOR LOCATION OF BOLT, SEE SECTION ABOVE

$P_{up} = 2000^{\#}$ (SEISMIC)

$P_u = 2000^{\#}/0.7 = 2860^{\#}$

USE: 5/8" Ø F1554 GRADE 36 HEAVY HEX BOLT W/ 6" MIN EMBEDMENT LENGTH, 3 3/4" MIN EDGE DISTANCE

SEE 'SIMPSON ANCHOR DESIGNER FOR ACI 318' RESULTS

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Holdown into (E) Footing - Line N

Date/Time : 2/5/2010 8:00:04 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
5/8" SET-XP	A307 GR. C	1	5	1

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
B tension and shear	18	No
Hole Condition	Inspection	Temp. Range
Dry Concrete	Continuous	1

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
1740	0	0	0	0

e_x (in)	e_y (in)	Mod/high seismic	Anchor w/ sustained tension	Anchor only resists wind/seis loads	Apply entire shear @ front row
0	0	Yes	No	Yes	No

Individual Anchor Tension Loads

N_{ua1} (lb)
1740.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)
0.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

13110# >> 2.5x1740#

Anchor Design OK

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
13110	9832.50	1740.00	0.1770

Concrete Breakout (Φ = 0.65 , Φ_{seis} = 0.75)

N_{cb}(lb)	ΦN_{cb}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{cb}$
5854.03	2853.84	1740.00	0.6097

Adhesive (Φ = 0.65 , Φ_{seis} = 0.75)

N_a(lb)	ΦN_a(lb)	N_{ua}(lb)	N_{ua} /ΦN_a
4342.15	2116.80	1740.00	0.8220

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.65 , $\alpha_{v.seis}$ = 0.71)

V_{eq}(lb)	ΦV_{eq}(lb)	V_{ua}(lb)	V_{ua} /ΦV_{eq}
5584.15	3629.70	0.00	0.0000

Concrete Breakout (case 1) (Φ = 0.70 , Φ_{seis} = 0.75)

V_{cbx}(lb)	ΦV_{cbx}(lb)	V_{uax}(lb)	V_{uax} /ΦV_{cbx}
2746.17	1441.74	0.00	0.0000

V_{cby}(lb)	ΦV_{cby}(lb)	V_{uay}(lb)	V_{uay} /ΦV_{cby}	V_{ua} /ΦV_{cb}
8181.50	4295.29	0.00	0.0000	0.0000

Concrete Breakout (case 2) does not apply to single anchor layout

Concrete Breakout (case 3) (Φ = 0.70 , Φ_{seis} = 0.75)

c_{x1} edge

V_{cby}(lb)	ΦV_{cby}(lb)	V_{uay}(lb)	V_{uay} /ΦV_{cby}
40262.18	21137.64	0.00	0.0000

c_{y1} edge

V_{cbx}(lb)	ΦV_{cbx}(lb)	V_{uax}(lb)	V_{uax} /ΦV_{cbx}
21781.91	11435.50	0.00	0.0000

c_{x2} edge

V_{cby}(lb)	ΦV_{cby}(lb)	V_{uay}(lb)	V_{uay} /ΦV_{cby}
5492.35	2883.48	0.00	0.0000

c_{y2} edge

V_{cbx}(lb)	ΦV_{cbx}(lb)	V_{uax}(lb)	V_{uax} /ΦV_{cbx}	V_{ua} /ΦV_{cb}
21781.91	11435.50	0.00	0.0000	0.0000

Pryout (Φ = 0.70 , Φ_{seis} = 0.75)

V_{cp}(lb)	ΦV_{cp}(lb)	V_{uax}(lb)	V_{uax} /ΦV_{cp}
8684.30	4559.26	0	0.0000

V_{cp}(lb)	ΦV_{cp}(lb)	V_{uay}(lb)	V_{uay} /ΦV_{cp}	V_{ua} /ΦV_{cp}
8684.30	4559.26	0	0.0000	0.0000

Interaction check

V.Max(0) <= 0.2 and T.Max(0.82) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 5/8" diameter A307 GR. C SET-XP anchor(s) with 5 in. embedment

BRITTLE FAILURE GOVERNS: Governing anchor failure mode is brittle failure. Per 2006 IBC Section 1908.1.16, anchors shall be governed by a ductile steel element in structures assigned to Seismic Design Category C, D, E, or F. Alternatively the minimum design strength of the anchor(s) shall be at least 2.5 times the factored forces or the anchor attachment to the structure shall undergo ductile yielding at a load level less than the design strength of the anchor(s). Designer must exercise own judgement to determine if this design is suitable.

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Holdown into (N) Footing

Date/Time : 2/5/2010 8:27:09 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
5/8" Heavy Hex Bolt	F1554 GR. 36	1	6	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
B tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
2860	0	0	0	0

e_x(in)	e_y(in)	Mod/high seismic	Apply entire shear @ front row
0	0	Yes	No

Individual Anchor Tension Loads

N_{ua1} (lb)
2860.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)
0.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

13100#>>2.5*2860#

Anchor Design OK

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
13100	9825.00	2860.00	0.2911

Concrete Breakout (Φ = 0.70 , Φ_{seis} = 0.75)

N_{cb}(lb)	ΦN_{cb}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{cb}$
9196.65	4828.24	2860.00	0.5923

Pullout ($\Phi = 0.70$, $\Phi_{seis} = 0.75$)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	$N_{ua} / \Phi N_{pn}$
13420.00	7045.50	2860.00	0.4059

Side-Face Blowout does not apply

Shear Strengths

Steel ($\Phi = 0.65$)

V_{eq}(lb)	ΦV_{eq}(lb)	V_{ua}(lb)	$V_{ua} / \Phi V_{eq}$
7865	5112.25	0.00	0.0000

Concrete Breakout (case 1) ($\Phi = 0.70$, $\Phi_{seis} = 0.75$)

V_{cbx}(lb)	ΦV_{cbx}(lb)	V_{uax}(lb)	$V_{uax} / \Phi V_{cbx}$
3045.60	1598.94	0.00	0.0000

V_{cby}(lb)	ΦV_{cby}(lb)	V_{uay}(lb)	$V_{uay} / \Phi V_{cby}$	$V_{ua} / \Phi V_{cb}$
8306.98	4361.17	0.00	0.0000	0.0000

Concrete Breakout (case 2) does not apply to single anchor layout

Concrete Breakout (case 3) ($\Phi = 0.70$, $\Phi_{seis} = 0.75$)

c_{x1} edge

V_{cby}(lb)	ΦV_{cby}(lb)	V_{uay}(lb)	$V_{uay} / \Phi V_{cby}$
37745.79	19816.54	0.00	0.0000

c_{y1} edge

V_{cbx}(lb)	ΦV_{cbx}(lb)	V_{uax}(lb)	$V_{uax} / \Phi V_{cbx}$
22008.81	11554.62	0.00	0.0000

c_{x2} edge

V_{cby}(lb)	ΦV_{cby}(lb)	V_{uay}(lb)	$V_{uay} / \Phi V_{cby}$
6091.20	3197.88	0.00	0.0000

c_{y2} edge

V_{cbx}(lb)	ΦV_{cbx}(lb)	V_{uax}(lb)	$V_{uax} / \Phi V_{cbx}$	$V_{ua} / \Phi V_{cb}$
22008.81	11554.62	0.00	0.0000	0.0000

Pryout ($\Phi = 0.70$, $\Phi_{seis} = 0.75$)

V_{cp}(lb)	ΦV_{cp}(lb)	V_{uax}(lb)	$V_{uax} / \Phi V_{cp}$
18393.31	9656.49	0	0.0000

V_{cp}(lb)	ΦV_{cp}(lb)	V_{uay}(lb)	$V_{uay} / \Phi V_{cp}$	$V_{ua} / \Phi V_{cp}$
18393.31	9656.49	0	0.0000	0.0000

Interaction check

V.Max(0) <= 0.2 and T.Max(0.59) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 5/8" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 6 in. embedment

BRITTLE FAILURE GOVERNS: Governing anchor failure mode is brittle failure. Per 2006 IBC Section 1908.1.16, anchors shall be governed by a ductile steel element in structures assigned to Seismic Design Category C, D, E, or F. Alternatively the minimum design strength of the anchor(s) shall be at least 2.5 times the factored forces or the anchor attachment to the structure shall undergo ductile yielding at a load level less than the design strength of the anchor(s). Designer must exercise own judgement to determine if this design is suitable.

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name : Holdown into (N) Footing - 12" Wide Ftg

Date/Time : 2/5/2010 9:18:16 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
5/8" Heavy Hex Bolt	F1554 GR. 36	1	6	N/A

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Normal weight	Yes	2500.0	1.00

Condition	Thickness (in)	Suppl. Edge Reinforcement
B tension and shear	18	No

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
2860	0	0	0	0

e_x(in)	e_y(in)	Mod/high seismic	Apply entire shear @ front row
0	0	Yes	No

Individual Anchor Tension Loads

N_{ua1} (lb)
2860.00

e'_{Nx}(in)	e'_{Ny}(in)
0.00	0.00

Individual Anchor Shear Loads

V_{ua1} (lb)
0.00

e'_{Vx}(in)	e'_{Vy}(in)
0.00	0.00

13100#>>2.5x2860#

Anchor OK

Tension Strengths

Steel (Φ = 0.75)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
13100	9825.00	2860.00	0.2911

Concrete Breakout (Φ = 0.70 , Φ_{seis} = 0.75)

N_{cb}(lb)	ΦN_{cb}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{cb}$
10274.31	5394.01	2860.00	0.5302

Pullout (Φ = 0.70 , Φ_{seis} = 0.75)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	N_{ua} /ΦN_{pn}
13420.00	7045.50	2860.00	0.4059

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.65)

V_{eq}(lb)	ΦV_{eq}(lb)	V_{ua}(lb)	V_{ua} /ΦV_{eq}
7865	5112.25	0.00	0.0000

Concrete Breakout (case 1) (Φ = 0.70 , Φ_{seis} = 0.75)

V_{cbx}(lb)	ΦV_{cbx}(lb)	V_{uax}(lb)	V_{uax} /ΦV_{cbx}
6163.86	3236.03	0.00	0.0000

V_{cby}(lb)	ΦV_{cby}(lb)	V_{uay}(lb)	V_{uay} /ΦV_{cby}	V_{ua} /ΦV_{cb}
4649.08	2440.76	0.00	0.0000	0.0000

Concrete Breakout (case 2) does not apply to single anchor layout

Concrete Breakout (case 3) (Φ = 0.70 , Φ_{seis} = 0.75)

c_{x1} edge

V_{cby}(lb)	ΦV_{cby}(lb)	V_{uay}(lb)	V_{uay} /ΦV_{cby}
12327.72	6472.06	0.00	0.0000

c_{y1} edge

V_{cbx}(lb)	ΦV_{cbx}(lb)	V_{uax}(lb)	V_{uax} /ΦV_{cbx}
11622.69	6101.91	0.00	0.0000

c_{x2} edge

V_{cby}(lb)	ΦV_{cby}(lb)	V_{uay}(lb)	V_{uay} /ΦV_{cby}
12327.72	6472.06	0.00	0.0000

c_{y2} edge

V_{cbx}(lb)	ΦV_{cbx}(lb)	V_{uax}(lb)	V_{uax} /ΦV_{cbx}	V_{ua} /ΦV_{cb}
11622.69	6101.91	0.00	0.0000	0.0000

Pryout (Φ = 0.70 , Φ_{seis} = 0.75)

V_{cp}(lb)	ΦV_{cp}(lb)	V_{uax}(lb)	V_{uax} /ΦV_{cp}
20548.61	10788.02	0	0.0000

V_{cp}(lb)	ΦV_{cp}(lb)	V_{uay}(lb)	V_{uay} /ΦV_{cp}	V_{ua} /ΦV_{cp}
20548.61	10788.02	0	0.0000	0.0000

Interaction check

V.Max(0) <= 0.2 and T.Max(0.53) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 5/8" diameter F1554 GR. 36 Heavy Hex Bolt anchor(s) with 6 in. embedment

BRITTLE FAILURE GOVERNS: Governing anchor failure mode is brittle failure. Per 2006 IBC Section 1908.1.16, anchors shall be governed by a ductile steel element in structures assigned to Seismic Design Category C, D, E, or F. Alternatively the minimum design strength of the anchor(s) shall be at least 2.5 times the factored forces or the anchor attachment to the structure shall undergo ductile yielding at a load level less than the design strength of the anchor(s). Designer must exercise own judgement to determine if this design is suitable.

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZAL EXPANSION 09.9-2/2
SHEET NO. ____ OF ____
CALCULATED BY ____ DATE 12/09
CHECKED BY ____ DATE ____
SCALE ____

(E) FLOOR DIAPHRAGM DEFLECTION

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188\,L\,e_n + \frac{\Sigma(\Delta_c X)}{2b} \quad \text{(BLOCKED DIAPHRAGM)}$$

A = DBL 2x6 (MIN) = 16.5 in²
b = 41'
E = DFL #2 = 1.6×10^6 psi
e_n = ASSUME MAX SHEAR FOR UNBLOCKED DIAPH. PERP TO JOISTS = 0.01(1.2) = 0.012
v = 215 plf / # OF NAILS PER FOOT = 112.5 #/NAIL (CASE 2 LOADING)
Gt = 46,500 #/in
L = 20'
v = 215 plf (MAX LOAD PER CASE 2 OF UNBLOCKED DIAPHRAGM
Δ_c = 0.03 (TENSION) / 0.005 (COMPRESSION) (PER APA RESEARCH REPORT 138)
X = 16' SPLICES AT 8' CENTERS (ASSUMED)

$$\Delta_{BEND} = \frac{5vL^3}{8EA_b} = 9.9 \times 10^{-4}\,''$$

$$\Delta_{SHEAR} = \frac{vL}{4Gt} = 0.023''$$

$$\Delta_{NAIL\ SLIP} = 0.188\,L\,e_n = 0.045''$$

$$\Delta_{CHORD\ SPLICE} = \frac{\Sigma \Delta_c X}{2b} = \text{TENSION} = 0.03(8) + 0.03(16) = 0.72$$

COMPRESSION = 0.005(8') + 0.005(16') = 0.12

$$= (0.72 + 0.12)/2(41') = 0.01''$$

$$\Delta_{TOTAL} = (9.9 \times 10^{-4} + 0.023 + 0.045 + 0.01)\,2.5 = 0.197''$$

└ FACTOR FOR UNBLOCKED DIAPHRAGM

D PRODUCT 207

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277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORZAL EXPANSION 09.9-2/2
SHEET NO. ________ OF ________
CALCULATED BY ________ DATE 12/09
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SCALE ________

(H) FLOOR DIAPHRAGM DEFLECTION

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188\,Le_n + \frac{\Sigma(\Delta_c X)}{2b}$$ (BLOCKED DIAPHRAGM)

A = AREA OF CHORD X-SECTION = DBL 2×6 = 16.5 in^2
b = DIAPHRAGM WIDTH = 23.83'
E = ELASTIC MODULUS OF CHORDS = DF-L #2 = 1.6×10^6 psi
e_n = NAIL DEFORMATION = 85 plf / 2 NAILS/ft = 43#/NAIL w/10d NAILS = 0.00
INCREASE 0.00 × 20% FOR NON STRUCT I DIAPHRAGM = 0.00
Gt = PANEL RIGIDITY (PER TABLE 2305.2.2(2) = 60/40 PANEL RATING, 1 1/8" THICK
= 46,500 #/in
L = DIAPHRAGM LENGTH = 61'
v = MAX SHEAR = 85 plf
$\Sigma(\Delta_c X)$ = IND. CHORD SLIP VALUES
Δ_c = 0.03 (PER APA RESEARCH REPORT 138) / 0.005" IN COMPRESSION CHORD SPLICES LOCATED AT 8' CENTERS (ASSUMED)
X = DIST FROM CHORD SPLICE TO CLOSEST SUPPORTING SHEARWALL
= 16' (ASSUMED)

$$\Delta_{BEND} = \frac{5vL^3}{8EAb} = 0.019$$

$$\Delta_{SHEAR} = \frac{vL}{4Gt} = 0.027$$

$$\Delta_{NAIL\ SLIP} = 0.188\,Le_n = \emptyset$$

$$\Delta_{CHORD\ SPLICE} = \frac{\Sigma(\Delta_c X)}{2b} = \text{TENSION} = (0.03(8) + 0.03(16) + 0.03(24) + 0.03(32))\,2 = 4.8$$
$$= \text{COMPRESSION} = (0.005(8+16+24+32))(2) = 0.8$$
$$= (4.8 + 0.8)/2(23.83)$$
$$= 0.117$$

$$\Delta_{TOTAL} = (0.019 + 0.027 + 0.000 + 0.117)\underbrace{2.5}_{\text{FACTOR FOR UN BLOCKED DIAPHRAGM}} = 0.408(2.5) = 1.02''$$

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277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

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(14) SHEARWALL DEFLECTION

USE SHEARWALLS ON LINE 1.5 FOR ANALYSIS

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + \frac{hd_a}{b}$$

$v = 184\,plf$

$h = 11'$

$E = 4\times6$ DF#2 $= 1.6\times10^6\,psi$

$A = 19.25\,in^2$

$Gt = 3/8"$ SHTG w/ 24/0 SPAN RATING $= 25{,}000\ \#/in$

$b = 5'\text{-}6''$

$e_n =$ LOAD PER NAIL $= 184\,plf / 2\,NAILS/ft = 92\#/NAIL$

$= 1.2\,(v_{NAIL}/616)^{3.018} = 0.003$

$d_a =$ HDU2 $= 0.088$ @ $3075\#$ (ALLOWABLE LOAD)

ASSUMING HOLD-DOWN SLIP TO BE LINEAR

$0.088\,(2000/3075) = 0.057$

$\Delta_{BEND} = \frac{8vh^3}{EAb} = 0.012$

$\Delta_{SHEAR} = \frac{vh}{Gt} = 0.081$

$\Delta_{NAIL\ SLIP} = 0.75he_n = 0.025$

$\Delta_{HD} = \frac{hd_a}{b} = 0.114$

$\Delta_t = 0.012 + 0.081 + 0.025 + 0.114 = 0.232''$

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277 N. Beechnut Ave.
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STORY DRIFT / BUILDING SEPARATION

BASED ON MAX DIAPHRAGM DEFLECTION

(N) DIAPHRAGM DEFLECTION = 1.02" (1/0.7) = 1.46" ← CONTROLS
ADJUSTMENT BACK TO STRENGTH DESIGN

(E) DIAPHRAGM DEFLECTION = 0.197 (1/0.7) = 0.28"

(N) DIAPHRAGM SHEARWALL DEFLECTION = 0.232" (1/0.7) = 0.33"

δ_{xe} = 1.46" + 0.33" = 1.79"
C_d = 4 - BEARING WALL SYSTEMS / LT. FRAMED WALL W/ SHEATHING

$\delta_x = \frac{C_d \delta_{xe}}{I} = 4(1.79")/1.0 = 7.16"$

IF (N) DIAPHRAGM DEFLECTION IS BLOCKED THEN
δ_{DIAPH} = 0.408" (1/0.7) = 0.583"

δ_{xe} = 0.583 + 0.33" = 0.913"

δ_x = 4(0.913)/1 = 3.65"

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(805) 801-9385

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SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 2/10
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SCALE ______

(E) CMU WALL OUT-OF-PLANE LOADS - LINE 9 $h = 10'\text{-}8"$

$F_p = 0.40 S_{DS} I W_w \Rightarrow 0.40(0.844)(1.0)\, 78_{psf}$
$= 26.4_{psf}$

WALL TO FLOOR OUT-OF-PLANE ANCHORAGE

MIN ANCHORAGE REQ'S
a) $F_p = 0.40 S_{DS} I W_w$ $26.4_{psf}(10.67/2) = 145_{plf}$
b) $400 S_{DS} I =$ 325_{plf} ← GOVERNS SECTION 12.11.2
c) 280_{plf}

PER SECTION 12.11.2.1 FOR SDC C-F,

$F_p = 0.8 S_{DS} I W_w = 52.8_{psf}(10.67/2) = 282_{plf} < 325_{plf}$

ANCHORAGE REACTIONS:
$T = C$ AT $24"$ o.c. $= 650^{\#}$
$T = C$ AT $48"$ o.c. $= 1300^{\#}$

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JOB BORZM EXPANSION
SHEET NO. ____ OF ____
CALCULATED BY ____ DATE 4/10
CHECKED BY ____ DATE ____
SCALE ____

REPLACEMENT OF (E) PA 28 ANCHOR

CAPACITY OF (E) PA 28 ——— 2815#

TRY HDU2 TO SIDE OF (N) 4x FLOOR BM W/ 5/8"Ø A-307 ATR EPOXIED TO (E) CMU WALL W/ SIMPSON SET EPOXY

EDGE DISTANCE = 12"
TENSION CAPACITY = 1645# W/ 5" EMBED
= LOAD ADJUSTMENT FACTOR 0.92 (EDGE DIST.)
" " " 1.33 (SEISMIC FORCE)
= 1645#(0.92)(1.33)
= 2010# < 2815#

TO INCREASE CAPACITY, CONTINUE BOLT THROUGH CMU WALL & USE 6"SQ x 3/16" PL WASHER @ OPP. FACE

USE: HDU 2 HOLDOWN W/ 5/8"Ø A-307 ALL THREAD BOLT THROUGH (E) CMU WALL W/ SIMPSON 'SET' EPOXY & 6"SQ x 3/16" PL WASHER AT OPPOSITE FACE

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LATERAL ANALYSIS

Conference Room Expansion

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277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORRALL EXPANSION 09.07-2/2
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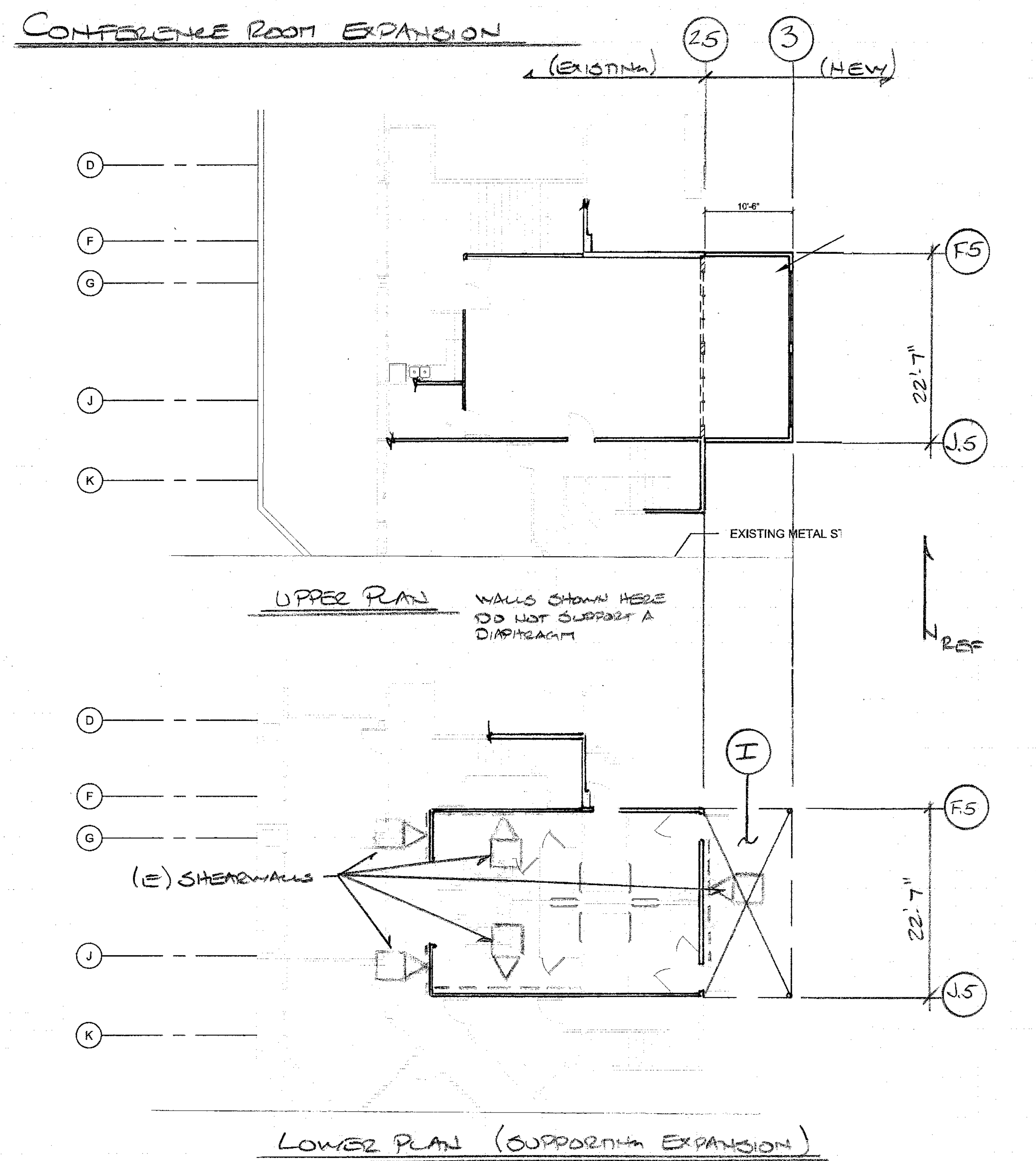
SCALE ______



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JOB Borzall Expansion 09.9-2/2
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 11/09
CHECKED BY ______ DATE ______
SCALE Conference Room Expansion

Diaphragm Loads

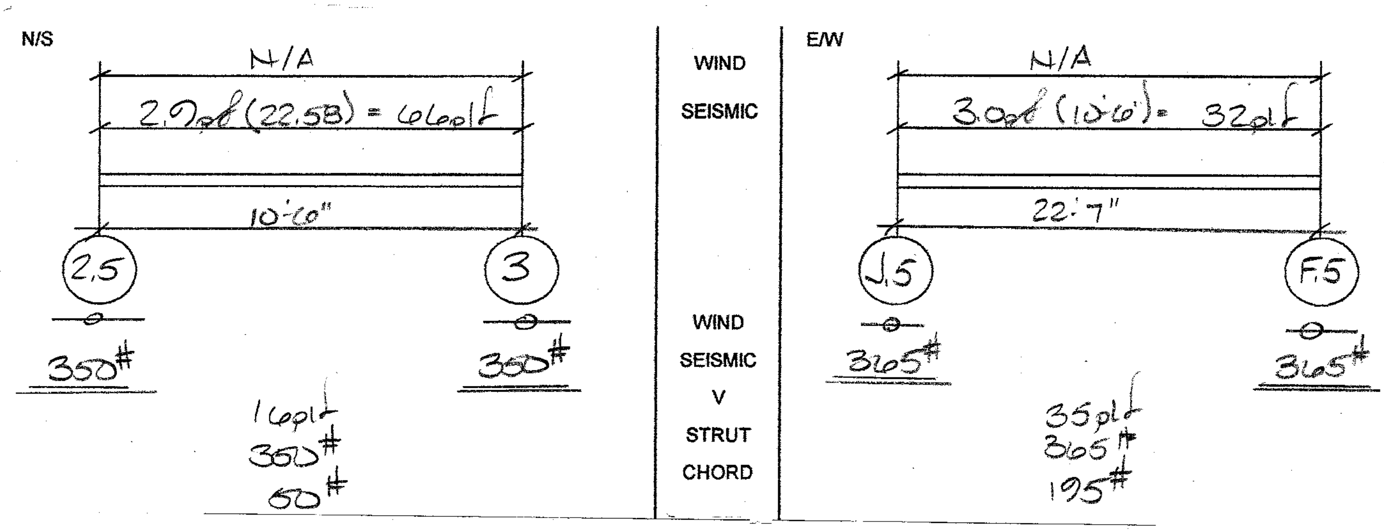
(I) N/S: 14psf + 12.33' [9psf (2.0)] 1/22.58 = 23.7psf
E/W: 14psf + 12.33' [9psf (1.0)] 1/10'6" = 24.6psf

SEIS	EXP
2.9psf	N/A
3.0psf	N/A

Lateral Load

WIND: N/A (NO EXPOSURE TO WIND LOADING)
SEISMIC: 0.0942 e = 0.122 W_D

DIAPHRAGM: (I) ~~RF~~ FLR



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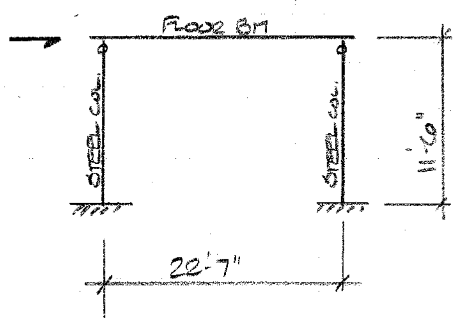
JOB: Borzall Expansion 09.9.-2/2
SHEET NO.: OF:
CALCULATED BY: DATE: 11/09
CHECKED BY: DATE:
SCALE: Conference Room Expansion

SEISMIC RESISTANT AT LINE 3

LATERAL LOAD — $350^{\#}$ $R = 6.5$

IF CANTILEVERED COLUMN SYSTEM USED $R = 1.5$

LATERAL LOAD $= 350^{\#}\left(6.5/1.5\right) = 1520^{\#}$



LOAD PER COLUMN $= 1520^{\#}/2$ COLUMNS
$= 760^{\#}$/COLUMN

$M_{MAX_{LAT}} = 760^{\#}(11.5') = 8740^{\#}ft$

$S_{x_{REQD}} = M_{MAX}(\Omega)/F_b' = 8740^{\#}ft(1.25)/0.6(F_y)$

$= 6.24 in^3$ (A53 GR B PIPE)
$= 5.20 in^3$ (A500 GR B 42ksi ROUND HSS)
$= 4.75 in^3$ (A500 GR B 46ksi RECT HSS)

START w/ 6" SCHED 40 (STD) ø PIPE, HSS 5.000 x 0.375 ROUND, OR HSS $4\frac{1}{2}$ x $4\frac{1}{2}$" x $\frac{5}{16}$ RECT

DRIFT LIMITS

$\Delta_a = 0.020 h_{sx} = 2.76$

SDS CAT D $\Delta_a/\rho = 2.12"$

$\delta_x = C_d\, \delta_{xe}/I$

$\delta_x = \Delta_a/\rho \therefore \delta_{xe_{MAX}} = \dfrac{\Delta_a/\rho\,(I)}{C_d}$

$\delta_{xe} = 2.12(1.0)/1.25$

$= 1.70"$ MAX DEFLECTION

$I_{xx} = P\ell^3/3E\delta_{xe}$

$= 13.51 in^4$

ALL SECTIONS PREVIOUSLY NOTED STILL MEET DRIFT REQUIREMENTS

D PRODUCT 207

Steel Column

Description : Cantilevered Column at Line 3

General Information

Code Ref : 2006 IBC, AISC Manual 13th Edition

Steel Section Name : **HSS6X6X3/16**
Analysis Method : 2006 IBC & ASCE 7-05
Steel Stress Grade , A-500, Grade B, Fy = 46 ksi, Carbon Steel
Fy : Steel Yield 46.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi
Load Combination : Allowable Stress

Overall Column Height 11.50 ft
Top & Bottom Fixity Top Free, Bottom Fixed

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 10ft, K = 2.1
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 10 ft, K = 2.1

Applied Loads

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

Column self weight included : 166.83 lbs * Dead Load Factor
AXIAL LOADS . . .
Floor Beam on Line 3: Axial Load at 11.50 ft, D = 2.920, L = 7.035 k
BENDING LOADS . . .
Seismic Load: Lat. Point Load at 11.50 ft creating Mx-x, E = 1.090 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.4818** : 1
Load Combination +D+0.70E+H
Location of max.above base 0.0 ft
At maximum location values are . . .
Pu : Axial 3.087 k
Pn / Omega : Allowable 51.146 k
Mu-x : Applied 8.775 k-ft
Mn-x / Omega : Allowable 19.431 k-ft
Mu-y : Applied 0.0 k-ft
Mn-y / Omega : Allowable 19.809 k-ft

PASS Maximum Shear Stress Ratio = **0.02270** : 1
Load Combination +D+0.70E+H
Location of max.above base 0.0 ft
At maximum location values are . . .
Vu : Applied 0.7630 k
Vn / Omega : Allowable 33.613 k

Maximum SERVICE Load Reactions . .
Top along X-X 0.0 k
Bottom along X-X 0.0 k
Top along Y-Y 0.0 k
Bottom along Y-Y 1.090 k

Maximum SERVICE Load Deflections . . .
Along Y-Y -1.469 in at 11.50 ft above base
for load combination : E Only
Along X-X 0.0 in at 0.0 ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D	0.060	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+L+H	0.198	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+Lr+H	0.060	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750Lr+0.750L+H	0.164	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.70E+H	0.482	PASS	0.00 ft	0.023	PASS	0.00 ft
+D-0.70E+H	0.482	PASS	0.00 ft	0.023	PASS	0.00 ft
+D+0.750Lr+0.750L+0.5250E+H	0.420	PASS	0.00 ft	0.017	PASS	0.00 ft
+D+0.750Lr+0.750L-0.5250E+H	0.420	PASS	0.00 ft	0.017	PASS	0.00 ft
+D+0.750L+0.750S+0.5250E+H	0.420	PASS	0.00 ft	0.017	PASS	0.00 ft
+D+0.750L+0.750S-0.5250E+H	0.420	PASS	0.00 ft	0.017	PASS	0.00 ft
+0.60D+0.70E+H	0.470	PASS	0.00 ft	0.023	PASS	0.00 ft
+0.60D-0.70E+H	0.470	PASS	0.00 ft	0.023	PASS	0.00 ft

Maximum Reactions - Unfactored

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top
D Only				
L Only				
E Only			1.090	
-E Only			-1.090	

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

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CHECKED BY ____ DATE ____
SCALE CONFERENCE ROOM EXTENSION

✓ SEISMIC RESTRAINT AT LINE (3) (CONT.)

BASE PL DESIGN

$M_{MAX} = 8740^{\#}$ ft $(12''/ft)(\Omega) = 131{,}100^{\#''}$

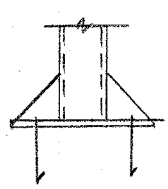
TENSION AT FACE OF COLUMN PIPE = $M_{MAX}/6''$ = $21850^{\#}$
ROUND HSS = $M_{MAX}/5''$ = $26220^{\#}$
RECT HSS = $M_{MAX}/4.5'$ = $29135^{\#}$

WELD REQ'D = TENSION / $(900^{\#}/in/\tfrac{1}{16})(\ell)$
PIPE = 1/2" FILLET ($\ell = 3''$)
ROUND HSS = 3/4" FILLET ($\ell = 2.5''$)
RECT HSS = 1/2" FILLET ($\ell = 4$)

DUE TO THICKNESS OF FILLET REQ'D
GUSSET PL'S WILL BE REQ'D OR
USE WIDE FLANGE SECTION

MIN W SECTION = W6 × 12

USING 3" × 3" × 3/4" GUSSET PL'S (1) BOTH SIDES



WELD REQ'D PIPE = 3/16" FILLET ($\ell = 9''$)
ROUND HSS = 1/4" FILLET ($\ell = 8.5''$)
RECT HSS = 1/4" FILLET ($\ell = 10''$)

✓ ANCHOR BOLTS

M_{MAX} PER ACI 318 SECTION 9.2 ⇒ 1.0E = $8740^{\#}$ ft $(1/0.7)$ ← ASD → STRENGTH
= $12490^{\#}$ ft

V_{MAX} PER ACI 318 SECTION 9.2 ⇒ 1.0E = $760^{\#}$/COLUMN $(1/0.7)$
= $1090^{\#}$

SUMMARY: LINES (3) & (E.5) USE: (6) 1/2"Ø F1554 GRADE 36 A.B. W/ HEAVY HEX NUT & WASHER EMBED 16" (MIN)

LINES (3) & (J.5) USE: (4) 1/2"Ø A449 A.B. W/ HEAVY HEX NUT & WASHER, EMBED 12" (MIN)

SEE 'SIMPSON ANCHOR DESIGNER FOR ACI 318' RESULTS

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Steel Base Plate Design

Description : Base Plate - Column at Intersection of Lines J.5 & 3

General Information

Calculations per 13th AISC & AISC Design Guide No. 1, 1990 by DeWolf & Ricker

Material Properties

AISC Design Method: Allowable Stress Design
Steel Plate Fy = 36.0 ksi
Concrete Support f'c = 2.50 ksi
Assumed Bearing Area : Full Bearing

ASIF : Allowable Stress Increase Factor 1.0
ABIF : Allowable Bearing Increase Factor 1.0
Ω_c : ASD Safety Factor. 2.50
Allowable Bearing Fp per J8 3.188 ksi

Column & Plate

Column Properties

Steel Section : HSS6X6X3/16

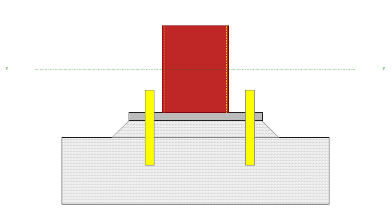
Depth	6	in	Area	3.98	in^2
Width	6	in	Ixx		in^4
Flange Thickness	0.174	in	Iyy		in^4
Web Thickness		in			

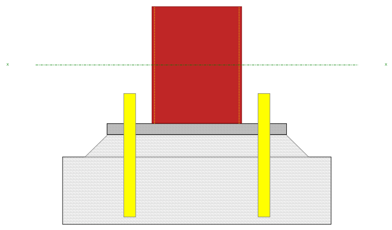
Plate Dimensions

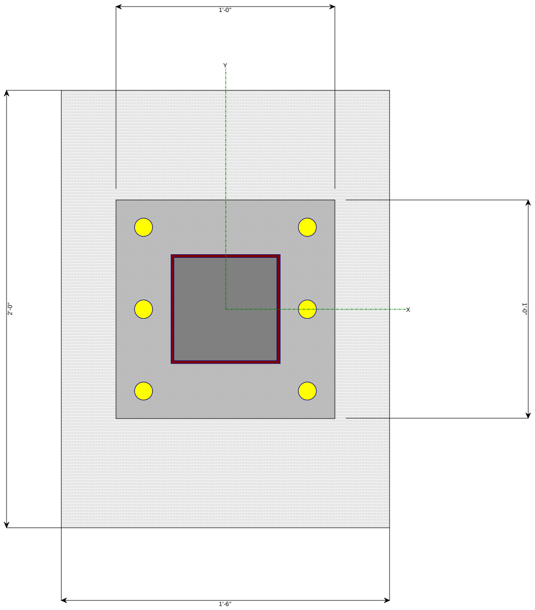
N : Length 12.0 in
B : Width 12.0 in
Thickness 0.750 in
Column assumed welded to base plate.

Support Dimensions

Support width along "X" 24.0 in
Length along "Z" 18.0 in



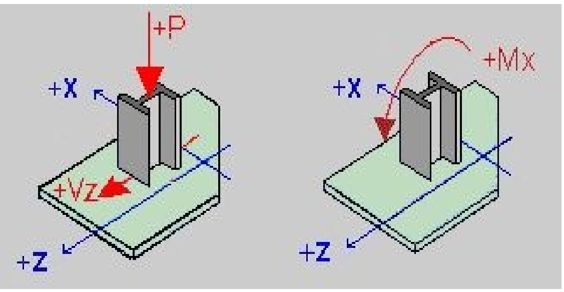




Applied Loads

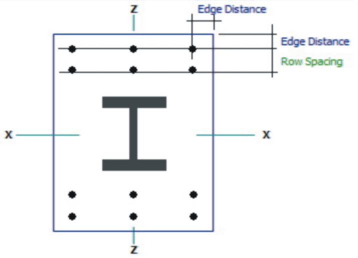
	P-Y	V-Z	M-X
D : Dead Load	k	k	k-ft
L : Live	k	k	k-ft
Lr : Roof Live	k	k	k-ft
S : Snow	k	k	k-ft
W : Wind	k	k	k-ft
E : Earthquake	k	1.090 k	12.490 k-ft
H : Lateral Earth	k	k	k-ft

" P " = Gravity load, "+" sign is downward. "+" Moments create higher soil pressure at +Z edge.
"+" Shears push plate towards +Z edge.



Anchor Bolts

Anchor Bolt or Rod Description	Anchor Bolt	
Max of Tension or Pullout Capacity..........	6.180	k
Shear Capacity..	2.570	k
Edge distance : bolt to plate..................	1.50	in
Number of Bolts in each Row..................	2.0	
Number of Bolt Rows.......................	3.0	
Row Spacing................................	4.50	in



Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Steel Base Plate Design

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ENERCALC, INC. 1983-2009, Ver: 6.0.24, N:40336

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Description : Base Plate - Column at Intersection of Lines J.5 & 3

General Information

Calculations per 13th AISC & AISC Design Guide No. 1, 1990 by DeWolf & Ricker

Material Properties

AISC Design Method	Allowable Stress Design		ASIF : Allowable Stress Increase Factor	1.0
Steel Plate Fy	=	36.0 ksi	ABIF : Allowable Bearing Increase Factor	1.0
Concrete Support f'c	=	2.50 ksi	Ω_c : ASD Safety Factor.	2.50
Assumed Bearing Area :	Full Bearing		Allowable Bearing Fp per J8	3.188 ksi

Column & Plate

Column Properties

Steel Section : HSS6X6X3/16

Depth	6 in		Area	3.98 in^2
Width	6 in		Ixx	in^4
Flange Thickness	0.174 in		Iyy	in^4
Web Thickness	in			

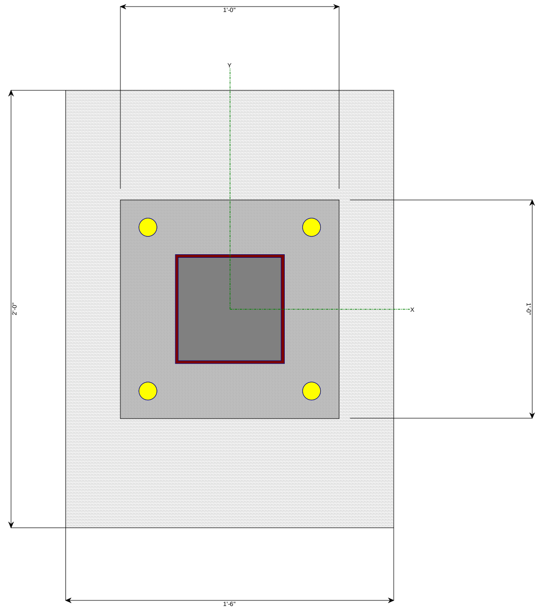


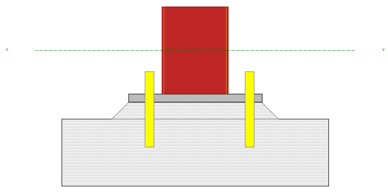
Plate Dimensions

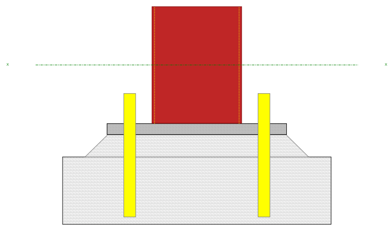
N : Length	12.0 in
B : Width	12.0 in
Thickness	0.750 in

Column assumed welded to base plate.

Support Dimensions

Support width along "X"	24.0 in
Length along "Z'	18.0 in

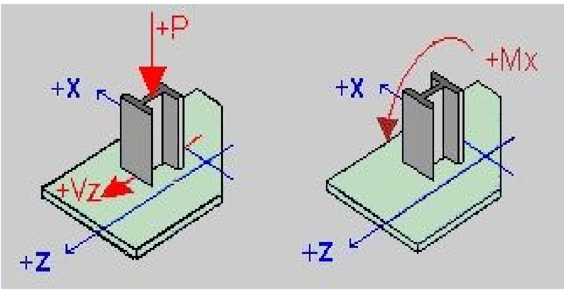




Applied Loads

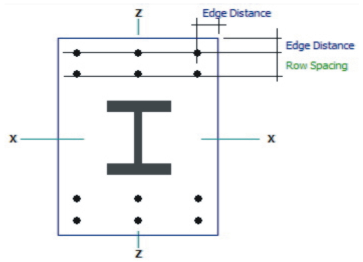
	P-Y	V-Z	M-X
D : Dead Load	k	k	k-ft
L : Live	k	k	k-ft
Lr : Roof Live	k	k	k-ft
S : Snow	k	k	k-ft
W : Wind	k	k	k-ft
E : Earthquake	k	1.090 k	12.490 k-ft
H : Lateral Earth	k	k	k-ft

" P " = Gravity load, "+" sign is downward.
"+" Moments create higher soil pressure at +Z edge.
"+" Shears push plate towards +Z edge.



Anchor Bolts

Anchor Bolt or Rod Description	Anchor Bolt
Max of Tension or Pullout Capacity...........	6.180 k
Shear Capacity...	2.570 k
Edge distance : bolt to plate..................	1.50 in
Number of Bolts in each Row..................	2.0
Number of Bolt Rows.......................	2.0
Row Spacing................................	9.0 in



Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.0)

Job Name : Borzall Expansion Date/Time : 11/5/2009 2:03:41 PM

1) Input

Calculation Method : ACI 318 Appendix D For Cracked Concrete

Calculation Type : Design

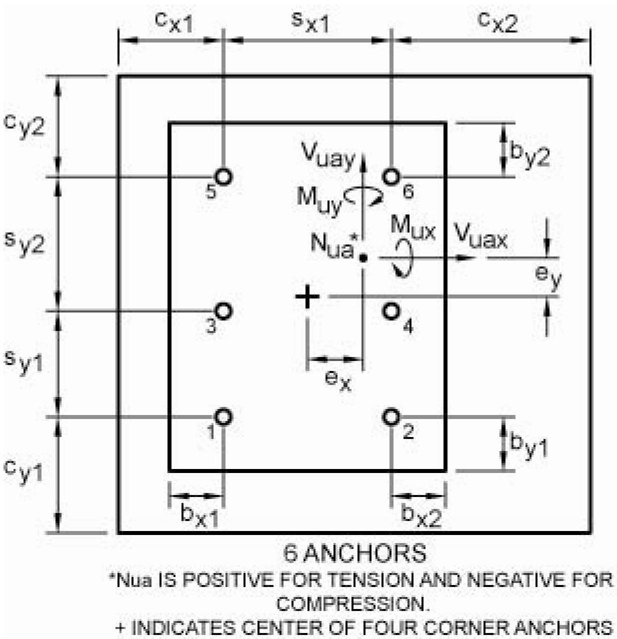
a) Layout

Anchor : 1/2" PAB4 Number of Anchors : 6

Steel Grade: AB Embedment Depth : 16 in

Built-up Grout Pads : Yes

Steel Grade AB = F1554 Grade 36



Anchor Layout Dimensions :

c_{x1} : 13.5 in

c_{x2} : 13.5 in

c_{y1} : 6 in

c_{y2} : 22 in

b_{x1} : 1.5 in

b_{x2} : 1.5 in

b_{y1} : 1.5 in

b_{y2} : 1.5 in

s_{x1} : 9 in

s_{y1} : 4.5 in

s_{y2} : 4.5 in

Base Plate will bear on both concrete pad footing and compression nuts to transfer load into pad footing

WARNING: EXCESS BEARING PRESSURE!
Calculated bearing pressure is 3072.26 psi and exceeds the permissible bearing stress of ϕ Fp per ACI 318 Section 10.17. Designer must exercise own judgement to determine if this design is suitable.

b) Base Material

Concrete : Normal weight

Cracked Concrete : Yes

Condition : A tension and shear

Thickness, h : 24 in

Supplementary edge reinforcement : No

f'_c : 2500.0 psi

$\Psi_{c,V}$: 1.20

ϕF_p : 1381.3 psi

c) Factored Loads

Load factor source : ACI 318 Section 9.2

N_{ua} : 0 lb

V_{uay} : -1090 lb

M_{uy} : 0 lb*ft

e_x : 0 in

e_y : 1.8125 in

V_{uax} : 0 lb

M_{ux} : -12490 lb*ft

Moderate/high seismic risk or intermediate/high design category : Yes

Apply entire shear load at front row for breakout : No

d) Anchor Parameters

Anchor Model = PAB4	d_o = 0.5 in
Category = N/A	h_{ef} = 15 in
h_{min} = 16.75 in	c_{ac} = 22.5 in
c_{min} = 3 in	s_{min} = 3 in
Ductile = Yes	

2) Tension Force on Each Individual Anchor

Anchor #1 N_{ua1} = 5784.02 lb

Anchor #2 N_{ua2} = 5784.02 lb

Anchor #3 N_{ua3} = 3046.53 lb

Anchor #4 N_{ua4} = 3046.53 lb

Anchor #5 N_{ua5} = 309.03 lb

Anchor #6 N_{ua6} = 309.03 lb

Sum of Anchor Tension ΣN_{ua} = 18279.17 lb

a_x = 0.00 in

a_y = 11.01 in

e'_{Nx} = 0.00 in

e'_{Ny} = 2.70 in

3) Shear Force on Each Individual Anchor

Resultant shear forces in each anchor:

Anchor #1 V_{ua1} = 181.67 lb (V_{ua1x} = 0.00 lb , V_{ua1y} = -181.67 lb)

Anchor #2 V_{ua2} = 181.67 lb (V_{ua2x} = 0.00 lb , V_{ua2y} = -181.67 lb)

Anchor #3 V_{ua3} = 181.67 lb (V_{ua3x} = 0.00 lb , V_{ua3y} = -181.67 lb)

Anchor #4 V_{ua4} = 181.67 lb (V_{ua4x} = 0.00 lb , V_{ua4y} = -181.67 lb)

Anchor #5 V_{ua5} = 181.67 lb (V_{ua5x} = 0.00 lb , V_{ua5y} = -181.67 lb)

Anchor #6 V_{ua6} = 181.67 lb (V_{ua6x} = 0.00 lb , V_{ua6y} = -181.67 lb)

Sum of Anchor Shear ΣV_{uax} = 0.00 lb, ΣV_{uay} = -1090.00 lb

e'_{Vx} = 0.00 in

e'_{Vy} = 0.00 in

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

$N_{sa} = nA_{se}f_{uta}$ [Eq. D-3]

Number of anchors acting in tension, n = 6

N_{sa} = 8235 lb (for each individual anchor)

ϕ = 0.75 [D.4.4]

ϕN_{sa} = 6176.25 lb (for each individual anchor)

5) Concrete Breakout Strength of Anchor Group in Tension [Sec. D.5.2]

$N_{cbg} = A_{Nc}/A_{Nco}\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ [Eq. D-5]

Number of influencing edges = 4

h_{ef} (adjusted for edges per D.5.2.3) = 14.667 in

A_{Nco} = 1936.00 in^2 [Eq. D-6]

A_{Nc} = 1332.00 in^2

$\Psi_{ec,Nx}$ = 1.0000 [Eq. D-9]

$\Psi_{ec,Ny}$ = 0.8908 [Eq. D-9]

$\Psi_{ec,N}$ = 0.8908 (Combination of x-axis & y-axis eccentricity factors.)

$\Psi_{ed,N}$ = 0.7818 [Eq. D-10 or D-11]

Note: Cracking shall be controlled per D.5.2.6

$\Psi_{c,N}$ = 1.0000 [Sec. D.5.2.6]

$\Psi_{cp,N}$ = 1.0000 [Eq. D-12 or D-13]

$N_b = 16\sqrt{f'_c}\, h_{ef}^{5/3}$ = 70303.30 lb [Eq. D-8]

N_{cbg} = 33688.52 lb [Eq. D-5]

ϕ = 0.75 [D.4.4]

ϕ_{seis} = 0.75

ϕN_{cbg} = 18949.79 lb (for the anchor group)

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

$N_p = 8A_{brg}f'_c$ [Eq. D-15]

A_{brg} = 1.3662 in^2

$N_{pn} = \Psi_{c,p}N_p$ [Eq. D-14]

$\Psi_{c,p}$ = 1.0 [D.5.3.6]

N_{pn} = 27324.00 lb

ϕ = 0.70 [D.4.4]

ϕ_{seis} = 0.75

$\phi\ N_{pn} = \phi\ N_{eq}$ = 14345.10 lb (for each individual anchor)

7) Side Face Blowout of Anchor in Tension [Sec. D.5.4]

Concrete side face blowout strength is only calculated for headed anchors in tension close to an edge, $c_{a1} < 0.4h_{ef}$. Not applicable in this case.

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

V_{eq} = 4940.00 lb (for each individual anchor)

ϕ = 0.65 [D.4.4]

$\phi\ V_{eq}$ = 3211.00 lb (for each individual anchor)

ϕV_{eq} is multiplied by 0.8 due to built-up grout pads...[Sec D.6.1.3]

ϕV_{eq} = 2568.80lb (for each individual anchor)

9) Concrete Breakout Strength of Anchor Group in Shear [Sec D.6.2]

Case 1: Anchor(s) closest to edge checked against sum of anchor shear loads at the edge
In x-direction...

$V_{cbgx} = A_{vcx}/A_{vcox}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-22]

c_{a1} = 13.50 in

A_{vcx} = 713.81 in^2

A_{vcox} = 820.13 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 0.7889 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{bx} = 18606.80 lb

V_{cbgx} = 15331.08 lb [Eq. D-22]

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgx} = 8623.74 lb (for the anchor group)

In y-direction...

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 6.00 in

A_{vcy} = 243.00 in^2

A_{vcoy} = 162.00 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{by} = 5513.13 lb

V_{cbgy} = 9923.63 lb [Eq. D-22]

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgy} = 5582.04 lb (for the anchor group)

Case 2: Anchor(s) furthest from edge checked against total shear load

In x-direction...

$V_{cbgx} = A_{vcx}/A_{vcox}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-22]

c_{a1} = 16.00 in (adjusted for edges per D.6.2.4)

A_{vcx} = 888.00 in^2

A_{vcox} = 1152.00 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 0.7750 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{bx} = 24007.73 lb

V_{cbgx} = 17210.54 lb [Eq. D-22]

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgx} = 9680.93 lb (for the entire anchor group)

In y-direction...

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 15.00 in

A_{vcy} = 810.00 in^2

A_{vcoy} = 1012.50 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 0.8800 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{by} = 21792.54 lb

V_{cbgy} = 18410.34 lb [Eq. D-22]

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgy} = 10355.82 lb (for the entire anchor group)

Case 3: Anchor(s) closest to edge checked for parallel to edge condition

Check anchors at c_{x1} edge

$V_{cbgx} = A_{vcx}/A_{vcox}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-22]

c_{a1} = 13.50 in

A_{vcx} = 713.81 in^2

A_{vcox} = 820.13 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{bx} = 18606.80 lb

V_{cbgx} = 19433.77 lb [Eq. D-22]

$V_{cbgy} = 2 * V_{cbgx}$ [Sec. D.6.2.1(c)]

V_{cbgy} = 38867.54 lb

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgy} = 21862.99 lb (for the anchor group)

Check anchors at c_{y1} edge

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 6.00 in

A_{vcy} = 243.00 in^2

A_{vcoy} = 162.00 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{by} = 5513.13 lb

V_{cbgy} = 9923.63 lb [Eq. D-22]

$V_{cbgx} = 2 * V_{cbgy}$ [Sec. D.6.2.1(c)]

V_{cbgx} = 19847.25 lb

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgx} = 11164.08 lb (for the anchor group)

Check anchors at c_{x2} edge

$V_{cbgx} = A_{vcx}/A_{vcox}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-22]

c_{a1} = 13.50 in

A_{vcx} = 713.81 in^2

A_{vcox} = 820.13 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Eq. D-27 or D-28] [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f'_c} (c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{bx} = 18606.80 lb

V_{cbgx} = 19433.77 lb [Eq. D-22]

$V_{cbgy} = 2 * V_{cbgx}$ [Sec. D.6.2.1(c)]

V_{cbgy} = 38867.54 lb

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgy} = 21862.99 lb (for the anchor group)

Check anchors at c_{y2} edge

$V_{cbgy} = A_{vcy}/A_{vcoy} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} V_{by}$ [Eq. D-22]

c_{a1} = 16.00 in (adjusted for edges per D.6.2.4)

A_{vcy} = 864.00 in^2

A_{vcoy} = 1152.00 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f'_c} (c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{by} = 24007.73 lb

V_{cbgy} = 21606.95 lb [Eq. D-22]

$V_{cbgx} = 2 * V_{cbgy}$ [Sec. D.6.2.1(c)]

V_{cbgx} = 43213.91 lb

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgx} = 24307.82 lb (for the anchor group)

10) Concrete Pryout Strength of Anchor Group in Shear [Sec. D.6.3]

$V_{cpg} = k_{cp}N_{cbg}$ [Eq. D-30]

k_{cp} = 2 [Sec. D.6.3.1]

e_{Nx} = 0.00 in (Applied shear load eccentricity relative to anchor group c.g.)

e_{Ny} = 1.81 in (Applied shear load eccentricity relative to anchor group c.g.)

$\Psi_{ec,Nx}$ = 1.0000 [Eq. D-9] (Calulated using applied shear load eccentricity)

$\Psi_{ec,Ny}$ = 0.9239 [Eq. D-9] (Calulated using applied shear load eccentricity)

$\Psi_{ec,N'}$ = 0.9239 (Combination of x-axis & y-axis eccentricity factors)

$N_{cbg} = (A_{Nca}/A_{Nc})(\Psi_{ec,N'}/\Psi_{ec,N})N_{cbg}$

N_{cbg} = 33688.52 lb (from Section (5) of calculations)

A_{Nc} = 1332.00 in^2 (from Section (5) of calculations)

A_{Nca} = 1332.00 in^2 (considering all anchors)

$\Psi_{ec,N}$ = 0.8908 (from Section(5) of calculations)

N_{cbg} = 34938.00 lb (considering all anchors)

V_{cpg} = 69876.00 lb

ϕ = 0.70 [D.4.4]

ϕ_{seis} = 0.75

ϕV_{cpg} = 36684.90 lb (for the anchor group)

11) Check Demand/Capacity Ratios [Sec. D.7]

Tension

- Steel : 0.9365
- Breakout : 0.9646
- Pullout : 0.4032
- Sideface Blowout : N/A

Breakout Ratio does not include additional resistance provided by (E) pad footing that will be epoxied to. This will provide additional breakout strength, therefore creating a ductile system

Shear

- Steel : 0.0707
- Breakout (case 1) : 0.0651
- Breakout (case 2) : 0.1053
- Breakout (case 3) : 0.0249
- Pryout : 0.0297

V.Max(0.11) <= 0.2 and T.Max(0.96) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 1/2" diameter PAB4 anchor(s) with 16 in. embedment

BRITTLE FAILURE GOVERNS: Governing anchor failure mode is brittle failure. Per 2006 IBC

Section 1908.1.16, anchors shall be governed by a ductile steel element in structures assigned to Seismic Design Category C, D, E, or F. Alternatively the minimum design strength of the anchor(s) shall be at least 2.5 times the factored forces or the anchor attachment to the structure shall undergo ductile yielding at a load level less than the design strength of the anchor(s). Designer must exercise own judgement to determine if this design is suitable.

Steel Base Plate Design

Description : Base Plate - Column at Intersection of Lines F.5 & 3

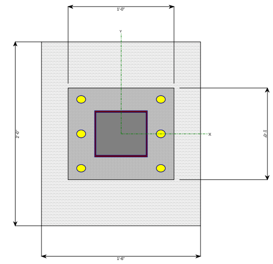
General Information

Calculations per 13th AISC & AISC Design Guide No. 1, 1990 by DeWolf & Ricker

Material Properties

AISC Design Method	Allowable Stress Design	ASIF : Allowable Stress Increase Factor	1.0
Steel Plate Fy	= 36.0 ksi	ABIF : Allowable Bearing Increase Factor	1.0
Concrete Support f'c	= 2.50 ksi	Ω_c : ASD Safety Factor.	2.50
Assumed Bearing Area :	Full Bearing	Allowable Bearing Fp per J8	3.188 ksi

Column & Plate



Column Properties

Steel Section : HSS6X6X3/16

Depth	6 in	Area	3.98 in^2
Width	6 in	Ixx	22.3 in^4
Flange Thickness	0.174 in	Iyy	22.3 in^4
Web Thickness	0 in		

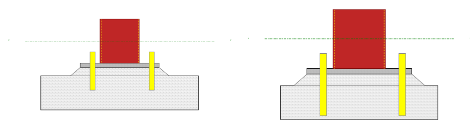
Plate Dimensions

N : Length	12.0 in
B : Width	12.0 in
Thickness	0.750 in

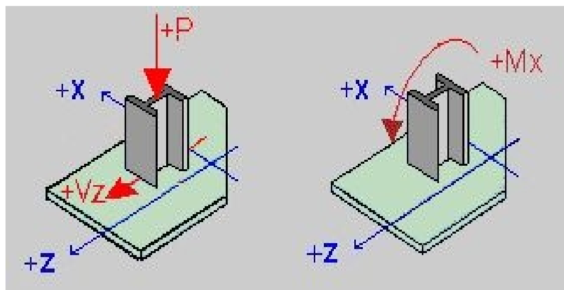
Column assumed welded to base plate.

Support Dimensions

Support width along "X"	24.0 in
Length along "Z'	18.0 in



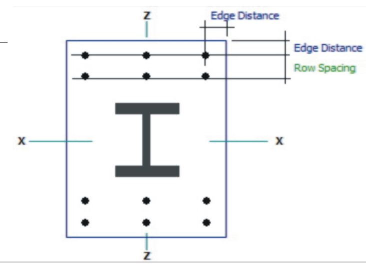
Applied Loads



	P-Y	V-Z	M-X
D : Dead Load	k	k	k-ft
L : Live	k	k	k-ft
Lr : Roof Live	k	k	k-ft
S : Snow	k	k	k-ft
W : Wind	k	k	k-ft
E : Earthquake	k	1.090 k	12.490 k-ft
H : Lateral Earth	k	k	k-ft

" P " = Gravity load, "+" sign is downward. "+" Moments create higher soil pressure at +Z edge.
"+" Shears push plate towards +Z edge.

Anchor Bolts



Anchor Bolt or Rod Description	Anchor Bolt
Max of Tension or Pullout Capacity...........	6.180 k
Shear Capacity..	2.570 k
Edge distance : bolt to plate..................	1.50 in
Number of Bolts in each Row..................	2.0
Number of Bolt Rows.......................	3.0
Row Spacing................................	4.50 in

GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method	Allowable Stress Design
Governing Load Combination	+D+0.70E+H
Governing Load Case Type	Axial + Moment, Eccentricity <= L/6
Design Plate Size	1'-0" x 1'-0" x 0 -3/4"
Pa : Axial Load	0.000 k
Ma : Moment	8.743 k-ft

fv : Actual	0.365 ksi
Fv : Allowable = 0.60 * Fy / 1.5 (per G2)	21.557 ksi
Stress Ratio	0.017

Shear Stress OK

Mu : Max. Moment	1.491 k-in
fb : Max. Bending Stress	15.905 ksi
Fb : Allowable : Fy * ASIF / Omega	21.557 ksi
Stress Ratio	0.738

Bending Stress OK

fu : Max. Plate Bearing Stress	0.364 ksi
Fp : Allowable : min(0.85*f'c*sqrt(A2/A1), 1.7* f'c)*Ome	1.275 ksi
Stress Ratio	0.286

Bearing Stress OK

Steel Base Plate Design

Description : Base Plate - Column at Intersection of Lines F.5 & 3

Load Comb. : +D+0.70E+H

Axial Load + Moment, Ecc. < L/6

Loading		Bearing Stresses	
Pa : Axial Load	0.000 k	Fp : Allowable	1.275 ksi
Ma : Moment	8.743 k-ft	fa : Max. Bearing Pressu	0.364 ksi
Eccentricity	0.000 in	Stress Ratio	0.286
A1 : Plate Area	144.000 in^2	**Plate Bending Stresses**	
A2 : Support Area	324.000 in^2	Mmax1 = Fu * m^2 / 2	1.491 k-in
e A2/A1	1.500	Mmax2 = Fu * n^2 / 2	0.623 k-in
		Mmax	1.491 k-in
Distance for Moment Calculation		fb : Actual	15.905 ksi
" m "	3.150 in	Fb : Allowable	21.557 ksi
" n "	3.600 in	Stress Ratio	0.738
		Shear Stress	
		fv : Actual	0.365 ksi
		Fv : Allowable	21.557 ksi
		Stress Ratio	0.017

Load Comb. : +D-0.70E+H

Axial Load + Moment, Ecc. < L/6

Loading		Bearing Stresses	
Pa : Axial Load	0.000 k	Fp : Allowable	1.275 ksi
Ma : Moment	8.743 k-ft	fa : Max. Bearing Pressu	0.364 ksi
Eccentricity	0.000 in	Stress Ratio	0.286
A1 : Plate Area	144.000 in^2	**Plate Bending Stresses**	
A2 : Support Area	324.000 in^2	Mmax1 = Fu * m^2 / 2	1.491 k-in
e A2/A1	1.500	Mmax2 = Fu * n^2 / 2	0.623 k-in
		Mmax	0.623 k-in
Distance for Moment Calculation		fb : Actual	6.645 ksi
" m "	3.150 in	Fb : Allowable	21.557 ksi
" n "	3.600 in	Stress Ratio	0.308
		Shear Stress	
		fv : Actual	0.365 ksi
		Fv : Allowable	21.557 ksi
		Stress Ratio	0.017

Load Comb. : +D+0.750Lr+0.750L+0.5250E+H

Axial Load + Moment, Ecc. < L/6

Loading		Bearing Stresses	
Pa : Axial Load	0.000 k	Fp : Allowable	1.275 ksi
Ma : Moment	6.557 k-ft	fa : Max. Bearing Pressu	0.273 ksi
Eccentricity	0.000 in	Stress Ratio	0.214
A1 : Plate Area	144.000 in^2	**Plate Bending Stresses**	
A2 : Support Area	324.000 in^2	Mmax1 = Fu * m^2 / 2	1.118 k-in
e A2/A1	1.500	Mmax2 = Fu * n^2 / 2	0.467 k-in
		Mmax	1.118 k-in
Distance for Moment Calculation		fb : Actual	11.928 ksi
" m "	3.150 in	Fb : Allowable	21.557 ksi
" n "	3.600 in	Stress Ratio	0.553
		Shear Stress	
		fv : Actual	0.274 ksi
		Fv : Allowable	21.557 ksi
		Stress Ratio	0.013

Steel Base Plate Design

Description : Base Plate - Column at Intersection of Lines F.5 & 3

Load Comb. : +D+0.750Lr+0.750L-0.5250E+H

Axial Load + Moment, Ecc. < L/6

Loading		Bearing Stresses	
Pa : Axial Load	0.000 k	Fp : Allowable	1.275 ksi
Ma : Moment	6.557 k-ft	fa : Max. Bearing Pressu	0.273 ksi
Eccentricity	0.000 in	Stress Ratio	0.214
A1 : Plate Area	144.000 in^2	Plate Bending Stresses	
A2 : Support Area	324.000 in^2	Mmax1 = Fu * m^2 / 2	1.118 k-in
e A2/A1	1.500	Mmax2 = Fu * n^2 / 2	0.467 k-in
		Mmax	0.467 k-in
Distance for Moment Calculation		fb : Actual	4.984 ksi
" m "	3.150 in	Fb : Allowable	21.557 ksi
" n "	3.600 in	Stress Ratio	0.231
		Shear Stress	
		fv : Actual	0.274 ksi
		Fv : Allowable	21.557 ksi
		Stress Ratio	0.013

Load Comb. : +D+0.750L+0.750S+0.5250E+H

Axial Load + Moment, Ecc. < L/6

Loading		Bearing Stresses	
Pa : Axial Load	0.000 k	Fp : Allowable	1.275 ksi
Ma : Moment	6.557 k-ft	fa : Max. Bearing Pressu	0.273 ksi
Eccentricity	0.000 in	Stress Ratio	0.214
A1 : Plate Area	144.000 in^2	Plate Bending Stresses	
A2 : Support Area	324.000 in^2	Mmax1 = Fu * m^2 / 2	1.118 k-in
e A2/A1	1.500	Mmax2 = Fu * n^2 / 2	0.467 k-in
		Mmax	1.118 k-in
Distance for Moment Calculation		fb : Actual	11.928 ksi
" m "	3.150 in	Fb : Allowable	21.557 ksi
" n "	3.600 in	Stress Ratio	0.553
		Shear Stress	
		fv : Actual	0.274 ksi
		Fv : Allowable	21.557 ksi
		Stress Ratio	0.013

Load Comb. : +D+0.750L+0.750S-0.5250E+H

Axial Load + Moment, Ecc. < L/6

Loading		Bearing Stresses	
Pa : Axial Load	0.000 k	Fp : Allowable	1.275 ksi
Ma : Moment	6.557 k-ft	fa : Max. Bearing Pressu	0.273 ksi
Eccentricity	0.000 in	Stress Ratio	0.214
A1 : Plate Area	144.000 in^2	Plate Bending Stresses	
A2 : Support Area	324.000 in^2	Mmax1 = Fu * m^2 / 2	1.118 k-in
e A2/A1	1.500	Mmax2 = Fu * n^2 / 2	0.467 k-in
		Mmax	0.467 k-in
Distance for Moment Calculation		fb : Actual	4.984 ksi
" m "	3.150 in	Fb : Allowable	21.557 ksi
" n "	3.600 in	Stress Ratio	0.231
		Shear Stress	
		fv : Actual	0.274 ksi
		Fv : Allowable	21.557 ksi
		Stress Ratio	0.013

Steel Base Plate Design

Description : Base Plate - Column at Intersection of Lines F.5 & 3

Load Comb. : +0.60D+0.70E+H

Axial Load + Moment, Ecc. < L/6

Loading		
Pa : Axial Load	0.000	k
Ma : Moment	8.743	k-ft
Eccentricity	0.000	in
A1 : Plate Area	144.000	in^2
A2 : Support Area	324.000	in^2
e A2/A1	1.500	
Distance for Moment Calculation		
" m "	3.150	in
" n "	3.600	in

Bearing Stresses		
Fp : Allowable	1.275	ksi
fa : Max. Bearing Pressu	0.364	ksi
Stress Ratio	0.286	
Plate Bending Stresses		
Mmax1 = Fu * m^2 / 2	1.491	k-in
Mmax2 = Fu * n^2 / 2	0.623	k-in
Mmax	1.491	k-in
fb : Actual	15.905	ksi
Fb : Allowable	21.557	ksi
Stress Ratio	0.738	
Shear Stress		
fv : Actual	0.365	ksi
Fv : Allowable	21.557	ksi
Stress Ratio	0.017	

Load Comb. : +0.60D-0.70E+H

Axial Load + Moment, Ecc. < L/6

Loading		
Pa : Axial Load	0.000	k
Ma : Moment	8.743	k-ft
Eccentricity	0.000	in
A1 : Plate Area	144.000	in^2
A2 : Support Area	324.000	in^2
e A2/A1	1.500	
Distance for Moment Calculation		
" m "	3.150	in
" n "	3.600	in

Bearing Stresses		
Fp : Allowable	1.275	ksi
fa : Max. Bearing Pressu	0.364	ksi
Stress Ratio	0.286	
Plate Bending Stresses		
Mmax1 = Fu * m^2 / 2	1.491	k-in
Mmax2 = Fu * n^2 / 2	0.623	k-in
Mmax	0.623	k-in
fb : Actual	6.645	ksi
Fb : Allowable	21.557	ksi
Stress Ratio	0.308	
Shear Stress		
fv : Actual	0.365	ksi
Fv : Allowable	21.557	ksi
Stress Ratio	0.017	

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.0)

Job Name : Borzall Expansion Date/Time : 11/5/2009 3:53:26 PM

1) Input

Calculation Method : ACI 318 Appendix D For Cracked Concrete

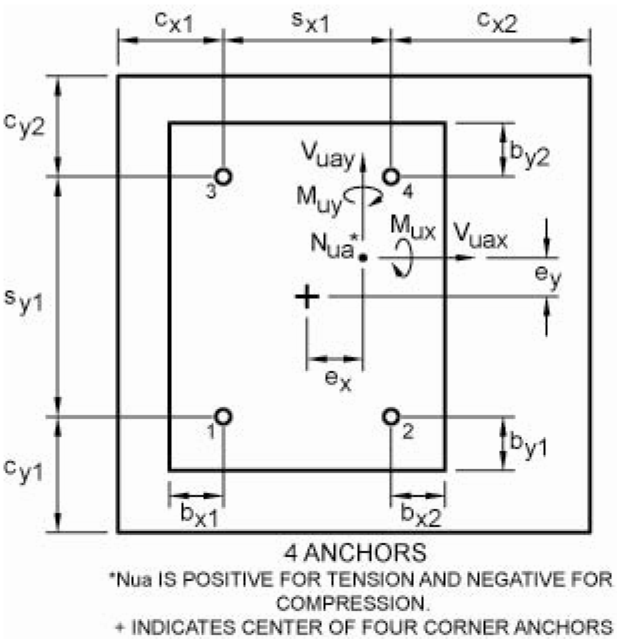
Calculation Type : Design

a) Layout

Anchor : 1/2" PAB4H Number of Anchors : 4

Steel Grade: AB_H Embedment Depth : 12 in

Built-up Grout Pads : Yes



Anchor Layout Dimensions :

c_{x1} : 19.5 in

c_{x2} : 19.5 in

c_{y1} : 19.5 in

c_{y2} : 19.5 in

b_{x1} : 1.5 in

b_{x2} : 1.5 in

b_{y1} : 1.5 in

b_{y2} : 1.5 in

s_{x1} : 9 in

s_{y1} : 4.5 in

WARNING: EXCESS BEARING PRESSURE!
Calculated bearing pressure is 7740.14 psi and exceeds the permissible bearing stress of ϕ Fp per ACI 318 Section 10.17. Designer must exercise own judgement to determine if this design is suitable.

b) Base Material

Concrete : Normal weight — f'_c : 2500.0 psi

Cracked Concrete : Yes — $\Psi_{c,V}$: 1.20

Condition : A tension and shear — ϕF_p : 1381.3 psi

Thickness, h : 24 in

Supplementary edge reinforcement : No

c) Factored Loads

Load factor source : ACI 318 Section 9.2

N_{ua} : 0 lb — V_{uax} : 0 lb

V_{uay} : -1090 lb — M_{ux} : -12490 lb*ft

M_{uy} : 0 lb*ft

e_x : 0 in

e_y : 0 in

Moderate/high seismic risk or intermediate/high design category : Yes

Apply entire shear load at front row for breakout : No

d) Anchor Parameters

Anchor Model = PAB4H	d_o = 0.5 in
Category = N/A	h_{ef} = 11 in
h_{min} = 12.75 in	c_{ac} = 16.5 in
c_{min} = 3 in	s_{min} = 3 in
Ductile = Yes	

2) Tension Force on Each Individual Anchor

Anchor #1 N_{ua1} = 12767.49 lb

Anchor #2 N_{ua2} = 12767.49 lb

Anchor #3 N_{ua3} = 2050.51 lb

Anchor #4 N_{ua4} = 2050.51 lb

Sum of Anchor Tension ΣN_{ua} = 29636.00 lb

a_x = 0.00 in

a_y = 6.86 in

e'_{Nx} = 0.00 in

e'_{Ny} = 1.63 in

3) Shear Force on Each Individual Anchor

Resultant shear forces in each anchor:

Anchor #1 V_{ua1} = 272.50 lb (V_{ua1x} = 0.00 lb , V_{ua1y} = -272.50 lb)

Anchor #2 V_{ua2} = 272.50 lb (V_{ua2x} = 0.00 lb , V_{ua2y} = -272.50 lb)

Anchor #3 V_{ua3} = 272.50 lb (V_{ua3x} = 0.00 lb , V_{ua3y} = -272.50 lb)

Anchor #4 V_{ua4} = 272.50 lb (V_{ua4x} = 0.00 lb , V_{ua4y} = -272.50 lb)

Sum of Anchor Shear ΣV_{uax} = 0.00 lb, ΣV_{uay} = -1090.00 lb

e'_{Vx} = 0.00 in

e'_{Vy} = 0.00 in

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

$N_{sa} = nA_{se}f_{uta}$ [Eq. D-3]

Number of anchors acting in tension, n = 4

N_{sa} = 17040 lb (for each individual anchor)

ϕ = 0.75 [D.4.4]

ϕN_{sa} = 12780.00 lb (for each individual anchor)

5) Concrete Breakout Strength of Anchor Group in Tension [Sec. D.5.2]

$N_{cbg} = A_{Nc}/A_{Nco}\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ [Eq. D-5]

Number of influencing edges = 0

h_{ef} = 11 in

h'_{ef} = 11.458 in [Sec. D.5.2.8]

Note: A_{Nco} and A_{Nc} are calculated using h'_{ef} for anchor with washer

A_{Nco} = 1181.64 in^2 [Eq. D-6]

A_{Nc} = 1686.20 in^2

$\Psi_{ec,Nx}$ = 1.0000 [Eq. D-9]

$\Psi_{ec,Ny}$ = 0.9102 [Eq. D-9]

$\Psi_{ec,N}$ = 0.9102 (Combination of x-axis & y-axis eccentricity factors.)

$\Psi_{ed,N}$ = 1.0000 [Eq. D-10 or D-11]

Note: Cracking shall be controlled per D.5.2.6

$\Psi_{c,N}$ = 1.0000 [Sec. D.5.2.6]

$\Psi_{cp,N}$ = 1.0000 [Eq. D-12 or D-13]

$N_b = 16\sqrt{f'_c}\, h_{ef}^{5/3}$ = 43525.57 lb [Eq. D-8]

N_{cbg} = 56535.34 lb [Eq. D-5]

ϕ = 0.75 [D.4.4]

ϕ_{seis} = 0.75

ϕN_{cbg} = 31801.13 lb (for the anchor group)

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

$N_p = 8A_{brg}f'_c$ [Eq. D-15]

A_{brg} = 1.3662 in^2

$N_{pn} = \Psi_{c,p}N_p$ [Eq. D-14]

$\Psi_{c,p}$ = 1.0 [D.5.3.6]

N_{pn} = 27324.00 lb

ϕ = 0.70 [D.4.4]

ϕ_{seis} = 0.75

$\phi N_{pn} = \phi N_{eq}$ = 14345.10 lb (for each individual anchor)

7) Side Face Blowout of Anchor in Tension [Sec. D.5.4]

Concrete side face blowout strength is only calculated for headed anchors in tension close to an edge, $c_{a1} < 0.4h_{ef}$. Not applicable in this case.

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

V_{eq} = 10225.00 lb (for each individual anchor)

ϕ = 0.65 [D.4.4]

ϕV_{eq} = 6646.25 lb (for each individual anchor)

ϕV_{eq} is multiplied by 0.8 due to built-up grout pads...[Sec D.6.1.3]

ϕV_{eq} = 5317.00lb (for each individual anchor)

9) Concrete Breakout Strength of Anchor Group in Shear [Sec D.6.2]

Case 1: Anchor(s) closest to edge checked against sum of anchor shear loads at the edge
In x-direction...

$V_{cbgx} = A_{vcx}/A_{vcox}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-22]

c_{a1} = 16.00 in (adjusted for edges per D.6.2.4)

A_{vcx} = 1044.00 in^2

A_{vcox} = 1152.00 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 0.9438 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{bx} = 24007.73 lb

V_{cbgx} = 24639.80 lb [Eq. D-22]

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgx} = 13859.89 lb (for the anchor group)

In y-direction...

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 16.00 in (adjusted for edges per D.6.2.4)

A_{vcy} = 1152.00 in^2

A_{vcoy} = 1152.00 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 0.9438 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{by} = 24007.73 lb

V_{cbgy} = 27188.75 lb [Eq. D-22]

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgy} = 15293.67 lb (for the anchor group)

Case 2: Anchor(s) furthest from edge checked against total shear load

In x-direction...

$V_{cbgx} = A_{vcx}/A_{vcox}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-22]

c_{a1} = 16.00 in (adjusted for edges per D.6.2.4)

A_{vcx} = 1044.00 in^2

A_{vcox} = 1152.00 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 0.9438 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{bx} = 24007.73 lb

V_{cbgx} = 24639.80 lb [Eq. D-22]

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgx} = 13859.89 lb (for the entire anchor group)

In y-direction...

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 16.00 in (adjusted for edges per D.6.2.4)

A_{vcy} = 1152.00 in^2

A_{vcoy} = 1152.00 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 0.9438 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{by} = 24007.73 lb

V_{cbgy} = 27188.75 lb [Eq. D-22]

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgy} = 15293.67 lb (for the entire anchor group)

Case 3: Anchor(s) closest to edge checked for parallel to edge condition
Check anchors at c_{x1} edge

$V_{cbgx} = A_{vcx}/A_{vcox}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-22]

c_{a1} = 16.00 in (adjusted for edges per D.6.2.4)

A_{vcx} = 1044.00 in^2

A_{vcox} = 1152.00 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{bx} = 24007.73 lb

V_{cbgx} = 26108.40 lb [Eq. D-22]

$V_{cbgy} = 2 * V_{cbgx}$ [Sec. D.6.2.1(c)]

V_{cbgy} = 52216.80 lb

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgy} = 29371.95 lb (for the anchor group)

Check anchors at c_{y1} edge

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 16.00 in (adjusted for edges per D.6.2.4)

A_{vcy} = 1152.00 in^2

A_{vcoy} = 1152.00 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{by} = 24007.73 lb

V_{cbgy} = 28809.27 lb [Eq. D-22]

$V_{cbgx} = 2 * V_{cbgy}$ [Sec. D.6.2.1(c)]

V_{cbgx} = 57618.54 lb

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgx} = 32410.43 lb (for the anchor group)

Check anchors at c_{x2} edge

$V_{cbgx} = A_{vcx}/A_{vcox}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-22]

c_{a1} = 16.00 in (adjusted for edges per D.6.2.4)

A_{vcx} = 1044.00 in^2

A_{vcox} = 1152.00 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Eq. D-27 or D-28] [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{bx} = 24007.73 lb

V_{cbgx} = 26108.40 lb [Eq. D-22]

$V_{cbgy} = 2 * V_{cbgx}$ [Sec. D.6.2.1(c)]

V_{cbgy} = 52216.80 lb

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgy} = 29371.95 lb (for the anchor group)

Check anchors at c_{y2} edge

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 16.00 in (adjusted for edges per D.6.2.4)

A_{vcy} = 1152.00 in^2

A_{vcoy} = 1152.00 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.2000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 4.00 in

V_{by} = 24007.73 lb

V_{cbgy} = 28809.27 lb [Eq. D-22]

$V_{cbgx} = 2 * V_{cbgy}$ [Sec. D.6.2.1(c)]

V_{cbgx} = 57618.54 lb

ϕ = 0.75

ϕ_{seis} = 0.75

ϕV_{cbgx} = 32410.43 lb (for the anchor group)

10) Concrete Pryout Strength of Anchor Group in Shear [Sec. D.6.3]

$V_{cpg} = k_{cp}N_{cbg}$ [Eq. D-30]

$k_{cp} = 2$ [Sec. D.6.3.1]

$e_{Nx} = 0.00$ in (Applied shear load eccentricity relative to anchor group c.g.)

$e_{Ny} = 0.00$ in (Applied shear load eccentricity relative to anchor group c.g.)

$\Psi_{ec,Nx} = 1.0000$ [Eq. D-9] (Calulated using applied shear load eccentricity)

$\Psi_{ec,Ny} = 1.0000$ [Eq. D-9] (Calulated using applied shear load eccentricity)

$\Psi_{ec,N'} = 1.0000$ (Combination of x-axis & y-axis eccentricity factors)

$N_{cbg} = (A_{Nca}/A_{Nc})(\Psi_{ec,N'}/\Psi_{ec,N})N_{cbg}$

$N_{cbg} = 56535.34$ lb (from Section (5) of calculations)

$A_{Nc} = 1686.20$ in^2 (from Section (5) of calculations)

$A_{Nca} = 1686.20$ in^2 (considering all anchors)

$\Psi_{ec,N} = 0.9102$ (from Section(5) of calculations)

$N_{cbg} = 62111.06$ lb (considering all anchors)

$V_{cpg} = 124222.13$ lb

$\phi = 0.70$ [D.4.4]

$\phi_{seis} = 0.75$

$\phi V_{cpg} = 65216.62$ lb (for the anchor group)

11) Check Demand/Capacity Ratios [Sec. D.7]

Tension

- Steel : 0.9990
- Breakout : 0.9319
- Pullout : 0.8900
- Sideface Blowout : N/A

Shear

- Steel : 0.0513
- Breakout (case 1) : 0.0356
- Breakout (case 2) : 0.0713
- Breakout (case 3) : 0.0186
- Pryout : 0.0167

V.Max(0.07) <= 0.2 and T.Max(1) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 1/2" diameter PAB4H anchor(s) with 12 in. embedment

BRITTLE FAILURE GOVERNS: Governing anchor failure mode is brittle failure. Per 2006 IBC Section 1908.1.16, anchors shall be governed by a ductile steel element in structures assigned to Seismic Design Category C, D, E, or F. Alternatively the minimum design strength of the anchor(s) shall be at least 2.5 times the factored forces or the anchor attachment to the structure shall undergo ductile yielding at a load level less than the design strength of the anchor(s). Designer must exercise own judgement to determine if this design is suitable.

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using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Printed: 9 NOV 2009, 9:21AM

Steel Base Plate Design

Y:\@Shawn Pierce Engineering\09_Projects\Borzall Addition_02\Engineering\Design\melford borzall expansion.ec6
ENERCALC, INC. 1983-2008, Ver: 6.0.221, N:40336

Lic. # : KW-06008640 **License Owner : PIERCE ENGINEERING**

Description : Base Plate - Column at Intersection of Lines J.5 & 3

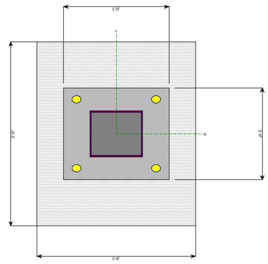
General Information

Calculations per 13th AISC & AISC Design Guide No. 1, 1990 by DeWolf & Ricker

Material Properties

AISC Design Method	Allowable Stress Design	ASIF : Allowable Stress Increase Factor	1.0
Steel Plate Fy	= 36.0 ksi	ABIF : Allowable Bearing Increase Factor	1.0
Concrete Support f'c	= 2.50 ksi	Ω_c : ASD Safety Factor.	2.50
Assumed Bearing Area :	Full Bearing	Allowable Bearing Fp per J8	3.188 ksi

Column & Plate



Column Properties

Steel Section : HSS6X6X3/16

Depth	6 in	Area	3.98 in^2
Width	6 in	Ixx	22.3 in^4
Flange Thickness	0.174 in	Iyy	22.3 in^4
Web Thickness	in		

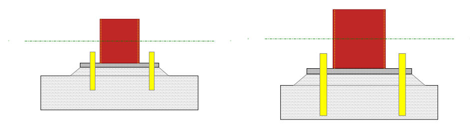
Plate Dimensions

N : Length	12.0 in
B : Width	12.0 in
Thickness	0.750 in

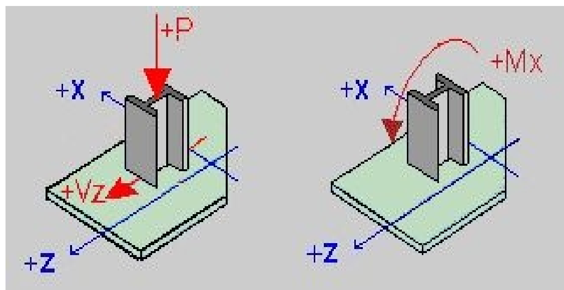
Column assumed welded to base plate.

Support Dimensions

Support width along "X"	24.0 in
Length along "Z'	18.0 in



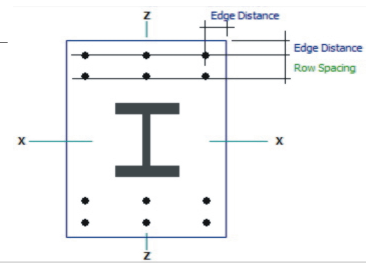
Applied Loads



	P-Y	V-Z	M-X
D : Dead Load	k	k	k-ft
L : Live	k	k	k-ft
Lr : Roof Live	k	k	k-ft
S : Snow	k	k	k-ft
W : Wind	k	k	k-ft
E : Earthquake	k	1.090 k	12.490 k-ft
H : Lateral Earth	k	k	k-ft

" P " = Gravity load, "+" sign is downward.
"+" Moments create higher soil pressure at +Z edge.
"+" Shears push plate towards +Z edge.

Anchor Bolts



Anchor Bolt or Rod Description	Anchor Bolt
Max of Tension or Pullout Capacity...........	6.180 k
Shear Capacity...	2.570 k
Edge distance : bolt to plate...................	1.50 in
Number of Bolts in each Row...................	2.0
Number of Bolt Rows........................	2.0
Row Spacing.................................	9.0 in

GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method	Allowable Stress Design
Governing Load Combination	+D+0.70E+H
Governing Load Case Type	Axial + Moment, Eccentricity <= L/6
Design Plate Size	1'-0" x 1'-0" x 0 -3/4"
Pa : Axial Load	0.000 k
Ma : Moment	8.743 k-ft

fv : Actual .... 0.365 ksi
Fv : Allowable = 0.60 * Fy / 1.5 (per G2) 21.557 ksi
Stress Ratio 0.017
Shear Stress OK

Mu : Max. Moment .... 1.491 k-in
fb : Max. Bending Stress .... 15.905 ksi
Fb : Allowable : 21.557 ksi
Fy * ASIF / Omega
Stress Ratio 0.738
Bending Stress OK

fu : Max. Plate Bearing Stress .... 0.364 ksi
Fp : Allowable : 1.275 ksi
min(0.85*f'c*sqrt(A2/A1), 1.7* f'c)*Ome
Stress Ratio 0.286
Bearing Stress OK

Steel Base Plate Design

Description : Base Plate - Column at Intersection of Lines J.5 & 3

Load Comb. : +D+0.70E+H

Axial Load + Moment, Ecc. < L/6

Loading		Bearing Stresses	
Pa : Axial Load	0.000 k	Fp : Allowable	1.275 ksi
Ma : Moment	8.743 k-ft	fa : Max. Bearing Pressu	0.364 ksi
Eccentricity	0.000 in	Stress Ratio	0.286
A1 : Plate Area	144.000 in^2	**Plate Bending Stresses**	
A2 : Support Area	324.000 in^2	Mmax1 = Fu * m^2 / 2	1.491 k-in
e A2/A1	1.500	Mmax2 = Fu * n^2 / 2	0.623 k-in
Distance for Moment Calculation		Mmax	1.491 k-in
		fb : Actual	15.905 ksi
" m "	3.150 in	Fb : Allowable	21.557 ksi
" n "	3.600 in	Stress Ratio	0.738
		Shear Stress	
		fv : Actual	0.365 ksi
		Fv : Allowable	21.557 ksi
		Stress Ratio	0.017

Load Comb. : +D-0.70E+H

Axial Load + Moment, Ecc. < L/6

Loading		Bearing Stresses	
Pa : Axial Load	0.000 k	Fp : Allowable	1.275 ksi
Ma : Moment	8.743 k-ft	fa : Max. Bearing Pressu	0.364 ksi
Eccentricity	0.000 in	Stress Ratio	0.286
A1 : Plate Area	144.000 in^2	**Plate Bending Stresses**	
A2 : Support Area	324.000 in^2	Mmax1 = Fu * m^2 / 2	1.491 k-in
e A2/A1	1.500	Mmax2 = Fu * n^2 / 2	0.623 k-in
Distance for Moment Calculation		Mmax	0.623 k-in
		fb : Actual	6.645 ksi
" m "	3.150 in	Fb : Allowable	21.557 ksi
" n "	3.600 in	Stress Ratio	0.308
		Shear Stress	
		fv : Actual	0.365 ksi
		Fv : Allowable	21.557 ksi
		Stress Ratio	0.017

Load Comb. : +D+0.750Lr+0.750L+0.5250E+H

Axial Load + Moment, Ecc. < L/6

Loading		Bearing Stresses	
Pa : Axial Load	0.000 k	Fp : Allowable	1.275 ksi
Ma : Moment	6.557 k-ft	fa : Max. Bearing Pressu	0.273 ksi
Eccentricity	0.000 in	Stress Ratio	0.214
A1 : Plate Area	144.000 in^2	**Plate Bending Stresses**	
A2 : Support Area	324.000 in^2	Mmax1 = Fu * m^2 / 2	1.118 k-in
e A2/A1	1.500	Mmax2 = Fu * n^2 / 2	0.467 k-in
Distance for Moment Calculation		Mmax	1.118 k-in
		fb : Actual	11.928 ksi
" m "	3.150 in	Fb : Allowable	21.557 ksi
" n "	3.600 in	Stress Ratio	0.553
		Shear Stress	
		fv : Actual	0.274 ksi
		Fv : Allowable	21.557 ksi
		Stress Ratio	0.013

Steel Base Plate Design

Description : Base Plate - Column at Intersection of Lines J.5 & 3

Load Comb. : +D+0.750Lr+0.750L-0.5250E+H

Axial Load + Moment, Ecc. < L/6

Loading		Bearing Stresses	
Pa : Axial Load	0.000 k	Fp : Allowable	1.275 ksi
Ma : Moment	6.557 k-ft	fa : Max. Bearing Pressu	0.273 ksi
Eccentricity	0.000 in	Stress Ratio	0.214
A1 : Plate Area	144.000 in^2	**Plate Bending Stresses**	
A2 : Support Area	324.000 in^2	Mmax1 = Fu * m^2 / 2	1.118 k-in
e A2/A1	1.500	Mmax2 = Fu * n^2 / 2	0.467 k-in
Distance for Moment Calculation		Mmax	0.467 k-in
" m "	3.150 in	fb : Actual	4.984 ksi
" n "	3.600 in	Fb : Allowable	21.557 ksi
		Stress Ratio	0.231
		Shear Stress	
		fv : Actual	0.274 ksi
		Fv : Allowable	21.557 ksi
		Stress Ratio	0.013

Load Comb. : +D+0.750L+0.750S+0.5250E+H

Axial Load + Moment, Ecc. < L/6

Loading		Bearing Stresses	
Pa : Axial Load	0.000 k	Fp : Allowable	1.275 ksi
Ma : Moment	6.557 k-ft	fa : Max. Bearing Pressu	0.273 ksi
Eccentricity	0.000 in	Stress Ratio	0.214
A1 : Plate Area	144.000 in^2	**Plate Bending Stresses**	
A2 : Support Area	324.000 in^2	Mmax1 = Fu * m^2 / 2	1.118 k-in
e A2/A1	1.500	Mmax2 = Fu * n^2 / 2	0.467 k-in
Distance for Moment Calculation		Mmax	1.118 k-in
" m "	3.150 in	fb : Actual	11.928 ksi
" n "	3.600 in	Fb : Allowable	21.557 ksi
		Stress Ratio	0.553
		Shear Stress	
		fv : Actual	0.274 ksi
		Fv : Allowable	21.557 ksi
		Stress Ratio	0.013

Load Comb. : +D+0.750L+0.750S-0.5250E+H

Axial Load + Moment, Ecc. < L/6

Loading		Bearing Stresses	
Pa : Axial Load	0.000 k	Fp : Allowable	1.275 ksi
Ma : Moment	6.557 k-ft	fa : Max. Bearing Pressu	0.273 ksi
Eccentricity	0.000 in	Stress Ratio	0.214
A1 : Plate Area	144.000 in^2	**Plate Bending Stresses**	
A2 : Support Area	324.000 in^2	Mmax1 = Fu * m^2 / 2	1.118 k-in
e A2/A1	1.500	Mmax2 = Fu * n^2 / 2	0.467 k-in
Distance for Moment Calculation		Mmax	0.467 k-in
" m "	3.150 in	fb : Actual	4.984 ksi
" n "	3.600 in	Fb : Allowable	21.557 ksi
		Stress Ratio	0.231
		Shear Stress	
		fv : Actual	0.274 ksi
		Fv : Allowable	21.557 ksi
		Stress Ratio	0.013

Steel Base Plate Design

Description : Base Plate - Column at Intersection of Lines J.5 & 3

Load Comb. : +0.60D+0.70E+H

Axial Load + Moment, Ecc. < L/6

Loading

Pa : Axial Load	0.000	k
Ma : Moment	8.743	k-ft
Eccentricity	0.000	in
A1 : Plate Area	144.000	in^2
A2 : Support Area	324.000	in^2
e A2/A1	1.500	

Distance for Moment Calculation

" m "	3.150	in
" n "	3.600	in

Bearing Stresses

Fp : Allowable	1.275	ksi
fa : Max. Bearing Pressu	0.364	ksi
Stress Ratio	0.286	

Plate Bending Stresses

Mmax1 = Fu * m^2 / 2	1.491	k-in
Mmax2 = Fu * n^2 / 2	0.623	k-in
Mmax	1.491	k-in
fb : Actual	15.905	ksi
Fb : Allowable	21.557	ksi
Stress Ratio	0.738	

Shear Stress

fv : Actual	0.365	ksi
Fv : Allowable	21.557	ksi
Stress Ratio	0.017	

Load Comb. : +0.60D-0.70E+H

Axial Load + Moment, Ecc. < L/6

Loading

Pa : Axial Load	0.000	k
Ma : Moment	8.743	k-ft
Eccentricity	0.000	in
A1 : Plate Area	144.000	in^2
A2 : Support Area	324.000	in^2
e A2/A1	1.500	

Distance for Moment Calculation

" m "	3.150	in
" n "	3.600	in

Bearing Stresses

Fp : Allowable	1.275	ksi
fa : Max. Bearing Pressu	0.364	ksi
Stress Ratio	0.286	

Plate Bending Stresses

Mmax1 = Fu * m^2 / 2	1.491	k-in
Mmax2 = Fu * n^2 / 2	0.623	k-in
Mmax	0.623	k-in
fb : Actual	6.645	ksi
Fb : Allowable	21.557	ksi
Stress Ratio	0.308	

Shear Stress

fv : Actual	0.365	ksi
Fv : Allowable	21.557	ksi
Stress Ratio	0.017	

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB: BORZALE EXPANSION 09.9-2/2
SHEET NO.: OF:
CALCULATED BY: DATE: 11/09
CHECKED BY: DATE:
SCALE: CONFERENCE ROOM EXTENSION

✓ SEISMIC RESTRAINT AT LINE (3) (CONT.)

DUE TO SEISMIC LOAD & POTENTIAL P-DELTA EFFECTS THE FOLLOWING COLUMN SHALL BE USED:

HSS 6×6×3/16 A-500 GRADE B

SEE ENERCALC RESULTS

✓ FOOTING REQUIREMENTS

LINE (3) AT (F.5) - DUE TO PROXIMITY OF (N) COLUMN TO (E) PAD FOOTING, THE FOLLOWING DESIGN WILL REVIEW HOW THE (N) PAD FOOTING SHOULD BE CONNECTED TO THE (E) FOOTING TO HELP DISTRIBUTE THE LATERAL MOMENT TO THE SOIL.

$P_D = 2920^{\#}$
$P_L = 7035^{\#}$
$M_{MAX} = 8900^{\#'}$

TO TRANSFER MOMENT TO (E) FTG

$T = M_{MAX} / (d_{(E)FTG} - 3\frac{1}{2}'') = 5210^{\#}$ TENSION FACE

CAPACITY OF #5 A615 GR 60 BAR

$A_s f_y = 0.31\,in^2 (60{,}000\,psi) = 18.6k/BAR$

CAPACITY OF BAR $= 18.6/5.2 = 3.6 \times$ FACTORED FORCE

THEREFORE BRITTLE FAILURE DOES NOT NEED TO BE MET.

USE: (2) #5 A615 GR 60 BARS EPOXIED INTO FACE OF (E) PAD FTG W/ SIMPSON 'SET-XP', EMBED = 9"

SEE 'SIMPSON ANCHOR DESIGNER FOR ACI318' RESULTS

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.0)

Job Name : Tension Rebar　　　　Date/Time : 11/7/2009 6:59:19 PM

1) Input

Calculation Method : ACI 318 Appendix D For Cracked Concrete

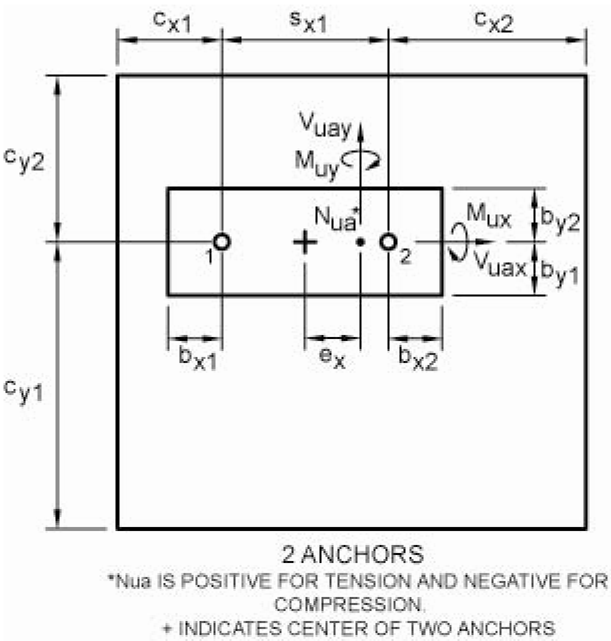
Calculation Type : Analysis

a) Layout

Anchor : 5/8" SET-XP　　　　Number of Anchors : 2

Steel Grade: A615 GR. 60　　　　Embedment Depth : 9 in

Built-up Grout Pads : No



Anchor Layout Dimensions :

c_{x1} : 6.75 in

c_{x2} : 26 in

c_{y1} : 3.5 in

c_{y2} : 20.5 in

s_{x1} : 9.75 in

b) Base Material

Concrete : Normal weight　　　　f'_c : 2500.0 psi

Cracked Concrete : Yes　　　　$\Psi_{c,V}$: 1.00

Condition : B tension and shear　　　　ϕF_p : 1381.3

psi

Thickness, h : 48 in

Supplementary edge reinforcement : No

Hole Condition : Dry Concrete

Inspection : Continuous

Temperature Range : 1 (Maximum 110 ^{o}F short term and 75 ^{o}F long term temp.)

c) Factored Loads

Load factor source : ACI 318 Section 9.2

N_{ua} : 5210 lb

V_{uax} : 0 lb

V_{uay} : 0 lb

M_{ux} : 0 lb*ft

M_{uy} : 0 lb*ft

e_x : 0 in

e_y : 0 in

Moderate/high seismic risk or intermediate/high design category : Yes

Anchor w/ sustained tension : No

Anchors only resist wind and/or seismic loads : Yes

Apply entire shear load at front row for breakout : No

d) Anchor Parameters

From C-SAS-2009:

Anchor Model = SETXP	d_o = 0.625 in
Category = 1	h_{ef} = 9 in
h_{min} = 12.125 in	c_{ac} = 27 in
c_{min} = 1.75 in	s_{min} = 3 in

Ductile = Yes

2) Tension Force on Each Individual Anchor

Anchor #1 N_{ua1} = 2605.00 lb

Anchor #2 N_{ua2} = 2605.00 lb

Sum of Anchor Tension ΣN_{ua} = 5210.00 lb

e'_{Nx} = 0.00 in

e'_{Ny} = 0.00 in

3) Shear Force on Each Individual Anchor

Resultant shear forces in each anchor:

Anchor #1 V_{ua1} = 0.00 lb (V_{ua1x} = 0.00 lb , V_{ua1y} = 0.00 lb)

Anchor #2 V_{ua2} = 0.00 lb (V_{ua2x} = 0.00 lb , V_{ua2y} = 0.00 lb)

Sum of Anchor Shear ΣV_{uax} = 0.00 lb, ΣV_{uay} = 0.00 lb

e'_{Vx} = 0.00 in

e'_{Vy} = 0.00 in

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

$N_{sa} = nA_{se}f_{uta}$ [Eq. D-3]

Number of anchors acting in tension, n = 2

N_{sa} = 27900 lb (for each individual anchor) [C-SAS-2009]

ϕ = 0.75 [D.4.4]

ϕN_{sa} = 20925.00 lb (for each individual anchor)

5) Concrete Breakout Strength of Anchor Group in Tension [Sec. D.5.2]

$N_{cbg} = A_{Nc}/A_{Nco}\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ [Eq. D-5]

Number of influencing edges = 2

h_{ef} = 9 in

A_{Nco} = 729.00 in^2 [Eq. D-6]

A_{Nc} = 510.00 in^2

$\Psi_{ec,Nx}$ = 1.0000 [Eq. D-9]

$\Psi_{ec,Ny}$ = 1.0000 [Eq. D-9]

$\Psi_{ec,N}$ = 1.0000 (Combination of x-axis & y-axis eccentricity factors.)

Smallest edge distance, $c_{a,min}$ = 3.50 in

$\Psi_{ed,N}$ = 0.7778 [Eq. D-10 or D-11]

Note: Cracking shall be controlled per D.5.2.6

$\Psi_{c,N}$ = 1.0000 [Sec. D.5.2.6]

$\Psi_{cp,N}$ = 1.0000 [Eq. D-12 or D-13]

$N_b = k_c\sqrt{f'_c}\, h_{ef}^{1.5}$ = 22950.00 lb [Eq. D-7]

k_c = 17 [Sec. D.5.2.6]

N_{cbg} = 12487.65 lb [Eq. D-5]

ϕ = 0.65 [D.4.4]

ϕ_{seis} = 0.75

ϕN_{cbg} = 6087.73 lb (for the anchor group)

6) Adhesive Strength of Anchor in Tension [Sec. D.5.3 (AC308 Sec.3.3)]

$\tau_{k,cr}$ = 718 psi [C-SAS-2009]

$\tau_{k,max,cr} = k_{cr}\sqrt{h_{ef}}\sqrt{f'_c}/(\pi\ d_o)$ [Eq. D-16i]
$k_{cr} = 17$ [C-SAS-2009]
h_{ef} (unadjusted) = 9 in
$\tau_{k,max,cr} = 1298.70$ psi
$N_{ao} = \tau_{k,cr}\pi\ d_o h_{ef} = 12688.11$ lb [Eq. D-16f]
$\tau_{k,uncr} = 2263.00$ psi for use in [Eq. D-16d]
$s_{cr,Na} = \min[20d_o\sqrt{(\tau_{k,uncr}/1450)}\ , 3h_{ef}] = 15.616$ in [Eq. D-16d]
$c_{cr,Na} = s_{cr,Na}/2 = 7.808$ in [Eq. D-16e]
$N_{ag} = A_{Na}/A_{Nao}\Psi_{ed,Na}\Psi_{g,Na}\Psi_{ec,Na}\Psi_{p,Na}N_{ao}$ [Eq. D-16b]
$A_{Nao} = 243.86\ in^2$ [Eq. D-16c]
$A_{Na} = 274.87\ in^2$
$\Psi_{ec,Nax} = 1/(1+2e'_{Nx}/s_{cr,Na}) = 1.0000$ [Eq. D-16j]
$\Psi_{ec,Nay} = 1/(1+2e'_{Ny}/s_{cr,Na}) = 1.0000$ [Eq. D-16j]
$\Psi_{ec,Na} = 1.0000$ (Combination of x-axis and y-axis eccentricity factors.)
Smallest edge distance, $c_{a,min} = 3.50$ in
$\Psi_{ed,Na} = \min[0.7+0.3c_{a,min}/c_{cr,Na}\ , 1.0] = 0.8345$ [Eq. D-16m]
$\Psi_{p,Na} = 1.0000$ [Sec. D.5.3.14]
$\Psi_{g,Na} = \Psi_{g,Nao}+(s/s_{cr,Na})^{0.5}(1-\Psi_{g,Nao})$ [Eq. D-16g]
s = 10 in (largest spacing)
$\Psi_{g,Nao} = \max\{\sqrt{n} - [(\sqrt{n} - 1)(\tau_{k,cr}/\tau_{k,max,cr})^{1.5}], 1.0\}$ [Eq. D-16h]
$\Psi_{g,Nao} = 1.2439$
$\Psi_{g,Na} = 1.0512$
$N_{ag} = 12545.52$ lb [Eq. D-16b]
$\phi = 0.65$ [C-SAS-2009]
$\phi_{seis} = 0.75$
$\phi N_{ag} = 6115.94$ lb (for the anchor group)

7) Side Face Blowout of Anchor in Tension [Sec. D.5.4]

Concrete side face blowout strength is only calculated for headed anchors in tension close to an edge, $c_{a1} < 0.4h_{ef}$. Not applicable in this case.

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

$V_{sa} = 16740.00$ lb (for each individual anchor)

$V_{eq} = V_{sa}\alpha_{v.seis}$ [AC308 Eq. 11-27]

$\alpha_{v.seis}$ = 0.80 [C-SAS-2009]

V_{eq} = 13392.00 lb

ϕ = 0.65 [D.4.4]

$\phi\ V_{eq}$ = 8704.80 lb (for each individual anchor)

9) Concrete Breakout Strength of Anchor Group in Shear [Sec D.6.2]

Case 1: Anchor(s) closest to edge checked against sum of anchor shear loads at the edge
In x-direction...

$V_{cbx} = A_{vcx}/A_{vcox}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-21]

c_{a1} = 26.00 in

A_{vcx} = 936.00 in^2

A_{vcox} = 3042.00 in^2 [Eq. D-23]

$\Psi_{ed,V}$ = 0.7269 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{\ d_o}\sqrt{\ f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{bx} = 55601.44 lb

V_{cbx} = 12436.30 lb [Eq. D-22]

ϕ = 0.70

ϕ_{seis} = 0.75

ϕV_{cbx} = 6529.06 lb (for a single anchor)

In y-direction...

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 20.50 in

A_{vcy} = 1306.88 in^2

A_{vcoy} = 1891.13 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 0.7659 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{\ d_o}\sqrt{\ f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{by} = 38927.53 lb

V_{cbgy} = 20602.34 lb [Eq. D-22]

ϕ = 0.70

ϕ_{seis} = 0.75

ϕV_{cbgy} = 10816.23 lb (for the anchor group)

ϕV_{cby} = 5408.11 lb (for a single anchor - divided ϕV_{cbgy} by 2)

Case 2: Anchor(s) furthest from edge checked against total shear load
In x-direction...

$V_{cbx} = A_{vcx}/A_{vcox}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-21]

c_{a1} = 32.00 in (adjusted for edges per D.6.2.4)

A_{vcx} = 1152.00 in^2

A_{vcox} = 4608.00 in^2 [Eq. D-23]

$\Psi_{ed,V}$ = 0.7219 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{bx} = 75919.09 lb

V_{cbx} = 13701.02 lb [Eq. D-22]

ϕ = 0.70

ϕ_{seis} = 0.75

ϕV_{cbx} = 7193.04 lb (for a single anchor)

In y-direction...

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 20.50 in

A_{vcy} = 1306.88 in^2

A_{vcoy} = 1891.13 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 0.7659 [Eq. D-27 or D-28]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{by} = 38927.53 lb

V_{cbgy} = 20602.34 lb [Eq. D-22]

ϕ = 0.70

ϕ_{seis} = 0.75

ϕV_{cbgy} = 10816.23 lb (for the entire anchor group)

Case 3: Anchor(s) closest to edge checked for parallel to edge condition
Check anchors at c_{x1} edge

$V_{cbx} = A_{vcx}/A_{vcox}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-21]

c_{a1} = 6.75 in

A_{vcx} = 137.95 in^2

A_{vcox} = 205.03 in^2 [Eq. D-23]

$\Psi_{ed,V}$ = 1.0000 [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{\ d_o}\sqrt{\ f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{bx} = 7354.98 lb

V_{cbx} = 4948.72 lb [Eq. D-22]

$V_{cby} = 2 * V_{cbx}$ [Sec. D.6.2.1(c)]

V_{cby} = 9897.45 lb

ϕ = 0.70

ϕ_{seis} = 0.75

ϕV_{cby} = 5196.16 lb (for a single anchor)

Check anchors at c_{y1} edge

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 3.50 in

A_{vcy} = 106.31 in^2

A_{vcoy} = 55.13 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{\ d_o}\sqrt{\ f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{by} = 2746.17 lb

V_{cbgy} = 5296.19 lb [Eq. D-22]

$V_{cbgx} = 2 * V_{cbgy}$ [Sec. D.6.2.1(c)]

V_{cbgx} = 10592.39 lb

ϕ = 0.70

ϕ_{seis} = 0.75

ϕV_{cbgx} = 5561.00 lb (for the anchor group)

Check anchors at c_{x2} edge

$V_{cbx} = A_{vcx}/A_{vcox}\Psi_{ed,V}\Psi_{c,V}V_{bx}$ [Eq. D-21]

c_{a1} = 26.00 in

A_{vcx} = 936.00 in^2

A_{vcox} = 3042.00 in^2 [Eq. D-23]

$\Psi_{ed,V}$ = 1.0000 [Eq. D-27 or D-28] [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{bx} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{bx} = 55601.44 lb

V_{cbx} = 17108.14 lb [Eq. D-22]

$V_{cby} = 2 * V_{cbx}$ [Sec. D.6.2.1(c)]

V_{cby} = 34216.27 lb

ϕ = 0.70

ϕ_{seis} = 0.75

ϕV_{cby} = 17963.54 lb (for a single anchor)

Check anchors at c_{y2} edge

$V_{cbgy} = A_{vcy}/A_{vcoy}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_{by}$ [Eq. D-22]

c_{a1} = 20.50 in

A_{vcy} = 1306.88 in^2

A_{vcoy} = 1891.13 in^2 [Eq. D-23]

$\Psi_{ec,V}$ = 1.0000 [Eq. D-26]

$\Psi_{ed,V}$ = 1.0000 [Sec. D.6.2.1(c)]

$\Psi_{c,V}$ = 1.0000 [Sec. D.6.2.7]

$V_{by} = 7(l_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ [Eq. D-24]

l_e = 5.00 in

V_{by} = 38927.53 lb

V_{cbgy} = 26901.14 lb [Eq. D-22]

$V_{cbgx} = 2 * V_{cbgy}$ [Sec. D.6.2.1(c)]

V_{cbgx} = 53802.28 lb

ϕ = 0.70

ϕ_{seis} = 0.75

ϕV_{cbgx} = 28246.20 lb (for the anchor group)

10) Concrete Pryout Strength of Anchor Group in Shear [Sec. D.6.3]

$V_{cpg} = \min[k_{cp}N_{ag}, k_{cp}N_{cbg}]$ [Eq. D-30b]

k_{cp} = 2 [Sec. D.6.3.2]

e_{Nx} = 0.00 in (Applied shear load eccentricity relative to anchor group c.g.)

e_{Ny} = 0.00 in (Applied shear load eccentricity relative to anchor group c.g.)

$\Psi_{ec,Nx}$ = 1.0000 [Eq. D-9] (Calulated using applied shear load eccentricity)

$\Psi_{ec,Ny}$ = 1.0000 [Eq. D-9] (Calulated using applied shear load eccentricity)

$\Psi_{ec,N'}$ = 1.0000 (Combination of x-axis & y-axis eccentricity factors)

$N_{ag} = (A_{Naa}/A_{Na})(\Psi_{ec,N'}/\Psi_{ec,Na})N_{ag}$

N_{ag} = 12545.52 lb (from Section (6) of calculations)

A_{Na} = 274.87 in^2 (from Section (6) of calculations)

A_{Naa} = 274.87 in^2 (considering all anchors)

$\Psi_{ec,Na}$ = 1.0000 (from Section(6) of calculations)

N_{ag} = 12545.52 lb (considering all anchors)

$N_{cbg} = (A_{Nca}/A_{Nc})(\Psi_{ec,N'}/\Psi_{ec,N})N_{cbg}$

N_{cbg} = 12487.65 lb (from Section (5) of calculations)

A_{Nc} = 510.00 in^2 (from Section (5) of calculations)

A_{Nca} = 510.00 in^2 (considering all anchors)

$\Psi_{ec,N}$ = 1.0000 (from Section(5) of calculations)

N_{cbg} = 12487.65 lb (considering all anchors)

V_{cpg} = 24975.31 lb

ϕ = 0.70 [D.4.4]

ϕ_{seis} = 0.75

ϕV_{cpg} = 13112.04 lb (for the anchor group)

11) Check Demand/Capacity Ratios [Sec. D.7]

Tension

- Steel : 0.1245
- Breakout : 0.8558
- Adhesive : 0.8519
- Sideface Blowout : N/A

Shear

- Steel : 0.0000
- Breakout (case 1) : 0.0000
- Breakout (case 2) : 0.0000
- Breakout (case 3) : 0.0000
- Pryout : 0.0000

V.Max(0) <= 0.2 and T.Max(0.86) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 5/8" diameter A615 GR. 60 SET-XP anchor(s) with 9 in. embedment

BRITTLE FAILURE GOVERNS: Governing anchor failure mode is brittle failure. Per 2006 IBC Section 1908.1.16, anchors shall be governed by a ductile steel element in structures assigned to Seismic Design Category C, D, E, or F. Alternatively the minimum design strength of the anchor(s) shall be at least 2.5 times the factored forces or the anchor attachment to the structure shall undergo ductile yielding at a load level less than the design strength of the anchor(s). Designer must exercise own judgement to determine if this design is suitable.

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Nipomo, CA 93444
(805) 801-9385

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SEISMIC RESTRAINT AT LINE 3 (CONT.)

FOOTING REQUIREMENTS (CONT.)

LINE 3 AT F.5 (CONT.)

DUE TO PROXIMITY OF (E) FTG TO (N) COLUMN LOCATION, A GRADE BM THAT RUNS ALONG LINE F.5 IMMEDIATELY ADJACENT TO THE (E) PAD FTG, WILL BE USED TO RESIST THE VERTICAL COMPONENTS OF THE COLUMN.

USE 18" W x 24" DP CONT. FTG w/ (2)-#5 TOP & BTM

SEE ENERCALC RESULTS

D PRODUCT 207

Shawn Pierce Engineering
277 N. Beechnut Ave
Nipomo, CA 93444
Phone: (805) 801-9385
Email: pierce.engineering@gmail.co

Title : Melford Borzall Expansion
Dsgnr: Shawn D. Pierce
Project Desc.:

Project Notes :

Job # 09.9-2/2

Printed: 7 APR 2010, 3:08PM

Concrete Beam Design

Z:\@Shawn Pierce Engineering\09_Projects\Borzal Addition_02\Engineering\Design\melford borzall expansion.ec6
ENERCALC, INC. 1983-2010, Ver: 6.1.03, N:40336

Lic. # : KW-06008640 | **License Owner : PIERCE ENGINEERING**

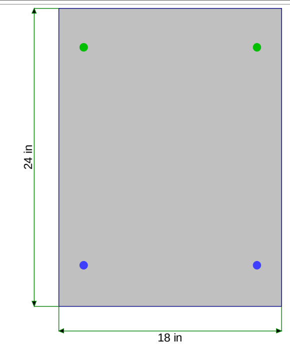
Description : Continuous footing under new column

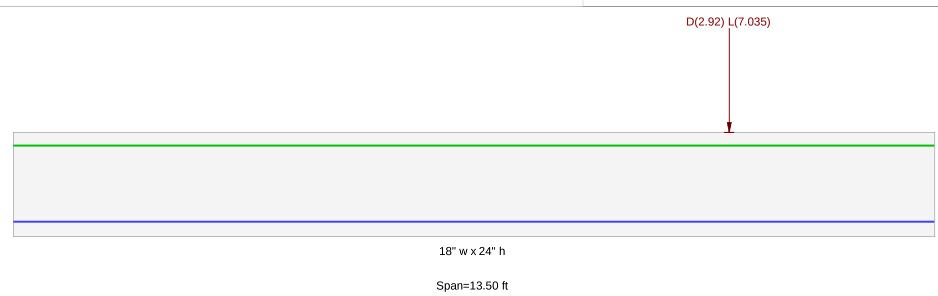
Material Properties

Calculations per IBC 2006, CBC 2007, ACI 318-05

f'c	=	2.50 ksi	ϕ Phi Values Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	375.0 psi	Shear :	0.750
ψ Density	=	145.0 pcf	β_1 =	0.850
Elastic Modulus	=	2,850.0 ksi		
Load Combination		2006 IBC & ASCE 7-05		
Fy - Main Rebar	=	60.0 ksi	Fy - Stirrups =	40.0 ksi
E - Main Rebar	=	29,000.0 ksi	E - Stirrups =	29,000.0 ksi
			Stirrup Bar Size # =	# 3
			Num of Bars Crossing Inclined Crack =	2

Beam is supported on an elastic foundation.





Cross Section & Reinforcing Details

Rectangular Section, Width = 18.0 in, Height = 24.0 in
Span #1 Reinforcing....
2-#5 at 3.313 in from Bottom, from 0.0 to 13.50 ft in this span
2-#5 at 3.125 in from Top, from 0.0 to 13.50 ft in this span

Applied Loads

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

Load for Span Number 1
Point Load : D = 2.920, L = 7.035 k @ 10.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.216** : 1	Maximum Deflection	
Section used for this span	**Typical Section**	Max Downward L+Lr+S Deflection	0.000 in
Mu : Applied	12.985 k-ft	Max Upward L+Lr+S Deflection	0.000 in
Mn * Phi : Allowable	60.210 k-ft	Max Downward Total Deflection	0.067 in
Load Combination	+1.20D+0.50Lr+1.60L+1.60	Max Upward Total Deflection	-0.035 in
Location of maximum on span	10.488 ft		
Span # where maximum occurs	Span # 1		

Maximum Soil Pressure = **1.455** ksf at 13.50 ft

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	-0.001	0.227
D Only	-0.000	0.066
D+L	-0.001	0.227

Shear Stirrup Requirements

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	10.488	12.98	60.21	0.22
+1.40D					
Span # 1	1	10.488	3.60	60.21	0.06
+1.20D+0.50Lr+1.60L+1.60H					
Span # 1	1	10.488	12.98	60.21	0.22
+1.20D+0.50L+0.50S+1.60H					
Span # 1	1	10.488	6.18	60.21	0.10
+1.20D+1.60Lr+0.50L					

Concrete Beam Design

Description : Continuous footing under new column

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 1	1	10.488	6.18	60.21	0.10
+1.20D+1.60Lr+0.50L+0.80W					
Span # 1	1	10.488	6.18	60.21	0.10
+1.20D+0.50L+1.60S					
Span # 1	1	10.488	6.18	60.21	0.10
+1.20D+0.50L+1.60S+0.80W					
Span # 1	1	10.488	6.18	60.21	0.10
+1.20D+0.50Lr+0.50L+1.60W					
Span # 1	1	10.488	6.18	60.21	0.10
+1.20D+0.50L+0.50S+1.60W					
Span # 1	1	10.488	6.18	60.21	0.10
+1.20D+0.50L+0.20S+E					
Span # 1	1	10.488	6.18	60.21	0.10
+0.90D+1.60W+1.60H					
Span # 1	1	10.488	2.31	60.21	0.04
+0.90D+E+1.60H					
Span # 1	1	10.488	2.31	60.21	0.04

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
Span 1	1	0.0674	13.500	Span 1	-0.0352	0.000

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277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB: BORZALL EXPANSION 09.9.2/2
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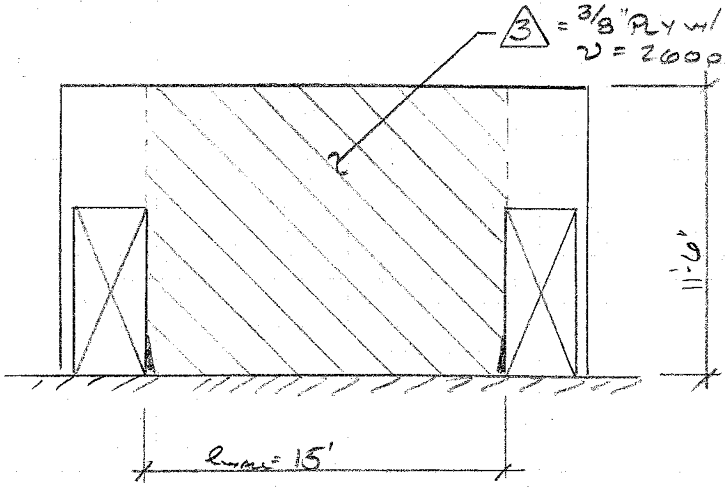
SCALE: CONFERENCE ROOM EXTENSION

– SHEARWALL ON LINE (2.5)

(E) SHEARWALL



HOLDOWNS = LTT20B w/ 1/2" ⌀ A.B. 10" EMBED INTO FTG

(E) SHEARWALL CAPACITY
$= v\,(l_{wall})$
$= 3900^{\#}$

(E) SHEARWALL UPLIFT (MAX)
$= 3900^{\#}\,(11'\text{-}6")/14'\text{-}6"$
$= 3090^{\#}$ >> CAPACITY OF LTT20B

PER SIMPSON CATALOG 98
ALLOWABLE TENSION LOAD $= 1750^{\#}$

- BASED ON CAPACITY OF (E) HOLDOWNS THE MAXIMUM SHEAR LOAD SHOULD BE $1750^{\#}\,(14'\text{-}6")/11'\text{-}6" = 2210^{\#}$
- USING THIS VALUE & ADDING THE LOAD FROM THE EXTENSION
  $v_{NEW} = (2210^{\#} + 350^{\#})/15'$
  $= 171\,plf < v = 260\,plf$
  (E) SHEARWALL NAILING OK
- W/ INCREASED LOAD (H) UPLIFT REQUIREMENTS ARE
  $(2210^{\#} + 350^{\#})(11'\text{-}6")/(14'\text{-}6")$
  $= 2030^{\#} = 116\%$ OF CAPACITY OF LTT

SUMMARY

(E) SHEARWALL NAILING IS OK FOR ADD'L LOADING. THE (E) HOLDOWN REQUIRES RETROFITTING, SEE NEXT PAGE

D PRODUCT 207

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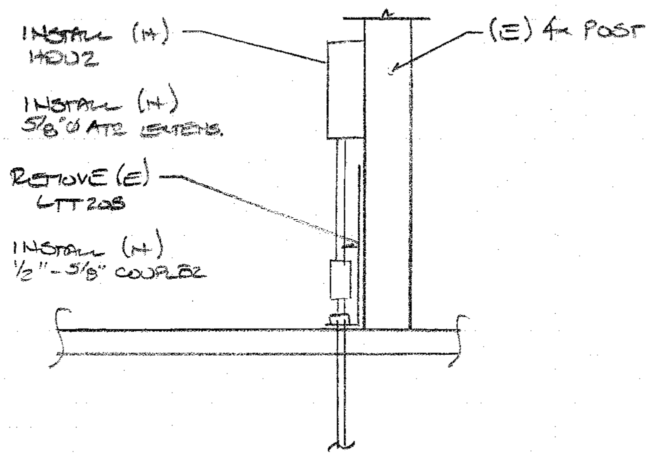
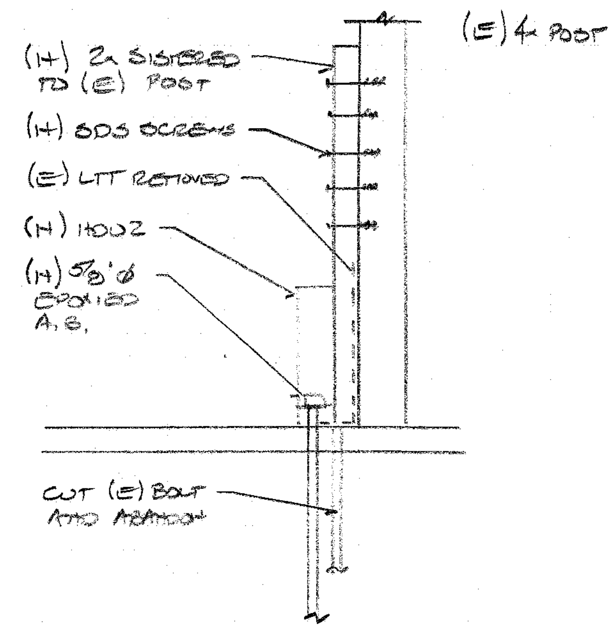
SCALE CONFERENCE ROOM EXTENSION

✓ SHEARWALL ON LINE 2.5 (CONT.)



1/2"ø A36 ANCHOR BOLT 'L' BOLT EMBEDDED 10" MIN INTO FTN

PER CALCULATIONS OF ACI 318-05 (E) ANCHOR BOLT FAILS IN 'PULLOUT' ∴ USE OF (E) ANCHOR BOLT WILL NOT BE ACCEPTABLE PER 2007 CBC



5/8"ø A307 GRADE C ALL THREAD BOLT EPOXIED INTO (E) FOOTING W/ SIMPSON 'SET' EPOXY & 5" MIN EMBEDMENT

SEE SIMPSON 'ANCHOR DESIGNER' FOR 3B' RESULTS

CAPACITY OF ROD = $0.31 in^2 (58 ksi)$
$= 17.98k \gg 2.5 (2030^{\#})$
NO BRITTLE FAILURE

✓ SDS SCREWS

$2030^{\#} / ((190^{\#}/screw)(1.6)) = 7$ SCREWS

USE (7) SDS 25212 SCREWS FROM 2x SISTERED TO (E) POST

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.0)

Job Name : Borzall - 5/8"Dia Anchor

Date/Time : 11/9/2009 6:51:40 AM

1) Input

Calculation Method : ACI 318 Appendix D For Cracked Concrete

Calculation Type : Analysis

a) Layout

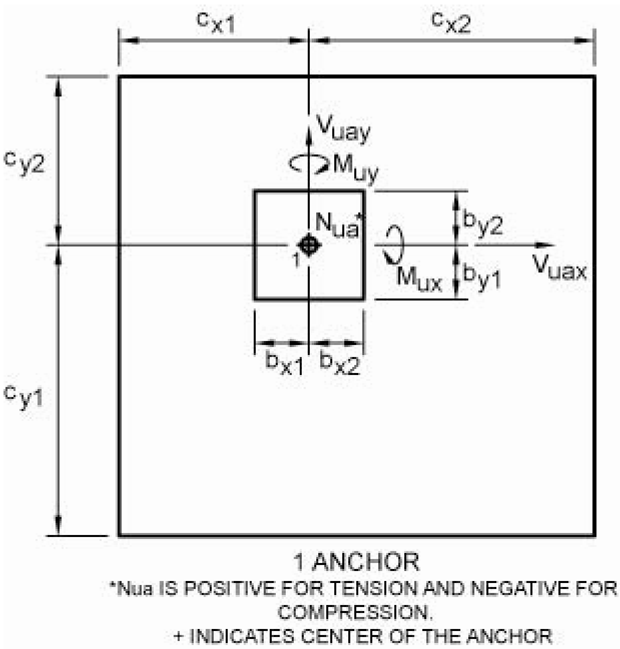
Anchor : 5/8" SET-XP

Number of Anchors : 1

Steel Grade: A307 GR. C

Embedment Depth : 5 in

Built-up Grout Pads : No



Anchor Layout Dimensions :

c_{x1} : 6 in

c_{x2} : 6 in

c_{y1} : 60 in

c_{y2} : 60 in

b) Base Material

Concrete : Normal weight

f'_c : 2500.0 psi

Cracked Concrete : Yes

$\Psi_{c,V}$: 1.00

Condition : A tension, B shear

ϕF_p : 1381.3 psi

Thickness, h : 18 in
Supplementary edge reinforcement : No
Hole Condition : Dry Concrete
Inspection : Continuous
Temperature Range : 1 (Maximum 110 ^{o}F short term and 75 ^{o}F long term temp.)

c) Factored Loads

Load factor source : ACI 318 Section 9.2

N_{ua} : 2030 lb

V_{uax} : 0 lb

V_{uay} : 0 lb

M_{ux} : 0 lb*ft

M_{uy} : 0 lb*ft

e_x : 0 in

e_y : 0 in

Moderate/high seismic risk or intermediate/high design category : Yes
Anchor w/ sustained tension : No
Anchors only resist wind and/or seismic loads : Yes
Apply entire shear load at front row for breakout : No

d) Anchor Parameters

From C-SAS-2009:

Anchor Model = SETXP	d_o = 0.625 in
Category = 1	h_{ef} = 5 in
h_{min} = 8.125 in	c_{ac} = 15 in
c_{min} = 1.75 in	s_{min} = 3 in
Ductile = Yes	

2) Tension Force on Each Individual Anchor

Anchor #1 N_{ua1} = 2030.00 lb

Sum of Anchor Tension ΣN_{ua} = 2030.00 lb

e'_{Nx} = 0.00 in

e'_{Ny} = 0.00 in

3) Shear Force on Each Individual Anchor

Resultant shear forces in each anchor:
Anchor #1 V_{ua1} = 0.00 lb (V_{ua1x} = 0.00 lb , V_{ua1y} = 0.00 lb)

Sum of Anchor Shear ΣV_{uax} = 0.00 lb, ΣV_{uay} = 0.00 lb

e'_{Vx} = 0.00 in

e'_{Vy} = 0.00 in

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

$N_{sa} = nA_{se}f_{uta}$ [Eq. D-3]

Number of anchors acting in tension, n = 1

N_{sa} = 13110 lb (for a single anchor) [C-SAS-2009]

ϕ = 0.75 [D.4.4]

ϕN_{sa} = 9832.50 lb (for a single anchor)

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

$N_{cb} = A_{Nc}/A_{Nco}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ [Eq. D-4]

Number of influencing edges = 2

h_{ef} = 5 in

A_{Nco} = 225.00 in^2 [Eq. D-6]

A_{Nc} = 180.00 in^2

Smallest edge distance, $c_{a,min}$ = 6.00 in

$\Psi_{ed,N}$ = 0.9400 [Eq. D-10 or D-11]

Note: Cracking shall be controlled per D.5.2.6

$\Psi_{c,N}$ = 1.0000 [Sec. D.5.2.6]

$\Psi_{cp,N}$ = 1.0000 [Eq. D-12 or D-13]

$N_b = k_c\sqrt{f'_c}\, h_{ef}^{1.5}$ = 9503.29 lb [Eq. D-7]

k_c = 17 [Sec. D.5.2.6]

N_{cb} = 7146.47 lb [Eq. D-4]

ϕ = 0.75 [D.4.4]

ϕ_{seis} = 0.75

ϕN_{cb} = 4019.89 lb (for a single anchor)

6) Adhesive Strength of Anchor in Tension [Sec. D.5.3 (AC308 Sec.3.3)]

$\tau_{k,cr}$ = 718 psi [C-SAS-2009]

k_{cr} = 17 [C-SAS-2009]

h_{ef} (unadjusted) = 5 in

$N_{ao} = \tau_{k,cr}\pi\, d_o h_{ef}$ = 7048.95 lb [Eq. D-16f]

$\tau_{k,uncr}$ = 2263.00 psi for use in [Eq. D-16d]

$s_{cr,Na} = \min[20d_o\sqrt{(\tau_{k,uncr}/1450)}\ , 3h_{ef}]$ = 15.000 in [Eq. D-16d]

$c_{cr,Na} = s_{cr,Na}/2$ = 7.500 in [Eq. D-16e]

$N_a = A_{Na}/A_{Nao}\Psi_{ed,Na}\Psi_{p,Na}N_{ao}$ [Eq. D-16a]

A_{Nao} = 225.00 in^2 [Eq. D-16c]

A_{Na} = 180.00 in^2

Smallest edge distance, $c_{a,min}$ = 6.00 in

$\Psi_{ed,Na} = \min[0.7+0.3c_{a,min}/c_{cr,Na}, 1.0] = 0.9400$ [Eq. D-16m]

$\Psi_{p,Na}$ = 1.0000 [Sec. D.5.3.14]

N_a = 5300.81 lb [Eq. D-16a]

ϕ = 0.65 [C-SAS-2009]

ϕ_{seis} = 0.75

ϕN_a = 2584.14 lb (for a single anchor)

7) Side Face Blowout of Anchor in Tension [Sec. D.5.4]

Concrete side face blowout strength is only calculated for headed anchors in tension close to an edge, $c_{a1} < 0.4h_{ef}$. Not applicable in this case.

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

V_{sa} = 7865.00 lb (for a single anchor)

$V_{eq} = V_{sa}\alpha_{v.seis}$ [AC308 Eq. 11-27]

$\alpha_{v.seis}$ = 0.71 [C-SAS-2009]

V_{eq} = 5584.15 lb

ϕ = 0.65 [D.4.4]

ϕV_{eq} = 3629.70 lb (for a single anchor)

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

Concrete breakout strength has not been evaluated against applied shear load(s) per user option. Refer to Section D.4.2.1 of ACI 318 for conditions where calculations of the concrete breakout strength may not be required.

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

$V_{cp} = \min[k_{cp}N_a, k_{cp}N_{cb}]$ [Eq. D-30a]

k_{cp} = 2 [Sec. D.6.3.2]

N_a = 5300.81 lb (from Section (6) of calculations)

N_{cb} = 7146.47 lb (from Section (5) of calculations)

V_{cp} = 10601.62 lb

ϕ = 0.70 [D.4.4]

ϕ_{seis} = 0.75

ϕV_{cp} = 5565.85 lb (for a single anchor)

11) Check Demand/Capacity Ratios [Sec. D.7]

Tension

- Steel : 0.2065
- Breakout : 0.5050
- Adhesive : 0.7856
- Sideface Blowout : N/A

Shear

- Steel : 0.0000
- Breakout : N/A
- Pryout : 0.0000

V.Max(0) <= 0.2 and T.Max(0.79) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 5/8" diameter A307 GR. C SET-XP anchor(s) with 5 in. embedment

BRITTLE FAILURE GOVERNS: Governing anchor failure mode is brittle failure. Per 2006 IBC Section 1908.1.16, anchors shall be governed by a ductile steel element in structures assigned to Seismic Design Category C, D, E, or F. Alternatively the minimum design strength of the anchor(s) shall be at least 2.5 times the factored forces or the anchor attachment to the structure shall undergo ductile yielding at a load level less than the design strength of the anchor(s). Designer must exercise own judgement to determine if this design is suitable.

Shawn Pierce Engineering
277 N. Beechnut Ave.
Nipomo, CA 93444
(805) 801-9385

JOB BORBAL EXPANSION 09.9-2/2
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE 11/09
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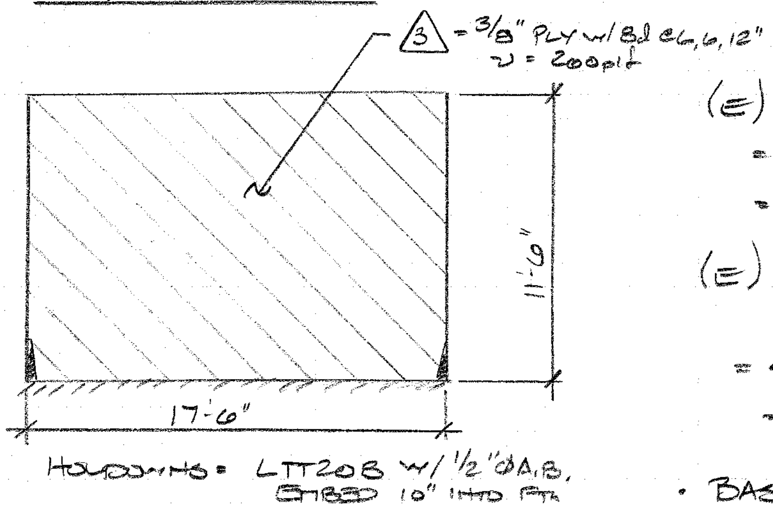
SCALE CONFERENCE ROOM EXTENSION

✓ SHEARWALLS ON LINES (F.5) & (J.5)

(E) SHEARWALL



(E) SHEARWALL CAPACITY
= v (l_{wall})
= 4550 #

(E) SHEARWALL UPLIFT (MAX)
(ASSUMING NO RESISTANT MOMENT)
= 4550# (11'-6")/17'
= 3080 # >> CAPACITY OF LTT20B (SEE PREV. CALC.)

- BASED ON CAPACITY OF LTT20B
  1750# (17'-0")/11'-6" = 2590#

- USING ABOVE VALUE & ADDING (N) LOAD
  2590# + 365# = 2955 # / 17'-6"
  v = 169plf < 260plf
  (E) SHEARWALL OK

- w/ INCREASED LOADS (N) UPLIFT IS
  2955# (11'-6")/17' = 2000#
  = 114% OF CAPACITY

- BASED ON CALC FOR LINE (2.5)
  ADD (N) HOLDOWN & EPOXY ANCHOR AS NOTED IN SUMMARY

SUMMARY

- (E) SHEARWALL NAILING OK FOR ADDITIONAL LOAD
- REMOVE (E) HOLDOWN & CUT (E) BOLT TO TOP OF SILL PL
- INSTALL (N) HDU2 HOLDOWN w/ 5/8"Ø A307 GRADE C ATR EPOXIED INTO (E) FTG w/ SIMPSON 'SET-XP' EPOXY & 5" MIN EMBED.
- INSTALL (N) 2x SISTERED TO (E) 4x POST w/ (7) SIMPSON SDS 25212 SCREWS

D PRODUCT 207

APPENDIX

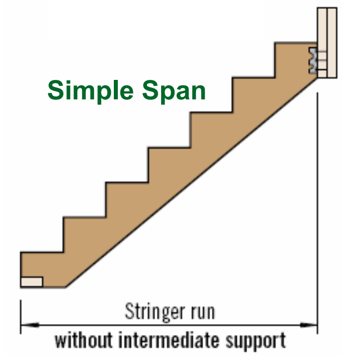
Trus Joist® 1¾" 1.55E TimberStrand® LSL Stair Stringer Tables

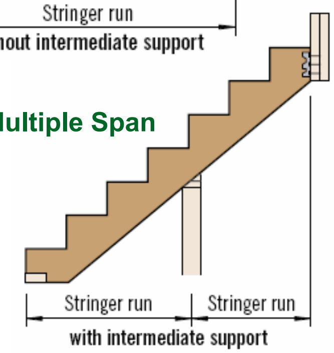
Maximum Stringer Run – 40 psf Live Load and 12 psf Dead Load

1¾" 1.55E LSL Stair Stringer Depth	36" Tread Width 2 Stringers With 2x4 Reinforcement	36" Tread Width 3 Stringers With 2x4 Reinforcement	42" Tread Width 3 Stringers With 2x4 Reinforcement	44" Tread Width 3 Stringers With 2x4 Reinforcement	48" Tread Width 3 Stringers With 2x4 Reinforcement
9.5"	7'-6"	8'-4"	7'-6"	7'-6"	7'-6"
11⅞"	10'-10"	12'-6"	12'-6"	11'-8"	11'-8"
14"	14'-2"	14'-2"	14'-2"	14'-2"	14'-2"

Maximum Stringer Run – 100 psf Live Load and 12 psf Dead Load

1¾" 1.55E LSL Stair Stringer Depth	36" Tread Width 2 Stringers With 2x4 Reinforcement	36" Tread Width 3 Stringers With 2x4 Reinforcement	42" Tread Width 3 Stringers With 2x4 Reinforcement	44" Tread Width 3 Stringers With 2x4 Reinforcement	48" Tread Width 3 Stringers With 2x4 Reinforcement
9.5"	5'-0"	5'-10"	5'-10"	5'-10"	5'-0"
11⅞"	8'-4"	9'-2"	9'-2"	8'-4"	8'-4"
14"	10'-0"	11'-8"	10'-10"	10'-10"	10'-10"





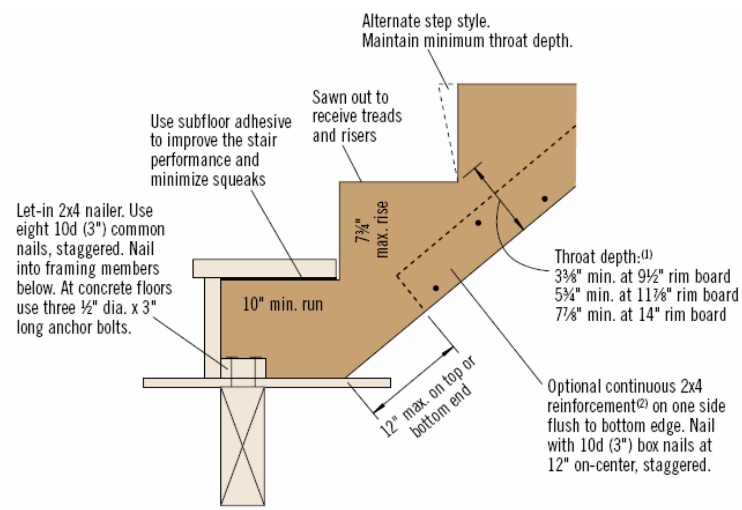
General Guidelines for Calculating Step Rise and Run

- The rise times the run should equal approximately 75".
- Two times the rise plus one run should equal approximately 25".
- Rise plus run should be 17" to 18".

General Notes

- Maximum stringer runs shown are valid for U.S codes (working stress) or Canadian codes (Limit State Design). Loads shown are unfactored.
- Deflection criteria of L/360 live load and L/240 total load.
- Stairway assembly is unstable until treads are installed.
- Use subfloor adhesive to improve the stair performance and minimize squeaks.
- Tables based on 7¾" maximum rise and 10" minimum run. Local codes may be more restrictive.
- Maximum rise between floors or landings permitted by code is 12'-0".
- Keep materials dry. Add vapor barrier at bottom of stair stringer if it is in contact with concrete
- The attachment detail shown is a suggestion only; alternate details are possible. Responsibility remains with the project designer or engineer of record.
- For assistance with loading conditions and stair configurations not shown, contact your iLevel representative.

Low End Attachment – 40 psf Live Load 12 psf Dead Load



(1) Minimum throat depths may be reduced by an additional ¼" for 11⅞" and 14" material depths if 2x4 reinforcement is used and provided total rises and runs are limited to table values for unreinforced stringers.
(2) Minimum No. 2 hem-fir, spruce-pine-fir or better grade.

See iLevel® Trus Joist® TimberStrand® LSL Stair Stringer Design Guide TJ-8003 for More Information and Installation Details

CONTACT US ▪ 1.888.iLevel8 (1.888.453.8358) ▪ www.iLevel.com

A3 Metal Building Foundations

A3.1 Introduction

This section will not attempt to cover an in-depth discussion of the fundamentals of foundation design, since it is a complicated subject that is sufficiently covered in a number of other books. This section will, however, illustrate several common foundation systems used by engineers on metal building projects to give the reader a general overview of the types of systems available. In addition, there will be a general discussion of the types of loads metal buildings impose on their foundations, as well as some mechanisms to resist those loads.

The manufacturer is not responsible for the design, materials and workmanship of the foundation. See Section 3.2.2 of the Common Industry Practices for more information about design responsibilities regarding foundations. It is strongly recommended that the anchorage and foundation of the building be designed by a Registered Professional Engineer experienced in the design of such structures.

The metal building manufacturer will generally provide an anchor bolt plan and data showing the diameter, location, and projection of all anchor bolts, base plate sizes, and the column reactions (magnitude and direction) for the metal building system. The manufacturer is responsible for determining the quantity and diameter of the anchor bolts to permit the transfer of forces between the base plate and the anchor bolt in shear and tension, but is not responsible for the transfer of anchor bolt forces to the concrete, or the adequacy of the anchor bolt in relation to the concrete.

A3.2 Types of Forces

Metal building foundations are subjected to all of the same loads as "conventional" building foundations, but with two major differences.

A3.2.1 Large Column Uplift Force

Because of design optimization, metal buildings are, by nature, light-weight structures. Therefore, the wind uplift force, as calculated using building codes, represents a significantly larger ratio of wind vs. building dead load than many conventional forms of construction. Even so, the total wind load on a frame has a diminished effect on the foundation because of response time. For this reason, it has been suggested that only 70 percent of the total wind load on a frame need be considered in the design of the foundation (Ref. B3.24 and B3.25). Anchor bolts should be designed for 100 percent of the wind load.

A3.2.2 Horizontal Thrust Force

Many metal buildings utilize rigid frames, which can generate a substantial horizontal "thrust" force from gravity loads. This force is usually directed outward and tends to make the foundation want to spread. For a pinned-base frame, Figure A3.2(a) illustrates this point. Figure A3.2(b) shows the fixed-base case with a moment "M" added.

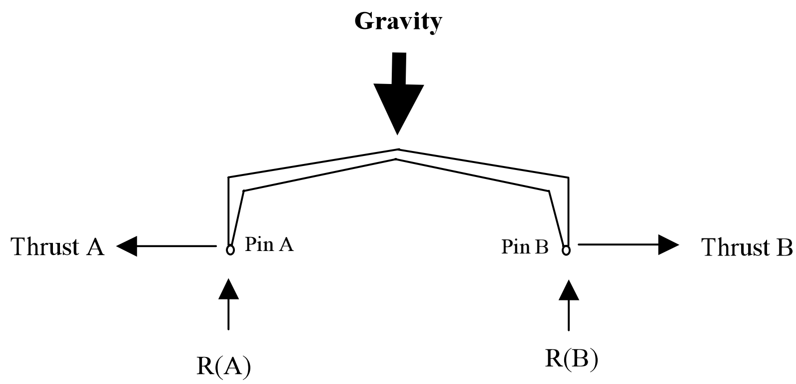


Figure A3.2(a) Horizontal "Thrust" Force (Pinned-base)

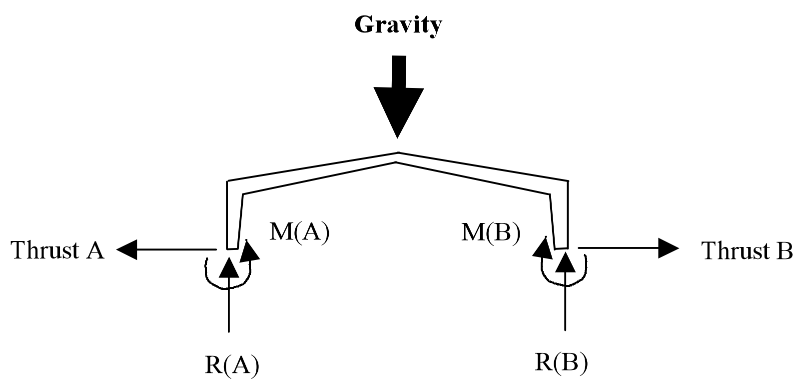


Figure A3.2(b) Horizontal "Thrust" Force (Fixed-base)

A3.3 Methods of Lateral Load Resistance

The lateral forces transferred to foundations by the metal building system can be resisted by the use of one of the following methods:

- Tension Ties
- “Hairpin” Rods
- Shear Block

A3.4 Tension Ties

A rod may be connected from a column (or pier) to the column (or pier) on the opposite side of the building, thus balancing the horizontal forces. (See Figure A3.4)

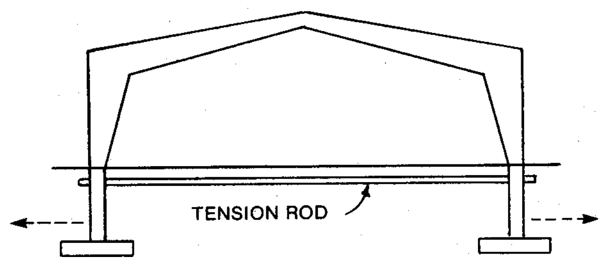


Figure A3.4 Tension rod

A3.5 Hairpin Rods

Another method of resisting horizontal forces involves use of a bent re-bar, or “hairpin”, which is cast into the slab-on-grade (See Figure A3.5). The slab-on-grade, if properly reinforced, can provide the required resistance to the horizontal shear force. The force is transferred from the anchor bolts to the hairpin rod, then to the concrete (through bond with the re-bar), and then into the mesh in the slab which acts as the final tensile element. Figure A3.5 illustrates the use of the spread tie for resisting horizontal thrusts.

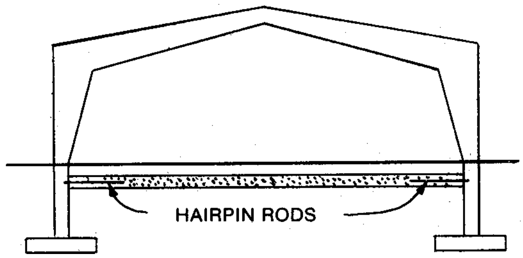


Figure - A3.5(a) Hairpin Rods

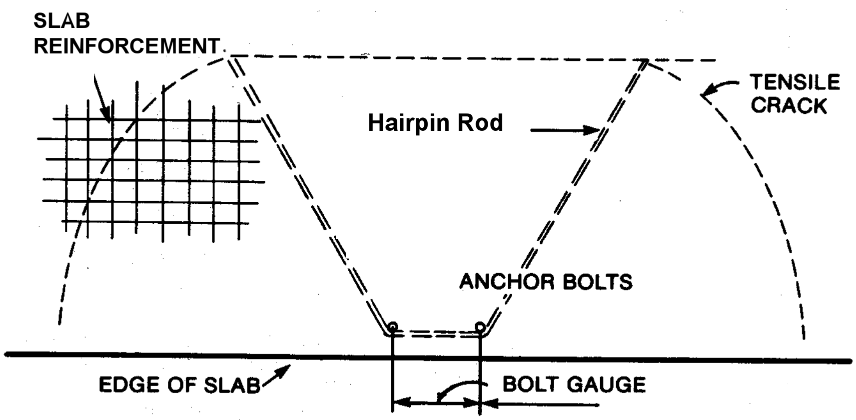


Figure A3.5(b) Spread Tie Rod

To adequately resist the horizontal thrusts, the spread tie rod must extend into the slab a sufficient distance so that the length of the “failure tensile crack” will have enough reinforcing mesh crossing it – such that a proper factor of safety is developed.

A3.6 Shear Blocks

In certain structures, it may not be possible to provide these tension ties or hairpin rods. The only means by which the horizontal forces can be resisted, to prevent sliding, is by either friction between footing and soil, use of a shear block, or a combination of the two. (For instance, retaining walls must commonly provide resistance to sliding, and use this method to do so). When the friction force is insufficient to resist the total horizontal force (with a proper factor of safety), it may be necessary to add a shear block. A shear block is nothing more than a depression on the bottom of a footing as shown in Figure A3.6.

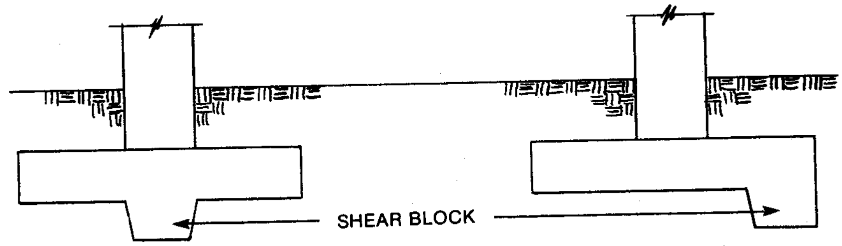


Figure A3.6 Examples of Shear Blocks

SET-XP™ *Structural Epoxy-Tie Anchoring Adhesive for Cracked and Uncracked Concrete*

TESTED TO MEET IBC® 2006

SIMPSON Strong-Tie ANCHOR SYSTEMS®

SET-XP™ is a 1:1 two component, high solids epoxy-based anchoring adhesive formulated for optimum performance in both cracked and uncracked concrete. SET-XP™ adhesive has been rigorously tested in accordance with ICC-ES AC308 and 2006 IBC requirements and has proven to offer increased reliability in the most adverse conditions, including performance in cracked concrete under static and seismic loading. SET-XP™ adhesive is teal in color in order to be identified as a high-performance adhesive for adverse conditions. Resin and hardener are dispensed and mixed simultaneously through the mixing nozzle. SET-XP™ adhesive exceeds the ASTM C881 specification for Type I and Type IV, Grade 3, Class C epoxy.

USES: • When SET-XP™ adhesive is used with the IXP™ anchor, all thread rod or rebar, the system can be used in tension and seismic zones where there is a risk of cracks occurring that pass through the anchor location. It is also suitable for uncracked concrete conditions.

CODES: ICC-ES ESR-2508; City of L.A. pending; Florida FL 11506.5 NSF/ANSI Standard 61 (216 in²/1000 gal). ⚠ The load tables list values based upon results from the most recent testing and may not reflect those in current code reports. Where code jurisdictions apply, consult the current reports for applicable load values.

APPLICATION: Surfaces to receive epoxy must be clean. The base-material temperature must be 50º F or above at the time of installation. For best results, material should be 70–80º F at the time of application. Cartridges should not be immersed in water to facilitate warming. To warm cold material, the cartridges should be stored in a warm, uniformly-heated area or storage container for a sufficient time to allow epoxy to warm completely. Mixed material in nozzle can harden in 5–7 minutes at a temperature of 40º F or above.

DESIGN EXAMPLE: See pages 26–28

INSTALLATION: See pages 31–31

SHELF LIFE: 24 months from date of manufacture in unopened side-by-side cartridge.

STORAGE CONDITIONS: For best results, store between 45–90° F. To store partially used cartridges, leave hardened nozzle in place. To re-use, attach new nozzle.

COLOR: Resin – white, hardener – black-green. When properly mixed, SET-XP adhesive will be a uniform teal color.

CLEAN UP: Uncured material – Wipe up with cotton cloths. If desired, scrub area with abrasive, waterbased cleaner and flush with water. If approved, solvents such as ketones (MEK, acetone, etc.), lacquer thinner or adhesive remover can be used. DO NOT USE SOLVENTS TO CLEAN ADHESIVE FROM SKIN. Take appropriate precautions when handling flammable solvents. Solvents may damage surfaces to which they are applied.
Cured Material – chip or grind off surface.

TEST CRITERIA: Anchors installed with SET-XP™ Epoxy-Tie® adhesive have been tested in accordance with ICC-ES's *Acceptance Criteria for Post-Installed Adhesive Anchors in Concrete Elements (AC308)* for the following:

- Seismic and wind loading in cracked and uncracked concrete
- Static tension and shear loading in cracked and uncracked concrete
- Horizontal and overhead installations
- Long-term creep at elevated-temperatures
- Static loading at elevated-temperatures
- Damp holes
- Freeze-thaw conditions
- Critical and minimum edge distance and spacing

PROPERTY	TEST METHOD	RESULTS
Consistency	ASTM C881	Passed, non-sag
Glass transition temperature	ASTM E1356	155°F
Bond strength (moist cure)	ASTM C882	3,742 psi at 2 days
Water absorption	ASTM D570	0.10%
Compressive yield strength	ASTM D695	14,830 psi
Compressive modulus	ASTM D695	644,000 psi
Gel time	ASTM C881	49 minutes

CHEMICAL RESISTANCE: Very good to excellent against distilled water, in-organic acids and alkalis. Fair to good against organic acids and alkalis, and many organic solvents. Poor against ketones. For more detailed information visit our website or contact Simpson Strong-Tie.

SET-XP™

EMN22i

EDT22B

SET-XP™ Adhesive

ANCHOR DESIGNER ACI 318 Software Supported

- IMPORTANT -
SEE Pages 31–32 FOR INSTALLATION INSTRUCTIONS

SET-XP Cartridge System

Model No.	Capacity ounces (cubic inches)	Cartridge Type	Carton Quantity	Dispensing tool(s)	Mixing Nozzle
SET-XP22	22 (39.7)	side-by-side	10	EDT22B, EDT22AP, EDT22CKT	EMN22i

1. Cartridge estimation guides are available on page 64.
2. Detailed information on dispensing tools, mixing nozzles and other adhesive accessories is available on pages 87–92.
3. Use only appropriate Simpson Strong-Tie mixing nozzle in accordance with Simpson Strong-Tie instructions. Modification or improper use of mixing nozzle may impair epoxy performance.

Cure Schedule

Base Material Temerature		Cure Time
ºF	ºC	
50	10	72 hrs.
70	21	24 hrs.
90	32	24 hrs.
110	43	24 hrs.

SUGGESTED SPECIFICATION: Anchoring adhesive shall be a two-component high-solids, epoxy-based system supplied in manufacturer's standard cartridge and dispensed through a static-mixing nozzle supplied by the manufacturer. The adhesive anchor shall have been tested and qualified for performance in cracked and uncracked concrete per ICC-ES AC308. Adhesive shall be SET-XP™ Epoxy-Tie® adhesive from Simpson Strong-Tie, Pleasanton, CA. Anchors shall be installed per Simpson Strong-Tie instructions for SET-XP Epoxy-Tie adhesive.

ACCESSORIES: See pages 87–92 for information on dispensing tools, mixing nozzles and other accessories.

SET-XP™ Epoxy Anchor Installation Information and Additional Data for Threaded Rod and Rebar in Normal-Weight Concrete[1]

Characteristic		Symbol	Units	Nominal Anchor Diameter				
				½ / #4	⅝ / #5	¾ / #6	⅞ / #7	1 / #8
Installation Information								
Drill Bit Diameter		d	in.	⅝	¾	⅞	1	1⅛
Maximum Tightening Torque		T_{inst}	ft-lb	40	90	130	200	300
Permitted Embedment Depth (h_{ef}) Range[2]	Minimum	-	in.	2¾	3⅛	3½	3¾	4
	Maximum	-	in.	10	12½	15	17 1/2	20
Minimum Concrete Thickness		h_{min}	in.	2.25 x h_{ef}				
Critical Edge Distance		c_{ac}	in.	3 x h_{ef}				
Minimum Edge Distance		c_{min}	in.	1¾				
Minimum Anchor Spacing		s_{min}	in.	3				

1. The information presented in this table is to be used in conjunction with the design criteria of ICC-ES AC308. See pages 18–19.
2. Minimum and maximum embedment depths are set so as to fit the ICC-ES AC308 design model.

SET-XP™ Epoxy Tension Design Data for Threaded Rod and Rebar in Normal-Weight Concrete[1,12]

*See page 10 for an explanation of the load table icons

Characteristic			Symbol	Units	Nominal Anchor Diameter (inch) / Rebar Size				
					½ / #4	⅝ / #5	¾ / #6	⅞ / #7	1 / #8
Steel Strength in Tension									
Threaded Rod	Minimum Tensile Stress Area		A_{se}	in²	0.142	0.226	0.334	0.462	0.606
	Tension Resistance of Steel - ASTM A193, Grade B7		N_{sa}	lb.	17,750	28,250	41,750	57,750	75,750
	- ASTM A307, Grade C				8,235	13,110	19,370	26,795	35,150
	- Type 410 Stainless (ASTM A193, Grade B6)				15,620	24,860	36,740	50,820	66,660
	- Type 304 Stainless (ASTM A193, Grade B8)				10,650	16,950	25,050	34,650	45,450
	Strength Reduction Factor - Steel Failure		ϕ	-	0.75[9]				
Rebar	Minimum Tensile Stress Area		A_{se}	in²	0.20	0.31	0.44	0.60	0.79
	Tension Resistance of Steel - Rebar (ASTM A615, Grade 60)		N_{sa}	lb.	18,000	27,900	39,600	54,000	71,100
	Strength Reduction Factor - Steel Failure		ϕ	-	0.65[9]				
Concrete Breakout Strength in Tension[15]									
Effectiveness Factor - Uncracked Concrete			k_{uncr}	-	24				
Effectiveness Factor - Cracked Concrete			k_{cr}	-	17				
Strength Reduction Factor - Breakout Failure			ϕ	-	0.65[11]				
Bond Strength in Tension (2,500 psi ≤ f'c ≤ 8,000 psi)[15]									
Temp. Range 1 for Uncracked Concrete[2,4,5]	Characteristic Bond Strength[8]		$\tau_{k,uncr}$	psi	2,422	2,263	1,942	1,670	2,003
	Permitted Embedment Depth Range	Minimum	h_{ef}	in	2¾	3⅛	3½	3¾	4
		Maximum			10	12½	15	17½	20
Temp. Range 1 for Cracked Concrete[2,4,5]	Characteristic Bond Strength[8,13,14]		$\tau_{k,cr}$	psi	1,040	718	1,003	619	968
	Permitted Embedment Depth Range	Minimum	h_{ef}	in	4	5	6	7	8
		Maximum			10	12½	15	17½	20
Temp. Range 2 for Uncracked Concrete[3,4,5]	Characteristic Bond Strength[6,8]		$\tau_{k,uncr}$	psi	1,250	1,170	1,005	860	1,035
	Permitted Embedment Depth Range	Minimum	h_{ef}	in	2¾	3⅛	3½	3¾	4
		Maximum			10	12½	15	17½	20
Temp. Range 2 for Cracked Concrete[3,4,5]	Characteristic Bond Strength[6,8,13,14]		$\tau_{k,cr}$	psi	537	371	518	320	500
	Permitted Embedment Depth Range	Minimum	h_{ef}	in	4	5	6	7	8
		Maximum			10	12½	15	17½	20
Bond Strength in Tension - Bond Strength Reduction Factors for Continuous Special Inspection									
Strength Reduction Factor - Dry Concrete			$\phi_{dry,ci}$	-	0.65[10]				
Strength Reduction Factor - Water-saturated Concrete			$\phi_{sat,ci}$	-	0.45[10]				
Additional Factor for Water-saturated Concrete[7]			$K_{sat,ci}$	-	0.57				
Bond Strength in Tension - Bond Strength Reduction Factors for Periodic Special Inspection									
Strength Reduction Factor - Dry Concrete			$\phi_{dry,pi}$	-	0.55[10]				
Strength Reduction Factor - Water-saturated Concrete			$\phi_{sat,pi}$	-	0.45[10]				
Additional Factor for Water-saturated Concrete[7]			$K_{sat,pi}$	-	0.48				

1. The information presented in this table is to be used in conjunction with the design criteria of ICC-ES AC308, except as modified below. See pages 18–19.
2. Temperature Range 1: Maximum short-term temperature of 110°F. Maximum long-term temperature of 75°F.
3. Temperature Range 2: Maximum short-term temperature of 150°F. Maximum long-term temperature of 110°F.
4. Short-term concrete temperatures are those that occur over short intervals (diurnal cycling).
5. Long-term concrete temperature are constant temperatures over a significant time period.
6. For anchors that only resist wind or seismic loads, bond strengths may be increased by 72%.
7. In water-saturated concrete, multiply $\tau_{k,uncr}$ and $\tau_{k,cr}$ by K_{sat}.
8. For anchors installed in overhead and subjected to tension resulting from sustained loading, multiply the value calculated for N_a according to ICC-ES AC308 by 0.75. See page 18.
9. The value of ϕ applies when the load combinations of ACI 318 Section 9.2 are used. If the load combinations of ACI 318 Appendix C are used, refer to Section D4.5 to determine the appropriate value of ϕ.
10. The value of ϕ applies when both the load combinations of ACI 318 Section 9.2 are used and the requirements of Section D4.4(c) for Condition B are met. If the load combinations of ACI 318 Appendix C are used, refer to Section D4.5 to determine the appropriate value of ϕ.
11. The value of ϕ applies when both the load combinations of ACI 318 Section 9.2 are used and the requirements of Section D4.4(c) for Condition B are met. If the load combinations of ACI 318 Section 9.2 are used and the requirements of Section D4.4(c) for Condition A are met, refer to Section D4.4 to determine the appropriate value of ϕ. If the load combinations of ACI 318 Appendix C are used, refer to Section D4.5 to determine the appropriate value of ϕ.
12. Sand-lightweight and all-lightweight concrete are beyond the scope of this table.
13. For anchors installed in regions assigned to Seismic Design Category C, D, E or F, the bond strength values for 7/8" anchors or #7 rebar anchors must be multiplied by $\alpha_{N,seis}$ = 0.80.
14. For anchors installed in regions assigned to Seismic Design Category C, D, E or F, the bond strength values for 1" anchors or #8 rebar anchors must be multiplied by $\alpha_{N,seis}$ = 0.92
15. The values of f'_c used for calculation purposes must not exceed 8,000 psi (55.1 MPa) for uncracked concrete. The value of f'_c used for calculation purposes must not exceed 2500 psi (17.2 MPa) for cracked concrete when calculating concrete breakout strength in tension and pullout strength in tension.

SET-XP™ Epoxy Shear Design Data for Threaded Rod and Rebar in Normal-Weight Concrete[1,5]

*See page 10 for an explanation of the load table icons

Characteristic		Symbol	Units	Nominal Anchor Diameter (inch) / Rebar Size				
				½ / #4	⅝ / #5	¾ / #6	⅞ / #7	1 / #8
Steel Strength in Shear								
Threaded Rod	Minimum Shear Stress Area	A_{se}	in²	0.142	0.226	0.334	0.462	0.606
	Shear Resistance of Steel - ASTM A193, Grade B7	V_{sa}[6]	lb.	10,650	16,950	25,050	34,650	45,450
	- ASTM A307, Grade C			4,940	7,865	11,625	16,080	21,090
	- Type 410 Stainless (ASTM A193, Grade B6)			9,370	14,910	22,040	30,490	40,000
	- Type 304 Stainless (ASTM A193, Grade B8)			6,390	10,170	15,030	20,790	27,270
	Reduction for Seismic Shear - ASTM A307, Grade C[6]	$\alpha_{V,seis}$	-	0.71				
	Reduction for Seismic Shear - ASTM A193, Grade B7[6]			0.71				
	Reduction for Seismic Shear - Stainless (ASTM A193, Grade B6)[6]			0.80				
	Reduction for Seismic Shear - Stainless (ASTM A193, Grade B8)[6]			0.80				
	Strength Reduction Factor - Steel Failure	ϕ	-	0.65[2]				
Rebar	Minimum Shear Stress Area	A_{se}	in²	0.20	0.31	0.44	0.60	0.79
	Shear Resistance of Steel - Rebar (ASTM A615, Grade 60)	V_{sa}[6]	lb.	10,800	16,740	23,760	32,400	42,660
	Reduction for Seismic Shear - Rebar (ASTM A615, Grade 60)[6]	$\alpha_{V,seis}$	-	0.80				
	Strength Reduction Factor - Steel Failure	ϕ	-	0.60[2]				
Concrete Breakout Strength in Shear								
Outside Diameter of Anchor		d_o	in.	0.500	0.625	0.750	0.875	1.000
Load Bearing Length of Anchor in Shear		ℓ_e	in.	h_{ef}				
Strength Reduction Factor - Breakout Failure		ϕ	-	0.70[3]				
Concrete Pryout Strength in Shear[7]								
Coefficient for Pryout Strength		k_{cp}	-	2.0				
Strength Reduction Factor - Pryout Failure		ϕ	-	0.70[4]				

1. The information presented in this table is to be used in conjunction with the design criteria of ICC-ES AC308, except as modified below. See pages 18–19.
2. The value of ϕ applies when the load combinations of ACI 318 Section 9.2 are used. If the load combinations of AC 318 Appendix C are used, refer to Section D4.5 to determine the appropriate value of ϕ.
3. The value of ϕ applies when both the load combinations of ACI 318 Section 9.2 are used and the requirements of Section D4.4(c) for Condition B are met. If the load combinations of ACI 318 Section 9.2 are used and the requirements of Section D4.4(c) for Condition A are met, refer to Section D4.4 to determine the appropriate value of ϕ. If the load combinations of ACI 318 Appendix C are used, refer to Section D4.5 to determine the appropriate value of ϕ.
4. The value of ϕ applies when both the load combinations of ACI 318 Section 9.2 are used and the requirements of Section D4.4(c) for Condition B are met. If the load combinations of ACI 318 Appendix C are used, refer to Section D4.5 to determine the appropriate value of ϕ.
5. Sand-lightweight and all-lightweight concrete are beyond the scope of this table.
6. The values of V_{sa} are applicable for both cracked and uncracked concrete. For anchors installed in regions assigned to Seismic Design Category C, D, E or F, V_{sa} must be multiplied by $\alpha_{V,seis}$ for the corresponding anchor material.
7. The values of f'_c used for calculation purposes must not exceed 8,000 psi (55.1 MPa) for uncracked concrete. The value of f'_c used for calculation purposes must not exceed 2500 psi (17.2 MPa) for cracked concrete when calculating concrete pryout strength in shear.

January 19, 2009

Subject: Determination of the Minimum Concrete Thickness for SET-XP™ adhesive in Cracked and Uncracked Concrete.

To Whom It May Concern:

Please be advised that based upon the results of additional testing by an independent testing and evaluation agency, the value of the minimum base material thickness, hmin, for SET-XP™ adhesive (ICC-ES ESR-2508) in both cracked and uncracked concrete may be determined using the following relationship:

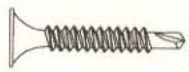
$$h_{min} = h_{ef} + 5d_o$$

where:
h_{min} = Minimum concrete thickness (inches) as defined in Tables 1 and 4 of ICC-ES ESR-2508;
h_{ef} = Permissible embedment depth (inches) as defined in Table 5 of ICC-ES ESR-2508 ; and
d_o = Nominal insert diameter (inches) as defined in Tables 2, 3 and 4 of ICC-ES ESR-2508

Sincerely,

Simpson Strong-Tie Anchor Systems®

Simpson Strong-Tie Anchor Systems® 5956 W. Las Positas Boulevard Pleasanton, CA 94588 Phone: 925.560.9000 Fax: 925.847.3871 www.simpsonanchors.com

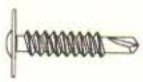


BUGLE HEAD - SELF DRILLING

For drywall attachment to metal studs and joists. Also used for attaching cabinets through gypsum board and insulation board.
Metal thickness – 20, 18 16 and 14 gauge.

BUGLE HEAD

For drywall attachment to metal studs and joists. Also used for attaching cabinets through gypsum board and insulation board.
Metal thickness – 25 and 20 gauge.



WAFER HEAD - SELF DRILLING

For metal to metal framing connections when drywall, plywood and other similar wall materials are used on metal studs and joists.
Metal thickness – 20, 18, 16 and 14 gauge.

WAFER HEAD

For metal to metal framing connections when drywall, plywood and other similar wall materials are used on metal studs and joists.
Metal thickness – 25 and 20 gauge.

HEX HEAD - SELF DRILLING

For attaching steel deck, backup plates, door frames, lathers channel to metal framing and structural connections.
Metal thickness – 20, 18, 16 and 14 gauge.

PAN HEAD - SELF DRILLING

For attaching steel deck, backup plates, door frames, lathers channel to metal framing and structural connections.
Metal thickness – 18, 16 and 14 gauge.

Screw Size / Section Gauge	No. 12 (d=0.209")		No. 10 (d=0.183")		No. 8 (d=0.161")		No. 6 (d=0.135")	
	Shear	Pullout	Shear	Pullout	Shear	Pullout	Shear	Pullout
25	74	50	69	44	65	39	60	32
20	185	92	173	81	163	71	149	60
18	276	120	258	105	242	93	222	78
16	387	151	363	132	341	116	N/A	N/A
14	548	190	513	166	N/A	N/A	N/A	N/A
Min. Edge Dist. And O.C. Spacing	11/16"		9/16"		1/2"		1/2"	

Notes:
1. All values based on connected parts having a minimum yield stress, Fy=33 ksi and a minimum ultimate stress, Fu=45 ksi.
2. When connecting materials of different gauge thickness, use loads shown for the lighter gauge.
3. Applied shear loads may be multiplied by 0.75 for wind or earthquake loads per AISI A4.4.
4. For screws in tension, the head of the screw, or washer if provided, shall have a minimum diameter of 5/16 inch.

Tapfast® Fasteners with Flo-Seal® Washers

Self-drilling Screws with Integral Washer

Ease Installation and Maximize Performance in Metal-to-Wood Applications

End costly wood frame fastening problems with Tapfast® fasteners, engineered for superior sealing and holding power, easier installation and outstanding corrosion resistance. Each incorporates a Flo-Seal® sealing washer to provide a custom seal.

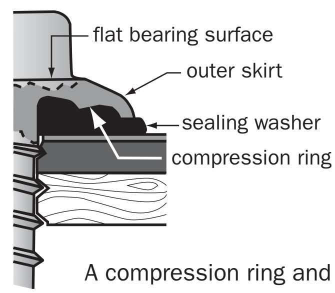
Eases Installation
- Flat bearing surface at the base of the hex head means easier, more stable driving
- Washer design compensates for different installation situations and techniques – engineered to provide reliable and consistent sealing even in under-tightened, over-tightened or angle-driven installations
- Formed cutting point easily penetrates multiple panel thicknesses, to simplify installation of light-gage sheet metal to wood framing

Special rugged sealing washer
- Designed to prevent spin-out and tearing
- UV- and heat-resistant – counteracts the effects of thermal change and building movement
- No metal-to-metal contact between fastener head and washer

Stalgard® coating standard
- Provides 800 hours of salt spray resistance (per ASTM B117)

Maximum in-place performance
- Flo-Seal® system ensures long-term protection against leakage
- #10 diameter and deep, spaced thread provide high pull-out values

Flo-Seal® Sealing System



A compression ring and outer skirt are designed right into the fastener head. When driven, this configuration controls the flow of the special sealing washer, ensuring sealing in critical areas.

Standard Tapfast® Fasteners

Description	Drive/ Head Style	Finish	Part No.	Qty./ Shipper
10# x 1"	Hex Washer	Silver Stalgard®	HFHH210	3000
10# x 1-1/2"			HFHH230	2500
10# x 2"			HFHH250	1500
10# x 2-1/2"			HFHH265	1000
10# x 3"			HFHH280	800

Tapfast® Specifications

Diameter: #10

Lengths: 1" to 3"

Head Style: Hex washer

Point Type: Pierce (cutting) for up to 18 gage steel

Material: Carbon steel

Washer Design: EPDM one-piece integral washer, 9/16" O.D.

Finish: Silver Stalgard® coating

Installation Tools: Can be installed with standard drive tools

Distributed By: