Smith Monroe Gray
ENGINEERS, INC.

CLIENT
PROJECT
BY DATE REV.
JOB NO. SHEET OF

Bin Wall / Footing Design (cont'd)

Check Uplift

$P_U = 42^k$

Weight of Wall = 7.4^k/ft

Length Reqd = $42^k \times 1.5$ (FOS) / 7.4^k/ft = 8.5 ft < H, OK By Inspection

Check Sliding (20' Trib Section)

$P_{SLIDE} = 3.4^{klf} \times 20' = 68^{kips}$

Weight of Wall = $7.4^k \times 20' = 148$ kips

Column Reaction = 10 kips (Min. Column Dead Load)

$P_{RESIST} = (10^k + 148^k) \times 0.5 = 79^{kips} > P_{SLIDE}$ OK ✓

(0.5 = Friction Factor)

Smith Monroe Gray
ENGINEERS, INC.

CLIENT: LAKESIDE INDUSTRIES
PROJECT: MAPLE VALLEY ASPHALT PLANT
FOUNDATION DESIGN
BY: BS DATE: 8/17/2018
JOB #: 18-183B SHEET OF

DESIGN OF RECTANGULAR FOOTING WITH OVERTURNING MOMENT

FOOTING:

LOADING PARAMETERS:

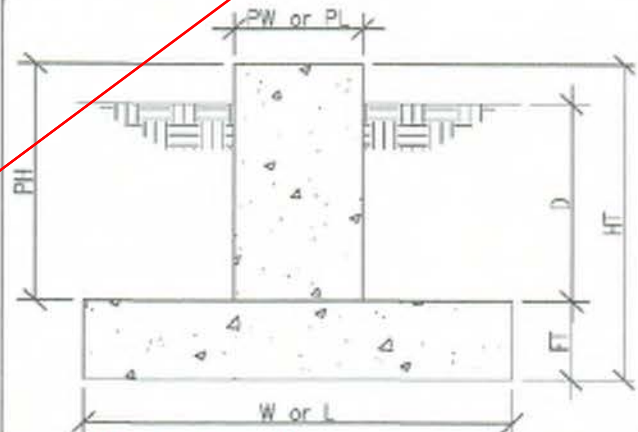
ALLOWABLE SOIL BEARING =	3,000	PSF
SOIL WEIGHT =	120	PCF
REQD. O.T. SAFETY FACTOR =	1.5	
STR.INCR.FOR HORIZ. LOADS =	1.00	
VERTICAL DEAD LOAD =	3.00	KIPS
VERTICAL LIVE LOAD =	0.00	KIPS
HORIZONTAL LOAD =	3.40	KIPS
MOMENT @ TOP OF FOOTING =	(25.00)	FT-KIPS

Surcharge only

NEGLECTS STOCKPILE RESISTING, CONSERVATIVE

FOOTING DIMENSIONS:

FTG. LENGTH (L) =	9.0	FT (PAR.TO LOAD)
FTG. WIDTH (W) =	1.0	FT (PERP.TO LOAD)
FTG. THICKNESS (FT) =	1.50	FT
FOOTING DEPTH (D) =	0.0	FT
PIER LENGTH (PL) =	3.0	FT
PIER WIDTH (PW) =	1.0	FT
PIER HEIGHT (PH) =	12.0	FT
CONCRETE WEIGHT =	7.4	KIPS
SOIL WEIGHT =	0.0	KIPS
TOTAL WEIGHT =	7.4	KIPS



$3.4^K \times (4' + 1.5')$
1/3 H
footing thickness

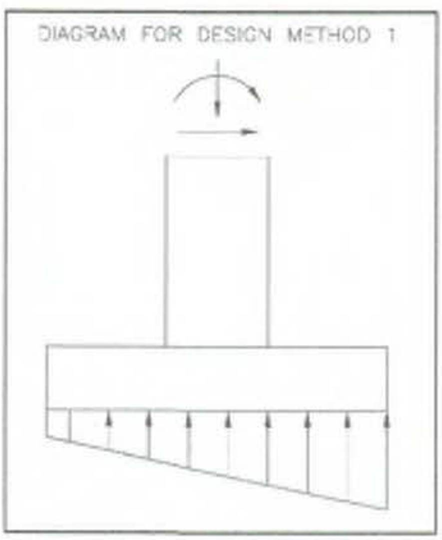
DESIGN METHOD 1

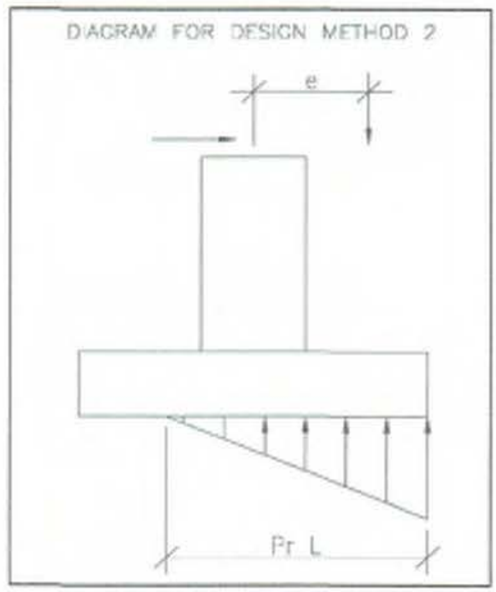
OVERTURNING MOM. =	20.9	FT-KIPS
SOIL PR. FROM DL =	1,158.3	PSF
SOIL PR. FROM MOM. =	(1,548.1)	PSF
MIN. PRESSURE =	(389.8)	PSF
MAX. PRESSURE =	2,706.5	PSF

DOES NOT APPLY AS UPLIFT AT BACK OF FOOTING

DESIGN METHOD 2

e =	2.00	FT	
Pr L =	7.49	FT	
MAX. PR =	2,785.3	PSF	<--- GOVERNS





	ACTUAL		ALLOWABLE		
LL + DL BEARING =	1,158	PSF	3,000	PSF	OK
DL + HORIZ. BEARING =	2,785	PSF	3,000	PSF	OK
F.S. OF OVERTURNING =	2.24		1.5		OK

Smith Monroe Gray
ENGINEERS, INC.

CLIENT: LAKESIDE INDUSTRIES
PROJECT: MAPLE VALLEY ASPHALT PLANT
FOUNDATION DESIGN
BY: BS DATE: 8/17/2018
JOB #: 18-183B SHEET OF

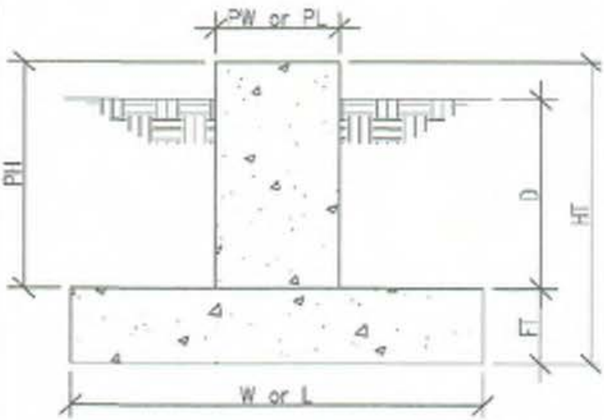
DESIGN OF RECTANGULAR FOOTING WITH OVERTURNING MOMENT

FOOTING:

LOADING PARAMETERS:

ALLOWABLE SOIL BEARING =	3,000	PSF
SOIL WEIGHT =	120	PCF
REQD. O.T. SAFETY FACTOR =	1.5	
STR.INCR.FOR HORIZ. LOADS =	1.33	
VERTICAL DEAD LOAD =	20.00	KIPS
VERTICAL LIVE LOAD =	0.00	KIPS
HORIZONTAL LOAD =	74.00	KIPS
MOMENT @ TOP OF FOOTING =	(517.00)	FT-KIPS

SURCHARGE + WIND



FOOTING DIMENSIONS:

FTG. LENGTH (L) =	9.0	FT (PAR.TO LOAD)
FTG. WIDTH (W) =	20.0	FT (PERP.TO LOAD)
FTG. THICKNESS (FT) =	1.50	FT
FOOTING DEPTH (D) =	0.0	FT
PIER LENGTH (PL) =	3.0	FT
PIER WIDTH (PW) =	20.0	FT
PIER HEIGHT (PH) =	12.0	FT
CONCRETE WEIGHT =	148.5	KIPS
SOIL WEIGHT =	0.0	KIPS
TOTAL WEIGHT =	148.5	KIPS

NEGLECTS STOCKPILE RESISTING, CONSERVATIVE

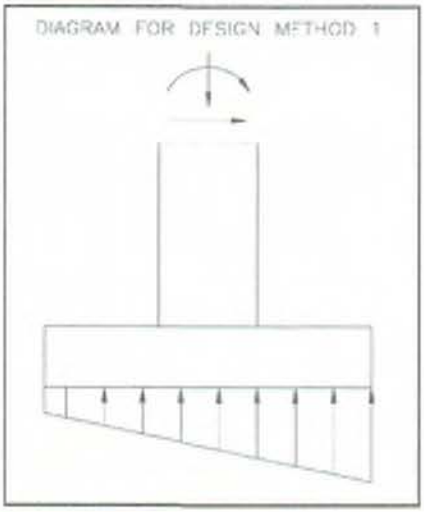
DESIGN METHOD 1

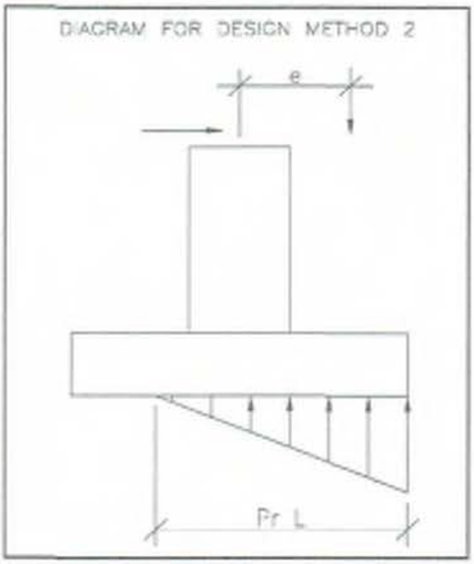
OVERTURNING MOM. =	482.0	FT-KIPS
SOIL PR. FROM DL =	936.1	PSF
SOIL PR. FROM MOM. =	(1,785.2)	PSF
MIN. PRESSURE =	(849.1)	PSF
MAX. PRESSURE =	2,721.3	PSF

DOES NOT APPLY AS UPLIFT AT BACK OF FOOTING

DESIGN METHOD 2

e =	2.86	FT	
Pr L =	4.92	FT	
MAX. PR =	3,425.9	PSF	<--- GOVERNS





	ACTUAL		ALLOWABLE		
LL + DL BEARING =	936	PSF	3,000 PSF		OK
DL + HORIZ. BEARING =	3,426	PSF	4,000 PSF		OK
F.S. OF OVERTURNING =	1.57		1.5		OK

Smith Monroe Gray
ENGINEERS, INC.

CLIENT ______
PROJECT ______
BY ______ DATE ______ REV. ______
JOB NO. ______ SHEET ______ OF ______

* PROVIDE SUPPLEMENTAL CALCULATIONS TO INCREASE BIN WALL STICK-UP TO 14 FT

TALLER WALL ONLY OCCURS @ BUILDING #2, ASSUME WORST CASE LOADING FROM 12 FT WALL CALCULATIONS, (CONSERVATIVE)

MAXIMUM DEMAND (PER MODEL)

FOOTING (LC #1 CONTROLS)

$M_U = 21.8^{k\cdot ft}/ft < \phi M_n = 28^{k\cdot ft}/ft$ OK ✓

$V_U = 11.5^{k}/ft < \phi V_n = 16^{k}/ft$ OK ✓

WALL (LC #3 CONTROLS, W/ WEIGHT OF SOIL ADDED)

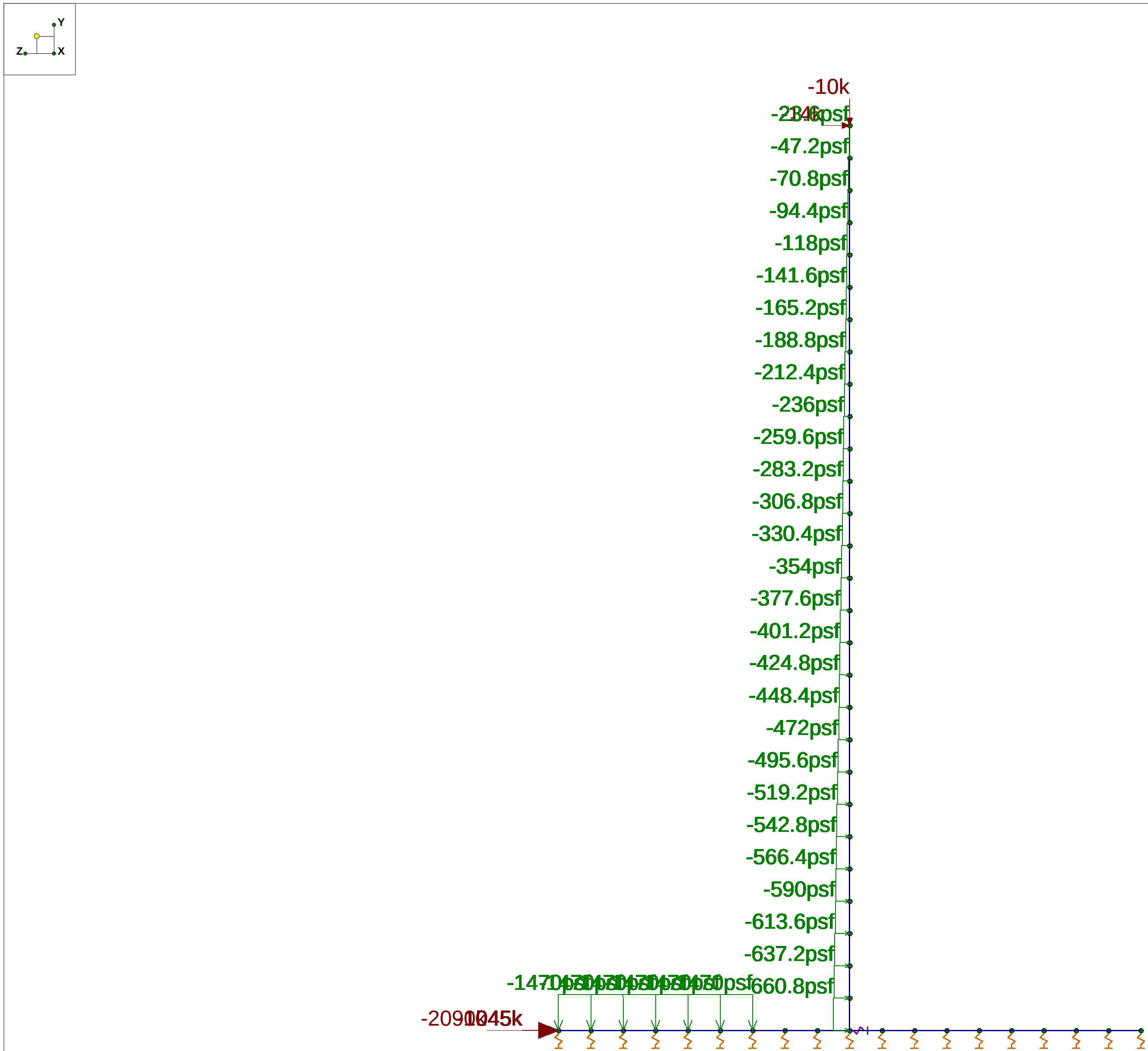
$M_U = 31.8^{k\cdot ft}/ft < \phi M_n = 64^{k\cdot ft}/ft$ OK ✓

$V_U = 4^{k}/ft < \phi V_n = 37^{k}/ft$ OK ✓

ORIGINAL REINFORCEMENT IS SUFFICIENT FOR EXTENDED HEIGHT

MAX BEARING PER MODEL ≈ 700# (6"x6") → 2800 PSF
< 3000 PSF
ALLOWABLE, OK ✓

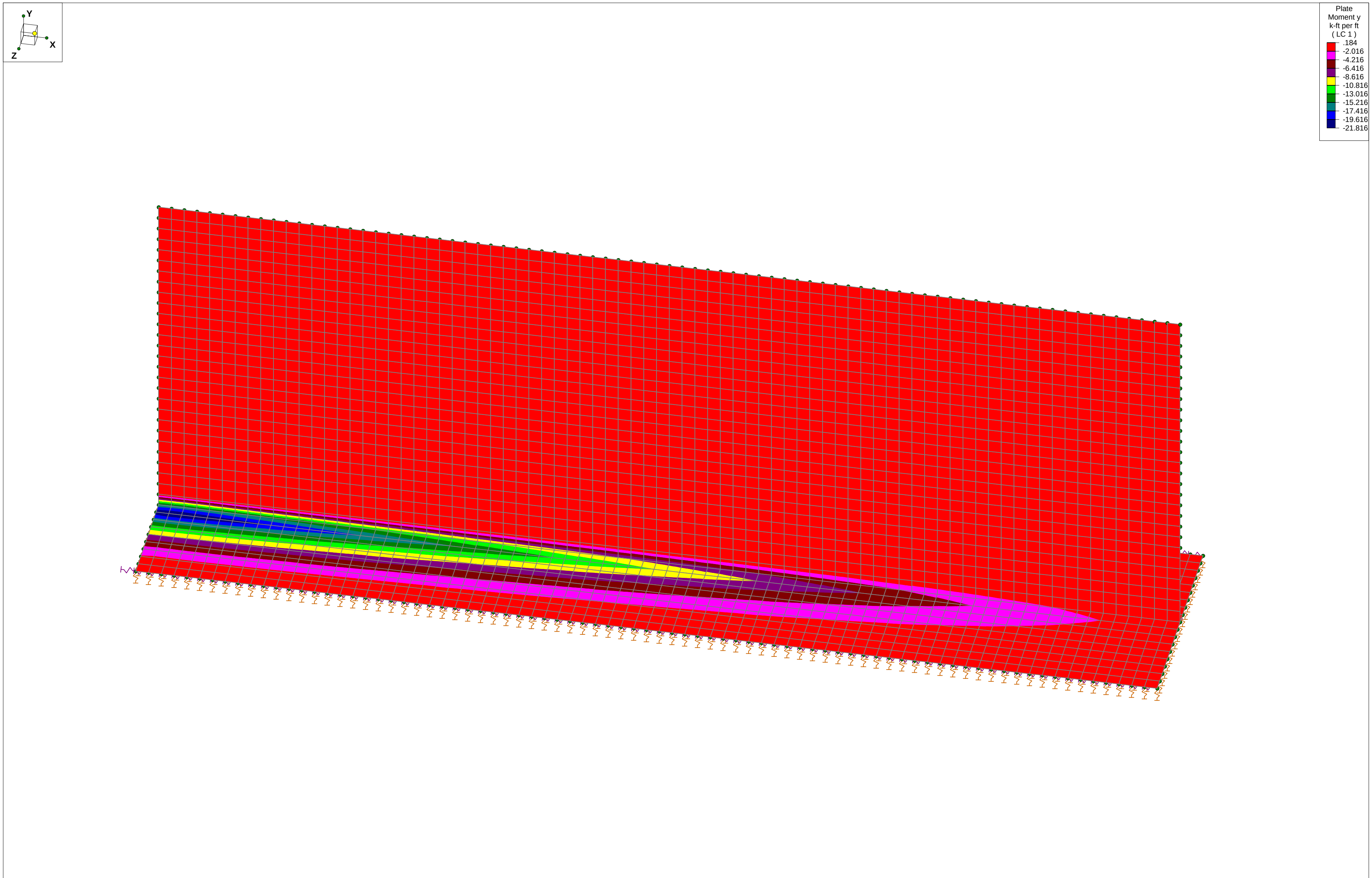
* UPLIFT & SLIDING CONTROLLED BY SHORTER WALL



Loads: LC 3, DL + SURCHARGE + WIND

SMG ENGINEERS	AGG STORAGE FOUNDATION	SK -
BS		Apr 9, 2019 at 5:55 PM
18-183B		AGG BUILDING FOUNDATION 15ft Rev1.r3d

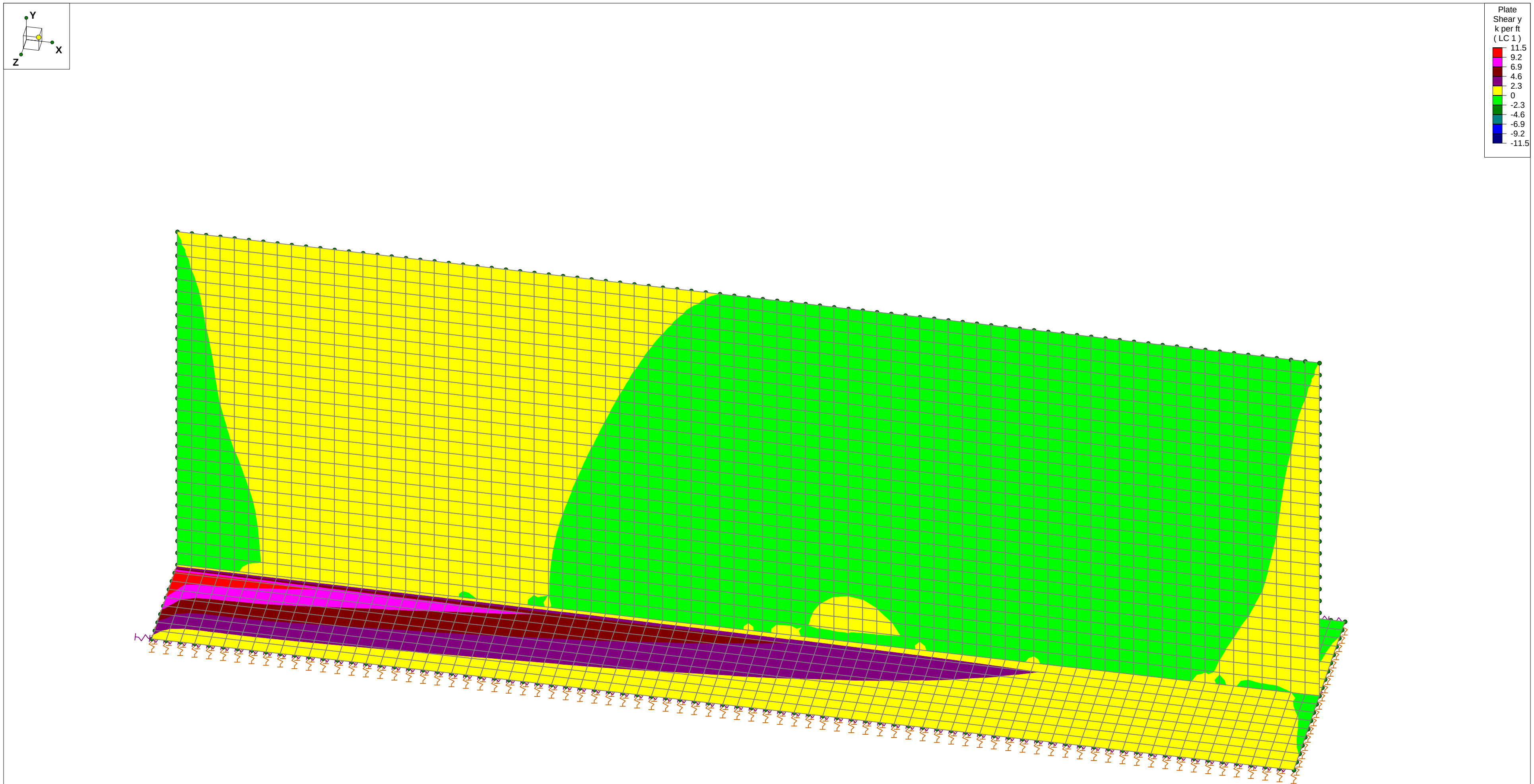
SHEET B154



Loads: BLC 1, SELF WEIGHT
Results for LC 1, DL + MAX COLUMN LOAD

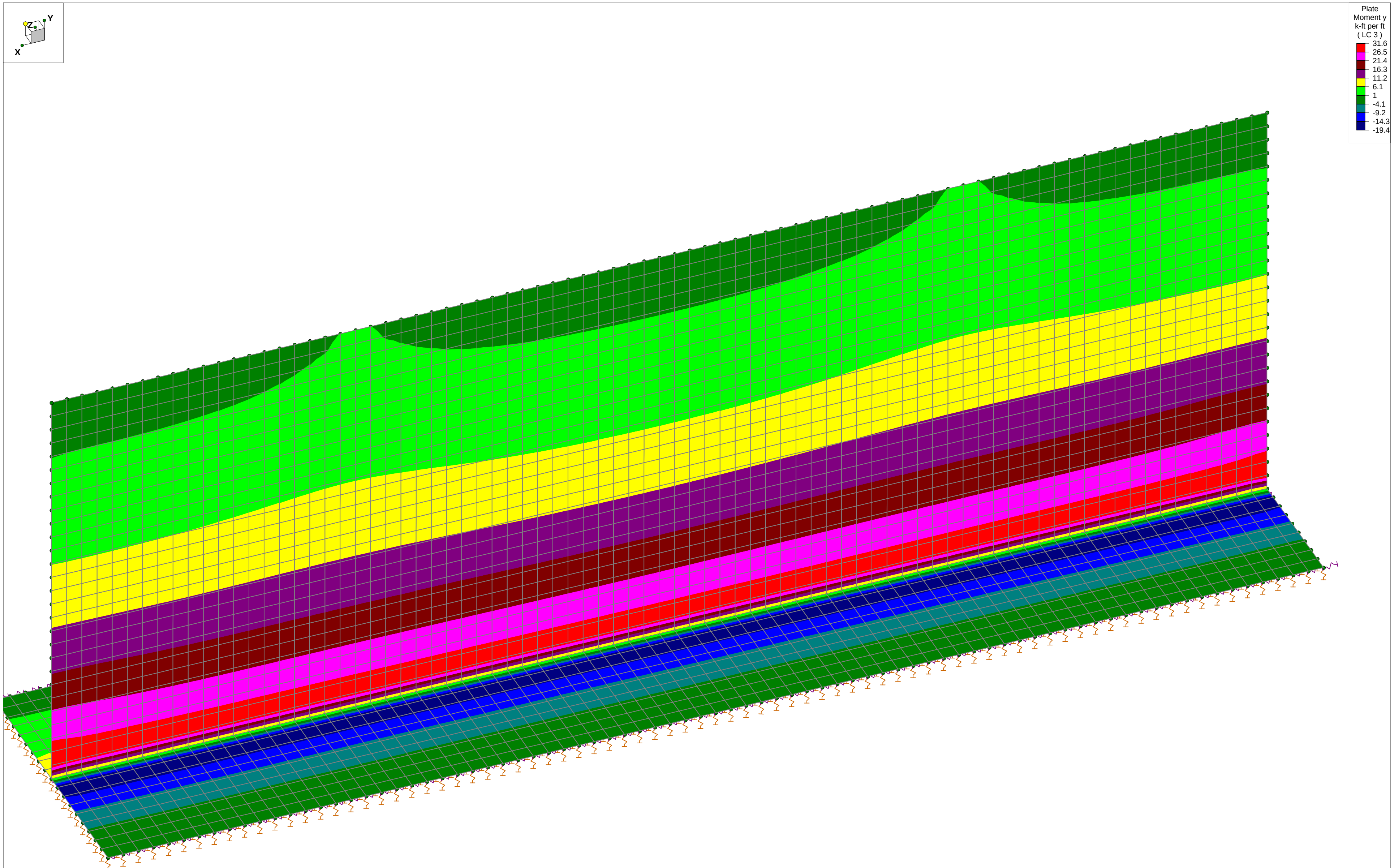
SMG ENGINEERS	AGG STORAGE FOUNDATION	SK - 1
BS		Apr 9, 2019 at 5:48 PM
18-183B		AGG BUILDING FOUNDATION 15ft Rev1.r3d

SHEET B155



Loads: BLC 1, SELF WEIGHT
Results for LC 1, DL + MAX COLUMN LOAD

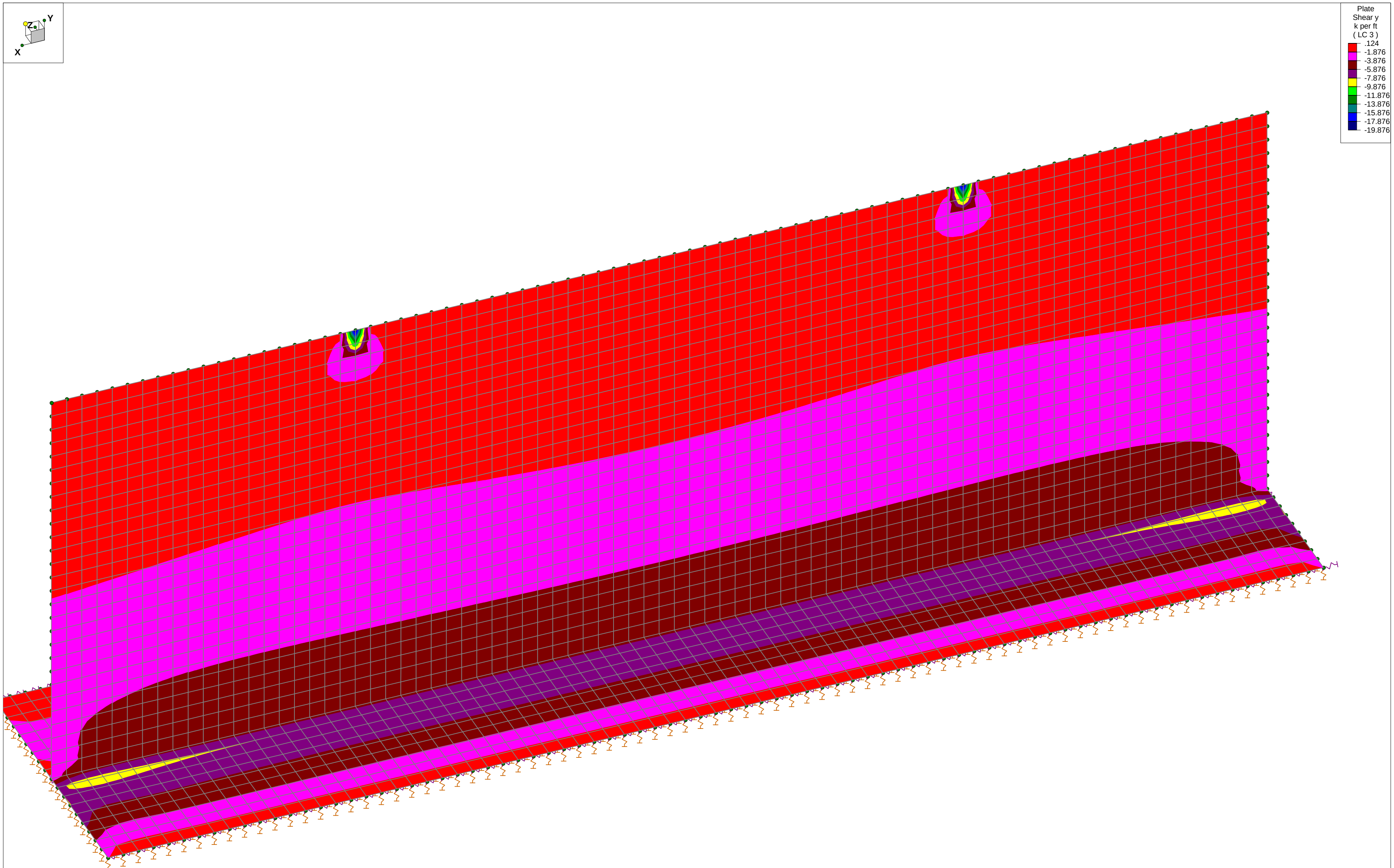
SMG ENGINEERS	AGG STORAGE FOUNDATION	SK -
BS		Apr 9, 2019 at 5:48 PM
18-183B		AGG BUILDING FOUNDATION 15ft Rev1.r3d



Loads: BLC 1, SELF WEIGHT
Results for LC 3, DL + SURCHARGE + WIND

SMG ENGINEERS	AGG STORAGE FOUNDATION	SK -
BS		Apr 9, 2019 at 5:49 PM
18-183B		AGG BUILDING FOUNDATION 15ft Rev1.r3d

SHEET B157



Loads: BLC 1, SELF WEIGHT
Results for LC 3, DL + SURCHARGE + WIND

SMG ENGINEERS	AGG STORAGE FOUNDATION	SK -
BS		Apr 9, 2019 at 5:50 PM
18-183B		AGG BUILDING FOUNDATION 15ft Rev1.r3d

SHEET B158

Smith Monroe Gray
ENGINEERS, INC.

CLIENT
PROJECT

BY DATE REV.
JOB NO. SHEET OF

14^{FT} WALL OVERTURNING

SURCHARGE + WIND

WIND COLUMN $R_{XN} = 10^{K}$ @ 20^{FT} OC (LATERAL)

SURCHARGE FORCE = 105 PCF · 0.45 · $14^{FT} \cdot 14^{FT}/2$ = $4630^{\#}/_{FT}$

↑ K_A

WALL DEMAND @ 20^{FT} SECTION

$P_{WIND} = 10^{K}$

$P_{SURCHARGE} = 4630^{\#}/_{FT} \cdot 20' = 92600^{\#} = 92.6^{K}$

MIN. COLUMN LOAD = 0 KIPS

M_{OVER} @ FOOTING = $10^{K} \cdot 15.5' + 92.6^{K} \cdot (14'/3 + 1.5') = 726^{K\text{-}FT}$

ASPHALT WT RESISTANCE

$W_T = 105\ PCF \cdot 14' \cdot 3' \cdot 20' = 88.2^{K}$

$ECC = 3^{FT}$

$M_R = 3^{FT} \cdot 88.2^{K} = 264^{K\text{-}FT}$

NET $M_{OVER} = 726 - 264 = 462^{K\text{-}FT}$

Smith Monroe Gray ENGINEERS, INC.

CLIENT: LAKESIDE INDUSTRIES
PROJECT: MAPLE VALLEY ASPHALT PLANT
FOUNDATION DESIGN
BY: BS DATE: 4/9/2019
JOB #: 18-183B SHEET OF

DESIGN OF RECTANGULAR FOOTING WITH OVERTURNING MOMENT

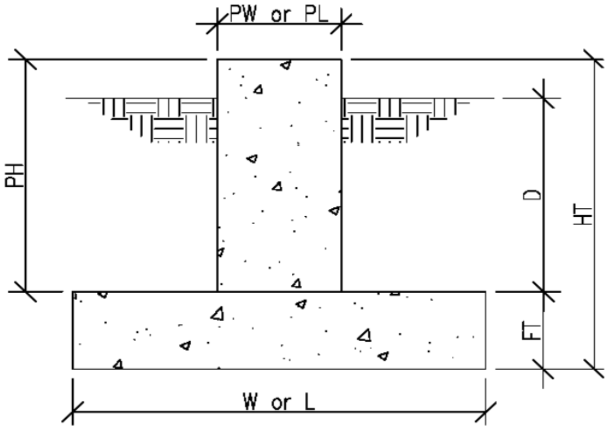
FOOTING:

LOADING PARAMETERS:

ALLOWABLE SOIL BEARING =	3,000	PSF
SOIL WEIGHT =	105	PCF
REQD. O.T. SAFETY FACTOR =	1.5	
STR.INCR.FOR HORIZ. LOADS =	1.33	
VERTICAL DEAD LOAD =	0.00	KIPS
VERTICAL LIVE LOAD =	0.00	KIPS
HORIZONTAL LOAD =	102.60	KIPS
MOMENT @ TOP OF FOOTING =		FT-KIPS

FOOTING DIMENSIONS:

FTG. LENGTH (L) =	9.0	FT (PAR.TO LOAD)
FTG. WIDTH (W) =	20.0	FT (PERP.TO LOAD)
FTG. THICKNESS (FT) =	1.50	FT
FOOTING DEPTH (D) =	0.0	FT
PIER LENGTH (PL) =	3.0	FT
PIER WIDTH (PW) =	20.0	FT
PIER HEIGHT (PH) =	14.0	FT
CONCRETE WEIGHT =	166.5	KIPS
SOIL WEIGHT =	0.0	KIPS
TOTAL WEIGHT =	166.5	KIPS



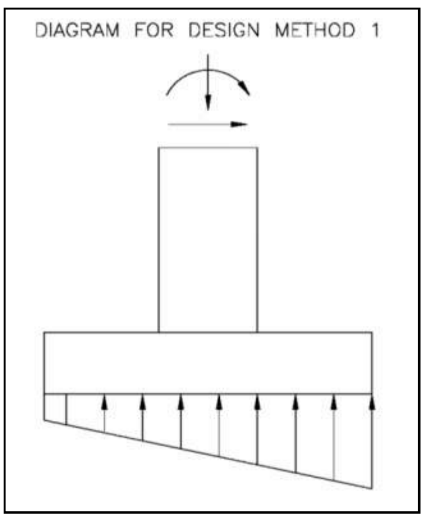
DESIGN METHOD 1

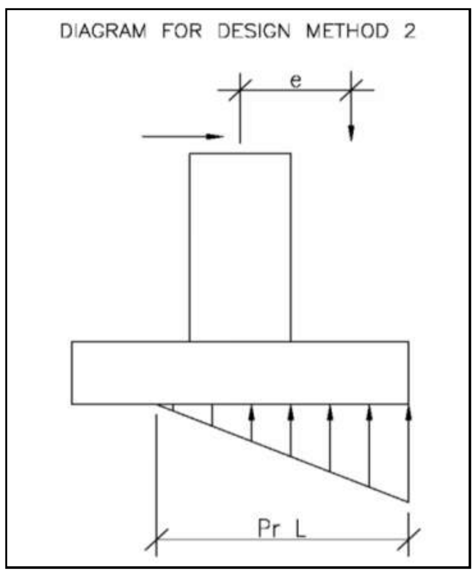
OVERTURNING MOM. =	462.0	FT-KIPS
SOIL PR. FROM DL =	925.0	PSF
SOIL PR. FROM MOM. =	(1,711.1)	PSF
MIN. PRESSURE =	(786.1)	PSF
MAX. PRESSURE =	2,636.1	PSF

DOES NOT APPLY AS UPLIFT AT BACK OF FOOTING

DESIGN METHOD 2

e =	2.77	FT	
Pr L =	5.18	FT	
MAX. PR =	3,217.0	PSF	**<--- GOVERNS**





	ACTUAL		ALLOWABLE		
LL + DL BEARING =	**925**	PSF	**3,000**	PSF	**OK**
DL + HORIZ. BEARING =	**3,217**	PSF	**4,000**	PSF	**OK**
F.S. OF OVERTURNING =	**1.62**		**1.5**		**OK**

Smith Monroe Gray
ENGINEERS, INC.

CLIENT
PROJECT
BY DATE REV.
JOB NO. SHEET OF

RAP CRUSHER & STORAGE BUILDING (BUILDING #3)

* SIMILAR IN DESIGN TO AGGREGATE STORAGE BUILDING, REFERENCE SHT. FOR INFO NOT SHOWN HERE

* NO INTERIOR BINS / WALLS, CENTER OF BUILDING USES A MOMENT FRAME FOR TRANSVERSE STABILITY

* SEE MODEL OUTPUT FOR MEMBER DEMANDS

* FOUNDATION DESIGN SIMILAR TO AGG BUILDING, COLUMN REACTIONS < AGG BUILDING, OK BY INSPECTION

USE ISLAND FOUNDATION TO SUPPORT SINGLE COLUMN @ OPEN FACE

DEMAND

$P_{max} = 131^{K}$ (LRFD)

$P_H = 10^{KIPS}$

$P_{DL} = 70^{KIPS}$ (BUILDING ONLY, NO FOOTING)

* SLIDING OK BY INSPECTION

SEE SPREADSHEET

USE 14" X 9' SQ FTG w/ #6's @ 12 OC

STABILITY (ASD) RXNS

MAX. TENSION = 12^{KIPS} LC26

$V_{LONG} = 24^{KIPS}$ LC37

$V_{TRANS} = 8^{KIPS}$ LC28

Smith Monroe Gray
ENGINEERS, INC.

CLIENT: LAKESIDE INDUSTRIES
PROJECT: MAPLE VALLEY ASPHALT PLANT
FOUNDATION DESIGN
BY: BS
SHEET: _____ OF _____
DATE: 11/2/2018

SINGLE REINFORCED RECTANGULAR CONCRETE FOOTING ANALYSIS

FOOTING CRITERIA:

L:	9.00	FT
W:	9.00	FT
DEPTH:	14.00	IN
COVER:	3.00	IN

APPLIED LOAD:

P:	131.00	K
LOAD FACTOR:	1.00	
P_u:	131	K

REINFORCEMENT IN 9.00 FOOT DIRECTION:

BAR SIZE: 6
QUANTITY: 10

CONCRETE DESIGN CRITERIA:

CONC f'_c:	4000	PSI
REINF F_y:	60000	PSI
β_1 =	0.85	

REINFORCEMENT IN 9.00 FOOT DIRECTION:

BAR SIZE: 6
QUANTITY: 10

COLUMN BEARING AREA:

L:	52.00	IN
W:	36.00	IN

ALLOWABLE SOIL BEARING PRESSURE: 3.00 KSF

SOIL BEARING:

FTG DL=	14.18	K	f_p=	1.79	KSF (P+DL)	**DEMAND= 0.60**
P+DL=	145.18	K	f_p=	1.62	KSF (P_u ONLY)	

PUNCHING SHEAR:

V_u=	99.12	K	ϕV_n=	403.77	K	**DEMAND= 0.25**

CONCRETE DESIGN FOR 9.00 FOOT DIRECTION:

A_s=	4.40	IN^2	a=	0.72	IN
$A_{s,min}$ =	2.07	IN^2	c= β_1*a =	0.85	IN
d=	10.625	IN	ε_t = [(d-c)/c]*0.003 =	0.0347	**> 0.005, OK**

ULTIMATE FORCES:			NOMINAL STRENGTH:			DEMAND RATIOS:	
M_u=	39.62	FT-K	ϕM_n=	203.26	FT-K	**Mu/φMn=**	**0.19**
V_u=	21.08	K	ϕV_n=	108.86	K	**Vu/φVn=**	**0.19**

CONCRETE DESIGN FOR 9.00 FOOT DIRECTION:

A_s=	4.40	IN^2	a=	0.72	IN
$A_{s,min}$ =	1.92	IN^2	c= a/β_1 =	0.85	IN
d=	9.875	IN	ε_t = [(d-c)/c]*0.003 =	0.0320	**> 0.005, OK**

ULTIMATE FORCES:			NOMINAL STRENGTH:			DEMAND RATIOS:	
M_u=	65.50	FT-K	ϕM_n=	188.41	FT-K	**Mu/φMn=**	**0.35**
V_u=	31.69	K	ϕV_n=	101.18	K	**Vu/φVn=**	**0.31**

Smith Monroe Gray
ENGINEERS, INC.

CLIENT
PROJECT
BY DATE REV.
JOB NO. SHEET OF

Bar Development

Cover Req's:
Exposed = 2"
Cast Against Earth = 3"

Hooked Development

$Min\ \phi = 6 \cdot d_b$ (#3 - #8)

$l_{ext} = 12 \cdot d_b$

$l_{dh_1} = \left(\frac{f_y \cdot \psi_e \cdot \psi_c \cdot \psi_r}{50 \cdot \lambda \cdot \sqrt{f'_c}}\right) \cdot d_b = 13.3 \cdot d_b \rightarrow$ #7 = 11.6" Min

$f_y = 60000$ psi
$\psi_e = 1.0$ (Uncoated)
$\psi_c = 0.7$ (Cover > 2½")
$\psi_r = 1.0$
$f'_c = 4000$ psi

$l_{dh_2} = 8 d_b$

$l_{dh_3} = 6"$

Splice Design

* Dr < 50% → Class B Splice

Splice, $l_{st} = 1.3 \cdot l_d$ or 12"

$l_d = \left(\frac{f_y \cdot \psi_t \cdot \psi_e}{X \cdot \lambda \cdot \sqrt{f'_c}}\right) d_b \rightarrow$ 49.3 d_b @ #6 = 37"
61.6 d_b @ #7 = 54"

$f_y = 60000$ psi
$\psi_t = 1.3$
$\psi_e = 1.0$ (Uncoated)
$f'_c = 4000$ psi

X = 25 @ #6
20 @ #7

DESIGN OF RECTANGULAR FOOTING WITH OVERTURNING MOMENT

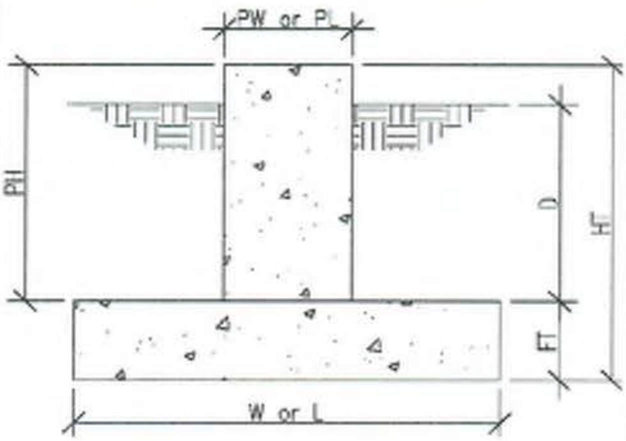
FOOTING:

LOADING PARAMETERS:

ALLOWABLE SOIL BEARING =	3,000	PSF
SOIL WEIGHT =	120	PCF
REQD. O.T. SAFETY FACTOR =	1.5	
STR.INCR.FOR HORIZ. LOADS =	1.33	
VERTICAL DEAD LOAD =	(12.00)	KIPS
VERTICAL LIVE LOAD =	0.00	KIPS
HORIZONTAL LOAD =	8.00	KIPS
MOMENT @ TOP OF FOOTING =	0.00	FT-KIPS

FOOTING DIMENSIONS:

FTG. LENGTH (L) =	9.0	FT (PAR.TO LOAD)
FTG. WIDTH (W) =	13.5	FT (PERP.TO LOAD)
FTG. THICKNESS (FT) =	1.50	FT
FOOTING DEPTH (D) =	0.0	FT
PIER LENGTH (PL) =	3.0	FT
PIER WIDTH (PW) =	13.5	FT
PIER HEIGHT (PH) =	12.0	FT
CONCRETE WEIGHT =	100.2	KIPS
SOIL WEIGHT =	0.0	KIPS
TOTAL WEIGHT =	100.2	KIPS



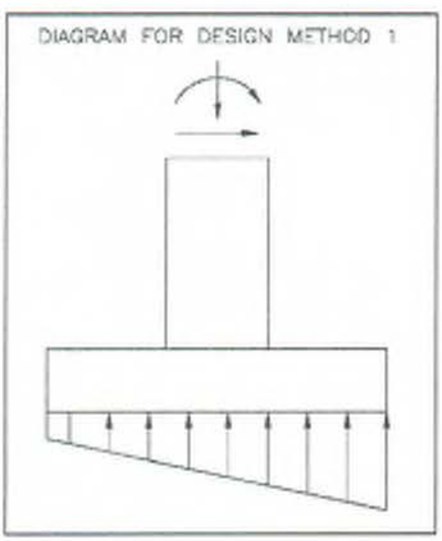
DESIGN METHOD 1

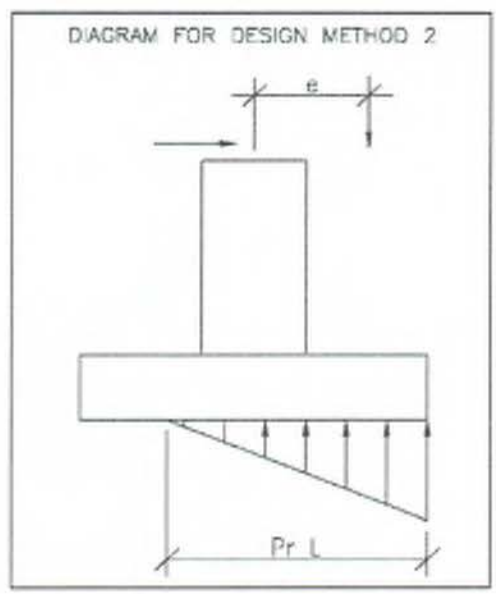
OVERTURNING MOM. =	108.0	FT-KIPS	
SOIL PR. FROM DL =	726.2	PSF	
SOIL PR. FROM MOM. =	(592.6)	PSF	
MIN. PRESSURE =	133.6	PSF	
MAX. PRESSURE =	1,318.8	PSF	**GOVERNS**

DESIGN METHOD 2

e =	1.22	FT
Pr L =	9.83	FT
MAX. PR =	1,330.1	PSF

DOES NOT APPLY AS NO UPLIFT AT BACK OF FOOTING





	ACTUAL		ALLOWABLE		
LL + DL BEARING =	726	PSF	3,000	PSF	OK
DL + HORIZ. BEARING =	1,319	PSF	4,000	PSF	OK
F.S. OF OVERTURNING =	3.68		1.5		OK

Smith Monroe Gray
ENGINEERS, INC.

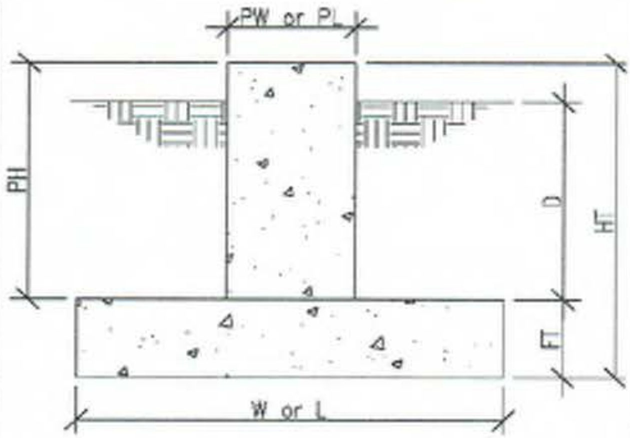
CLIENT: LAKESIDE INDUSTRIES
PROJECT: MAPLE VALLEY ASPHALT PLANT
FOUNDATION DESIGN
BY: BS DATE: 9/29/2018
JOB #: 18-183B SHEET OF

DESIGN OF RECTANGULAR FOOTING WITH OVERTURNING MOMENT

FOOTING:

LOADING PARAMETERS:

ALLOWABLE SOIL BEARING =	3,000	PSF
SOIL WEIGHT =	120	PCF
REQD. O.T. SAFETY FACTOR =	1.5	
STR.INCR.FOR HORIZ. LOADS =	1.33	
VERTICAL DEAD LOAD =	(24.00)	KIPS
VERTICAL LIVE LOAD =	0.00	KIPS
HORIZONTAL LOAD =	48.00	KIPS
MOMENT @ TOP OF FOOTING =	0.00	FT-KIPS



FOOTING DIMENSIONS:

FTG. LENGTH (L) =	27.0	FT (PAR.TO LOAD)
FTG. WIDTH (W) =	9.0	FT (PERP.TO LOAD)
FTG. THICKNESS (FT) =	1.50	FT
FOOTING DEPTH (D) =	0.0	FT
PIER LENGTH (PL) =	27.0	FT
PIER WIDTH (PW) =	3.0	FT
PIER HEIGHT (PH) =	12.0	FT
CONCRETE WEIGHT =	200.5	KIPS
SOIL WEIGHT =	0.0	KIPS
TOTAL WEIGHT =	200.5	KIPS

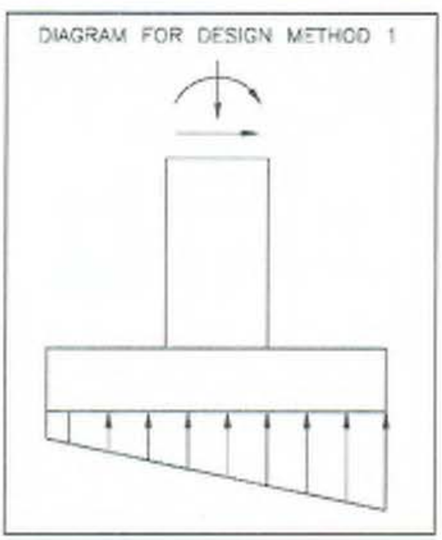
DESIGN METHOD 1

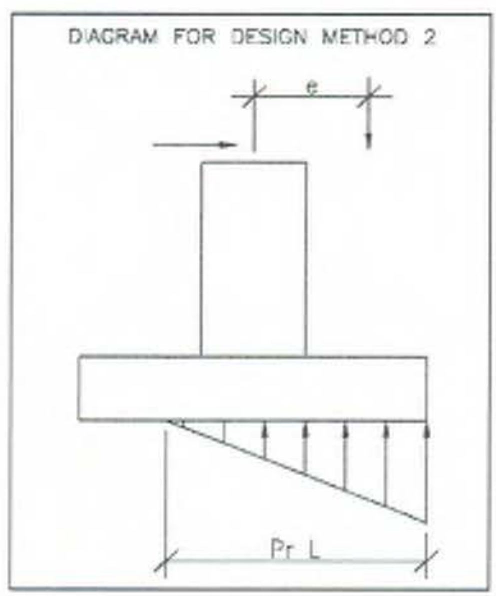
OVERTURNING MOM. =	648.0	FT-KIPS	
SOIL PR. FROM DL =	726.2	PSF	
SOIL PR. FROM MOM. =	(592.6)	PSF	
MIN. PRESSURE =	133.6	PSF	
MAX. PRESSURE =	1,318.8	PSF	**GOVERNS**

DESIGN METHOD 2

e =	3.67	FT
Pr L =	29.48	FT
MAX. PR =	1,330.1	PSF

DOES NOT APPLY AS NO UPLIFT AT BACK OF FOOTING





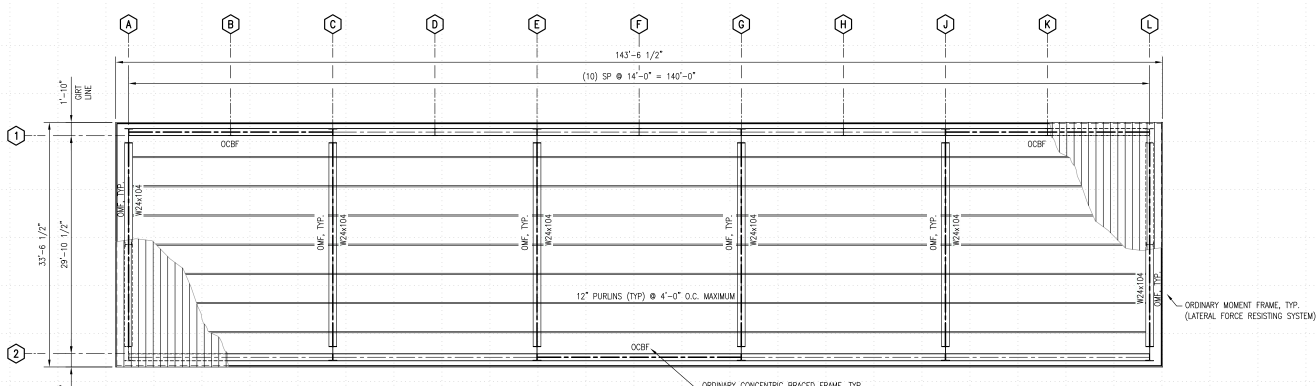
	ACTUAL		ALLOWABLE		
LL + DL BEARING =	726	PSF	3,000	PSF	OK
DL + HORIZ. BEARING =	1,319	PSF	4,000	PSF	OK
F.S. OF OVERTURNING =	3.68		1.5		OK

EQUIPMENT STORAGE BUILDING:

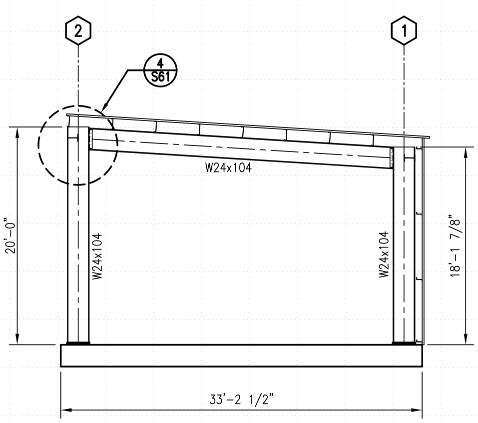
DL = SELF WEIGHT
LL = 125 PSF (LIGHT STORAGE)

SNOW LOAD = 25 PSF <- CONTROLS
ROOF LIVE LOAD = 20 PSF

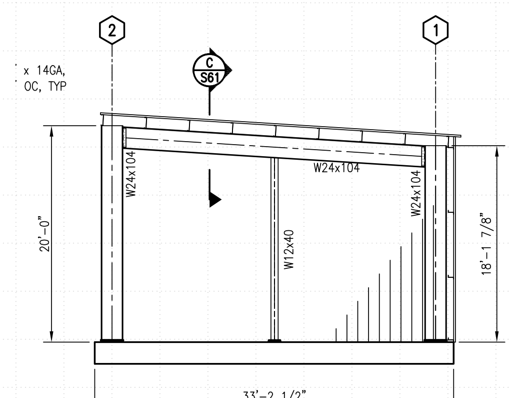
WIND & SEISMIC (SEE DESIGN CRITERIA)



D S60 ROOF PLAN 1/8" = 1'-0"



G S60 SECTION 1/8" = 1'-0"



F S60 NORTH ELEVATION 1/8" = 1'-0"

EQUIPMENT STORAGE: ROOF DESIGN

MAX. PRESSURE

DECKING DL = 3 PSF

WIND, 0.6W = VARIES, 24 PSF (POSITIVE), 25 PSF (NEGATIVE)

SNOW = 25 PSF

ROOF LIVE LOAD = 25 PSF

MAX. LOAD COMBINATION PRESSURE = 28 PSF (DL + SNOW)

TRY VERCO PLB 36 x 22GA w/ Z-PURLINS @ 4' OC

DECKING, $S_{MIN} = 0.176\ in^3 \rightarrow M_n/\Omega = \frac{F_y \cdot S}{1.67} = \frac{50\ ksi \cdot 0.176\ in^3}{1.67} = 5.27\ ^{k\text{-}in}/_{ft}$

$M_n/\Omega = 5.27\ ^{k\text{-}in}/_{ft} \times 1^{ft}/12^{in} \times 1000\ ^{\#}/_{k} = 439\ ^{\#\cdot ft}/_{ft}$

$M_{max} = wl^2/8 = 29\ PLF\ (4')^2/8 = 58\ ^{\#\cdot ft}/_{ft}$ OK ✓

Z-PURLIN LOAD, $w = 28\ PSF \times 4'\ OC = 112\ PLF$

@ 30' SPAN, 12" x 3½" x 12GA → $W_{ALLOW} = 153\ PLF$

USE VERCO PLB 36 x 22GA w/ 12" x 3½" x 12GA Z PURLINS @ 4' OC MAX.

Equipment Storage: Wall Design

Max. Wind Pressure, 0.6W = 21 PSF

Try Verco PLB 36 x 22 GA Decking w/ Z-Purlin @ 6' OC Max.

Per Previous, $M_n/\Omega = 439^{\#\cdot FT}$

$M_{MAX} = 21\ PSF \times 1^{FT} \times (6')^2/8 = 95^{\#\cdot FT}$ OK ✓

Z Purlin Load = $21\ PSF \times 6^{FT} = 126\ PLF$

@ 15' Span → 6" x 2½" x 14 GA → $W_{ALLOW} = 147\ PLF$

Use Verco PLB 36x22 GA w/ 6" x 2½" x 14 GA Z Purlins @ 6' OC Max.

FLEXOSPAN - CEE AND ZEE LOAD TABLES

Allowable Uniform Loads In Pounds Per Lineal Foot

CEE		Simple Span					
		16 Gauge		14 Gauge		12 Gauge	
Section	Bay	2 1/2" Flan	3 1/2" Flan	2 1/2" Flan	3 1/2" Flan	2 1/2" Flan	3 1/2" Flan
6" Web	10 ft	251	-	331	-	480	-
	12 ft	174	-	230	-	333	-
	14 ft	128	-	169	-	244	-
	15 ft	111	-	147	-	213	-
	18 ft	77	-	102	-	148	-
	20 ft	62	-	82	-	120	-
	22 ft	51	-	68	-	99	-
	24 ft	43	-	57	-	83	-
	25 ft	40	-	53	-	76	-
	28 ft	32	-	42	-	61	-
8" Web	12 ft	260	269	341	365	493	545
	14 ft	191	198	250	268	362	400
	15 ft	166	172	218	233	315	349
	18 ft	115	119	151	162	219	242
	20 ft	93	97	122	131	177	196
	22 ft	77	80	101	108	146	162
	24 ft	65	67	85	91	123	136
	25 ft	60	62	78	84	113	125
	28 ft	47	49	62	67	90	100
	30 ft	41	43	54	58	78	87
10" Web	20 ft	115	-	168	173	243	266
	22 ft	95	-	139	143	200	220
	24 ft	80	-	116	120	168	185
	25 ft	74	-	107	111	155	170
	28 ft	59	-	85	88	124	136
	30 ft	51	-	74	77	108	118
	32 ft	45	-	65	67	94	104
	34 ft	40	-	58	60	84	92
	35 ft	37	-	54	56	79	87
	38 ft	32	-	46	48	67	73
12" Web	20 ft	-	-	185	206	293	344
	24 ft	-	-	128	143	203	239
	25 ft	-	-	118	132	187	220
	28 ft	-	-	94	105	149	175
	30 ft	-	-	82	91	130	153
	32 ft	-	-	72	80	114	134
	34 ft	-	-	64	71	101	119
	35 ft	-	-	60	67	95	112
	38 ft	-	-	51	57	81	95
	40 ft	-	-	46	51	73	88

ZEE		Simple Span						3 or More Spans, Std. Lap					
		16 Gauge		14 Gauge		12 Gauge		16 Gauge		14 Gauge		12 Gauge	
Section	Bay	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.
6" Web	10 ft	254	-	331	-	499	-	-	-	-	-	-	-
	12 ft	126	-	230	-	346	-	-	-	-	-	-	-
	14 ft	129	-	169	-	254	-	-	-	-	-	-	-
	15 ft	113	-	147	-	221	-	176	-	235	-	357	-
	18 ft	78	-	102	-	154	-	118	-	157	-	238	-
	20 ft	63	-	82	-	124	-	94	-	124	-	188	-
	22 ft	52	-	68	-	103	-	76	-	101	-	153	-
	24 ft	44	-	57	-	86	-	63	-	84	-	126	-
	25 ft	40	-	53	-	79	-	58	-	76	-	116	-
	28 ft	32	-	42	-	63	-	46	-	60	-	91	-
8" Web	12 ft	260	265	340	364	510	545	-	-	-	-	-	-
	14 ft	191	195	250	267	374	401	-	-	-	-	-	-
	15 ft	166	169	218	233	326	349	-	-	-	-	-	-
	18 ft	115	117	151	161	226	242	156	158	222	235	345	368
	20 ft	93	95	122	131	183	196	127	129	178	188	275	293
	22 ft	77	78	101	108	151	162	105	107	145	154	223	238
	24 ft	65	66	85	91	127	136	88	90	121	129	185	198
	25 ft	59	61	78	83	117	125	81	83	111	118	170	181
	28 ft	47	48	62	66	93	100	65	66	87	93	133	142
	30 ft	41	42	54	58	81	87	56	57	76	81	115	123
10" Web	20 ft	115	119	168	-	250	266	131	133	217	-	365	387
	24 ft	80	82	116	-	173	185	96	97	154	-	248	263
	25 ft	74	76	107	-	160	170	89	91	142	-	228	242
	28 ft	59	60	85	-	127	136	72	74	114	-	179	191
	30 ft	51	52	74	-	111	118	64	65	99	-	155	165
	32 ft	45	46	65	-	97	104	56	58	87	-	135	144
	35 ft	37	38	54	-	81	87	48	49	73	-	112	119
	38 ft	32	33	46	-	69	73	41	42	61	-	94	101
12" Web	20 ft	-	-	183	203	301	345	-	-	210	220	418	463
	24 ft	-	-	127	141	209	239	-	-	153	162	289	324
	25 ft	-	-	117	130	192	220	-	-	142	151	265	299
	28 ft	-	-	93	103	153	176	-	-	116	124	210	238
	30 ft	-	-	81	90	133	153	-	-	102	109	183	207
	32 ft	-	-	71	79	117	134	-	-	90	97	160	181
	35 ft	-	-	59	66	98	112	-	-	76	82	133	151
	37 ft	-	-	53	59	88	100	-	-	68	74	119	135
	40 ft	-	-	45	50	75	86	-	-	59	64	101	115

Notes: 1.The weight of the section has not been subtracted from these values. 2. Both flanges of member must be fully braced. 3. These loads are based on the transfer of the support loads directly to the web of the section by the use of clips or plates. For flanges bearing directly on structural, contact factory for section selection. 4. See back page for weights per lineal foot of memebers shown here. 5. Loads shown are stress governing. When deflection limits are specified, contact factory. 6. These sample calculations are very basic. Many different variables can affect loading. For instance; drift loading, building height, geographic location, etc. Please consult Flexospan if special conditions exist. 7. The selection of sections for your application is subject to final approval by your design professional. 8. Capacity values have been calculated in accordance with the AISI 2001 design manual. 9. Values shown in the load tables for three or more spans are based on uniform bay spacings. If non-uniform bay spacings exist, contact factory. UNCONTROLLED COPY

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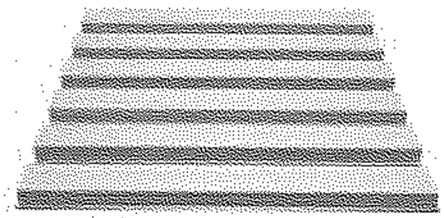
VERCO DECKING, INC.
a NUCOR company

ROOF DECK

PLB™-36 or HSB®-36



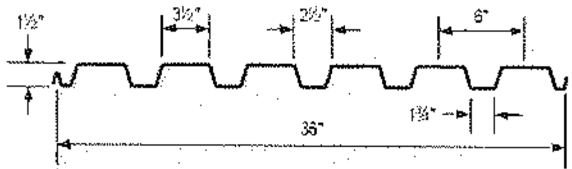
The difference between the PLB™-36 profile and HSB®-36 profile is the method of sidelap attachment; the panels themselves are identical in both geometry and material properties. The prefix "PL" designates a PunchLok® II version of the B profile, while "HS" (high shear) indicates a top seam weld, button punch, or screw version of the same profile.

Type B profiles are 1.5-inch deep structural roof deck that provide both vertical load and diaphragm shear capacity. The profile contains 6 ribs and is 36 inches wide with male and female edges, creating an interlocking side lap when installed. The wide ribs make the profile an ideal structural substrate to uniformly support roofing systems applied on top of the deck. Type B profiles are typically used for span conditions of 10 feet or less.

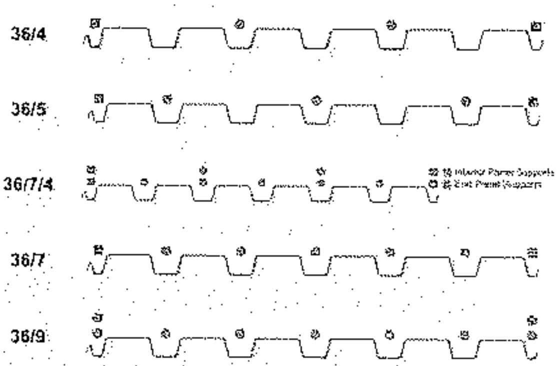
Extensive full scale diaphragm testing is an ongoing effort with B deck to produce a more efficient roof diaphragm in terms of capacity and installation. The current industry use of mechanical fasteners (Hilti and Pneutek), restraining elements (ShearTranz® Systems) and the innovative PunchLok® II side lap attachment system are all direct results of testing.

CAD DRAWINGS

Profile Dimensions



Attachment Patterns to Supports



Section Properties

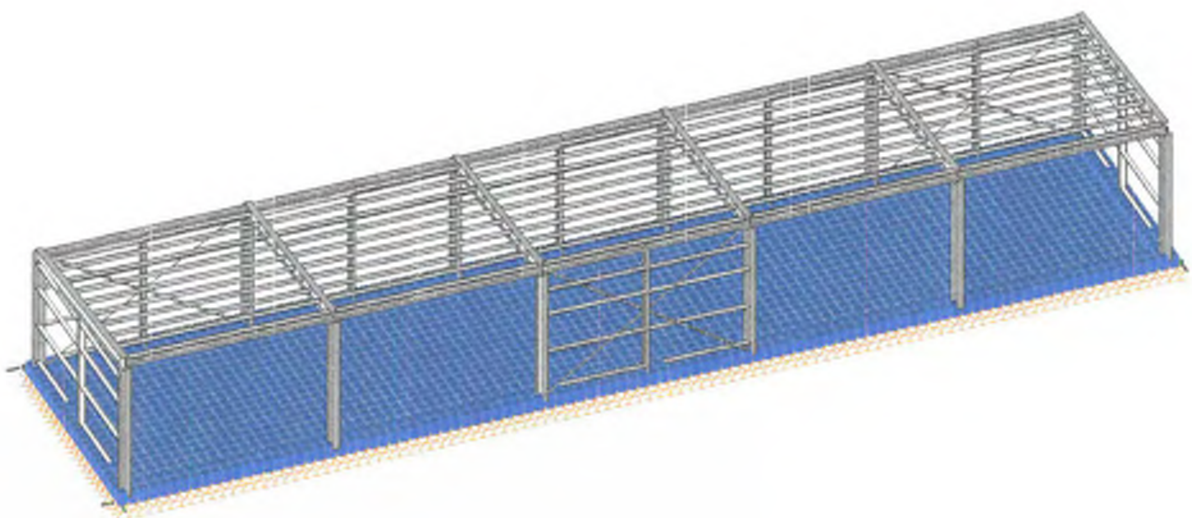
Deck Gage	Deck Weight Galv G60	Deck Weight Phos/ Painted	I_d for Deflection Single Span	I_d for Deflection Multiple Spans	Moment $+S_{eff}$	Moment $-S_{eff}$
	(psf)	(psf)	(in^4/ft)	(in^4/ft)	(in^3/ft)	(in^3/ft)
22	1.9	1.8	0.177	0.192	0.176	0.188
20	2.3	2.2	0.219	0.231	0.230	0.237
18	2.9	2.8	0.302	0.306	0.314	0.331
16	3.5	3.4	0.381	0.381	0.399	0.410

NOTE: Section properties based on F_y = 50 ksi.

Allowable Reactions

Deck Gage	End Bearing 2"	End Bearing 3"	End Bearing 4"	Interior Bearing 3"	Interior Bearing 4"
22	935	1076	1163	1559	1671
20	1301	1492	1609	2190	2340
18	2181	2484	2667	3714	3950
16	3265	3699	3955	5607	5938

NOTE: Allowable reactions are in pounds per foot of deck width and are based on F_y = 50 ksi.



SMG ENGINEERS	EQUIPMENT STORAGE BUILDING	SK -
BS		Sept 28, 2018 at 5:46 PM
18-183B		EQUIP STORAGE BLDG Rev_0 9...

Basic Load Cases

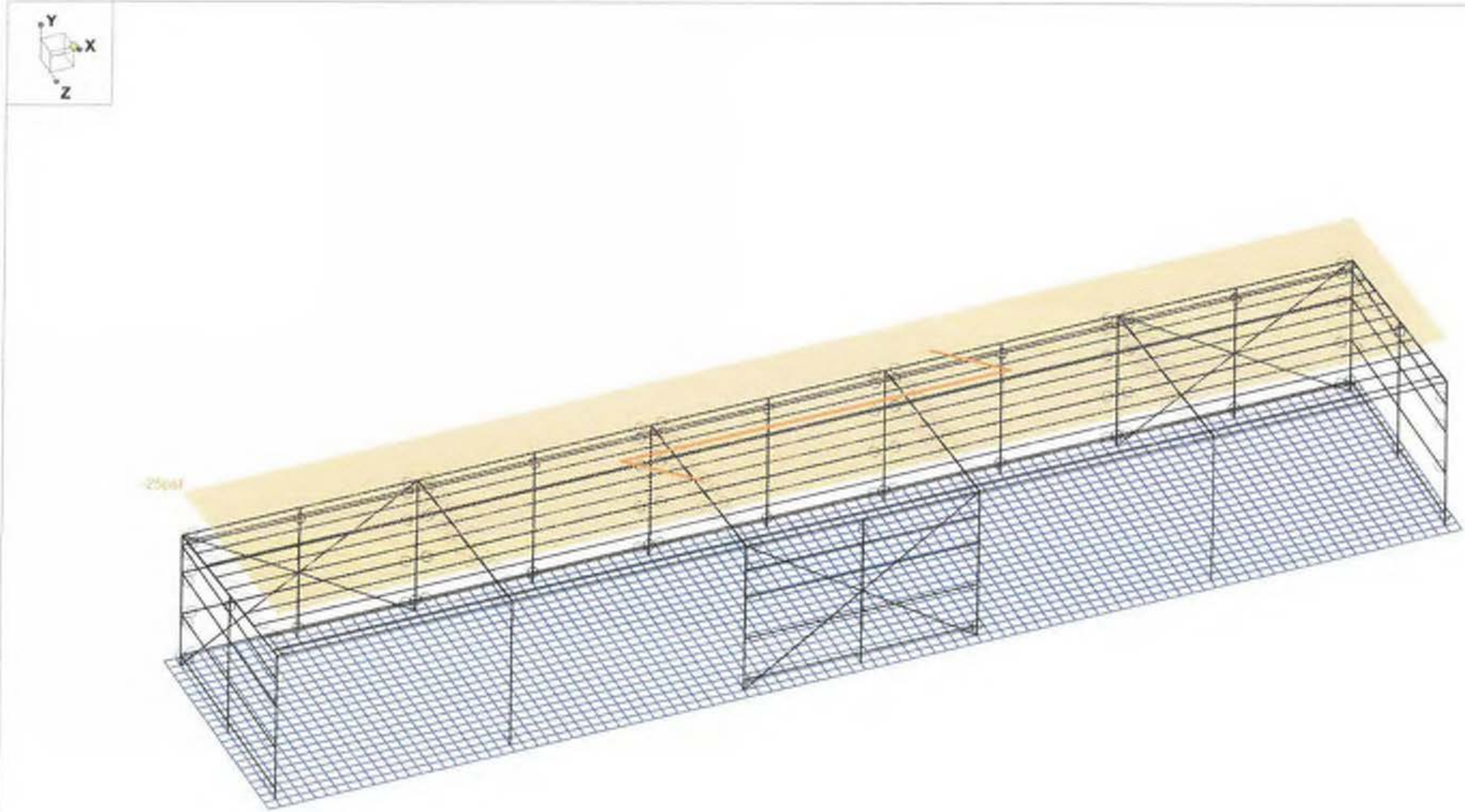
	BLC Description	Category	X Gr...	Y Gr...	Z Gra...	Joint	Point	Distri...	Area(...	Surfa...
1	DEAD LOAD	None		-1.05					1	
3	ROOF SNOW LOAD	None							1	
5	WIND TRANS - WINDWARD + GCpi	None							5	
6	-GCpi	None							5	
7	WIND TRANS - LEEWARD + GCpi	None							5	
8	-GCpi	None							5	
9	WIND LONG + GCpi	None							5	
10	-GCpi	None							5	
16	SEISMIC LONG	ELX	-.27							
17	SEISMIC TRANSVERSE	ELZ			-.25					

Load Combinations

	Description	S...	PDelta	S...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
1	DEAD		Y		1	1																
2	LIVE		Y		2	1																
3	SNOW LOAD		Y		3	1																
4	WIND TRANS1 (WINDWAR...		Y		5	1																
5	WIND TRANS1 (WINDWAR...		Y		6	1																
6	WIND TRANS2 (LEEWARD ...		Y		7	1																
7	WIND TRANS2 (LEEWARD -...		Y		8	1																
8	WIND LONG + GCpi)		Y		9	1																
9	WIND LONG - GCpi)		Y		10	1																
10	SEISMIC - LONG		Y		16	1																
11	SEISMIC - TRANS		Y		17	1																
12																						
13	ASD LOAD COMBINATIONS																					
14	D + L	Yes	Y		1	1	2	1														
15	D + S	Yes	Y		1	1	3	1														
16	D + 0.75L + 0.75S	Yes	Y		1	1	2	.75	3	.75												
17	D + 0.6W TRANS1 (WINDW...	Yes	Y		1	1	5	.6														
18	-GCpi	Yes	Y		1	1	6	.6														
19	D + 0.6W TRANS2 (LEEWA...	Yes	Y		1	1	7	.6														
20	-GCpi	Yes	Y		1	1	8	.6														
21	D + 0.6W (LONG)	Yes	Y		1	1	9	.6														
22	-GCpi	Yes	Y		1	1	10	.6														
23	D + 0.6W (75% TRANS1 + 7...	Yes	Y		1	1	5	.45	9	.45												
24	-GCpi	Yes	Y		1	1	6	.45	10	.45												
25	D + 0.6W (75% TRANS2 + 7...	Yes	Y		1	1	7	.45	9	.45												
26	-GCpi	Yes	Y		1	1	8	.45	10	.45												
27	D + 0.75L + 0.75(0.6W[T1+]) ...	Yes	Y		1	1	2	.75	3	.75	5	.45										
28	D + 0.75L + 0.75(0.6W[T1-]) ...	Yes	Y		1	1	2	.75	3	.75	6	.45										
29	D + 0.75L + 0.75(0.6W[T2+]) ...	Yes	Y		1	1	2	.75	3	.75	7	.45										
30	D + 0.75L + 0.75(0.6W[T2-]) ...	Yes	Y		1	1	2	.75	3	.75	8	.45										
31	D + 0.75L + 0.75(0.6W[L+]) +...	Yes	Y		1	1	2	.75	3	.75	9	.45										
32	D + 0.75L + 0.75(0.6W[L-]) + ...	Yes	Y		1	1	2	.75	3	.75	10	.45										
33	D + 0.75L + 0.75(0.6W[0.75T...	Yes	Y		1	1	2	.75	3	.75	5	.338	9	.338								
34	D + 0.75L + 0.75(0.6W[0.75T...	Yes	Y		1	1	2	.75	3	.75	6	.338	10	.338								
35	D + 0.75L + 0.75(0.6W[0.75T...	Yes	Y		1	1	2	.75	3	.75	7	.338	9	.338								
36	D + 0.75L + 0.75(0.6W[0.75T...	Yes	Y		1	1	2	.75	3	.75	8	.338	10	.338								
37	D + 0.7Ex (LONG)	Yes	Y		1	1	16	.7														
38	D + 0.7Ez (TRANS)	Yes	Y		1	1	17	.7														
39	D - 0.7Ez (TRANS)	Yes	Y		1	1	17	-.7														
40	D + 0.75L + 0.75(0.7Ex) + 0....	Yes	Y		1	1	2	.75	16	.525	3	.75										
41	D + 0.75L - 0.75(0.7Ez) + 0.7...	Yes	Y		1	1	2	.75	17	.525	3	.75										
42		Yes	Y		1	1	2	.75	17	-.5...	3	.75										
43																						
44	LRFD LOAD COMBINATIONS																					
45	1.4D		Y		1	1.4																
46	1.2D + 1.6L + 0.5S		Y		1	1.2	2	1.6	3	.5												
47	1.2D + 1.6S + 0.5W (DOES ...		Y																			
48	*1.2D + L+ 0.5S		Y		1	1.2	2	1	3	.5												
49	1.2D + 1.0W (TRANS1) + L +...		Y		L48	1	5															
50	-GCpi		Y		L48	1	6															
51	TRANS2		Y		L48	1	7															
52	-GCpi		Y		L48	1	8															
53	LONG		Y		L48	1	9															
54	-GCpi		Y		L48	1	10															
55	COMB1		Y		L48	1	5	.75	9	.75												
56	-GCpi		Y		L48	1	6	.75	10	.75												

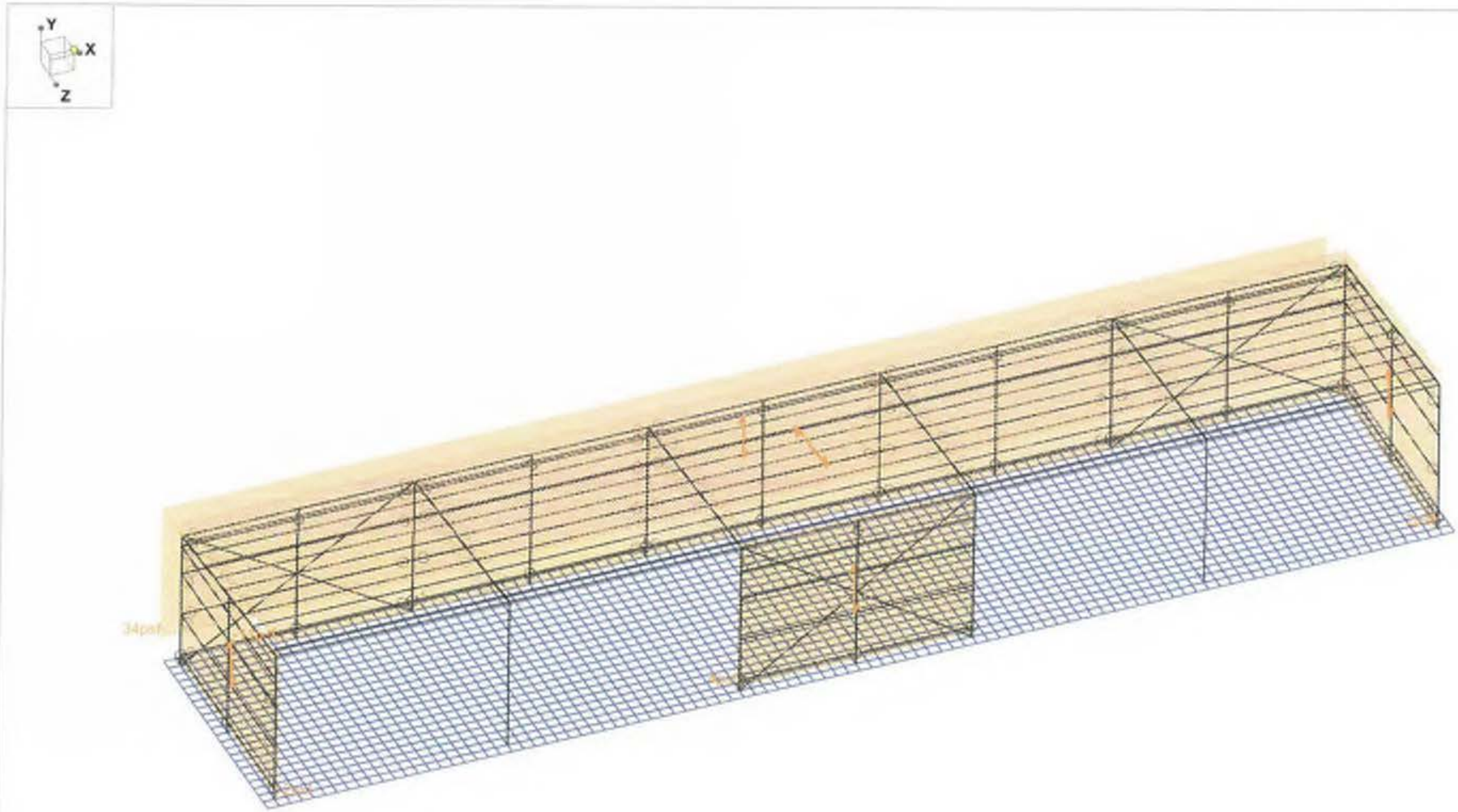
Load Combinations (Continued)

	Description	S...	PDelta	S...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
57	COMB2		Y		L48	1	7	.75	9	.75														
58	-GCpi		Y		L48	1	8	.75	10	.75														
59	1.2D + 1.0Ex + L + 0.2S		Y		1	1.2	E...	1	2	1	3	.2												
60	1.2D +1.0Ez + L + 0.2S		Y		1	1.2	E...	1	2	1	3	.2												
61	1.2D - 1.0Ez + L + 0.2S		Y		1	1.2	E...	-1	2	1	3	.2												



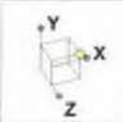
Loads: BLC 3, ROOF SNOW LOAD

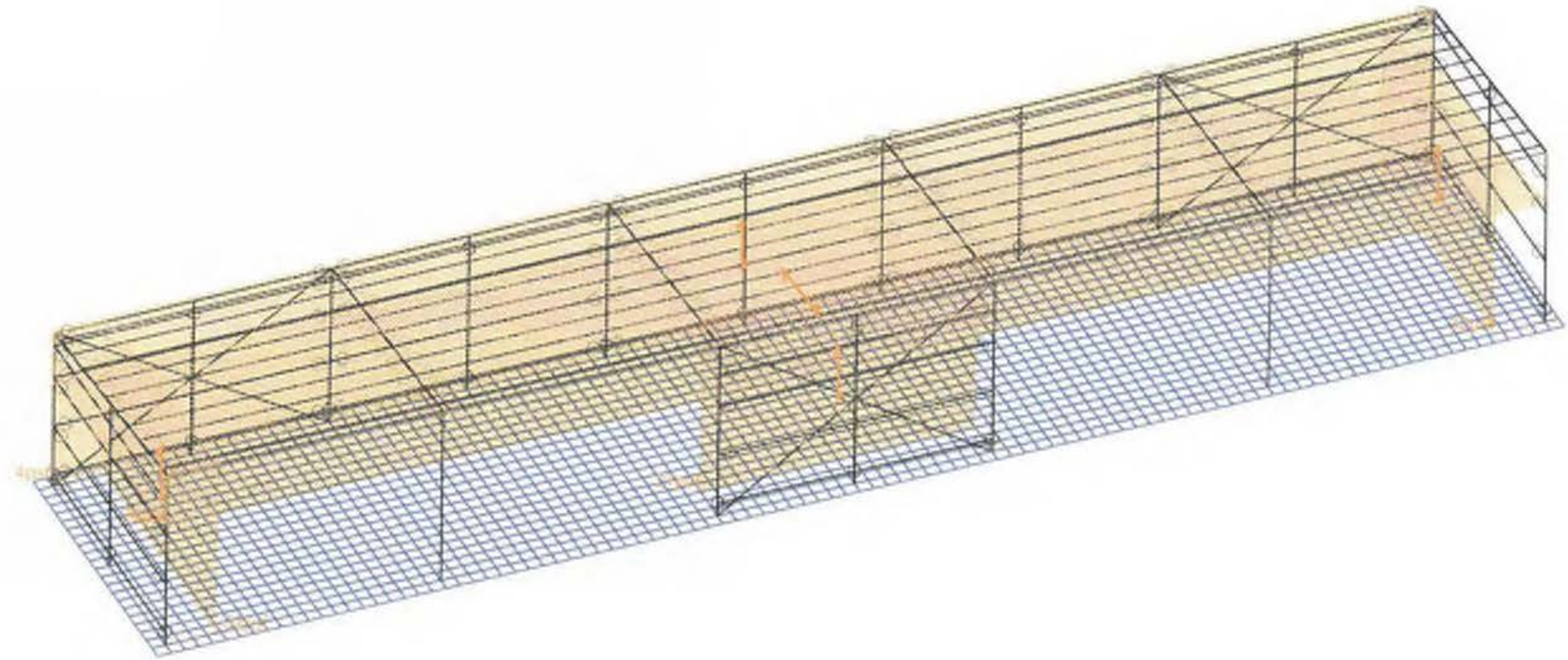
SMG ENGINEERS	EQUIPMENT STORAGE BUILDING	SK -
BS		Sept 28, 2018 at 5:53 PM
18-183B		EQUIP STORAGE BLDG Rev_0 9.21.18.r3d



Loads: BLC 5, WIND TRANS - WINDWARD + GCpi

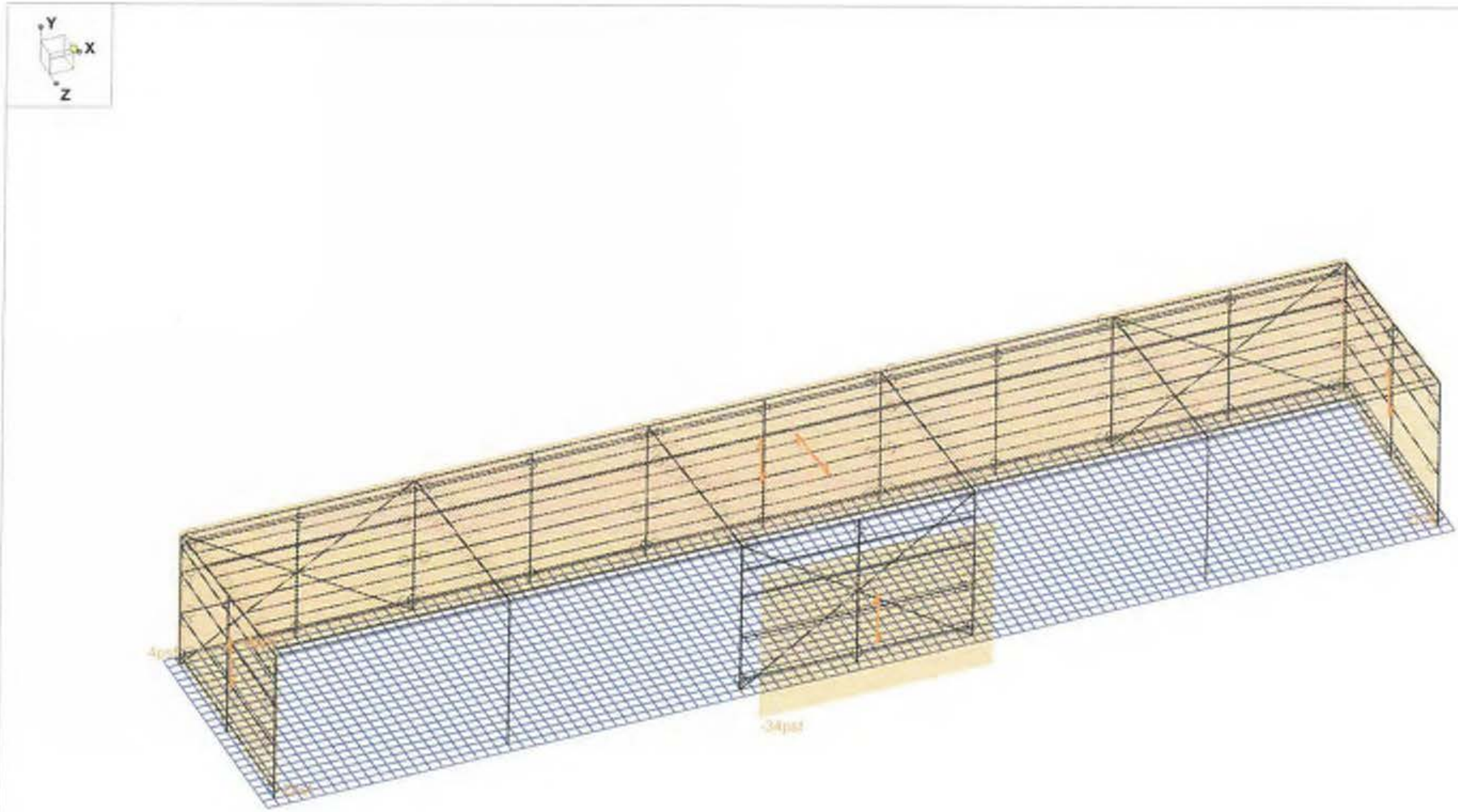
SMG ENGINEERS	EQUIPMENT STORAGE BUILDING	SK -
BS		Sept 28, 2018 at 5:53 PM
18-183B		EQUIP STORAGE BLDG Rev_0 9.21.18.r3d





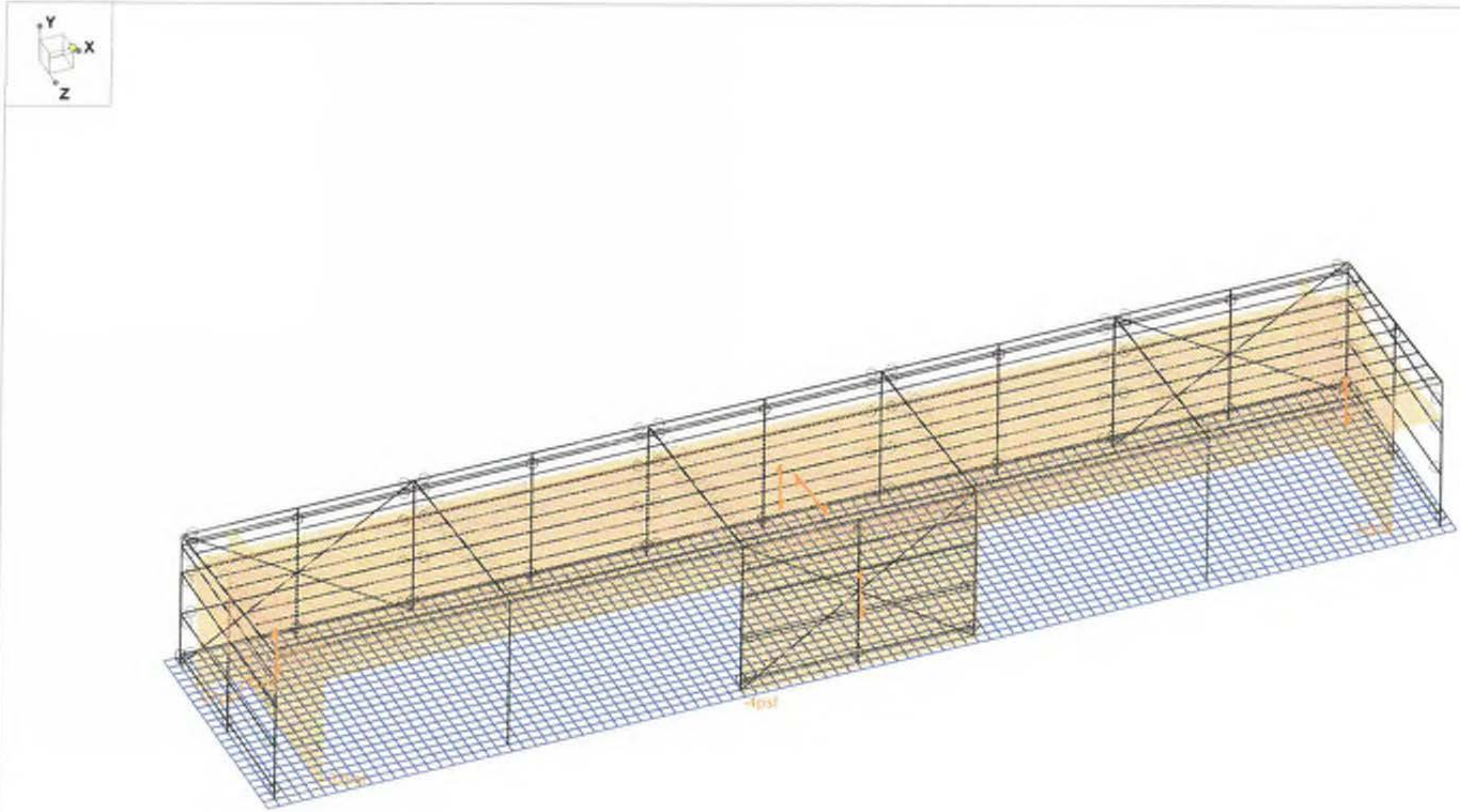
Loads: BLC 6, -GCpi

SMG ENGINEERS	EQUIPMENT STORAGE BUILDING	SK -
BS		Sept 28, 2018 at 5:54 PM
18-183B		EQUIP STORAGE BLDG Rev_0 9.21.18.r3d



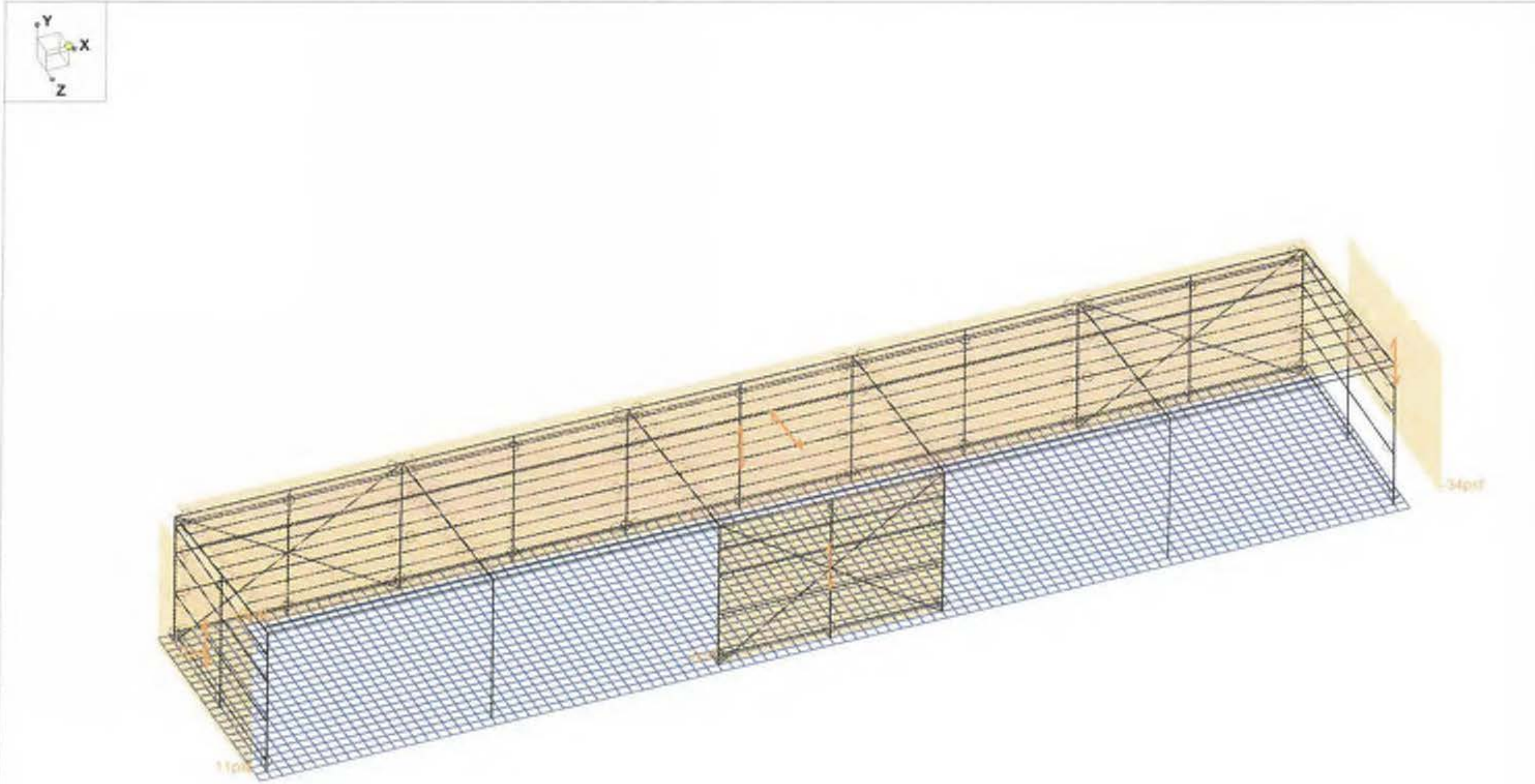
Loads: BLC 7, WIND TRANS - LEEWARD + GCpi

SMG ENGINEERS	EQUIPMENT STORAGE BUILDING	SK -
BS		Sept 28, 2018 at 5:54 PM
18-183B		EQUIP STORAGE BLDG Rev_0 9.21.18.r3d



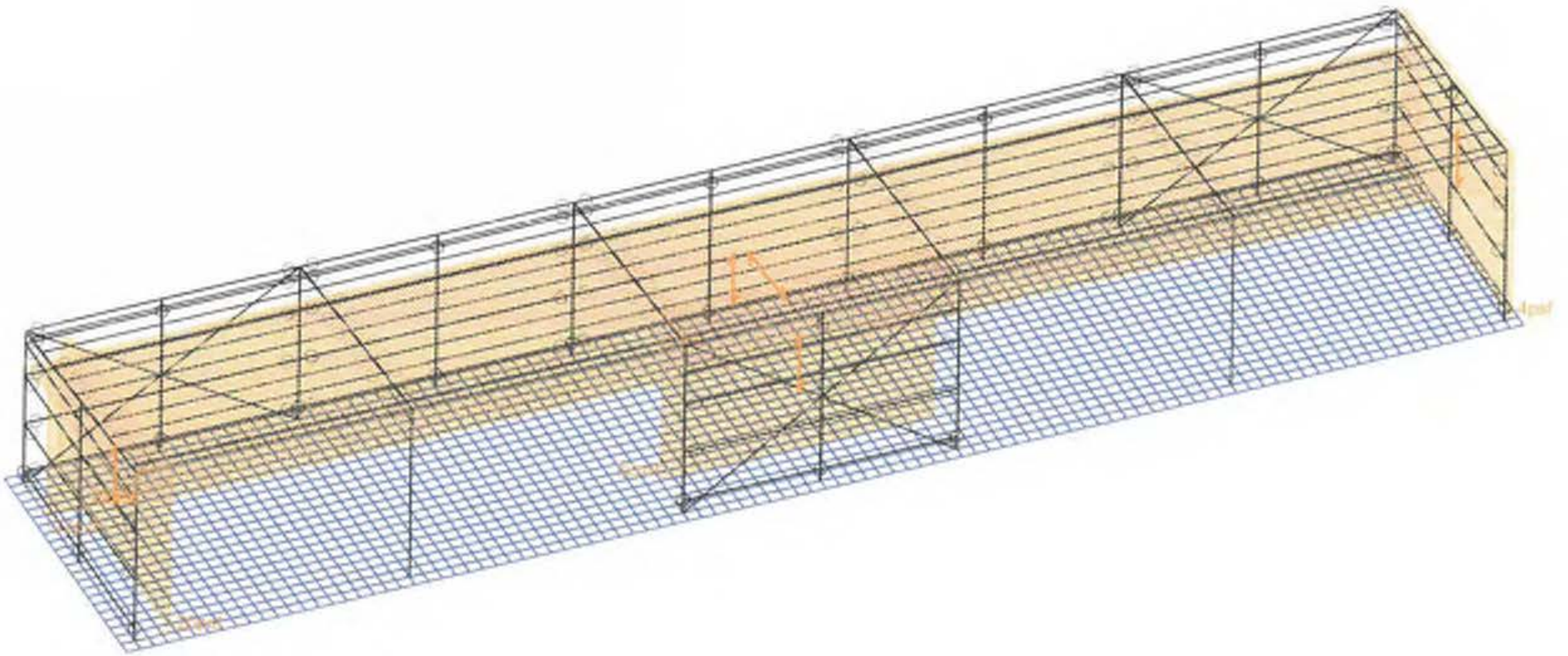
Loads: BLC 8, -GCpi

SMG ENGINEERS	EQUIPMENT STORAGE BUILDING	SK -
BS		Sept 28, 2018 at 5:54 PM
18-183B		EQUIP STORAGE BLDG Rev_0 9.21.18.r3d



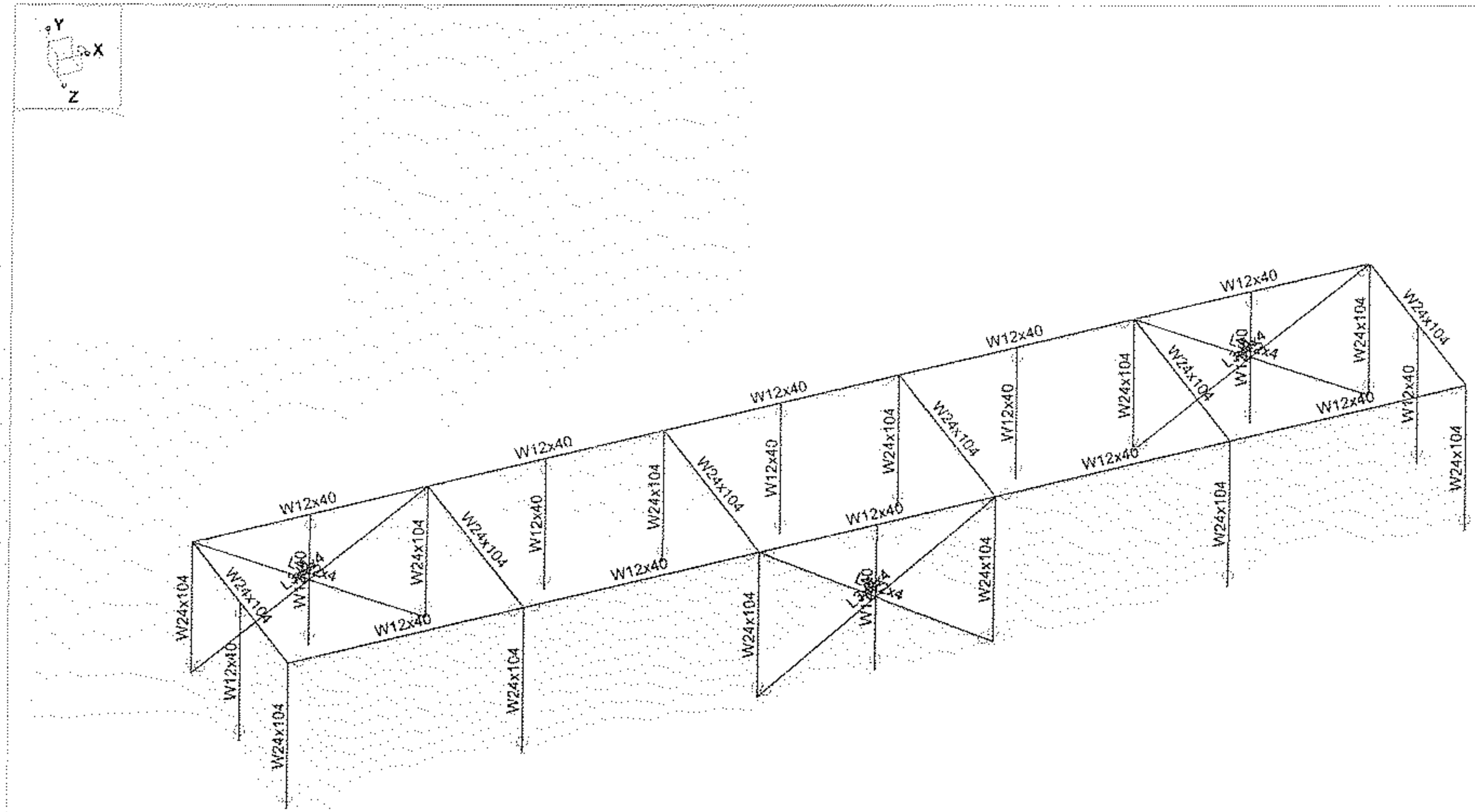
Loads: BLC 9, WIND LONG + GCpi

SMG ENGINEERS	EQUIPMENT STORAGE BUILDING	SK -
BS		Sept 28, 2018 at 5:54 PM
18-183B		EQUIP STORAGE BLDG Rev_0 9.21.18.r3d

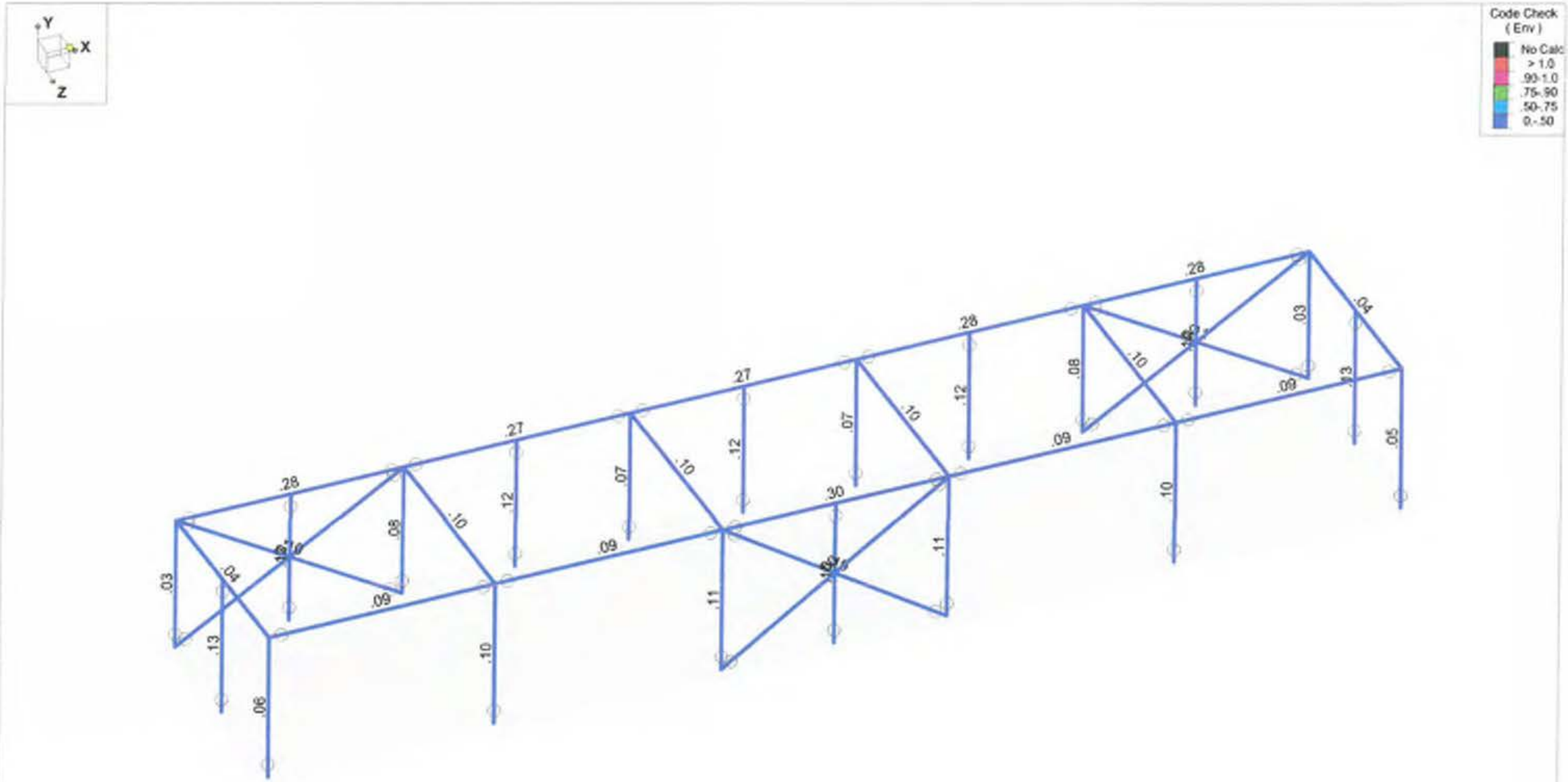


Loads: BLC 10, -GCpi

SMG ENGINEERS	EQUIPMENT STORAGE BUILDING	SK -
BS		Sept 28, 2018 at 5:55 PM
18-183B		EQUIP STORAGE BLDG Rev_0 9.21.18.r3d



SMG ENGINEERS	EQUIPMENT STORAGE BUILDING	SK -
BS		Sept 28, 2018 at 5:47 PM
18-183B		EQUIP STORAGE BLDG Rev_0 9.21.18.r3d



Member Code Checks Displayed (Enveloped)
Envelope Only Solution

SMG ENGINEERS	EQUIPMENT STORAGE BUILDING	SK -
BS		Sept 28, 2018 at 5:50 PM
18-183B		EQUIP STORAGE BLDG Rev_0 9.21.18.r3d

SHEET C18

Envelope AISC 14th(360-10): LRFD Steel Code Checks

	Memb...	Shape	Code Check	Loc[ft]	LC	Sh...	Loc[ft]	Dir	LC	phi*...	phi*Pn...	phi*Mn...	phi*Mn z...	Cb	Eqn
1	M26	W12x40	.300	15	24	.023	30	y	24	76.8...	526.5	63	109.609	1.283	H1...
2	M23	W12x40	.277	15	26	.020	30	y	26	76.8...	526.5	63	111.109	1.3	H1...
3	M19	W12x40	.276	15	26	.020	0	y	26	76.8...	526.5	63	111.108	1.3	H1...
4	M22	W12x40	.275	15	26	.020	0	y	26	76.8...	526.5	63	111.109	1.3	H1...
5	M20	W12x40	.275	15	26	.020	30	y	26	76.8...	526.5	63	111.11	1.3	H1...
6	M21	W12x40	.275	15	26	.020	0	y	26	76.8...	526.5	63	111.107	1.3	H1...
7	M29	L3x2x4	.195	0	37	.000	0	z	35	1.366	38.88	.826	1.291	1	H2...
8	M124	W12x40	.146	7.656	24	.044	0	y	24	156....	526.5	63	156.941	1.153	H1...
9	M123	W12x40	.130	7.5	21	.035	0	y	21	172....	526.5	63	164.766	1.164	H1...
10	M119	W12x40	.130	7.5	26	.040	0	y	24	172....	526.5	63	164.679	1.163	H1...
11	M125	W12x40	.120	7.521	26	.041	0	y	26	191....	526.5	63	174.578	1.187	H1...
12	M126	W12x40	.119	7.521	26	.041	0	y	26	191....	526.5	63	174.575	1.187	H1...
13	M120	W12x40	.118	7.521	26	.041	0	y	26	191....	526.5	63	174.578	1.187	H1...
14	M121	W12x40	.118	7.521	26	.041	0	y	26	191....	526.5	63	174.572	1.187	H1...
15	M122	W12x40	.117	7.521	26	.041	0	y	26	191....	526.5	63	174.575	1.187	H1...
16	M33	L3x2x4	.110	0	37	.000	0	y	27	1.366	38.88	.826	1.291	1	H2...
17	M7	W24x104	.106	21	27	.014	0	y	27	796....	1381.5	234	1083.75	1.656	H1...
18	M5	W24x104	.106	21	27	.014	0	y	27	796....	1381.5	234	1083.75	1.656	H1...
19	M9	W24x104	.104	21	27	.013	0	y	27	796....	1381.5	234	1083.75	1.667	H1...
20	M3	W24x104	.104	21	27	.013	0	y	27	796....	1381.5	234	1083.75	1.667	H1...
21	M31	L3x2x4	.101	0	37	.000	0	z	33	1.366	38.88	.826	1.291	1	H2...
22	M16	W24x104	.098	0	27	.052	0	y	27	449....	1381.5	234	1083.75	1.764	H1...
23	M15	W24x104	.098	0	27	.052	0	y	27	449....	1381.5	234	1083.75	1.764	H1...
24	M17	W24x104	.095	0	27	.051	0	y	27	449....	1381.5	234	1083.75	1.739	H1...
25	M14	W24x104	.095	0	27	.051	0	y	27	449....	1381.5	234	1083.75	1.739	H1...
26	M27	W12x40	.093	15	23	.003	0	z	23	76.8...	526.5	63	85.463	1	H1...
27	M28	W12x40	.092	15	23	.003	0	z	33	76.8...	526.5	63	85.463	1	H1...
28	M25	W12x40	.087	15	23	.003	0	z	23	76.8...	526.5	63	85.463	1	H1...
29	M24	W12x40	.087	15	23	.003	0	z	33	76.8...	526.5	63	85.463	1	H1...
30	M10	W24x104	.077	19	36	.018	0	y	36	878....	1381.5	234	1083.75	1.425	H1...
31	M4	W24x104	.077	19	36	.018	0	y	36	878....	1381.5	234	1083.75	1.425	H1...
32	M8	W24x104	.071	19	29	.016	0	y	36	878....	1381.5	234	1083.75	1.7	H1...
33	M6	W24x104	.071	19	29	.016	0	y	36	878....	1381.5	234	1083.75	1.7	H1...
34	M1	W24x104	.061	13.5...	26	.008	0	y	26	796....	1381.5	234	1083.75	1.428	H1...
35	M11	W24x104	.046	13.5...	26	.008	0	y	26	796....	1381.5	234	1083.75	1.425	H1...
36	M18	W24x104	.040	15.0...	21	.020	15.033	y	27	449....	1381.5	234	1083.75	1.72	H1...
37	M13	W24x104	.038	0	26	.020	15.033	y	27	449....	1381.5	234	1083.75	2.06	H1...
38	M12	W24x104	.032	19	26	.008	17.615	y	17	878....	1381.5	234	1083.75	1.496	H1...
39	M2	W24x104	.032	19	26	.008	17.615	y	17	878....	1381.5	234	1083.75	1.497	H1...
40	M32	L3x2x4	.011	0	18	.000	0	z	27	1.366	38.88	.826	1.291	1	H2...
41	M30	L3x2x4	.001	0	18	.000	0	z	29	1.366	38.88	.826	1.291	1	H2...
42	M34	L3x2x4	.000	0	14	.000	0	z	27	1.366	38.88	.826	1.291	1	H1...

Equipment Storage: Member & Connection Design

Main Column: W24x104

Demand (DR = 0.11 Per Risa, OK ✓)

Axial, P_{max} = 23 kips (compression) 9 kips (tension)

Moment, $M_{x,max}$ = 102 k-ft
$M_{y,max}$ = 8 k-ft

Shear, $V_{y,max}$ = 5 kips
$V_{x,max}$ = 2 kips

Defl, Δy = 1/2" → L/500
Δx = 3/16" → L/1300

Base Plate Design

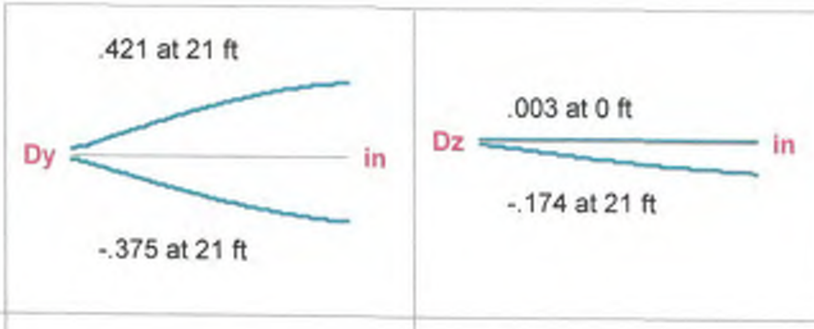
Max Compression = 23 kips
Max Tension = 9 kips

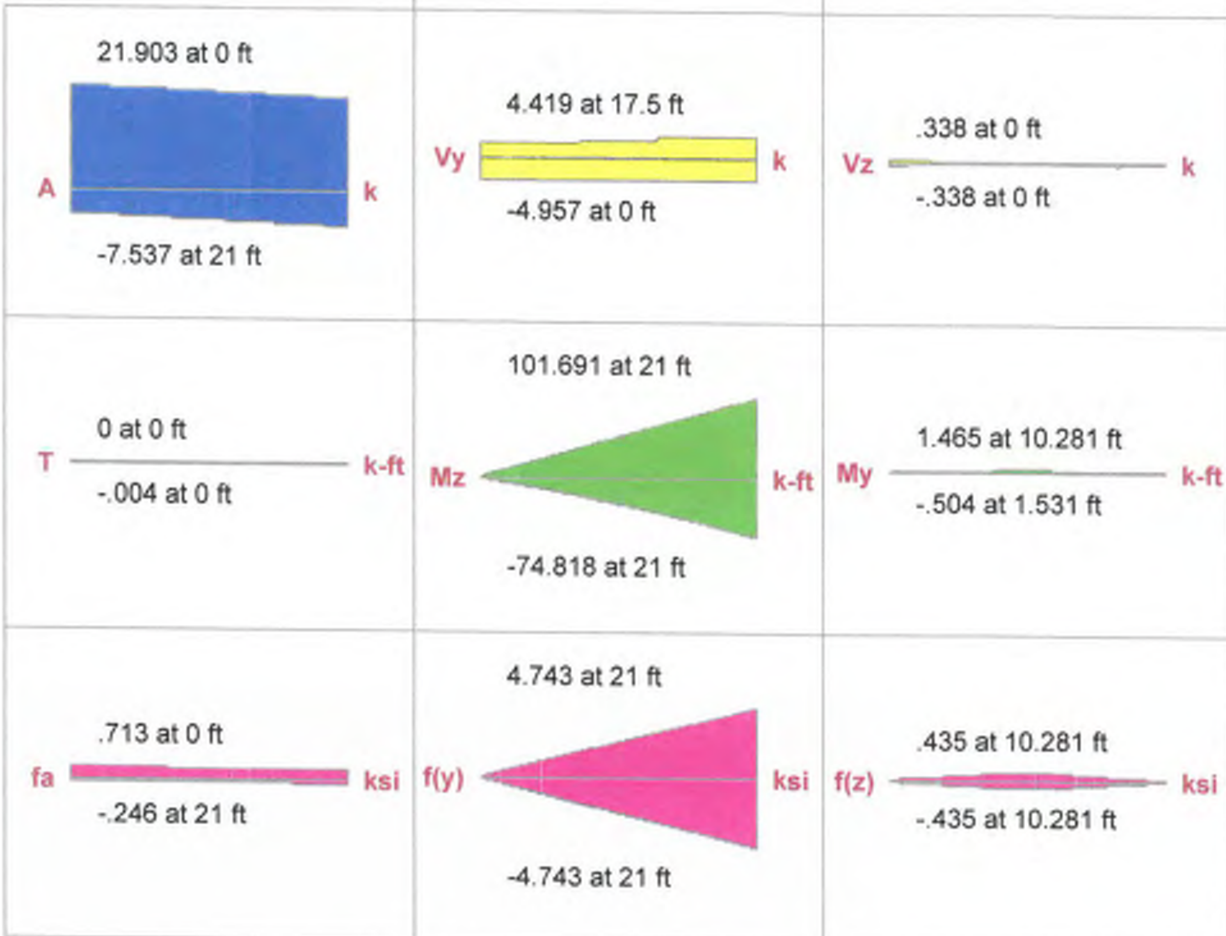
V_x max = 5 kips
V_y max = 2 kips

1" Base Plate w/ (6) 1"Ø Anchors OK By Inspection

Column: *M7*

Shape: **W24x104**
Material: **A572 Gr.50**
Length: **21 ft**
I Joint: **N4**
J Joint: **N22**
Envelope
Code Check: **0.106 (LC 27)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	0.106 (LC 27)	Max Shear Check	0.014 (y) (LC 27)
Location	21 ft	Location	0 ft
Equation	H1-1b	Max Defl Ratio	L/2445

Bending Flange	Compact	Compression Flange	Non-Slender	Qs=1
Bending Web	Compact	Compression Web	Slender	Qa=1

Fy	50 ksi		y-y	z-z
phi*Pnc	796.756 k	Lb	21 ft	21 ft
phi*Pnt	1381.5 k	KL/r	86.76	25.078
phi*Mny	234 k-ft	L Comp Flange	21 ft	
phi*Mnz	1083.75 k-ft	L-torque	21 ft	
phi*Vny	361.5 k	Tau_b	1	
phi*Vnz	518.4 k			
Cb	1.656			

EQUIPMENT STORAGE: MEMBER & CXN DESIGN (CONT'D)

MAIN BEAM: W24x104

DEMAND (DR = 0.10 PER RISA, OK ✓)

AXIAL, $P_{MAX} = 6^{KIPS}$
$T_{MAX} = 6^{KIPS}$

MOMENT, $M_{x\,MAX} = 102^{K\text{-}FT}$
$M_{y\,MAX} = 6^{K\text{-}FT}$

SHEAR, $V_{y}\ MAX = 19^{KIPS}$
$V_{x}\ MAX = 2^{KIPS}$

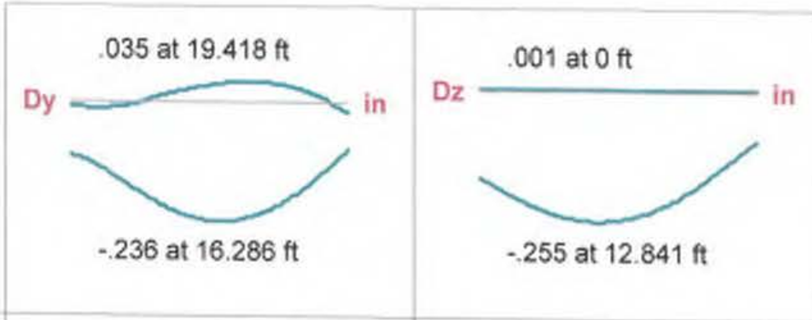
DEFL, $\Delta_y = 1/4''$ → L/1400
$\Delta_x = 1/4''$

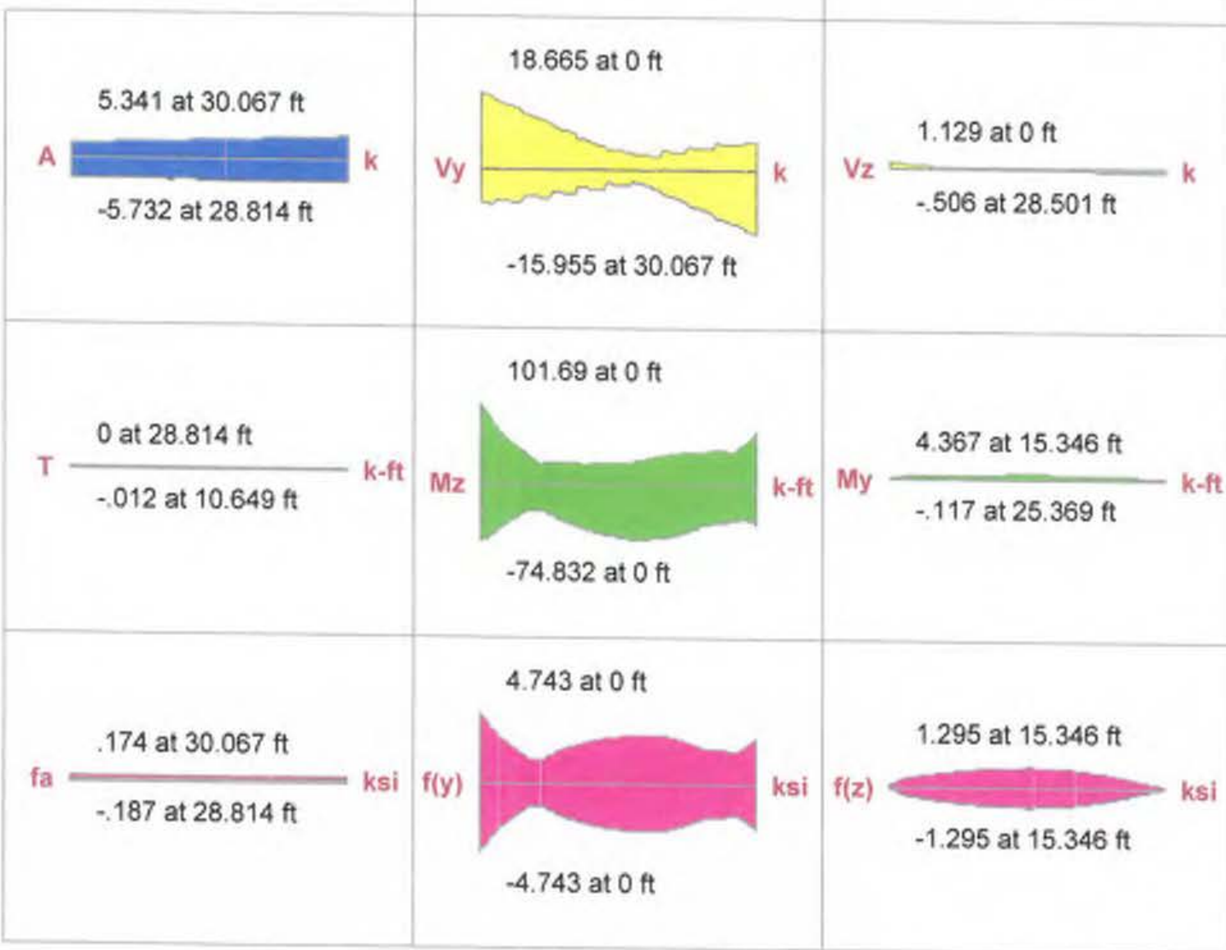
* FULL CSP CXN TO COLUMN OK BY INSPECTION

SEE BUILDING 1, 2, 3 CALCULATIONS FOR CJP W24 BEAM REQUIREMENTS (SIMILAR)

Beam: *M15*

Shape: W24x104
Material: A572 Gr.50
Length: 30.067 ft
I Joint: N21
J Joint: N15
Envelope
Code Check: 0.098 (LC 27)
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	0.098 (LC 2	Max Shear Check	0.052 (y) (LC 2	Max Defl Ratio	L/2575
Location	0 ft	Location	0 ft	Location	16.286 ft
Equation	H1-1b			Span	1

Bending Flange	Compact	Compression Flange	Non-Slender	Qs=1
Bending Web	Compact	Compression Web	Slender	Qa=1

			y-y	z-z
Fy	50 ksi	Lb	30.067 ft	30.067 ft
phi*Pnc	449.478 k	KL/r	124.218	35.905
phi*Pnt	1381.5 k			
phi*Mny	234 k-ft	L Comp Flange	30.067 ft	
phi*Mnz	1083.75 k-ft	L-torque	30.067 ft	
phi*Vny	361.5 k	Tau_b	1	
phi*Vnz	518.4 k			
Cb	1.764			

EQUIPMENT STORAGE: MEMBER & CONNECTION DESIGN

LONGITUDINAL BEAM: W12x40 (ROTATED 90°)

DEMAND (DR = 0.30 PER RISA, OK✓)

AXIAL, P_{MAX} = 2 KIPS
T_{MAX} = 3 KIPS

MOMENT, $M_{x\,MAX}$ = 30 K-FT
$M_{y\,MAX}$ = 6 K-FT

SHEAR, $V_{y\,MAX}$ = 3 KIPS
$V_{x\,MAX}$ = 1 KIPS

DEFL, Δ_y = < 1/8" OK✓
Δ_x = ~1" (GROSS), 1/2" (NET) → L/720

CONNECTION TO MAIN COLUMN (SHEAR ONLY)

DBL CLIP ANGLE w/ (3) ROWS OF 3/4"Ø BOLTS OK BY INSPECTION

X-BRACING: L3x2x1/4

MAX. TENSION = 7.6 KIPS DR = 0.20 < 1.0 OK✓

CONNECTION

USE (2) 3/4"Ø BOLTS, V_n/Ω = 11.9K x 2 = 23.8 KIPS OK✓

>> 7.6K x 2.0 = 15.2 KIPS

SEISMIC OVERSTRENGTH

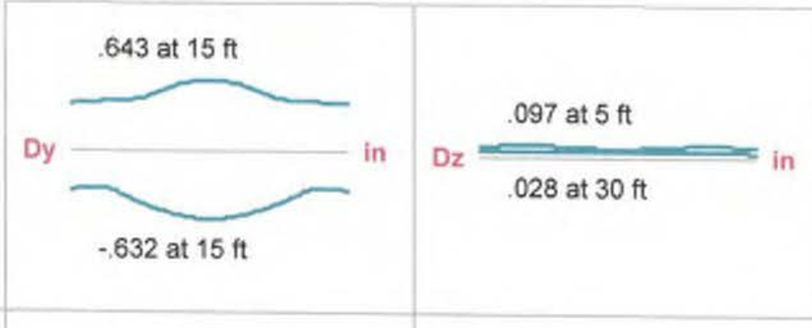
REQ'D 1/4" WELD LENGTH = 15.2 kip/3.71 k/in = 4.1"

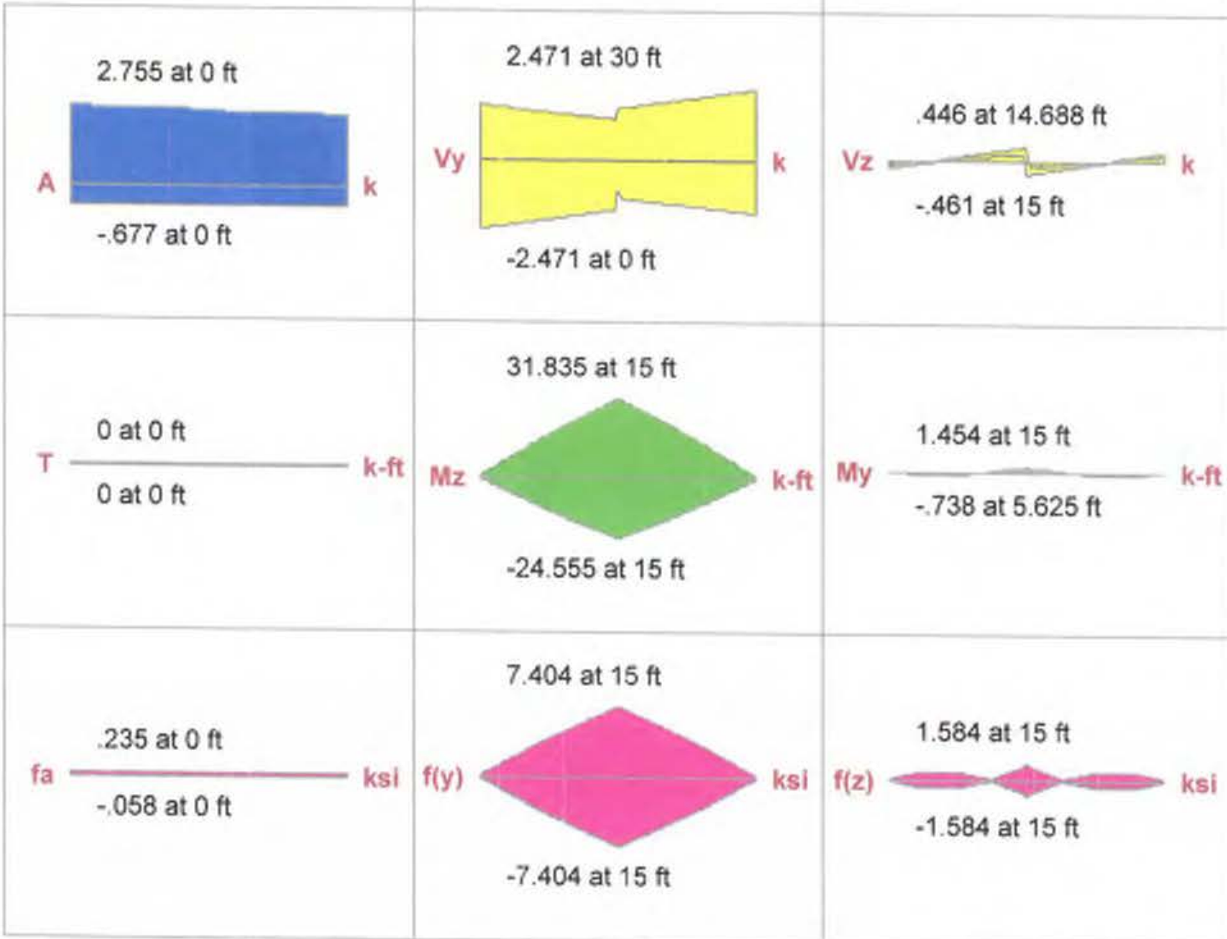
REQ'D 1/2" GUSSET = 15.2kip/(36ksi*0.5"*1"/1.67)= 1.41"

1/2" GUSSET PL & 1/4" WELD, OK BY INSP.

Beam: *M26*

Shape: **W12x40**
Material: **A572 Gr.50**
Length: **30 ft**
I Joint: **N21**
J Joint: **N22**
Envelope
Code Check: **0.300 (LC 24)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

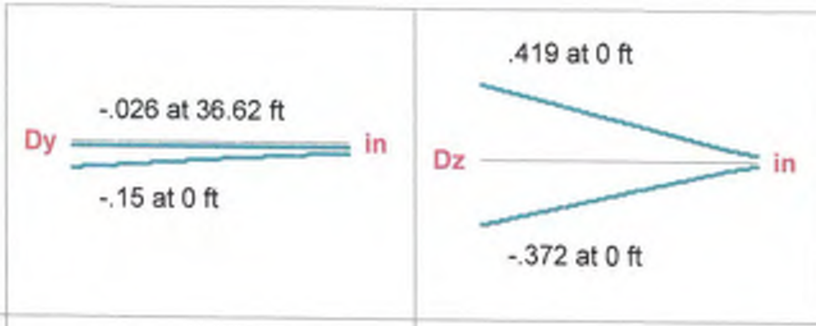
Max Bending Check	**0.300 (LC**	Max Shear Check	**0.023 (y) (LC 2**	Max Defl Ratio	**L/1517**
Location	**15 ft**	Location	**30 ft**	Location	**21.563 ft**
Equation	**H1-1b**			Span	**2**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=1**

			y-y	z-z
Fy	**50 ksi**	Lb	**30 ft**	**30 ft**
phi*Pnc	**76.873 k**	KL/r	**185.428**	**70.279**
phi*Pnt	**526.5 k**			
phi*Mny	**63 k-ft**	L Comp Flange	**30 ft**	
phi*Mnz	**109.609 k-ft**	L-torque	**30 ft**	
phi*Vny	**105.315 k**	Tau_b	**1**	
phi*Vnz	**222.758 k**			
Cb	**1.283**			

VBrace: *M29*

Shape: L3x2x4
Material: A36 WTLS
Length: 36.62 ft
I Joint: N21
J Joint: N4
Envelope
Code Check: 0.195 (LC 37)
Report Based On 97 Sections



AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	0.195 (LC 37)	Max Shear Check	0.000 (z) (LC 35)
Location	0 ft	Location	0 ft
Equation	H2-1*	Max Defl Ratio	L/10000
Bending Flange	Compact	Compression Flange	Non-Slender
Bending Web	Compact	Compression Web	Non-Slender

Fy	36 ksi		y-y'	z-z'
phi*Pnc	1.366 k	Lb	16 ft	16 ft
phi*Pnt	38.88 k	KL/r	445.476	187.59
phi*Mny'	.826 k-ft			
phi*Mnz'	1.291 k-ft	L Comp Flange	16 ft	
phi*Vny	14.58 k	L-torque	36.62 ft	
phi*Vnz	9.72 k	Tau_b	1	
Cb	1			

Smith Monroe Gray
ENGINEERS, INC.

CLIENT LAKESIDE INDUSTRIES, INC.
PROJECT MAPLE VALLEY ASPHALT PLANT
EQUIPMENT STORAGE BUILDING
BY BS DATE 4/8/2019 REV.
JOB NO. 18-183B SHEET OF

EQUIPMENT STORAGE: MEMBER & CONNECTION DESIGN

SECONDARY COLUMN: W12x40

DEMAND (DR = 0.15 PER RISA, OK ✓)

AXIAL, P_{MAX} = 14 KIPS
T_{MAX} = 1 KIP

MOMENT, M_{YMAX} = 23 K-FT
M_{yMAX} = —

SHEAR, V_{yMAX} = 5 KIPS
V_{xMAX} = —

DEFL, Δ_y = 0.665" → L/380

BASE PLATE DESIGN

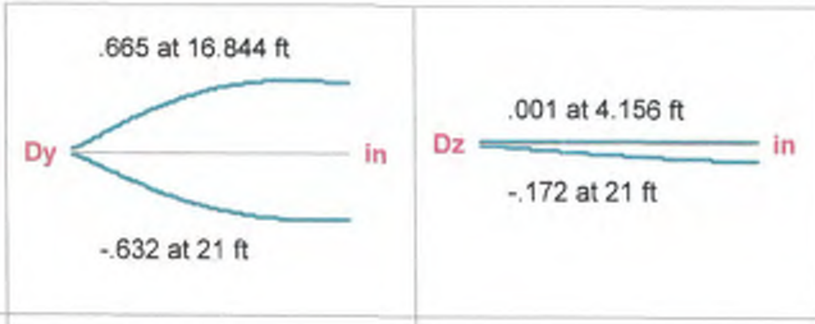
PL 3/4" w/ (4) 1"⌀ ANCHORS OK BY INSPECTION

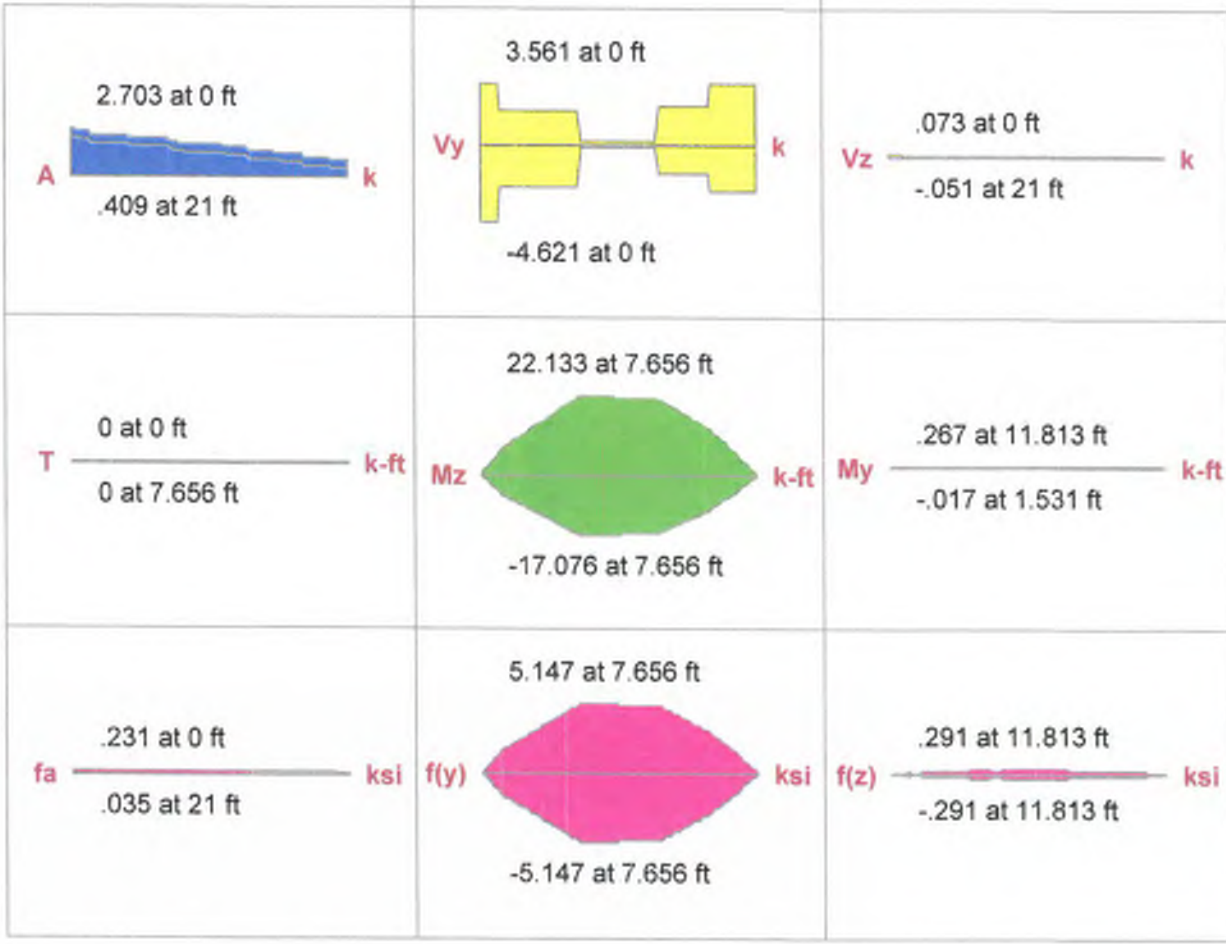
END CONNECTIONS

DBL CLIP ANGLE w/ (2) 3/4"⌀ BOLTS OK BY INSPECTION

Column: *M124*

Shape: W12x40
Material: A572 Gr.50
Length: 21 ft
I Joint: N123
J Joint: N2491
Envelope
Code Check: 0.146 (LC 24)
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	0.146 (LC 24)	Max Shear Check	0.044 (y) (LC 24)
Location	7.656 ft	Location	0 ft
Equation	H1-1b	Max Defl Ratio	L/987

Bending Flange	Compact	Compression Flange	Non-Slender	Qs=1
Bending Web	Compact	Compression Web	Slender	Qa=1

			y-y	z-z
Fy	50 ksi	Lb	21 ft	21 ft
phi*Pnc	156.884 k	KL/r	129.8	49.195
phi*Pnt	526.5 k			
phi*Mny	63 k-ft	L Comp Flange	21 ft	
phi*Mnz	156.941 k-ft	L-torque	21 ft	
phi*Vny	105.315 k	Tau_b	1	
phi*Vnz	222.758 k			
Cb	1.153			

Smith Monroe Gray
ENGINEERS, INC.

CLIENT
PROJECT

BY DATE REV.
JOB NO. SHEET OF

EQUIPMENT STORAGE FOUNDATION

MAX. BEARING REACTION = 1.1 KIPS (ASD)

BEARING AREA = 1.5' x 1.5' = 2.25 ft^2

BEARING PRESSURE = 1.1^K/2.25 ft^2 = 490 PSF << 3000 PSF ALLOWABLE

SLIDING CHECK

MAX. HORIZ. LOAD = 131 KIPS

DEAD LOAD = 770 KIPS

FRICTION RESISTANCE = 0.5 x 770 KIPS = 385 KIPS

FS = 385^K/131^K = 2.9 OK ✓

* NO UPLIFT BY INSPECTION

* NO OVERTURNING BY INSPECTION

Envelope Joint Reactions

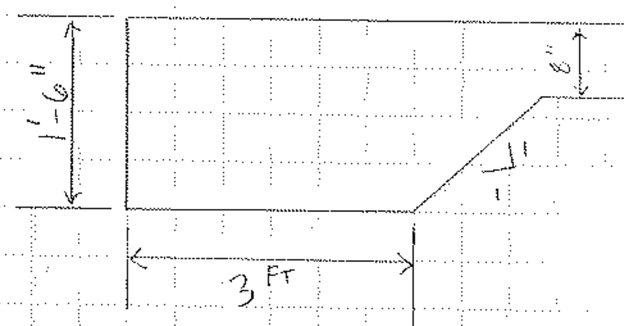
	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N3	max	0	14	1.075	33	0	14	0	14	0	14	0	14
2		min	0	14	.483	26	0	14	0	14	0	14	0	14
3	N141	max	0	14	1.068	33	0	14	0	14	0	14	0	14
4		min	0	14	.488	26	0	14	0	14	0	14	0	14
5	N144	max	0	14	1.059	33	0	14	0	14	0	14	0	14
6		min	0	14	.481	26	0	14	0	14	0	14	0	14
7	N4	max	0	14	1.059	27	0	14	0	14	0	14	0	14
8		min	0	14	.438	26	0	14	0	14	0	14	0	14
9	N347	max	0	14	1.052	27	0	14	0	14	0	14	0	14
10		min	0	14	.444	26	0	14	0	14	0	14	0	14
11	N139	max	0	14	1.052	33	0	14	0	14	0	14	0	14
12		min	0	14	.494	26	0	14	0	14	0	14	0	14
13	N344	max	0	14	1.043	27	0	14	0	14	0	14	0	14
14		min	0	14	.437	26	0	14	0	14	0	14	0	14
15	N6	max	0	14	1.038	27	0	14	0	14	0	14	0	14
16		min	0	14	.614	26	0	14	0	14	0	14	0	14
17	N1	max	0	14	1.038	27	0	14	0	14	0	14	0	14
18		min	0	14	.613	26	0	14	0	14	0	14	0	14
19	N349	max	0	14	1.037	27	0	14	0	14	0	14	0	14
20		min	0	14	.453	26	0	14	0	14	0	14	0	14
21	N146	max	0	14	1.034	33	0	14	0	14	0	14	0	14
22		min	0	14	.48	26	0	14	0	14	0	14	0	14
23	N137	max	0	14	1.029	33	0	14	0	14	0	14	0	14
24		min	0	14	.501	26	0	14	0	14	0	14	0	14
25	N9	max	0	14	1.021	35	0	14	0	14	0	14	0	14
26		min	0	14	.504	24	0	14	0	14	0	14	0	14
27	N342	max	0	14	1.019	27	0	14	0	14	0	14	0	14
28		min	0	14	.438	26	0	14	0	14	0	14	0	14
29	N351	max	0	14	1.015	27	0	14	0	14	0	14	0	14
30		min	0	14	.464	26	0	14	0	14	0	14	0	14
31	N10	max	0	14	1.012	15	0	14	0	14	0	14	0	14
32		min	0	14	.49	24	0	14	0	14	0	14	0	14
33	N529	max	0	14	1.012	35	0	14	0	14	0	14	0	14
34		min	0	14	.505	24	0	14	0	14	0	14	0	14
35	N7	max	0	14	1.01	40	0	14	0	14	0	14	0	14
36		min	0	14	.678	18	0	14	0	14	0	14	0	14
37	N532	max	0	14	1.01	35	0	14	0	14	0	14	0	14
38		min	0	14	.506	24	0	14	0	14	0	14	0	14
39	N12	max	0	14	1.005	15	0	14	0	14	0	14	0	14
40		min	0	14	.68	18	0	14	0	14	0	14	0	14
41	N135	max	0	14	1.003	33	0	14	0	14	0	14	0	14
42		min	0	14	.508	26	0	14	0	14	0	14	0	14
43	N495	max	0	14	1.003	15	0	14	0	14	0	14	0	14
44		min	0	14	.492	24	0	14	0	14	0	14	0	14
45	N492	max	0	14	1.002	15	0	14	0	14	0	14	0	14
46		min	0	14	.492	24	0	14	0	14	0	14	0	14
47	N148	max	0	14	1.002	33	0	14	0	14	0	14	0	14
48		min	0	14	.48	26	0	14	0	14	0	14	0	14
49	N527	max	0	14	.996	35	0	14	0	14	0	14	0	14
50		min	0	14	.509	24	0	14	0	14	0	14	0	14

EQUIPMENT STORAGE FOUNDATION

FOOTING DEMANDS PER RISA (LRFD LOAD COMBINATIONS)

		LC	
M_{ux} =	$17.4^{k\cdot ft}$	55	→ $2.4^{k\cdot ft}$ @ SLAB
	$-13.3^{k\cdot ft}$	55	
M_{uy} =	$2.2^{k\cdot ft}$	55	
	$-13.1^{k\cdot ft}$	55	→ $3.3^{k\cdot ft}$ @ SLAB
V_{uy} =	6.4^{k}	55	→ 3.7^{k} @ SLAB

USE 3' x 18" FOOTING w/ 8" SLAB



Smith Monroe Gray
ENGINEERS, INC.

Client: LAKESIDE INDUSTRIES

Job #: 18-183B

By: BS

Project:

Date: 8/16/2018 Sheet of

Concrete Slab Design per ACI 318-08

IN COMPLIANCE W/ACI 318-14

Applied Forces:

Ultimate Shear, V_u = 10 kips

Ultimate Moment, M_u = 18 ft-kips

Slab Properties:

Width = 12 in

Depth = 18 in

Cover = 3 in.

d = 14.69 in.

f'_c = 4000 psi

β_1 = 0.85

Capacity:

Shear: $\phi = 0.75$

$\Phi V_c = \Phi V_n = \phi*2*b*d*\sqrt{f'c}$

$\Phi V_c = \Phi V_n =$ 16.72 kips

Bending: $\phi = 0.9$

$\Phi M_n = \phi(As*fy*(d-a/2))$

$\Phi M_n =$ 26.75 k-ft.

Longitudinal Reinforcement:

Bar Size = 5

Spacing = 9 inches o.c.

f_y = 60000 psi

A_s = 0.41 in^2

a = 0.61 in

c = 0.72 in

Shrinkage and Temperature Reinforcing

Min. reinf. ratio = 0.0018

A_s min = 0.32 in^2 OK

max. spacing = 18.0 in

Check Tension Controlled (ACI 10.3.4)

$\varepsilon_t = [(d-c)/c]*0.003$

ε_t = 0.0586 **> 0.005, OK**

Demand Ratios:

$V_u/\Phi V_n$ =	0.60	**SLAB IS OK IN SHEAR**
$M_u/\Phi M_n$ =	0.67	**SLAB IS OK IN BENDING**

Smith Monroe Gray
ENGINEERS, INC.

Client: LAKESIDE INDUSTRIES

Job #: 18-183B

By: BS

Project:

Date: 8/16/2018 Sheet of

Concrete Slab Design per ACI 318-08

IN COMPLIANCE W/ACI 318-14

Applied Forces:

Ultimate Shear, V_u = 3.7 kips

Ultimate Moment, M_u = 3.3 ft-kips

Slab Properties:

Width = 12 in

Depth = 8 in

Cover = 4 in.

d = 3.69 in.

f'_c = 4000 psi

β_1 = 0.85

Capacity:

Shear: $\phi = 0.75$

$\Phi V_c = \Phi V_n = \phi*2*b*d*\sqrt{f'c}$

$\Phi V_c = \Phi V_n =$ 4.20 kips

Bending: $\phi = 0.9$

$\Phi M_n = \phi(As*fy*(d-a/2))$

$\Phi M_n =$ 4.83 k-ft.

Longitudinal Reinforcement:

Bar Size = 5

Spacing = 12 inches o.c.

f_y = 60000 psi

A_s = 0.31 in^2

a = 0.46 in

c = 0.54 in

Shrinkage and Temperature Reinforcing

Min. reinf. ratio = 0.0018

A_s min = 0.08 in^2 OK

max. spacing = 18.0 in

Check Tension Controlled (ACI 10.3.4)

$\varepsilon_t = [(d-c)/c]*0.003$

ε_t = 0.0176 **> 0.005, OK**

Demand Ratios:

$V_u/\Phi V_n$ = 0.88 **SLAB IS OK IN SHEAR**

$M_u/\Phi M_n$ = 0.68 **SLAB IS OK IN BENDING**

Envelope Plate Forces (per ft)

	Plate		Qx [k]	LC	Qy [k]	LC	Mx [k-ft]	LC	My [k-ft]	LC	Mxy [k-ft]	LC	Fx [k]	LC	Fy [k]	LC	Fxy [k]	LC
1	P123	max	6.371	55	.856	58	17.239	55	1.971	55	0	47	.46	61	.382	58	.962	55
2		min	-.777	58	-2.56	55	-5.516	58	-.638	58	-1.856	55	-12.291	59	-.353	55	-1.101	58
3	P124	max	6.122	55	.069	55	11.649	55	.425	55	0	47	.824	60	.25	55	.57	61
4		min	-.62	58	-.019	58	-3.721	58	-.122	58	-2.665	55	-12.708	59	-.271	58	-.658	58
5	P125	max	5.838	55	.031	56	6.982	55	.14	55	0	47	1.615	60	.077	61	.472	61
6		min	-.442	58	-.078	55	-2.288	58	-.031	56	-3.092	55	-13.09	59	-.058	60	-.44	58
7	P164	max	5.642	55	2.807	55	17.153	55	2.195	55	1.475	55	12.13	61	1.308	55	.591	55
8		min	-.988	58	-.764	58	-5.301	58	-.779	58	0	47	-12.285	60	-1.409	58	-.57	58
9	P126	max	5.567	55	.016	56	3.089	55	.094	55	0	47	2.394	60	.073	61	.424	61
10		min	-.261	58	-.033	55	-1.186	56	-.014	56	-3.267	55	-13.473	59	-.058	60	-.392	60
11	P127	max	5.309	55	.007	56	0	47	.061	57	0	47	3.164	60	.064	61	.396	61
12		min	-.075	58	-.014	57	-.511	56	0	56	-3.414	55	-13.886	59	-.058	60	-.385	60
13	P103	max	5.143	55	.431	58	17.042	55	2.059	55	0	47	13.249	61	.224	58	.945	55
14		min	-.422	58	-2.728	55	-2.713	58	-.301	58	-.432	56	-12.776	60	-.389	55	-.607	58
15	P128	max	5.072	55	.002	55	.175	58	.036	57	0	47	3.924	60	.062	61	.378	61
16		min	0	47	-.006	60	-2.783	55	0	47	-3.511	55	-14.351	59	-.059	60	-.376	60
17	P146	max	5.04	55	.137	58	0	47	.271	58	5.024	55	2.346	61	.697	61	.591	61
18		min	0	47	0	47	-10.551	55	0	47	0	47	-2.36	60	-.697	60	-.583	60
19	P129	max	4.858	55	.014	55	.473	58	.039	58	0	47	4.671	60	.061	61	.364	61
20		min	0	47	-.012	58	-4.971	55	0	47	-3.565	55	-14.884	59	-.06	60	-.366	60
21	P326	max	4.756	57	1.739	57	10.852	57	1.336	57	1.685	57	1.407	58	.215	55	.537	55
22		min	0	47	-.074	56	-.923	56	-.04	56	0	47	-11.843	59	-.631	58	-1.733	58
23	P104	max	4.741	55	.112	55	10.943	55	.386	55	0	47	12.417	61	.266	55	.515	55
24		min	-.329	58	-.016	58	-1.716	58	-.038	58	-.994	55	-12.263	60	-.145	58	-.378	58
25	P399	max	4.672	57	0	47	0	47	.267	45	0	47	1.687	60	.232	60	.996	60
26		min	0	47	-.127	55	-9.79	55	0	47	-5.233	55	-1.915	59	-.232	61	-1	61
27	P130	max	4.668	55	.022	55	.556	58	.054	58	0	47	5.405	60	.061	61	.35	61
28		min	0	47	-.02	58	-6.793	55	-.018	55	-3.585	55	-15.502	59	-.06	60	-.354	60
29	P145	max	4.664	55	1.37	55	0	47	0	47	6.703	55	.675	58	2.166	61	.166	60
30		min	0	47	0	47	-9.372	55	-3.199	55	0	47	-.581	55	-2.171	60	-.156	61
31	P417	max	4.62	57	0	47	11.82	57	1.594	57	0	47	11.252	60	2.3	58	.296	60
32		min	0	47	-2.131	57	0	47	0	47	-1.614	57	-11.582	61	-.708	55	-1.012	58
33	P327	max	4.559	57	0	47	7.069	57	.283	57	2.233	57	1.946	61	.441	58	.548	61
34		min	0	47	-.055	57	-.902	56	0	47	0	47	-12.231	59	-.095	55	-1.011	58
35	P131	max	4.502	55	.026	55	.45	58	.07	58	0	47	6.123	60	.061	61	.339	59
36		min	0	47	-.027	58	-8.33	55	-.027	55	-3.583	55	-16.221	59	-.061	60	-.341	60
37	P147	max	4.489	55	.203	55	0	47	.306	58	4.842	55	4.132	61	.597	61	.403	61
38		min	0	47	0	47	-12.125	55	0	47	0	47	-4.149	60	-.597	60	-.394	60
39	P165	max	4.406	55	.163	45	11.407	55	.594	55	1.484	55	12.638	61	.318	55	.459	60
40		min	-.601	58	0	47	-3.356	58	-.262	56	0	47	-12.61	60	-.337	58	-.329	56
41	P132	max	4.36	55	.024	55	.181	58	.086	58	0	47	6.825	60	.061	61	.413	59
42		min	0	47	-.035	58	-9.644	55	-.026	55	-3.572	55	-17.065	59	-.061	60	-.327	60
43	P328	max	4.342	57	.049	55	3.951	57	.098	55	2.523	57	3.022	61	.072	58	.559	61
44		min	0	47	-.006	56	-.908	56	0	47	0	47	-12.585	59	-.048	61	-.669	58
45	P184	max	4.3	55	2.67	55	14.226	55	1.784	55	.689	56	15.094	61	1.44	55	.645	55
46		min	-.835	58	-.7	58	-5.503	58	-.777	58	0	47	-15.213	60	-.782	58	-.79	58
47	P105	max	4.291	55	.009	58	5.919	55	.087	59	0	47	11.728	61	.071	61	.34	61
48		min	-.227	58	-.045	55	-.862	58	0	47	-1.365	55	-11.75	60	-.052	60	-.267	60
49	P133	max	4.242	55	.018	55	0	47	.103	58	0	47	7.515	60	.06	61	.507	59
50		min	0	47	-.043	58	-10.774	55	-.012	55	-3.57	55	-18.061	59	-.06	60	-.313	60

Envelope Plate Forces (per ft)

	Plate		Qx [k]	LC	Qy [k]	LC	Mx [k-ft]	LC	My [k-ft]	LC	Mxy [k-ft]	LC	Fx [k]	LC	Fy [k]	LC	Fxy [k]	LC
1	P213	max	.893	58	.852	58	17.36	55	1.979	55	1.718	55	14.193	61	.384	58	.967	58
2		min	-6.257	55	-2.564	55	-5.398	58	-.63	58	0	47	-13.657	60	-.353	55	-.867	55
3	P212	max	.729	58	.065	55	11.733	55	.433	55	2.537	55	13.484	61	.25	55	.527	58
4		min	-6.015	55	-.023	58	-3.638	58	-.114	58	0	47	-13.266	60	-.27	58	-.459	55
5	P72	max	.829	58	2.807	55	16.995	55	2.171	55	0	47	.082	61	1.308	55	.724	61
6		min	-5.797	55	-.762	58	-5.457	58	-.803	58	-1.638	55	-12.455	59	-1.415	58	-1.084	60
7	P211	max	.546	58	.027	56	7.034	55	.147	55	2.972	55	12.928	61	.075	61	.313	58
8		min	-5.737	55	-.081	55	-2.237	58	-.024	56	0	47	-12.883	60	-.055	60	-.267	61
9	P210	max	.359	58	.013	56	3.115	55	.1	55	3.156	55	12.464	61	.071	61	.215	58
10		min	-5.471	55	-.036	55	-1.16	56	-.008	56	0	47	-12.504	60	-.056	60	-.223	61
11	P209	max	.169	58	.005	56	0	47	.067	57	3.31	55	12.048	61	.062	61	.19	60
12		min	-5.217	55	-.017	57	-.507	56	0	47	0	47	-12.127	60	-.056	60	-.2	61
13	P266	max	0	47	1.902	57	12.026	57	1.469	57	0	47	13.516	60	.198	55	1.537	58
14		min	-5.092	57	0	47	0	47	0	47	-1.759	57	-13.006	61	-.634	58	-.552	55
15	P90	max	0	47	.146	58	0	47	.279	58	0	47	16.679	60	1.065	60	6.593	59
16		min	-5.046	55	0	47	-10.772	55	0	47	-5.074	55	-21.259	59	-2.391	59	-3.529	60
17	P208	max	0	47	0	47	.164	58	.041	57	3.414	55	11.66	61	.06	61	.186	60
18		min	-4.984	55	-.007	58	-2.794	55	0	47	0	47	-11.754	60	-.057	60	-.187	61
19	P265	max	0	47	0	47	7.928	57	.322	57	0	47	12.618	60	.432	58	.838	58
20		min	-4.889	57	-.052	57	-.008	56	0	47	-2.373	57	-12.428	61	-.135	55	-.333	55
21	P207	max	0	47	.012	55	.45	58	.044	58	3.473	55	11.29	61	.059	61	.18	60
22		min	-4.773	55	-.014	58	-4.994	55	0	47	0	47	-11.387	60	-.058	60	-.177	61
23	P52	max	.449	58	3.044	55	16.94	55	2.172	55	.224	59	11.177	61	1.422	55	.26	61
24		min	-4.679	55	-.32	58	-2.74	58	-.382	58	-.414	56	-11.313	60	-.798	58	-.635	60
25	P264	max	0	47	.062	55	4.566	57	.121	55	0	47	11.863	60	.073	58	.499	58
26		min	-4.662	57	0	47	-.268	56	0	47	-2.706	57	-11.849	61	-.05	61	-.304	61
27	P91	max	0	47	1.43	55	0	47	0	47	0	47	17.707	60	1.261	60	8.109	59
28		min	-4.658	55	0	47	-9.691	55	-3.237	55	-6.818	55	-17.901	59	-4.505	59	-5.882	60
29	P233	max	.919	58	.81	58	14.336	55	1.759	55	.607	56	18.04	61	.197	58	.646	58
30		min	-4.654	55	-2.356	55	-5.468	58	-.607	58	0	47	-17.547	60	-.417	55	-.658	55
31	P206	max	0	47	.02	55	.526	58	.059	58	3.499	55	10.938	61	.06	61	.171	60
32		min	-4.585	55	-.022	58	-6.823	55	-.014	55	0	47	-11.032	60	-.06	60	-.166	61
33	P250	max	0	47	.061	45	0	47	.199	45	0	47	1.14	60	.043	60	.47	60
34		min	-4.581	55	0	47	-10.405	57	0	47	-4.637	55	-2.148	59	-.043	61	-.472	61
35	P71	max	.43	58	.163	45	11.204	55	.568	55	0	47	.983	61	.319	55	.869	61
36		min	-4.574	55	0	47	-3.56	58	-.287	56	-1.66	55	-12.149	59	-.343	58	-1.088	60
37	P251	max	0	47	.077	45	0	47	.183	45	0	47	1.979	60	.053	60	.462	60
38		min	-4.567	55	0	47	-10.605	57	0	47	-4.419	55	-2.231	59	-.053	61	-.464	61
39	P393	max	0	47	0	47	0	47	.274	59	4.955	55	17.393	61	1.766	61	5.88	61
40		min	-4.558	57	-.137	55	-10.097	55	0	47	0	47	-20.583	59	-2.344	59	-6.474	59
41	P252	max	0	47	.069	45	0	47	.19	45	0	47	2.829	60	.053	60	.459	60
42		min	-4.548	55	0	47	-10.373	57	0	47	-4.171	55	-2.936	61	-.053	61	-.461	61
43	P253	max	0	47	.064	45	0	47	.165	45	0	47	3.682	60	.053	60	.455	60
44		min	-4.522	55	0	47	-9.75	57	0	47	-3.948	55	-3.793	61	-.053	61	-.458	61
45	P256	max	0	47	.354	45	0	47	.319	45	0	47	6.22	60	.219	55	1.18	58
46		min	-4.51	57	0	47	-6.139	57	0	47	-3.429	57	-6.345	61	-.407	58	-.678	55
47	P249	max	0	47	0	47	0	47	0	47	0	47	.328	60	.093	60	.476	60
48		min	-4.51	55	-.227	55	-9.849	55	-.21	57	-4.926	55	-2.072	59	-.094	61	-.478	61
49	P254	max	0	47	.057	56	0	47	.137	45	0	47	4.534	60	.052	60	.507	58
50		min	-4.502	55	0	47	-8.805	57	0	47	-3.769	55	-4.649	61	-.052	61	-.454	61

Envelope Plate Forces (per ft)

	Plate		Qx [k]	LC	Qy [k]	LC	Mx [k-ft]	LC	My [k-ft]	LC	Mxy [k-ft]	LC	Fx [k]	LC	Fy [k]	LC	Fxy [k]	LC
1	P213	max	.893	58	.852	58	17.36	55	1.979	55	1.718	55	14.193	61	.384	58	.967	58
2		min	-6.257	55	-2.564	55	-5.398	58	-.63	58	0	47	-13.657	60	-.353	55	-.867	55
3	P214	max	2.414	55	.852	58	17.294	55	1.975	55	2.208	55	14.536	61	.385	58	.699	55
4		min	-1.912	58	-2.562	55	-5.307	58	-.631	58	0	47	-14.004	60	-.354	55	-.733	58
5	P123	max	6.371	55	.856	58	17.239	55	1.971	55	0	47	.46	61	.382	58	.962	55
6		min	-.777	58	-2.56	55	-5.516	58	-.638	58	-1.856	55	-12.291	59	-.353	55	-1.101	58
7	P163	max	1.641	58	2.804	55	17.218	55	2.208	55	2.753	55	11.815	61	1.309	55	1.238	58
8		min	-2.205	55	-.764	58	-5.392	58	-.778	58	0	47	-11.968	60	-1.411	58	-1.071	55
9	P164	max	5.642	55	2.807	55	17.153	55	2.195	55	1.475	55	12.13	61	1.308	55	.591	55
10		min	-.988	58	-.764	58	-5.301	58	-.779	58	0	47	-12.285	60	-1.409	58	-.57	58
11	P122	max	2.034	58	.856	58	17.132	55	1.966	55	0	47	1.226	61	.383	58	.595	58
12		min	-2.293	55	-2.557	55	-5.468	58	-.639	58	-2.357	55	-11.972	59	-.353	55	-.604	55
13	P73	max	2.061	55	2.804	55	17.101	55	2.186	55	0	47	.419	60	1.309	55	1.527	61
14		min	-1.786	58	-.763	58	-5.506	58	-.8	58	-2.904	55	-12.793	59	-1.418	58	-1.628	58
15	P102	max	1.006	58	.43	58	17.062	55	2.06	55	0	47	13.548	61	.218	58	.368	58
16		min	-4.085	55	-2.729	55	-2.717	58	-.3	58	-.824	55	-13.004	60	-.386	55	-.768	55
17	P103	max	5.143	55	.431	58	17.042	55	2.059	55	0	47	13.249	61	.224	58	.945	55
18		min	-.422	58	-2.728	55	-2.713	58	-.301	58	-.432	56	-12.776	60	-.389	55	-.607	58
19	P72	max	.829	58	2.807	55	16.995	55	2.171	55	0	47	.082	61	1.308	55	.724	61
20		min	-5.797	55	-.762	58	-5.457	58	-.803	58	-1.638	55	-12.455	59	-1.415	58	-1.084	60
21	P52	max	.449	58	3.044	55	16.94	55	2.172	55	.224	59	11.177	61	1.422	55	.26	61
22		min	-4.679	55	-.32	58	-2.74	58	-.382	58	-.414	56	-11.313	60	-.798	58	-.635	60
23	P53	max	3.826	55	3.042	55	16.921	55	2.17	55	0	47	10.925	61	1.42	55	1.222	55
24		min	-.958	58	-.321	58	-2.735	58	-.386	58	-1.196	55	-11.133	60	-.795	58	-.942	58
25	P233	max	.919	58	.81	58	14.336	55	1.759	55	.607	56	18.04	61	.197	58	.646	58
26		min	-4.654	55	-2.356	55	-5.468	58	-.607	58	0	47	-17.547	60	-.417	55	-.658	55
27	P234	max	3.342	55	.81	58	14.318	55	1.759	55	.955	55	17.878	61	.246	58	1.058	55
28		min	-1.763	58	-2.356	55	-5.51	58	-.606	58	0	47	-17.316	60	-.36	55	-.33	58
29	P183	max	1.656	58	2.669	55	14.244	55	1.784	55	1.256	55	15.259	61	1.406	55	.257	58
30		min	-3.14	55	-.701	58	-5.46	58	-.778	58	0	47	-15.449	60	-.811	58	-1.179	55
31	P184	max	4.3	55	2.67	55	14.226	55	1.784	55	.689	56	15.094	61	1.44	55	.645	55
32		min	-.835	58	-.7	58	-5.503	58	-.777	58	0	47	-15.213	60	-.782	58	-.79	58
33	P306	max	3.858	57	2.04	57	12.235	57	1.533	57	.219	59	13.106	60	.223	55	.574	55
34		min	-.843	56	-.495	56	-3.829	56	-.382	56	-.01	56	-12.722	61	-.525	58	-1.286	58
35	P305	max	.778	56	2.039	57	12.196	57	1.531	57	.672	57	13.441	60	.218	55	1.032	58
36		min	-3.045	57	-.495	56	-3.793	56	-.382	56	-.102	56	-13.058	61	-.524	58	-.397	55
37	P286	max	.898	56	2.038	57	12.16	57	1.53	57	.076	56	17.785	60	.219	55	1.138	58
38		min	-3.806	57	-.498	56	-3.914	56	-.386	56	-.129	57	-17.421	61	-.525	58	-.492	55
39	P356	max	2.799	57	.35	56	12.143	57	1.566	57	.942	57	10.704	60	1.924	58	.838	55
40		min	-.664	56	-2.328	57	-3.825	56	-.564	56	-.18	56	-10.987	61	-.795	55	-1.613	58
41	P287	max	3.095	57	2.037	57	12.137	57	1.528	57	.164	56	17.771	60	.222	55	.479	55
42		min	-.727	56	-.497	56	-3.86	56	-.385	56	-.612	57	-17.407	61	-.524	58	-1.179	58
43	P355	max	.806	56	.35	56	12.105	57	1.559	57	.118	59	10.996	60	1.925	58	.844	58
44		min	-3.587	57	-2.328	57	-3.789	56	-.564	56	-.059	60	-11.28	61	-.798	55	-.211	60
45	P436	max	.596	56	.35	56	12.069	57	1.556	57	.252	56	14.932	60	1.923	58	1.189	58
46		min	-2.865	57	-2.328	57	-3.908	56	-.575	56	-.872	57	-15.232	61	-.797	55	-.502	55
47	P437	max	3.527	57	.35	56	12.048	57	1.551	57	.128	59	14.921	60	1.924	58	.532	55
48		min	-.869	56	-2.329	57	-3.855	56	-.574	56	-.039	61	-15.221	61	-.796	55	-1.267	58
49	P266	max	0	47	1.902	57	12.026	57	1.469	57	0	47	13.516	60	.198	55	1.537	58
50		min	-5.092	57	0	47	0	47	0	47	-1.759	57	-13.006	61	-.634	58	-.552	55

Envelope Plate Forces (per ft)

	Plate		Qx [k]	LC	Qy [k]	LC	Mx [k-ft]	LC	My [k-ft]	LC	Mxy [k-ft]	LC	Fx [k]	LC	Fy [k]	LC	Fxy [k]	LC
1	P87	max	0	47	.212	55	0	47	.378	58	0	47	12.487	60	.107	58	3.051	59
2		min	-3.372	55	0	47	-13.32	55	0	47	-4.233	55	-21.475	59	-.829	59	-1.524	60
3	P149	max	3.349	55	.213	55	0	47	.384	58	4.176	55	6.213	61	.415	61	.045	57
4		min	0	47	0	47	-13.246	55	0	47	0	47	-6.236	60	-.415	60	-.049	56
5	P137	max	3.947	55	0	47	0	47	.175	45	0	47	10.462	60	0	45	1.434	59
6		min	0	47	-.069	58	-13.184	55	0	47	-3.962	55	-24.763	59	-.222	59	-.394	60
7	P199	max	0	47	0	47	0	47	.175	45	3.907	55	9.825	61	.132	61	.24	61
8		min	-3.867	55	-.07	58	-13.108	55	0	47	0	47	-9.868	60	-.132	60	-.236	60
9	P86	max	0	47	.19	55	0	47	.348	58	0	47	11.236	60	.044	58	2.442	59
10		min	-2.978	55	0	47	-13.102	55	0	47	-4.003	55	-20.538	59	-.571	59	-1.317	60
11	P88	max	0	47	.237	55	0	47	.4	45	0	47	13.857	60	.296	60	3.877	59
12		min	-3.884	55	0	47	-13.099	55	0	47	-4.544	55	-22.175	59	-1.219	59	-1.876	60
13	P150	max	2.948	55	.191	55	0	47	.355	58	3.943	55	6.874	61	.344	61	.174	60
14		min	0	47	0	47	-13.062	55	0	47	0	47	-6.9	60	-.345	60	-.166	61
15	P136	max	4.008	55	0	47	0	47	.157	58	0	47	9.627	60	.035	61	1.055	59
16		min	0	47	-.067	58	-13.003	55	0	47	-3.778	55	-22.481	59	-.145	59	-.326	60
17	P148	max	3.868	55	.238	55	0	47	.4	45	4.491	55	5.343	61	.495	61	.169	61
18		min	0	47	0	47	-12.982	55	0	47	0	47	-5.363	60	-.495	60	-.16	60
19	P200	max	0	47	0	47	0	47	.159	58	3.72	55	9.682	61	.109	61	.103	61
20		min	-3.93	55	-.068	58	-12.962	55	0	47	0	47	-9.732	60	-.109	60	-.099	60
21	P138	max	3.872	55	0	47	0	47	.176	45	0	47	11.505	60	.054	60	2.021	59
22		min	0	47	-.074	58	-12.927	55	0	47	-4.196	55	-27.768	59	-.348	59	-.554	60
23	P198	max	0	47	0	47	0	47	.176	45	4.143	55	10.189	61	.164	61	.434	61
24		min	-3.79	55	-.074	58	-12.809	55	0	47	0	47	-10.225	60	-.164	60	-.43	60
25	P85	max	0	47	.175	55	0	47	.302	58	0	47	10.106	60	.112	61	1.986	59
26		min	-2.7	55	0	47	-12.556	55	0	47	-3.857	55	-19.543	59	-.405	59	-1.201	60
27	P151	max	2.661	55	.176	55	0	47	.311	58	3.791	55	7.412	61	.292	61	.288	60
28		min	0	47	0	47	-12.544	55	0	47	0	47	-7.439	60	-.292	60	-.279	61
29	P135	max	4.071	55	0	47	0	47	.14	58	0	47	8.892	60	.05	61	.804	59
30		min	0	47	-.06	58	-12.49	55	0	47	-3.657	55	-20.692	59	-.097	59	-.303	60
31	P201	max	0	47	0	47	0	47	.143	58	3.595	55	9.699	61	.091	61	0	45
32		min	-3.993	55	-.062	58	-12.477	55	0	47	0	47	-9.757	60	-.092	60	0	45
33	P89	max	0	47	.201	55	0	47	.305	58	0	47	15.295	60	.634	60	5.001	59
34		min	-4.501	55	0	47	-12.29	55	0	47	-4.892	55	-22.304	59	-1.773	59	-2.444	60
35	P139	max	3.778	55	0	47	0	47	.193	45	0	47	12.94	60	.171	60	2.918	59
36		min	0	47	-.06	45	-12.149	55	0	47	-4.418	55	-31.839	59	-.502	59	-.886	60
37	P147	max	4.489	55	.203	55	0	47	.306	58	4.842	55	4.132	61	.597	61	.403	61
38		min	0	47	0	47	-12.125	55	0	47	0	47	-4.149	60	-.597	60	-.394	60
39	P197	max	0	47	0	47	0	47	.193	45	4.367	55	10.85	61	.167	61	.686	61
40		min	-3.693	55	-.06	45	-11.982	55	0	47	0	47	-10.88	60	-.167	60	-.683	60
41	P152	max	2.466	55	.168	55	0	47	.261	58	3.71	55	7.88	61	.254	61	.373	60
42		min	0	47	0	47	-11.779	55	-.063	55	0	47	-7.907	60	-.254	60	-.364	61
43	P84	max	0	47	.167	55	0	47	.253	58	0	47	9.082	60	.149	61	1.637	59
44		min	-2.512	55	0	47	-11.77	55	-.072	55	-3.781	55	-18.579	59	-.299	59	-1.14	60
45	P202	max	0	47	.004	55	0	47	.125	58	3.526	55	9.829	61	.08	61	.061	60
46		min	-4.069	55	-.053	58	-11.736	55	0	47	0	47	-9.894	60	-.08	60	-.056	61
47	P134	max	4.147	55	.006	55	0	47	.121	58	0	47	8.198	60	.057	61	.631	59
48		min	0	47	-.052	58	-11.729	55	0	47	-3.592	55	-19.25	59	-.068	59	-.303	60
49	P391	max	0	47	0	47	0	47	.419	45	4.415	55	17.713	61	.771	61	3.826	61
50		min	-3.454	57	-.228	45	-11.305	57	0	47	0	47	-21.483	59	-1.195	59	-3.812	59

Envelope Plate Forces (per ft)

	Plate		Qx [k]	LC	Qy [k]	LC	Mx [k-ft]	LC	My [k-ft]	LC	Mxy [k-ft]	LC	Fx [k]	LC	Fy [k]	LC	Fxy [k]	LC
1	P163	max	1.641	58	2.804	55	17.218	55	2.208	55	2.753	55	11.815	61	1.309	55	1.238	58
2		min	-2.205	55	-.764	58	-5.392	58	-.778	58	0	47	-11.968	60	-1.411	58	-1.071	55
3	P164	max	5.642	55	2.807	55	17.153	55	2.195	55	1.475	55	12.13	61	1.308	55	.591	55
4		min	-.988	58	-.764	58	-5.301	58	-.779	58	0	47	-12.285	60	-1.409	58	-.57	58
5	P73	max	2.061	55	2.804	55	17.101	55	2.186	55	0	47	.419	60	1.309	55	1.527	61
6		min	-1.786	58	-.763	58	-5.506	58	-.8	58	-2.904	55	-12.793	59	-1.418	58	-1.628	58
7	P52	max	.449	58	3.044	55	16.94	55	2.172	55	.224	59	11.177	61	1.422	55	.26	61
8		min	-4.679	55	-.32	58	-2.74	58	-.382	58	-.414	56	-11.313	60	-.798	58	-.635	60
9	P72	max	.829	58	2.807	55	16.995	55	2.171	55	0	47	.082	61	1.308	55	.724	61
10		min	-5.797	55	-.762	58	-5.457	58	-.803	58	-1.638	55	-12.455	59	-1.415	58	-1.084	60
11	P53	max	3.826	55	3.042	55	16.921	55	2.17	55	0	47	10.925	61	1.42	55	1.222	55
12		min	-.958	58	-.321	58	-2.735	58	-.386	58	-1.196	55	-11.133	60	-.795	58	-.942	58
13	P102	max	1.006	58	.43	58	17.062	55	2.06	55	0	47	13.548	61	.218	58	.368	58
14		min	-4.085	55	-2.729	55	-2.717	58	-.3	58	-.824	55	-13.004	60	-.386	55	-.768	55
15	P103	max	5.143	55	.431	58	17.042	55	2.059	55	0	47	13.249	61	.224	58	.945	55
16		min	-.422	58	-2.728	55	-2.713	58	-.301	58	-.432	56	-12.776	60	-.389	55	-.607	58
17	P213	max	.893	58	.852	58	17.36	55	1.979	55	1.718	55	14.193	61	.384	58	.967	58
18		min	-6.257	55	-2.564	55	-5.398	58	-.63	58	0	47	-13.657	60	-.353	55	-.867	55
19	P214	max	2.414	55	.852	58	17.294	55	1.975	55	2.208	55	14.536	61	.385	58	.699	55
20		min	-1.912	58	-2.562	55	-5.307	58	-.631	58	0	47	-14.004	60	-.354	55	-.733	58
21	P123	max	6.371	55	.856	58	17.239	55	1.971	55	0	47	.46	61	.382	58	.962	55
22		min	-.777	58	-2.56	55	-5.516	58	-.638	58	-1.856	55	-12.291	59	-.353	55	-1.101	58
23	P122	max	2.034	58	.856	58	17.132	55	1.966	55	0	47	1.226	61	.383	58	.595	58
24		min	-2.293	55	-2.557	55	-5.468	58	-.639	58	-2.357	55	-11.972	59	-.353	55	-.604	55
25	P184	max	4.3	55	2.67	55	14.226	55	1.784	55	.689	56	15.094	61	1.44	55	.645	55
26		min	-.835	58	-.7	58	-5.503	58	-.777	58	0	47	-15.213	60	-.782	58	-.79	58
27	P183	max	1.656	58	2.669	55	14.244	55	1.784	55	1.256	55	15.259	61	1.406	55	.257	58
28		min	-3.14	55	-.701	58	-5.46	58	-.778	58	0	47	-15.449	60	-.811	58	-1.179	55
29	P234	max	3.342	55	.81	58	14.318	55	1.759	55	.955	55	17.878	61	.246	58	1.058	55
30		min	-1.763	58	-2.356	55	-5.51	58	-.606	58	0	47	-17.316	60	-.36	55	-.33	58
31	P233	max	.919	58	.81	58	14.336	55	1.759	55	.607	56	18.04	61	.197	58	.646	58
32		min	-4.654	55	-2.356	55	-5.468	58	-.607	58	0	47	-17.547	60	-.417	55	-.658	55
33	P416	max	.201	56	0	47	11.912	57	1.608	57	0	47	11.097	60	2.296	58	1.95	58
34		min	-1.186	57	-2.128	57	0	47	0	47	-2.568	57	-11.26	61	-.707	55	-.817	61
35	P417	max	4.62	57	0	47	11.82	57	1.594	57	0	47	11.252	60	2.3	58	.296	60
36		min	0	47	-2.131	57	0	47	0	47	-1.614	57	-11.582	61	-.708	55	-1.012	58
37	P462	max	0	47	3.389	55	1.323	55	1.589	55	.74	58	.239	57	8.66	60	.859	58
38		min	-1.605	55	-.112	58	0	47	-4.926	58	-.777	55	-.208	56	-8.671	61	-.78	55
39	P356	max	2.799	57	.35	56	12.143	57	1.566	57	.942	57	10.704	60	1.924	58	.838	55
40		min	-.664	56	-2.328	57	-3.825	56	-.564	56	-.18	56	-10.987	61	-.795	55	-1.613	58
41	P355	max	.806	56	.35	56	12.105	57	1.559	57	.118	59	10.996	60	1.925	58	.844	58
42		min	-3.587	57	-2.328	57	-3.789	56	-.564	56	-.059	60	-11.28	61	-.798	55	-.211	60
43	P436	max	.596	56	.35	56	12.069	57	1.556	57	.252	56	14.932	60	1.923	58	1.189	58
44		min	-2.865	57	-2.328	57	-3.908	56	-.575	56	-.872	57	-15.232	61	-.797	55	-.502	55
45	P437	max	3.527	57	.35	56	12.048	57	1.551	57	.128	59	14.921	60	1.924	58	.532	55
46		min	-.869	56	-2.329	57	-3.855	56	-.574	56	-.039	61	-15.221	61	-.796	55	-1.267	58
47	P306	max	3.858	57	2.04	57	12.235	57	1.533	57	.219	59	13.106	60	.223	55	.574	55
48		min	-.843	56	-.495	56	-3.829	56	-.382	56	-.01	56	-12.722	61	-.525	58	-1.286	58
49	P305	max	.778	56	2.039	57	12.196	57	1.531	57	.672	57	13.441	60	.218	55	1.032	58
50		min	-3.045	57	-.495	56	-3.793	56	-.382	56	-.102	56	-13.058	61	-.524	58	-.397	55

Envelope Plate Forces (per ft)

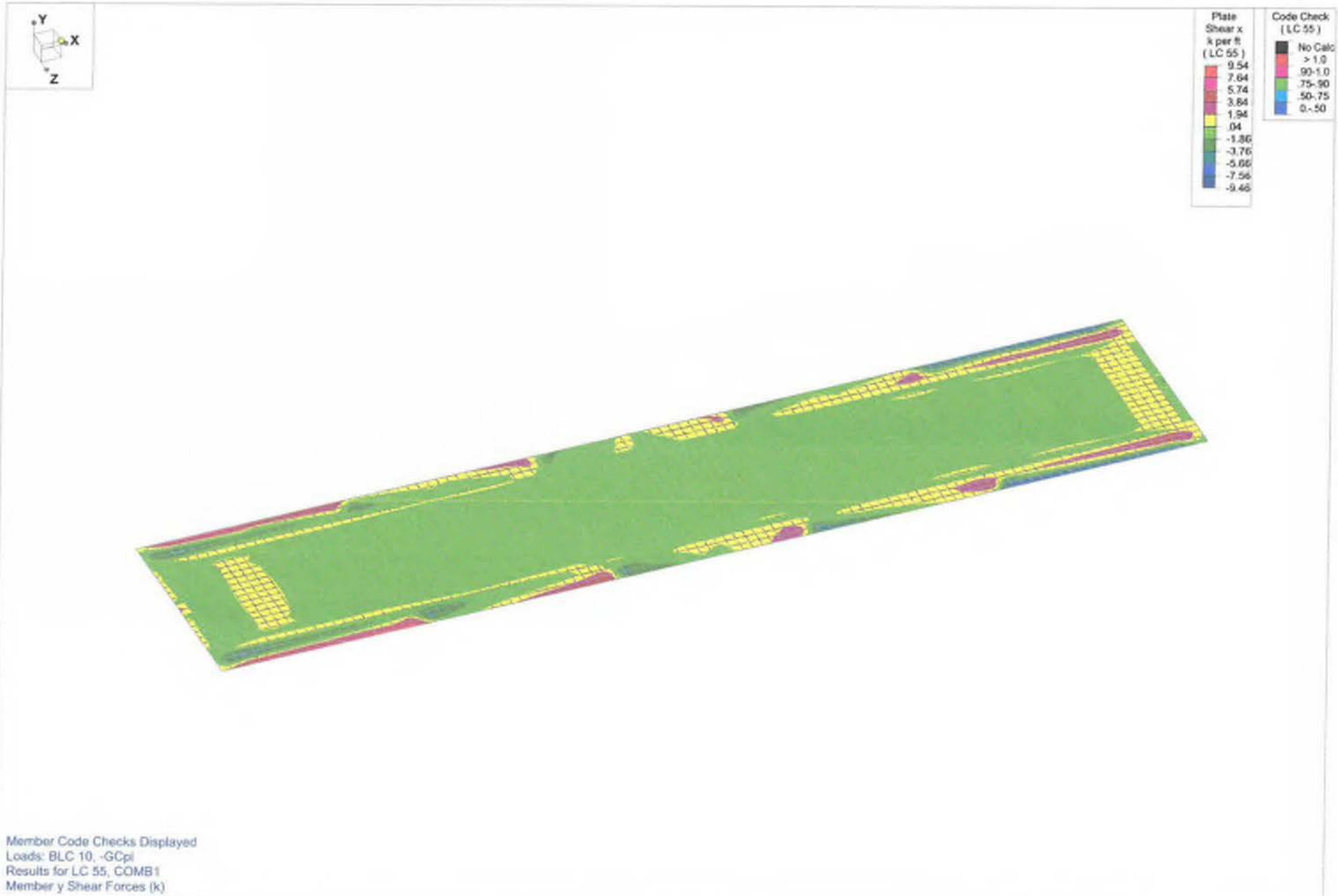
	Plate		Qx [k]	LC	Qy [k]	LC	Mx [k-ft]	LC	My [k-ft]	LC	Mxy [k-ft]	LC	Fx [k]	LC	Fy [k]	LC	Fxy [k]	LC
1	P39	max	0	47	2.334	57	0	47	0	47	0	47	3.357	60	21.087	60	7.993	61
2		min	-2.025	55	0	47	-2.021	57	-13.101	55	-4.951	57	-3.36	61	-21.112	61	-8.001	60
3	P481	max	2.001	55	2.197	57	0	47	0	47	4.741	45	3.146	60	12.032	60	6.168	60
4		min	0	47	0	47	-1.913	57	-12.965	55	0	47	-3.148	61	-12.019	61	-6.152	61
5	P35	max	0	47	2.066	58	.569	55	0	47	0	47	1.415	60	21.199	60	3.968	61
6		min	-.193	55	0	47	0	47	-12.8	55	-2.564	58	-1.414	61	-21.211	61	-3.98	60
7	P37	max	0	47	2.438	57	.582	55	0	47	0	47	2.031	60	21.811	60	5.545	61
8		min	-.349	55	0	47	0	47	-12.683	55	-2.839	57	-2.03	61	-21.828	61	-5.556	60
9	P477	max	.194	55	1.879	58	.587	55	0	47	2.39	60	1.246	60	14.896	60	4.036	60
10		min	0	47	0	47	0	47	-12.643	55	0	47	-1.243	61	-14.882	61	-4.024	61
11	P36	max	.106	55	0	47	.295	55	0	47	0	47	.396	60	23.845	60	1.914	61
12		min	0	47	-2.496	57	0	47	-12.636	55	-2.339	58	-.395	61	-23.83	61	-1.922	60
13	P38	max	0	47	0	47	.005	61	0	47	0	47	.596	59	27.966	60	2.825	61
14		min	-.178	55	-2.466	58	-.06	58	-12.57	55	-2.718	57	-.35	61	-27.938	61	-2.835	60
15	P479	max	.355	55	2.332	45	.595	55	0	47	2.738	45	1.695	60	14.337	60	5.132	60
16		min	0	47	0	47	0	47	-12.551	55	0	47	-1.693	61	-14.322	61	-5.118	61
17	P476	max	0	47	0	47	.301	55	0	47	2.189	60	.361	60	22.548	60	2.277	60
18		min	-.109	55	-2.352	60	0	47	-12.474	55	0	47	-.36	61	-22.527	61	-2.27	61
19	P447	max	2.023	55	0	47	0	47	0	47	0	47	.567	60	2.232	60	.967	60
20		min	0	47	-2.653	55	-2.514	55	-12.456	57	-6.18	55	-.582	61	-2.232	61	-.966	61
21	P478	max	.17	55	0	47	.011	59	0	47	2.612	45	.516	60	26.945	60	3.182	60
22		min	0	47	-2.326	60	-.045	58	-12.429	55	0	47	-.516	61	-26.913	61	-3.174	61
23	P5	max	0	47	0	47	0	47	0	47	5.926	55	2.505	59	.745	58	.193	61
24		min	-1.962	55	-2.716	55	-2.438	55	-12.368	57	0	47	-2.24	61	-1.377	59	-.198	60
25	P33	max	0	47	1.833	58	.657	55	0	47	0	47	.913	60	20.026	60	3.012	61
26		min	-.232	55	0	47	0	47	-12.159	55	-2.322	58	-.911	61	-20.037	61	-3.024	60
27	P475	max	.232	55	1.636	58	.676	55	0	47	2.098	58	.838	60	14.543	60	3.281	60
28		min	0	47	0	47	0	47	-11.973	55	0	47	-.836	61	-14.531	61	-3.271	61
29	P34	max	.125	55	0	47	.271	55	0	47	0	47	.242	60	20.978	60	1.256	61
30		min	0	47	-2.495	57	0	47	-11.922	55	-2.146	58	-.241	61	-20.974	61	-1.262	60
31	P9	max	0	47	0	47	.526	45	0	47	3.319	55	.699	59	5.846	60	.957	59
32		min	-.181	59	-2.406	55	0	47	-11.907	57	0	47	-.531	61	-5.857	61	-.385	61
33	P7	max	0	47	0	47	.518	57	0	47	3.642	55	.799	59	3.588	60	1.113	59
34		min	-.31	57	-2.964	55	0	47	-11.817	57	0	47	-.649	61	-3.593	61	-.622	61
35	P10	max	.098	45	3.385	55	.28	45	0	47	3.055	55	.203	59	14.201	59	1.068	59
36		min	0	47	0	47	0	47	-11.76	57	0	47	-.138	61	-13	61	-.784	61
37	P474	max	0	47	0	47	.278	55	0	47	1.94	58	.245	60	19.259	60	1.639	60
38		min	-.128	55	-2.316	57	0	47	-11.733	55	0	47	-.244	61	-19.248	61	-1.634	61
39	P8	max	0	47	3.268	55	.063	58	0	47	3.491	55	.588	59	15.869	59	1.555	59
40		min	-.204	55	0	47	-.079	55	-11.732	57	0	47	-.496	61	-14.086	61	-1.248	61
41	P449	max	.31	57	0	47	.515	45	0	47	0	47	.086	60	2.701	60	1.01	60
42		min	0	47	-2.973	55	0	47	-11.584	57	-3.824	55	-.222	59	-2.703	61	-1.012	61
43	P448	max	.218	55	3.596	55	.051	58	0	47	0	47	.052	60	1.62	58	.451	60
44		min	0	47	0	47	-.09	55	-11.505	57	-3.685	55	-.051	61	-1.267	61	-.455	61
45	P451	max	.173	45	0	47	.526	45	0	47	0	47	.066	58	3.253	60	1.117	60
46		min	0	47	-2.479	55	0	47	-11.479	57	-3.521	55	-.198	59	-3.254	61	-1.122	61
47	P11	max	0	47	0	47	.628	45	0	47	2.885	55	.579	59	7.466	60	.708	59
48		min	-.218	57	-1.689	55	0	47	-11.35	57	0	47	-.415	61	-7.48	61	-.078	61
49	P450	max	0	47	3.684	55	.28	45	0	47	0	47	0	45	2.2	60	.464	60
50		min	-.098	45	0	47	0	47	-11.346	57	-3.253	55	-.062	59	-2.187	61	-.468	61

Envelope Plate Forces (per ft)

	Plate		Qx [k]	LC	Qy [k]	LC	Mx [k-ft]	LC	My [k-ft]	LC	Mxy [k-ft]	LC	Fx [k]	LC	Fy [k]	LC	Fxy [k]	LC
1	P394	max	0	47	0	47	0	47	0	47	6.942	55	14.702	61	3.304	61	7.086	61
2		min	-4.148	57	-1.654	55	-10.165	55	-3.12	55	0	47	-17.279	59	-4.433	59	-7.942	59
3	P145	max	4.664	55	1.37	55	0	47	0	47	6.703	55	.675	58	2.166	61	.166	60
4		min	0	47	0	47	-9.372	55	-3.199	55	0	47	-.581	55	-2.171	60	-.156	61
5	P5	max	0	47	0	47	0	47	0	47	5.926	55	2.505	59	.745	58	.193	61
6		min	-1.962	55	-2.716	55	-2.438	55	-12.368	57	0	47	-2.24	61	-1.377	59	-.198	60
7	P395	max	0	47	0	47	0	47	0	47	5.55	55	8.842	61	5.547	61	5.277	61
8		min	-3.334	57	-3.378	57	-4.732	55	-5.952	55	0	47	-10.115	59	-7.312	59	-6.341	59
9	P146	max	5.04	55	.137	58	0	47	.271	58	5.024	55	2.346	61	.697	61	.591	61
10		min	0	47	0	47	-10.551	55	0	47	0	47	-2.36	60	-.697	60	-.583	60
11	P393	max	0	47	0	47	0	47	.274	59	4.955	55	17.393	61	1.766	61	5.88	61
12		min	-4.558	57	-.137	55	-10.097	55	0	47	0	47	-20.583	59	-2.344	59	-6.474	59
13	P147	max	4.489	55	.203	55	0	47	.306	58	4.842	55	4.132	61	.597	61	.403	61
14		min	0	47	0	47	-12.125	55	0	47	0	47	-4.149	60	-.597	60	-.394	60
15	P144	max	3.635	55	3.612	55	0	47	0	47	4.819	55	5.264	60	6.99	60	4.563	60
16		min	0	47	0	47	-3.757	57	-6.03	55	0	47	-5.264	61	-6.976	61	-4.551	61
17	P392	max	0	47	0	47	0	47	.317	59	4.762	55	18.082	61	1.242	61	4.661	61
18		min	-4.055	57	-.189	59	-10.981	57	0	47	0	47	-21.61	59	-1.738	59	-4.913	59
19	P481	max	2.001	55	2.197	57	0	47	0	47	4.741	45	3.146	60	12.032	60	6.168	60
20		min	0	47	0	47	-1.913	57	-12.965	55	0	47	-3.148	61	-12.019	61	-6.152	61
21	P343	max	4.119	55	0	47	0	47	0	47	4.647	55	32.043	61	.537	61	3.64	61
22		min	0	47	-.214	55	-10.144	55	-.191	57	0	47	-36.5	59	-.717	59	-4.106	59
23	P196	max	0	47	.186	55	0	47	0	47	4.615	55	11.906	61	.469	61	1.068	61
24		min	-3.526	55	0	47	-10.624	55	-.208	55	0	47	-11.93	60	-.47	60	-1.065	60
25	P148	max	3.868	55	.238	55	0	47	.4	45	4.491	55	5.343	61	.495	61	.169	61
26		min	0	47	0	47	-12.982	55	0	47	0	47	-5.363	60	-.495	60	-.16	60
27	P391	max	0	47	0	47	0	47	.419	45	4.415	55	17.713	61	.771	61	3.826	61
28		min	-3.454	57	-.228	45	-11.305	57	0	47	0	47	-21.483	59	-1.195	59	-3.812	59
29	P344	max	3.241	45	0	47	0	47	0	47	4.377	55	38.749	61	1.371	61	5.436	61
30		min	0	47	-.983	55	-8.12	55	-.994	55	0	47	-44.233	59	-1.807	59	-6.34	59
31	P197	max	0	47	0	47	0	47	.193	45	4.367	55	10.85	61	.167	61	.686	61
32		min	-3.693	55	-.06	45	-11.982	55	0	47	0	47	-10.88	60	-.167	60	-.683	60
33	P342	max	4.215	55	.062	59	0	47	.199	45	4.345	55	27.134	61	.357	61	2.603	61
34		min	0	47	0	47	-10.848	57	0	47	0	47	-30.954	59	-.492	59	-2.868	59
35	P195	max	0	47	.981	55	0	47	0	47	4.253	55	13.401	61	.713	61	2.348	61
36		min	-2.677	45	0	47	-7.851	55	-1.019	55	0	47	-13.419	60	-.713	60	-2.347	60
37	P149	max	3.349	55	.213	55	0	47	.384	58	4.176	55	6.213	61	.415	61	.045	57
38		min	0	47	0	47	-13.246	55	0	47	0	47	-6.236	60	-.415	60	-.049	56
39	P198	max	0	47	0	47	0	47	.176	45	4.143	55	10.189	61	.164	61	.434	61
40		min	-3.79	55	-.074	58	-12.809	55	0	47	0	47	-10.225	60	-.164	60	-.43	60
41	P341	max	4.223	55	.077	45	0	47	.183	45	4.108	55	23.474	61	.22	61	1.883	61
42		min	0	47	0	47	-11.13	57	0	47	0	47	-26.951	59	-.341	59	-1.988	59
43	P390	max	0	47	0	47	0	47	.398	45	4.078	55	16.799	61	.441	61	3.239	61
44		min	-2.93	57	-.209	45	-11.07	57	0	47	0	47	-20.795	59	-.814	59	-3.226	60
45	P150	max	2.948	55	.191	55	0	47	.355	58	3.943	55	6.874	61	.344	61	.174	60
46		min	0	47	0	47	-13.062	55	0	47	0	47	-6.9	60	-.345	60	-.166	61
47	P199	max	0	47	0	47	0	47	.175	45	3.907	55	9.825	61	.132	61	.24	61
48		min	-3.867	55	-.07	58	-13.108	55	0	47	0	47	-9.868	60	-.132	60	-.236	60
49	P340	max	4.22	55	.069	45	0	47	.19	45	3.845	55	20.636	61	.114	61	1.429	61
50		min	0	47	0	47	-10.922	57	0	47	0	47	-23.993	59	-.218	59	-1.423	60

Envelope Plate Forces (per ft)

	Plate		Qx [k]	LC	Qy [k]	LC	Mx [k-ft]	LC	My [k-ft]	LC	Mxy [k-ft]	LC	Fx [k]	LC	Fy [k]	LC	Fxy [k]	LC
1	P398	max	4.177	57	0	47	0	47	0	47	0	47	1.346	60	.839	60	.987	60
2		min	0	47	-1.692	55	-10.121	55	-3.245	55	-7.271	55	-1.83	59	-.833	61	-.988	61
3	P91	max	0	47	1.43	55	0	47	0	47	0	47	17.707	60	1.261	60	8.109	59
4		min	-4.658	55	0	47	-9.691	55	-3.237	55	-6.818	55	-17.901	59	-4.505	59	-5.882	60
5	P447	max	2.023	55	0	47	0	47	0	47	0	47	.567	60	2.232	60	.967	60
6		min	0	47	-2.653	55	-2.514	55	-12.456	57	-6.18	55	-.582	61	-2.232	61	-.966	61
7	P397	max	3.197	57	0	47	0	47	0	47	0	47	.386	60	1.642	58	.484	59
8		min	0	47	-3.239	57	-4.883	55	-6.188	55	-5.835	55	-1.71	59	-1.112	61	-.263	58
9	P399	max	4.672	57	0	47	0	47	.267	45	0	47	1.687	60	.232	60	.996	60
10		min	0	47	-.127	55	-9.79	55	0	47	-5.233	55	-1.915	59	-.232	61	-1	61
11	P90	max	0	47	.146	58	0	47	.279	58	0	47	16.679	60	1.065	60	6.593	59
12		min	-5.046	55	0	47	-10.772	55	0	47	-5.074	55	-21.259	59	-2.391	59	-3.529	60
13	P400	max	4.235	57	0	47	0	47	.302	45	0	47	2.092	60	.19	60	1.097	60
14		min	0	47	-.185	45	-10.523	57	0	47	-5.069	55	-2.172	61	-.19	61	-1.101	61
15	P92	max	0	47	3.661	55	0	47	0	47	0	47	17.637	60	17.592	60	12.109	61
16		min	-3.679	55	0	47	-3.954	57	-6.104	55	-4.971	55	-17.689	61	-17.622	61	-12.089	60
17	P39	max	0	47	2.334	57	0	47	0	47	0	47	3.357	60	21.087	60	7.993	61
18		min	-2.025	55	0	47	-2.021	57	-13.101	55	-4.951	57	-3.36	61	-21.112	61	-8.001	60
19	P249	max	0	47	0	47	0	47	0	47	0	47	.328	60	.093	60	.476	60
20		min	-4.51	55	-.227	55	-9.849	55	-.21	57	-4.926	55	-2.072	59	-.094	61	-.478	61
21	P89	max	0	47	.201	55	0	47	.305	58	0	47	15.295	60	.634	60	5.001	59
22		min	-4.501	55	0	47	-12.29	55	0	47	-4.892	55	-22.304	59	-1.773	59	-2.444	60
23	P401	max	3.69	57	0	47	0	47	.419	45	0	47	2.593	60	.176	60	1.152	60
24		min	0	47	-.228	45	-10.77	57	0	47	-4.749	55	-2.674	61	-.177	61	-1.156	61
25	P140	max	3.611	55	.193	55	0	47	0	47	0	47	15.076	60	0	45	4.182	59
26		min	0	47	0	47	-10.844	55	-.21	55	-4.672	55	-37.482	59	-.73	59	-1.344	60
27	P250	max	0	47	.061	45	0	47	.199	45	0	47	1.14	60	.043	60	.47	60
28		min	-4.581	55	0	47	-10.405	57	0	47	-4.637	55	-2.148	59	-.043	61	-.472	61
29	P248	max	0	47	0	47	0	47	0	47	0	47	.312	61	.319	58	.24	60
30		min	-3.584	55	-1.005	55	-8.01	55	-1.047	55	-4.617	55	-1.997	59	-.27	61	-.245	61
31	P88	max	0	47	.237	55	0	47	.4	45	0	47	13.857	60	.296	60	3.877	59
32		min	-3.884	55	0	47	-13.099	55	0	47	-4.544	55	-22.175	59	-1.219	59	-1.876	60
33	P402	max	3.21	57	0	47	0	47	.398	45	0	47	3.163	60	.169	60	1.18	60
34		min	0	47	-.209	45	-10.513	57	0	47	-4.432	55	-3.244	61	-.17	61	-1.185	61
35	P251	max	0	47	.077	45	0	47	.183	45	0	47	1.979	60	.053	60	.462	60
36		min	-4.567	55	0	47	-10.605	57	0	47	-4.419	55	-2.231	59	-.053	61	-.464	61
37	P139	max	3.778	55	0	47	0	47	.193	45	0	47	12.94	60	.171	60	2.918	59
38		min	0	47	-.06	45	-12.149	55	0	47	-4.418	55	-31.839	59	-.502	59	-.886	60
39	P141	max	2.679	55	.999	55	0	47	0	47	0	47	18.465	60	.678	60	6.446	59
40		min	0	47	0	47	-8.077	55	-1.029	55	-4.318	55	-45.357	59	-1.824	59	-1.551	60
41	P87	max	0	47	.212	55	0	47	.378	58	0	47	12.487	60	.107	58	3.051	59
42		min	-3.372	55	0	47	-13.32	55	0	47	-4.233	55	-21.475	59	-.829	59	-1.524	60
43	P138	max	3.872	55	0	47	0	47	.176	45	0	47	11.505	60	.054	60	2.021	59
44		min	0	47	-.074	58	-12.927	55	0	47	-4.196	55	-27.768	59	-.348	59	-.554	60
45	P252	max	0	47	.069	45	0	47	.19	45	0	47	2.829	60	.053	60	.459	60
46		min	-4.548	55	0	47	-10.373	57	0	47	-4.171	55	-2.936	61	-.053	61	-.461	61
47	P403	max	2.809	57	0	47	0	47	.335	45	0	47	3.776	60	.162	60	1.194	60
48		min	0	47	-.193	45	-9.862	57	0	47	-4.166	55	-3.856	61	-.163	61	-1.199	61
49	P86	max	0	47	.19	55	0	47	.348	58	0	47	11.236	60	.044	58	2.442	59
50		min	-2.978	55	0	47	-13.102	55	0	47	-4.003	55	-20.538	59	-.571	59	-1.317	60



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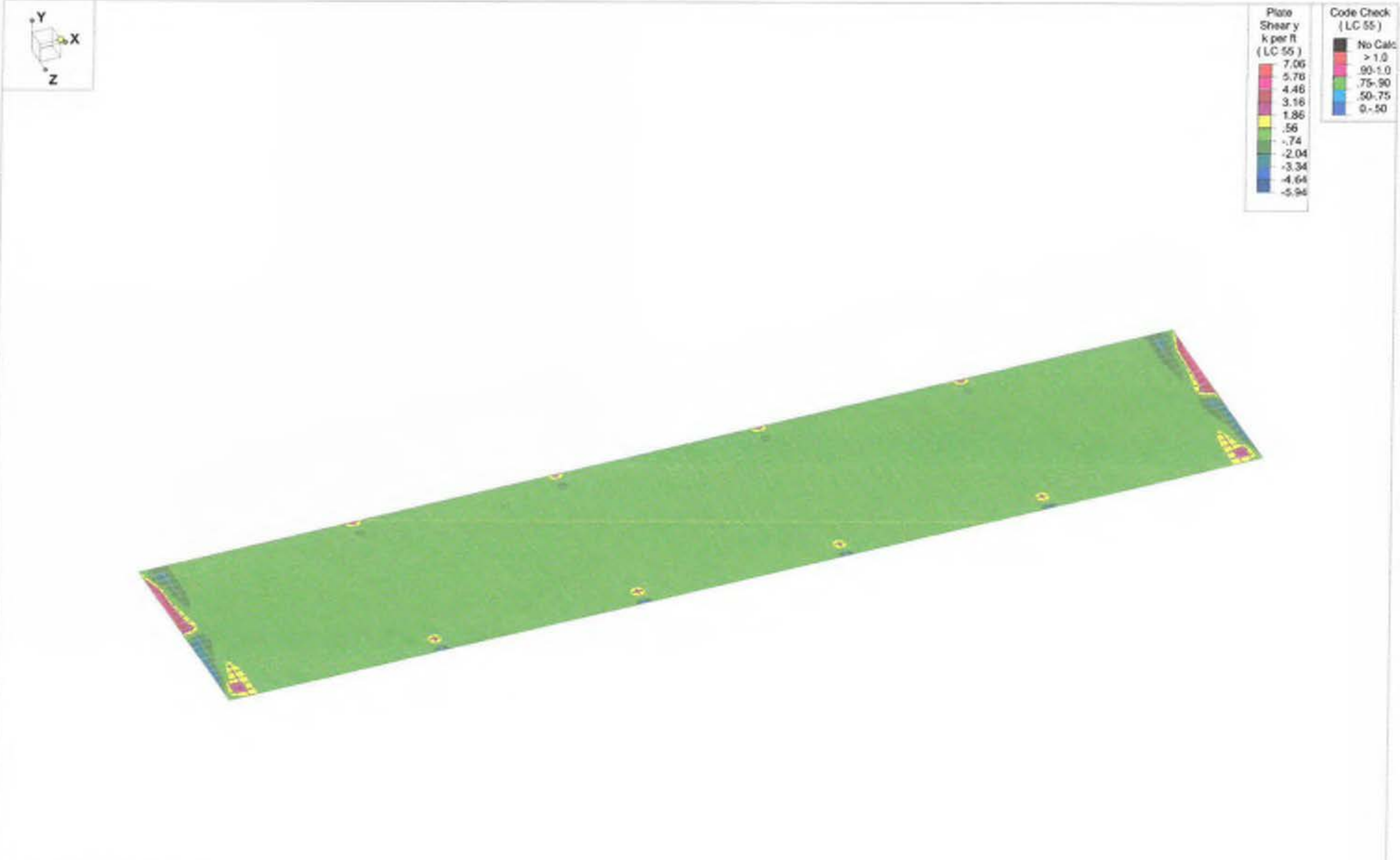
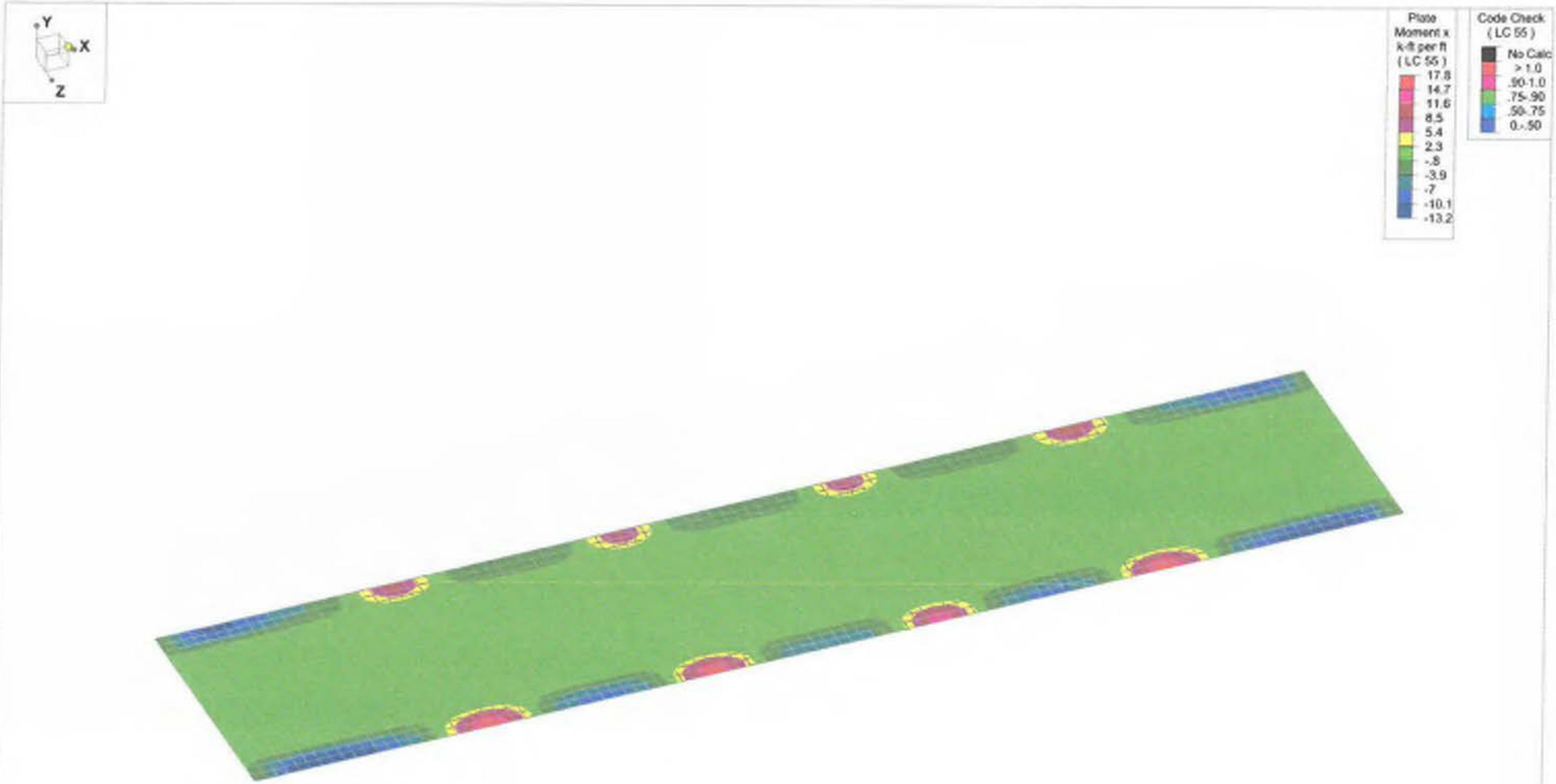
18-183B		EQUIP STORAGE BLDG Rev_0 9.21.18.r3d

SHEET C42



Member Code Checks Displayed
Loads: BLC 10, -GCpi
Results for LC 55, COMB1
Member y Shear Forces (k)

SMG ENGINEERS	EQUIPMENT STORAGE BUILDING	SK -
BS		Sept 28, 2018 at 6:45 PM
18-183B		EQUIP STORAGE BLDG Rev_0 9.21.18.r3d



Member Code Checks Displayed
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Results for LC 55, COMB1
Member y Shear Forces (k)

SMG ENGINEERS	EQUIPMENT STORAGE BUILDING	SK -
BS		Sept 28, 2018 at 6:45 PM
18-183B		EQUIP STORAGE BLDG Rev_0 9.21.18.r3d

SHEET C44

Member Code Checks Displayed
Loads: BLC 10, -GCpi
Results for LC 55, COMB1
Member y Shear Forces (k)

SMG ENGINEERS	EQUIPMENT STORAGE BUILDING	SK -
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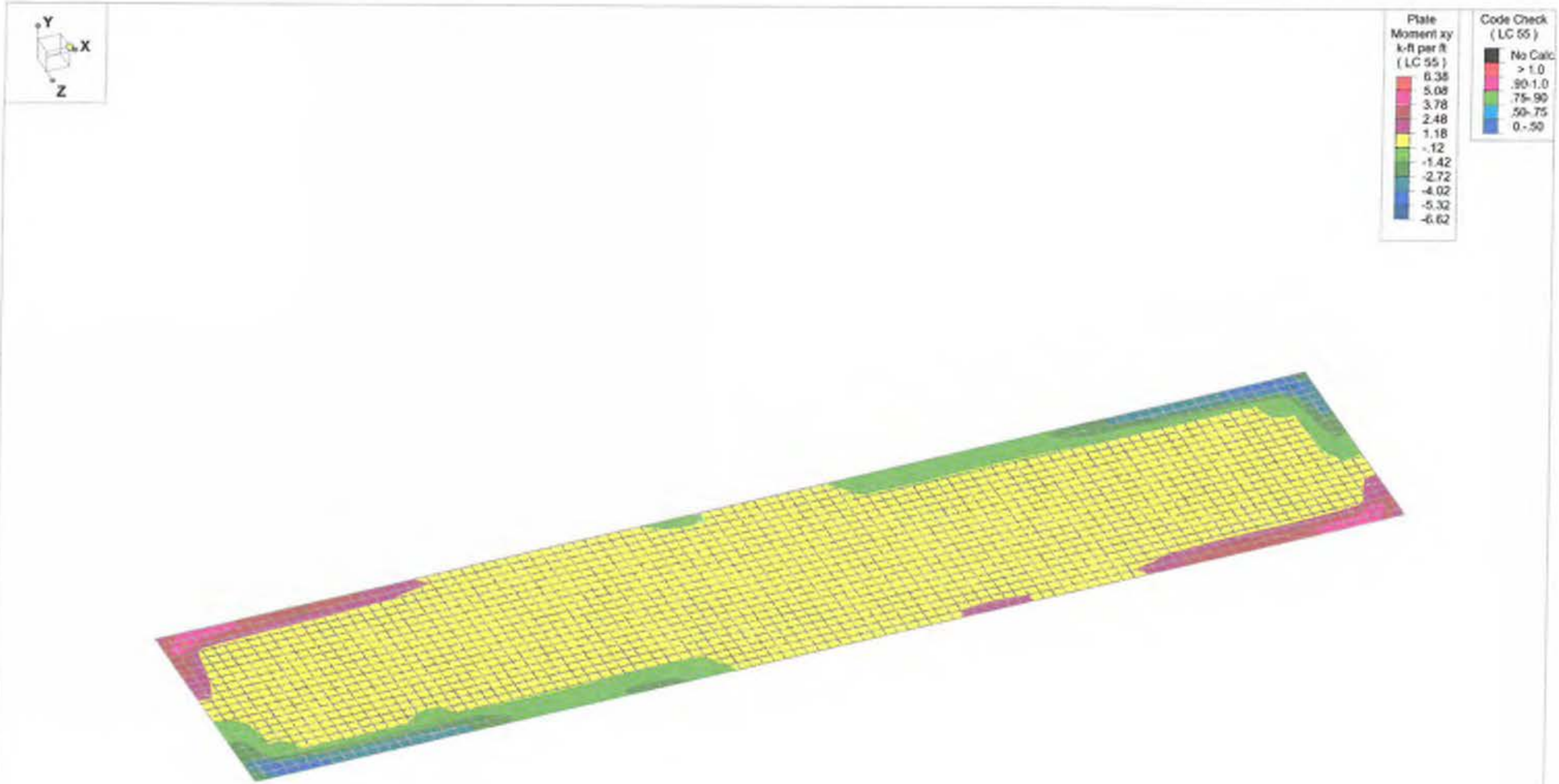
18-183B		EQUIP STORAGE BLDG Rev_0 9.21.18.r3d



Member Code Checks Displayed
Loads: BLC 10, -GCpi
Results for LC 55, COMB1
Member y Shear Forces (k)

SMG ENGINEERS	EQUIPMENT STORAGE BUILDING	SK -
BS		Sept 28, 2018 at 6:46 PM
18-183B		EQUIP STORAGE BLDG Rev_0 9.21.18.r3d

SHEET C46

Company:
Specifier:
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Sub-Project I Pos. No.:
Date: 9/28/2018

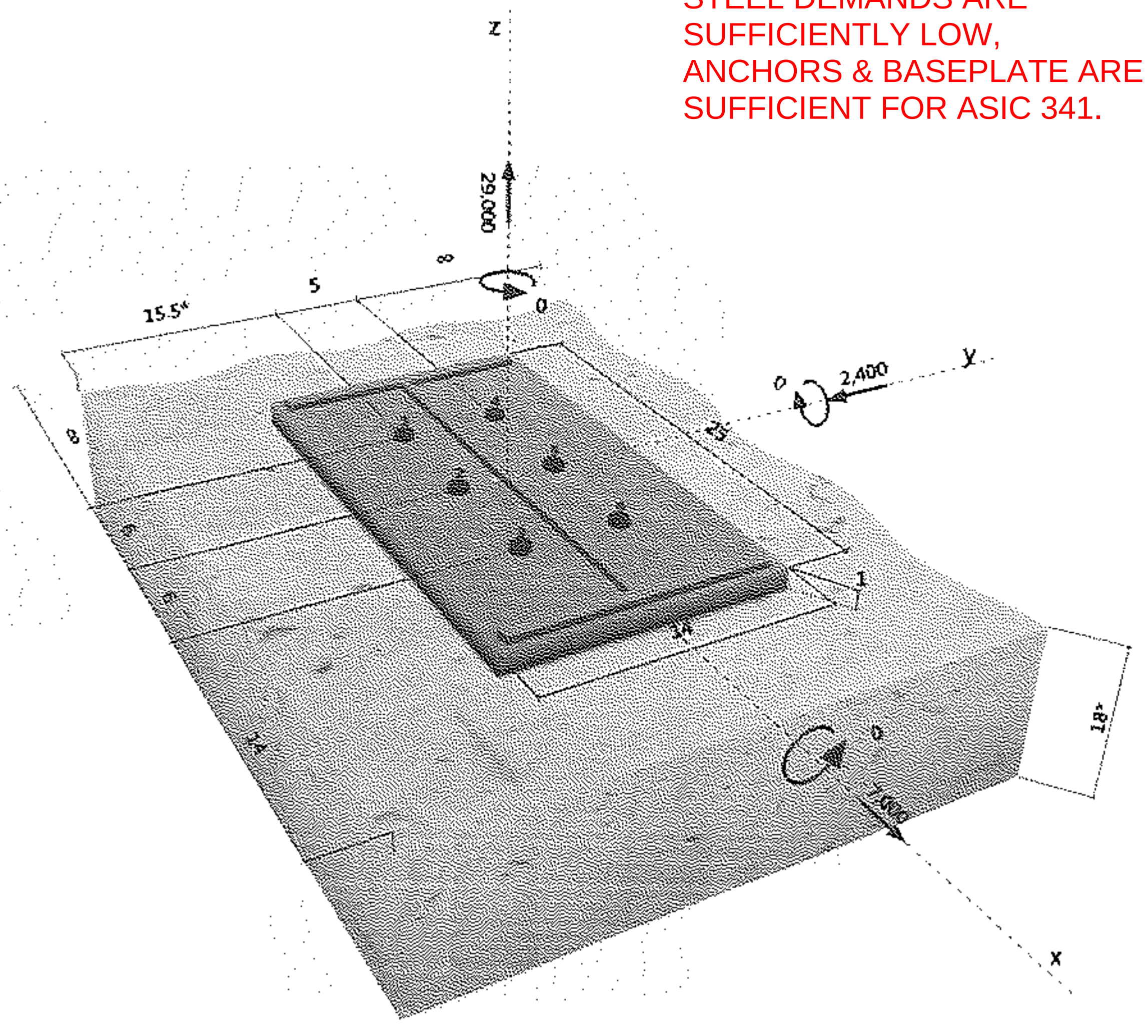
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 + HAS-E 1**
Effective embedment depth:	$h_{ef,act}$ = 10.000 in. ($h_{ef,limit}$ = - in.)
Material:	5.8
Evaluation Service Report:	ESR-3187
Issued I Valid:	11/1/2016 | 3/1/2018
Proof:	Design method ACI 318-14 / Chem
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate:	l_x x l_y x t = 25.000 in. x 14.000 in. x 1.000 in.; (Recommended plate thickness: not calculated
Profile:	W shape (AISC); (L x W x T x FT) = 24.100 in. x 12.800 in. x 0.500 in. x 0.750 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 18.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

Geometry [in.] & Loading [lb, in.lb]

STEEL DEMANDS ARE SUFFICIENTLY LOW, ANCHORS & BASEPLATE ARE SUFFICIENT FOR ASIC 341.

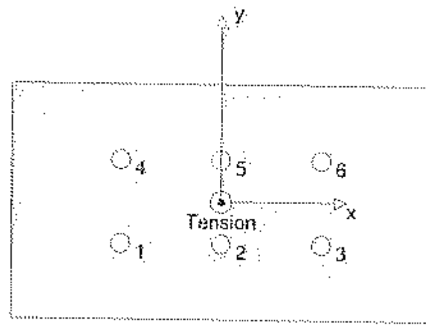


2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	4,833	1,233	1,167	-400
2	4,833	1,233	1,167	-400
3	4,833	1,233	1,167	-400
4	4,833	1,233	1,167	-400
5	4,833	1,233	1,167	-400
6	4,833	1,233	1,167	-400



max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 29,000 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	4,833	28,541	17	OK
Bond Strength**	29,000	43,943	66	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	29,000	34,532	84	OK

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3187
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	72,500

Calculations

N_{sa} [lb]
43,910

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
43,910	0.650	28,541	4,833

3.2 Bond Strength

$N_{ag} = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ec1,Na}\, \psi_{ec2,Na}\, \psi_{ed,Na}\, \psi_{cp,Na}\, N_{ba}$ ACI 318-14 Eq. (17.4.5.1.b)

$\phi\ N_{ag} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Na} = see ACI 318-14, Section 17.4.5.1, Fig. R 17.4.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-14 Eq. (17.4.5.1c)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ ACI 318-14 Eq. (17.4.5.1d)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e_N}{c_{Na}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.5.3)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.5.4b)

$\psi_{cp,Na} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.5.5b)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-14 Eq. (17.4.5.2)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{k,c}$ [psi]
2,327	1.000	10.000	14.000	1,326

$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	c_{ac} [in.]	λ_a
0.000	0.000	20.543	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
14.478	1,374.52	838.50	0.990

$\psi_{ec1,Na}$	$\psi_{ec2,Na}$	$\psi_{cp,Na}$	N_{ba} [lb]
1.000	1.000	1.000	41,654

Results

N_{ag} [lb]	ϕ_{bond}	$\phi\ N_{ag}$ [lb]	N_{ua} [lb]
67,604	0.650	43,943	29,000

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3.3 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b$ ACI 318-14 Eq. (17.4.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1+\frac{2\, e_N}{3\, h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\, \lambda_a \sqrt{f'_c}\, h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
10.000	0.000	0.000	14.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
20.543	17	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,435.00	900.00	1.000	1.000	0.980	1.000	34,000

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
53,127	0.650	34,532	29,000

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	1,233	15,807	8	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Concrete Breakout Strength controls)**	7,400	74,378	10	OK
Concrete edge failure in direction x+**	7,400	17,593	43	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = (0.6\ A_{se,V}\ f_{uta})$ refer to ICC-ES ESR-3187

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$(0.6\ A_{se,V}\ f_{uta})$ [lb]
0.61	72,500	26,345

Calculations

V_{sa} [lb]
26,345

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
26,345	0.600	15,807	1,233

4.2 Pryout Strength (Concrete Breakout Strength controls)

$V_{cpg} = k_{cp}\left[\left(\frac{A_{Nc}}{A_{Nc0}}\right)\psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b\right]$ ACI 318-14 Eq. (17.5.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	10.000	0.000	0.000	14.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	20.543	17	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,435.00	900.00	1.000	1.000	0.980	1.000	34,000

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
106,254	0.700	74,378	7,400

4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\, \psi_{ed,V}\, \psi_{c,V}\, \psi_{h,V}\, \psi_{parallel,V}\, V_b$ — ACI 318-14 Eq. (17.5.2.1b)

$\phi\ V_{cbg} \geq V_{ua}$ — ACI 318-14 Table 17.3.1.1

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5\, c_{a1}^2$ — ACI 318-14 Eq. (17.5.2.1c)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e_v}{3c_{a1}}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.5.2.5)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.5.2.6b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-14 Eq. (17.5.2.8)

$V_b = 9\, \lambda_a\, \sqrt{f'_c}\, c_{a1}^{1.5}$ — ACI 318-14 Eq. (17.5.2.2b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
14.000	15.500	0.000	1.000	18.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
8.000	1.000	1.000	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
747.00	882.00	1.000	0.921	1.080	29,817

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
25,133	0.700	17,593	7,400

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.840	0.421	5/3	99	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-14, Section 17.8.1.

Fastening meets the design criteria!

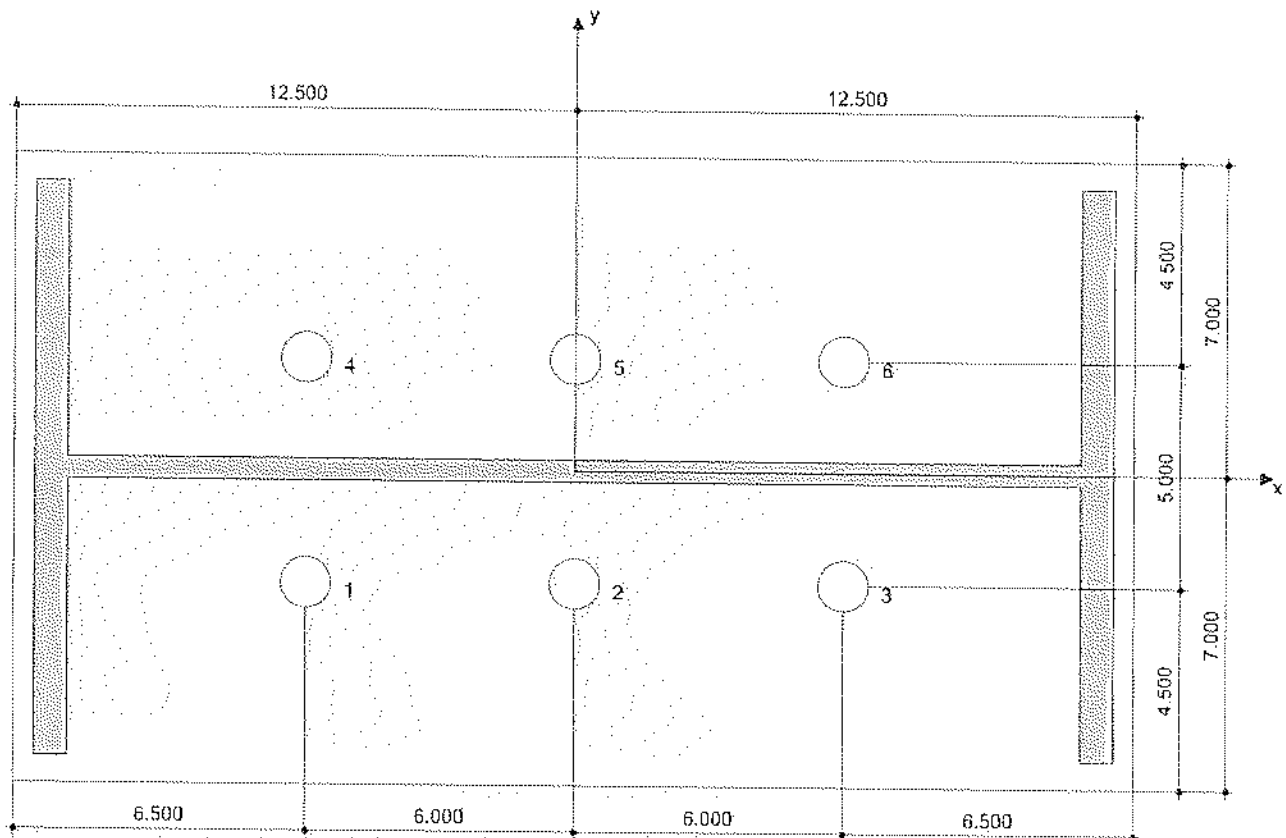
7 Installation data

Anchor plate, steel: -
Profile: W shape (AISC); 24.100 x 12.800 x 0.500 x 0.750 in.
Hole diameter in the fixture: d_f = 1.125 in.
Plate thickness (input): 1.000 in.
Recommended plate thickness: not calculated
Drilling method: Hammer drilled
Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-HY 200 + HAS-E 1
Installation torque: 1,800.003 in.lb
Hole diameter in the base material: 1.125 in.
Hole depth in the base material: 10.000 in.
Minimum thickness of the base material: 12.250 in.

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-6.000	-2.500	-	26.000	15.500	-
2	0.000	-2.500	-	20.000	15.500	-
3	6.000	-2.500	-	14.000	15.500	-

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
4	-6.000	2.500	-	26.000	20.500	-
5	0.000	2.500	-	20.000	20.500	-
6	6.000	2.500	-	14.000	20.500	-

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8 Remarks; Your Cooperation Duties

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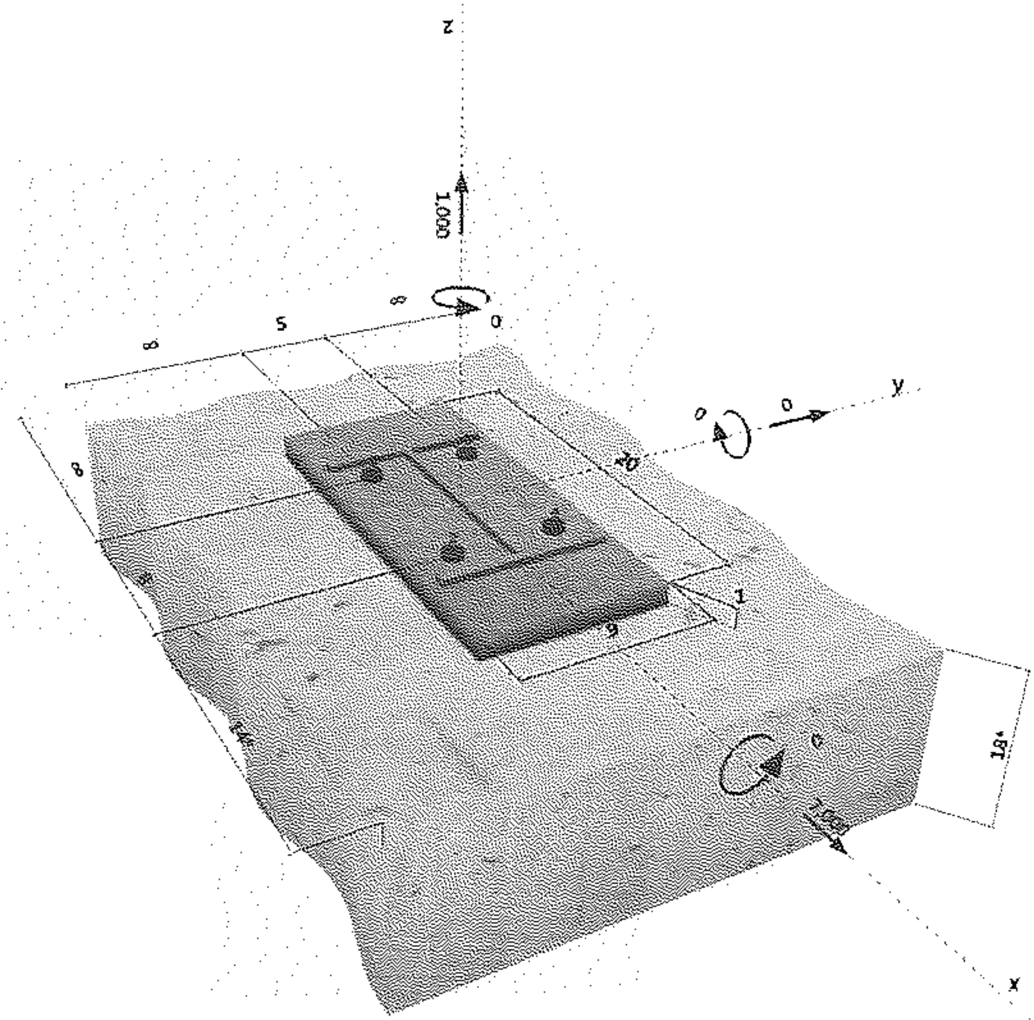
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 + HAS-E 1**
Effective embedment depth:	$h_{ef,act}$ = 10.000 in. ($h_{ef,limit}$ = - in.)
Material:	5.8
Evaluation Service Report:	ESR-3187
Issued I Valid:	11/1/2016 | 3/1/2018
Proof:	Design method ACI 318-11 / Chem
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate:	l_x x l_y x t = 20.000 in. x 9.000 in. x 1.000 in.; (Recommended plate thickness: not calculated
Profile:	W shape (AISC); (L x W x T x FT) = 11.900 in. x 8.010 in. x 0.295 in. x 0.515 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 18.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

Geometry [in.] & Loading [lb, in.lb]



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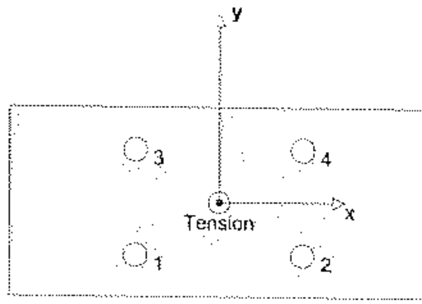
2 Load case/Resulting anchor forces

Load case: Design loads



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	250	1,750	1,750	0
2	250	1,750	1,750	0
3	250	1,750	1,750	0
4	250	1,750	1,750	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 1,000 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	250	28,541	1	OK
Bond Strength**	1,000	39,600	3	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	1,000	31,163	4	OK

* anchor having the highest loading **anchor group (anchors in tension)

DEMAND OK BY INSPECTION FOR INCREASED SEISMIC PER AISC 341

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3187
ϕ $N_{sa} \geq N_{ua}$ ACI 318-11 Table D.4.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	72,500

Calculations

N_{sa} [lb]
43,910

Results

N_{sa} [lb]	ϕ_{steel}	ϕ N_{sa} [lb]	N_{ua} [lb]
43,910	0.650	28,541	250

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3.2 Bond Strength

$N_{ag} = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ec1,Na}\ \psi_{ec2,Na}\ \psi_{ed,Na}\ \psi_{cp,Na}\ N_{ba}$ ACI 318-11 Eq. (D-19)

$\phi\ N_{ag} \geq N_{ua}$ ACI 318-11 Table D.4.1.1

A_{Na} = see ACI 318-11, Part D.5.5.1, Fig. RD.5.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-11 Eq. (D-20)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ ACI 318-11 Eq. (D-21)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e_N}{c_{Na}}}\right) \leq 1.0$ ACI 318-11 Eq. (D-23)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$ ACI 318-11 Eq. (D-25)

$\psi_{cp,Na} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ ACI 318-11 Eq. (D-27)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-11 Eq. (D-22)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{k,c}$ [psi]
2,327	1.000	10.000	14.000	1,326

$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	c_{ac} [in.]	λ_a
0.000	0.000	20.543	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
14.478	1,238.69	838.50	0.990

$\psi_{ec1,Na}$	$\psi_{ec2,Na}$	$\psi_{cp,Na}$	N_{ba} [lb]
1.000	1.000	1.000	41,654

Results

N_{ag} [lb]	ϕ_{bond}	$\phi\ N_{ag}$ [lb]	N_{ua} [lb]
60,924	0.650	39,600	1,000

3.3 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b$	ACI 318-11 Eq. (D-4)
$\phi\ N_{cbg} \geq N_{ua}$	ACI 318-11 Table D.4.1.1
A_{Nc} see ACI 318-11, Part D.5.2.1, Fig. RD.5.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-11 Eq. (D-5)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N}{3\ h_{ef}}}\right) \leq 1.0$	ACI 318-11 Eq. (D-8)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-11 Eq. (D-10)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-11 Eq. (D-12)
$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$	ACI 318-11 Eq. (D-6)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
10.000	0.000	0.000	14.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
20.543	17	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,295.00	900.00	1.000	1.000	0.980	1.000	34,000

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
47,944	0.650	31,163	1,000

Company:
Specifier:
Address:
Phone I Fax: |
E-Mail:

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	1,750	15,807	12	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Concrete Breakout Strength controls)**	7,000	67,121	11	OK
Concrete edge failure in direction x+**	7,000	21,624	33	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = (0.6\, A_{se,V}\, f_{uta})$ refer to ICC-ES ESR-3187
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-11 Table D.4.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$(0.6\, A_{se,V}\, f_{uta})$ [lb]
0.61	72,500	26,345

Calculations

V_{sa} [lb]
26,345

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
26,345	0.600	15,807	1,750

4.2 Pryout Strength (Concrete Breakout Strength controls)

$V_{cpg} = k_{cp}\left[\left(\frac{A_{Nc}}{A_{Nc0}}\right)\psi_{ec,N}\,\psi_{ed,N}\,\psi_{c,N}\,\psi_{cp,N}\,N_b\right]$ ACI 318-11 Eq. (D-41)
$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-11 Table D.4.1.1
A_{Nc} see ACI 318-11, Part D.5.2.1, Fig. RD.5.2.1(b)
$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-11 Eq. (D-5)
$\psi_{ec,N} = \left(\frac{1}{1+\frac{2\,e_N}{3\,h_{ef}}}\right) \leq 1.0$ ACI 318-11 Eq. (D-8)
$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-11 Eq. (D-10)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-11 Eq. (D-12)
$N_b = k_c\,\lambda_a\,\sqrt{f'_c}\,h_{ef}^{1.5}$ ACI 318-11 Eq. (D-6)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	10.000	0.000	0.000	14.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	20.543	17	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,295.00	900.00	1.000	1.000	0.980	1.000	34,000

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
95,888	0.700	67,121	7,000

4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ — ACI 318-11 Eq. (D-31)

$\phi\ V_{cbg} \geq V_{ua}$ — ACI 318-11 Table D.4.1.1

A_{Vc} see ACI 318-11, Part D.6.2.1, Fig. RD.6.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ — ACI 318-11 Eq. (D-32)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e_V}{3c_{a1}}}\right) \leq 1.0$ — ACI 318-11 Eq. (D-36)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-11 Eq. (D-38)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-11 Eq. (D-39)

$V_b = 9\ \lambda_a\ \sqrt{f'_c}\ c_{a1}^{1.5}$ — ACI 318-11 Eq. (D-34)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
14.000	-	0.000	1.000	18.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
8.000	1.000	1.000	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
846.00	882.00	1.000	1.000	1.080	29,817

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
30,892	0.700	21,624	7,000

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.032	0.324	5/3	16	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-11, Part D.9.1

Fastening meets the design criteria!

Company:
Specifier:
Address:
Phone I Fax: |
E-Mail:

Page: 7
Project:
Sub-Project I Pos. No.:
Date: 9/28/2018

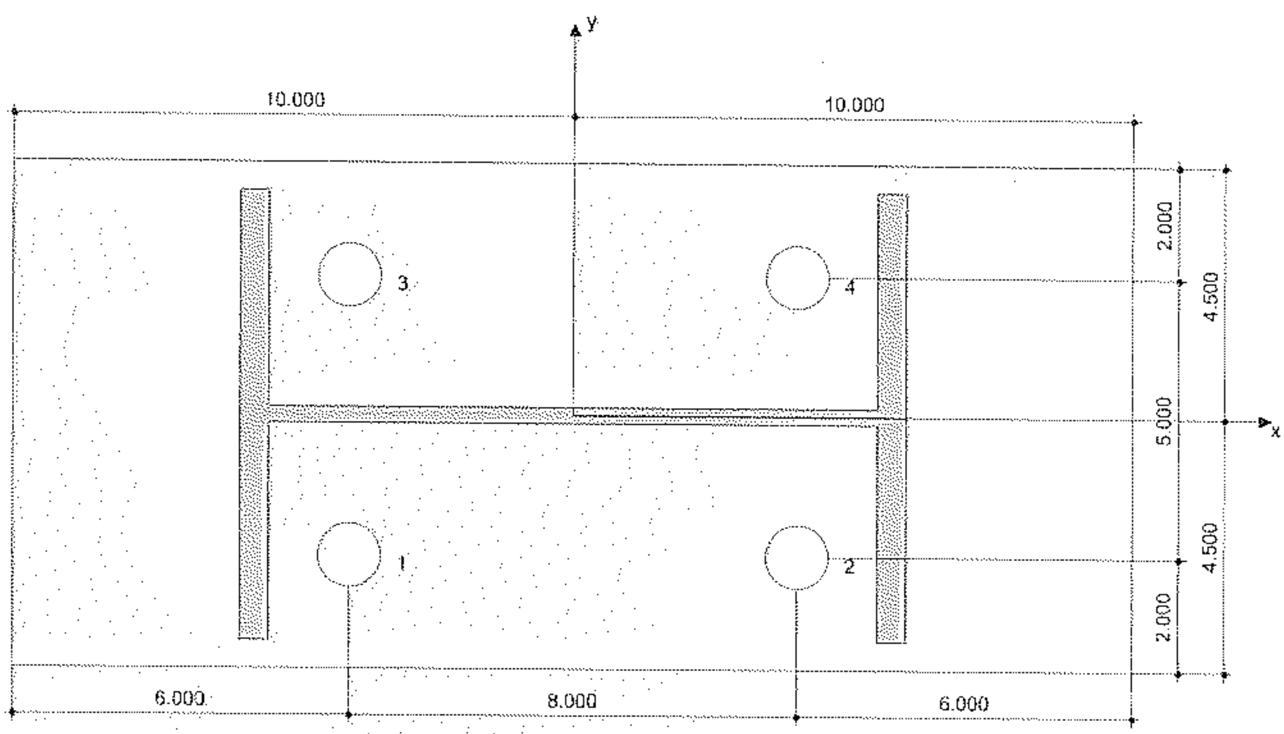
7 Installation data

Anchor plate, steel: -
Profile: W shape (AISC); 11.900 x 8.010 x 0.295 x 0.515 in.
Hole diameter in the fixture: d_f = 1.125 in.
Plate thickness (input): 1.000 in.
Recommended plate thickness: not calculated
Drilling method: Hammer drilled
Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-HY 200 + HAS-E 1
Installation torque: 1,800.003 in.lb
Hole diameter in the base material: 1.125 in.
Hole depth in the base material: 10.000 in.
Minimum thickness of the base material: 12.250 in.

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-4.000	-2.500	-	22.000	-	-
2	4.000	-2.500	-	14.000	-	-
3	-4.000	2.500	-	22.000	-	-
4	4.000	2.500	-	14.000	-	-

Company:		Page:	8
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	9/28/2018
E-Mail:			

8 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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CLIENT

PROJECT

BY DATE REV.

JOB NO. SHEET OF

Secondary Containment

Volume = $5' \cdot (36' \times 36') = 6480\ ft^3$

Net Volume = $6480\ ft^3 - 3 \cdot (\pi \cdot (12')^2/4) \cdot 5' = 4783\ ft^3$

$1.1 \times 30{,}000$ GAL = 33,000 GAL → 4411 ft^3 OK ✓

Containment Volume is Sufficient for 110% of Tank Volume

SECONDARY CONTAINMENT STRUCTURE IS CAPABLE OF CONTAINING A SINGLE 30,000 GAL TANK IN THE EVENT OF A LEAK/FAILURE.

Governing code: 2012 IBC <- 2015 IBC , CALCS ARE IN COMPLIANCE)
- ASCE 7-10
- AWWA D100-5

Design criteria:

DL:
- $1246+[(\pi(5.75^2-5.73^2))44]490 = 17144$ LBS
- SW of Conc. 150 pcf

LL:
- 30,000 gal (4) AC Tank
  - $8.56(30000) = 256.8$ kips

WIND CALC IS CONSERVATIVE, RISK II STRUCTURE & 110 mph WIND, ACTUAL

WL: wind speed V = 115 MPH; Exposure C; Risk III

$q_z = 0.00256\, k_z k_{zt} k_d V^2$ [ASCE 7-10 Eq 29.3-1]

where:

$k_d = 0.95$ [ASCE 7-10 Tb 26.6-1]

$k_z = 1.065$ [ASCE 7-10 Tb 29.3-1]

$k_{zt} = 1$ [ASCE 7-10 26.8.2]

$q_z = 0.00256(1.065)(1)(0.95)(115)^2 = 34.3$ Psi

$h/D \rightarrow 45/11.5 = 3.91$

$C_f = 0.549$ [ASCE 7-10 Fig 29.5-1]

$F = q_z G C_f A_f$ [ASCE 7-10 Eq 29.5-1]

$G = 0.85$ [ASCE 7-10 26.9]

$A_f = 11.5(44.04) = 507$ Ft^2

$F = 34.3(0.85)(0.549)(507) = 8.12$ kips

AWWA D100-5 ⟶ EQ:

$M_s = \sqrt{[A_i(W_s x_s + W_r H_t + W_i x_i)]^2 + [A_c W_c x_c]^2}$ [EQ 13-23]

$A_I = \frac{S_{aI} I_E}{1.4 R_i} \geq \frac{0.36 S_1 I_E}{R_i}$ (EQ 13-17)

$S_{aI} = S_{DS} = 0.884g$

$I_E = 1$ (Table 24) $R_i = 3$

$S_1 = 0.495$

EQ Continued:

$$A_i = \frac{0.884(1)}{1.4(3)} = 0.210 \geq 0.0594 = \frac{0.36(.495)(1)}{3}$$

$$W_s = 17144 \text{ LBS}$$

$$X_s = 22 \text{ ft}$$

*Assume 45 PSF $\quad W_r = \pi r^2(45) \rightarrow \pi(5.75)^2(45) = 4674 \text{ LBS}$

$$H_T = 44 \text{ Ft}$$

[EQ 13-25]

$$D/H = 11.5/44 = 0.261 \therefore W_i = \left[1.0 - 0.218\frac{D}{H}\right]W_T$$

$$W_i = [1 - 0.218(0.261)]\,256.8 = 242.2 \text{ kips}$$

$$X_i = \left[0.5 - 0.094\frac{D}{H}\right]H \quad \text{[EQ 13-29]}$$

$$X_i = [0.5 - 0.094(0.261)]\,44 = 20.9 \text{ Ft}$$

$$A_c = \frac{S_{ac} I_E}{1.4 R_c} \quad \text{[EQ 13-18]}$$

$$T_c = 2\pi\sqrt{\frac{D}{3.68g \,\text{Tanh}\left(\frac{3.68H}{D}\right)}} \quad \text{[EQ 13-22]}$$

$$T_c = 2\pi\sqrt{\frac{11.5}{3.68\,\text{Tanh}\left(\frac{3.68(44)}{11.5}\right)}} = 11.107s < 16s$$

$$\therefore S_{AC} = \frac{kS_{D1}}{T_c} \rightarrow \frac{1(0.497)}{11.11} = 0.045$$

$$R_c = 1.5 \quad \text{[Table 28]}$$

$$A_c = \frac{0.045(1)}{1.4(1.5)} = 0.0214$$

$$W_c = 0.230\frac{D}{H}\tanh\left(\frac{3.67H}{D}\right)W_T \quad \text{(EQ 13-16)}$$

$$W_c = 0.230\frac{11.5}{44}\tanh\left(\frac{3.67(44)}{11.5}\right)256800 = 15437 \text{ LBS}$$

[EQ 13-30] $$X_c = \left[1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1}{\frac{3.67H}{D}\sinh\left(\frac{3.67H}{D}\right)}\right]H \rightarrow \left[1.0 - \frac{\cosh\left(\frac{3.67(44)}{11.5}\right) - 1}{\frac{3.67(44)}{11.5}\sinh\left(\frac{3.67(44)}{11.5}\right)}\right](44)$$

$$X_c = 40.9 \text{ Ft}$$

EQ continued:

$$M_S = \sqrt{[0.21(17144(22) + 4674(44) + 242(20.9))]^2 + [0.0214(15437)(40.9)]^2}$$

↳ 242 kips

moment @ base of shell → $M_S = 1{,}185$ kip·ft

$$M_{mf} = \sqrt{[A_i(W_S X_S + W_r H_t + W_i X_{imf})]^2 + [A_c W_c X_{cmf}]^2} \quad \text{[EQ 13-32]}$$

$$X_{imf} = [0.5 + 0.06\, D/H]H \quad \text{[EQ 13-34]} \rightarrow [0.5 + 0.06\, 11.5/44]44 = 22.69 \text{ ft}$$

$$X_{cmf} = \left[1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1.937}{\frac{3.67H}{D}\sinh\left(\frac{3.67H}{D}\right)}\right]H \quad \text{[EQ 13-35]}$$

$$X_{cmf} = \left[1.0 - \frac{\cosh\left(\frac{3.67(44)}{11.5}\right) - 1.937}{\frac{3.67(44)}{11.5}\sinh\left(\frac{3.67(44)}{11.5}\right)}\right](44) = 40.9 \text{ ft}$$

$$M_{mf} = \sqrt{[0.21(17144(22) + 4674(44) + 242200(22.69))]^2 + [0.0214(15437)(40.9)]^2}$$

moment @ Top of Foundation → $M_{mf} = 1{,}277$ k·ft

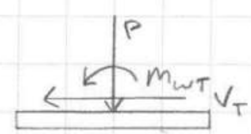
$$V_F = \sqrt{[A_i(W_S + W_r + W_F + W_i)]^2 + [A_c W_c]^2}$$

$$W_i = \pi(11.5)^2/4\,(0.25/12)\,490 = 1060 \text{ LBS}$$

$$V_F = \sqrt{[0.21(17144 + 4674 + 1060 + 242200)]^2 + [0.0214(15437)]^2} = 55.7 \text{ kips}$$

Check overturning:

Wind: $V_T = 4(8.12)(0.6) = 19.5$ kips



$M_W = 8.12(22)(0.6) = 107.2$ k·ft

$P = 4(17144 + 4674 + 1060)(0.6) = 54.9$ kips

$M_{WT} = 4(107.2) = 429$ k·ft

(See Spreadsheet)

Smith Monroe Gray
ENGINEERS, INC.

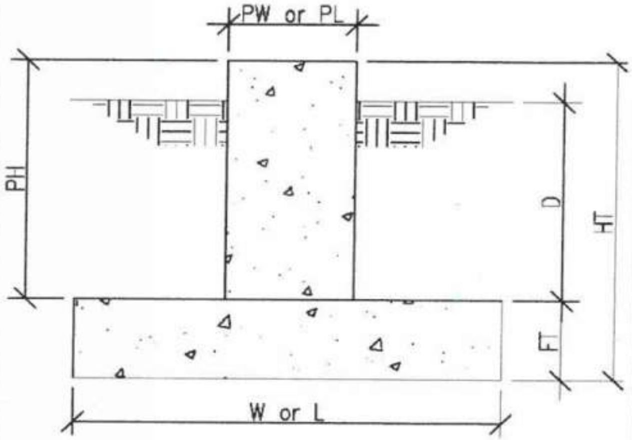
CLIENT:
PROJECT:
BY:
DATE: 8/13/2018
JOB #:
SHEET 4 OF 9

DESIGN OF RECTANGULAR FOOTING WITH OVERTURNING MOMENT

FOOTING:

LOADING PARAMETERS:

ALLOWABLE SOIL BEARING =	3,000	PSF
SOIL WEIGHT =	115	PCF
REQD. O.T. SAFETY FACTOR =	1.5	
STR.INCR.FOR HORIZ. LOADS =	1.33	
VERTICAL DEAD LOAD =	54.90	KIPS
VERTICAL LIVE LOAD =	0	KIPS
HORIZONTAL LOAD =	19.5	KIPS
MOMENT @ TOP OF FOOTING =	429	FT-KIPS



FOOTING DIMENSIONS:

FTG. LENGTH (L) =	37.0	FT (PAR.TO LOAD)
FTG. WIDTH (W) =	37.0	FT (PERP.TO LOAD)
FTG. THICKNESS (FT) =	2.00	FT
FOOTING DEPTH (D) =	0.0	FT
PIER LENGTH (PL) =	0.0	FT
PIER WIDTH (PW) =	0.0	FT
PIER HEIGHT (PH) =	0.0	FT
CONCRETE WEIGHT =	246.4	KIPS
SOIL WEIGHT =	0.0	KIPS
TOTAL WEIGHT =	246.4	KIPS

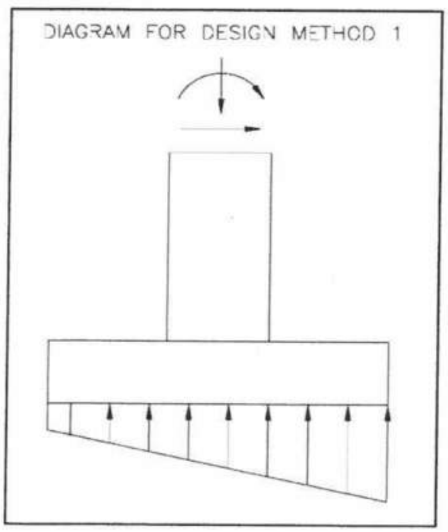
DESIGN METHOD 1

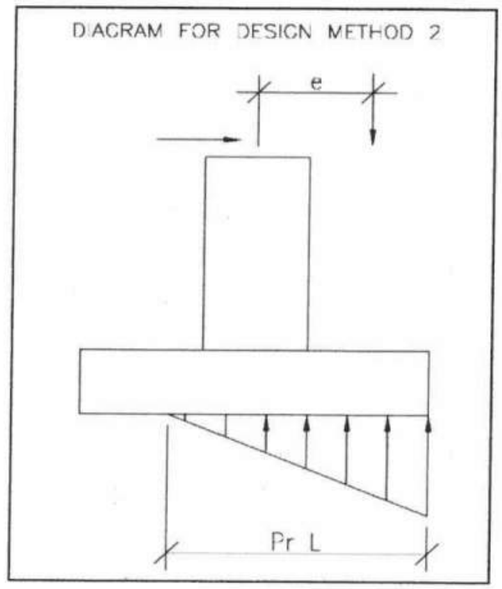
OVERTURNING MOM. =	468.0	FT-KIPS	
SOIL PR. FROM DL =	220.1	PSF	
SOIL PR. FROM MOM. =	(55.4)	PSF	
MIN. PRESSURE =	164.7	PSF	
MAX. PRESSURE =	275.5	PSF	**GOVERNS**

DESIGN METHOD 2

e =	1.55	FT
Pr L =	50.84	FT
MAX. PR =	320.4	PSF

DOES NOT APPLY AS NO UPLIFT AT BACK OF FOOTING





	ACTUAL		ALLOWABLE		
LL + DL BEARING =	220	PSF	3,000	PSF	OK
DL + HORIZ. BEARING =	276	PSF	3,990	PSF	OK
F.S. OF OVERTURNING =	11.91		1.5		OK

Check for Sliding:

Wind:

$FS_{sliding} = \frac{Resisting}{Acting} > 1.5 \rightarrow \frac{(54.9+246)(0.5) + 1/2(350)(2)^2(37)/1000}{1/2(35)(2)^2(37)/1000 + 19.5}$

$= 7.98 > 1.5$ (OK)

Check overturning:

Seismic:

$M_T = 1,277(4)(0.7) = 3576$ k·ft

$V_T = 55.7(4)(0.7) = 156$ kips

$P_{T_{LL}} = [(256.8)4 + 91.5](0.6) = 671.22$ kips

(See spreadsheet)

Check for sliding:

Seismic:

$FS_{sliding} = \frac{Resisting}{Acting} > 1.5 \rightarrow \frac{(54.9 + 246 + 616)0.5 + 1/2(350)(2)^2(37)/1000}{1/2(35)(2)^2(37)/1000 + 156}$

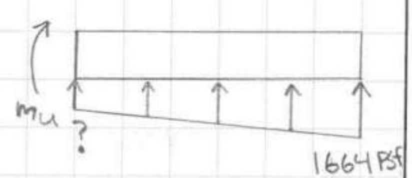
$= 3.05 > 1.5$ (OK)

Check Bending of Slab: (LRFD) 0.9DL + 1.0E

$M_T = 1,277(4) = 5,109$ k·ft

$V_T = 55.7(4) = 223$ kips

$P_T = [256.8(4) + 91.5]0.9 = 1007$ kips



$M_{Total} = 223(2) + 5,109 = 5555$ k·ft

$P_{Total} = 1007 + 410.7(0.9) = 1377$ kips

$\sigma_{max} = \frac{P}{A} + \frac{Mc}{I} \rightarrow \frac{1377(1000)}{37^2} + \frac{5555(1000)}{37(37)^2/6} = 1664$ PSF

$\sigma_{min} = \frac{P}{A} - \frac{Mc}{I} \rightarrow \frac{1377(1000)}{37^2} - \frac{5555(1000)}{37(37)^2/6} = 348$ PSF

Smith Monroe Gray
ENGINEERS, INC.

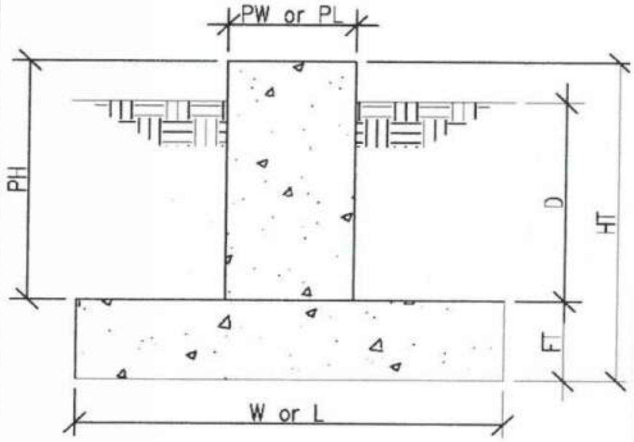
CLIENT:
PROJECT:
BY: DATE: 8/14/2018
JOB #: SHEET 6 OF 9

DESIGN OF RECTANGULAR FOOTING WITH OVERTURNING MOMENT

FOOTING:

LOADING PARAMETERS:

ALLOWABLE SOIL BEARING =	3,000	PSF
SOIL WEIGHT =	115	PCF
REQD. O.T. SAFETY FACTOR =	1.5	
STR.INCR.FOR HORIZ. LOADS =	1.33	
VERTICAL DEAD LOAD =	671.22	KIPS
VERTICAL LIVE LOAD =	0	KIPS
HORIZONTAL LOAD =	156.0	KIPS
MOMENT @ TOP OF FOOTING =	3,576	FT-KIPS



FOOTING DIMENSIONS:

FTG. LENGTH (L) =	37.0	FT (PAR.TO LOAD)
FTG. WIDTH (W) =	37.0	FT (PERP.TO LOAD)
FTG. THICKNESS (FT) =	2.00	FT
FOOTING DEPTH (D) =	0.0	FT
PIER LENGTH (PL) =	0.0	FT
PIER WIDTH (PW) =	0.0	FT
PIER HEIGHT (PH) =	0.0	FT
CONCRETE WEIGHT =	246.4	KIPS
SOIL WEIGHT =	0.0	KIPS
TOTAL WEIGHT =	246.4	KIPS

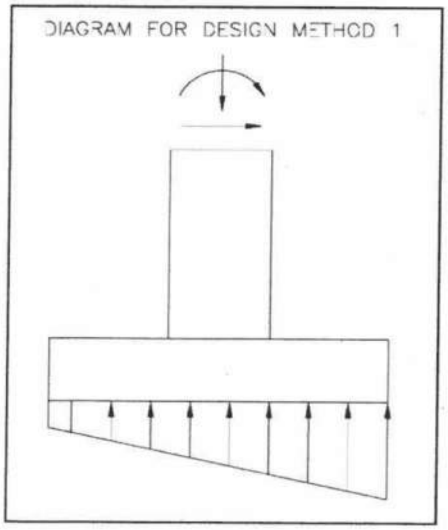
DESIGN METHOD 1

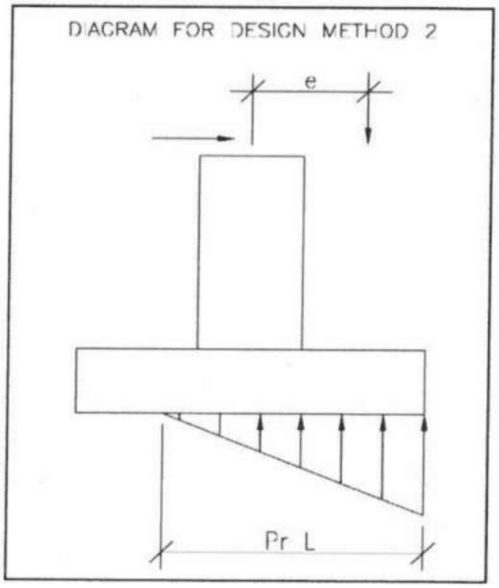
OVERTURNING MOM. =	3,888.0	FT-KIPS	
SOIL PR. FROM DL =	670.3	PSF	
SOIL PR. FROM MOM. =	(460.5)	PSF	
MIN. PRESSURE =	209.8	PSF	
MAX. PRESSURE =	1,130.8	PSF	**GOVERNS**

DESIGN METHOD 2

e =	4.24	FT
Pr L =	42.79	FT
MAX. PR =	1,159.2	PSF

DOES NOT APPLY AS NO UPLIFT AT BACK OF FOOTING





	ACTUAL		ALLOWABLE		
LL + DL BEARING =	670	PSF	3,000	PSF	OK
DL + HORIZ. BEARING =	1,131	PSF	3,990	PSF	OK
F.S. OF OVERTURNING =	4.37		1.5		OK

$\sigma_{min_u} \rightarrow \frac{1664-348}{37} = \frac{X-348}{32.5} \therefore \sigma_{min_u} = 1504$ PSF

$M_u = -1504(4.5)^2/2 - \frac{1}{2}(1664-1504)(4.5)(4.5/3) + M_u = 0$

$\therefore M_u = 15.8$ k·Ft/Ft or 189 k·in/Ft

$A_{s_{calc}} = \frac{M_u}{\phi f_y (d - \frac{a}{2})} \rightarrow \frac{189}{0.9(60)(21-\frac{.26}{2})} = 0.18$ in²/Ft

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.18(60)}{0.85(4)(12)} = 0.26$ in/Ft

$A_{s_{min}} = \frac{3\sqrt{f'_c}}{f_y} b_w d \rightarrow \frac{3\sqrt{4000}}{60000}(12)(21) = 0.80$ in²/Ft
(ACI 318-11 EQ 10-13)
$200 b_w d / f_y \rightarrow 200(12)(21)/60000 = 0.84$ in² Ft

$A_{s_{required}} = \frac{4}{3} A_{s_{calc}}$ [ACI 318-11 10.5.3] $= 4/3(0.18) = 0.24$ in²/Ft

*use No. 5 bars @ 12" o.c. → $A_s = 0.31$ in²/Ft > 0.24 in²/Ft (ok)
Both Directions

$\rho = \frac{A_s}{bd} \rightarrow \frac{0.266}{12(21)} = 0.0011 < 0.018$ (ok) $\phi = 0.9$

Check shear:

$\phi V_n \geq V_u \rightarrow \phi V_n = \phi(V_c + V_s)$

$V_c = 2\lambda\sqrt{f'_c} b_w d \rightarrow 2(1)\sqrt{4000}(222)(21) = 589.7$ kips

$\frac{1}{2}V_c > V_u \therefore$ no reinforcement needed.
[ACI 318 : 11.4.6.1]

$0.75 \frac{1}{2} V_c \Rightarrow 0.75(589.7)\frac{1}{2} = 221.1$ kips > 55.7 kips.

Tank Failure Foundation Walls:

$1.1(30) = 33k$ gallons $= 4411\ ft^3$ *use 5 ft tall walls

$5(32)(32) = 5120\ ft^3 > 4411\ ft^3$ (OK)

Check wall for Bending:

$P = \rho g h \quad 8.56/(0.133681) = 64\ LB/ft^3 = \rho$

$P = 64(5) = 320\ PSF$

$F = \frac{1}{2}bh \rightarrow \frac{1}{2}320(5) = 800\ Lbs/ft$

$m_u = 1.6[(800)(1.667)] = 2134\ LB \cdot ft$ or $25.6\ k \cdot in$

$A_s = \dfrac{m_u}{\phi f_y(d - \frac{a}{2})} \rightarrow \dfrac{25.6}{0.9(60)(4 - \frac{.18}{2})} = 0.121\ in^2/ft$

$a = \dfrac{A_s f_y}{0.85 f'_c b} = \dfrac{0.121(60)}{0.85(4)(12)} = 0.18\ in$

$A_{sreq} = \frac{4}{3}(0.121) = 0.161\ in^2$

*use No. 4 bars @ 12" O.C $A_s = 0.2\ in^2 > 0.161\ in^2$

$\rho = \dfrac{A_s}{bd} = \dfrac{0.171}{12(4)} = 0.0036 < 0.018$ (OK) $\therefore \phi = 0.9$

VERIFY RELOCATED SILO FOUNDATION IS SUFFICIENT FOR THE NEW SITE

MAPLE VALLEY SITE PARAMETER SUMMARY
WIND, Vult = 110 mph

SITE CLASS D
SEISMIC DESIGN CATEGORY D
SEISMIC ACCELERATION PARAMETERS
Ss = 1.325g
S1 = 0.495g
SDS = 0.883g
SD1 = 0.496g

BY INSPECTION SITE PARAMETERS ARE APPROX. EQUIVALENT, ORIGINAL FOUNDATION DESIGN IS ADEQUATE PENDING ORIGINAL DESIGN SUFFICIENCY

ORIGINAL SILO FOUNDATION DESIGN PARAMETERS (ref. B&T DRAWING 16091-S1.1)

1

GENERAL NOTE
CODE: INTERNATIONAL BUILDING CODE -- 2015 EDITION
ALL ASTM'S CALLED OUT ARE TO BE THE LATEST EDITION

LIVE LOADS
BUILDING RISK CATEGORY II (IBC TABLE 1604.5)

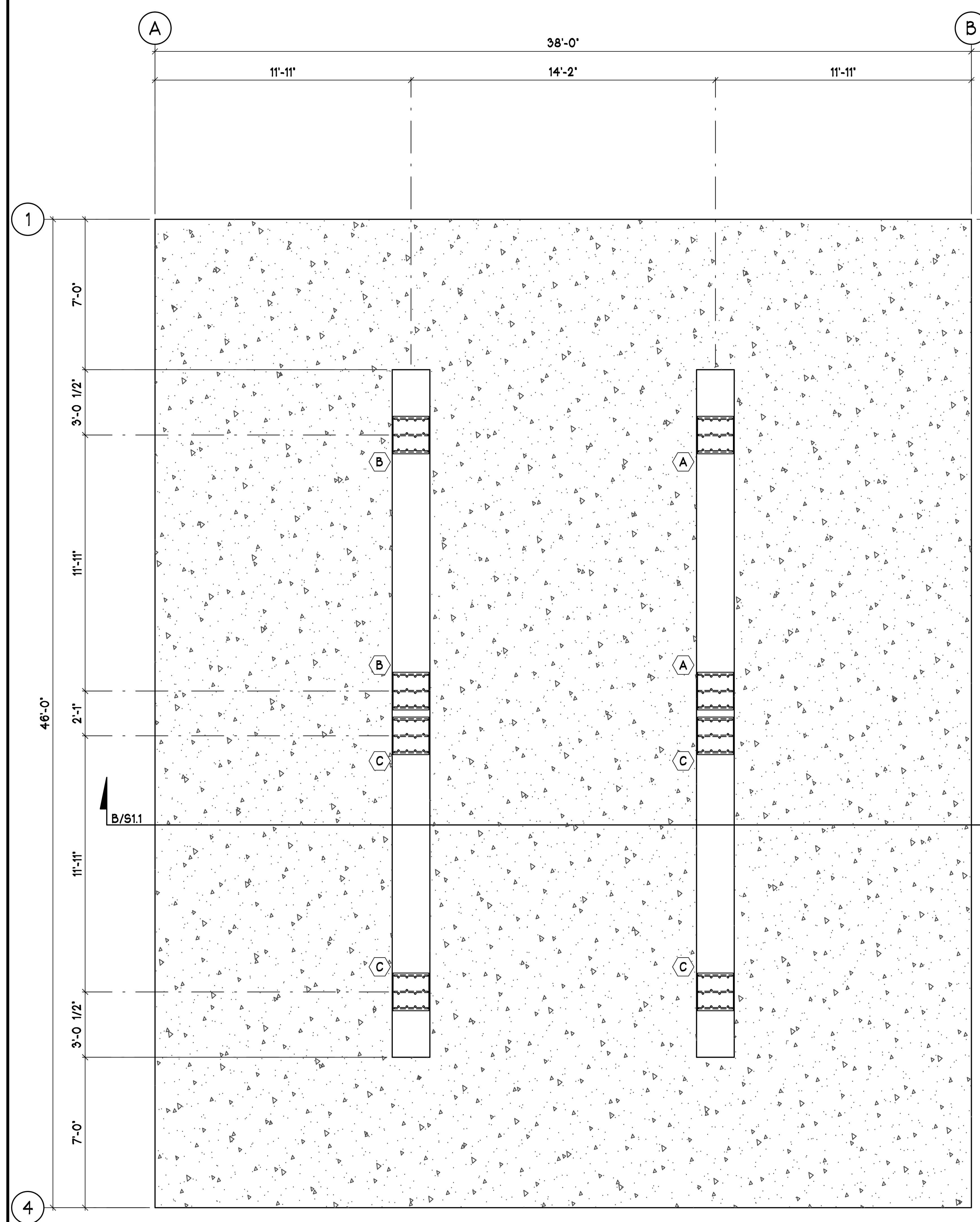
LATERAL LOADS:
WIND . . . . Vult = 120 MPH Vasd = 95 MPH
. . . . EXPOSURE "C" Kzt = 1.00
SEISMIC . . . . SITE CLASS "D"
SEISMIC DESIGN CATEGORY "D"
IMPORTANCE FACTOR Ie = 1.0
S_s = 1.253g S_1 = 0.472g
F_A = 1.000 Fv = 1.528
S_{DS} = 0.835g S_{D1} = 0.481g
R = 3.5 (ORDINARY MOMENT FRAME)

1

FOUNDATION
FOUNDATION DESIGN WAS BASED UPON SOILS REPORT NO. JN 16376 BY GEOTECH CONSULTANTS, INC., DATED AUGUST 16, 2016. THE FOLLOWING VALUES WERE USED:

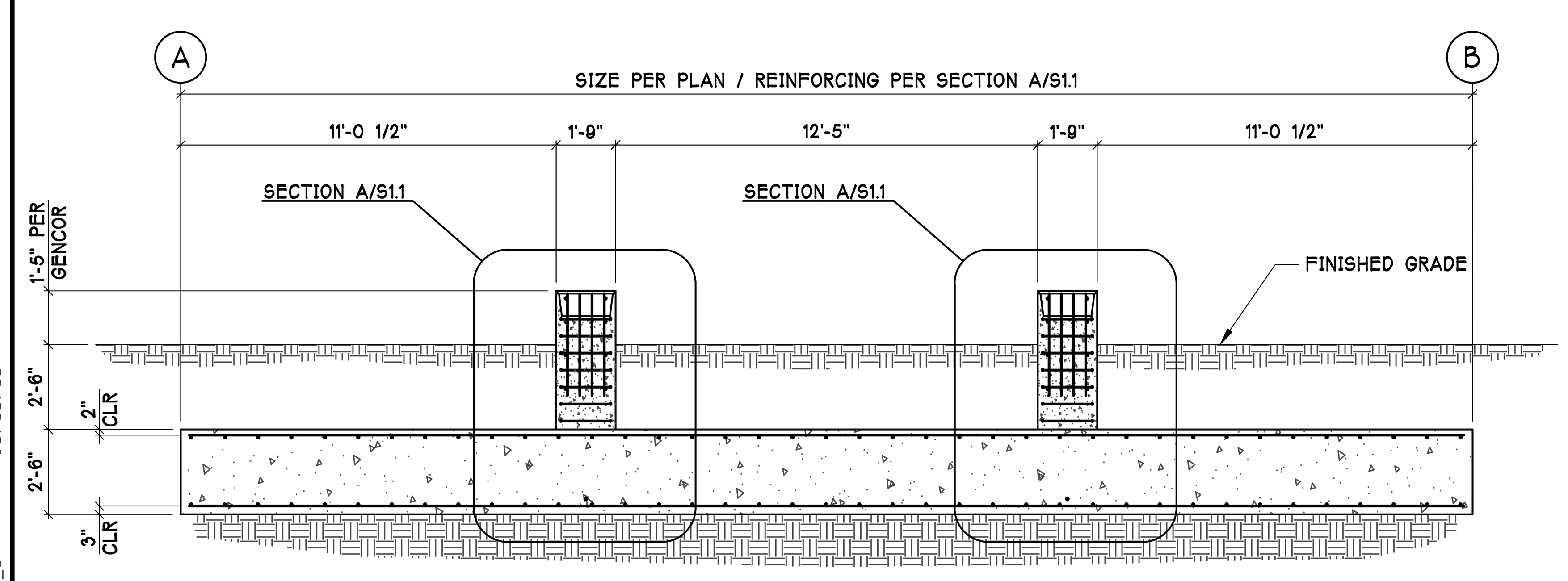
FOOTING BEARING PRESSURE:. . . . 3000 PSF ON DENSE NATIVE MATERIAL OR COMPACTED STRUCTURAL FILL (33% INCREASE FOR WIND OR SEISMIC)
LATERAL EARTH PRESSURE: . . . . . 35 PCF EQUIVALENT FLUID PRESSURE (ACTIVE-UNRESTRAINED)
. . . . . 50 PCF EQUIVALENT FLUID PRESSURE (ACTIVE-RESTRAINED)
. . . . . 350 PCF EQUIVALENT FLUID PRESSURE (PASSIVE)
COEFFICIENT OF FRICTION: 0.50

ALL EXTERIOR FOOTINGS SHALL BE A MINIMUM OF 1'-6" BELOW FINISH GRADE. SLABS AND FOOTINGS SHALL BEAR ON UNDISTURBED SOIL OR STRUCTURAL FILL COMPACTED TO 95% MAXIMUM DRY DENSITY PER ASTM D-1557 AS RECOMMENDED IN SOILS REPORT. CONTRACTOR SHALL PROVIDE PERMANENT POSITIVE DRAINAGE OF BUILDING PERIMETER. ALL SITE PREPARATION AND GRADING SHALL BE DONE IN ACCORDANCE WITH SOILS REPORT.



SILO FOUNDATION PLAN

1/4" = 1'-0"



B/S1.1 **SECTION** 1/4" = 1'-0"

REQUIRED SPECIAL INSPECTIONS AND TESTS OF SOILS (TABLE 1705.6 IBC 2015)

TYPE	CONTINUOUS SPECIAL INSPECTION	PERIODIC SPECIAL INSPECTION
1. VERIFY MATERIALS BELOW SHALLOW FOUNDATIONS ARE ADEQUATE TO ACHIEVE THE DESIGN BEARING CAPACITY.	–	X
2. VERIFY EXCAVATIONS ARE EXTENDED TO PROPER DEPTH AND HAVE REACHED PROPER MATERIAL.	–	X
3. PERFORM CLASSIFICATION AND TESTING OF COMPACTED FILL MATERIALS.	–	X
4. VERIFY USE OF PROPER MATERIALS, DENSITIES AND LIFT THICKNESSES DURING PLACEMENT AND COMPACTION OF COMPACTED FILL.	X	–
5. PRIOR TO PLACEMENT OF COMPACTED FILL, OBSERVE SUBGRADE AND VERIFY THAT SITE HAS BEEN PREPARED PROPERLY.	–	X

REQUIRED SPECIAL INSPECTIONS AND TEST OF CONCRETE CONSTRUCTION (TABLE 1705.3 IBC 2015)

TYPE	CONTINUOUS SPECIAL INSPECTION	PERIODIC SPECIAL INSPECTION	REFERENCED STANDARD[a]	IBC REFERENCED
1. INSPECTION OF REINFORCEMENT, INCLUDING PRESTRESSING TENDONS, AND VERIFY PLACEMENT.	–	X	ACI 318 CH. 20, 25.2, 25.3, 26.5.1–26.5.3	1908.4
2. REINFORCING BAR WELDING:			AWS D1.4 ACI 318 26.5.4	–
a. VERIFY WELDABILITY OF REINFORCING BARS OTHER THAN ASTM A 706;	–	X		
b. INSPECT SINGLE-PASS FILLET WELDS, MAXIMUM 5/16"; AND		X		
c. INSPECT ALL OTHER WELDS.	X			
3. INSPECT ANCHORS CAST IN CONCRETE.	–	X	ACI 318 17.8.2	–
4. INSPECT ANCHORS POST-INSTALLED IN HARDENED CONCRETE MEMBERS.[b]				–
a. ADHESIVE ANCHORS INSTALLED IN HORIZONTALLY OR UPWARDLY INCLINED ORIENTATIONS TO RESIST SUSTAINED TENSION LOADS.	X		ACI 318 17.8.2.4	
b. MECHANICAL ANCHORS AND ADHESIVE ANCHORS NOT DEFINED IN 4.a.		X	ACI 318 17.8.2	
5. VERIFY USE OF REQUIRED DESIGN MIX.	–	X	ACI 318 CH. 19, 26.4.3, 26.4.4	1904.1, 1904.2, 1908.2, 1908.3
6. PRIOR TO CONCRETE PLACEMENT, FABRICATE SPECIMENS FOR STRENGTH TESTS, PERFORM SLUMP AND AIR CONTENT TESTS, AND DETERMINE THE TEMPERATURE OF THE CONCRETE.	X	–	ASTM C 172 ASTM C 31 ACI 318: 26.4.5, 26.12	1908.10
7. INSPECT CONCRETE AND SHOTCRETE PLACEMENT FOR PROPER APPLICATION TECHNIQUES.	X	–	ACI 318 26.4.5	1908.6, 1908.7, 1908.8
8. VERIFY MAINTENANCE OF SPECIFIED CURING TEMPERATURE AND TECHNIQUES.	–	X	ACI 318 26.4.7–26.4.9	1908.9
9. INSPECT PRESTRESSED CONCRETE FOR:				–
a. APPLICATION OF PRESTRESSING FORCES; AND	–	–	ACI 318 26.9.2.1	
b. GROUTING OF BONDED PRESTRESSING TENDONS.	–	–	ACI 318 26.9.2.3	
10. INSPECT ERECTION OF PRECAST CONCRETE MEMBERS.	–	–	ACI 318 CH. 26.8	–
11. VERIFY IN-SITU CONCRETE STRENGTH, PRIOR TO STRESSING OF TENDONS IN POST-TENSIONSED CONCRETE AND PRIOR TO REMOVAL OF SHORES AND FORMS FROM BEAMS AND STRUCTURAL SLABS.	–	–	ACI 318: 26.10.2	–
12. INSPECT FORMWORK FOR SHAPE, LOCATION AND DIMENSIONS OF THE CONCRETE MEMBER BEING FORMED.	–	X	ACI 318: 26.10.1(b)	–

NOTES:

a. WHERE APPLICABLE, SEE ALSO SECTION 1705.12, SPECIAL INSPECTION FOR SEISMIC RESISTANCE.

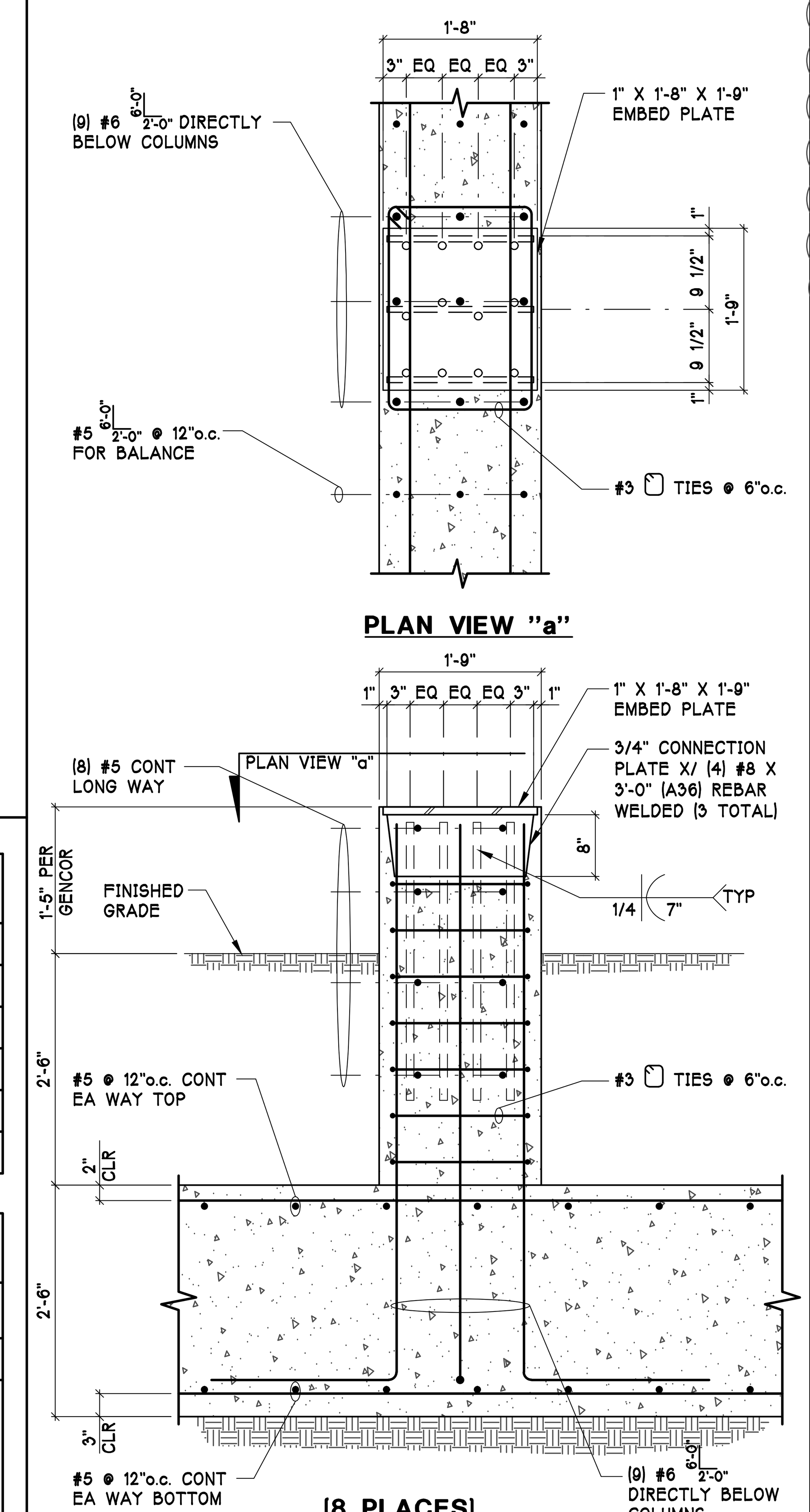
b. SPECIFIC REQUIREMENTS FOR SPECIAL INSPECTION SHALL BE INCLUDED IN THE RESEARCH REPORT FOR THE ANCHOR ISSUED BY AN APPROVED SOURCE IN ACCORDANCE WITH 17.8.2 IN ACI 318, OR OTHER QUALIFICATION PROCEDURES. WHERE SPECIFIC REQUIREMENTS ARE NOT PROVIDED, SPECIAL INSPECTION REQUIREMENTS SHALL BE SPECIFIED BY THE REGISTERED DESIGN PROFESSIONAL AND SHALL BE APPROVED BY THE BUILDING OFFICIAL PRIOR TO THE COMMENCEMENT OF THE WORK.

INSPECTION TABLES

BASE PLATE LOADS (KIPS)

SILO LEGS	A	B	C	D
TOTAL D+L	181k	84k	123k	29k
TOTAL UPLIFT	184k	184k	128k	128k
LATERAL LOAD D+L	16k	16k	16k	25k

BASE PLATE LOADS SCHEDULE



A/S1.1 **SECTION** 3/4" = 1'-0"

GENERAL NOTE
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BUILDING RISK CATEGORY II (IBC TABLE 1604.5)

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WIND . . . . Vult = 120 MPH Vasd = 95 MPH
. . . . EXPOSURE "C" Kzt = 1.00
SEISMIC . . . . SITE CLASS "D"
SEISMIC DESIGN CATEGORY "D"
IMPORTANCE FACTOR Ie = 1.0
S_S = 1.253g S_1 = 0.472g
F_A = 1.000 Fv = 1.528
S_{DS} = 0.835g S_{D1} = 0.481g
R = 3.5 (ORDINARY MOMENT FRAME)

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. . . . . 50 PCF EQUIVALENT FLUID PRESSURE (ACTIVE-RESTRAINED)
. . . . . 350 PCF EQUIVALENT FLUID PRESSURE (PASSIVE)
COEFFICIENT OF FRICTION: 0.50

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CONCRETE (MIX DESIGN – DEFERRED SUBMITTAL)
~~f'c = 3000 psi FOR RETAINING WALLS, FOOTINGS & SLABS ON GRADE~~
ULTIMATE STRENGTH DESIGN METHOD USED. MIXING AND PLACING OF ALL CONCRETE AND SELECTION OF MATERIALS SHALL BE IN ACCORDANCE WITH THE IBC AND ACI CODE 318. PROPORTIONING OF AGGREGATE TO CEMENT SHALL BE SUCH AS TO PRODUCE A DENSE WORKABLE MIX WITH 5" MAXIMUM SLUMP WHICH CAN BE PLACED WITHOUT SEGREGATION OR EXCESS FREE SURFACE WATER. FOR ADMIXTURES, SEE SPECIFICATIONS. 1/2" CHAMFER ALL EXPOSED EDGES, UNLESS INDICATED OTHERWISE ON ARCHITECTURAL DRAWINGS. WATER CURING SHALL BE USED. AIR ENTRAIN ALL CONCRETE EXPOSED TO WEATHER WITH 3% TO 6% AIR BY VOLUME.

REINFORCING STEEL
ALL CONCRETE REINFORCING STEEL SHALL BE DEFORMED PER ASTM A615, GRADE 60 (fy=60,000 psi) EXCEPT ALL #4 SLAB DOWELS SHALL BE GRADE 40 (fy=40,000 psi). LAP CONTINUOUS REINFORCING BARS 30 BAR DIAMETERS, 1'-7" MINIMUM UNLESS NOTED OTHERWISE. CORNER BARS (1'-7" BEND) WILL BE PROVIDED FOR ALL HORIZONTAL REINF. DETAIL STEEL IN ACCORDANCE WITH "ACI MANUAL OF STANDARD PRACTICE OF DETAILING REINFORCED CONCRETE STRUCTURES". WELDED WIRE FABRIC (WWF) TO CONFORM WITH ASTM A185. REINFORCING HOOKS TO COMPLY WITH STANDARD ACI HOOKS.

UNLESS NOTED OTHERWISE, COVER TO MAIN REINFORCEMENT TO BE:
CONCRETE CAST AGAINST &
PERMANENTLY EXPOSED TO EARTH . . . . 3 INCHES
CONCRETE EXPOSED TO EARTH OR WEATHER . . 1 1/2 INCHES (#5 BARS & SMALLER)
. . . . 2 INCHES (#6 THRU #18 BARS)
CONCRETE NOT EXPOSED TO WEATHER
OR IN CONTACT WITH GROUND . . . . 3/4 INCHES (#11 BAR & SMALLER)
. . . . 1 1/2 INCHES (#14 & #18 BARS)

MISCELLANEOUS STEEL PLATES
MISCELLANEOUS STEEL PLATES SHALL CONFORM TO ASTM A36 (fy=36,000 PSI). WELDS NOT SPECIFIED SHALL BE 1/4" CONTINUOUS FILLET MINIMUM. ALL WELDS TO BE BY WABO CERTIFIED WELDERS -- USE FRESH E70 ELECTRODES. MISCELLANEOUS HANGERS TO BE SIMPSON OR I.C.C. APPROVED EQUAL. NAIL ALL HOLES WITH NAILS AS SPECIFIED BY MANUFACTURER UNLESS NOTED OTHERWISE ON DRAWINGS. MACHINE BOLTS TO BE A-307. ANCHOR BOLTS INTO CONCRETE SHALL HAVE MINIMUM EMBEDMENT PER IBC TABLE 1908.2.

SHOP DRAWINGS
SUBMIT THREE SETS OF SHOP DRAWINGS TO THE ENGINEER AND ONE SET TO THE BUILDING DEPARTMENT FOR APPROVAL PRIOR TO FABRICATION FOR:
REINFORCING STEEL & MISC. STEEL

SPECIAL INSPECTIONS
INSPECTIONS ARE TO BE PER IBC CHAPTER 17, SECTIONS 1704 AND 1705 AND ARE TO BE BY AN INDEPENDENT TESTING LAB AND APRROVED BY THE OWNER AND BUILDING DEPARTMENT AND ENGAGED BY AND PAID FOR BY THE OWNER PRIOR TO STARTING CONSTRUCTION.

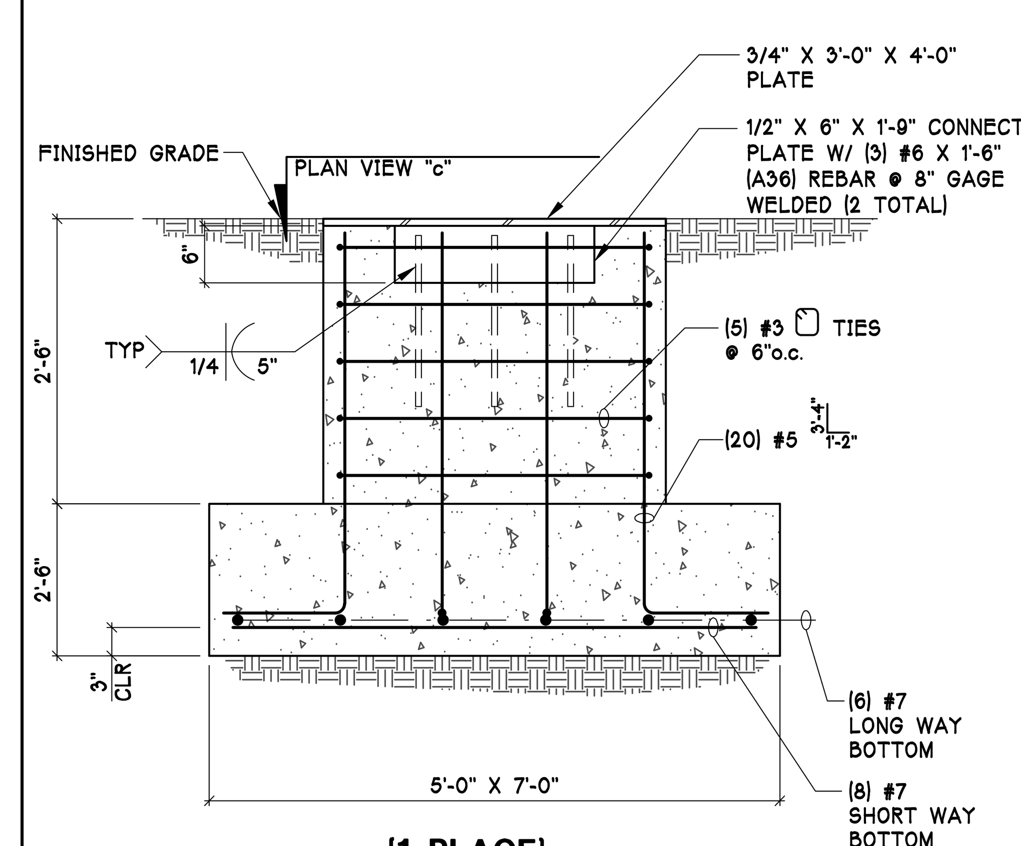
FOUNDATION: INSPECT FOOTINGS AND EXCAVATIONS JUST PRIOR TO CONCRETE PLACEMENT TO INSURE MATERIAL IS DRY AND DENSE.

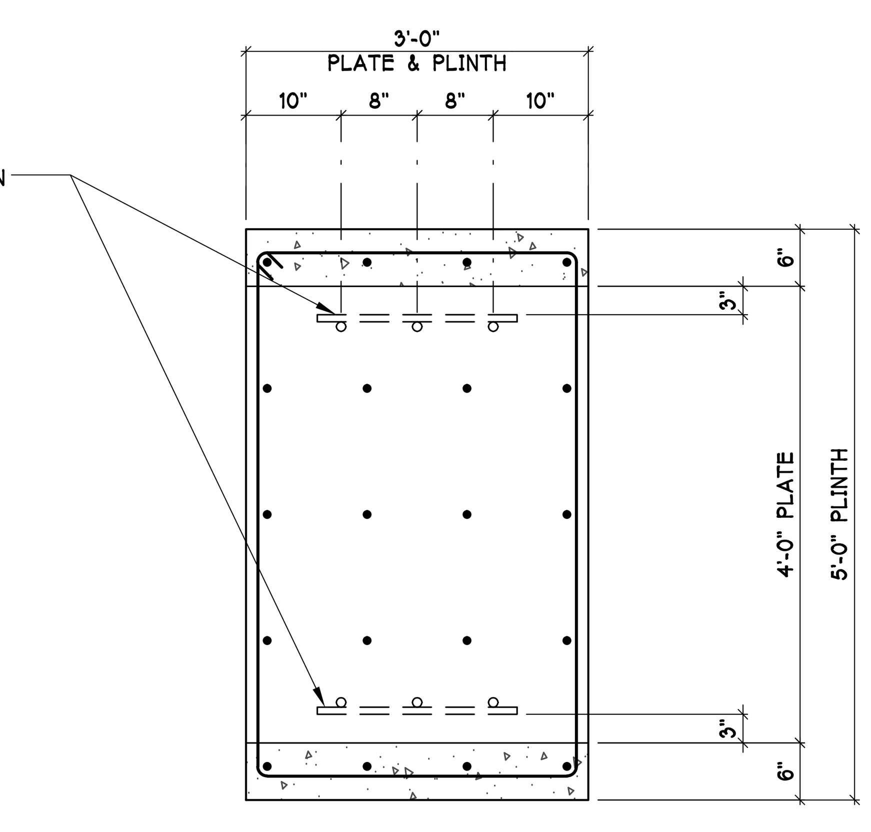
CONCRETE: TAKE CONCRETE CYLINDERS PER IBC SECTION 1705.3. VERIFY MIX DESIGN AND SLUMPS.

REINFORCING: VERIFY ALL REINFORCING IS PLACED IN ACCORDANCE WITH APPROVED PLANS. CHECK FOR REQUIRED COVER, SIZE AND GRADE.

SPECIAL CONDITIONS
CONTRACTOR SHALL NOTIFY ENGINEER OF RECORD OF ANY UNUSUAL OR UNSAFE CONDITIONS WHICH ARE DISCOVERED DURING CONSTRUCTION. SOME DETAILS WERE BASED UPON ASSUMED CONDITIONS SINCE THEY WERE NOT CLEARLY SHOWN ON ORIGINAL DRAWINGS. ANY DISCREPANCIES BETWEEN OUR DETAILS AND ACTUAL CONDITIONS SHOULD BE CALLED TO OUR ATTENTION PRIOR TO CONTINUATION OF WORK. CONTRACTOR SHALL VERIFY ALL DIMENSIONS IN FIELD AND SHALL PROVIDE ADEQUATE SHORING AND BRACING OF ALL STRUCTURAL MEMBERS DURING CONSTRUCTION. CONTRACTOR SHALL NOTIFY ENGINEER OF ALL FIELD CHANGES PRIOR TO INSTALLATION.

GENERAL NOTES





PLAN VIEW "c"

C/S1.1 **SECTION** (SLATT CONVEYOR) 3/4" = 1'-0"

REVISIONS		
DATE	DESCRIPTION	
08/18/16	CITY OF COVINGTON COMMENTS	1

B&T DESIGN & ENGINEERING
B & T DESIGN & ENGINEERING, INC.
250 E. SUNSET WAY
ISSAQUAH, WA 98027
(425) 557-0779 (PHONE)
(425) 557-0765 (FAX)

TERRY E. BALDWIN — STATE OF WASHINGTON — 23915 — PROFESSIONAL STRUCTURAL ENGINEER

LAKESIDE INDUSTRIES SILOS
18808 SE 256TH ST
COVINGTON, WA 98042

PROJECT TITLE

FOUNDDATION
PLAN & DETAILS

SHEET TITLE

PROJECT	16091
DATE	05/19/16
DRAWN	NH
CHKD.	JT

SHEET : **S1.1** OF

08/18/16
16091-S1_1

Smith Monroe Gray
ENGINEERS, INC.

CLIENT ____ PROJECT ____
BY ____ DATE ____ REV. ____
JOB NO. ____ SHEET ____ OF ____

Vaporizer Fndn

$H_T = \sim 34'-8''$

CG $H_T = \sim 17'-2''$

Empty $W_T = 6500^{\#}$

Operating $W_T = 7000^{\#}$

(4) Base Plates w/ (4) 1" Ø Anchors

Seismic Parameters

* On Symmetrically Braced Legs → $R = 3.0$

$$C_S = \frac{S_{DS}}{R/I_e} = \frac{0.883}{(3.0/1.0)} = 0.29$$

$$E_V = 0.2 \cdot S_{DS} \cdot D = 0.18 D$$

Wind

$V = 110$ mph $K_d = 0.85$ $K_{zt} = 1.0$ $K_Z = 0.85$

$$q_z = 0.00256 \cdot K_d \cdot K_z \cdot K_{zt} \cdot V^2 = 22.4 \text{ PSF}$$

$$F_W = G \cdot q_z \cdot C_f = 0.85 \cdot 22.4 \text{ PSF} \cdot 1.65 = 31.4 \text{ PSF}$$

$C_f = 1.65$ $(B/S = 0.24)$

$0.6 \cdot F_W = 18.8$ PSF

SMG
Smith Monroe Gray
ENGINEERS, INC.

CLIENT ______
PROJECT ______
BY ______ DATE ______ REV. ______
JOB NO. ______ SHEET ______ OF ______

BASE RXN - SEISMIC

$W_T = 7000^{\#}$

$V_x = 7000^{\#} \cdot 0.29 = 2030^{\#} \times 0.7 = 1421^{\#}$
$V_y = 7000^{\#} \cdot 0.18 = 1260^{\#} \times 0.7 = 882^{\#}$

$M_{OVER} = 2030^{\#} \cdot (206'' / 12''/ft) = 34848^{\#\cdot ft} \times 0.7 = \underline{24394^{\#\cdot ft}}$

BASE RXN - WIND

$W_T = 7000^{\#}$

$V_x = 18.8\ psf \cdot (416'' \cdot 48'') / 144\ in^2 = 5322^{\#}$

$M_{OVER} = 5322^{\#} \cdot (416'' / 12''/ft) / 2 = \underline{92248^{\#\cdot ft}}$

ANCHORAGE DESIGN

$V_{max} = 5322^{\#}/4 = 1330^{\#}$ / BASE PLATE $/0.6 = 2216^{\#} = V_U$

$T_{MAX} = \dfrac{92248^{\#\cdot ft}}{(84''/12''/ft) \cdot 2} = 6589^{\#}$ / BASE PLATE $/0.6 = T_U = 10982^{\#}$

USE (4) 1"DIA ANCHORS W/ 10" EMBED (SEE HILTI OUTPUT)

FOOTING PRESSURE - TRY 24" x 12' SQ

VERT PRESSURE $= \dfrac{7000^{\#}}{144\ ft^2} + 300\ psf = 348\ psf \times 12 = 4176\ PLF$

TOTAL = 50112

$M_{OVERT} = 92248^{\#\cdot ft}$

CHECK MIDDLE 1/3
RXN ECC = 2^{ft} → $92248^{\#\cdot ft} / 2^{ft} = 46124^{\#} < 50112^{\#}$ OK✓

Company:		Page:	1
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	4/8/2019
E-Mail:			

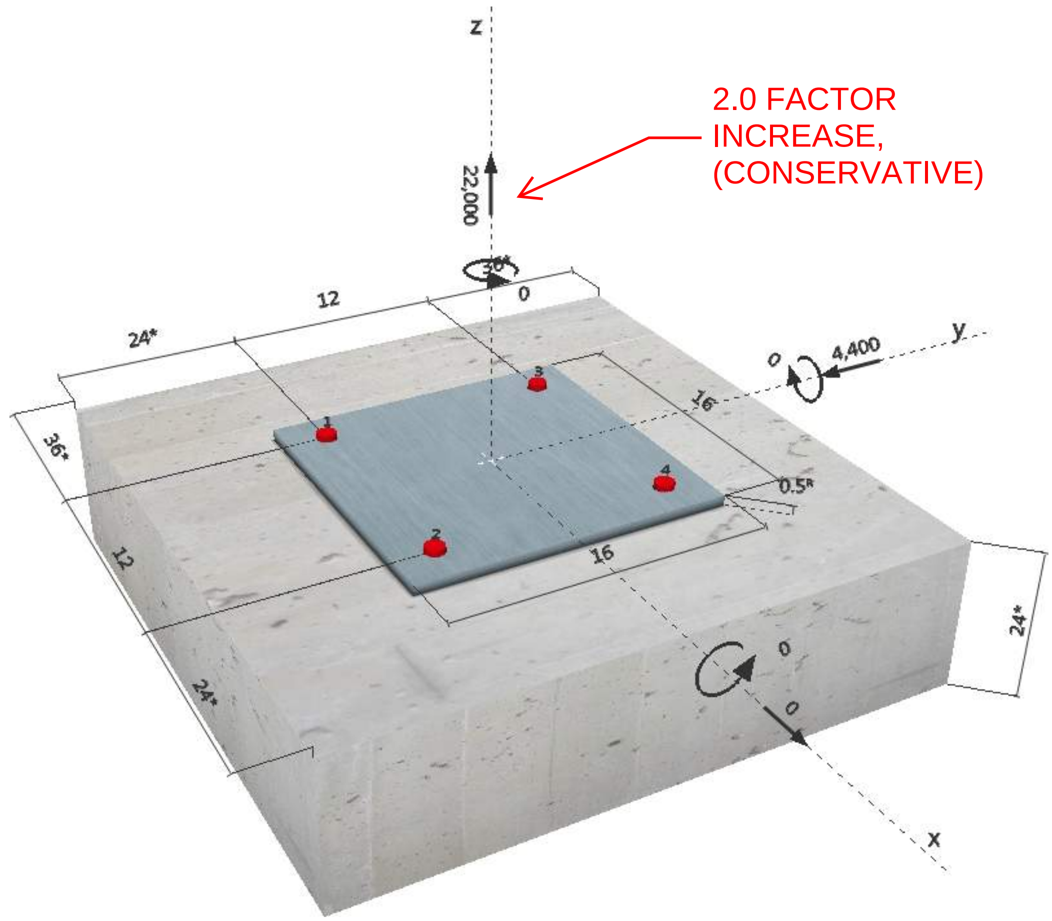
Specifier's comments:

1 Input data

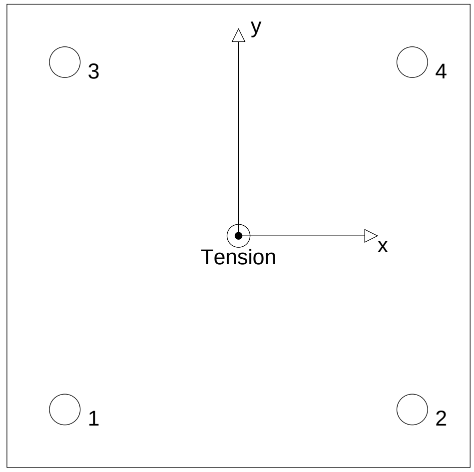
Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 1**
Effective embedment depth:	h_{ef} = 10.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-14 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 16.000 in. x 16.000 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile:	no profile
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 24.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d))
	Shear load: yes (17.2.3.5.3 (c))

R - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



2 Load case/Resulting anchor forces



Load case: Design loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	5,500	1,100	0	-1,100
2	5,500	1,100	0	-1,100
3	5,500	1,100	0	-1,100
4	5,500	1,100	0	-1,100

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 22,000 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	5,500	26,361	21	OK
Pullout Strength*	5,500	25,217	22	OK
Concrete Breakout Strength**	22,000	49,392	45	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

$N_{sa} = A_{se,N}\ f_{uta}$ ACI 318-14 Eq. (17.4.1.2)
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

N_{sa} [lb]
35,148

Results

N_{sa} [lb]	ϕ_{steel}	ϕ N_{sa} [lb]	N_{ua} [lb]
35,148	0.750	26,361	5,500

3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-14 Eq. (17.4.3.1)

$N_p = 8 A_{brg} f'_c$ ACI 318-14 Eq. (17.4.3.4)

$\phi N_{pN} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.50	1.000	4,000

Calculations

N_p [lb]
48,032

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{pn} [lb]	N_{ua} [lb]
48,032	0.700	0.750	1.000	25,217	5,500

3.3 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$ ACI 318-14 Eq. (17.4.2.1b)

$\phi N_{cbg} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
10.000	0.000	0.000	24.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,764.00	900.00	1.000	1.000	1.000	1.000	48,000

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{cbg} [lb]	N_{ua} [lb]
94,080	0.700	0.750	1.000	49,392	22,000

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	1,100	13,708	9	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	4,400	131,712	4	OK
Concrete edge failure in direction y-**	4,400	34,426	13	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-14 Eq. (17.5.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

V_{sa} [lb]
21,089

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
21,089	0.650	13,708	1,100

4.2 Pryout Strength

$V_{cpg} = k_{cp}\left[\left(\frac{A_{Nc}}{A_{Nc0}}\right)\psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b\right]$ ACI 318-14 Eq. (17.5.3.1b)
$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1
A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)
$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)
$\psi_{ec,N} = \left(\frac{1}{1+\frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)
$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)
$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	10.000	0.000	0.000	24.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,764.00	900.00	1.000	1.000	1.000	1.000	48,000

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
188,160	0.700	1.000	1.000	131,712	4,400

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4.3 Concrete edge failure in direction y-

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\, \psi_{ed,V}\, \psi_{c,V}\, \psi_{h,V}\, \psi_{parallel,V}\, V_b$ ACI 318-14 Eq. (17.5.2.1b)

$\phi\ V_{cbg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ ACI 318-14 Eq. (17.5.2.1c)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e_V'}{3c_{a1}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.5)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.6b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-14 Eq. (17.5.2.8)

$V_b = 9\ \lambda_a \sqrt{f_c'}\ c_{a1}^{1.5}$ ACI 318-14 Eq. (17.5.2.2b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
24.000	24.000	0.000	1.000	24.000

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
8.000	1.000	1.000	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
1,728.00	2,592.00	1.000	0.900	1.225	66,925

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
49,180	0.700	1.000	1.000	34,426	4,400

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.445	0.128	5/3	30	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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6 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.

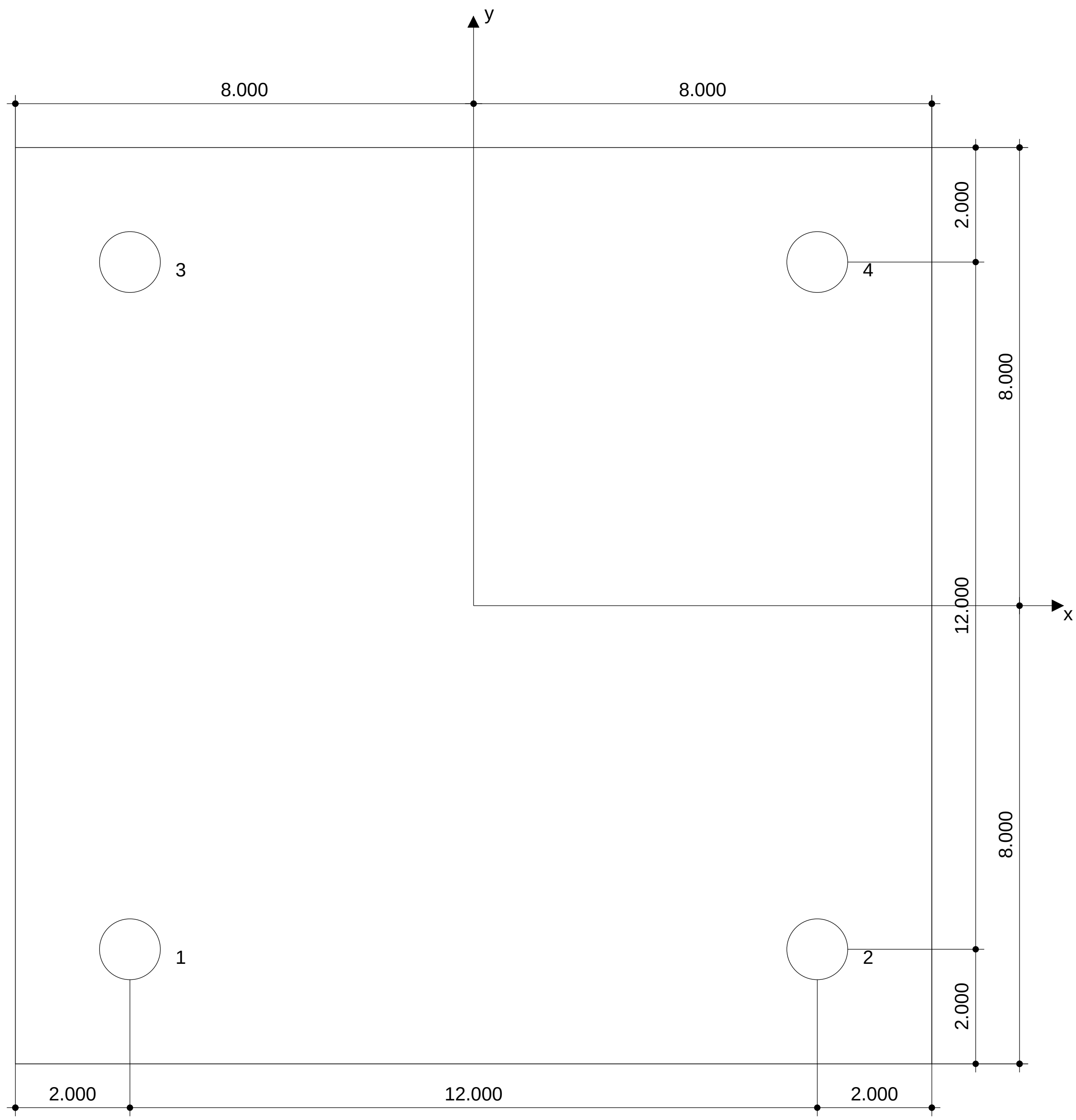
Fastening meets the design criteria!

Company:
Specifier:
Address:
Phone I Fax: |
E-Mail:

7 Installation data

Anchor plate, steel: -
Profile: no profile
Hole diameter in the fixture: d_f = 1.063 in.
Plate thickness (input): 0.500 in.
Recommended plate thickness: not calculated

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 1
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 10.000 in.
Minimum thickness of the base material: 11.172 in.



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-6.000	-6.000	36.000	36.000	24.000	48.000
2	6.000	-6.000	48.000	24.000	24.000	48.000
3	-6.000	6.000	36.000	36.000	36.000	36.000
4	6.000	6.000	48.000	24.000	36.000	36.000

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8 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

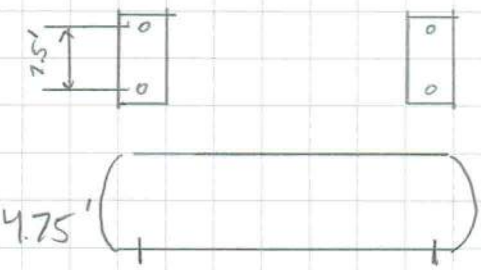
Smith Monroe Gray
ENGINEERS, INC.

CLIENT ______
PROJECT ______
BY ______ DATE ______ REV. ______
JOB NO. ______ SHEET ______ OF ______

LNG Foundation

Tank Wt. = 25.9 kips ± 3% → 26.7 kips
LNG Wt. = 54.6 kips

Σ Design Wt. = 81.3 kips



Vert. C.G. = 8.5'/2 + (9' − 8.5') = 4.75'

Seismic Parameters (similar to Vaporizer)

C_s = 0.44
E_v = 0.18·D
) Controls over Wind

Anchor Rxn Summary

LC		Total Rxn	Rxn Per Anchor
DL		81.3k	20.3k ↓
Ex-Shear	81.3k·0.44 = 35.8k	35.8k	9.0k →
E-Overturning	35.8k·4.75'/7.5'	22.7k ↑↓	11.4k ↑↓
Ev	81.3k·0.18 =	14.6k	3.7k ↑↓
Max ↓	81.3k + 14.6k	95.9k	31.4k ↓
Max ↑	0.6D + 0.7E	38.6k ↓	1.6k ↓

(2) 5'x10' Footings OK by Insp., Sp = 1110 PSF OK

LNG FOUNDATION (CONT'D)

FOUNDATION OVERTURNING

MIN. WEIGHT $= 0.9 \cdot 81.3^{k} + 0.9 \cdot 5' \cdot 12' \cdot 2 \cdot 0.15\ kcf$
$= 89.4^{k\text{-}FT}$

$M_{OVER} = 35.8^{k} \cdot (4.75') = 170^{k\text{-}FT}$

$M_{RESIST} = 89.4^{KIPS} \cdot 12'/6 = 178.8^{k\text{-}FT}$ OK✓

MAX. WEIGHT $= 1.2 \cdot 81.3^{k} + 1.0 \cdot 14.6^{k} + 1.2 \cdot 5' \cdot 12' \cdot 2 \cdot 0.15\ kcf$
$= 133.8^{KIPS}$

MAX. PRESSURE $= 133.8^{KIPS} / (2 \cdot 5' \cdot 12') \times 2 = 2.23$ ksf @ TRIANGULAR

< 3000 PSF OK✓

ANCHORAGE RXN

T_U = NA * DOES NOT OCCUR (MIN COMPRESSION = 1.6^{k})
$V_U = 9.0^{k} \times 0.7 = 6.3^{k}$

ASD DEMAND
LRFD DEMAND = 9.0 KIPS

USE A SINGLE 1 3/8" DIA ANCHOR W/ 6" MIN EMBED
(SEE HILTI OUTPUT)

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Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	4/8/2019
E-Mail:			

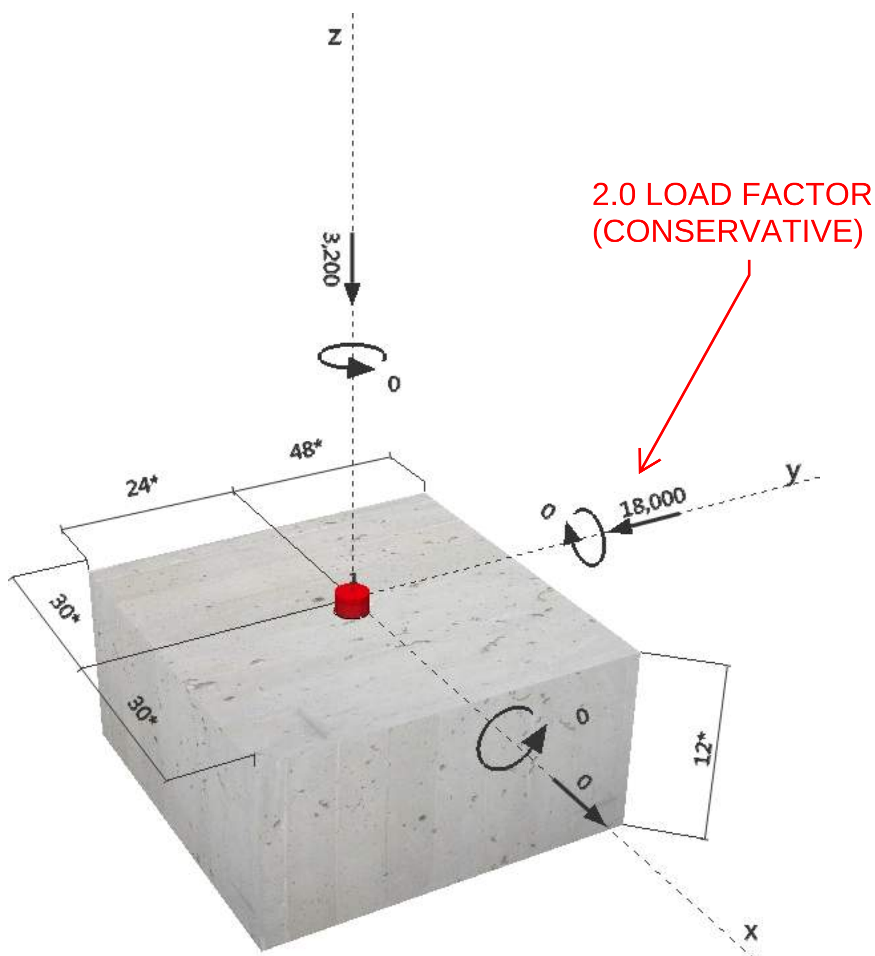
Specifier's comments:

1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 1 3/8**
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-14 / CIP
Stand-off installation:	- (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 12.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d))
	Shear load: yes (17.2.3.5.3 (c))

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



Company:
Specifier:
Address:
Phone I Fax: |
E-Mail:

2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	-3,200	18,000	0	-18,000

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	-3,200	50,460	7	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-14 Eq. (17.4.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
1.16	58,000

Calculations

N_{sa} [lb]
67,280

Results

N_{sa} [lb]	ϕ_{steel}	ϕ N_{sa} [lb]	N_{ua} [lb]
67,280	0.750	50,460	-3,200

The steel proof was done for the highest absolute force per anchor - in this case compression loading. Please be aware that buckling should be verified separately

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	18,000	26,239	69	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	18,000	31,232	58	OK
Concrete edge failure in direction y-**	18,000	22,540	80	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\, A_{se,V}\, f_{uta}$ ACI 318-14 Eq. (17.5.1.2b)

$\phi\, V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
1.16	58,000

Calculations

V_{sa} [lb]
40,368

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\, V_{sa}$ [lb]	V_{ua} [lb]
40,368	0.650	26,239	18,000

4.2 Pryout Strength

$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b \right]$ ACI 318-14 Eq. (17.5.3.1a)

$\phi\, V_{cp} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e_N'}{3\, h_{ef}}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\, \lambda_a \sqrt{f_c'}\, h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	24.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
324.00	324.00	1.000	1.000	1.000	1.000	22,308

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\, V_{cp}$ [lb]	V_{ua} [lb]
44,617	0.700	1.000	1.000	31,232	18,000

4.3 Concrete edge failure in direction y-

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ ACI 318-14 Eq. (17.5.2.1a)

$\phi\ V_{cb} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ ACI 318-14 Eq. (17.5.2.1c)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e_V'}{3c_{a1}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.5)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.6b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-14 Eq. (17.5.2.8)

$V_b = 9\ \lambda_a \sqrt{f_c'}\ c_{a1}^{1.5}$ ACI 318-14 Eq. (17.5.2.2b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
20.000	30.000	0.000	1.000	12.000

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
6.000	1.000	1.375	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
720.00	1,800.00	1.000	1.000	1.581	50,912

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cb}$ [lb]	V_{ua} [lb]
32,199	0.700	1.000	1.000	22,540	18,000

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.063	0.799	5/3	70	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Company:		Page:	5
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	4/8/2019
E-Mail:			

6 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to done separately.
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.

Fastening meets the design criteria!

Company:
Specifier:
Address:
Phone I Fax: |
E-Mail:

7 Installation data

Anchor plate, steel: -
Profile: -
Hole diameter in the fixture: -
Plate thickness (input): -
Recommended plate thickness: -

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 1 3/8
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 6.000 in.
Minimum thickness of the base material: 7.406 in.

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	30.000	30.000	24.000	48.000

8 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

Sound Attentuation Wall 1 (18' Max Height)

Max. Ht. = 18'

Design Wind Speed = 110 MPH = V

Solid Sign Wind Loading Per ASCE 7-14,

$s/h = 1.0$ $B/s = 2.0$ min → $C_f = 1.40$
= 20 max → $C_f = 1.30$

Max. $C_f = 1.40$

$q_z = 0.00256 \cdot K_d \cdot K_{zt} \cdot K_z \cdot V^2 = 22.4$ PSF

$K_d = 0.85$
$K_{zt} = 1.0$
$K_z = 0.85$ @ H = 0-15', Exp. C

$F_W = G \, q_z \cdot C_f = 26.7$ PSF

$G = 0.85$
$q_z = 22.4$ PSF
$C_f = 1.4$

Seismic Load - Nonbuilding Structure (Mass Cantilever)

$C_s = 0.29$ ($R = 3.0$)

$W_{6"} = 6"/12" \cdot 150 \text{ PCF} \cdot 0.29 = 21.75$ PLF ← Wind Controls

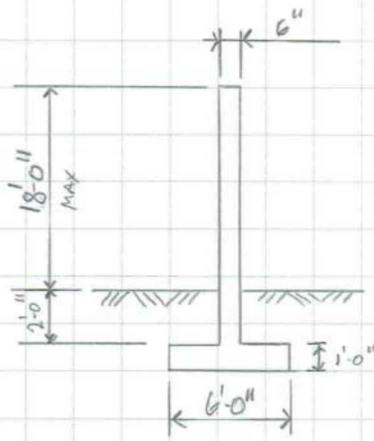
Smith Monroe Gray
ENGINEERS, INC.

CLIENT ______
PROJECT ______

BY ______ DATE ______ REV. ______
JOB NO. ______ SHEET ______ OF ______

SOUND ATTENUATION WALL (CONT'D)

WALL DEMAND (DL + 1.0WL LOAD COMBO CONTROLS)



$w = 26.7\ \text{PLF}$

$M_U = (26.7\ \text{PLF} \cdot 18') \cdot (18'/2 + 2') = \underline{5286\ ^{\#\cdot FT}/FT}$

$V_U = 26.7\ \text{PLF} \cdot 18' = \underline{481\ ^{\#}/FT}$

USE 6" WALL w/ #6 @ 9" OC @ CENTER, COVER = $2\frac{5}{8}$"

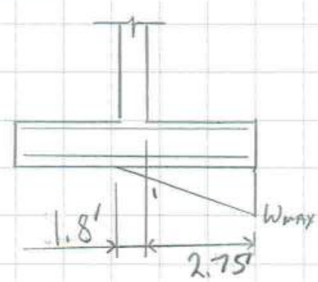
WALL OVERTURNING ANALYSIS

$A_{WALL} = 6.0' \cdot 1.0' + 18' \cdot 6"/12" = 15.0\ ft^2$

$W_{WALL} = 150\ \text{PCF} \cdot 15.0\ ft^2/ft = 2250\ \text{PLF}$

*SEE SPREADSHEET, $W_{MAX} = 1570$ PSF (L = 4.55')
$FS_{OVER} = 2.0 > 1.5$ OK✓

FOOTING DEMAND



$W_{MAX} = 1570\ \text{PLF}$

$W_1 = 1570\ \text{PLF}/4.55' \times 1.8' = 621\ \text{PLF}$

$M_{U1} = 621\ \text{PLF} \cdot (2.75')^2/2 = 2348\ ^{\#\cdot FT}$

$M_{U2} = \frac{(1570\ \text{PLF} - 621\ \text{PLF}) \cdot 2.75^{FT}}{2} \cdot \frac{2}{3} \cdot 2.75' = 2392\ ^{\#\cdot FT}$

$\underline{M_U = 4740\ ^{\#\cdot FT}}$

$V_U = 621\ \text{PLF} \cdot 2.75' + (1570 - 621) \cdot 2.75/2 = 3012\ ^{\#}$

#4 @ 12" OC OK✓

Smith Monroe Gray
ENGINEERS, INC.

Client: LAKESIDE INDUSTRIES

Job #: 18-183B

By: BS

Project:

Date: 11/1/2018 Sheet of

Concrete Slab Design per ACI 318-08

Applied Forces:

Ultimate Shear, V_u = 0.48 kips

Ultimate Moment, M_u = 5.286 ft-kips

Slab Properties:

Width = 12 in

Depth = 6 in

Cover = 2.625 in.

d = 3.00 in.

f'_c = 4000 psi

β_1 = 0.85

Capacity:

Shear: $\phi = 0.75$

$\Phi V_c = \Phi V_n = \phi*2*b*d*\sqrt{f'c}$

$\Phi V_c = \Phi V_n$ = 3.42 kips

Bending: $\phi = 0.9$

$\Phi M_n = \phi(As*fy*(d-a/2))$

ΦM_n = 6.78 k-ft.

Longitudinal Reinforcement:

Bar Size = 6

Spacing = 9 inches o.c.

f_y = 60000 psi

A_s = 0.59 in^2

a = 0.86 in

c = 1.01 in

Shrinkage and Temperature Reinforcing

Min. reinf. ratio = 0.0018

A_s min = 0.06 in^2 OK

max. spacing = 18.0 in

Check Tension Controlled (ACI 10.3.4)

$\varepsilon_t = [(d-c)/c]*0.003$

ε_t = 0.0059 **> 0.005, OK**

Demand Ratios:

$V_u/\Phi V_n$ =	**0.14**	**SLAB IS OK IN SHEAR**
$M_u/\Phi M_n$ =	**0.78**	**SLAB IS OK IN BENDING**

Smith Monroe Gray
ENGINEERS, INC.

CLIENT: LAKESIDE INDUSTRIES
PROJECT: MAPLE VALLEY ASPHALT PLANT
FOUNDATION DESIGN
BY: BS DATE: 11/1/2018
JOB #: 18-183B SHEET OF

DESIGN OF RECTANGULAR FOOTING WITH OVERTURNING MOMENT

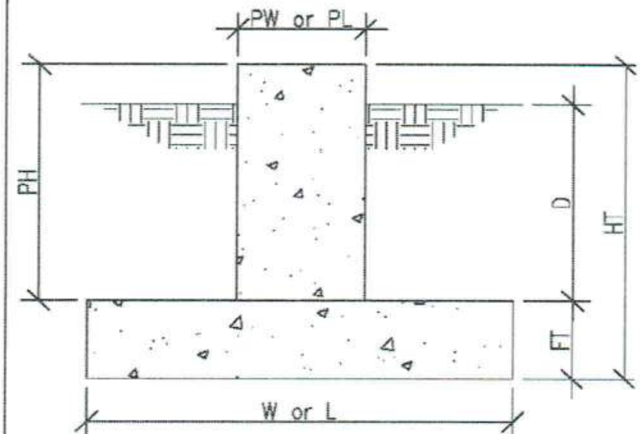
FOOTING:

LOADING PARAMETERS:

ALLOWABLE SOIL BEARING =	3,000	PSF
SOIL WEIGHT =	120	PCF
REQD. O.T. SAFETY FACTOR =	1.5	
STR.INCR.FOR HORIZ. LOADS =	1.33	
VERTICAL DEAD LOAD =	0.00	KIPS
VERTICAL LIVE LOAD =	0.00	KIPS
HORIZONTAL LOAD =	0.35	KIPS
MOMENT @ TOP OF FOOTING =	0.00	FT-KIPS

FOOTING DIMENSIONS:

FTG. LENGTH (L) =	6.00	FT (PAR.TO LOAD)
FTG. WIDTH (W) =	1.0	FT (PERP.TO LOAD)
FTG. THICKNESS (FT) =	1.00	FT
FOOTING DEPTH (D) =	2.0	FT
PIER LENGTH (PL) =	0.5000	FT
PIER WIDTH (PW) =	1.0	FT
PIER HEIGHT (PH) =	18.0	FT
CONCRETE WEIGHT =	2.25	KIPS
SOIL WEIGHT =	1.32	KIPS
TOTAL WEIGHT =	3.57	KIPS



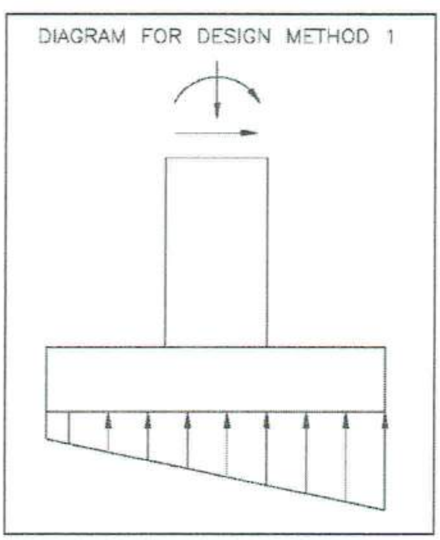
DESIGN METHOD 1

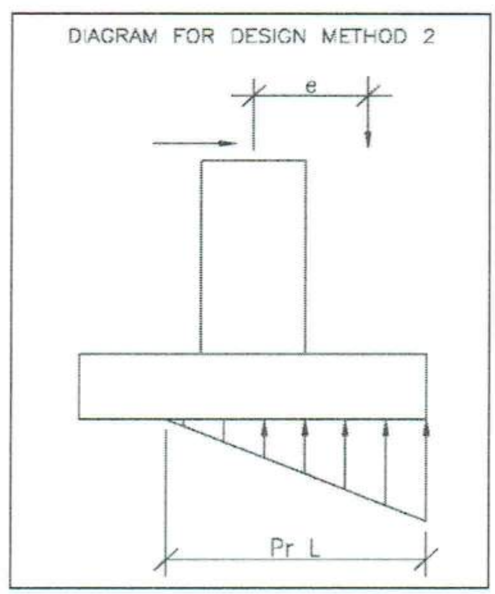
OVERTURNING MOM. =	5.3	FT-KIPS
SOIL PR. FROM DL =	595.0	PSF
SOIL PR. FROM MOM. =	(883.3)	PSF
MIN. PRESSURE =	(288.3)	PSF
MAX. PRESSURE =	1,478.3	PSF

DOES NOT APPLY AS UPLIFT AT BACK OF FOOTING

DESIGN METHOD 2

e =	1.48	FT	
Pr L =	4.55	FT	
MAX. PR =	1,570.5	PSF	**<--- GOVERNS**





	ACTUAL		ALLOWABLE		
LL + DL BEARING =	595	PSF	3,000	PSF	OK
DL + HORIZ. BEARING =	1,571	PSF	4,000	PSF	OK
F.S. OF OVERTURNING =	2.02		1.5		OK

Smith Monroe Gray
ENGINEERS, INC.

Client: LAKESIDE INDUSTRIES
Job #: 18-183B
By: BS
Project:
Date: 11/1/2018 Sheet of

Concrete Slab Design per ACI 318-08

Applied Forces:

Ultimate Shear, V_u = 3.01 kips
Ultimate Moment, M_u = 4.74 ft-kips

Slab Properties:

Width = 12 in
Depth = 12 in
Cover = 3 in.
d = 8.75 in.
f'_c = 4000 psi
β_1 = 0.85

Capacity:

Shear: $\phi = 0.75$
$\Phi V_c = \Phi V_n = \phi*2*b*d*\sqrt{f'c}$
$\Phi V_c = \Phi V_n$ = 9.96 kips

Bending: $\phi = 0.9$
$\Phi M_n = \phi(As*fy*(d-a/2))$
ΦM_n = 7.74 k-ft.

Demand Ratios:

$V_u/\Phi V_n$ = 0.30 **SLAB IS OK IN SHEAR**

$M_u/\Phi M_n$ = 0.61 **SLAB IS OK IN BENDING**

Longitudinal Reinforcement:

Bar Size = 4
Spacing = 12 inches o.c.
f_y = 60000 psi

A_s = 0.20 in^2
a = 0.29 in
c = 0.35 in

Shrinkage and Temperature Reinforcing

Min. reinf. ratio = 0.0018
A_s min = 0.19 in^2 OK
max. spacing = 18.0 in

Check Tension Controlled (ACI 10.3.4)

$\varepsilon_t = [(d-c)/c]*0.003$
ε_t = 0.0729 **> 0.005, OK**

Sound Attenuation Wall 2 (30' Max. Height)

Max Height = 30'

Per Previous $q_z = 22.4$ PSF

$s/h = 1.0$, $B/s = 6.0 \rightarrow C_s = 1.35$

$F_w = G \cdot q_z \cdot C_s = \underline{25.7 \text{ PSF}}$

$G = 0.85$
$C_s = 1.35$

$W_{12"} = 1^{FT} \cdot 150 \text{ PCF} \cdot 0.29$ ← C_s
$= \underline{43.5 \text{ PLF}}$ ← Controls

Wall Demand

$M_u = wl^2/2 = 43.5 \text{ PLF} (32')^2/2 = 22300^{\#\cdot FT}/FT$

$V_u = 43.5 \text{ PLF} \cdot 32' = 1392^{\#}/FT$

Use 12" Wall w/ #6 @ 8" OC @ (2) Faces

Wall Overturning (See Spreadsheet)

Wind:

$W_{max} = 2211$ PSF ($L = 4.88'$)
$FS_{over} = 2.25 > 1.5$ OK✓

Seismic:

$W_{max} = 3178$ PSF $L = 4.89^{FT}$
$FS_{over} = 1.57 > 1.5$

Footing Design

$W_1 = 3178 \text{ PLF} / 4.89' \times 1.39' = 903 \text{ PLF}$

$M_{u_1} = 903 \text{ PLF} (4.5')^2/2 = 9143^{\#\cdot FT}$ $M_{u_2} = \frac{(3178 \text{ PLF} - 903 \text{ PLF}) \cdot 4.5'}{2} \cdot \frac{2}{3} \cdot 4.5' = 15356^{\#\cdot FT}$

$\Sigma M_u = 24500^{\#\cdot FT}$ $V_u = 903 \text{ PLF} \cdot 4.5' + (3178 \text{ PLF} - 903 \text{ PLF}) \cdot 4.5/2 = 9182^{\#}$

Use 12" Footing w/ #7 @ 10" OC

Smith Monroe Gray
ENGINEERS, INC.

Client: LAKESIDE INDUSTRIES

Job #: 18-183B

By: BS

Project:

Date: 4/7/2019 Sheet of

Concrete Slab Design per ACI 318-14

Applied Forces:

Ultimate Shear, V_u = 1.34 kips

Ultimate Moment, M_u = 22.3 ft-kips

Slab Properties:

Width = 12 in

Depth = 12 in

Cover = 3 in.

d = 8.63 in.

f'_c = 4000 psi

β_1 = 0.85

Capacity:

Shear: $\phi = 0.75$

$\Phi V_c = \Phi V_n = \phi*2*b*d*\sqrt{f'c}$

$\Phi V_c = \Phi V_n$ = 9.82 kips

Bending: $\phi = 0.9$

$\Phi M_n = \phi(As*fy*(d-a/2))$

ΦM_n = 24.17 k-ft.

Demand Ratios:

$V_u/\Phi V_n$ =	0.14	**SLAB IS OK IN SHEAR**
$M_u/\Phi M_n$ =	0.92	**SLAB IS OK IN BENDING**

Longitudinal Reinforcement:

Bar Size = 6

Spacing = 8 inches o.c.

f_y = 60000 psi

A_s = 0.66 in^2

a = 0.97 in

c = 1.14 in

Shrinkage and Temperature Reinforcing

Min. reinf. ratio = 0.0018

A_s min = 0.19 in^2 OK

max. spacing = 18.0 in

Check Tension Controlled (ACI 10.3.4)

$\varepsilon_t = [(d-c)/c]*0.003$

ε_t = 0.0197 **> 0.005, OK**

DESIGN OF RECTANGULAR FOOTING WITH OVERTURNING MOMENT

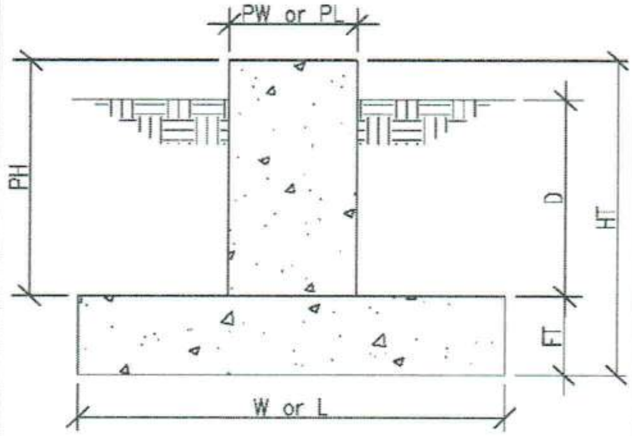
FOOTING: WIND

LOADING PARAMETERS:

ALLOWABLE SOIL BEARING =	3,000	PSF
SOIL WEIGHT =	120	PCF
REQD. O.T. SAFETY FACTOR =	1.5	
STR.INCR.FOR HORIZ. LOADS =	1.33	
VERTICAL DEAD LOAD =	0.00	KIPS
VERTICAL LIVE LOAD =	0.00	KIPS
HORIZONTAL LOAD =	0.77	KIPS
MOMENT @ TOP OF FOOTING =	0.00	FT-KIPS

FOOTING DIMENSIONS:

FTG. LENGTH (L) =	8.00	FT (PAR.TO LOAD)
FTG. WIDTH (W) =	1.0	FT (PERP.TO LOAD)
FTG. THICKNESS (FT) =	1.00	FT
FOOTING DEPTH (D) =	2.0	FT
PIER LENGTH (PL) =	1.0	FT
PIER WIDTH (PW) =	1.0	FT
PIER HEIGHT (PH) =	30.0	FT
CONCRETE WEIGHT =	5.70	KIPS
SOIL WEIGHT =	1.68	KIPS
TOTAL WEIGHT =	7.38	KIPS



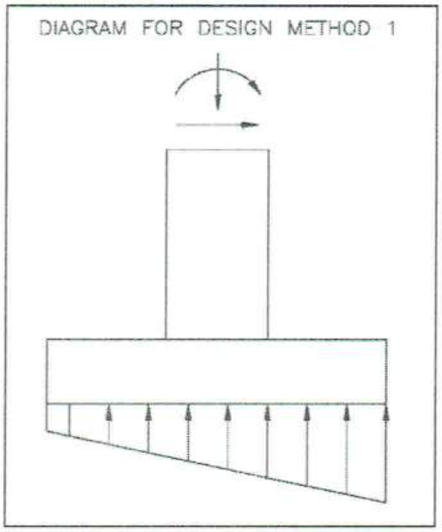
DESIGN METHOD 1

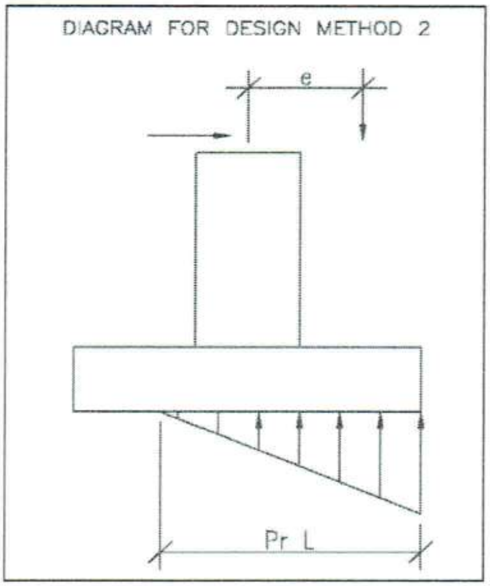
OVERTURNING MOM. =	13.1	FT-KIPS
SOIL PR. FROM DL =	922.5	PSF
SOIL PR. FROM MOM. =	(1,228.1)	PSF
MIN. PRESSURE =	(305.6)	PSF
MAX. PRESSURE =	2,150.6	PSF

DOES NOT APPLY AS UPLIFT AT BACK OF FOOTING

DESIGN METHOD 2

e =	1.78	FT
Pr L =	6.67	FT
MAX. PR =	2,211.3	PSF **<--- GOVERNS**





	ACTUAL		ALLOWABLE		
LL + DL BEARING =	923	PSF	3,000	PSF	OK
DL + HORIZ. BEARING =	2,211	PSF	4,000	PSF	OK
F.S. OF OVERTURNING =	2.25		1.5		OK

DESIGN OF RECTANGULAR FOOTING WITH OVERTURNING MOMENT

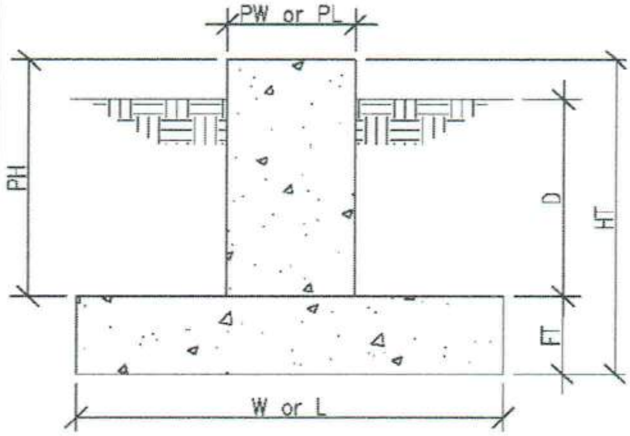
FOOTING:

LOADING PARAMETERS:

ALLOWABLE SOIL BEARING =	3,000	PSF
SOIL WEIGHT =	120	PCF
REQD. O.T. SAFETY FACTOR =	1.5	
STR.INCR.FOR HORIZ. LOADS =	1.33	
VERTICAL DEAD LOAD =	0.00	KIPS
VERTICAL LIVE LOAD =	0.00	KIPS
HORIZONTAL LOAD =	1.39	KIPS
MOMENT @ TOP OF FOOTING =	0.00	FT-KIPS

FOOTING DIMENSIONS:

FTG. LENGTH (L) =	9.00	FT (PAR.TO LOAD)
FTG. WIDTH (W) =	1.0	FT (PERP.TO LOAD)
FTG. THICKNESS (FT) =	1.00	FT
FOOTING DEPTH (D) =	2.0	FT
PIER LENGTH (PL) =	1.0	FT
PIER WIDTH (PW) =	1.0	FT
PIER HEIGHT (PH) =	30.0	FT
CONCRETE WEIGHT =	5.85	KIPS
SOIL WEIGHT =	1.92	KIPS
TOTAL WEIGHT =	7.77	KIPS



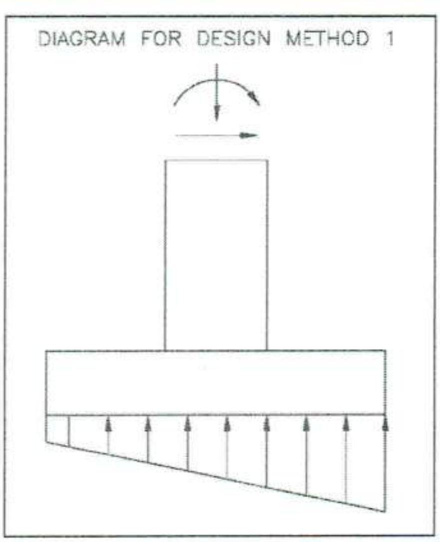
DESIGN METHOD 1

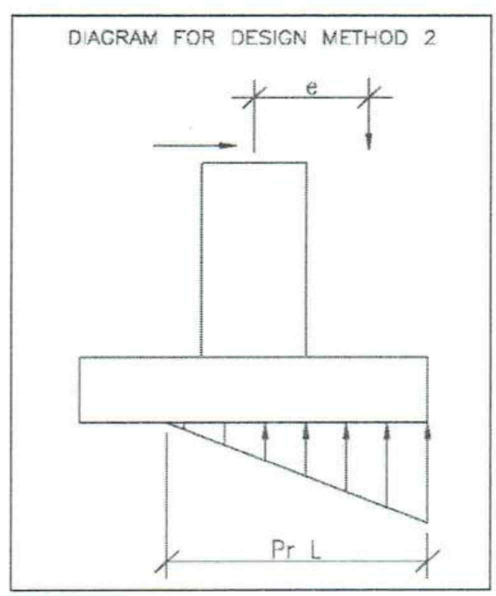
OVERTURNING MOM. =	22.3	FT-KIPS
SOIL PR. FROM DL =	863.3	PSF
SOIL PR. FROM MOM. =	(1,651.9)	PSF
MIN. PRESSURE =	(788.5)	PSF
MAX. PRESSURE =	2,515.2	PSF

DOES NOT APPLY AS UPLIFT AT BACK OF FOOTING

DESIGN METHOD 2

e =	2.87	FT	
Pr L =	4.89	FT	
MAX. PR =	3,177.9	PSF	**<--- GOVERNS**





	ACTUAL		**ALLOWABLE**		
LL + DL BEARING =	863	PSF	3,000	PSF	OK
DL + HORIZ. BEARING =	**3,178**	PSF	4,000	PSF	OK
F.S. OF OVERTURNING =	**1.57**		1.5		OK

Smith Monroe Gray
ENGINEERS, INC.

Client: LAKESIDE INDUSTRIES

Job #: 18-183B

By: BS

Project:

Date: 4/7/2019 Sheet of

Concrete Slab Design per ACI 318-14

Applied Forces:

Ultimate Shear, V_u = 9.2 kips

Ultimate Moment, M_u = 25 ft-kips

Slab Properties:

Width = 12 in

Depth = 12 in

Cover = 3 in.

d = 8.56 in.

f'_c = 4000 psi

β_1 = 0.85

Capacity:

Shear: $\phi = 0.75$

$\Phi V_c = \Phi V_n = \phi*2*b*d*\sqrt{f'c}$

$\Phi V_c = \Phi V_n$ = 9.75 kips

Bending: $\phi = 0.9$

$\Phi M_n = \phi(As*fy*(d-a/2))$

ΦM_n = 26.03 k-ft.

Demand Ratios:

$V_u/\Phi V_n$ = **0.94** **SLAB IS OK IN SHEAR**

$M_u/\Phi M_n$ = **0.96** **SLAB IS OK IN BENDING**

Longitudinal Reinforcement:

Bar Size = 7

Spacing = 10 inches o.c.

f_y = 60000 psi

A_s = 0.72 in^2

a = 1.06 in

c = 1.25 in

Shrinkage and Temperature Reinforcing

Min. reinf. ratio = 0.0018

A_s min = 0.18 in^2 OK

max. spacing = 18.0 in

Check Tension Controlled (ACI 10.3.4)

$\varepsilon_t = [(d-c)/c]*0.003$

ε_t = 0.0176 **> 0.005, OK**

DRAWN BY: ADAM COCHRAN

WALL HT H	TYPE 2A W	TYPE 2A t	TYPE 2A BARS "A"&"F"	TYPE 2A BAR "B"	TYPE 2A BARS "D"&"E"	TYPE 2B W	TYPE 2B t	TYPE 2B BARS "A"&"F"	TYPE 2B BAR "B"	TYPE 2B BARS "D"&"E"
6' - 0"	2' - 0"	5"	3 ~ #4	#4 @ 18"	#3 @ 15"	2' - 3"	5"	3 ~ #4	#4 @ 18"	#3 @ 12"
8' - 0"	2' - 3"	5"	3 ~ #4	#4 @ 18"	#3 @ 12"	2' - 9"	5"	3 ~ #4	#4 @ 18"	#4 @ 15"
10' - 0"	2' - 6"	5"	3 ~ #4	#4 @ 18"	#3 @ 9"	3' - 3"	5"	5 ~ #4	#4 @ 18"	#4 @ 10"
12' - 0"	3' - 0"	5"	3 ~ #4	#4 @ 18"	#4 @ 12"	3' - 9"	5"	5 ~ #4	#4 @ 18"	#4 @ 10"
14' - 0"	3' - 3"	5"	3 ~ #4	#4 @ 18"	#4 @ 10"	4' - 3"	5"	5 ~ #4	#4 @ 18"	#5 @ 11"
16' - 0"	3' - 9"	5"	5 ~ #4	#4 @ 18"	#4 @ 9"	4' - 9"	5"	5 ~ #4	#4 @ 18"	#6 @ 12"
18' - 0"	4' - 0"	5"	5 ~ #4	#4 @ 18"	#5 @ 11"	5' - 3"	5"	5 ~ #4	#4 @ 18"	#6 @ 9"
20' - 0"	5' - 0"	5"	5 ~ #4	#4 @ 18"	#5 @ 9"	6' - 0"	6"	5 ~ #4	#4 @ 18"	#6 @ 9"
22' - 0"	5' - 6"	5"	5 ~ #4	#4 @ 18"	#5 @ 7"	6' - 3"	6"	5 ~ #4	#4 @ 12"	#6 @ 8"
24' - 0"	6' - 0"	5"	5 ~ #4	#4 @ 15"	#5 @ 6"	6' - 9"	7"	5 ~ #4	#4 @ 11"	#6 @ 8"

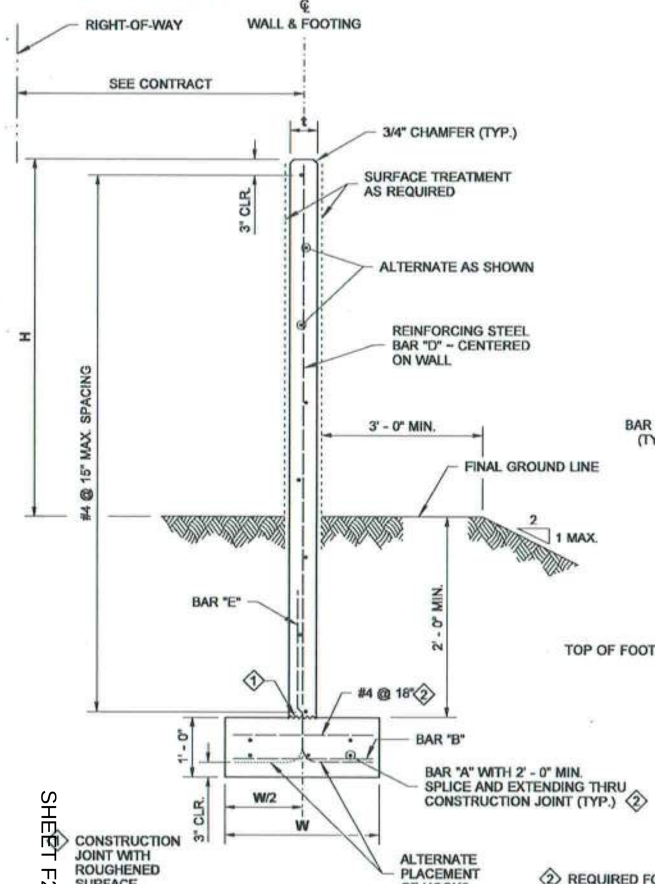
TYPE 2C W	TYPE 2C t	TYPE 2C BARS "A"&"F"	TYPE 2C BAR "B"	TYPE 2C BARS "D"&"E"	TYPE 2D W	TYPE 2D t	TYPE 2D BARS "A"&"F"	TYPE 2D BAR "B"	TYPE 2D BARS "D"&"E"	WALL HT H
2' - 0"	5"	3 ~ #4	#4 @ 18"	#3 @ 15"	2' - 6"	5"	3 ~ #4	#4 @ 18"	#3 @ 11"	6' - 0"
2' - 6"	5"	3 ~ #4	#4 @ 18"	#3 @ 10"	3' - 3"	5"	5 ~ #4	#4 @ 18"	#4 @ 12"	8' - 0"
2' - 9"	5"	3 ~ #4	#4 @ 18"	#4 @ 12"	3' - 6"	5"	5 ~ #4	#4 @ 18"	#4 @ 10"	10' - 0"
3' - 3"	5"	3 ~ #4	#4 @ 18"	#4 @ 10"	4' - 3"	5"	5 ~ #4	#4 @ 18"	#5 @ 12"	12' - 0"
3' - 9"	5"	3 ~ #4	#4 @ 18"	#4 @ 9"	4' - 9"	5"	5 ~ #4	#4 @ 18"	#5 @ 8"	14' - 0"
4' - 3"	5"	5 ~ #4	#4 @ 18"	#5 @ 11"	5' - 6"	5"	5 ~ #4	#4 @ 18"	#6 @ 9"	16' - 0"
4' - 6"	5"	5 ~ #4	#4 @ 18"	#6 @ 12"	6' - 0"	6"	5 ~ #4	#4 @ 18"	#6 @ 9"	18' - 0"
5' - 3"	5"	5 ~ #4	#4 @ 18"	#6 @ 10"	6' - 6"	6"	5 ~ #4	#4 @ 12"	#6 @ 7 1/4"	20' - 0"
5' - 9"	6"	5 ~ #4	#4 @ 18"	#6 @ 9"	7' - 0"	7"	5 ~ #4	#4 @ 11"	#6 @ 7"	22' - 0"
6' - 3"	6"	5 ~ #4	#4 @ 12"	#6 @ 8"	7' - 6"	7"	5 ~ #4	#4 @ 9"	#6 @ 6 1/4"	24' - 0"

t = WALL THICKNESS

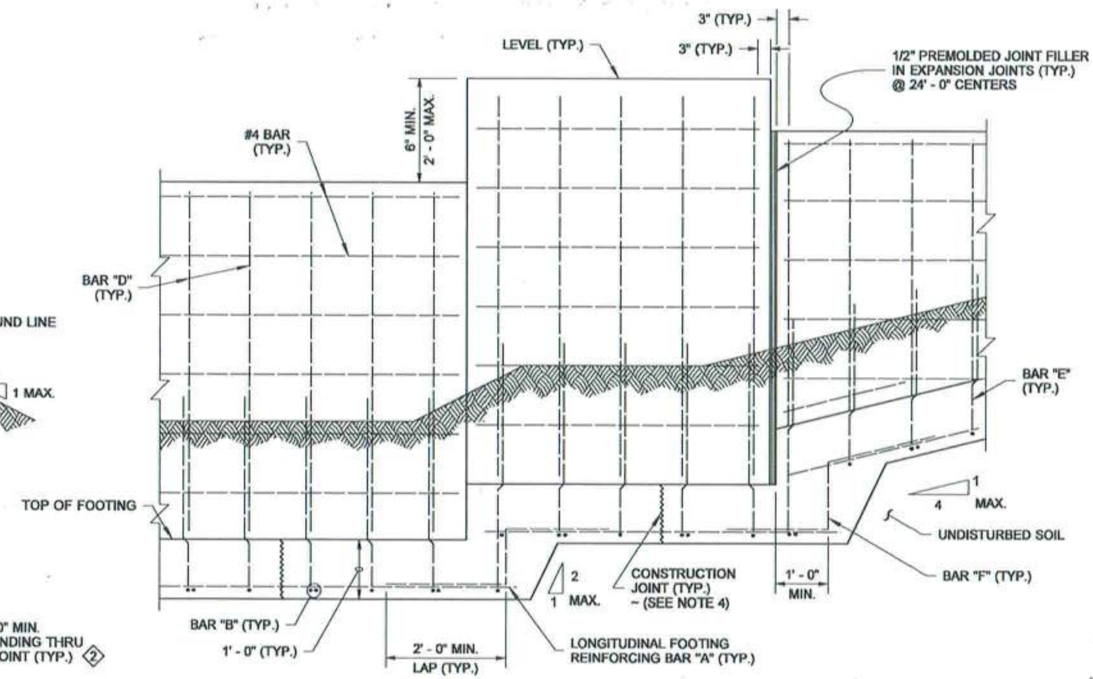
NOTES

1. Wall to be designated Noise Barrier Wall Type 2A, 2B, 2C or 2D. The Contract specifies actual wall designations.
2. For intermediate wall heights not listed, use the next higher H.
3. Panels shall have at least 3 feet of level ground on each side.
4. Construction joints in the footing shall be spaced at 120 feet maximum.

WIND EXPOSURE & VELOCITY		
NOISE BARRIER TYPE	WIND EXPOSURE	WIND VELOCITY (MPH)
2A	B1	80
2B	B1	90
2C	B2	80
2D	B2	90



TYPICAL SECTION



ELEVATION

REQUIRED FOR WALLS 24' - 0" ~ TYPE 2C, WALL HEIGHT 22' - 0" & 24' - 0" ~ TYPE 2B & WALLS 20' - 0", 22' - 0" & 24' - 0" ~ TYPE 2D

CAST-IN-PLACE CONCRETE WALL ON SPREAD FOOTING

EXPIRES AUGUST 23, 2006

NOTE: THIS PLAN IS NOT A LEGAL ENGINEERING DOCUMENT BUT AN ELECTRONIC DUPLICATE. THE ORIGINAL, SIGNED BY THE ENGINEER AND APPROVED FOR PUBLICATION, IS KEPT ON FILE AT THE WASHINGTON STATE DEPARTMENT OF TRANSPORTATION. A COPY MAY BE OBTAINED UPON REQUEST.

NOISE BARRIER WALL TYPE 2

STANDARD PLAN D-2.04-00

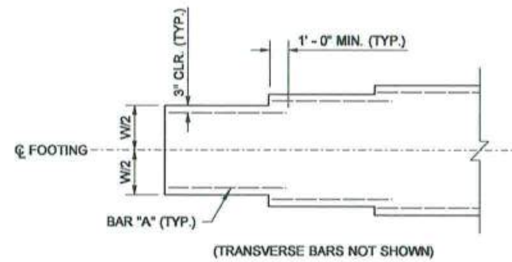
SHEET 1 OF 2 SHEETS

APPROVED FOR PUBLICATION

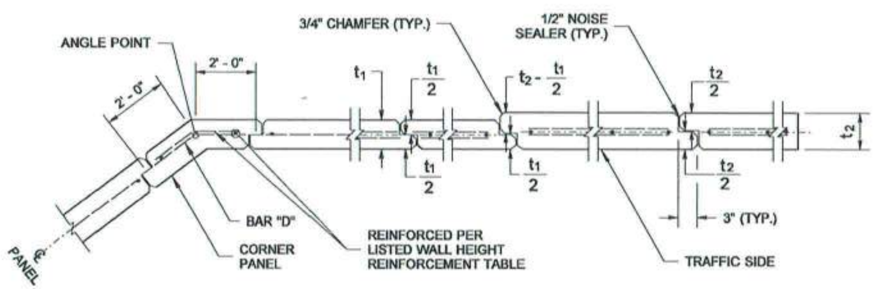
Harold J. Peterfeso — STATE DESIGN ENGINEER — 11-10-05 DATE

Washington State Department of Transportation

DRAWN BY: ADAM COCHRAN



FOOTING WIDTH TRANSITION DETAIL
FOR LOCATIONS WITHOUT FOOTING STEP



JOINT AND CORNER DETAIL

CAST-IN-PLACE CONCRETE WALL ON SPREAD FOOTING

BIJAN KHALEGHI
STATE OF WASHINGTON
27695
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES AUGUST 23, 2006

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NOISE BARRIER WALL TYPE 2

STANDARD PLAN D-2.04-00

SHEET 2 OF 2 SHEETS

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Washington State Department of Transportation

RAMBOLL

Onsite Structures and Noise Barriers

Lakeside Maple Valley Asphalt Plant Site
King County, Washington

FIGURE 2

DRAFTED BY: KB | DATE: 10/30/2018 | Project: 1690002813