EQUILIBRIUM
ENGINEERS LLC

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August 12, 2013

Paul Smith
JRJ architects, llc
15455 NW Greenbrier Parkway, Suite 260
Beaverton, OR 97006

RE: Providence Portland Medical Center
Suite 156-ASC
EE LLC Job No. 12043

13-100880-FA

Dear Paul:

Attached please find calculation sheets C1 through C32, which verify the structural adequacy of the following:

- Floor infill at removed hot tub
- Replacement of east wall demo
- Analysis of floor structure for new mechanical unit
- New ceilings, soffits, and interior non-bearing walls
- Infill of existing pool and slab on grade at pool footprint
- Anchorage of dual LED overhead light boom to structure

These are shown on the structural drawings S1.0 through S3.2, dated August 12, 2013. Design is based on the provisions of the 2009 International Building Code, 2010 OSSC, information indicated on the original structural drawings for the building, and information gathered during our site visit on July 19, 2013.

Please call if you have any questions.

Sincerely,

Justin Fenton, PE
Project Engineer

Enclosure

STRUCTURAL
REGISTERED PROFESSIONAL ENGINEER
OREGON
July 22, 1997
EDWARD P. QUESENBERRY
EXPIRES 12/31/2014

16325 Boones Ferry Road, Suite 202 ■ Lake Oswego, Oregon 97035
Phone 503.636.8388 ■ Cell 503.803.8576 ■ Fax 503.636.2090 ■ Email edq@equilibriumllc.com

Project PPI ASC	EQUILIBRIUM ENGINEERS	By FENTON	Sheet # C1
Location PORTLAND, OR		Date 7/31/13	
Client		Revised	Job # 12043
		Date	

T.I NARRATIVE

DESIGN INCLUDES PARTITIONS, SUSPENDED CEILINGS SUSPENDED COFFITS, CEILINGS, POOL INFILL, TUB INFILL, EQUIPMENT ANCHORAGE

DESIGN CRITERIA

2009 IBC
2010 OSSC
ASC 7-05

DEAD LOADS

SOFFITS 7.5 PSF
PARTITIONS 10 PSF
SUSPENDED CEILING 4 PSF

LATERAL LOAD (INTERIOR)

5 PSF MIN. OR AS DETERMINED BY F_p FOR EA. ITEM

SEISMIC, GENERAL

LATITUDE: 45.526843
LONGITUDE: -122.610753
PER USGS:
$S_s = 0.975g$ $S_1 = 0.333g$
$F_a = 1.11$ $F_v = 1.933$
OCCUPANCY CATEGORY IV
$I_p = 1.5$
$S_{DS} = 0.721$ $S_{D1} = 0.365$
SEISMIC DESIGN CATEGORY D

$$F_p = \frac{0.4\, a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2(z/H)\right)$$

$z/H = 14.5/97.5 = 0.15$

$F_{PH} = 0.5624\, W_p \left(a_p / R_p\right)$

$0.7 F_{PH} = 0.394 \left(a_p / R_p\right) W_p$

$0.7 F_{PH_{MIN}} = 0.227\, W_p$

INTERIOR PARTITIONS

$a_p = 1.0$
$R_p = 2.5$ (1.5 FOR ANCHORS)

$0.7 F_p = 0.394 \times 10\text{PSF} \left(1/2.5\right) = 1.576 \text{ PSF} < 5 \text{ PSF}$

$0.7 F_{p_{MIN}} = 0.227 W_p = 0.227 \times 10 \text{ PSF} = 2.27 F_p < 5 \text{ PSF}$

∴ 5 PSF GOVERNS PARTITION DESIGN INCLUDING ANCHOR DESIGN
$R_p = 1.5$

- TYPICAL PARTITIONS

3625125-30 @ 16" o.c.
SPAN = 15'-6" FOR L/240

OR 3625125-43 @ 24" o.c.
SPAN = 15'-3" FOR L/240

PER SSMA CATALOG

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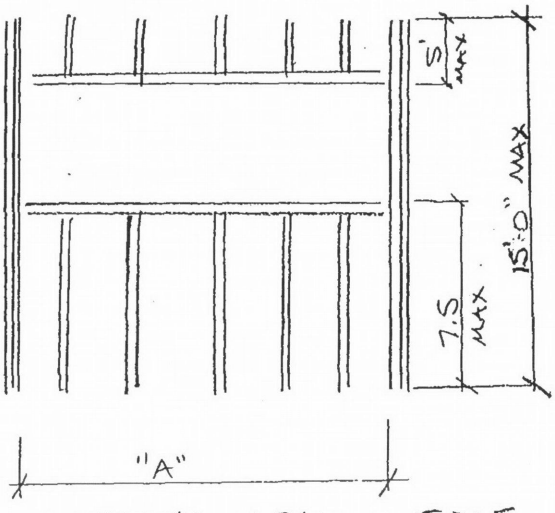
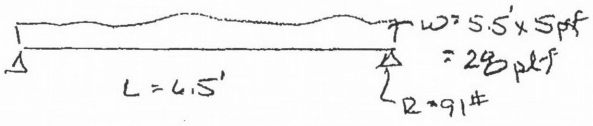
INTERIOR PARTITIONS



LATERAL LOAD = 5 PSF (ASD)

• WHERE "A" ≤ 6'-6"

SILL



$M = 0.147^{'k}$

$S_{req} = \frac{0.147^{'k} \times 12}{0.66 \times 33^{ksi}} = 0.0814 \text{ in}^3$

$S_{362T125\cdot30} = 0.152 \text{ in}^3$ ∴ OK

$\Delta_{req} = \frac{6.5 \times 12}{240} = 0.325''$

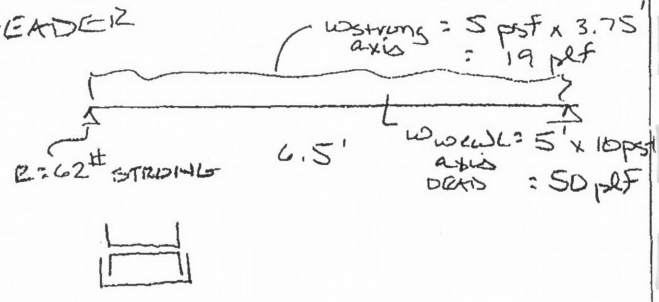
$I_{req} = \frac{5(0.028/12)(6.5 \times 12)^4}{384 \times 0.325'' \times 29000^{ksi}}$

$= 0.119 \text{ in}^4$

$I_{362T125\cdot30} = 0.339 \text{ in}^4$ ∴ OK

∴ (1) 362 T125-30 OK FOR SILL

HEADER



BY OBSERVATION (CALC FOR SILL)

(1) 362T125-30 OK FOR STRONG AXIS LOAD OF 19 PLF

WEAK AXIS:

$M = 0.264^{'k}$

$S_{req} = \frac{0.264^{'k} \times 12}{0.66 \times 33^{ksi}} = 0.145 \text{ in}^3$

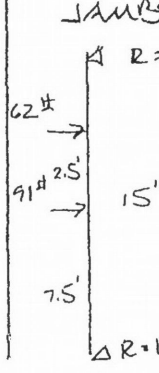
$S_{y\ 362T125\cdot30} = 0.1028 \text{ in}^3$
$\times\ 3$
0.3084 in^3
$> 0.145 \text{ in}^3$
∴ OK
(3) 362T125-30

$I_{req} = \frac{5(0.05/12)(6.5 \times 12)^4}{384 \times 0.325'' \times 29000^{ksi}}$

$= 0.213 \text{ in}^4$

$I_{y\ actual} = 0.242 \text{ in}^4$
$> 0.213 \text{ in}^4$
∴ (3) 362T125-30 OK

JAMBS



Try (2) 362 S125-30
$w_{DL} = 5$ plf
$S = 0.35 \text{ in}^3$
$I = 0.75 \text{ in}^4$

$M = 0.64^{'k}$

$\Delta_{w8 \times 10} = 0.022''$

$S_{req} = 0.64^{'k} \times 12 / 0.66 \times 33^{ksi}$
$= 0.353 \text{ in}^3$

$I_{req} = \frac{0.022 \times 30.8 \text{ in}^4}{15' \times 12/180} = 0.678 \text{ in}^4$

(2) 362S125-30 OK FOR

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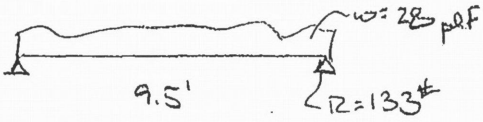
EQUILIBRIUM ENGINEERS

• WHERE 6'-6" < "A" ≤ 9'-6"

SILL



$S_{req} = \frac{0.315^{k} \times 12}{0.66 \times 33^{ksi}} = 0.174\ in^3$

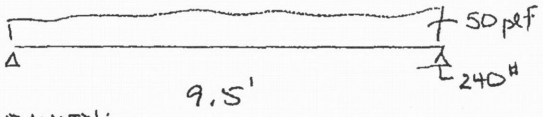
∴ (3)362T125-30 OK
$3 \times 0.152\ in^3 = 0.456\ in^3 > 0.174\ in^3$

$I_{req} = \frac{5(0.028/12)(9.5 \times 12)^4}{384 \times \frac{9.5' \times 12}{180} \times 29000}$

$= 0.279\ in^3$

(3)362T125-30 OK
$3 \times 0.339\ in^4 > 0.279\ in^4$

HEADER

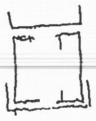


GRAVITY:
$M = 0.564^{'k}$

$S_{req} = \frac{0.564^{'k} \times 12}{0.66 \times 33^{ksi}} = 0.31\ in^3$

$I_{req} = \frac{5(0.05/12)(9.5' \times 12)^4}{384 \times 0.475'' \times 29000}$

$= 0.665\ in^4$



(4)362T125-30

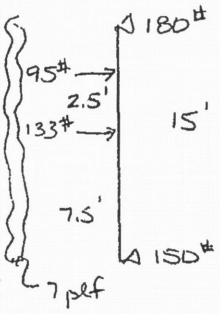
$S = 0.304\ in^3 \approx 0.31\ in^3$
$I = 0.678\ in^4 > 0.665\ in^4$
∴ OK

OUT-OF-PLANE

(2)362T125-30
$I = 2 \times 0.339\ in^4 = 0.678\ in^4$
$S = 2 \times 0.152\ in^3 = 0.304\ in^3$

per calc for sill,
(2)362T125-30 OK

JAMB



$M = 1.0^{'k}$

$\Delta_{MC\ 3 \times 7.1} = 0.436''$

$S_{req} = \frac{1.0^{'k} \times 12}{0.66 \times 33^{ksi}} = 0.551\ in^3$

$I_{req} = \frac{0.436'' \times 2.74\ in^4}{15' \times 12/180}$

$= 1.19\ in^4$

(2)362T125-30
+ (2)362S125-30

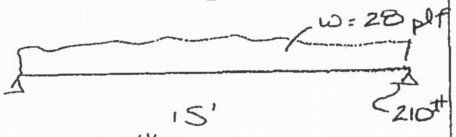
$S = 2 \times .175\ in^3 + 2 \times .152\ in^3$
$= 0.654\ in^3 > 0.551\ in^3$

$I = 2 \times .375\ in^4 + 2 \times .339\ in^4$
$= 1.43\ in^4 > 1.19\ in^4$

∴ OK

• WHERE 9'-6" < "A" ≤ 15'-0"

SILL



$M = 0.788^{'k}$

$S_{req} = 0.434\ in^3$

$I_{req} = 1.09\ in^4$

(2)+rackS + (1) STUD
362T125-30 362S125-30

$S = 2 \times .152\ in^3 + 1 \times .175\ in^3 = 0.479\ in$

$I = 2 \times .339\ in^4 + 1 \times .375\ in^4 = 1.05\ in^4$

∴ (2)362T125+(1)362S125-30

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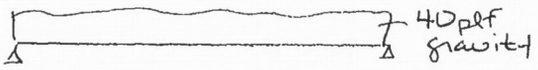
EQUILIBRIUM ENGINEERS

HEADER

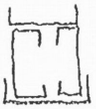


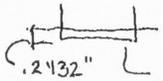
15'

GRAVITY $M = 1.125^{'k}$

$S_{req} = 0.620 in^3$

$I_{req} = 2.09 in^4$





$S_x = 0.0248 in^3$

$I_x = 0.025 in^4$

$A = .182 in^2$

$I = 2 \times 0.375 in^4$

$+ 0.025 in^4 + .182 in^2 (1.57)^2$

$+ 0.025 in^4 + .182 in^2 (2.06)^2$

$= 2.02 in^4 \approx 2.09 in^4$

$\therefore$ OK

$S = \frac{2.02 in^4}{1.81 in} = 1.11 in^3 > 0.62 in^3$

$\therefore$ (2)362S125-30

+(2)362T125-30 OK

OUT-OF-PLANE

15'

$M = 0.563^{'k}$

$S_{req} = 0.3099 in^3$

$I_{req} = 0.785 in^4$

$I = 2 \times 0.339 in^4$

$+ [0.025 in^4 + .182 in^2 (1.57)^2] 2$

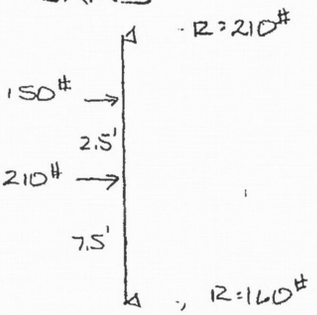
$= 1.63 in^4 > 0.785 in^4$

$S = \frac{1.63 in^4}{1.81 in} = 0.901 in^3 > 0.3099 in^3$

$\therefore$ (2)362S125-30

+(2)362T125-30 OK

JAMB



$M = 1.16^{'k}$

$A_{MC3x7.1} = 0.525$

$S_{req} = \frac{1.16^{'k} \times 12}{0.66 \times 33^{ksi}} = 0.639 in^3$

$I_{req} = \frac{0.525'' \times 2.72^2}{15' \times 12/180} = 1.42 in^4$

(2)362S125-30

+(2)362T125-30

$S = 0.654 in^3 > 0.639 in^3$

$I = 1.43 in^4 \geq 1.42 in^4$

$\therefore$ OK

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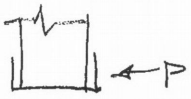
EQUILIBRIUM ENGINEERS

• PARTITION CONNECTIONS

- TO CONCRETE & CONC. FILLED DECK BASE CONN.

TYP. STUD 

$P = 5PSF \times 16/12 \times 15'/2$

$= 50^{\#}$ (ASD) (INCLUDES $R_p = 1.5$)

0.145" Ø HILTI X-DNI

EMBED 1"

$V_a = 212^{\#} \times .5 = 106^{\#}$ (SF; $f_c' = 3ksi$)

$P_{applied} = 50^{\#} \times 1.3$ (S.F. per ASCE 13.4.2)

$= 65^{\#} < V_a = 106^{\#}$

∴ 0.145" Ø HILTI X-DNI

EMBED 1"

@16" o.c. OK

AT JAMB

$P_{max} = 160^{\#} \times 1.3$ (S.F.)

$= 208^{\#}$

$\text{No. Req} = \frac{208^{\#}}{106^{\#}} = 2.0$

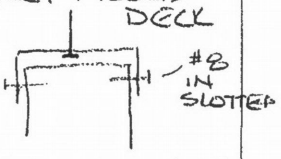
∴ (3) 0.145" Ø

HILTI X-DNI

@4" o.c.

EMBED 1"

- HEAD CONN. TO CONC. FILLED DECK

TYP. STUD 

$P_{max} = 50^{\#}$ (ASD - INCLUDES $R_p = 1.5$)

$\times 1.3$

$65^{\#}$

(1) 0.145" Ø HILTI X-DNI

EMBED 1"

$V_a = 320^{\#}$ per ESR-1663 TABLE 3

$\times .5$ (SF)

$160^{\#} > 65^{\#}$

∴ OK

V_a #8 screw in 30 GA = $50^{\#}$ per ESR-2196 for HILTI KWIK PRO

$V_{applied} = 50^{\#}/2$ screws

$= 25^{\#}$

$T_a = 50^{\#} > 25^{\#}$ ∴ OK

CHECK GA. OF TRACK

$t_{req} = \sqrt{\frac{7.5 \times (50^{\#}/2 \text{ sides})(1.125")}{33{,}000 psi \times 9.5"}}$

$= 0.026"$

∴ 18 ga OK

$t = 0.043"$

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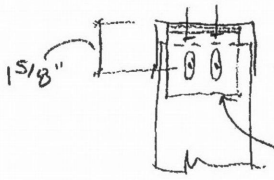
EQUILIBRIUM ENGINEERS

HEAD CONN. AT JAMBS

AT CONC. FILLED DECK



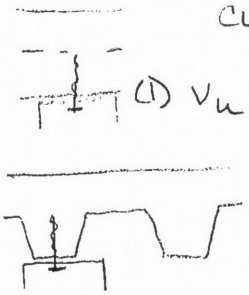
R = ← fastener to conn.
← 210# x 1.3
= 273#

L3 1/4 x 2 x 14GA
x 3 1/2" LONG

(2) #8 SCREWS IN 30 GA. & 14 GA.

V_a = 150# x 2 screws
= 300# > 210#

(1) 3/8" Ø TITEN HD SCREW
1 1/2" EMBED AT LOW FLUTE
CLOSEST TO JAMB

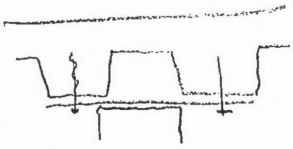


(1) $V_u = \frac{273^{\#}}{.7} = 390^{\#}$

or $V_u = \frac{210^{\#}}{.7} \times 2.5$ ← per IBC 1908.1.16
= 750# ← governs

UNITY = 0.558
ϕV_c = 1344# ∴ OK

∴ OK PER ATTACHED



0.145" Ø HILTI
X-DNI
EMBED 1"

V_a = 320# per ESR-1663
S.F. x.5 TABLE 3
160#

$V_{applied}$ = 273#

No. Req'd = $\frac{273^{\#}}{160^{\#}}$ = 1.7

∴ (4) 0.145" Ø
HILTI X-DNI
OK

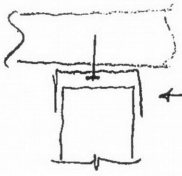
if substitute Hilti 3/8" Ø
KB-TZ, Embed 2"

$V_{app.}$ = 273#

V_{allow} = 944# per ESR-1917 TABLE 11

∴ OK TO SUBSTITUTE
3/8" Ø HILTI KB-TZ
EMBED 2"
FOR 3/8" Ø TITEN HD.

– FOR CONNECTION TO ROOF DECK APPLICATION
(ASSUME MIN. 20 GA. DECK)



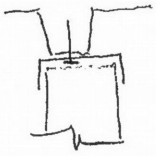
← P_{MAX} = 50# @ TYP
210# @ JAMB

#8 SCREW IN
18 GA TO 20 GA
DECK

V_{allow} = 174# PER ESR-2196

∴ #8 HILTI KWIK PRO
@ 16" O.C. MAX
AT LOW FLUTE
OK

∴ (2) #8 @ JAMBS OK



(1) #8 @ TYP OK
(2) #8 @ JAMBS OK
PER ABOVE

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name :

Date/Time : 2/8/2010 3:28:32 PM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
3/8" Titen HD	N/A	1	1.5	1

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Sand lightweight over Metal Deck	Yes	3000.0	N/A

Condition	Thickness (in)	Suppl. Edge Reinforcement
B tension and shear	see figure	N/A

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
0	0	750	0	0

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

Individual Anchor Tension Loads

N_{ua1} (lb)
0.00

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

Individual Anchor Shear Loads

V_{ua1} (lb)
750.00

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

Tension Strengths

Steel (Φ = 0.65)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
10890	7078.50	0.00	0.0000

Concrete Breakout does not apply

Pullout (Φ = 0.65 , Φ_{seis} = 0.75)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{pn}$
580.00	282.75	0.00	0.0000

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.60)

V_{eq}(lb)	ΦV_{eq}(lb)	V_{ua}(lb)	$V_{ua}/\Phi V_{eq}$
2240	1344.00	750.00	0.5580

Concrete Breakout does not apply

Pryout does not apply

Interaction check

T.Max(0) <= 0.2 and V.Max(0.56) <= 1.0 [Sec D.7.2]

Interaction check: PASS

Use 3/8" diameter Titen HD anchor(s) with 1.5 in. embedment

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

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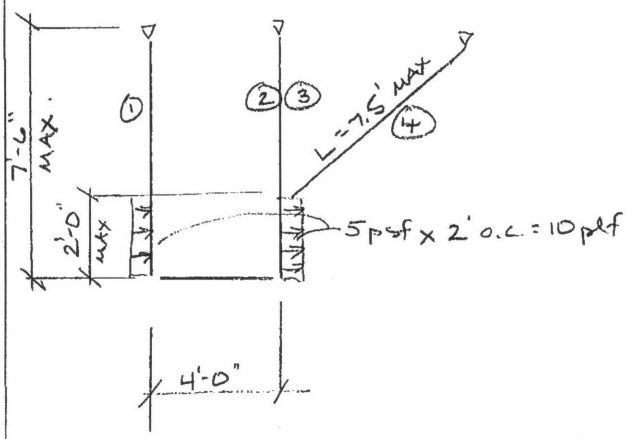
Client		Revised		Job #	12043
		Date			

EQUILIBRIUM ENGINEERS LLC

SUSPENDED SOFFITS

- AT CONCRETE OVER METAL DECK:



DEAD LOAD = 7.5 PSF x 2' = 15 plf

HANGER DL = 15 plf x 7.5'
+ 15 plf x 2'
143#

MEMBER 1:

$M_{D1} = 0.1^{'k} = 1.2^{''k}$

$\Delta = 0.08''$ ∴ 362S125-30 OK @ 24" o.c

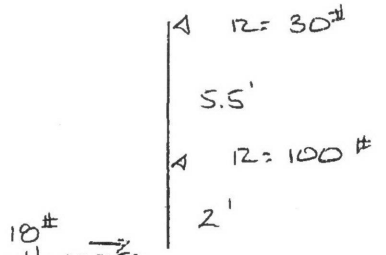
$M_a = 3.46^{''k} > 1.2^{''k}$

$R_{H\ TOP} = 4^{\#}$
$R_{H\ BOT} = 18^{\#}$
$R_{V\ TOP} = 143^{\#} + 22^{\#} \times .7$ (Fpv)
$= 158^{\#}$

MEMBER 2:

NO KICKER LOCATION
SAME AS MEMBER 1
∴ 362S125-30 @ 24" o.c. OK

MEMBER 3 AT KICKER @ 48" o.c.



18#
x 4 VERTS
72#

$R_V = 143^{\#}$ + VERT. KICKER + .7F COMPONENT
$= 143^{\#} + 100^{\#} + 15^{\#} = 258^{\#}$

$M = 0.144^{'k} = 1.73^{''k} < 3.46^{''k}$ M_a 362S125-30
∴ OK

$\Delta = 0.082''$ $l/585$ ∴ OK

∴ 362S125-30 @ 24" o.c OK
w/ KICKER @ 48" o.c

MEMBER 4 KICKER

$\frac{kl}{r_y} = \frac{7.5' \times 12}{.378} = 238 > 200$ ∴ n

∴ use (2) 362T125-30 for lengths > 5'-6"

$\frac{kl}{r_y} = \frac{5.5' \times 12}{.378} = 174$

∴ use (1) 362T125-30 @ 48" o.c.

$P = 100^{\#}/\cos 45 = 142^{\#}$

$P_a = 0.6 \times 33^{ksi} \times 2 \times .191\ in^2 \times .658^{(1.47)}$
$= 4.1^{k} > .142^{k}$

∴ OK 362T125-30 FOR KICKER

$V_{(2)\#8\ SMS} = 145^{\#}$ PER ESR-2196
∴ (2) #8 OK > 142#

Project PPP ASC	EQUILIBRIUM ENGINEERS	By FENTON	Sheet # C9
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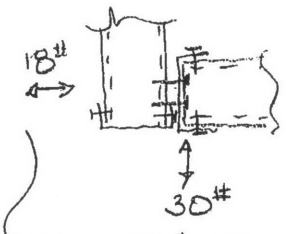
CONNECTIONS:

GRAVITY AT BOTTOM



MAX = 54# AT KICKER STUD

SMS #8

$V_a = 145^{\#}$ IN 30 MIL.

$T_a = 62^{\#}$ IN 30 MIL (PER ESR-2196)

$T_{max} = 54^{\#} < 62^{\#}$

∴ (1) SCREW TO CARRY TENSION

$V_{MAX} = 54^{\#} < 145^{\#}$

∴ (1) SCREW TO TRANSFER SHEAR

∴ (2) #8 SMS OK AT EA. CONNECTION (HILTI KWIK-PRO'S)

TOP CONNECTION AT HANGER w/o KICKER

$T_{ASD} = 158^{\#}$ $T_u = 194^{\#}$

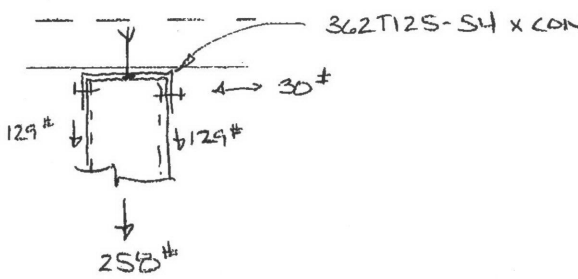
$V_{ASD} = 4^{\#}$ $V_u = 6^{\#}$

TOP CONNECTION AT HANGER w/ KICKER

$T_{ASD} = 258^{\#}$ $T_u = 336^{\#}$

$V_{ASD} = 30^{\#}$ $V_u = 43^{\#}$

PERPENDICULAR TO DECK



$$t_{track\ reqd} = \sqrt{\frac{.129^k \times 1.81''}{\frac{1}{6}(12'')(0.9 \times 50^{ksi})}}$$

$= 0.0509$ ∴ 16 GA. TRACK OK

$V_{screws} = 129^{\#}$

V_a in 30 GA $= 145^{\#} > 129^{\#}$

∴ #8 SMS EA. SIDE OK

$T_{screws} = 15^{\#}$

$T_a = 62.5^{\#}$ ∴ OK

#8 SMS EA. SIDE

ANCHOR TO DECK:

3/8" Ø HILTI KB-TZ, 2" EMBED

$T_{a\,max} = 709^{\#}$ per ESR-1917

$V_{a\,max} = 944^{\#}$

$$\left(\frac{258}{709}\right) + \left(\frac{30}{944}\right) = 0.4 < 1.2$$

(if brittle failure $.4 \times 2.5 = 1.0 < 1.2$) ∴ OK

if use 1/2" Ø SIMPSON Titen HD EMBED 3 1/4"

$T_u = 336^{\#} \times 2.5 = 840^{\#}$

$V_u = 43^{\#} \times 2.5 = 108^{\#}$

∴ unity tension $= 0.93 < 1.0$

∴ OK

(see attached)

Anchor Calculations AT SOFFIT HANGER

Anchor Designer for ACI 318 (Version 4.2.0.2)

Job Name :

Date/Time : 2/9/2010 11:54:56 AM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
1/2" Titen HD	N/A	1	3.25	1

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Sand lightweight over Metal Deck	Yes	3000.0	N/A

Condition	Thickness (in)	Suppl. Edge Reinforcement
B tension and shear	see figure	N/A

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
840	108	0	0	0

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

Individual Anchor Tension Loads

N_{ua1} (lb)
840.00

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

Individual Anchor Shear Loads

V_{ua1} (lb)
108.00

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

Tension Strengths

Steel (Φ = 0.65)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
20130	13084.50	840.00	0.0642

Concrete Breakout does not apply

Pullout (Φ = 0.65 , Φ_{seis} = 0.75)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{pn}$
1851.00	902.36	840.00	0.9309

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.60)

V_{eq}(lb)	ΦV_{eq}(lb)	V_{ua}(lb)	$V_{ua}/\Phi V_{eq}$
4098	2458.80	108.00	0.0439

Concrete Breakout does not apply

Pryout does not apply

Interaction check

V.Max(0.04) <= 0.2 and T.Max(0.93) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 1/2" diameter Titen HD anchor(s) with 3.25 in. embedment

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

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Project PPP ASC	EQUILIBRIUM ENGINEERS	By FENTON	Sheet # C11
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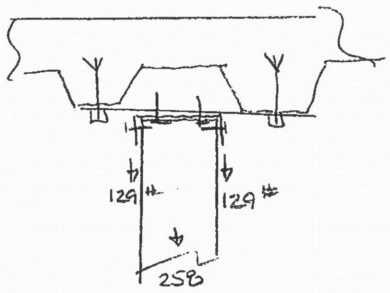
PARALLEL TO DECK



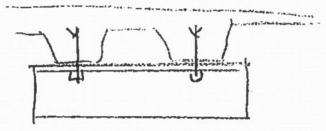
PL thickness $= \sqrt{\dfrac{\frac{258^{\#} \times 12''}{4}}{\frac{1}{6}(12'')(0.9 \times 50^{ksi})}}$

$= 0.093''$ ∴ 12 GA. OK

Per previous calc
(2) 3/8" ⌀ HILTI KB-TZ
EMBED 2" OK

(2) 1/2" ⌀ SIMPSON TITEN HD, EMBED 3 1/4"
OK

KICKER CONNECTION



$T_{ASD} = 100^{\#}$ $T_u = 143^{\#}$

$V_{ASD} = 100^{\#}$ $V_u = 143^{\#}$

(2) 3/8" ⌀ HILTI KB-TZ, EMBED 2"

$T_a = 709$
$V_a = 944^{\#}$

unity $= \dfrac{100 \times 2.5/2}{709} + \dfrac{100 \times 2.5/2}{944}$

$= 0.31 < 1.2$ ∴ OK

(2) 1/2" ⌀ SIMPSON TITEN HD
EMBED 3 1/4"

$T_u = \dfrac{143^{\#}}{2 \text{ anchors}} \times 2.5 = 178^{\#}$

$V_u = \dfrac{143^{\#}}{2} \times 2.5 = 178^{\#}$

unity = .20 ∴ OK

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2) – AT KICKER CONN.

Job Name :

Date/Time : 2/9/2010 12:08:15 PM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
1/2" Titen HD	N/A	1	3.25	1

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi'_{c,V}$
Sand lightweight over Metal Deck	Yes	3000.0	N/A

Condition	Thickness (in)	Suppl. Edge Reinforcement
B tension and shear	see figure	N/A

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
178	178	0	0	0

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

Individual Anchor Tension Loads

N_{ua1} (lb)
178.00

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

Individual Anchor Shear Loads

V_{ua1} (lb)
178.00

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

Tension Strengths

Steel (Φ = 0.65)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
20130	13084.50	178.00	0.0136

Concrete Breakout does not apply

Pullout (Φ = 0.65 , Φ_{seis} = 0.75)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{pn}$
1851.00	902.36	178.00	0.1973

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.60)

V_{sa}(lb)	ΦV_{sa}(lb)	V_{ua}(lb)	$V_{ua}/\Phi V_{sa}$
4098	2458.80	178.00	0.0724

Concrete Breakout does not apply

Pryout does not apply

Interaction check

V.Max(0.07) <= 0.2 and T.Max(0.2) <= 1.0 [Sec D.7.1]

Interaction check: PASS

Use 1/2" diameter Titen HD anchor(s) with 3.25 in. embedment

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

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Project PPP ASC	EQUILIBRIUM ENGINEERS	By FENTON	Sheet # C13
Location PORTLAND OR		Date 7/31/13	
Client		Revised	Job # 12043
		Date	

SUSPENDED CEILING

- LATERAL DESIGN IN CONC.

FOR ALL BRACING COMPONENTS EXCEPT ANCHORS IN CONCRETE

$$0.7F_p \text{ bracing comp.} = \frac{(0.7)(0.4)a_p S_{DS} W_p (1+2z/h)}{R_p/I_p}$$

FOR ANCHORS IN CONCRETE NOT GOVERNED BY THE STRENGTH OF A DUCTILE STEEL ELEMENT OR NOT COMPLYING w/ ACI 355.2

$$0.7F_{p\,anchors} = \frac{(0.7)(1.3)(0.4)a_p S_{DS} W_p (1+2z/h)}{R_p/I_p}$$

$0.7F_{p\,MIN} = 0.7(0.3)S_{DS} I_p W_p$

$a_p = 1.0$

$S_{DS} = 0.77$

$W_p = 4$ psf

$I_p = 1.0$

$R_p = 2.5$ for bracing components except anchors in concrete

$R_p = 1.5$ for anchors in concrete not governed by the strength of ductile steel element or not complying w/ACI 355.2

$z/h = 1.0$

$.7F_{p\,bracing\,comp} = 1.03$ psf ← governs

$.7F_{p\,min} = 0.65$ psf

$.7F_{p\,anchors} = 2.24$ psf ← governs for anchors

SPACING OF STRUTS & SPLAYS IN EA. DIRECTION = 12' MAX

TRIB AREA = 12' x 12' = 144 sf

$0.7F_{p\,brace} = .7F_{p\,brace} \times 144$ sf
$= 1.03 \times 144$ ~~sf~~
$= 148^{\#}$

$0.7F_{p\,anchors} = .7F_{p\,anchors} \times 144$ sf
$= 2.24 \times 144$ ~~sf~~
$= 323^{\#}$ shear & tension in anchor

- COMPRESSION STRUTS ARE PER PRESCRIPTIVE PER ATTACHED STATEWIDE DESIGN METHOD 06-01, PG 9 OF 19

- DIAGONAL WIRE SPLAYS

CAPACITY OF 12 GA. WIRE $= 0.6F_y A (1.33)$
$= .60(55^{ksi})(0.0088 in^2)(1.33)$
$= 386^{\#}$

$$.7F_{p\,SPLAY} = \frac{148^{\#}}{\cos 45} = 209^{\#} < 386^{\#}$$

∴ 12 GA. WIRE OK

2) Based on the maximum spacing of 12 feet on-center in each direction, the following strut sizes and corresponding maximum lengths may be used:

Strut Size	Max. Length
½" diameter conduit (EMT)	5'-10"
¾" diameter conduit (EMT)	7'-8"
1" diameter conduit (EMT)	9'-9"
Single 162S125-33 metal stud (1-5/8" x 20 gauge)	12'-0"
Back to Back 162S125-33 metal studs (1-5/8" x 20 gauge)	15'-0"
Single 250S125-33 metal stud (2-1/2" x 20 gauge)	13'-6"
Back to Back 250S125-33 metal studs (2-1/2" x 20 gauge)	15'-0"

Where utilizing a strut that is not indicated on the table above, the strut shall be capable of resisting the vertical component of 180 pounds induced by the wires.

e. Seismic separation joints required. [See Diagram 7]

1) General. Ceilings with an area greater than 2,500 square feet shall be provided with seismic separation joints. Separation joints shall divide the ceiling area into areas less than 2,500 square feet and shall allow for a minimum of 2 inches of lateral movement of the ceiling providing 1 inch of movement for the portions of ceiling located on either side of the separation joint. Each 2,500 square foot ceiling area shall have its grid attached to a closure angle or other approved device on two adjacent sides as required in section D 1 above.

2) Alternate. In lieu of seismic separation joints, the ceiling may be divided into areas less than 2,500 square feet by the use of partitions or soffits as described below. [See Diagram 6]

a) Partitions. Where used, partitions shall extend a minimum of 6 inches above the level of the plane of the grid and shall be independently braced to the structure above.

b) Soffits. Where used. soffits shall extend to a point at least level with the plane of the grid and shall be

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EQUILIBRIUM ENGINEERS

OR

- 1/2" Ø SIMPSON TITEN HD EMBED 2 3/4"

$T_u = \frac{323^{\#}}{.7} = 461^{\#}$

$V_u = \frac{323^{\#}}{.7} = 461^{\#}$

above loads include factor of 1.3 x 1.5 = 1.95 x applied load
$\approx R_p$

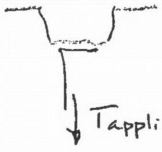
if loads are increased by 1.28 x Loads Above to meet 2.5 x factored load for IBC Requirements for brittle Failure:

$T_{umax} = 461^{\#} \times 1.28 = 590^{\#}$
$V_{umax} = 590^{\#}$

Per attached output
$\text{unity} = 1.11 < 1.2$

∴ 1/2" Ø SIMPSON TITEN HD EMBED 2 3/4" OK

HANGER ATTACHMENT (GRAVITY LOAD ONLY)



$T_{applied} = 4\,PSF \times 16\,\square' = 64^{\#}$

$T_{min\ design} = 100^{\#}$

RAMSET SPC 114 ANGLE CLIP FASTENER

$T_a = 157^{\#} > 100^{\#}$ ∴ OK

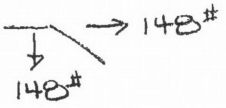
└ per ICC ESR-1799 TABLE 6

0.150" Ø fastener w/ 1 1/8" penetration

OR APPROVED EQUAL

- LATERAL CONNECTION TO ROOF DECK

- SPLAY WIRE



(2) #12-14 HILTI KWIK PRO @ 3/4" O.C. IN 20 GA.

$T_a = 99^{\#}$, $V_a = 200^{\#}$ } per ESR-2196

$\left(\frac{148/2}{99}\right) + \left(\frac{148/2}{200}\right) = 1.11 < 1.20$

∴ OK (2) #12-14

- HANGER

#12-14 HILTI KWIK PRO

$T_a = 99^{\#} \approx 100^{\#}$

∴ OK

Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.2)

SPLAY WIRE FASTENER

Job Name :

Date/Time : 2/9/2010 1:58:28 PM

Calculation Summary - ACI 318 Appendix D For Cracked Concrete

Anchor

Anchor	Steel	# of Anchors	Embedment Depth (in)	Category
1/2" Titen HD	N/A	1	2.75	1

Concrete

Concrete	Cracked	f'_c(psi)	$\Psi_{c,V}$
Sand lightweight over Metal Deck	Yes	3000.0	N/A

Condition	Thickness (in)	Suppl. Edge Reinforcement
B tension and shear	see figure	N/A

Factored Loads

N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	M_{ux} (lb*ft)	M_{uy} (lb*ft)
590	590	0	0	0

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

Individual Anchor Tension Loads

N_{ua1} (lb)
590.00

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

Individual Anchor Shear Loads

V_{ua1} (lb)
590.00

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

Tension Strengths

Steel (Φ = 0.65)

N_{sa}(lb)	ΦN_{sa}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{sa}$
20130	13084.50	590.00	0.0451

Concrete Breakout does not apply

Pullout (Φ = 0.65 , Φ_{seis} = 0.75)

N_{pn}(lb)	ΦN_{pn}(lb)	N_{ua}(lb)	$N_{ua}/\Phi N_{pn}$
1473.00	718.09	590.00	0.8216

Side-Face Blowout does not apply

Shear Strengths

Steel (Φ = 0.60)

V_{oq}(lb)	ΦV_{oq}(lb)	V_{ua}(lb)	$V_{ua}/\Phi V_{oq}$
3433	2059.80	590.00	0.2864

Concrete Breakout does not apply

Pryout does not apply

Interaction check

T.Max(0.82) + V.Max(0.29) = 1.11 <= 1.2 [Sec D.7.3]

Interaction check: PASS

Use 1/2" diameter Titen HD anchor(s) with 2.75 in. embedment

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

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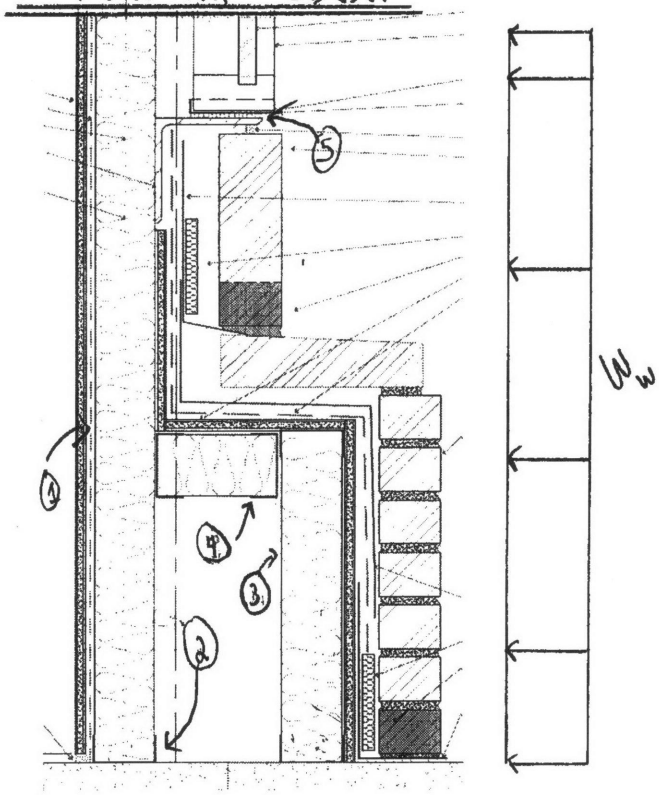
Project PPP ASC	EQUILIBRIUM ENGINEERS	By FENTON	Sheet # C7
Location PORTLAND, OR		Date 7/31/13	
Client		Revised	Job # 12043
		Date	

EXTERIOR WALL DESIGN



MEMBER 1

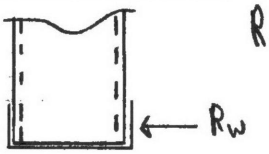
HT: 15' W_W: 25 PSF (CONSERVATIVE)

TRY 362S200-54, 50KSI @ 12" o.c.

H_{ALLOW}: 18'-4" > 15' ∴ OK!

$\boxed{\text{USE 362S200-54, 50 KSI @ 12" o.c.}}$

MEMBER 2



R_W: 25 PSF (15'/2)(1') = 187.5 LBS/STUD

REACTION @ EA. TRACK LEG = 187.5 LB/2 = 93.75 LBS

$$T_{REQ'D} = \left[\frac{6M}{0.6F_y b}\right]^{1/2} = \left[\frac{6(93.75\,LBS)(1")}{0.6(50{,}000\,PSI)(12")}\right]^{1/2}$$

= 0.04" → USE 43 MIL OR GREATER

$\boxed{\text{USE 362T125-54 FOR EXT. WALL TRACK}}$

CONNECTION TO BASE

TRY 3/8" Ø HILTI KB-TZ w/ 2" EMBED

$$V_{ALLOW} = \frac{1380\,LBS}{1.4} = 985.7\,LBS/BOLT$$

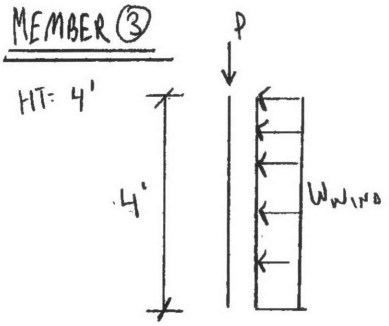
Ω_{ASD}

$V_{APPLIED}$ = 187.5 LBS/FT

$S_{REQ'D}$ = 985 LBS / 187.5 LB/FT = 5.25 FT

$\boxed{\text{USE (1) 3/8" Ø HILTI KB-TZ w/ 2" EMBED @ 24" o.c.}}$

MEMBER 3



W_{WIND}: 25PSF (12" o.c.) = 25PLF

TRY 362S200-54 STUD, 362T125-54 TRACK

P_{ALLOW} = > 4.27 KIPS ∴ OK!

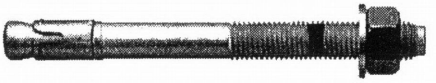
$\boxed{\text{USE 362S200-54 STUD, 362T125-54 TRACK}}$

C18

Company:	Equilibrium Engineers LLC	Page:	1
Specifier:	Justin Fenton	Project:	PPP ASC
Address:	16325 Boones Ferry Rd. Suite 202	Sub-Project I Pos. No.:	#12043
Phone I Fax:	503.636.8388 | 503.636.2090	Date:	8/8/2013
E-Mail:	justin@equilibriumllc.com		

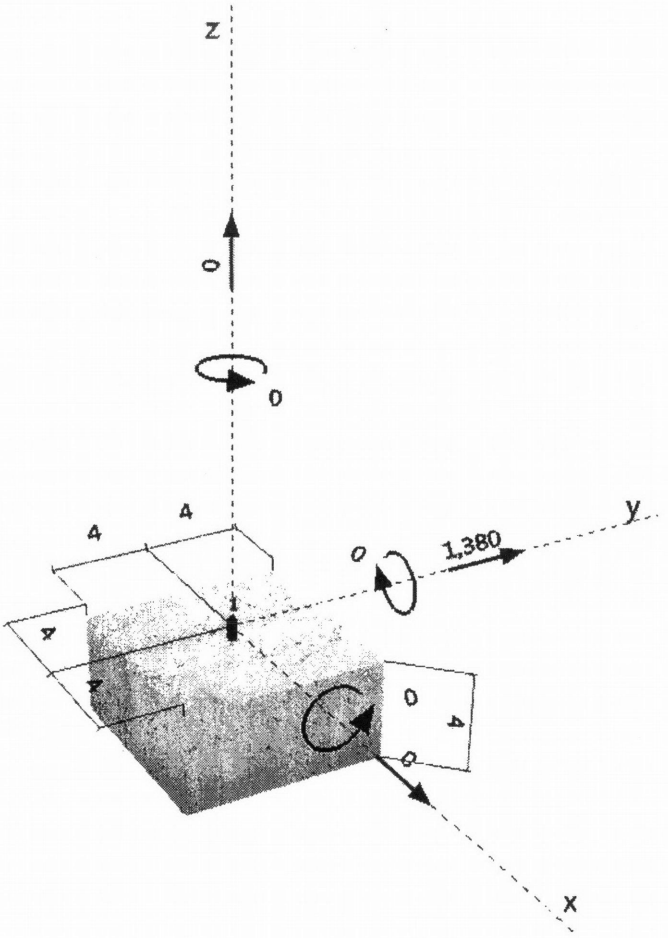
Specifier's comments: 3/8" Dia. Hilti KB-TZ w/ 2" Embed Capacity

1 Input data



Anchor type and diameter:	**Kwik Bolt TZ - CS 3/8 (2)**
Effective embedment depth:	h_{ef} = 2.000 in., h_{nom} = 2.313 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-1917
Issued I Valid:	5/1/2013 | 5/1/2015
Proof:	design method ACI 318 / AC193
Stand-off installation:	- (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 3000, f_c' = 3000 psi; h = 4.000 in.
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: > No. 4 bar with stirrups
Seismic loads (cat. C, D, E, or F)	no

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



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C11

Company:	Equilibrium Engineers LLC	Page:	2
Specifier:	Justin Fenton	Project:	PPP ASC
Address:	16325 Boones Ferry Rd. Suite 202	Sub-Project I Pos. No.:	#12043
Phone I Fax:	503.636.8388 | 503.636.2090	Date:	8/8/2013
E-Mail:	justin@equilibriumllc.com		

2 Proof I Utilization (Governing Cases)

		Design values [lb]		Utilization	
Loading	**Proof**	**Load**	**Capacity**	β_N / β_V [%]	**Status**
Tension	-	-	-	- / -	-
Shear	Concrete edge failure in direction y+	1380	1400	- / 99	OK

Loading	β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
Combined tension and shear loads	-	-	-	-	-

3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

4 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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Project	PPP ASC	By	FENTON	Sheet #	C20
Location	PORTLAND, OR	Date	7/31/13		
Client		Revised		Job #	12043
		Date			

EQUILIBRIUM ENGINEERS

EXTERIOR WALL DESIGN CONTINUED

MEMBER (4)

$\boxed{\text{USE STUD AND TRACKS FROM MEMBER (3)}}$

CONNECTIONS →

V_{ALLOW} #8 SMS $= 332 \text{ LBS/SCREW} \; (2/\text{TRACK}) = 664 \text{ LBS}$

∴ OK!

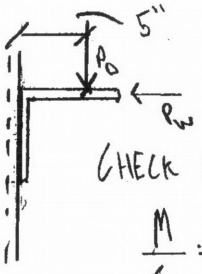
$\boxed{\text{USE (1) \#8 SMS @ EA. STUD, @ EA. TRACK}}$

MEMBER (5)

WINDOW SYSEM WEIGHT = 10 PSF (5') = 50 PLF

WIND LOAD: 25 PSF (1'-1") (8'½) (48") = 4,800 LB-IN

TRY L6x6 ¼



P: 50 PLF

M = 50 PLF (5") = 250 LB-IN/FT

CHECK BENDING IN ANGLE

$$\frac{M}{S} = \frac{250 \text{ LB-IN.}}{12'' \left(\frac{1}{4}''\right)^2/6)(0.66)} = 3{,}030 \text{ PSI}$$

∴ OK!

DESIGN WELD: (ASD)

$$S_W = \frac{d^2}{6} = \frac{6^2}{6} = 6 \text{ IN}^2$$ (WELD LINE @ WEB SIDE)

$$f = M/S_W = \frac{250 \text{ LB-IN}}{6 \text{ IN}^2} = 41.667 \text{ LB/IN.}$$ REQ'D WELD

FROM SSMA → $W_{ALLOW} = 822 \text{ LBS/IN} > 41.67 \text{ LB/IN.}$

∴ OK!

$\boxed{\text{USE } \tfrac{1}{8}'' \text{ FILLET AND L 6x6x}\tfrac{1}{4} \text{ ANGLE}}$

CHECK STUD FOR WINDOW MOMENT

$M_{APPLIED} = 250 \text{ LB-IN} + 4{,}800 \text{ LB-IN} = 5{,}050 \text{ LB-IN.}$

$M_{ALLOW} = 14.66 \text{ K-IN} = 14{,}660 \text{ LB-IN}$ ∴ OK!

CONNECTION AT HEAD (DEFLECTION HEAD)

DESIGN TRACK:

$$P = \frac{250 \text{ LB-IN}}{(8' \times 12''/\text{FT})} + 187.5 \text{ LBS} = 190.1 \text{ LBS}$$

$P_{ALLOW} = 308 \text{ LBS}$ (FROM CLARK DIETRICH)

↳ SEE ATTACHED

$\boxed{\text{USE 16 GAUGE SLOTTED TRACK @ HEAD}}$

C21

ClarkDietrich
BUILDING SYSTEMS

Product Submittal Sheet

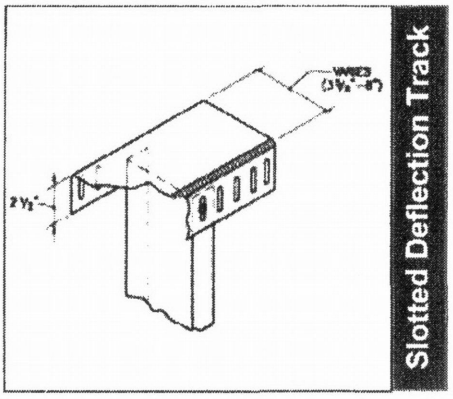
Tech Support: 888-437-3244
Engineering Services: 877-832-3206

Sales: 800-543-7140
clarkdietrich.com

05.40.00 (Cold-Formed Metal Framing)

Brady SLP-TRK® Sliptrack Structural System

Structural slotted deflection track for interior & exterior walls

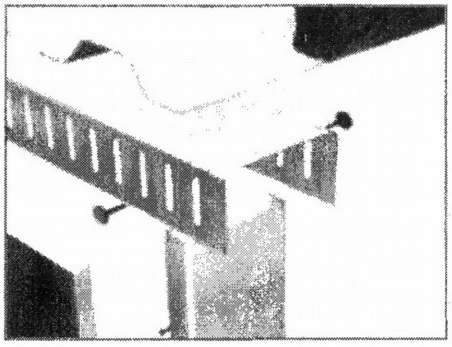


Brady's SLP-TRK® system provides a positive attachment for wall strength and allows for 1" vertical movement. Movement caused by normal head-of-wall and floor extension or compression is absorbed by SLP-TRK® to prevent damage to the wall system.

Waferhead screws positively attach the stud through SLP-TRK's vertical slots. Floor deck movement is absorbed by SLP-TRK® at the center of the construction joint.

Brady's SLP-TRK® smoothly integrates with a variety of wall installation systems and is extremely user-friendly, as well as economical. The simple design and easy installation reduces the cost of materials and labor. And the installer can use the head-of-wall systems he prefers to work with. Sliptrack Systems complies with the toughest code requirements in the country, if not the world. It has been tested with a variety of leading fire-stop products, and has outperformed the requirements of stringent fire and hose stream testing ASTM E-119, ASTM E-814. Sliptrack also has met and exceeded the latest seismic cycling standards found in UL2079 and ASTM1966

- **Allows up to 1" vert. deflection**
- **UL Approved 1 & 2 hour systems**



Product Data & Ordering Information:

Material: All Material: Grade 50ksi yield strength, G60
54mils: 16 Gauge, 0.0566" Design Thickness, 0.0538" Min. Thickness
68mils: 14 Gauge, 0.0713" Design Thickness, 0.0677" Min. Thickness

Dimensions: 2-1/2" legs with an inside depth equal to the depth of the stud
- Available in 2-1/2", 4", 6" and 8" width systems
- Vertical slots are 1/4" wide x 1-1/2" long and spaced every 1"o.c.

ASTM & Code Standards:

- ASTM C955, A653 & A1003.
- MSDS & Product Certification Information is available at www.clarkdietrich.com

Other available Brady SLP-TRK Systems:

Available in 2-1/2", 3-5/8", 4", 6" and 8" width systems in 25ga, 20ga and 18ga G40

Allowable Lateral Loads:

- 54mils (16gauge) SLP-TRK® P = 308 lbs.
- 68mils (14gauge) SLP-TRK® P = 342 lbs.

Calculating slip track point load:
Point Load (P) = (wind pressure PSF) x (spacing FT) x (wall stud length FT) / 2
Example: (5 PSF) x (1.33 FT) x (9.5 FT) / 2 = 31.7 lbs.

For more installation and fire rated assemblies on either of these systems, refer to: www.clarkdietrich.com
SLP-TRK® is a registered trademark of SlipTrack Systems, Inc.

GREEN Benefits and Recycled Content:

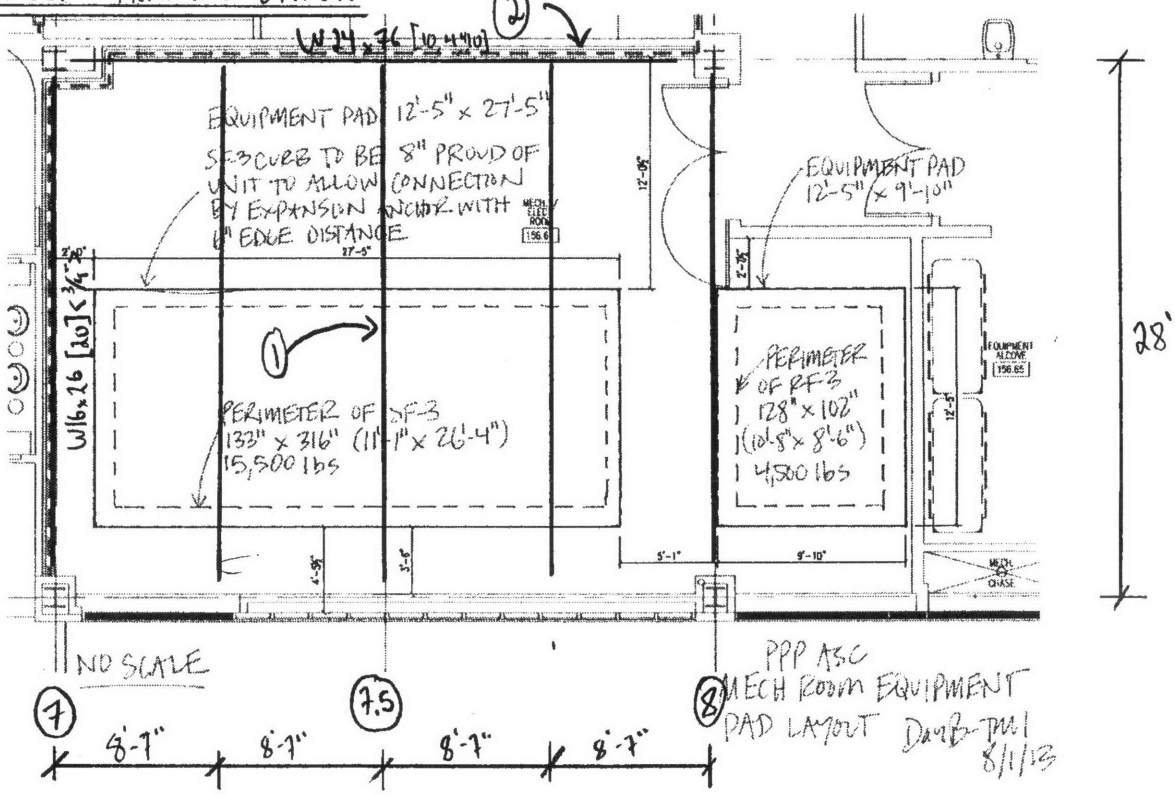
LEED Credit MR 2 - ClarkDietrich products are manufactured from cold-formed steel. Steel is 100% recyclable, which helps divert debris from the waste stream. The contribution to LEED must be calculated by the contractor based on weight or volume.
LEED Credit MR 4 - ClarkDietrich's steel products have a minimum recycled content of 34.9%, of which 24.3% is post-consumer, and 9.4% is pre-consumer. To report a higher number for your project or seek Credit MR 5, contact Technical Services at 888-437-3244 or visit www.clarkdietrich.com.

CD-SLP-TRK-STR © 04/23/13 ClarkDietrich Building Systems

Project Information	Contractor Information	Architect Information
Name:	Name:	Name:
Address:	Contact:	Contact:
	Phone:	Phone:
	Fax:	Fax:

Project PPP ASC	EQUILIBRIUM ENGINEERS	By FENTON	Sheet # C22
Location PORTLAND, OR		Date 7/31/13	
Client		Revised	Job # 12043
		Date	

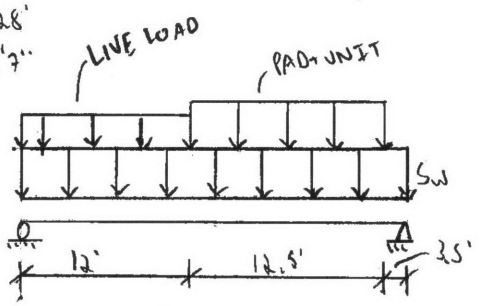
EQUIPMENT ROOM FRAMING CHECK



PPP ASC
MECH ROOM EQUIPMENT PAD LAYOUT
DwB-TM1
8/1/13

MEMBER 1 CHECK

SPAN: 28'
TRIB: 8'7"



SW (PER ENERCALC)

PAD + UNIT: $\frac{15{,}500\text{LBS}}{(27.5)(12.5')} + 4''(12.5\text{PSF}) = 95.1$ PSF

LIVE LOAD: 80 PSF

SEE ATTACHED ENERCALC

EXISTING STRUCTURE IS OK!

MEMBER 2 CHECK

SPAN: 34'-4"

REACTIONS → (M1) → D = 17.352^k, L = 6.625^k } W/ (N) MECH UNIT

REACTIONS (M1) → D = 13.8^k, L = 9.6^k } W/ CURRENT LOADS

MAX LOAD COMBINATIONS:

NEW → 1.2D + 1.6L = 1.2(17.352^k) + 1.6(6.625^k) = 31.422^k

OLD → 1.2D + 1.6L = 1.2(13.8) + 1.6(9.6^k) = 31.92^k

LOADS WENT DOWN, ∴ (E) BEAM IS OK!

SEE ATTACHED ENERCALC

C23

EQUILIBRIUM ENGINEERS
Justin Fenton
Project Engineer
Equilibrium Engineers LLC
16325 Boones Ferry Rd.
Lake Oswego, Oregon 97035
503.636.8388

Project Title:
Engineer:
Project Descr:
Project ID:

Printed: 9 AUG 2013, 4:21PM

Composite Steel Beam

File = C:\Users\JUSTIN~1\DOCUME~1\ENERCA~1\PPPASC~1.EC6
ENERCALC, INC. 1983-2013, Build:6.13.6.21, Ver:6.13.6.21

Lic. # : KW-06008381 **Licensee : EQUILIBRIUM ENGINEERS LLC**

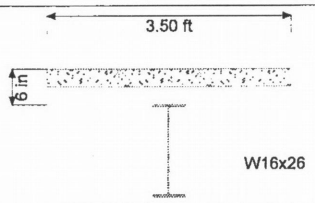
Description : Mech. Room W16 x 26 [20] <3/4"> Typ. Beam Stress Check with Unit + Slab

CODE REFERENCES

Calculations per AISC 360-05, IBC 2006, CBC 2007, ASCE 7-05
Load Combination Set : IBC 2009

Material Properties

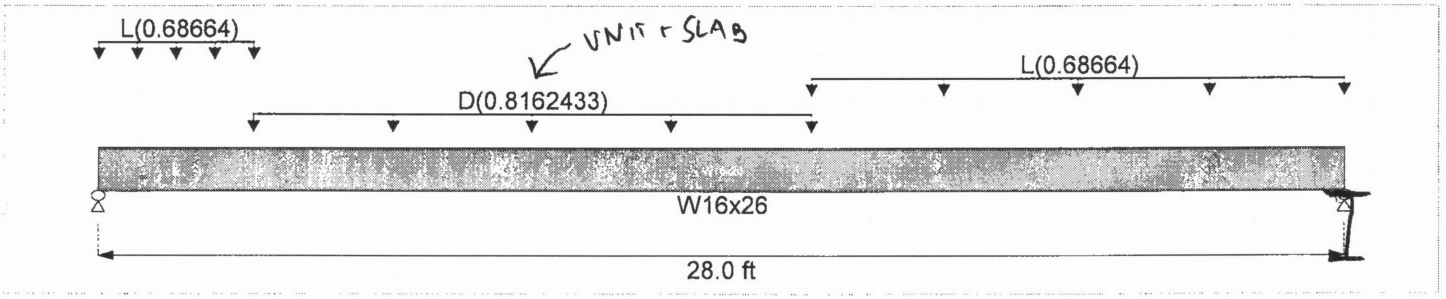
Analysis Method : Load Resistance Factor Design
Beam Bracing : Beam is Fully Braced against lateral-torsion buckling by attached sla
Load Combination IBC 2009
Fy : Steel Yield : 36.0 ksi E: Modulus : 29,000.0 ksi



Composite Beam Section Data

Beam is UNSHORED for Concrete Placement

Total Slab Thickness	6.0 in	Concrete f'c	3.0 ksi	Stud Diameter	3/4" in
Effective Width	3.50 ft	Concrete Density	145.0 pcf	Qn : Stud Capacity	11.0 k
Metal Deck . . .	Verco, PLW3 Formlok	Rib Height	3.0 in	Top Width	7.50 in
Ribs :	Perpendicular	Rib Spacing	12.0 in	Btm Width	4.50 in



Applied Loads

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

Beam + Slab self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.09510 ksf, Extent = 3.50 -->> 16.0 ft, Tributary Width = 8.583 ft, Post Composite Only
Uniform Load : L = 0.080 ksf, Extent = 16.0 -->> 28.0 ft, Tributary Width = 8.583 ft, Post Composite Only
Uniform Load : L = 0.080 ksf, Extent = 0.0 -->> 3.50 ft, Tributary Width = 8.583 ft, Post Composite Only

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.768** : 1
Section used for this span **W16x26**
Percent Composite Action **39** %

	Pre-Composite	Composite
Mu : Applied	67.618	158.627 k-ft
Mn * Phi : Allowable	119.340	206.647 k-ft

Load Combination Post Composite : +1.20D+0.50Lr+1.60L+1.60H
Location of maximum on span 14.187 ft
Span # where maximum occurs Span # 1

Maximum Shear Stress Ratio = **0.273** : 1
Section used for this span **W16x26**
Vu : Applied 23.142 k
Vn * Phi : Allowable 84.780 k
Load Combination span 1
Location of maximum on span 28.0 ft
Span # where maximum occurs Span # 1

Maximum Deflection

Max Downward Pre-Composite	0.789 in	Max Downward Composite Deflection	1.672 in
Max Upward Pre-Composite	0.000 in	Max Upward Composite Deflection	0.000 in
Pre-Composite Deflection Ratio	425	Composite Deflection Ratio	200

Shear Stud Requirements

From Support 1 to 14.19 ft use 10 studs.
From 14.19 ft to Support 2 use 10 studs.

Maximum Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	24.468	23.977
Precomposite Loads	6.900	6.900
Precomposite minus Construction	6.900	6.900
Postcomposite : D + Lr	20.450	17.352
Postcomposite : D + L	24.468	23.977
Postcomposite : D + Lr + L	24.468	23.977

D = 17.352 k
L = 23.977 k - 17.352 k = 6.625 k

EQUILIBRIUM ENGINEERS
Justin Fenton
Project Engineer
Equilibrium Engineers LLC
16325 Boones Ferry Rd.
Lake Oswego, Oregon 97035
503.636.8388

Project Title:
Engineer:
Project Descr:

Project ID:

Printed: 9 AUG 2013, 4:21PM

Composite Steel Beam

File = C:\Users\JUSTIN~1\DOCUME~1\ENERCA~1\PPPASC~1.EC6
ENERCALC, INC. 1983-2013, Build:6.13.6.21, Ver:6.13.6.21

Lic. # : KW-06008381 **Licensee : EQUILIBRIUM ENGINEERS LLC**

Description : Mech. Room W16 x 26 [20] <3/4"> Typ. Beam Stress Check with Unit + Slab

Maximum Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Postcomposite : D + L + S	24.468	23.977
Postcomposite : D + L + W + S/2	24.468	23.977
Postcomposite : D + L + S + W/2	24.468	23.977
Postcomposite : D + L + S + E/1.4	24.468	23.977

C25

EQUILIBRIUM ENGINEERS
Justin Fenton
Project Engineer
Equilibrium Engineers LLC
16325 Boones Ferry Rd.
Lake Oswego, Oregon 97035
503.636.8388

Project Title:
Engineer:
Project Descr:
Project ID:

Printed: 9 AUG 2013, 4:21PM

Composite Steel Beam

File = C:\Users\JUSTIN~1\DOCUME~1\ENERCA~1\PPPASC~1.EC6
ENERCALC, INC. 1983-2013, Build:6.13.6.21, Ver:6.13.6.21

Lic. # : KW-06008381 **Licensee : EQUILIBRIUM ENGINEERS LLC**

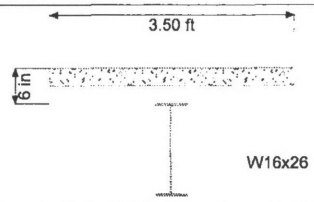
Description : Mech. Room W16 x 26 [20] <3/4"> Typ. Beam Stress Check no Mech Unit

CODE REFERENCES

Calculations per AISC 360-05, IBC 2006, CBC 2007, ASCE 7-05
Load Combination Set : IBC 2009

Material Properties

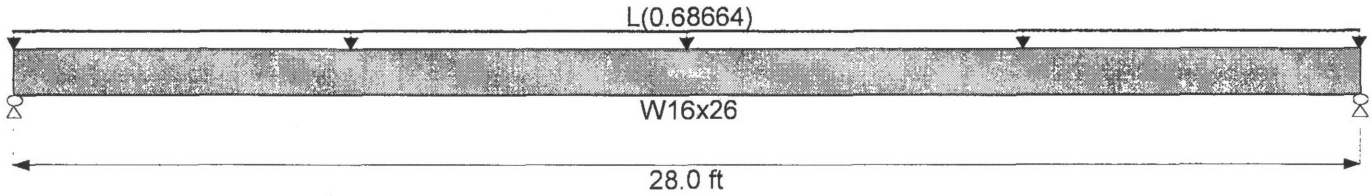
Analysis Method : Load Resistance Factor Design
Beam Bracing : Beam is Fully Braced against lateral-torsion buckling by attached sla
Load Combination IBC 2009
Fy : Steel Yield : 36.0 ksi E: Modulus : 29,000.0 ksi



Composite Beam Section Data

Beam is UNSHORED for Concrete Placement

Total Slab Thickness	6.0 in	Concrete f'c	3.0 ksi	Stud Diameter	3/4" in
Effective Width	3.50 ft	Concrete Density	145.0 pcf	Qn : Stud Capacity	11.0 k
Metal Deck . . .	Verco, PLW3 Formlok	Rib Height	3.0 in	Top Width	7.50 in
Ribs :	Perpendicular	Rib Spacing	12.0 in	Btm Width	4.50 in



Applied Loads

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

Beam + Slab self weight calculated and added to loads
Load for Span Number 1
Uniform Load : L = 0.080 ksf, Tributary Width = 8.583 ft, Post Composite Only

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.801** : 1
Section used for this span **W16x26**
Percent Composite Action **39** %

	Pre-Composite	Composite
Mu : Applied	67.618	165.624 k-ft
Mn * Phi : Allowable	119.340	206.647 k-ft

Load Combination Post Composite : +1.20D+0.50Lr+1.60L+1.60H
Location of maximum on span 14.0 ft
Span # where maximum occurs Span # 1

Maximum Shear Stress Ratio = **0.279** : 1
Section used for this span **W16x26**
Vu : Applied 23.661 k
Vn * Phi : Allowable 84.780 k
Load Combination span 1
Location of maximum on span 0.0 ft
Span # where maximum occurs Span # 1

Maximum Deflection

Max Downward Pre-Composite	0.789 in	Max Downward Composite Deflection	1.619 in
Max Upward Pre-Composite	0.000 in	Max Upward Composite Deflection	0.000 in
Pre-Composite Deflection Ratio	425	Composite Deflection Ratio	207

Shear Stud Requirements

From Support 1 to 14.00 ft use 10 studs.
From 14.00 ft to Support 2 use 10 studs.

Maximum Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	23.413	23.413
Precomposite Loads	6.900	6.900
Precomposite minus Construction	6.900	6.900
Postcomposite : D + Lr	13.800	13.800
Postcomposite : D + L	23.413	23.413
Postcomposite : D + Lr + L	23.413	23.413
Postcomposite : D + L + S	23.413	23.413
Postcomposite : D + L + W + S/2	23.413	23.413
Postcomposite : D + L + S + W/2	23.413	23.413

D = 13.8 k
L = 23.413 - 13.8 = 9.6 k

C16

EQUILIBRIUM ENGINEERS
Justin Fenton
Project Engineer
Equilibrium Engineers LLC
16325 Boones Ferry Rd.
Lake Oswego, Oregon 97035
503.636.8388

Project Title:
Engineer:
Project Descr:

Project ID:

Printed: 9 AUG 2013, 4:21PM

Composite Steel Beam

File = C:\Users\JUSTIN~1\DOCUME~1\ENERCA~1\PPPASC~1.EC6
ENERCALC, INC. 1983-2013, Build:6.13.6.21, Ver:6.13.6.21

Lic. # : KW-06008381 **Licensee : EQUILIBRIUM ENGINEERS LLC**

Description : Mech. Room W16 x 26 [20] <3/4"> Typ. Beam Stress Check no Mech Unit

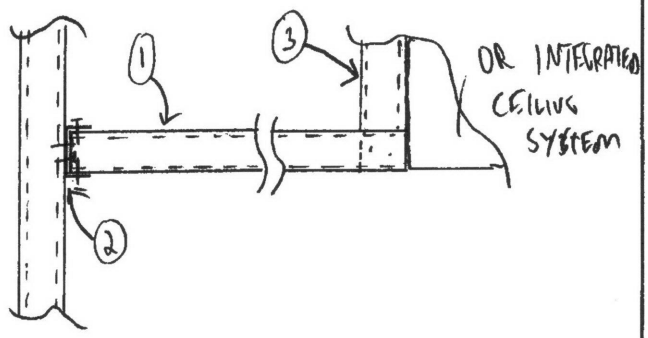
Maximum Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Postcomposite : D + L + S + E/1.4	23.413	23.413

Project PPP ASC	EQUILIBRIUM ENGINEERS	By FENTON	Sheet # C27
Location PORTLAND, OR		Date 8/5/13	
Client		Revised	Job # 12043
		Date	

OPERATING ROOM HARD LID DESIGN



MEMBER 1

TYPICAL STUD SIZE → 362S125-30 33KSI

FOR STUDS @ 24" o.c. → L/240 → MAX SPAN: 13'-6"

$\boxed{\text{SPANS UP TO 13' → USE 362S125-30}}$

TRY 600S125-30 33KSI

FOR STUDS @ 24" o.c. → L/240 → MAX SPAN: 18'-11"

$\boxed{\text{SPANS UP TO 18.5' → USE 600S125-30}}$

CHECK CONNECTION @ WALL MEMBER 2

FOR SPANS UP TO 13':

$V_{WALL} = 4\,PSF(2'\,o.c.)(13'/2) = 52\ LBS$

WALL TRACK THICKNESS REQ'D:

$$T_{REQ'D} = \left[\frac{6M}{0.6F_y b}\right]^{1/2} = \left[\frac{6(52\,LBS/2)(1")}{0.6(33{,}000\,PSI)(12")}\right]^{1/2} = 0.0256"$$

$\boxed{\text{USE 362T125-30 TRACK @ WALL UP TO 13'}}$

CHECK CONNECTION

$V_{REQ'D} = 52\ LBS$ → TRY #8 SMS

$V_{ALLOW\ \#8\ SMS} = 167\ LBS/SCREW$ ∴ OK!

$\boxed{\text{USE (2) \#8 SMS INTO WALL, (1) INTO EACH FLANGE AT TRACK TO JOIST}}$

FOR SPANS UP TO 18.5':

$V_{WALL} = 4\,PSF(2'\,o.c.)(18.5'/2) = 74\ LBS$

WALL THICKNESS REQ'D:

$$T_{REQ'D} = \left[\frac{6(74\,LBS/2)\,1"}{0.6(33{,}000\,PSI)(12")}\right]^{1/2} = 0.03\ INCHES$$

30 MILS = 0.03 INCHES ∴ OK

$\boxed{\text{USE 600T125-30 33KSI TRACK @ WALL UP TO 18.5'}}$

MEMBER 3

$\boxed{\text{USE TYPICAL WALL STUD CONNECTION PER SOFFIT CALCS (PREVIOUS PAGES)}}$

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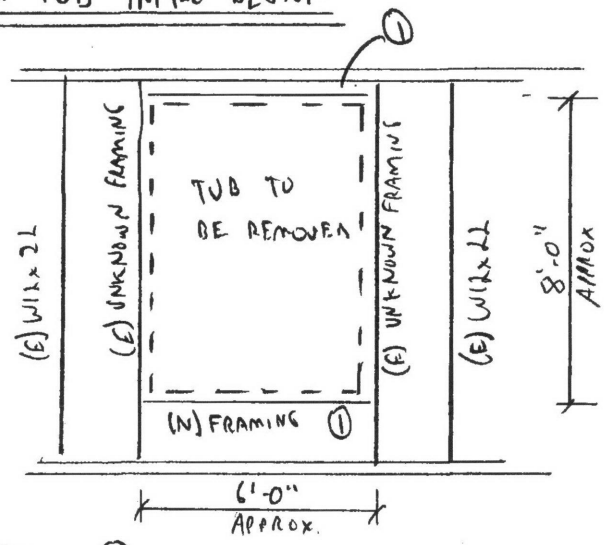
HOT TUB INFILL DESIGN



MEMBER 1

SPAN = 6'

TRIB = 4'

DL = 5½" CONC. → 50.6 PSF + SW BEAM

LL = 100 PSF

SEE ATTACHED ENERCALC

USE W8x10 BEAM MEMBER 1

REACTIONS: D = 640 LBS
L = 1,200 LBS

USE TYPICAL BOLTED BEAM TO BEAM CONN.

CHECK DECK TYPE:

TRY 3", 20 GA, W TYPE DECK W/ 5½" TOPPING

ALLOWABLE SUPERIMPOSED LOAD → 313 PSF ∴ OK!

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Project Title:
Engineer:
Project Descr:
Project ID:

Printed: 7 AUG 2013, 10:18AM

Steel Beam

File = C:\Users\JUSTIN~1\DOCUME~1\ENERCA~1\PPPASC~1.EC6
ENERCALC, INC. 1983-2013, Build:6.13.6.21, Ver:6.13.6.21

Lic. # : KW-06008381 **Licensee : EQUILIBRIUM ENGINEERS LLC**

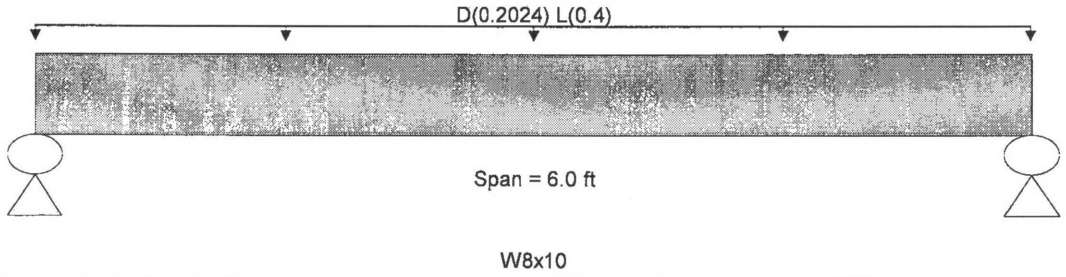
Description : Hot Tub Framing Member 1 W8x10

CODE REFERENCES

Calculations per AISC 360-05, IBC 2006, CBC 2007, ASCE 7-05
Load Combination Set : IBC 2009

Material Properties

Analysis Method :	Load Resistance Factor Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		
Load Combination	IBC 2009		



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.05060, L = 0.10 ksf, Tributary Width = 4.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.123** : 1	Maximum Shear Stress Ratio =	**0.067** : 1
Section used for this span	**W8x10**	Section used for this span	**W8x10**
Mu : Applied	4.027 k-ft	Vu : Applied	2.685 k
Mn * Phi : Allowable	32.871 k-ft	Vn * Phi : Allowable	40.239 k
Load Combination	+1.20D+0.50Lr+1.60L+1.60H	Load Combination	+1.20D+0.50Lr+1.60L+1.60H
Location of maximum on span	3.000ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.013 in	Ratio =	5,469
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.020 in	Ratio =	3572
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.0202	3.030		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.837	1.837
D Only	0.637	0.637
L Only	1.200	1.200
D+L	1.837	1.837

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LED SURGICAL LIGHT ANCHORAGE

LOADS (PROVIDED BY MFR):

DUAL LIGHT SYSTEM → 201 LBS, 567 FT-LBS } SERVICE LEVEL ASSUMED

SEISMIC LATERAL:

$F_{PH} = 0.5624\, w_p \left(\frac{a_p}{R_p}\right) = 0.5624\,(201\text{ LBS})\left(\frac{1.0}{1.5}\right)$

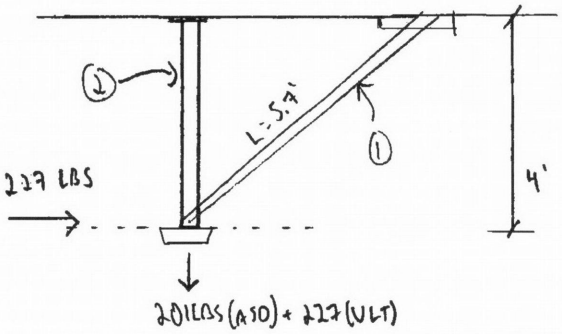
$= 75.36$ LBS (ULT)

LOAD COMBOS:

CONTROLS

$1.6L = \frac{1.6(567\text{ FT-LBS})}{4'} = 226.8$ LBS

$1.0E + 1.0L = 75.36\text{ LBS} + \frac{567\text{ FT-LBS}}{4'} = 217.2$ LBS



KICKER DESIGN MEMBER ①

$L = 5.7' \rightarrow$ SAY 6'

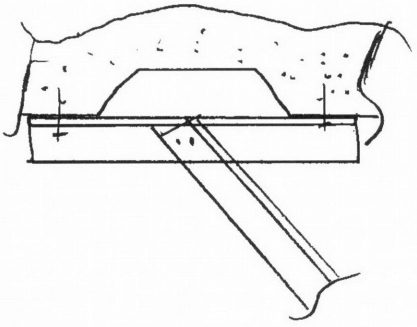
$P = 227\text{ LBS} / \sin 45 = 321$ LBS

SEE ATTACHED ENERCALC

USE L3x3x3/16 KICKER ANGLE

CONNECTION TO DECK

TRY L3x3x3/16 x 1'-4" LONG:



ANCHORS:

FROM SOFFIT KICKER DESIGN

$T_a = 709$ LBS $V_A = 944$ LBS

UNITY CHECK:

$\frac{321\text{ LBS}/2}{709\text{ LBS}} + \frac{321\text{ LBS}/2}{944\text{ LBS}} = 0.4 < 1.0 \therefore$ OK!

USE (2) 3/8" Ø HILTI KB-TZ w/ 2" EMBED

CONNECTION TO ANGLE

TRY (2) #8 SMS → $V_{ALLOW} = 725$ LBS/SCREW ∴ OK

OR

3/16" FILLET WELD ∴ OK BY INSPECTION

USE #8 SMS OR 3/16" FILLET WELD, TYP.

OR

USE (1) 5/8" Ø BOLT @ CL

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Project Title:
Engineer:
Project Descr:

Project ID: C31

Printed: 9 AUG 2013, 10:01AM

Steel Column

File = C:\Users\JUSTIN~1\DOCUME~1\ENERCA~1\PPPASC~1.EC6
ENERCALC, INC. 1983-2013, Build:6.13.6.21, Ver:6.13.6.21

Lic. # : KW-06008381 **Licensee : EQUILIBRIUM ENGINEERS LLC**

Description : vLED Kicker Angle

Code References

Calculations per AISC 360-05, IBC 2006, CBC 2007, ASCE 7-05
Load Combinations Used : IBC 2009

General Information

Steel Section Name : **L3x3x3/16**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 36.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi
Load Combination : IBC 2009

Overall Column Height 6.0 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 6.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for Y-Y Axis buckling = 6.0 ft, K = 1.0

Applied Loads

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

Column self weight included : 22.260 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 6.0 ft, L = 0.2010 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.02431** : 1
Load Combination +1.20D+0.50Lr+1.60L+1.60H
Location of max.above base 0.0 ft
At maximum location values are . . .
Pu 0.3483 k
0.9 * Pn 14.329 k
Mu-x 0.0 k-ft
0.9 * Mn-x : 1.160 k-ft
Mu-y 0.0 k-ft
0.9 * Mn-y : 1.160 k-ft

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

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

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

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LED SURGICAL LIGHT DESIGN CONT.

EQUIPMENT HANGER MEMBER ②

TRY HSS TUBE W/ PLATE AT TOP AND BOTTOM

SIZE PLATE:

$$\frac{567\ \text{FT-LBS}}{(11''/12)} = 619\ \text{LBS}$$

$$M = 619\ \text{LBS}(4'') = 2476\ \text{LB-IN.}$$

$$T_{REQ'D} = \left[\frac{6M}{0.6bF_y}\right]^{1/2} = \left[\frac{6(2476\ \text{LB-IN.})}{0.9(4'')(36{,}000\ \text{LB/IN}^2)}\right]^{1/2}$$

$$= 0.34'' \rightarrow \text{USE } 3/8''\ \text{PLATE}$$

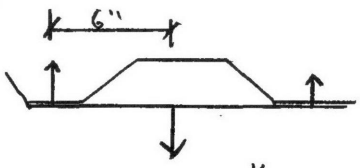
$\boxed{\text{USE } 3/8''\ \text{PLATE AT BOTTOM}}$

$\boxed{\text{USE HSS } 9\times4\times3/4 \text{ FOR DECK TO PLATE}}$

DESIGN TOP PLATE

TOTAL DOWNWARD LOAD:

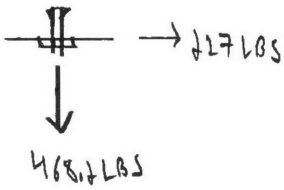
$$\underbrace{201\text{LBS}(1.2)}_{\text{DEAD}} + \underbrace{227\text{LBS}}_{\text{KICKER}} = 468.2\ \text{LBS}$$



$$T_{REQ'D} = \left[\frac{6(468.2\ \text{LBS})(6'')}{0.9(16'')(36{,}000\ \text{LB/IN}^2)}\right]^{1/2} = 0.18'' \rightarrow \boxed{\text{USE } 3/8''\ \text{PL}}$$

CHECK ANCHORS

WORST CASE, 1 ANCHOR FOR ALL TENSION, 4 ANCHORS FOR SHEAR



FOR: 3/8" Ø HILTI KB-TZ W/ 2" EMBED:

$T_{ALLOW} = 709$ LBS $\quad V_A = 944$ LBS

CHECK UNITY:

$$\frac{468.2\ \text{LBS}}{709\ \text{LBS}} + \frac{227\ \text{LBS}}{944\ \text{LBS}} = 0.9 < 1.0$$

∴ OK!

$\boxed{\text{USE (4) } 3/8''\ \text{Ø HILTI KB-TZ W/ 2" EMBED}}$

CONNECTION OF KICKER TO HSS

USE 3/8" THICK SQ. PLATE WELDED TO (2) SIDES AT BOTTOM OF HSS, BOLT THROUGH WITH 5/8" Ø BOLTS