at&t

velocitel
complete wireless solutions

STRUCTURAL ANALYSIS REPORT – Revision #1

PT16 – Lloyd Center
FA #: 10094222 at
915 NE Schuyler St.
Portland, OR 97212

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Velocitel Job #: 101AA10094222L1 ♦ August 21, 2013

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CONTENTS

1.0 SUBJECT & SCOPE

In accordance with AT&T's request, Velocitel performed a structural analysis to qualify the existing building structure for the addition of AT&T appurtenances and equipment at 915 NE Schuyler St., Portland, OR 97212

Velocitel was provided and/or procured the following existing drawings and reports for the previous AT&T installation:

- As-built construction drawings prepared by TRK Engineering, dated July 24, 2003.
- Structural analysis report prepared by TRK Engineering, dated November 21, 2003.
- Construction drawings prepared by Cornerstone Engineering, Inc., dated September 22, 2011.
- Structural analysis report prepared by Cornerstone Engineering, Inc., dated April 25, 2011.
- Structural analysis report prepared by Lund Wright Ospahl Structural Engineers, dated November 26, 2012.

The structural analysis is based on the above documents.

The overall structure is a 9-story residential building with a rooftop penthouse in which the structural system is comprised of cast-in-place concrete slabs supported by concrete beams and CMU walls. The existing equipment is supported inside an equipment room, supported on an existing concrete floor slab.

a)- Equipment Cabinets

Based on information provided by AT&T, the proposed additional equipment has the following specifications:

Cabinet	Height	Width	Depth	Weight
(2) 5 Shelf Marathon Battery Racks	84"	27"	24"	3100 lbs

Existing AT&T cabinets in the equipment room are as follows:

- (1) Indoor UMTS Cabinet (not used in analysis)
- (3) Nokia Indoor GSM Cabinet **(2 to be removed)**, weighing approximately 600 lbs each
- (1) LTE Rack, weighing 500 lbs (maximum)
- (1) Argus Power Rack (not used in analysis)
- (1) Battery String **(1 to be removed)**
- (1) FIF Rack (not used in analysis)
- (1) Telco Rack (not used in analysis)

b)- Antennas

Existing AT&T Appurtenances

Antenna	Mount	Coax and/or Fiber
(2) Kathrein 800-1022 (4) Kathrein 742-265 (1) P65-17-XLH-RR (2) KMW AM-X-CD-16-65 + (6) Nokia MHA CS72993.08 (6) LGP 21401 TMAs + (3) AWS RRH (3) 700 MHz RRH **(3 to be removed)** + (3) DC2 Surge Suppressors	(9) Pipe mounts attached to penthouse wall	(12) 7/8" + (1) Fiber Cable + (2) DC Cables

Proposed AT&T Appurtenances

Antenna	Mount	Coax and/or Fiber
(3) 1900 MHz RRH	Match Existing	n/a

Final Configuration of AT&T Appurtenances

Antenna	Mount	Coax and/or Fiber
(2) Kathrein 800-1022 (4) Kathrein 742-265 (1) P65-17-XLH-RR (2) KMW AM-X-CD-16-65 + (6) Nokia MHA CS72993.08 (6) LGP 21401 TMAs + (3) AWS RRH (3) 1900 MHz RRH + (3) DC2 Surge Suppressors	(9) Pipe mounts attached to penthouse wall	(12) 7/8" + (1) Fiber Cable + (2) DC Cables

2.0 CODES AND LOADING

The existing roof structure was analyzed according to:

- 2010 Oregon Structural Specialty Code
  - Design 3-second wind gust speed for Multnomah County, OR is 95 mph.
  - Exposure Category C.
  - Occupancy Category II; I = 1.0.

- Seismic Design category D.
- ASCE 7-05, Minimum Design Loads for Building and Other Structures.
- ACI 318-08, Building Code Requirements for Structural Concrete
- AISC Steel Construction Manual, 13th ed.

The following load combinations were used to analyze the existing structure and design the equipment anchorage. The numbers in parenthesis denote ASCE 7-05 equation numbers.

$1.2\ D + 1.6\ L$ (2.3.2.2)
$1.2\ D + 1.6\ W$ (2.3.2.4)
$(1.2\ D + 0.2\ S_{DS})\ D + E + L$ (12.14.3.1.3.5)
$(0.9 - 0.2S_{DS})D + E$ (12.14.3.1.3.7)

3.0 ASSUMPTIONS, ANALYSIS CRITERIA and METHOD

The following design values are assumed based on the existing drawings and based on typical wood framed building construction:

- The existing concrete compression strength, f'c, is assumed to be 3000 psi.
- All existing reinforcing steel is assumed to conform with ASTM A615 Grade 40, with a yield strength of 40 ksi.
- The existing floor dead load is assumed to be 5 psf, to account for miscellaneous loads.
- All existing steel pipe members are assumed to be ASTM A53, with a yield strength of 35 ksi.
- All other existing steel members are assumed to be ASTM A36, with a yield strength of 36 ksi.
- Existing non-load bearing wall is assumed to weigh 5 psf (metal stud wall construction).
- Existing machine room (above equipment room) live load is assumed to be 150 psf.

See attached pages from original building drawings in Appendix B for existing slab and beam construction information.

The existing structure is considered to have adequate strength for the proposed loading, if the existing structural members of the structure which will be used to support the proposed equipment are structurally adequate per the current code criteria or the additions and alterations to the existing structure do not increase the load in any structural element by more than 5% of its capacity.

4.0 CALCULATIONS AND OUTPUT

Calculations and Software output for this analysis are provided in Appendix A of this report.

5.0 CONCLUSION

The existing concrete floor structure is found **to have adequate** capacity for the additional equipment loads. At the worst case, the slab is stressed to 71.5% of its capacity.

The proposed Marathon 5 Shelf Battery Rack should be attached to the existing floor structure with ½" diameter Hilti KWIK TZ expansion bolts with 3 1/4" embedment, which is **adequate** for the code required seismic loads on the equipment. Please reference the Velocitel construction drawings for details of this proposed attachment.

The existing antenna mounts and connections are found **to have adequate** capacity for the additional appurtenance loads, and at the worst case are stressed to 5% of their respective capacities.

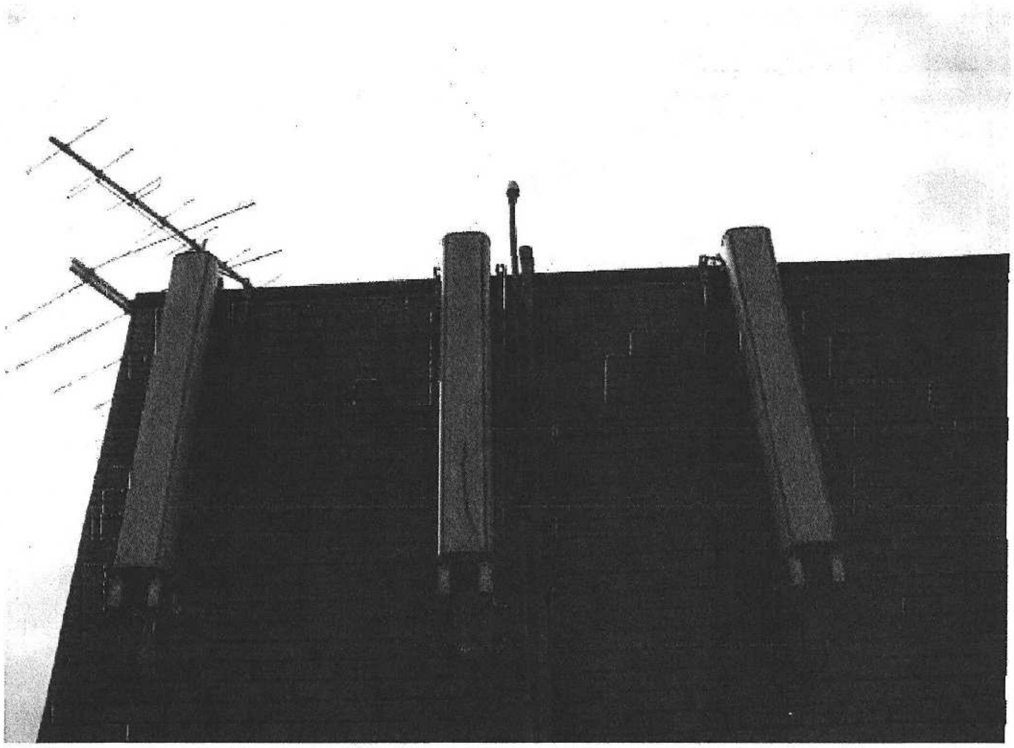
Therefore, the proposed additions and alterations can be implemented as intended, with the conditions outlined in this report.

The conclusions reached by Velocitel, Inc. in this report are only applicable for the previously mentioned existing structural members supporting the proposed telecommunication equipment cabinets and antennas. Any deviation of the support, load and placement, etc., will require Velocitel to generate an additional structural analysis. Further, no structural qualification is made or implied by this report for existing structural members not supporting the aforementioned telecommunication equipment. Velocitel will accept no liability due to discrepancies between the as built drawing(s) and the as built condition of the structure. Contractor should inspect the condition of the existing structure, mounts and connections and notify Velocitel of any discrepancies and deficiencies.

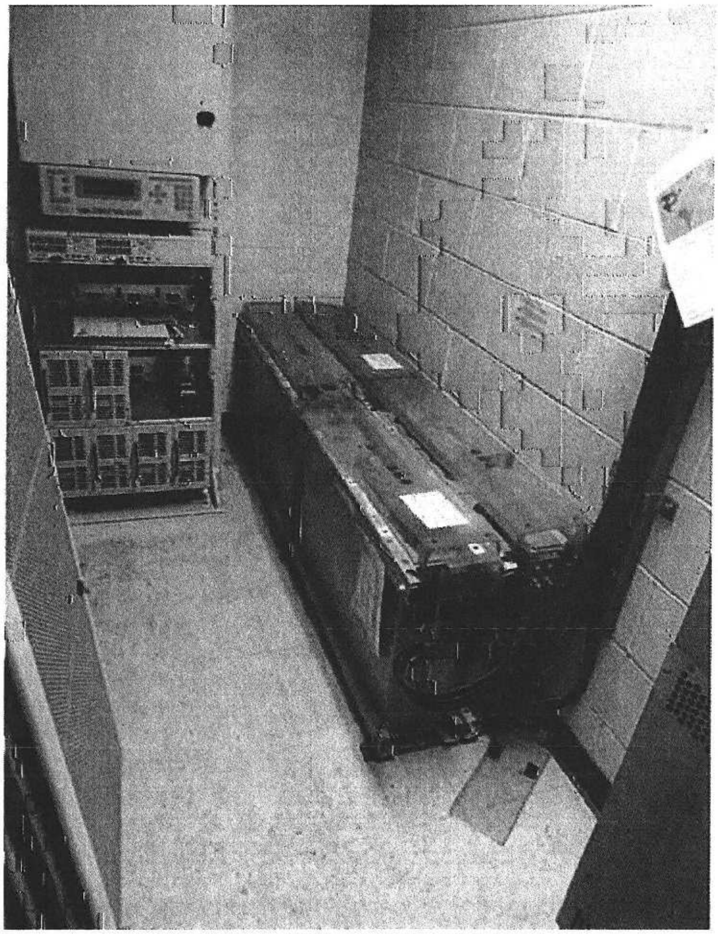
If you have any questions or concerns regarding the contents or results of the report, please feel free to contact me at 770-645-5900 x 104.

Very truly yours,
Velocitel Inc.

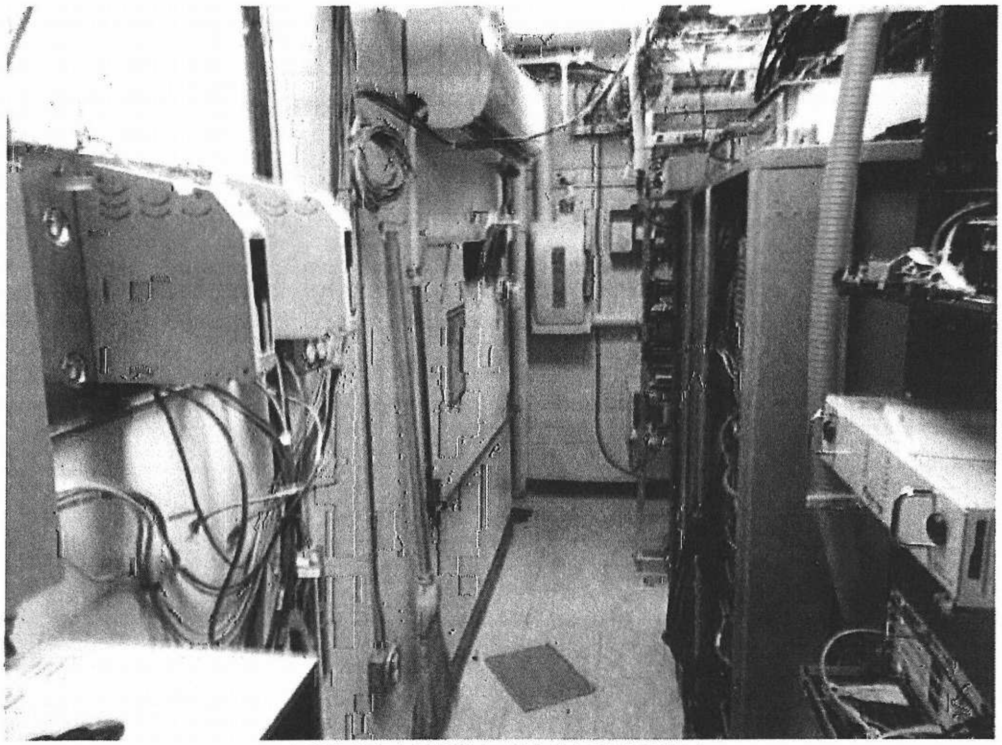
Neal Turbow, P.E.
OR Professional Engineer
License #86331-PE



EXISTING ANTENNAS



EXISTING EQUIPMENT



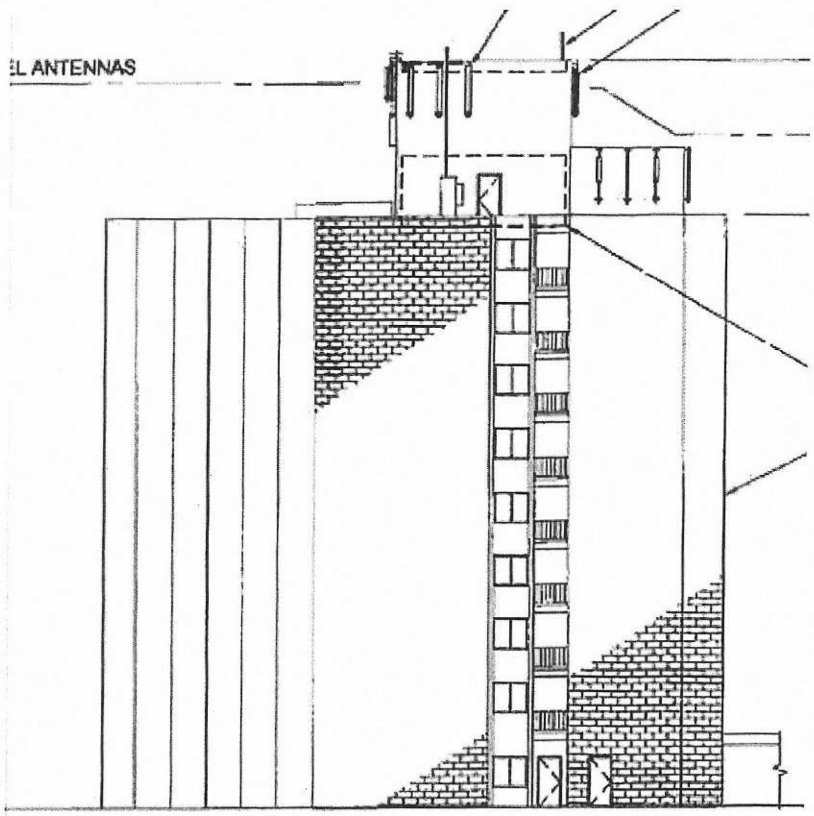
EXISTING EQUIPMENT

A)- PURPOSE

The purpose of these calculations is to qualify the existing floor structure supporting the proposed AT&T equipment installation in the equipment room located at:

915 NE Schuyler St., Portland, OR 97212

Reference : Structural Analysis Report by Cornerstone Engineering, Inc., dated 4/25/11
Structural Analysis Report by TRK Engineering, dated 11/21/03
Structural Analysis Report by Lund Wright Opsahl, dated 11/26/12
Construction Drawings by Cornerstone Engineering, Inc., dated 9/22/11
2010 Oregon Structural Specialty Code, referencing ASCE 7-05



B)- LOADS

ASCE-7-05 References

1)- Dead Loads

For equipment:

Existing equipment:

(3) Nokia Indoor GSM Cabinets **(2 to be removed)**, weighing 600 lbs each
(1) Argus Power Cabinet
(1) UMTS Cabinet
(1) Fiber Rack
(1) LTE Rack, weighing 500 lbs
(1) Equipment Rack
(2) Battery Strings **(2 to be removed)**

Proposed equipment:

(2) BBU Rack w/ 20 marathon batteries each, weighing 3100 lbs (each)

For Appurtenances:

Existing Appurtenances:

(2) Kathrein 80010122
(4) Kathrein 742-265
(1) P65-17-XLH-RR, weighing 70 lbs
(2) KMWAM-X-CD-16-65
(6) Nokia MHA CS72993.08
(6) LGP24101 TMAs
(3) 2100 MHz RRH (3 to be removed)
(3) 700 MHz RRH, weighing 44 lbs

Proposed Appurtenances:

(3) 1900 MHz RRH, weighing 46 lbs

2)- Wind Load

Wind loads are calculated in accordance with TIA-222-G and ASCE-7 requirements. TIA-222-G was designed using the ASCE-7 wind loading and the following wind loads are fully compliant with TIA-222-G.

			Reference, ASCE-7-05
Classification:	II		table 1.1 pg 3
Velocity pressure exposure coefficient:	$K_z := 1.27$	$Exp := C$	table 6-3 pg 79
Topographic factor:	$K_{zt} := 1.0$		sect 6.5.7.2 pg. 26

Wind directionality factor:	$K_d := 0.85$	table 6-4
Basic wind speed:	$V := 95$ mph	Oregon State Code
Importance factor:	$I := 1.0$	table 6-1 pg 77
Antenna height: (RAD center):	$h := 102$ ft > 60 ft	
Gust response factor:	$G := 0.85$	section 6.5.8 pg 26

Velocity pressure: $q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I \cdot psf$ $q_z = 24.94 \cdot psf$ (Eq6 − 15)

For Existing AT&T Antenna (P65-17-XLH-RR)

$H_1 := 96 \cdot in$

$W_1 := 12 \cdot in$

$D_1 := 6 \cdot in$

$\frac{H_1}{W_1} = 8$ $C_f := 1.43$

$\frac{H_1}{D_1} = 16$ $C_{fside} := 1.55$

$F_{ant1} := q_z \cdot G \cdot C_f \cdot H_1 \cdot W_1$

$F_{ant1} = 243\,lbf$

$F_{ant1side} := q_z \cdot G \cdot C_{fside} \cdot H_1 \cdot D_1$

$F_{ant1side} = 131\,lbf$

For Existing RRH 700 MHz:

$H_{700} := 28.8 \cdot in$

$W_{700} := 12.6 \cdot in$

$D_{700} := 5.7 \cdot in$

$\frac{H_{700}}{W_{700}} = 2.286$ $C_f := 1.32$

$\frac{H_{700}}{D_{700}} = 5.053$ $C_{fside} := 1.37$

$F_{700} := q_z \cdot G \cdot C_f \cdot H_{700} \cdot W_{700}$

$F_{700} = 71\,lbf$

$F_{700side} := q_z \cdot G \cdot C_{fside} \cdot H_{700} \cdot D_{700}$

$F_{700side} = 33.11\,lbf$

For Proposed 1900 Mhz RRH:

$H_{1900} := 20.1 \cdot in$

$W_{1900} := 11.2 \cdot in$

$D_{1900} := 7.9 \cdot in$

$\frac{H_{1900}}{W_{1900}} = 1.795 \quad C_f := 1.31$

$\frac{H_{1900}}{D_{1900}} = 2.544 \quad C_{fside} := 1.33$

$F_{1900} := q_z \cdot G \cdot C_f \cdot H_{1900} \cdot W_{1900}$

$F_{1900} = 43\,lbf$

$F_{1900side} := q_z \cdot G \cdot C_{fside} \cdot H_{1900} \cdot D_{1900}$

$F_{1900side} = 31.091\,lbf$

For 2" STD Pipe:

$H_{pipe} := 10 \cdot ft$

$D_{pipe} := 2.375 \cdot in$

$\frac{D_{pipe}}{ft} \sqrt{\frac{q_z}{psf}} = 0.988$ < 2.5

$\frac{H_{pipe}}{D_{pipe}} = 50.526 \quad C_f := 1.2$

$F_{pipe} := q_z \cdot G \cdot C_f \cdot D_{pipe} = 5.035 \cdot plf$

3)- Seismic Load

$S_s := .977$

$S_1 := .340$

$F_a := 1.109$ table 11.4-1

Assumed Site Class D

$F_v := 1.721$ table 11.4-2

$S_{DS} := \frac{2 \cdot F_a \cdot S_s}{3}$ Equation 11.4-1 and 3

$S_{D1} := \frac{2 \cdot F_v \cdot S_1}{3}$ Equation 11.4-2 and 4

$S_{DS} = 0.722$ Seismic Design Category D

$S_{D1} = 0.39$

$I_E := 1$

Seismic loads on cabinets:

$a_p := 1$ Table 13.6-1

Force on Battery Rack

$W_{pBBU} := 3100\,lbf$

$R_p := 2.5$ Table 13.6-1

$z := 1$ ft

$h := 1$ ft

$$F_{pBBU} := \frac{.4 \cdot a_p \cdot S_{DS} \cdot W_{pBBU}}{\left(\frac{R_p}{I_E}\right)} \cdot \left[1 + 2 \cdot \left(\frac{z}{h}\right)\right]$$ Equation 13.3-1

$F_{pBBU} = 1074.825\,lbf$ Seismic load at center of Battery Rack

$F_{pmin} := .3 \cdot S_{DS} \cdot W_{pBBU} \cdot I_E$ Equation 13.3-3

$F_{pmin} = 671.766 \cdot lbf$

Force on 700 MHz RRH

$W_{pRRH700} := 44\,lbf$

$R_p := 2.5$ Table 13.6-1

$z := 1$ ft

$h := 1$ ft

$$F_{pRRH700} := \frac{.4 \cdot a_p \cdot S_{DS} \cdot W_{pRRH700}}{\left(\frac{R_p}{I_E}\right)} \cdot \left[1 + 2 \cdot \left(\frac{z}{h}\right)\right]$$ Equation 13.3-1

$F_{pRRH700} = 15.256\,lbf$ Seismic load at center of 700 MHz RRH

$F_{pmin} := .3 \cdot S_{DS} \cdot W_{pRRH700} \cdot I_E$ Equation 13.3-3

$F_{pmin} = 9.535 \cdot lbf$

Force on 1900 MHz RRH

$W_{pRRH1900} := 46lbf$

$R_p := 2.5$ Table 13.6-1

$z := 1 \quad ft$

$h := 1 \quad ft$

$$F_{pRRH1900} := \frac{.4 \cdot a_p \cdot S_{DS} \cdot W_{pRRH1900}}{\left(\frac{R_p}{I_E}\right)} \cdot \left[1 + 2 \cdot \left(\frac{z}{h}\right)\right]$$ Equation 13.3-1

$F_{pRRH1900} = 15.949lbf$ Seismic load at center of Proposed 1900 MHz RRH

$F_{pmin} := .3 \cdot S_{DS} \cdot W_{pRRH1900} \cdot I_E$ Equation 13.3-3

$F_{pmin} = 9.968 \cdot lbf$

Force on Antenna Pipe Mount

$W_{pMount} := 3.66plf$

$R_p := 2.5$ Table 13.6-1

$z := 1 \quad ft$

$h := 1 \quad ft$

$$F_{pMount} := \frac{.4 \cdot a_p \cdot S_{DS} \cdot W_{pMount}}{\left(\frac{R_p}{I_E}\right)} \cdot \left[1 + 2 \cdot \left(\frac{z}{h}\right)\right]$$ Equation 13.3-1

$F_{pMount} = 1.269 \cdot plf$ Seismic load along length of Antenna Pipe Mount

$F_{pmin} := .3 \cdot S_{DS} \cdot W_{pMount} \cdot I_E$ Equation 13.3-3

$F_{pmin} = 0.793 \frac{1}{ft} \cdot lbf$

Force on Nokia GSM Cabinet

$W_{pNokia} := 600lbf$

$R_p := 2.5$ Table 13.6-1

$z := 1 \quad ft$

$h := 1 \quad ft$

$$F_{pNokia} := \frac{.4 \cdot a_p \cdot S_{DS} \cdot W_{pNokia}}{\left(\frac{R_p}{I_E}\right)} \cdot \left[1 + 2 \cdot \left(\frac{z}{h}\right)\right]$$ Equation 13.3-1

$F_{pNokia} = 208.031\,lbf$ Seismic load at center of Nokia GSM Cabinet

$F_{pmin} := .3 \cdot S_{DS} \cdot W_{pNokia} \cdot I_E$ Equation 13.3-3

$F_{pmin} = 130.019 \cdot lbf$

Force on LTE Rack

$W_{pLTE} := 500lbf$

$R_p := 2.5$ Table 13.6-1

$z := 1 \quad ft$

$h := 1 \quad ft$

$$F_{pLTE} := \frac{.4 \cdot a_p \cdot S_{DS} \cdot W_{pLTE}}{\left(\frac{R_p}{I_E}\right)} \cdot \left[1 + 2 \cdot \left(\frac{z}{h}\right)\right]$$ Equation 13.3-1

$F_{pLTE} = 173.359\,lbf$ Seismic load at center of LTE Rack

$F_{pmin} := .3 \cdot S_{DS} \cdot W_{pLTE} \cdot I_E$ Equation 13.3-3

$F_{pmin} = 108.349 \cdot lbf$

Force on P65-17-XLH-RR

$W_{pAnt} := 70lbf$

$R_p := 2.5$ Table 13.6-1

$z := 1 \quad ft$

$h := 1 \quad ft$

$$F_{pAnt} := \frac{.4 \cdot a_p \cdot S_{DS} \cdot W_{pAnt}}{\left(\frac{R_p}{I_E}\right)} \cdot \left[1 + 2 \cdot \left(\frac{z}{h}\right)\right]$$ Equation 13.3-1

$F_{pAnt} = 24.27\,lbf$ Seismic load at center of P65-17-XLH-RR

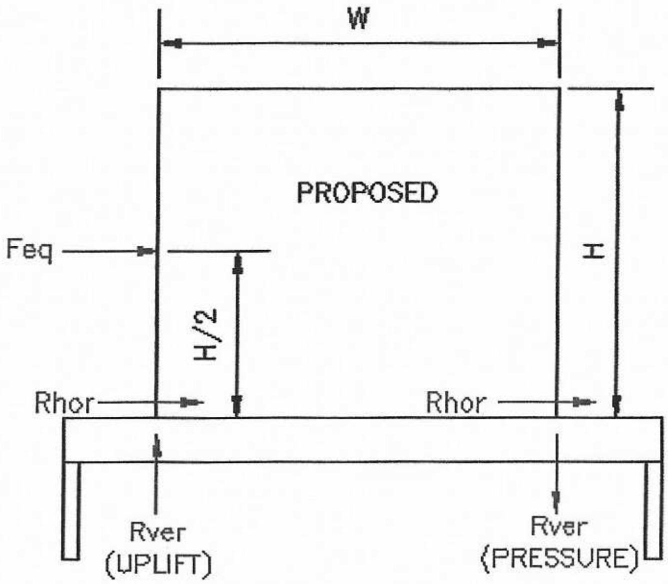
$F_{pmin} := .3 \cdot S_{DS} \cdot W_{pAnt} \cdot I_E$ Equation 13.3-3

$F_{pmin} = 15.169 \cdot lbf$

Equipment Dimensions:

For BBU Rack	For Nokia GSM Cabinet	For LTE Rack
$H_{BBU} := 7ft$	$H_{Nokia} := 70.9in$	$H_{LTE} := 84in$
$W_{BBU} := 27in$	$W_{Nokia} := 24in$	$W_{LTE} := 19in$
$D_{BBU} := 24in$	$D_{Nokia} := 24in$	$D_{LTE} := 12in$

Uplift on floor due to lateral loads on cabinets:



BBU Rack

Transverse:

$F_{pBBU} = 1074.825 lbf$

$$R_{verETBBU} := \frac{F_{pBBU} \cdot \frac{H_{BBU}}{2}}{D_{BBU}}$$

$R_{verETBBU} = 1880.944 lbf$

Longitudinal:

$F_{pBBU} = 1074.825 lbf$

$$R_{verELBBU} := \frac{F_{pBBU} \cdot \frac{H_{BBU}}{2}}{W_{BBU}}$$

$R_{verELBBU} = 1671.95 lbf$

Nokia GSM Cabinet:

Transverse:

$F_{pNokia} = 208.031\,lbf$

$$R_{verETNokia} := \frac{F_{pNokia} \cdot \frac{H_{Nokia}}{2}}{D_{Nokia}}$$

$R_{verETNokia} = 307.279\,lbf$

LTE Rack:

Transverse:

$F_{pLTE} = 173.359\,lbf$

$$R_{verETLTE} := \frac{F_{pLTE} \cdot \frac{H_{LTE}}{2}}{D_{LTE}}$$

$R_{verETLTE} = 606.756\,lbf$

Longitudinal:

$F_{pLTE} = 173.359\,lbf$

$$R_{verELLTE} := \frac{F_{pLTE} \cdot \frac{H_{LTE}}{2}}{W_{LTE}}$$

$R_{verELLTE} = 383.214\,lbf$

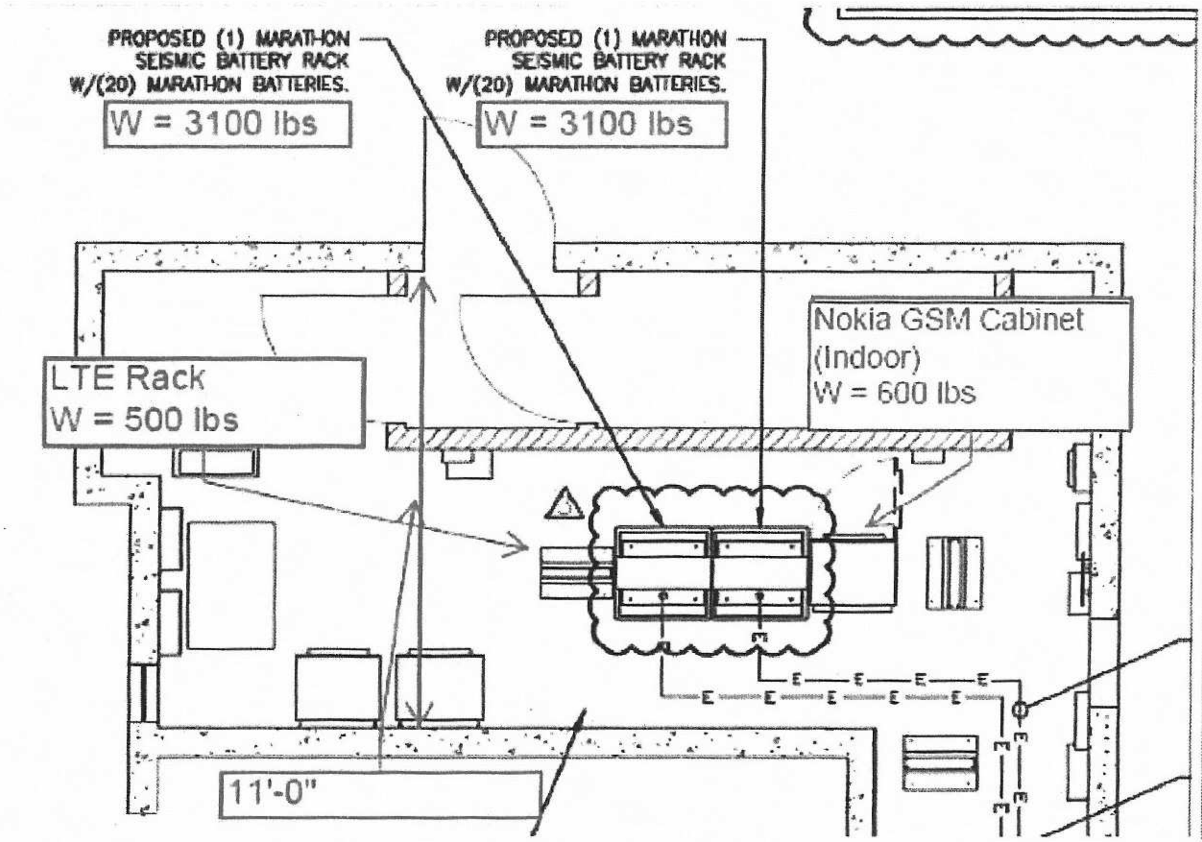
5)- Live Loads

Assume LL = 40 psf (for limited access, per ASCE 7-05)

C) - ANALYSIS

1. Existing elevated floor is comprised of 6" thick concrete slab reinforced with #4 bars at 8" O.C.

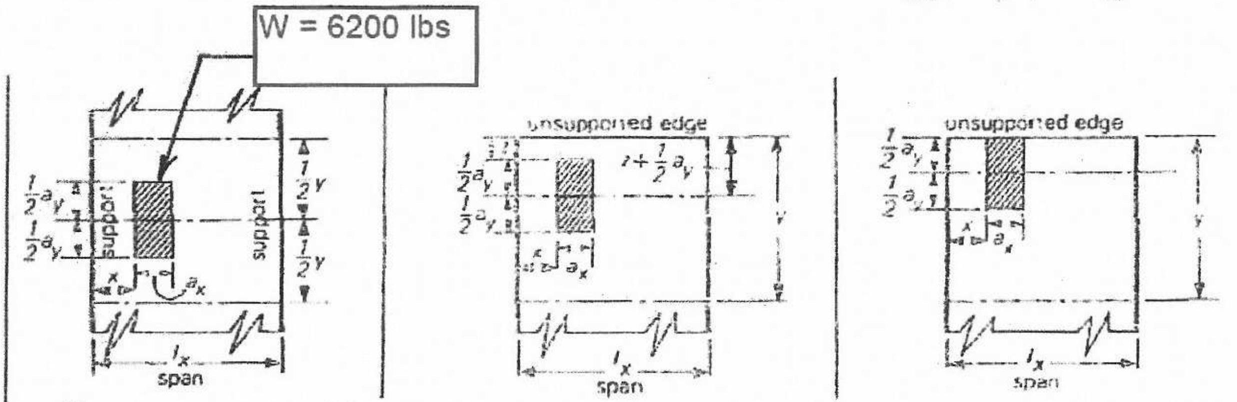
Span := 11ft



Existing Non-Load Bearing Wall - assume metal stud construction - W = 5 psf

$P_{wall} := 5psf \cdot 8ft$ $\qquad P_{wall} = 40 \cdot plf$

Calculate width of slab that resists the existing BBU loads:



For unsymmetrical load ($e < \frac{1}{2}l_x$, where $e = x + \frac{1}{2}a_x$)

Maximum bending moment on freely-supported span $= \frac{Fe}{y}\left[\left(1 - \frac{e}{l_x}\right)\left(1 - \frac{a_x}{2l_x}\right)\right]$ where

$y = a_y + 2.4e\left(1 - \frac{e}{l_x}\right)$ | $y = z + a_y + 1.2e\left(1 - \frac{e}{l_x}\right)$ | $y = a_y + 1.2e\left(1 - \frac{e}{l_x}\right)$

For symmetrical load ($e = \frac{1}{2}l_x$, where $e = x + \frac{1}{2}a_x$)

Maximum bending moment on freely supported slab $= \frac{F}{4y}(l_x - \frac{1}{2}a_x)$ (at midspan) where

$y = a_y + 0.6l_x$ | $y = z + a_y + 0.3l_x$ | $y = a_y + 0.3l_x$

If F is in kN and dimensions are in m, bending moments are in kN-m/m width
If F is in lb and dimensions are in ft, bending moments are in lb-ft/ft width

$a_y := 4.5\text{ft}$

$a_x := 2\text{ft}$

$x := 2.583\text{ft}$

$e := x + \frac{1}{2} \cdot a_x$

$e = 3.583\text{ft}$

$l_x := 11\text{ft}$

$y := a_y + 2.4 \cdot e \cdot \left(1 - \frac{e}{l_x}\right)$

$y = 10.298\text{ft}$

Analyze 10.3' wide concrete slab strip supporting BBUs, LTE rack and GSM cabinet

Proposed Equipment Loading:

$$w_{equip} := \frac{W_{pBBU} \cdot 2 + W_{pNokia} + W_{pLTE}}{8.083ft \cdot D_{BBU}}$$

$$w_{equip} = 451.565 \cdot psf$$

Existing Floor Loading:

Dead Loads:

$$DL_{slab} := 150pcf \cdot 6in \cdot y$$

$$DL_{slab} = 772.365 \cdot plf$$

$$DL_{misc} := 5psf \cdot y$$

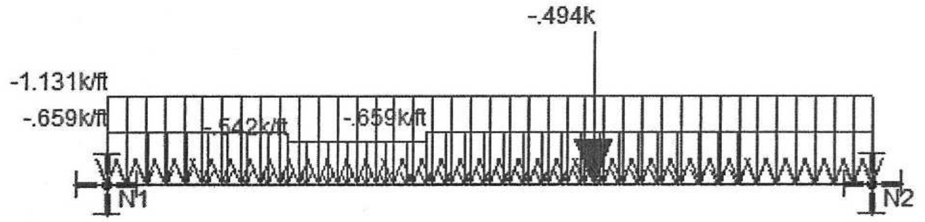
$$DL_{floor} := DL_{slab} + DL_{misc}$$

$$DL_{floor} = 823.856\, ft \cdot psf$$

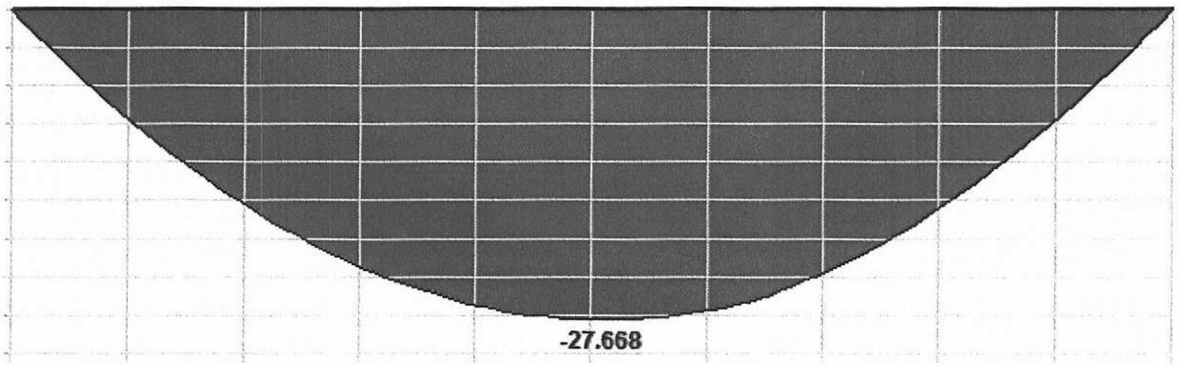
Live Loads:

$$LL_{floor} := 40psf \cdot y$$

Slab Loads (1.2 D + 1.6 L):

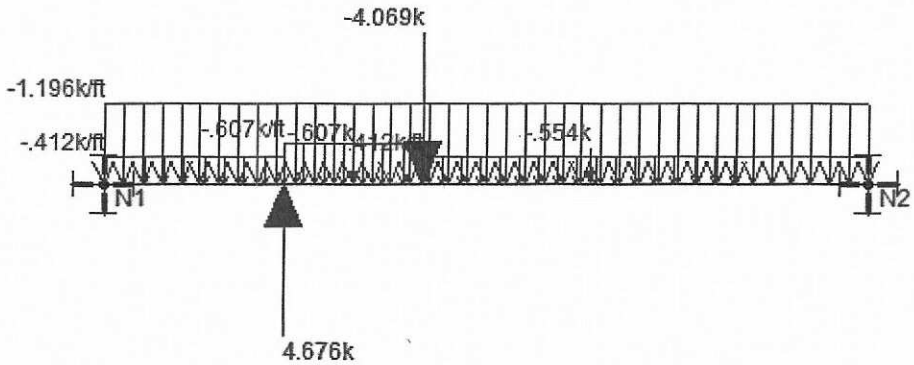


Slab Moment (1.2 D + 1.6 L):

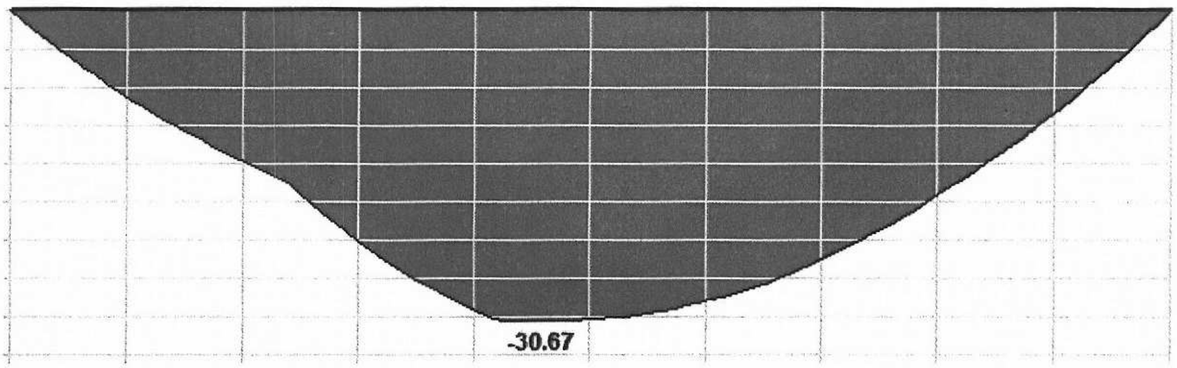


$M_{u1} := 27.7 kip \cdot ft$

Slab Loads ((1.2 + 0.2 Sds) D + E + L):



Slab Moment ((1.2 + 0.2 Sds) D + E + L):



$M_{u2} := 30.67 kip \cdot ft$

Existing Slab Capacity:

$b := 10.3ft$

$d := 6in - .75in - \frac{.5in}{2}$

$F_y := 40ksi$

$f_c := 3ksi$

Existing steel = #4 @ 8" O.C. $A_s := 16 \cdot .2in^2$

$A_s = 3.2 \cdot in^2$

$\rho := \frac{A_s}{b \cdot d}$

$\rho = 0.0052$

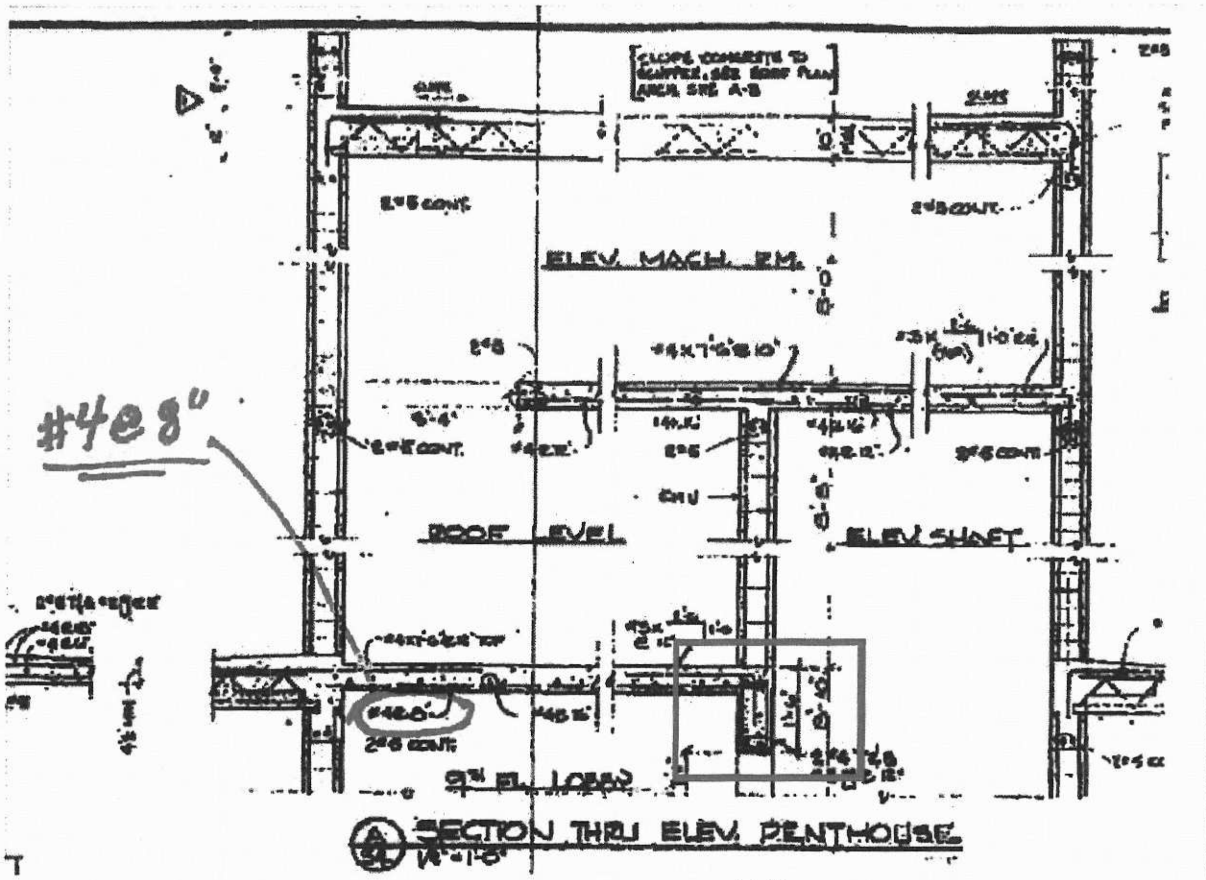
$\phi := .9$

$M_n := \phi \cdot b \cdot d^2 \cdot \rho \cdot F_y \cdot \left(1 - .59 \cdot \rho \cdot \frac{F_y}{f_c}\right)$

$M_n = 46.045 \cdot kip \cdot ft$ > $M_{u2} = 30.67 \cdot kip \cdot ft$ OK

2. Check concrete beam above elevator doorway.

8" W x 18" D w/ 2-#4 bars T & B



$DL_{slab} := 150pcf \cdot 6in + 5psf$

$Trib_{slab} := \frac{11ft}{2}$

$W_{wall} := 51psf \cdot 8.67ft$

Span = 5'-0" clear + 4" each side (typical for masonry lintels)

$LL_{mach\_rm} := 150psf$ Assumed

$LL_{equip} := 40psf$

Elevator shaft width: $a := 11.33ft$

Machine room floor cantilever width: $b := 8.33ft$

$$Trib_{mach\_rm} := \frac{(a+b)^2}{2a}$$

$$w_{DL} := DL_{slab} \cdot \frac{Trib_{slab}}{2} + DL_{slab} \cdot Trib_{mach\_rm} + W_{wall}$$

$$w_{DL} = 2026.744 \cdot plf$$

$$w_{LLmach\_rm} := LL_{mach\_rm} \cdot Trib_{mach\_rm}$$

$$w_{LLmach\_rm} = 2558.576 \cdot plf$$

$$w_{LL\_slab} := 166plf$$

$$w_{mtlstudwall} := P_{wall} \cdot \frac{4ft}{11ft}$$

$$w_{mtlstudwall} = 14.545 \cdot plf$$

Dist to battery: $a := 2.583ft$

$b := D_{BBU}$

$l := Span$

$$Trib_{batt} := \frac{b}{l} \cdot \left(l - a - \frac{b}{2}\right)$$

$$w_{batt} := \frac{W_{pBBU}}{W_{BBU} \cdot D_{BBU}} \cdot Trib_{batt}$$

$$w_{batt} = 928.998 \cdot plf$$

Calculate existing beam capacity (both negaive and positive moments - beam is continuous over openings at top of wall:

$$f_c = 3000psi$$

$$F_y = 40000psi$$

$$b := 8in$$

$$d := 18in - 1.5in - \frac{.5in}{2}$$

$$A_s := 2 \cdot .2in^2$$

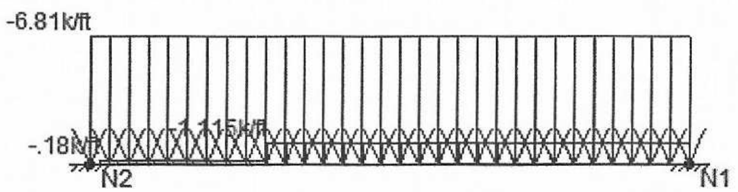
$$\rho := \frac{A_s}{b \cdot d}$$

$$\rho = 0.00308$$

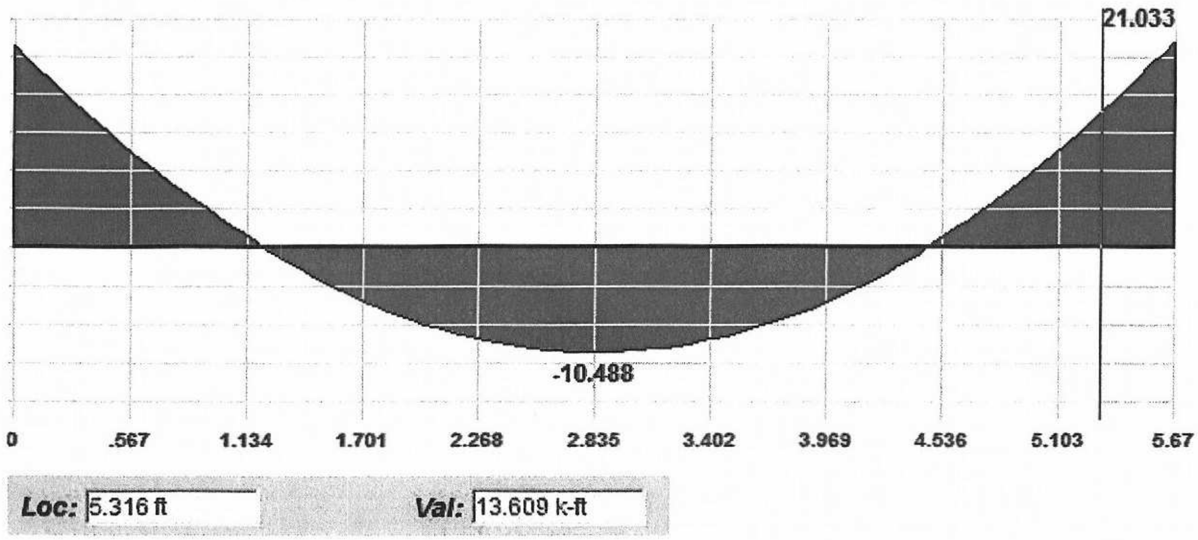
$$M_n := \phi \cdot \rho \cdot b \cdot d^2 \cdot F_y \cdot \left(1 - .59\rho \cdot \frac{F_y}{f_c}\right)$$

$$M_n = 19.028 \cdot kip \cdot ft$$

Member Loads (1.2 D + 1.6 L):



Member Moments (1.2 D + 1.6 L):



$$M_{uDL\_neg} := 13.61 kip \cdot ft$$

$$M_{uDL\_pos} := 10.5 kip \cdot ft \quad < \quad M_n = 19.028 \cdot kip \cdot ft$$

top or bottom, beam has adequate capacity for worst case positive and negative moments

Check shear in existing beam over elevator doorway for additional equipment loads:

Beam is reinforced with #3 stirrups @ 18" O.C.

$\phi_v := .75$

$A_v := 2 \cdot .11 in^2$

$V_c := 2 \cdot \sqrt{\frac{f_c}{psi}} \cdot b \cdot d \cdot psi$

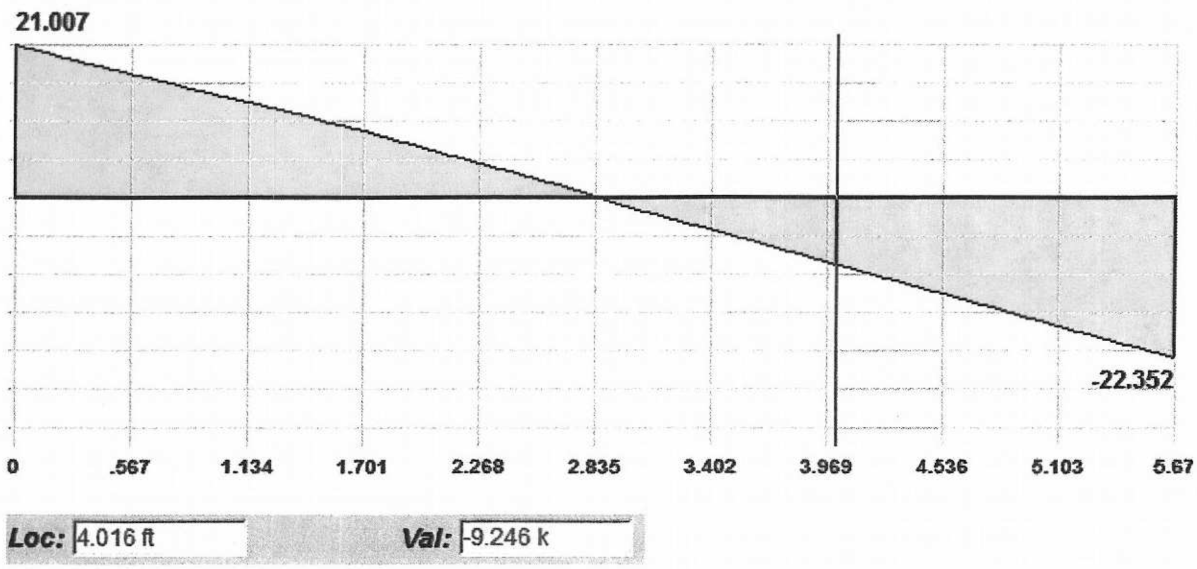
$V_s := \frac{A_v \cdot F_y \cdot d}{18in}$

$V_n := \phi_v \cdot (V_c + V_s)$

$V_n = 16.639 \cdot kip$

Critical shear occurs at "d" distance from face of support: $d = 1.354 \cdot ft$

Member Shear (1.2 D + 1.6 L):



$V_u := 9.25kip$ < $V_n = 16.639 \cdot kip$

Therefore, the existing equipment room floor has adequate capacity for the proposed battery racks.

D) - SEISMIC ANCHORAGE CALCULATIONS

1. Proposed Battery Cabinet bolts = **1/2" HILTI KWIK Bolt TZ w/ 3 1/2" embed**

   For anchorage of non-building components to concrete/masonry:

$k_H := 1.3$

$R_{pNB} := 1.5$

Total ultimate cabinet shear:

$$V_{cabinet} := F_{pBBU} \cdot k_H \cdot \frac{R_p}{R_{pNB}}$$

$$V_{cabinet} = 2328.788\,lbf \quad E$$

Uplift due to seismic overturning of battery rack:

$$F_{up} := R_{verETBBU} \cdot k_H \cdot \frac{R_p}{R_{pNB}}$$

$$F_{up} = 4.075 \times 10^3\,lbf$$

ASCE 7-05 Load cases for uplift:

12.4.2.3.7 - (0.9 D - 0.2 Sds)D + Ev

$Net\_DL := -(.9 - .2 \cdot S_{DS}) \cdot W_{pBBU}$ $\qquad Net\_DL = -2.342 \cdot kip$

Uplift per bolt:

$$Net\_Uplift := \frac{Net\_DL}{4} + \frac{F_{up}}{2}$$

$Net\_Uplift = 1.452 \times 10^3\,lbf$ @ each bolt location, worst case

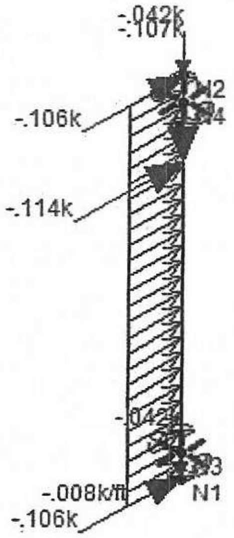
$$V_{bolt} := \frac{V_{cabinet}}{4}$$

$V_{bolt} = 582.197\,lbf$ @ each bolt location, typical

See attached Hilto Profis anchor design calculations for proposed 1/2" anchor bolt analysis results.

E) - ANTENNA MOUNT ANALYSIS

Proposed Antenna Mount Loads (1.2 D + 1.6 Wside):



Proposed Antenna Mount Usage (1.2 D + 1.6Wside):

OK

F) - ANTENNA MOUNT ANCHORAGE CALCULATIONS

Reactions on existing mounts:

Dead Load:

$V_{D2} := 123\,lbf$ per 2 bolts

$V_{D1} := 64\,lbf$ per 2 bolts

$$T_D := \frac{(V_{D1} + V_{D2}) \cdot 6in}{7ft}$$

$T_D = 13.357\,lbf$ per 2 bolts (top attachment only)

Wind Load (side):

$V_{W1} := 92\,lbf$ per 2 bolts

$V_{W2} := 151\,lbf$ per 2 bolts

$$T_{W1} := \frac{(V_{W1}) \cdot 6in}{8in}$$

$T_{W1} = 69\,lbf$ top attachment only

$$T_{W2} := \frac{(V_{W2}) \cdot 6in}{8in}$$

$T_{W2} = 113.25\,lbf$ bottom attachment only

Seismic Load (front)

$$T_{Ef1} := 23\,lbf \cdot k_H \cdot \frac{R_p}{R_{pNB}}$$

$T_{Ef1} = 49.833\,lbf$ per 2 bolts

$$T_{Ef2} := 43\,lbf \cdot \left(k_H \cdot \frac{R_p}{R_{pNB}}\right)$$

$T_{Ef2} = 93.167\,lbf$ per 2 bolts

Seismic Load (side)

$$V_{Es1} := 20\,lbf \cdot k_H \cdot \frac{R_p}{R_{pNB}}$$

$V_{Es1} = 43.333\,lbf$ per 2 bolts

$$V_{Es2} := 46\text{lbf} \cdot \left(k_H \cdot \frac{R_p}{R_{pNB}}\right)$$

$V_{Es2} = 99.667\text{lbf}$ per 2 bolts

$$T_{Es1} := \frac{(V_{Es1}) \cdot 6\text{in}}{8\text{in}}$$

$T_{Es1} = 32.5\text{lbf}$ 1 bolt only - top attachment only

$$T_{Es2} := \frac{(V_{Es2}) \cdot 6\text{in}}{8\text{in}}$$

$T_{Es2} = 74.75\text{lbf}$ 1 bolt only - bottom attachment only

For 1/2" anchor bolt with HY-120 epoxy (assume 2.5" embed):

$T_a := 1160\text{lbf}$ $T_{n\_steel} := 10.29\text{kip}$

$V_a := 1635\text{lbf}$ $V_{n\_steel} := 6.175\text{kip}$

Assume factor of safety for tension = 4 and factor of safety for shear = 3

$FS_T := 4$

$FS_V := 3$

$T_n := FS_T \cdot T_a$ $T_n = 4.64 \cdot \text{kip}$

$V_n := FS_V \cdot V_a$ $V_n = 4.905 \cdot \text{kip}$

$\phi_{vSeismic\_steel} := .7$

$\phi_{t\_steel} := .65$

$\phi_{v\_steel} := .6$

$\phi_{t\_conc} := .65$

$\phi_{v\_conc} := .7$

$V_{n\_steel} := \phi_{v\_steel} \cdot V_{n\_steel}$

$V_{n\_steel} = 3.705 \times 10^{3}\ lbf$

$V_{n\_seismic\_steel} := \phi_{v\_steel} \cdot \phi_{vSeismic\_steel} \cdot V_{n\_steel}$

$V_{n\_seismic\_steel} = 1.556 \times 10^{3}\ lbf$

$T_{n\_steel} := \phi_{t\_steel} \cdot T_{n\_steel}$

$T_{n\_steel} = 6.689 \times 10^{3}\ lbf$

$V_{n\_conc} := \phi_{v\_conc} \cdot V_{n}$

$V_{n\_conc} = 3.434 \times 10^{3}\ lbf$

$T_{n\_conc} := \phi_{t\_conc} \cdot T_{n}$

$T_{n\_conc} = 3.016 \times 10^{3}\ lbf$

$T_{n} := \min(T_{n\_steel}, T_{n\_conc})$

$T_{n} = 3.016 \times 10^{3}\ lbf$

$V_{n} := \min(V_{n\_steel}, V_{n\_conc})$

$V_{n} = 3.434 \times 10^{3}\ lbf$

$V_{n\_seismic} := \min(V_{n\_seismic\_steel}, V_{n\_conc})$

$V_{n\_seismic} = 1.556 \times 10^{3}\ lbf$

Load Combo 1 - 1.2 D + 1.6 W

$$V_{u1\_1} := \sqrt{\left(1.2 \cdot \frac{V_{D1}}{2}\right)^{2} + \left(1.6 \cdot \frac{V_{W1}}{2}\right)^{2}}$$

$V_{u1\_1} = 83.015\ lbf$ $<$ $V_{n} = 3.434 \times 10^{3}\ lbf$ OK

$$T_{u1\_1} := 1.2 \cdot \frac{T_{D}}{2} + 1.6 \cdot T_{W1}$$

$T_{u1\_1} = 118.414\ lbf$ $<$ $T_{n} = 3.016 \times 10^{3}\ lbf$ OK

$$\left(\frac{V_{u1\_1}}{V_n}\right)^{\frac{5}{3}} + \left(\frac{T_{u1\_1}}{T_n}\right)^{\frac{5}{3}} = 0.007$$ < 1 OK

$$V_{u1\_2} := \sqrt{\left(1.2 \cdot \frac{V_{D2}}{2}\right)^2 + \left(1.6 \cdot \frac{V_{W2}}{2}\right)^2}$$

$V_{u1\_2} = 141.559\,lbf$ < $V_n = 3.434 \times 10^3\,lbf$ OK

$T_{u1\_2} := 1.6 \cdot T_{W2}$

$T_{u1\_2} = 181.2\,lbf$ < $T_n = 3.016 \times 10^3\,lbf$ OK

$$\left(\frac{V_{u1\_2}}{V_n}\right)^{\frac{5}{3}} + \left(\frac{T_{u1\_2}}{T_n}\right)^{\frac{5}{3}} = 0.014$$ < 1 OK

Load Combo 2 - (1.2 + 0.2Sds) D - Ex

$$V_{u2\_1} := 1.344 \cdot \frac{V_{D1}}{2}$$

$V_{u2\_1} = 43.008\,lbf$ < $V_{n\_seismic} = 1.556 \times 10^3\,lbf$ OK

$$T_{u2\_1} := \frac{T_{Ef2}}{2} + 1.344 \cdot \frac{T_D}{2}$$

$T_{u2\_1} = 55.559\,lbf$ < $T_n = 3.016 \times 10^3\,lbf$ OK

$$\left(\frac{V_{u2\_1}}{V_n}\right)^{\frac{5}{3}} + \left(\frac{T_{u2\_1}}{T_n}\right)^{\frac{5}{3}} = 0.002$$ < 1 OK

$$V_{u2\_2} := 1.344 \cdot \frac{V_{D2}}{2}$$

$V_{u2\_2} = 82.656\,lbf$ < $V_{n\_seismic} = 1.556 \times 10^3\,lbf$ OK

$$T_{u2\_2} := \frac{T_{Ef2}}{2}$$

$T_{u2\_2} = 46.583\,\mathrm{lbf}$ < $T_n = 3.016 \times 10^3\,\mathrm{lbf}$ OK

$$\left(\frac{V_{u2\_2}}{V_n}\right)^{\frac{5}{3}} + \left(\frac{T_{u2\_2}}{T_n}\right)^{\frac{5}{3}} = 0.003$$ < 1 OK

Load Combo 3 - (1.2 + 0.2Sds) D - Ez

$$V_{u3\_1} := \sqrt{\left(1.344 \cdot \frac{V_{D1}}{2}\right)^2 + \left(\frac{V_{Es1}}{2}\right)^2}$$

$V_{u3\_1} = 48.157\,\mathrm{lbf}$ < $V_{n\_seismic} = 1.556 \times 10^3\,\mathrm{lbf}$ OK

$$T_{u3\_1} := T_{Es1} + 1.344 \cdot \frac{T_D}{2}$$

$T_{u3\_1} = 41.476\,\mathrm{lbf}$ < $T_n = 3.016 \times 10^3\,\mathrm{lbf}$ OK

$$\left(\frac{V_{u3\_1}}{V_n}\right)^{\frac{5}{3}} + \left(\frac{T_{u3\_1}}{T_n}\right)^{\frac{5}{3}} = 0.002$$ < 1 OK

$$V_{u3\_2} := \sqrt{\left(1.344 \cdot \frac{V_{D2}}{2}\right)^2 + \left(\frac{V_{Es2}}{2}\right)^2}$$

$V_{u3\_2} = 96.516\,\mathrm{lbf}$ < $V_{n\_seismic} = 1.556 \times 10^3\,\mathrm{lbf}$ OK

$$T_{u3\_2} := T_{Es2}$$

$T_{u3\_2} = 74.75\,\mathrm{lbf}$ < $T_n = 3.016 \times 10^3\,\mathrm{lbf}$ OK

$$\left(\frac{V_{u3\_2}}{V_n}\right)^{\frac{5}{3}} + \left(\frac{T_{u3\_2}}{T_n}\right)^{\frac{5}{3}} = 0.005$$ < 1 OK

Therefore, the existing antenna mount and connections have adequate capacity for the increased antenna loads.

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 45.535964
Longitude = -122.65598099999998
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.05 deg grid spacing

Period (sec)	Sa (g)
0.2	0.977 (Ss, Site Class B)
1.0	0.340 (S1, Site Class B)

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 45.535964
Longitude = -122.65598099999998
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.109 ,Fv = 1.721

Period (sec)	Sa (g)
0.2	1.084 (SMs, Site Class D)
1.0	0.585 (SM1, Site Class D)

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 45.535964
Longitude = -122.65598099999998
Design Spectral Response Accelerations SDs and SD1
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.109 ,Fv = 1.721

Period (sec)	Sa (g)
0.2	0.722 (SDs, Site Class D)
1.0	0.390 (SD1, Site Class D)

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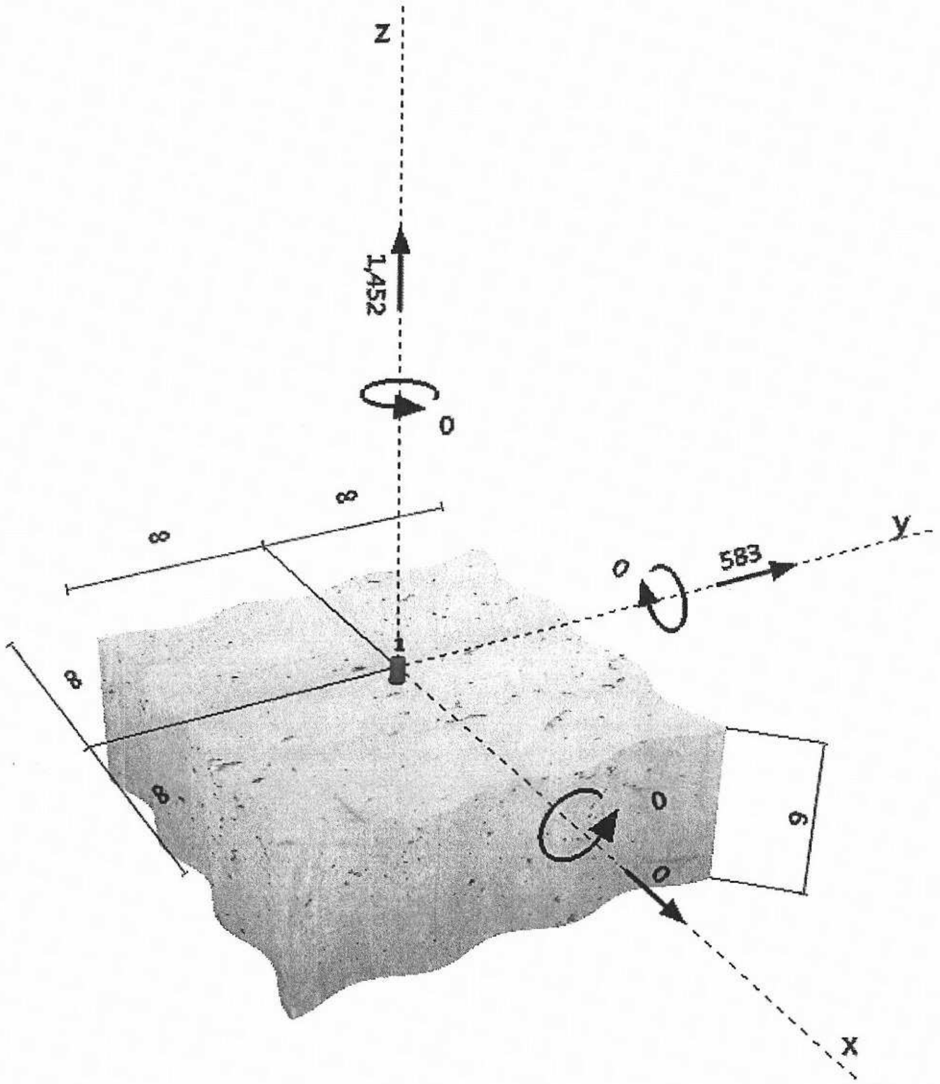
Phone I Fax:	|	Date:	6/25/2013
E-Mail:			

Specifier's comments:

1 Input data

Anchor type and diameter:	**Kwik Bolt TZ - CS 1/2 (3 1/4)**
Effective embedment depth:	h_{ef} = 3.250 in., h_{nom} = 3.625 in.
Material:	Carbon Steel
Evaluation Service Report::	ESR 1917
Issued I Valid:	4/1/2012 | 5/1/2013
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 = 6.000 in.
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	yes (D.3.3.6)

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	1452	583	0	583

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 ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	1452	8029	19	OK
Pullout Strength*	1452	2625	56	OK
Concrete Breakout Strength**	1452	2660	55	OK

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

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR 1917
$\phi N_{steel} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

Variables

n	$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
1	0.10	106000

Calculations

N_{sa} [lb]
10705

Results

N_{sa} [lb]	ϕ_{steel}	ϕN_{sa} [lb]	N_{ua} [lb]
10705	0.750	8029	1452

3.2 Pullout Strength

$N_{pn,f'_c} = N_{p,2500} \sqrt{\frac{f'_c}{2500}}$ refer to ICC-ES ESR 1917
$\phi N_{pn,f'_c} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

Variables

f'_c [psi]	$N_{p,2500}$ [lb]
3000	4915

Calculations

$\sqrt{\frac{f'_c}{2500}}$
1.095

Results

N_{pn,f'_c} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi N_{pn,f'_c}$ [lb]	N_{ua} [lb]
5384	0.650	0.750	1.000	2625	1452

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3.3 Concrete Breakout Strength

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b$ — ACI 318-08 Eq. (D-4)

$\phi\, N_{cb} \geq N_{ua}$ — ACI 318-08 Eq. (D-1)

A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ — ACI 318-08 Eq. (D-6)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e_N'}{3\, h_{ef}}}\right) \leq 1.0$ — ACI 318-08 Eq. (D-9)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ — ACI 318-08 Eq. (D-11)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-08 Eq. (D-13)

$N_b = k_c\, \lambda\, \sqrt{f_c'}\, h_{ef}^{1.5}$ — ACI 318-08 Eq. (D-7)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
3.250	0.000	0.000	∞	1.000

c_{ac} [in.]	k_c	λ	f_c' [psi]
7.500	17	1	3000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
95.06	95.06	1.000	1.000	1.000	1.000	5455

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\, N_{cb}$ [lb]	N_{ua} [lb]
5455	0.650	0.750	1.000	2660	1452

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	583	3572	17	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	583	5728	11	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

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

4.1 Steel Strength

V_{seis} = ESR value refer to ICC-ES ESR 1917

$\phi V_{steel} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

Variables

n	$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
1	0.10	106000

Calculations

V_{sa} [lb]
5495

Results

V_{sa} [lb]	ϕ_{steel}	ϕV_{sa} [lb]	V_{ua} [lb]
5495	0.650	3572	583

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 318-08 Eq. (D-30)

$\phi V_{cp} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-08 Eq. (D-6)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e_N'}{3\, h_{ef}}} \right) \leq 1.0$ ACI 318-08 Eq. (D-9)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-08 Eq. (D-11)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-08 Eq. (D-13)

$N_b = k_c \, \lambda \, \sqrt{f_c'} \, h_{ef}^{1.5}$ ACI 318-08 Eq. (D-7)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	3.250	0.000	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ	f_c' [psi]
1.000	7.500	17	1	3000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
95.06	95.06	1.000	1.000	1.000	1.000	5455

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕV_{cp} [lb]	V_{ua} [lb]
10911	0.700	0.750	1.000	5728	583

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.553	0.163	5/3	43	OK

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

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6 Warnings

- To avoid failure of the anchor plate the required thickness can be calculated in PROFIS Anchor. Load re-distributions 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 loading!
- 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 ACI 318, Part D.4.4(c).
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI318 or the relevant standard!
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-08 Appendix D, Part D.3.3.4 this requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, Part D.3.3.5 requires that the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. In lieu of D.3.3.4 and D.3.3.5, the minimum design strength of the anchors shall be multiplied by a reduction factor per D.3.3.6.
  An alternative anchor design approach to ACI 318-08, Part D.3.3 is given in IBC 2009, Section 1908.1.9. This approach contains "Exceptions" that may be applied in lieu of D.3.3 for applications involving "non-structural components" as defined in ASCE 7, Section 13.4.2.
  An alternative anchor design approach to ACI 318-08, Part D.3.3 is given in IBC 2009, Section 1908.1.9. This approach contains "Exceptions" that may be applied in lieu of D.3.3 for applications involving "wall out-of-plane forces" as defined in ASCE 7, Equation 12.11-1 or Equation 12.14-10.
- It is the responsibility of the user when inputting values for brittle reduction factors ($\phi_{nonductile}$) different than those noted in ACI 318-08, Part D.3.3.6 to determine if they are consistent with the design provisions of ACI 318-08, ASCE 7 and the governing building code.
  Selection of $\phi_{nonductile}$ = 1.0 as a means of satisfying ACI 318-08, Part D.3.3.5 assumes the user has designed the attachment that the anchor is connecting to undergo ductile yielding at a force level <= the design strengths calculated per ACI 318-08, Part D.3.3.3.

Fastening meets the design criteria!

Company:		Page:	6
Specifier:		Project:	10094222 Lloyd Center
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	I	Date:	6/25/2013
E-Mail:			

7 Installation data

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

Anchor type and diameter: Kwik Bolt TZ - CS, 1/2 (3 1/4)
Installation torque: 480.001 in.lb
Hole diameter in the base material: 0.500 in.
Hole depth in the base material: 3.625 in.
Minimum thickness of the base material: 6.000 in.

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	-	-	-	-

8 Remarks; Your Cooperation Duties

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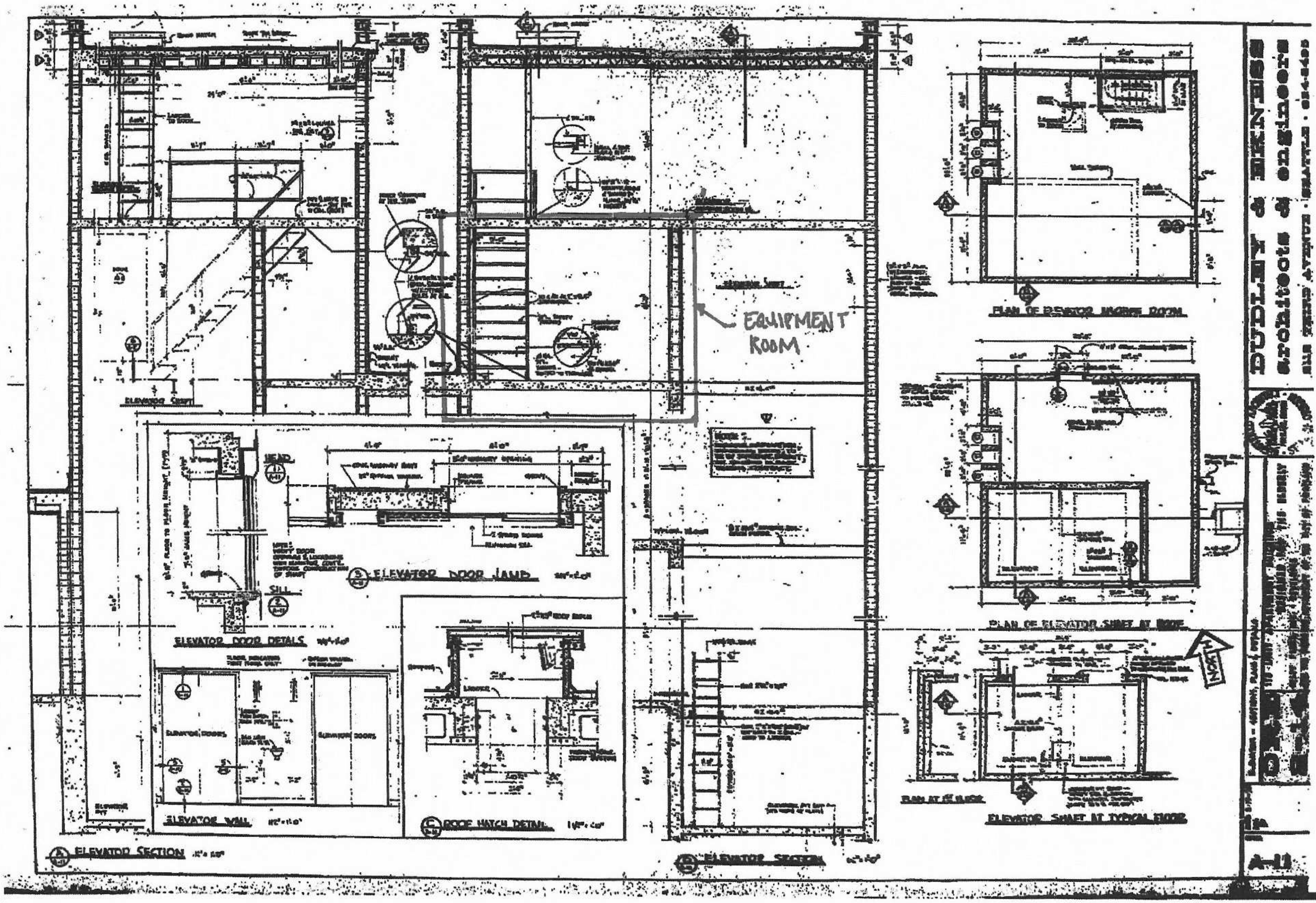
EQUIPMENT
ROOM
ELEVATOR SECTION
ELEVATOR DOOR JAMB
HEAD
SILL
ELEVATOR DOOR DETAILS
ELEVATOR WALL
ELEVATOR DOORS
ELEVATOR DOORS
ROOF HATCH DETAIL
ELEVATOR SECTION
PLAN OF ELEVATOR MACHINE ROOM
PLAN OF ELEVATOR SHAFT AT ROOF
NORTH
PLAN AT 1ST FLOOR
ELEVATOR SHAFT AT TYPICAL FLOOR
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