glassRAILINGS > UNI
STRUCTURAL TESTING DATA

Structural Design
of
Glass Guardrail
for
GlassRAILING > UNI

Prepared For

CARVART Glass
1441 Broadway 28th Floor
New York, NY 10018

Prepared By

TONGJI CONSULTING ASSOCIATE, INC
71-07 170th Street
Fresh Meadows, NY 11365

February 15, 2021

STATE OF NEW YORK
JIAN WANG
084654
LICENSED PROFESSIONAL ENGINEER
02/14/2021

Job Description

This worksheet is for the stress/deflection summary table of the guardrail glass panel with varied width,height and thickness for Carvart Glass product. (highlighted in green is the recommendation, see next pages)

Notes

1. laminated full tempered glass guardrail with varied thickness and height are checked stress and deflection with different type of interlayer material (SGP and PVB).
2. only interior guardrail glass panel is checked with 50 pounds force pier linear foot or minimum 200 pounds force live load is applied at top of glass panel per NYC building code 2014 Edition.
3. effective thickness method for laminated glass panel is used per ASTM E1300-16 considering the load duration time and temperature.
4. maximum 6 ksi allowable tension stress in glass panel is used per NYC building code 2014 edition chapter 24.
5. no direct code requirement on the glass guardrail deflection check under live load, based on engineering judgement, one (1) inch deflection is set as the limit of deflection. note that the deflection limit may varies per specific project specification. so the calculated deflection is listed for reference purpose,
6. for stress and deflection check, cantilever length with fix support at finished floor is used.
7. structural silicone stress is checked.
8. with this report with glass shoe strength test report, which is provided by the third party.
9. concrete anchor design is provided as a sample, not direct design for specific project, contractor engineer of project shall be responsible for final anchor/screw design.

Project: **Carvart Interior Glass Guardrail Design**
Subject: **Summary Table of Guardrail Design**
Designed by:J. W
Date: 02/15/2021

Index No. .
Job. No. .

Table 1: Live load Stress/Deflection of laminated 43" high glass guardrail

43" high Guardrail free standing panel width (ft)	4 ft (min.)		3 ft		2ft	
Laminated Glass thickness	LL Glass Stress (ksi) *	LL Glass deflection (inches)**	LL Glass Stress (ksi)	LL Glass deflection (inches)	LL Glass Stress (ksi)	LL Glass deflection (inches)
1/4" FT + 0.06" SGP interlayer + 1/4" FT (total thickness: 9/16")	4.50	1.16	-	-	-	-
1/4" FT + 0.06" PVB interlayer + 1/4" FT (total thickness: 9/16")	-	-	-	-	-	-
5/16" FT + 0.06" SGP interlayer + 5/16" FT (total thickness: 11/16")	2.65	0.52	3.58	0.71	5.58	1.19
5/16" FT + 0.06" PVB interlayer + 5/16" FT (total thickness: 11/16")	4.79	1.73	-	-	-	-
3/8" FT + 0.06" SGP interlayer + 3/8" FT (total thickness: 13/16")	1.85	0.30	2.48	0.40	3.69	0.59
3/8" FT + 0.06" PVB interlayer + 3/8" FT (total thickness: 13/16")	3.41	1.04	4.80	1.50	-	-
1/2" FT + 0.06" SGP interlayer + 1/2" FT (total thickness: 17/16")	1.09	0.13	1.46	0.18	2.32	0.32
1/2" FT + 0.06" PVB interlayer + 1/2" FT (total thickness: 17/16")	2.07	0.49	2.88	0.69	4.62	1.15

Project: **Carvart Interior Glass Guardrail Design**
Subject: **Summary Table of Guardrail Design**
Designed by:J. W
Date: 02/15/2021

Index No. .
Job. No. .

Table 2: Live load Stress/Deflection of laminated 55" high glass guardrail

55" high Guardrail free standing panel width (ft)	4 ft (min.)		3 ft		2ft	
Laminated Glass thickness	LL Glass Stress (ksi) *	LL Glass deflection (inches)**	LL Glass Stress (ksi)	LL Glass deflection (inches)	LL Glass Stress (ksi)	LL Glass deflection (inches)
1/4" FT + 0.06" SGP interlayer + 1/4" FT (total thickness: 9/16")	5.7	2.37	-	-	-	-
1/4" FT + 0.06" PVB interlayer + 1/4" FT (total thickness: 9/16")	-	-	-	-	-	-
5/16" FT + 0.06" SGP interlayer + 5/16" FT (total thickness: 11/16")	3.37	1.06	4.57	1.48	-	-
5/16" FT + 0.06" PVB interlayer + 5/16" FT (total thickness: 11/16")	5.86	3.36	-	-	-	-
3/8" FT + 0.06" SGP interlayer + 3/8" FT (total thickness: 13/16")	2.35	0.61	3.17	0.84	4.89	1.37
3/8" FT + 0.06" PVB interlayer + 3/8" FT (total thickness: 13/16")	4.19	2.03	-	-	-	-
1/2" FT + 0.06" SGP interlayer + 1/2" FT (total thickness: 17/16")	1.39	0.28	1.87	0.38	2.96	0.66
1/2" FT + 0.06" PVB interlayer + 1/2" FT (total thickness: 17/16")	2.56	0.97	3.68	1.45	5.9	2.39

Project: **Carvart Interior Glass Guardrail Design**
Subject: **Summary Table of Guardrail Design**
Designed by:J. W
Date: 02/15/2021

Index No. .
Job. No. .

Table 3: Live Load Stress/Deflection of laminated 72" high glass guardrail

72" high Guardrail panel free standing width (ft)	4 ft (min.)		3 ft		2ft	
Laminated Glass thickness	LL Glass Stress (ksi) *	LL Glass deflection (inches)**	LL Glass Stress (ksi)	LL Glass deflection (inches)	LL Glass Stress (ksi)	LL Glass deflection (inches)
3/8" FT + 0.06" SGP interlayer + 3/8" FT (total thickness: 13/16")	3.06	1.36	4.13	1.87	-	-
3/8" FT + 0.06" PVB interlayer + 3/8" FT (total thickness: 13/16")	-	-	-	-	-	-
1/2" FT + 0.06" SGP interlayer + 1/2" FT (total thickness: 17/16")	1.81	0.61	2.44	0.83	3.86	1.46
1/2" FT + 0.06" PVB interlayer + 1/2" FT (total thickness: 17/16")	3.34	2.14	4.8	3.2	-	-

Notes

* max. allowable stress in full tempered glass is 6 ksi. Engineer's recommendation is highlighted in green color.
** No specific deflection limit per NYC Building Code 2014 edition. Maximum 1 inches is recommended based on engineer judgement.
*** stress/deflection is calculated under 50 plf or 200 lbf concentrated live load applied at top of guardrail, with load duration 24 hours.
**** effective thickness method is applied for laminated full tempered glass with SGP and PVB interlayer per ASTM E1300-16.

Recommended Concrete Anchor:

Recommended anchor for glassRAILING>UNI: A. Side Mount:

1. 1/2" diameter HILITI KWIK BOLT TZ (KB-TZ) carbon steel anchor with minimum 3.75" concrete embedment @ 9" max. spacing with minimum 3.5" concrete edge distance.
2. applicable to 55.125" high glass guardrail with minimum 4 ft wide.

Recommended anchor for glassRAILING>UNI: A. Top Mount:

1. 1/2" diameter HILITI KWIK BOLT TZ (KB-TZ) carbon steel anchor with minimum 3.75" concrete embedment @ 12" max. spacing with minimum 3.5" concrete edge distance.
2. applicable to 55.125" high glass guardrail with minimum 4 ft wide.

Appendix:

Stress/Deflection check & Silicone, Anchor design

Job Description

This worksheet is for the structural design of the 43" high glass guardrail with varied thickness for Carvart Glass product: glassRAILINGS > PLAN. the following items are Included:

Design Notes and Results

1.) the scope of work: glass panel strength/deflection design,

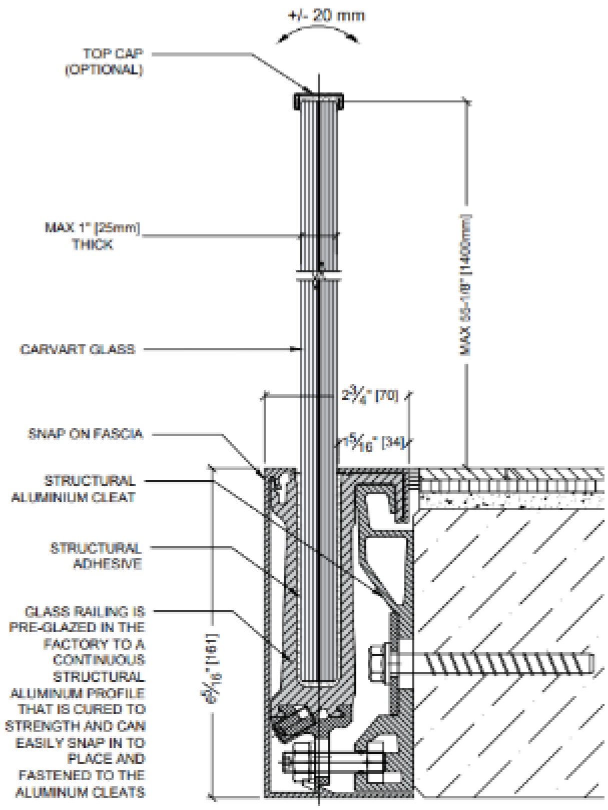
2.) No strength check of existing structure or sybstrate or items by others are in the scope of work.

3.) work this design with glass railing product.

References

1.) AISC steel construction Manual. 15th Edition
2.) NYC building construction Code. 2014
3.) ACI 318-14 Chapter 17
4.) ASTM E1300-16: Standard Practice for Determining load Resistance of Glass in Buildings

4.0 Design of Concrete Anchor *anchor (glassRAILING>UNI)*



A. SIDE MOUNT

Note
1. As a illustrative sample here, HILTI KWIK BOLT TZ (KB-TZ) carbon steel anchor is selected for design.
2. it is contractor engineer's responsibility/libaility to design the anchor. the design here is a recommendation .

$W_{guardail} := 4ft$ — width of guardrail panel

$Spacing_{anchor1} := 9in$ — spacing of lateral anchor

$N_{anchor} := floor\left(\dfrac{W_{guardail}}{Spacing_{anchor1}}\right) = 5.00$ — Number of anchor for one glass panel

$Edge_{anchor} := 2.5in$ — minimum concrete edge distance

$DL_{paneL\_plan\_A} := 160pcf \cdot 1in \cdot W_{guardail} \cdot (55.125in + 6.31in) \cdot 1.2 = 327.65\,lbf$ — dead load applied on one glass panel (assumed in conservative side)

$LL_{panel} := max\left(PLL, ULL \cdot W_{guardail}\right) = 200.00\,lbf$ — max. live load on one panel

$T_{anchor\_applied} := \dfrac{LL_{panel} \cdot (55.125in + 6.25in)}{Edge_{anchor} \cdot N_{anchor}} = 982.00\,lbf$ — tension load on one anchor

$V_{anchor\_applied} := \dfrac{DL_{paneL\_plan\_A} + LL_{panel}}{N_{anchor}} = 105.53\,lbf$ — shear load on one anchor

ESR – 1917

1/2" Dia. anchor bolt with 3.75" embedment

$d_{anchor} := \frac{1}{2} in$ — anchor bolt size

$Embedment_{anchor} := 3.25in$ — anchor bolt embedment depth

$V_{anchor\_factored} := V_{anchor\_applied} \cdot 1.6 = 0.17 \cdot kip$ — total shear factored load on one anchor, (1.6 factor is used to convert load from ASD to Strength Method) in conservative side

$T_{anchor\_factored} := 1.6 \cdot \left(T_{anchor\_applied}\right) = 1.57 \cdot kip$ — max. total Tension load of on one anchor

$N_{tension} := 1$ — number of tension bolt in group,

assuming cracked concrete

the nominal strength of one anchor rod

$N_{sa} := 10.705kip$ — Per Appendix: ESR_1917 Table 3

$\phi_{steel\_tension} := 0.75$ — Per Appendix: ESR_1917 Table 3

$\phi N_{sa} := \phi_{steel\_tension} \cdot N_{sa} = 8.03 \cdot kip$

concrete breakout strength of anchor in tension

For the definition of varies, see above figure.

$c_{a1} := 2.5in$ bolt edge distance (assumed)

$c_{a2} := 8in$ bolt edge distance (assumed)

$h_{ef} := 3.25in$

$C_{ac1} := 7.25in$ critical distance per ESR-1917 Report, Table 3 for min. 6" thickness concrete slab

$C_{ac} := \min\left(1.5 \cdot h_{ef}, C_{ac1}\right) = 4.88 \cdot in$ critical distance

$$A_{Nco} := \left(2 \cdot 1.5 \cdot h_{ef}\right) \cdot \left(2 \cdot 1.5 \cdot h_{ef}\right) = 95.06 \cdot in^2$$

$$A_{Nc} := \min\left[6in, \left(1.5 \cdot h_{ef} + \min\left(c_{a1}, 1.5 \cdot h_{ef}\right)\right)\right] \cdot \left(\min\left(c_{a2}, 1.5 \cdot h_{ef}\right) + 1.5 \cdot h_{ef}\right) = 58.50 \cdot in^2$$

$$\psi_{edN} := \min\left(1, 0.7 + 0.3 \cdot \frac{\min\left(c_{a1}, c_{a2}\right)}{1.5 \cdot h_{ef}}\right) = 0.85$$

Modification factor for anchor bolt group edge effect in tension. ACI 318-14 Eq. 17.4.2.4

$\psi_{cN} := 1.0$ Per ESR-1917 table 3

$$\psi_{cpN} := \min\left(1, \max\left(\frac{1.5 \cdot h_{ef}}{C_{ac}}, \frac{c_{a1}}{C_{ac}}\right)\right) = 1.00$$

Modification factor for anchor bolt group in tension for post-installed anchor ACI 38-14 Eq. 17.4.2.7a &7b

$k_{cr} := 17$ — Per ESR 1917 Table 3

$\lambda := 1.0$ — for normal weight concrete

$F_c := 2.5ksi$

$$N_{b1} := k_{cr} \cdot \lambda \cdot \left(\frac{F_c}{1psi}\right)^{0.5} \cdot \left(\frac{h_{ef}}{1in}\right)^{1.5} \cdot 1lbf = 4.98 \cdot kip$$

the basic concrete breakout strength of a single anchor in tension, ACI 318-14 Eq. 17.4.2.2a

$$N_{cb} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{edN} \cdot \psi_{cN} \cdot \psi_{cpN} \cdot N_{b1} = 2.62 \cdot kip$$

the nominal concrete breakout strength of anchor group in tension, ACI 318-14 Eq. 17.4.2.1a& 1b

$\phi_{co\_breakout} := 0.65$ — Per ESR-1917 Table 3

$$\phi N_{cb} := N_{cb} \cdot \phi_{co\_breakout} = 1.70 \cdot kip$$

Concrete Pullout/bond Strength of anchor in tension

$N_{P\_fc} := 4.915kip = 4.92 \cdot kip$ — Per ESR-1917 Table 3,

$\phi Pullout := N_{P\_fc} \cdot 0.65 = 3.19 \cdot kip$ — Per ESR-1917 Table 3, 0.65 reduction

Steel Strength of anchor in shear

$\phi Vsa := 5.495kip \cdot 0.65 = 3.57 \cdot kip$ — Per ESR-1917 Table 3, 0.65 reduction

Concrete breakout Strength of anchor in shear

Reference : ACI 318 -14 Chapter 17

$A_{vco} := 4.5 \cdot c_{a1}^{2} = 28.13 \cdot in^{2}$

ACI 318-14 Eq. 17.5.2.1c

$h_a := 6in = 6.00 \cdot in$

assumed the minimum depth of concrete slab

$C_{a1} := c_{a1} = 2.50 \cdot in$

$A_{vc} := \left(1.5 \cdot C_{a1} + \min\left(c_{a2}, 1.5 \cdot C_{a1}\right)\right) \cdot 1.5 \cdot C_{a1} = 28.13 \cdot in^{2}$

$$V_{b1} := 7 \cdot \left(\frac{\min\left(d_{anchor} \cdot 8, h_{ef}\right)}{d_{anchor}}\right)^{0.2} \cdot \left(\frac{d_{anchor}}{1in}\right)^{0.5} \cdot \left[\lambda \cdot \left(\frac{F_c}{1psi}\right)^{0.5} \cdot \left(\frac{C_{a1}}{1in}\right)^{1.5}\right] \cdot 1lbf = 1.42 \cdot kip$$

ACI 318-14 Eq. 17.5.2.2a

$$V_{b2} := 9\lambda \cdot \left(\frac{F_c}{1psi}\right)^{0.5} \cdot \left(\frac{C_{a1}}{1in}\right)^{1.5} \cdot (1lbf) = 1.78 \cdot kip$$

ACI 318-14 Eq. 17.5.2.2b

$V_b := \min\left(V_{b1}, V_{b2}\right) = 1.42 \cdot kip$

$e_V := 0.0in$

$$\psi_{ecV} := \frac{1}{1 + \frac{2 \cdot e_V}{3 \cdot C_{a1}}} = 1.00$$

Modification factor for anchor bolt group loaded eccentrically in shear
ACI 318-14 Eq. 17.5.2.5

$\psi_{edV} := 1.0$

Modification factor for anchor bolt group edge effect in shear
ACI 318-14 Eq. 17.5.2.6a for ca2> 1.5Ca1

$\psi_{cV} := 1.0$

Modification factor for anchor bolt group in shear for post-installed anchor
ACI 318-14 item 17.5.2.7

$\psi_{ch} := 1.0$

Modification factor for anchor bolt located in a concrete member where
ha < 1.5ca1, ACI 318-14 item 17.5.2.8

$$V_{cbg} := \frac{A_{vc}}{A_{vco}} \cdot \psi_{ecV} \cdot \psi_{edV} \cdot \psi_{cV} \cdot \psi_{ch} \cdot V_b = 1.42 \cdot kip$$

the nominal concrete breakout strength of anchor group in tension, ACI 318-14 Eq. 17.5.2.1a & 1b.

<u>Concrete pry out Strength of anchor in shear</u>

$K_{cp} := 2.0$

ESR-1917 Table 3

$\phi_{shear\_cr} := 0.70$

$$\phi V_{cpg} := \phi_{shear\_cr} \cdot K_{cp} \cdot N_{cb} = 3.66 \cdot kip$$

$$\phi N_b := \min\left(\phi N_{cb}, \phi N_{sa}, \phi Pullout\right) = 1.70 \cdot kip$$

$$\phi V_n := \min\left(\phi V_{cpg}, \phi Vsa, \phi_{shear\_cr} \cdot V_{cbg}\right) = 1.00 \cdot kip$$

$$ratio_{shear} := \frac{V_{anchor\_factored}}{\phi V_n} = 0.17$$

if Vu is less than 0.2 ϕV_n,,then full strength in tension shall be permitted. no need to check the interaction of tensile and shear forces

$$\frac{T_{anchor\_factored}}{\phi N_b} + \frac{V_{anchor\_factored}}{\phi V_n} = 1.09$$

$$Tension_{anchor} := \begin{cases} \text{"OK !!"} & \text{if } T_{anchor\_factored} \le \phi N_b \\ \text{"NG !!"} & \text{otherwise} \end{cases}$$

$Tension_{anchor} = \text{"OK !!"}$

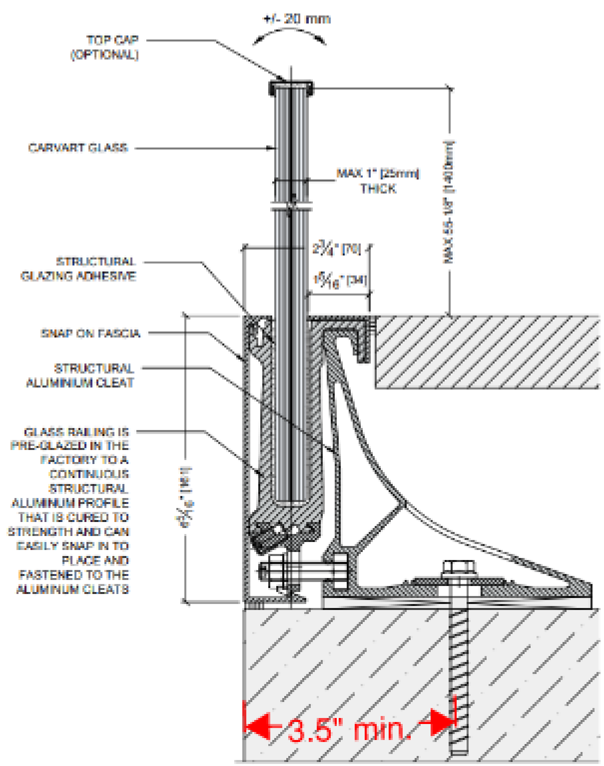
$$Shear_{anchor} := \begin{cases} \text{"OK !!"} & \text{if } V_{anchor\_factored} \le \phi V_n \\ \text{"NG !!"} & \text{otherwise} \end{cases}$$

$Shear_{anchor} = \text{"OK !!"}$

$$V\_T_{anchor\_check} := \begin{cases} \text{"OK !!"} & \text{if } \dfrac{T_{anchor\_factored}}{\phi N_b} + \dfrac{V_{anchor\_factored}}{\phi V_n} \le 1.2 \\ \text{"NG !!"} & \text{otherwise} \end{cases}$$

$V\_T_{anchor\_check} = \text{"OK !!"}$

5.0 Design of Concrete Anchor *anchor* *(glassRAILING>UNI): B. Top Mount*



B. TOP MOUNT

Note
1. As a illustrative sample here, HILTI KWIK BOLT TZ (KB-TZ) carbon steel anchor is selected for design.
2. it is contractor engineer's responsibility/libaility to design the anchor. the design here is a recommendation .

$W_{guardail} := 4ft$ width of guardrail panel

$Spacing_{anchor1} := 12in$ spacing of lateral anchor

$N_{anchor} := floor\left(\frac{W_{guardail}}{Spacing_{anchor1}}\right) = 4.00$ Number of anchor for one glass panel

$Edge_{anchor} := 2in$ minimum edge distance

$DL_{panel\_plan\_A} := 160pcf \cdot 1in \cdot W_{guardail} \cdot (55.125in + 6.31in) \cdot 1.2 = 327.65\,lbf$ dead load applied on one glass panel (assumed in conservative side)

$LL_{panel} := max\left(PLL, ULL \cdot W_{guardail}\right) = 200.00\,lbf$ max. live load on one panel

$T_{anchor\_applied} := \frac{LL_{panel} \cdot (55.125in + 6.25in)}{Edge_{anchor} \cdot N_{anchor}} = 1534.38\,lbf$ tension load on one anchor

$V_{anchor\_applied} := \frac{DL_{paneL\_plan\_A} + LL_{panel}}{N_{anchor}} = 131.91\,lbf$ shear load on one anchor

ESR – 1917

1/2" Dia. anchor bolt with 3.75" embedment

$d_{anchor} := \frac{1}{2} in$ — anchor bolt size

$Embedment_{anchor} := 3.25in$ — anchor bolt embedment depth

$V_{anchor\_factored} := V_{anchor\_applied} \cdot 1.6 = 0.21 \cdot kip$ — total shear factored load on one anchor, (1.6 factor is used to convert load from ASD to Strength Method) in conservative side

$T_{anchor\_factored} := 1.6 \cdot (T_{anchor\_applied}) = 2.46 \cdot kip$ — max. total Tension load of on one anchor

$N_{tension} := 1$ — number of tension bolt in group,

assuming cracked concrete

the nominal strength of one anchor rod

$N_{sa} := 10.705kip$ — Per Appendix: ESR_1917 Table 3

$\phi_{steel\_tension} := 0.75$ — Per Appendix: ESR_1917 Table 3

$\phi N_{sa} := \phi_{steel\_tension} \cdot N_{sa} = 8.03 \cdot kip$

concrete breakout strength of anchor in tension

For the definition of varies, see above figure.

$c_{a1} := 3.5in$ bolt edge distance (assumed)

$c_{a2} := 8in$ bolt edge distance (assumed)

$h_{ef} := 3.25in$

$C_{ac1} := 7.25in$ critical distance per ESR-1917 Report, Table 3 for min. 6" thickness concrete slab

$C_{ac} := \min(1.5 \cdot h_{ef}, C_{ac1}) = 4.88 \cdot in$ critical distance

$A_{Nco} := (2 \cdot 1.5 \cdot h_{ef}) \cdot (2 \cdot 1.5 \cdot h_{ef}) = 95.06 \cdot in^2$

$A_{Nc} := \min((1.5 \cdot h_{ef} + \min(c_{a1}, 1.5 \cdot h_{ef}))) \cdot (\min(c_{a2}, 1.5 \cdot h_{ef}) + 1.5 \cdot h_{ef}) = 81.66 \cdot in^2$

$\psi_{edN} := \min\left(1, 0.7 + 0.3 \cdot \frac{\min(c_{a1}, c_{a2})}{1.5 \cdot h_{ef}}\right) = 0.92$ Modification factor for anchor bolt group edge effect in tension. ACI 318-14 Eq. 17.4.2.4

$\psi_{cN} := 1.0$ Per ESR-1917 table 3

$\psi_{cpN} := \min\left(1, \max\left(\frac{1.5 \cdot h_{ef}}{C_{ac}}, \frac{c_{a1}}{C_{ac}}\right)\right) = 1.00$ Modification factor for anchor bolt group in tension for post-installed anchor

ACI 38-14 Eq. 17.4.2.7a &7b

$k_{cr} := 17$ Per ESR 1917 Table 3

$\lambda := 1.0$ for normal weight concrete

$F_c := 2.5\text{ksi}$

$$N_{b1} := k_{cr} \cdot \lambda \cdot \left(\frac{F_c}{1\text{psi}}\right)^{0.5} \cdot \left(\frac{h_{ef}}{1\text{in}}\right)^{1.5} \cdot 1\text{lbf} = 4.98 \cdot \text{kip}$$

the basic concrete breakout strength of a single anchor in tension, ACI 318-14 Eq. 17.4.2.2a

$$N_{cb} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{edN} \cdot \psi_{cN} \cdot \psi_{cpN} \cdot N_{b1} = 3.92 \cdot \text{kip}$$

the nominal concrete breakout strength of anchor group in tension, ACI 318-14 Eq. 17.4.2.1a& 1b

$\phi_{co\_breakout} := 0.65$ Per ESR-1917 Table 3

$$\phi N_{cb} := N_{cb} \cdot \phi_{co\_breakout} = 2.55 \cdot \text{kip}$$

Concrete Pullout/bond Strength of anchor in tension

$N_{P\_fc} := 4.915\text{kip} = 4.92 \cdot \text{kip}$ Per ESR-1917 Table 3,

$\phi Pullout := N_{P\_fc} \cdot 0.65 = 3.19 \cdot \text{kip}$ Per ESR-1917 Table 3, 0.65 reduction

Steel Strength of anchor in shear

$\phi Vsa := 5.495kip \cdot 0.65 = 3.57 \cdot kip$

Per ESR-1917 Table 3, 0.65 reduction

Concrete breakout Strength of anchor in shear

Reference : ACI 318 -14 Chapter 17

$A_{vco} := 4.5 \cdot c_{a1}^{2} = 55.12 \cdot in^{2}$

ACI 318-14 Eq. 17.5.2.1c

$h_a := 6in = 6.00 \cdot in$

assumed the minimum depth of concrete slab

$C_{a1} := c_{a1} = 3.50 \cdot in$

$A_{vc} := \left(1.5 \cdot C_{a1} + \min\left(c_{a2}, 1.5 \cdot C_{a1}\right)\right) \cdot 1.5 \cdot C_{a1} = 55.13 \cdot in^{2}$

$$V_{b1} := 7 \cdot \left(\frac{\min\left(d_{anchor} \cdot 8, h_{ef}\right)}{d_{anchor}}\right)^{0.2} \cdot \left(\frac{d_{anchor}}{1in}\right)^{0.5} \cdot \left[\lambda \cdot \left(\frac{F_c}{1psi}\right)^{0.5} \cdot \left(\frac{C_{a1}}{1in}\right)^{1.5}\right] \cdot 1lbf = 2.36 \cdot kip$$

ACI 318-14 Eq. 17.5.2.2a

$$V_{b2} := 9\lambda \cdot \left(\frac{F_c}{1psi}\right)^{0.5} \cdot \left(\frac{C_{a1}}{1in}\right)^{1.5} \cdot (1lbf) = 2.95 \cdot kip$$

ACI 318-14 Eq. 17.5.2.2b

$V_b := \min\left(V_{b1}, V_{b2}\right) = 2.36 \cdot kip$

$e_V := 0.0in$

$$\psi_{ecV} := \frac{1}{1 + \frac{2 \cdot e_V}{3 \cdot C_{a1}}} = 1.00$$

Modification factor for anchor bolt group loaded eccentrically in shear
ACI 318-14 Eq. 17.5.2.5

$\psi_{edV} := 1.0$

Modification factor for anchor bolt group edge effect in shear
ACI 318-14 Eq. 17.5.2.6a for ca2> 1.5Ca1

$\psi_{cV} := 1.0$

Modification factor for anchor bolt group in shear for post-installed anchor
ACI 318-14 item 17.5.2.7

$\psi_{ch} := 1.0$

Modification factor for anchor bolt located in a concrete member where
ha < 1.5ca1, ACI 318-14 item 17.5.2.8

$$V_{cbg} := \frac{A_{vc}}{A_{vco}} \cdot \psi_{ecV} \cdot \psi_{edV} \cdot \psi_{cV} \cdot \psi_{ch} \cdot V_b = 2.36 \cdot kip$$

the nominal concrete breakout strength of anchor group in tension, ACI 318-14 Eq. 17.5.2.1a & 1b.

Concrete pry out Strength of anchor in shear

$K_{cp} := 2.0$

ESR-1917 Table 3

$\phi_{shear\_cr} := 0.70$

$$\phi V_{cpg} := \phi_{shear\_cr} \cdot K_{cp} \cdot N_{cb} = 5.48 \cdot kip$$

$$\phi N_b := \min\left(\phi N_{cb}, \phi N_{sa}, \phi Pullout\right) = 2.55 \cdot kip$$

$$\phi V_n := \min\left(\phi V_{cpg}, \phi Vsa, \phi_{shear\_cr} \cdot V_{cbg}\right) = 1.65 \cdot kip$$

$$ratio_{shear} := \frac{V_{anchor\_factored}}{\phi V_n} = 0.13$$

if Vu is less than 0.2 ϕV_n,,then full strength in tension shall be permitted. no need to check the interaction of tensile and shear forces

$$\frac{T_{anchor\_factored}}{\phi N_b} + \frac{V_{anchor\_factored}}{\phi V_n} = 1.09$$

Note: 1 anchor bolt is in tension and 1 anchor bolt is in shear

$$Tension_{anchor} := \begin{cases} \text{"OK !!"} & \text{if } T_{anchor\_factored} \le \phi N_b \\ \text{"NG !!"} & \text{otherwise} \end{cases}$$

$Tension_{anchor} = \text{"OK !!"}$

$$Shear_{anchor} := \begin{cases} \text{"OK !!"} & \text{if } V_{anchor\_factored} \le \phi V_n \\ \text{"NG !!"} & \text{otherwise} \end{cases}$$

$Shear_{anchor} = \text{"OK !!"}$

$$V\_T_{anchor\_check} := \begin{cases} \text{"OK !!"} & \text{if } \frac{T_{anchor\_factored}}{\phi N_b} + \frac{V_{anchor\_factored}}{\phi V_n} \le 1.2 \\ \text{"NG !!"} & \text{otherwise} \end{cases}$$

$V\_T_{anchor\_check} = \text{"OK !!"}$