Geotechnical Information

Per email by NEAS (Geotechnical Engineer) on 10/14/2022: a Seismic Site Class of D - Stiff Soil, is recommended.

D

Elastic seismic response parameters

Risk Category	=	1	
Soil classification (Class/type)	=	'D'	(Default/Stiff Soil)
PGA (Peak Ground Acceleration, Site Class ~~B~~)	=	0.059	(ASCE 7-16 Seismic Hazard Tool & ATC Hazard Tool)
F_{PGA} (Site Factor)	=	1.6	
A_s (Acceleration Coefficient)	=	0.094	

25% of Permanent D.L shall be considered for horizontal seismic force (LRFD 3.10.9.2)

Permitted by BDM S1003.11

AASHTO LRFD 9TH ED. TABLE 3.10.3.2-1 or from ASCE 7-16....

AASHTO LRFD 9TH ED. EQN 3.10.4.2-2

The bridge is located in Seismic Performance Zone 1 and has an acceleration coefficient, As, of 0.094 > 0.05, the horizontal connection force shall not be less than 0.25 times the vertical reaction due to the tributary permanent DC and DW loading under the Extreme I Limit State.

For Extreme I Limit State:
DC Load Factor = 1.0 per LRFD Table 3.4.1-1
LL Load Factor = 0.0 per BDM C303.1.4.1.b

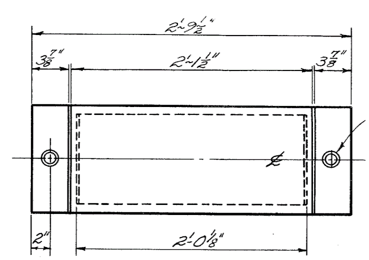
Total Unfactored Dead Loads, Kip (MDX Models)

	DC1	DC2	
	252.06	63.36	
	228.96	63.36	
	228.96	63.36	
	228.96	63.36	
	228.96	63.36	
	242.26	63.36	
	265.45	63.36	
Total DL (DC1 + DC2)	2119.13	kip	
# of Gir	7		(No. of beams)
DL per Gir	302.73286	kip	(Dead Load on each girder)
25% DL (per Gir)	75.683214	kip	(25% of DL for EQ forces)
# of Bolts per BRG	2		(Total # of bolts per BRG)
Horiz. Force per Bolt	37.842	kip	(Design Shear)

Scope: Check for minimum supports length for horizontal displacements at abutments and bearings strength at pier

Anchor Bolts Edge Distances & Spacing

BRG PL. Width	2.792	Ft	
Dist. b/w bolts	2.4583	Ft	(Bolt line dia)
Exis. Column Dia	3.3333	Ft	
Min. Edge Distance	5.2500	In	



VIEW J-J

FRA-71-19.36 under Cleveland Ave. (Pier Bearings Strength Check/Anchor Bolts)

These calculations provide the capacity of a single anchor or an anchor group according to AASHTO LRFD (9th Ed.) and ACI 318 Appendix D.

DEFINE CONSTANTS:

Grade 36 was selected as the material for the structural bolts.

Material

Min. Yield Strength $F_y := 36ksi$

Min. Tensile Strength $F_u := 58ksi$

Concrete $F_c := 4ksi$ $w_{conc} \equiv 0.15 \cdot \frac{kip}{ft^3}$

Resistance Factors [AASHTO LRFD Bridge Design 6.5.4.2]

for Flexure (Steel) $\phi_f := 1.0$

For ASTM F1554 bolts in Shear $\phi_s := 0.75$

LRFD 6.5.5 allows us to use a resistance factor of 1.0 for bolts used as anchor rods as long as the bots are F1554. F1554 did not exist the mid 90s and therefore postdates the 1958 construction. For this reason I've left the resistance factors alone, but let talk with Fred on this...get a second opinion.

GEOMETRY AND PROPERTIES

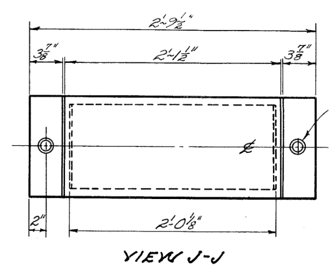
Nominal Anchor Bolt Diameter $d_{bolt} := 1.25in$

Nominal Anchor Bolt Area $A_{bolt} := \frac{\pi \, d_{bolt}^2}{4} = 1.23 \cdot in^2$

Anchor bolt bearing area $A_{bolt\_brg} := 1.817in^2$ Table A.2 (a), smallest bearing area used conservatively (Hex Head Bolt)

Number of Bolts per Bearing $N_{bolt} := 2$

Spacing of Bolts $S_{bolts} := 29.5in$



Check Minimum Spacing b/w Centers of Bolts (in Standard Holes) $S_{min} := 3 \cdot d_{bolt} = 3.75 \cdot in$

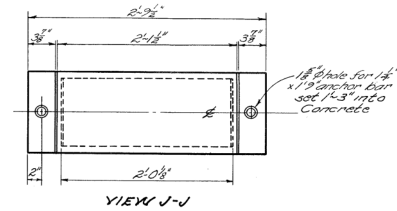
$$Check_{Spacing} := \begin{cases} \text{"OK"} & \text{if } S_{bolts} \geq S_{min} \\ \text{"NG, increase spacing"} & \text{otherwise} \end{cases}$$

$Check_{Spacing} = \text{"OK"}$

Clear Distance b/w Holes $L_c := S_{bolts} - \left(1 \cdot \frac{1}{16} in\right) = 29.44 \cdot in$

Edge Distance $d_{edge} := 5.25in$ As-built plans

Effective Embedment $h_{ef} := 15in$ As-built plans



DESIGN LOADS

Refer to the load calcs above for shear forces under seismic event.

Shear Force on Bolt due to seismic effects:

$Shear\_Load := 75.68kip$ Total shear force (for both bolts)

$V_u := \max(Shear\_Load) = 75.68 \cdot kip$ per bolt

Per bolt group/brg

ANCHOR BOLT CAPACITY

[AASHTO LRFD Bridge Design 14.8.3.1]

The shear resistance of anchor bolts shall be determined as specified in Article 6.13.2.12.

Shear Resistance

[AASHTO LRFD Bridge Design 6.13.2.12]

Calculation below is for shear resistance where threads are included in shear plane.

Factored Shear Resistance $R_r = \phi_s 0.50 \cdot A_{bolt} \cdot F_u \cdot N_s$

Design Shear Force $R_{shear} := V_u$ $R_{shear} = 75.68 \cdot kip$

No. of Shear Planes per Anchor Bolt $N_s := 1 N_{bolt}$

0.75 x 0.50 x Abolt x Fu x Ns

Nominal Shear Resistance $R_n := 0.48 \cdot A_{bolt} \cdot F_u \cdot N_s$ $R_n = 68.33 \cdot kip$

Shear Capacity Check $Check_s := \begin{cases} \text{"Design is OK"} & \text{if } \phi_s \cdot R_n > R_{shear} \\ \text{"Revise"} & \text{otherwise} \end{cases}$

$Check_s = \text{"Revise"}$

Change coding to "N.G"

CONCRETE CAPACITY

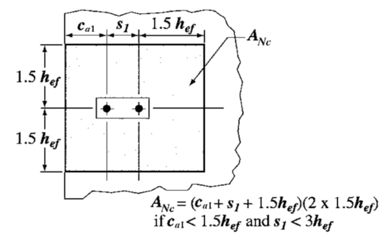
Concrete Breakout Strength in Tension (Calculated for concrete pryout strength): [ACI D.5.2.1]

$c_{a1} := d_{edge} = 5.25 \cdot in$ $\quad S_1 := S_{bolts} = 29.50 \cdot in$

$c_{a2} := c_{a1} = 5.25 \cdot in$

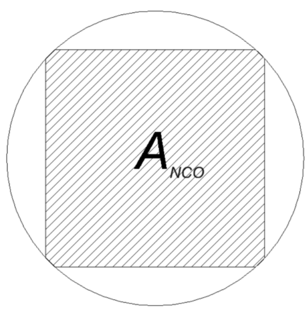
Cite: Per ACI 318-14, 17.4.2.3

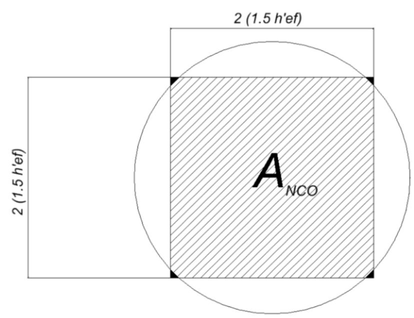
Note (RD. 5.2.3): Since three edge distances (i.e. 5.25 inches) are less than the 1.5 h_{ef} (i.e. 22.5 inches), h'_{ef} shall be calculated. h'_{ef} is taken largest of $c_{a,max}$/1.5 & 1/3 of bolts spacing. Therefore, (Bolts spacing/3 = 9.83") controls.



$$h'_{ef} := \max\left(\frac{c_{a1}}{1.5}, \frac{S_{bolts}}{3}\right) = 9.83 \cdot in$$

For A_{nco} & A_{nc}, use reduced embedment of anchor.

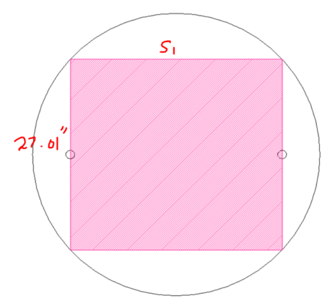




$A_{Nco} := 866.71 in^2$ $\quad N_{bolt} \cdot A_{Nco} = 1.73 \times 10^3 \cdot in^2$

(A_{nco} (for non ciruclar edges) = $9h_{ef}^2$ = 870 in². However, due to ciruclar edge constraints, the area is slightly reduced. Reduced (or effective) h'_{ef} shall be used for calculation of area.

$A_{Nc} := (S_1) \cdot (27.01 in)$ (27.01" is taken from CAD for given bolt spacing)



$A_{Nc} = 796.80 \cdot in^2$

$$check := \begin{cases} \text{"ok"} & \text{if } A_{Nc} \le N_{bolt} \cdot A_{Nco} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

check = "ok" $\quad A_{Nc} := \min(A_{Nc}, N_{bolt} \cdot A_{Nco}) = 796.80 \cdot in^2$

modification for eccentric load (D5.2.4)

$e_N := 0 in$ $\quad$ For single corner anchor in tension, e'_N shall be 0

$$\Psi_{ec,N} := \frac{1}{1 + \frac{2e_N}{3h_{ef}}} = 1.00 \qquad \Psi_{ec,N} := \min(\Psi_{ec,N}, 1.0) = 1.0$$

modification for edge effects (D5.2.5)

$$\Psi_{ed,N} := \begin{cases} 1 & \text{if } c_{a1} \geq 1.5h_{ef} \\ 0.7 + 0.3 \cdot \dfrac{c_{a1}}{1.5h_{ef}} & \text{if } c_{a1} < 1.5h_{ef} \end{cases} = 0.77$$

Per ACI 318-14, R17.4.2.3. Should he be h'ef here too?

Modification for cracked section (D5.2.6) $\Psi_{c,N} := 1.25$ for cast-in anchors

Modification for post-installed anchors (D5.2.6) $\Psi_{cp,N} := 1.0$ for cast-in anchors

Basic concrete breakout strength of a single anchor in tension in cracked concrete

$k_c := 24$ for cast-in-anchors

Note:
17 = Post-installed anchors
24 = Cast-in anchors

$$N_b := k_c \cdot \sqrt{\frac{F_c}{(psi)}} \cdot \left[\frac{h_{ef}}{(in)}\right]^{1.5} \cdot (lbf) = 88.18 \cdot kip$$

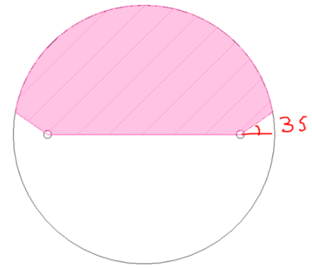
Nominal concrete breakout strength (D5.2)

$$N_{cbg} := \frac{A_{Nc}}{A_{Nco}} \cdot \left(\Psi_{ec,N} \cdot \Psi_{ed,N} \cdot \Psi_{c,N} \cdot \Psi_{cp,N} \cdot N_b\right) = 78.03 \cdot kip$$

Compare to Nua

Concrete Breakout Strength in Shear:

Assembly 1

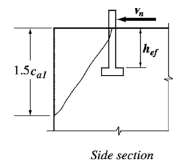


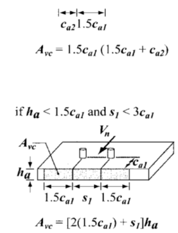
Projected Area for single anchor in a deep member

$$A_{Vco} := 4.5c_{a1}^{2} = 124.03 \cdot in^{2}$$

Projected Area of Failure Surface (at 35 deg cone) at its edge for a single or group of anchors.)

$$A_{Vc} := \left[S_1 + \left(\frac{c_{a1}}{\cos(35deg)}\right) \cdot 2\right] \cdot 1.5c_{a1} = 333.26 \cdot in^{2}$$





$$check := \begin{cases} \text{"ok"} & \text{if } A_{Vc} \le N_{bolt} \cdot A_{Vco} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$check = \text{"NG"}$$

$$A_{Vc} := \min\left(A_{Vc}, N_{bolt} \cdot A_{Vco}\right) = 248.06 \cdot in^{2}$$

Basic concrete breakout strength (cracked concrete)

$$V_b = 8 \cdot \left(\frac{l_e}{d_{bolt}}\right)^{0.2} \cdot \sqrt{\frac{d_{bolt}}{in}} \cdot \sqrt{\frac{F_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5} \cdot (lbf)$$

[ACI D.6.2.3]

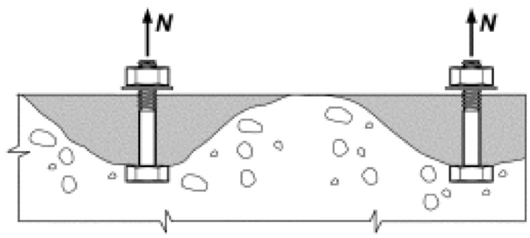
$d_{bolt} = \frac{5}{4} \cdot in \quad l_e := h_{ef} = 15.00 \cdot in$ D.6.2.2

$$V_b := 8 \cdot \left(\frac{l_e}{d_{bolt}}\right)^{0.2} \cdot \sqrt{\frac{d_{bolt}}{in}} \cdot \sqrt{\frac{F_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5} \cdot (lbf) = 11.19 \cdot kip$$

Modification for eccentricity (D6.2.5)

$$e_V := 0in$$

$$\Psi_{ec,V} := \frac{1}{1 + \frac{2 \cdot e_V}{3 \cdot c_{a1}}} = 1.00$$



(iii) Concrete breakout

Modification for edge effect (D6.2.6)

$$\Psi_{ed,V} := \begin{cases} 1 & \text{if } c_{a2} \geq 1.5c_{a1} \\ 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5c_{a1}} & \text{if } c_{a2} < 1.5c_{a1} \end{cases} = 0.90$$

$$\Psi_{ed,V} = 0.90$$

Modification for cracked concrete (D6.2.7)

$\Psi_{c,V} := 1.2$ for anchors in cracked concrete with supplementary reinforcement of a No. 4 bar or greater between the anchor and the edge

Nominal Concrete Breakout Strength

$$V_{cbg} := \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ec,V} \cdot \Psi_{ed,V} \cdot \Psi_{c,V} \cdot V_b = 24.16 \cdot kip$$ [ACI D.6.2.1]

Concrete Pryout Strength:

$$h_{ef} = 15.00 \cdot in$$ [ACI D.6.3.1]

$$k_{cp} := \begin{cases} 1.0 & \text{if } h_{ef} < 2.5in \\ 2.0 & \text{if } h_{ef} \geq 2.5in \end{cases} \qquad k_{cp} = 2.0$$

$$N_{cbg} = 7.80 \times 10^4 \cdot lbf$$

$$V_{cpg} := k_{cp} \cdot N_{cbg} = 1.56 \times 10^5 \cdot lbf$$

Factored Strength:

$$\phi V_n \geq V_u$$

$$\phi N_n \geq N_u$$

Note: Condition is determined from ACI 355.2

concrete breakout, side-face blowout, pullout or pryout strength

i) shear loads $\phi_{c,V} := 0.75$

ii) tension loads $\phi_{c,N} := 0.75$

Condition A is applied due to provision of supplemental reinf.

Shear:

Concrete breakout strength $\phi_{c,V} = 0.75$ $V_{cbg} = 24160 \cdot lbf$ $\phi_{c,V} \cdot V_{cbg} = 18 \cdot kip$

Concrete pryout strength $\phi_{c,V} = 0.75$ $V_{cpg} = 156056 \cdot lbf$ $\phi_{c,V} \cdot V_{cpg} = 117 \cdot kip$

FACTORED SHEAR CAPACITY $\boxed{\phi V_n := \min\left(\phi_{c,V} \cdot V_{cbg}, \phi_{c,V} \cdot V_{cpg}\right) = 18 \cdot kip}$

$$Check_2 := \begin{cases} \text{"OK"} & \text{if } \phi V_n > V_u \\ \text{"Not OK, Modify"} & \text{otherwise} \end{cases}$$

$Check_2 = \text{"Not OK, Modify"}$

Embedment Length $l_e = 15.00 \cdot in$

Minimum Support Length Requirements BDM 303.1.4.1.a & LRFD 9th Ed. 4.7.4.4 for seismic zone 1

Revise to read, Footing Thickness

L	=	266.68	FT	(Length b/w adjacent expansion joints, bridge length)				
H	=							
Column average heights								
Bot. of FTG elev.	=	793.9	FT	(Record Plans)				
Depth of FTG	=	3	FT	(Record Plans)				
Top of FTG elev.	=	790.9	FT					
Top of Col. 1 elev.	=	813.97	FT	(Record Plans)	Col. 1 Ht	=	23.07	FT
Top of Col. 2 elev.	=	814.23	FT	(Record Plans)	Col. 2 Ht	=	23.33	FT
Top of Col. 3 elev.	=	814.48	FT	(Record Plans)	Col. 3 Ht	=	23.58	FT
Top of Col. 4 elev.	=	814.72	FT	(Record Plans)	Col. 4 Ht	=	23.82	FT
Top of Col. 5 elev.	=	814.68	FT	(Record Plans)	Col. 5 Ht	=	23.78	FT
Top of Col. 6 elev.	=	814.63	FT	(Record Plans)	Col. 6 Ht	=	23.73	FT
Top of Col. 7 elev.	=	814.57	FT	(Record Plans)	Col. 7 Ht	=	23.67	FT
Average Ht. of a column	=	23.56857	FT					
Skew	=	55.9025	Deg					
N	=	21.16	In					

All the bearings are fixed. For the min support length at the abutments this should be 1/2 Span.

Should add the 3' footing thickness

.65

Per ODOT BDM 2020,C405.7

Thermal Movement

Per ODOT BDM 2020, S.1003.21

Expansion Coefficient, α	=	6.5E-06	1 / deg F	
T_{max}	=	120	F	
T_{min}	=	-30	F	
Construction Temp.	=	60	F	
Delt_T	=	90	F	
Span Expansion Length	=	770.64	In	
Thermal Force	=	0.45082	In	
Total Required Beam Seat	=	21.9684	In	
Available Beam Seat	=	36	In	(Record Plans)
Requirement	=	OK		

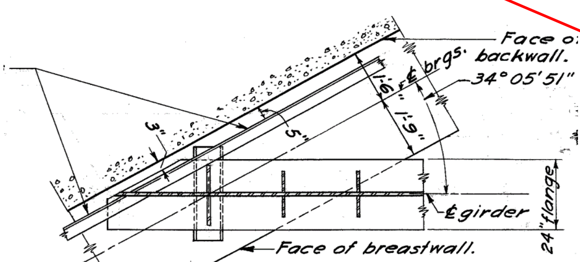
$$\Delta_T = \alpha L \left(T_{MaxDesign} - T_{MinDesign}\right) \quad (3.12.2.3\text{-}1)$$

where:

L = expansion length (in.)
α = coefficient of thermal expansion (in./in./°F)

LRFD 9th Ed. 3.12.2.3-1

Thermal Movement, DeltaT



TYPICAL SECTION G-G
AS INDICATED ON SHEET Nº 193.