FORT MILLER
THE FORT MILLER CO., INC.

P.O. Box 98
Schuylerville, N.Y. 12871-0098
(518) 695-5000
(518) 695-4970 FAX

PRECAST DESIGN RECOMMENDATIONS

Anchor Walls

Job #:	**25105**
Project:	**VT 110 BI5** **County of Orange, VT.**
Contractor:	**VAOT**
Date:	**November 15, 2023**
Design No:	

ATTACHED:

Shop drawings S1 thru S7

STATE OF VERMONT
PHILIP JUSTIN REED
NO. 135364
CIVIL
LICENSED
PROFESSIONAL ENGINEER

Accepted
BY: J. GRIFFIN
DATE: 11/21/2023
RESUBMIT: NO
RECEIVED: November 20, 2023
CKD BY: A. LEMIEUX
STATE OF VERMONT
AGENCY OF TRANSPORTATION
25105 - OFA - CALCS - 11-15-23.pdf

Designed by: Michael Santerre, PE
Prepared by: Michael Santerre, PE
Checked by: Philip J. Reed, PE

These calculations and drawings are stamped in accordance with the New York State Education Law and the Rules of the Board of Regents (including Section 29.3(b)(2) effective June 14, 1996 clarifying design responsibility relationships).
It is the responsibility of the primary design team or team of design professionals to provide "...all parameters which the design must satisfy;", "...to review and approve the design submitted by the delegatee for conformance with the established specifications and parameters..." and "to determine that the design prepared by the delegatee conforms to the overall project design and can be integrated into such design...". It is therefore incumbent on the owner and/or project engineer(s) to ensure that the design assumptions listed herein are in accordance with the contract specifications, contract plans and site conditions as they relate to the project. The professional engineer sealing these calculations and The Fort Miller Co., Inc. are not responsible for the site conditions, or the evaluation thereof, as they may relate to the product or the proper installation (Including rigging and handling) of the product.

Wingwall Design Assumptions

Soil:

Backfill Unit Weight:	γ_{BF} =	120.00	pcf	Subgrade Unit Weight:	γ_{SG} =	120.00	pcf
Backfill Int. Friction Angle	ϕ_{BF} =	34.00	deg.	Subgrade Int. Friction Angle	ϕ_{SG} =	34.00	deg.
Unit Weight of Water:		62.40	pcf	Resistance Factor Passive Earth	ϕ_{ep} =	0.50	
Backfill Submerged Unit Wgt.	γ_{sub} =	69.60	pcf				
Service Bearing Capacity:	q_n =	5.00	ksf (assumed)				
Resistance Factor for Bearing	ϕ_b =	0.45					
Resistance Factor for Sliding	ϕ_τ =	0.80					

Concrete/Reinforcement:

Concrete Unit Weight	w_c =	150.00	pcf	Steel Elastic Modulus	E_s =	29000	ksi
Concrete 28-day Strength	f'_c =	5,000	psi	Conc. Elastic Mod.	$E_c = 33000 w_c^{1.5}\sqrt{f'_c}$ =	4287	ksi
Steel Yield	f_y =	60,000	psi	Modular Ratio	$n = E_s / E_c$ =	6.765	
Max. Aggregate Size	a_g =	0.75	in.	Stress Block	β_1 =	0.800	
				Conc. Rupture Modulus fr = 0.20√f'$_c$ =		0.447	ksi

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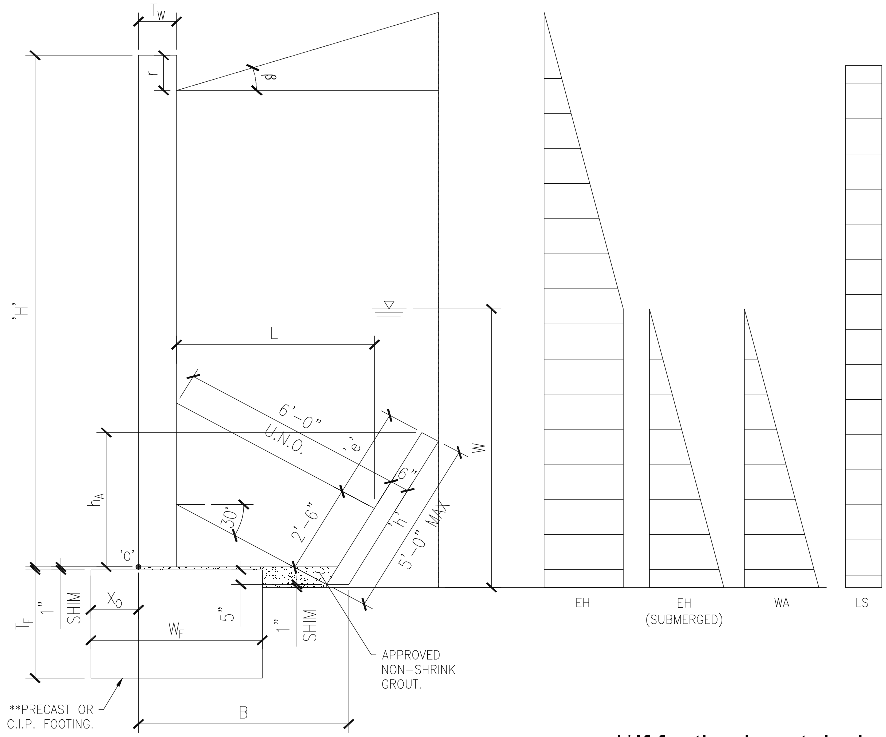

**If footing is not designed by FMC, T_F & W_F = 0

H =	13.350	ft	W_F =	0.000	ft
B =	6.678	ft	T_F =	0.000	ft
Individual Wall Segment =	10.000	ft	e =	2.500	ft
W =	0.000	ft	h =	5.000	ft (5'-0" max)
r =	0.500	ft	Xo =	0.750	ft
T_W =	1.000	ft	h_A =	3.830	ft
T_B =	0.000	ft	# anchors	2	per wall segment
$\beta =$	0.000	deg.	spacing	5.250	ft
$h_{eq} =$	2.000	ft	stem length	8.000	ft (6' min)
L =	6.928	ft	anchor width	4.750	ft

Wingwall Units will be designed as a segment with length shown above. Governing Unit: WW1

$$K_a = \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \right] \cos\beta = \quad 0.283$$

$$K_p = \left[\frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \right] \cos\beta = \quad 3.537$$

Load combinations/Load factors

	γ_{DC}	γ_{EV}	γ_{LS}	γ_{EH}	γ_{WA}
Service I	1.00	1.00	1.00	1.00	1.00
Strength Ia (min V, max H)	0.90	1.00	1.75	1.50	1.00
Strength Ib (max V, min H)	1.25	1.35	1.75	1.50	1.00

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Unfactored Vertical Forces/Moments about toe per wall segment:

	F_V (kips)	Arm (ft)	M_V (k-ft)
DC Wall (above water)	19.390	0.500	9.695
DC Wall (below water)	0.000	0.500	0.000
DC Anchor Stems	2.760	4.130	11.399
DC Anchor Faces	3.450	8.170	28.187
EV (above anchors)	75.580	5.080	383.946
EV (in anchor baskets)	30.180	4.580	138.224
LS (above anchor)	18.646	5.080	94.723
EH (uplift on soil anchor face)	0.000	7.928	0.000
EH (vertical component)	0.000	5.080	0.000
WA	0.000	5.080	0.000

Unfactored Horizontal Forces/Moments about toe per wall segment:

	F_H (kips)	Arm (ft)	M_H (k-ft)
EH (above water)	30.232	4.450	134.531
EH (below water, rectangular)	0.000	0.000	0.000
EH (below water, triangular)	0.000	0.000	0.000
LS	9.058	6.675	60.463
WA (triangular)	0.000	0.000	0.000

<u>Sliding Check</u> *

(Articles 11.6.3.6 & 10.6.3.4)

Strength I-a Limit State: Max. horizontal force, min. vertical resistance

$$R_R = \phi R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep}$$

for C.I.P. footings (grout poured between walll and anchor):

$$\phi_\tau R_\tau = \phi_\tau \left(\sum F_V\right) \tan\delta = (1.0)\phi_\tau \left(\sum F_V\right) \tan\phi_{SG}$$

$\tan\phi_{SG} =$ 0.675

Factored horizontal sliding forces:

	F_H (kips)
EH	45.348
LS	15.852
WA	0.000
$\sum F_H =$	**61.199**

Factored vertical resistances:

	F_V (kips)	
DC	23.040	
EV	105.760	
Net EH	0.000	
WA	0.000	
$(1.0)\phi_\tau \left(\sum F_V\right) \tan\phi_{SG} =$	**69.501**	**OK**

Key Height: (no key) 0 ft

Passive Resistance $\phi_{ep} R_{ep} =$ 0.000 kips

Total Resistance = **69.501** **OK**

(no key)

Sliding is realistically not an issue for the precast wall due to the grouted anchor hook behind the CIP footing preventing it from translating.

Overturn Check (Eccentricity)

(Article 11.6.3.3)

Strength I-a Limit State: Max. horizontal force, min. vertical resistance

Assumes only vertical earth above anchor face will contribute to overturning resistance.

Allowable eccentricity: $e_{max} = B/3 =$ 2.226 ft

Resultant location: $$X_0 = \frac{\sum M_V - \sum M_H}{\sum F_V}$$

Actual eccentricity: $$e_B = B/2 - X_0$$

Factored forces/moments:

	Vertical Forces F_V (kips)	Vertical Moments M_V (kip-ft)	Horizontal Moments M_H (kip-ft)
DC	23.040	44.352	
EV	105.760	522.171	
EH	0.000	0.000	201.797
LS	--	--	105.811
WA	0.000	0.000	0.000
	$\sum F_V =$ 128.800	$\sum M_V =$ 566.523	$\sum M_H =$ 307.608

$X_0 =$ 2.010 ft

$e_B =$ **1.329 ft OK**

Bearing Check on CIP Footing:

There is no moment connection to the CIP footing at the base of the wall. Thus, the bearing stress under the CIP footing due to the wingwall above it is soley due to the dead weight of the wall and the soil directly on top of the footing. The wingwall does not create an eccentricity in the bearing stress, and thus it is a rectangular distribution.

C.I.P. Footing width on drawings: **3.000 ft (MIN)**

Factored Vertical Bearing Forces:

DC_{wall}	**2.424**	**k/ft**
EV	**2.703**	**k/ft**
$DC_{footing}$	**1.125**	**k/ft**
	6.252	**k/ft**

Calculated bearing stress: **2.084 ksf**

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AASHTO LRFD Bridge Design Specifications - 8th Edition

Wingwall Stem Design

Member Geometry

Member Design Width b	12.00	in
Member Thickness t_s	12.00	in
Rebar clear cover	2.00	in
Design Bar Size	5	
A_b =	0.31	in^2
$d_e = t_s$-cover-$0.5d_b$ =	9.688	in

Unfactored Horizontal Forces/Moments above anchor at attachment location:

	F_H (kips)	Arm (ft)	M_H (k-ft)
EH (above water)	1.537	3.173	4.878
EH (below water, rectangular)	0.000	---	0.000
EH (below water, triangular)	0.000	---	0.000
LS	0.906	6.675	6.046
WA (triangular)	0.000	0.000	0.000

Controlling Forces: Strength Ib Limit State

Service Load Moment, M_a =	10.92	kip-ft
Design Ultimate Moment, M_u =	17.90	kip-ft
Design Ultimate Shear, V_u =	3.89	kips
Ultimate Axial Force, N_u =	24.24	kips (compressive)

Resistance Factors (Art. 5.5.4.2.1)

Flexure, φ_M = 0.9

Shear, φ_V = 0.9

Flexure

$$\rho = \left[1-\left(\sqrt{1-\frac{2M_u}{.85\phi b d^2 f_c'}}\right)\right]\bullet\frac{.85f_c'}{f_y} = 0.00362$$

A_s req'd = ρbd = 0.421 in^2/ft

Use #5's @ 6 in

A_s provided = **0.620 OK**

ρ provided = 0.00533

$$c = \frac{A_s f_y}{.85 f_c' \beta_1 b} = 0.912 \text{ in.}$$

$$c/d_e = 0.094 < 0.003/(0.003+e), \text{ OK (Art. 5.6.2.1)}$$

$$a = \beta_1 c = 0.729 \text{ in.} \qquad 1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} = 6.969 \text{ k-ft (Art. 5.6.3.3)}$$

$$I_g = \frac{bt_s^3}{12} = 1728 \text{ in}^4 \qquad \phi M_n = \phi A_s f_y (d_e - a/2) = \textbf{26.011 k-ft}$$

>Mu, OK

$$y_t = t_s - c = 11.088 \text{ in.}$$

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Wingwall Stem Design (cont'd)

Shear (Art. 5.7.3.4.2)

$d_v =$ 8.719 in. (Art. 5.8.2.9)

$s_x = \max(d_v, d_e - a/2) =$ 9.323 in.

$s_{xe} = s_x \dfrac{1.38}{a_g + 0.63} =$ 9.323 in., <12 in., use 12 in.

$\varepsilon_s = \dfrac{\dfrac{|M_u|}{d_v} - 0.5N_u + |V_u|}{E_s A_s} =$ 0.000913

$\beta = \dfrac{4.8}{(1+750\varepsilon_s)} \dfrac{51}{(39+s_{xe})} =$ 2.850

$\phi V_n = \phi V_c = \phi 0.0316 \beta \sqrt{f'_c} b_v d_v =$ **18.960 kips >Vu, OK**

Crack Control (Art 5.6.7)

$k = \sqrt{2\rho n + \rho n^2} - \rho n =$ 0.526

d_c= 2.31 in.

$j = 1 - \dfrac{k}{3} =$ 0.825

$\beta_s = 1 + \dfrac{d_c}{0.7(h - d_c)} =$ 1.341

Exposure γ_e = 0.75

$f_{ss} = \dfrac{M_a}{A_s jd} =$ 26.470 ksi

$s \le \dfrac{700\gamma_e}{\beta_s f_{ss}} - 2d_c =$ **10.165 in. OK**

Wall Minimum Steel (Art. 12.14.5.8)

(Stem horizontal reinforcement)

$A_s \ge \dfrac{1.3bh}{2(b+h)f_y} =$ 0.065 in^2

$(0.11 \le A_s \le 0.60)$

Minimum As req'd, ea. mat: 0.11 in^2

Use # 5 @ 18 in. max. spacing

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Wingwall Anchor Design

Member Geometry

Member Design Width b	6.00	in (thickness of Twall stem)
Member Thickness t_s	30.00	in (depth of Twall stem)
Rebar clear cover	2.00	in
Design Bar Size	6	
A_b =	0.44	in^2
$d_e = t_s$-cover-$0.5d_b$ =	27.625	in
Tributary width per anchor	5.000	ft

Loads for stem design at anchor location above are multiplied by tributary width:

Controlling Forces: Strength Ib Limit State

Service Load Moment, M_a =	54.62	kip-ft	Resistance Factors (Art. 5.5.4.2.1)	
Design Ultimate Moment, M_u =	89.49	kip-ft	Flexure, φ_M =	0.9

Flexure

$$\rho = \left[1-\left(\sqrt{1-\frac{2M_u}{.85\phi b d^2 f_c'}}\right)\right]\bullet\frac{.85 f_c'}{f_y} = \quad 0.00449$$

A_s req'd = ρbd = 0.743 in^2

Use #6's @ 6 in; i.e.: (2) #6 bars in stem

A_s provided = **0.880 OK**

ρ provided = 0.00531

$$c = \frac{A_s f_y}{.85 f_c' \beta_1 b} = \quad 2.588 \text{ in.}$$

$$c/d_e = \quad 0.094 \ <0.003/(0.003+e), \text{ OK (Art. 5.6.2.1)}$$

$$a = \beta_1 c = 2.071 \text{ in.} \qquad 1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} = \quad 22.025 \text{ k-ft (Art. 5.6.3.3)}$$

$$I_g = \frac{b t_s^3}{12} = 13500 \text{ in}^4 \qquad \phi M_n = \phi A_s f_y (d_e - a/2) = \quad \mathbf{105.295 \text{ k-ft}}$$

>Mu, OK

$$y_t = t_s - c = \quad 27.412 \text{ in.}$$

Crack Control (Art 5.6.7)

$$k = \sqrt{2\rho n + \rho n^2} - \rho n = \quad 0.525$$

d_c= 2.38 in.

$$j = 1 - \frac{k}{3} = \quad 0.825$$

$$\beta_s = 1 + \frac{d_c}{0.7(h-d_c)} = \quad 1.123$$

Exposure γ_e = 0.75

$$f_{ss} = \frac{M_a}{A_s j d} = \quad 32.685 \text{ ksi}$$

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c = \quad \mathbf{9.555 \text{ in. OK}}$$

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Development: (Art 5.10.8.2.4a)

Modification Factors:	M	
Epoxy Bar?	1.3	Yes
fy = 60	1.0	
Side Cover on hook	0.7	
Enclosed with stirrups?	1.0	No
As, req'd/As, prov	0.84	
Normal Weight Conc.	1.0	

$$l_{dh} = \frac{38.0 d_b}{\sqrt{f_c'}}(ModFactors) =$$ 9.799 in

Cover on front face	2.00	in	
ldh, prov =	10.000	in	**OK**

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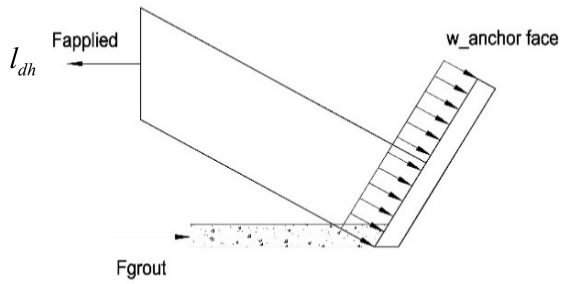
Anchor Design (continued)



FBD - ANCHOR

Force on Anchor face is conservatively taken as the entire lateral load on the wall distributed over the face area of the Anchor. Check bar in Anchor face for critical section where cantilevered extended face of Anchor meets the stem. Check horizontal bar in face.
Force in grout is conservatively taken as equal to entire lateral load on the wall. Check strength of grout.

Calculate force on rear of Anchor face:

As req'd from stem bar calculation: 0.743 in^2/ft

Factored Lateral Loads (EH and Surcharge Loads, where applicable): 61.199 kip
Anchor face height = 5.000 ft
To calculate load on wall, distribute load over face height, and to design per unit width, distribute over
Total Anchor width = 9.500 ft
per unit width of wall $w_{anchor-face} =$ 1.288 kip/ft
e = 2.500 ft

$$M_{critical\ section} = (w_{anchor-face} \quad anc\ \ or\ \ extension\text{^}2)/3$$

$M_{critical\ section} =$ 2.684 kip*ft

Member Geometry (design of vertical reinforcing in Anchor face)

Member Design Width b	12.00	in (per unit width of stem face)
Member Thickness t_s	6.00	in (thickness of Anchor face)
Rebar clear cover	2.38	in
Design Bar Size	5	
A_b =	0.31	in^2
$d_e = t_s$-cover-$0.5d_b$ =	3.313	in

Use M_critial section calculated above.

Anchor Design (continued)

Flexure

$$\rho = \left[1 - \left(\sqrt{1 - \frac{2M_u}{.85\phi b d^2 f_c'}}\right)\right] \bullet \frac{.85 f_c'}{f_y} = \quad 0.00469$$

A_s req'd = ρbd = 0.186 in^2

Use #5's @ 8 in

A_s provided = **0.460 OK**

ρ provided = 0.01158

$$c = \frac{A_s f_y}{.85 f_c' \beta_1 b} = \quad 0.677 \text{ in.}$$

$$c/d_e = \quad 0.204 \text{ <0.003/(0.003+e), OK (Art. 5.6.2.1)}$$

$$a = \beta_1 c = 0.541 \text{ in.} \quad 1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} = \quad 1.815 \text{ k-ft (Art. 5.6.3.3)}$$

$$I_g = \frac{b t_s^3}{12} = \quad 216 \text{ in}^4 \qquad \phi M_n = \phi A_s f_y (d_e - a/2) = \quad \textbf{6.299 k-ft}$$

>Mu, OK

$$y_t = t_s - c = \quad 5.323 \text{ in.}$$

Member Geometry (design of horizontal reinforcing in Anchor face)

Member Design Width b	12.00	in (per unit width of stem face)
Member Thickness t_s	6.00	in (thickness of Anchor face)
Rebar clear cover	2.00	in
Design Bar Size	5	
A_b =	0.31	in^2
$d_e = t_s$-cover-0.5d_b =	3.688	in

Take load on entire height of Anchor (5') and distribute over horizontal cantilever beam.

cantilever length: 1.625 ft

$w_{anchor-face}$ = 2.577 kip/ft

$$M_{critical\ section} = (w_{anchor-face} \quad cantilever\ lengt \ \hat{}2)/3$$

$$M_{critical\ section} = \quad 2.268 \text{ kip*ft}$$

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Flexure

$$\rho = \left[1 - \left(\sqrt{1 - \frac{2M_u}{.85\phi b d^2 f_c^{'}}}\right)\right] \bullet \frac{.85 f_c^{'}}{f_y} = \quad 0.00316$$

A_s req'd = ρbd = 0.140 in^2

Use #5's @ 8 in

A_s provided = **0.460 OK**

ρ provided = 0.01040

$$c = \frac{A_s f_y}{.85 f_c^{'} \beta_1 b} = \quad 0.677 \text{ in.}$$

$$c/d_e = \quad 0.184$$ <0.003/(0.003+e), OK (Art. 5.6.2.1)

$$a = \beta_1 c = 0.541 \text{ in.}$$

$$1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} = \quad 1.815 \text{ k-ft}$$ (Art. 5.6.3.3)

$$I_g = \frac{b t_s^3}{12} = \quad 216 \text{ in}^4$$

$$\phi M_n = \phi A_s f_y (d_e - a/2) = \quad 7.076 \text{ k-ft}$$

>Mu, OK

$$y_t = t_s - c = \quad 5.323 \text{ in.}$$

Wingwall Design Assumptions

Soil:

Backfill Unit Weight:	$\gamma_{BF} =$	120.00	pcf	Subgrade Unit Weight:	$\gamma_{SG} =$	120.00	pcf
Backfill Int. Friction Angle	$\phi_{BF} =$	34.00	deg.	Subgrade Int. Friction Angle	$\phi_{SG} =$	34.00	deg.
Unit Weight of Water:		62.40	pcf	Resistance Factor Passive Earth	$\phi_{ep} =$	0.50	
Backfill Submerged Unit Wgt.	$\gamma_{sub} =$	69.60	pcf				
Service Bearing Capacity:	$q_n =$	5.00	ksf (assumed)				
Resistance Factor for Bearing	$\phi_b =$	0.45					
Resistance Factor for Sliding	$\phi_\tau =$	0.80					

Concrete/Reinforcement:

Concrete Unit Weight	$w_c =$	150.00	pcf	Steel Elastic Modulus	$E_s =$	29000	ksi
Concrete 28-day Strength	$f'_c =$	5,000	psi	Conc. Elastic Mod.	$E_c = 33000 w_c^{1.5} \sqrt{f'_c} =$	4287	ksi
Steel Yield	$f_y =$	60,000	psi	Modular Ratio	$n = E_s / E_c =$	6.765	
Max. Aggregate Size	$a_g =$	0.75	in.	Stress Block	$\beta_1 =$	0.800	
				Conc. Rupture Modulus fr = 0.20√f'$_c$ =		0.447	ksi

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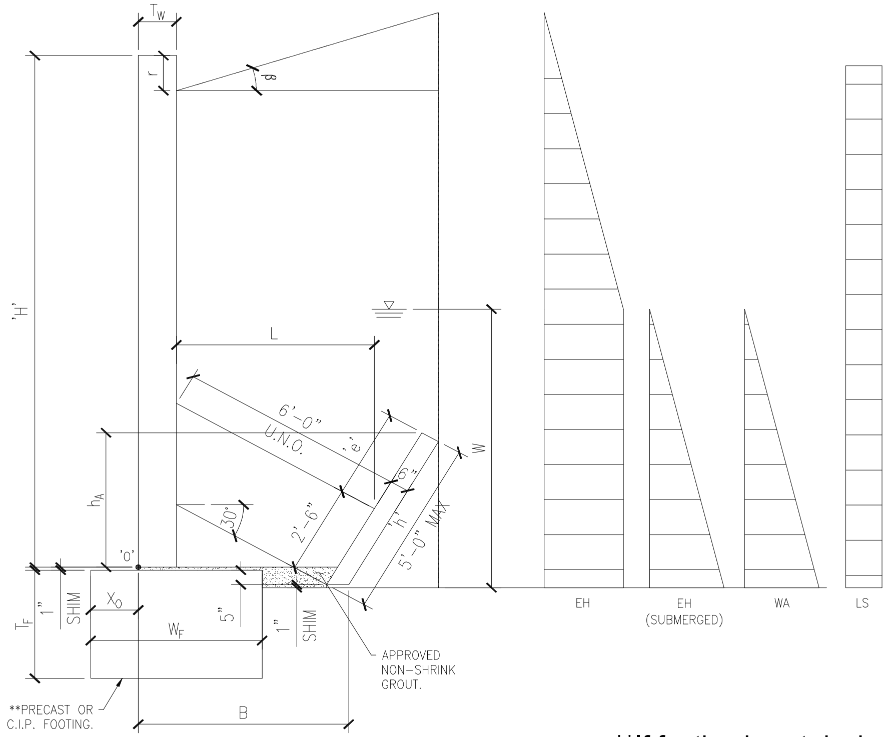

**If footing is not designed by FMC, T_F & W_F = 0

H =	11.660	ft	W_F =	0.000	ft
B =	6.678	ft	T_F =	0.000	ft
Individual Wall Segment =	5.000	ft	e =	2.500	ft
W =	0.000	ft	h =	5.000	ft (5'-0" max)
r =	0.500	ft	Xo =	0.750	ft
T_W =	1.000	ft	h_A =	3.830	ft
T_B =	0.000	ft	# anchors	1	per wall segment
$\beta =$	0.000	deg.	spacing	0.250	ft
$h_{eq} =$	2.000	ft	stem length	8.000	ft (6' min)
L =	6.928	ft	anchor width	4.750	ft

Wingwall Units will be designed as a segment with length shown above. Governing Unit: WW2

$$K_a = \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \right] \cos\beta = \quad 0.283$$

$$K_p = \left[\frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \right] \cos\beta = \quad 3.537$$

Load combinations/Load factors

	γ_{DC}	γ_{EV}	γ_{LS}	γ_{EH}	γ_{WA}
Service I	1.00	1.00	1.00	1.00	1.00
Strength Ia (min V, max H)	0.90	1.00	1.75	1.50	1.00
Strength Ib (max V, min H)	1.25	1.35	1.75	1.50	1.00

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Unfactored Vertical Forces/Moments about toe per wall segment:

	F_V (kips)	Arm (ft)	M_V (k-ft)
DC Wall (above water)	8.320	0.500	4.160
DC Wall (below water)	0.000	0.500	0.000
DC Anchor Stems	1.380	4.130	5.699
DC Anchor Faces	1.280	8.170	10.458
EV (above anchors)	31.660	5.080	160.833
EV (in anchor baskets)	13.720	4.580	62.838
LS (above anchor)	18.646	5.080	94.723
EH (uplift on soil anchor face)	0.000	7.928	0.000
EH (vertical component)	0.000	5.080	0.000
WA	0.000	5.080	0.000

Unfactored Horizontal Forces/Moments about toe per wall segment:

	F_H (kips)	Arm (ft)	M_H (k-ft)
EH (above water)	11.531	3.887	44.817
EH (below water, rectangular)	0.000	0.000	0.000
EH (below water, triangular)	0.000	0.000	0.000
LS	3.956	5.830	23.062
WA (triangular)	0.000	0.000	0.000

Sliding Check *

(Articles 11.6.3.6 & 10.6.3.4)

Strength I-a Limit State: Max. horizontal force, min. vertical resistance

$$R_R = \phi R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep}$$

for C.I.P. footings (grout poured between walll and anchor):

$$\phi_\tau R_\tau = \phi_\tau \left(\sum F_V\right)\tan\delta = (1.0)\phi_\tau \left(\sum F_V\right)\tan\phi_{SG}$$

$\tan\phi_{SG} =$ 0.675

Factored horizontal sliding forces:

	F_H (kips)
EH	17.297
LS	6.923
WA	0.000
$\sum F_H =$	**24.219**

Factored vertical resistances:

	F_V (kips)
DC	9.882
EV	45.380
Net EH	0.000
WA	0.000
$(1.0)\phi_\tau \left(\sum F_V\right)\tan\phi_{SG} =$	**29.820** **OK**

Key Height: (no key) 0 ft

Passive Resistance $\phi_{ep} R_{ep} =$ 0.000 kips

Total Resistance = **29.820** **OK**
(no key)

Sliding is realistically not an issue for the precast wall due to the grouted anchor hook behind the CIP footing preventing it from translating.

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Overturn Check (Eccentricity)

(Article 11.6.3.3)

Strength I-a Limit State: Max. horizontal force, min. vertical resistance

Assumes only vertical earth above anchor face will contribute to overturning resistance.

Allowable eccentricity: $e_{max} = B/3 =$ 2.226 ft

Resultant location: $X_0 = \dfrac{\sum M_V - \sum M_H}{\sum F_V}$

Actual eccentricity: $e_B = B/2 - X_0$

Factored forces/moments:

	Vertical Forces F_V (kips)	Vertical Moments M_V (kip-ft)	Horizontal Moments M_H (kip-ft)
DC	9.882	18.285	
EV	45.380	223.670	
EH	0.000	0.000	67.226
LS	--	--	40.359
WA	0.000	0.000	0.000
	$\sum F_V =$ 55.262	$\sum M_V =$ 241.956	$\sum M_H =$ 107.584

$X_0 =$ 2.432 ft

$e_B =$ **0.908 ft OK**

Bearing Check on CIP Footing:

There is no moment connection to the CIP footing at the base of the wall. Thus, the bearing stress under the CIP footing due to the wingwall above it is soley due to the dead weight of the wall and the soil directly on top of the footing. The wingwall does not create an eccentricity in the bearing stress, and thus it is a rectangular distribution.

C.I.P. Footing width on drawings: **3.000 ft (MIN)**

Factored Vertical Bearing Forces:

DC_{wall}	**2.080**	**k/ft**
EV	**2.361**	**k/ft**
$DC_{footing}$	**1.125**	**k/ft**
	5.566	**k/ft**

Calculated bearing stress: **1.855 ksf**

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AASHTO LRFD Bridge Design Specifications - 8th Edition

Wingwall Stem Design

Member Geometry

Member Design Width b	12.00	in
Member Thickness t_s	12.00	in
Rebar clear cover	2.00	in
Design Bar Size	5	
A_b =	0.31	in^2
$d_e = t_s$-cover-$0.5d_b$ =	9.688	in

Unfactored Horizontal Forces/Moments above anchor at attachment location:

	F_H (kips)	Arm (ft)	M_H (k-ft)
EH (above water)	1.040	2.610	2.714
EH (below water, rectangular)	0.000	---	0.000
EH (below water, triangular)	0.000	---	0.000
LS	0.791	5.830	4.612
WA (triangular)	0.000	0.000	0.000

Controlling Forces: Strength Ib Limit State

Service Load Moment, M_a =	7.33	kip-ft
Design Ultimate Moment, M_u =	12.14	kip-ft
Design Ultimate Shear, V_u =	2.94	kips
Ultimate Axial Force, N_u =	10.40	kips (compressive)

Resistance Factors (Art. 5.5.4.2.1)

Flexure, φ_M = 0.9

Shear, φ_V = 0.9

Flexure

$$\rho = \left[1-\left(\sqrt{1-\frac{2M_u}{.85\phi b d^2 f_c'}}\right)\right]\bullet\frac{.85 f_c'}{f_y} = \quad 0.00244$$

A_s req'd = ρbd = 0.283 in^2/ft

Use #5's @ 6 in

A_s provided = **0.620 OK**

ρ provided = 0.00533

$$c = \frac{A_s f_y}{.85 f_c' \beta_1 b} = 0.912 \text{ in.}$$

$$c/d_e = 0.094$$ <0.003/(0.003+e), OK (Art. 5.6.2.1)

$$a = \beta_1 c = 0.729 \text{ in.} \qquad 1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} = 6.969 \text{ k-ft (Art. 5.6.3.3)}$$

$$I_g = \frac{b t_s^3}{12} = 1728 \text{ in}^4 \qquad \phi M_n = \phi A_s f_y (d_e - a/2) =$$ **26.011 k-ft >Mu, OK**

$$y_t = t_s - c = 11.088 \text{ in.}$$

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Wingwall Stem Design (cont'd)

Shear (Art. 5.7.3.4.2)

$d_v =$ 8.719 in. (Art. 5.8.2.9)

$s_x = \max(d_v, d_e - a/2) =$ 9.323 in. $s_{xe} = s_x \dfrac{1.38}{a_g + 0.63} =$ 9.323 in., <12 in., use 12 in.

$$\varepsilon_s = \frac{\frac{|M_u|}{d_v} - 0.5N_u + |V_u|}{E_s A_s} =$$ 0.000804 $$\beta = \frac{4.8}{(1+750\varepsilon_s)} \frac{51}{(39+s_{xe})} =$$ 2.994

$\phi V_n = \phi V_c = \phi 0.0316\beta\sqrt{f'_c}\, b_v d_v =$ **19.922 kips >Vu, OK**

Crack Control (Art 5.6.7)

$k = \sqrt{2\rho n + \rho n^2} - \rho n =$ 0.526 d_c= 2.31 in.

$j = 1 - \dfrac{k}{3} =$ 0.825 $\beta_s = 1 + \dfrac{d_c}{0.7(h - d_c)} =$ 1.341

Exposure γ_e = 0.75

$f_{ss} = \dfrac{M_a}{A_s\, jd} =$ 17.752 ksi $s \le \dfrac{700\gamma_e}{\beta_s f_{ss}} - 2d_c =$ **17.428 in. OK**

Wall Minimum Steel (Art. 12.14.5.8)

(Stem horizontal reinforcement)

$A_s \ge \dfrac{1.3bh}{2(b+h)f_y} =$ 0.065 in^2

$(0.11 \le A_s \le 0.60)$

Minimum As req'd, ea. mat: 0.11 in^2

Use # 5 @ 18 in. max. spacing

Wingwall Anchor Design

Member Geometry

Member Design Width b	6.00	in (thickness of Twall stem)
Member Thickness t_s	30.00	in (depth of Twall stem)
Rebar clear cover	2.00	in
Design Bar Size	6	
A_b =	0.44	in^2
$d_e = t_s$-cover-$0.5d_b$ =	27.625	in
Tributary width per anchor	2.500	ft

Loads for stem design at anchor location above are multiplied by tributary width:

Controlling Forces: Strength Ib Limit State

Service Load Moment, M_a =	18.32	kip-ft	Resistance Factors (Art. 5.5.4.2.1)
Design Ultimate Moment, M_u =	30.36	kip-ft	Flexure, φ_M = 0.9

Flexure

$$\rho = \left[1-\left(\sqrt{1-\frac{2M_u}{.85\phi bd^2 f_c'}}\right)\right]\bullet\frac{.85f_c'}{f_y} = \quad 0.00149$$

A_s req'd = ρbd = 0.247 in^2

Use #6's @ 6 in; i.e.: (2) #6 bars in stem

A_s provided = **0.880 OK**

ρ provided = 0.00531

$$c = \frac{A_s f_y}{.85 f_c' \beta_1 b} = \quad 2.588 \text{ in.}$$

$$c/d_e = \quad 0.094 < 0.003/(0.003+e), \text{ OK (Art. 5.6.2.1)}$$

$$a = \beta_1 c = 2.071 \text{ in.} \qquad 1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} = \quad 22.025 \text{ k-ft (Art. 5.6.3.3)}$$

$$I_g = \frac{bt_s^3}{12} = 13500 \text{ in}^4 \qquad \phi M_n = \phi A_s f_y (d_e - a/2) = \quad \mathbf{105.295 \text{ k-ft}}$$

>Mu, OK

$$y_t = t_s - c = \quad 27.412 \text{ in.}$$

Crack Control (Art 5.6.7)

$$k = \sqrt{2\rho n + \rho n^2} - \rho n = \quad 0.525$$

d_c= 2.38 in.

$$j = 1 - \frac{k}{3} = \quad 0.825$$

$$\beta_s = 1 + \frac{d_c}{0.7(h-d_c)} = \quad 1.123$$

Exposure γ_e = 0.75

$$f_{ss} = \frac{M_a}{A_s jd} = \quad 10.960 \text{ ksi}$$

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c = \quad \mathbf{37.912 \text{ in. OK}}$$

Development: (Art 5.10.8.2.4a)

Modification Factors:	M	
Epoxy Bar?	1.3	Yes
fy = 60	1.0	
Side Cover on hook	0.7	
Enclosed with stirrups?	1.0	No
As, req'd/As, prov	0.28	
Normal Weight Conc.	1.0	

$$l_{dh} = \frac{38.0 d_b}{\sqrt{f_c'}}(ModFactors) = \quad 3.253 \text{ in}$$

Cover on front face 2.00 in

ldh, prov = 10.000 in **OK**

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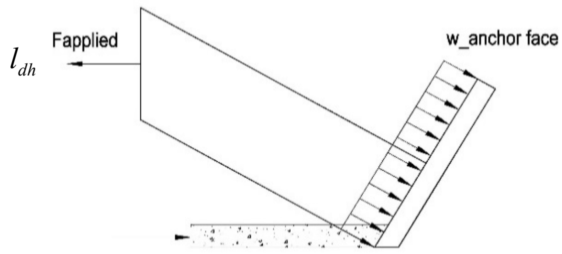
Anchor Design (continued)



FBD - ANCHOR

Force on Anchor face is conservatively taken as the entire lateral load on the wall distributed over the face area of the Anchor. Check bar in Anchor face for critical section where cantilevered extended face of Anchor meets the stem. Check horizontal bar in face.
Force in grout is conservatively taken as equal to entire lateral load on the wall. Check strength of grout.

Calculate force on rear of Anchor face:

As req'd from stem bar calculation: 0.247 in^2/ft

Factored Lateral Loads (EH and Surcharge Loads, where applicable): 24.219 kip
Anchor face height = 5.000 ft
To calculate load on wall, distribute load over face height, and to design per unit width, distribute over
Total Anchor width = 4.750 ft
per unit width of wall $w_{anchor-face}$ = 1.020 kip/ft
e = 2.500 ft

$$M_{critical\ section} = (w_{anchor-face}\quad anc\ \ or\ \ extension\text{^}2)/3$$

$M_{critical\ section}$ = 2.124 kip*ft

Member Geometry (design of vertical reinforcing in Anchor face)

Member Design Width b	12.00	in (per unit width of stem face)
Member Thickness t_s	6.00	in (thickness of Anchor face)
Rebar clear cover	2.38	in
Design Bar Size	5	
A_b =	0.31	in^2
$d_e = t_s$-cover-$0.5d_b$ =	3.313	in

Use M_critial section calculated above.

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Anchor Design (continued)

Flexure

$$\rho = \left[1-\left(\sqrt{1-\frac{2M_u}{.85\phi b d^2 f_c'}}\right)\right]\bullet\frac{.85 f_c'}{f_y} =$$ 0.00368

A_s req'd = ρbd = 0.146 in^2

Use #5's @ 8 in

A_s provided = **0.460 OK**

ρ provided = 0.01158

$$c = \frac{A_s f_y}{.85 f_c' \beta_1 b} =$$ 0.677 in.

$$c/d_e =$$ 0.204 <0.003/(0.003+e), OK (Art. 5.6.2.1)

$$a = \beta_1 c =$$ 0.541 in. $$1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} =$$ 1.815 k-ft (Art. 5.6.3.3)

$$I_g = \frac{bt_s^3}{12} =$$ 216 in^4 $$\phi M_n = \phi A_s f_y (d_e - a/2) =$$ **6.299 k-ft**

>Mu, OK

$$y_t = t_s - c =$$ 5.323 in.

Member Geometry (design of horizontal reinforcing in Anchor face)

Member Design Width b 12.00 in (per unit width of stem face)

Member Thickness t_s 6.00 in (thickness of Anchor face)

Rebar clear cover 2.00 in

Design Bar Size 5

A_b = 0.31 in^2

$d_e = t_s$-cover-0.5d_b = 3.688 in

Take load on entire height of Anchor (5') and distribute over horizontal cantilever beam.

cantilever length: 1.625 ft

$w_{anchor-face}$ = 2.039 kip/ft

$$M_{critical\ section} = (w_{anchor-face} \quad cantilever\ lengt \ \hat{}2)/3$$

$M_{critical\ section}$ = 1.795 kip*ft

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Flexure

$$\rho = \left[1 - \left(\sqrt{1 - \frac{2M_u}{.85\phi b d^2 f_c'}}\right)\right] \bullet \frac{.85 f_c'}{f_y} = \quad 0.00249$$

A_s req'd = ρbd = 0.110 in^2

Use #5's @ 8 in

A_s provided = **0.460 OK**

ρ provided = 0.01040

$$c = \frac{A_s f_y}{.85 f_c' \beta_1 b} = \quad 0.677 \text{ in.}$$

$$c/d_e = \quad 0.184 < 0.003/(0.003+e), \text{ OK (Art. 5.6.2.1)}$$

$$a = \beta_1 c = \quad 0.541 \text{ in.} \qquad 1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} = \quad 1.815 \text{ k-ft (Art. 5.6.3.3)}$$

$$I_g = \frac{bt_s^3}{12} = \quad 216 \text{ in}^4 \qquad \phi M_n = \phi A_s f_y (d_e - a/2) = \quad \mathbf{7.076 \text{ k-ft}}$$

>Mu, OK

$$y_t = t_s - c = \quad 5.323 \text{ in.}$$

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Wingwall Design Assumptions

Soil:

Backfill Unit Weight:	$\gamma_{BF} =$	120.00	pcf	Subgrade Unit Weight:	$\gamma_{SG} =$	120.00	pcf
Backfill Int. Friction Angle	$\phi_{BF} =$	34.00	deg.	Subgrade Int. Friction Angle	$\phi_{SG} =$	34.00	deg.
Unit Weight of Water:		62.40	pcf	Resistance Factor Passive Earth	$\phi_{ep} =$	0.50	
Backfill Submerged Unit Wgt.	$\gamma_{sub} =$	69.60	pcf				
Service Bearing Capacity:	$q_n =$	5.00	ksf (assumed)				
Resistance Factor for Bearing	$\phi_b =$	0.45					
Resistance Factor for Sliding	$\phi_\tau =$	0.80					

Concrete/Reinforcement:

Concrete Unit Weight	$w_c =$	150.00	pcf	Steel Elastic Modulus	$E_s =$	29000	ksi
Concrete 28-day Strength	$f_c' =$	5,000	psi	Conc. Elastic Mod.	$E_c = 33000 w_c^{1.5}\sqrt{f_c'} =$	4287	ksi
Steel Yield	$f_y =$	60,000	psi	Modular Ratio	$n = E_s / E_c =$	6.765	
Max. Aggregate Size	$a_g =$	0.75	in.	Stress Block	$\beta_1 =$	0.800	
				Conc. Rupture Modulus fr = $0.20\sqrt{f'_c}$ =		0.447	ksi

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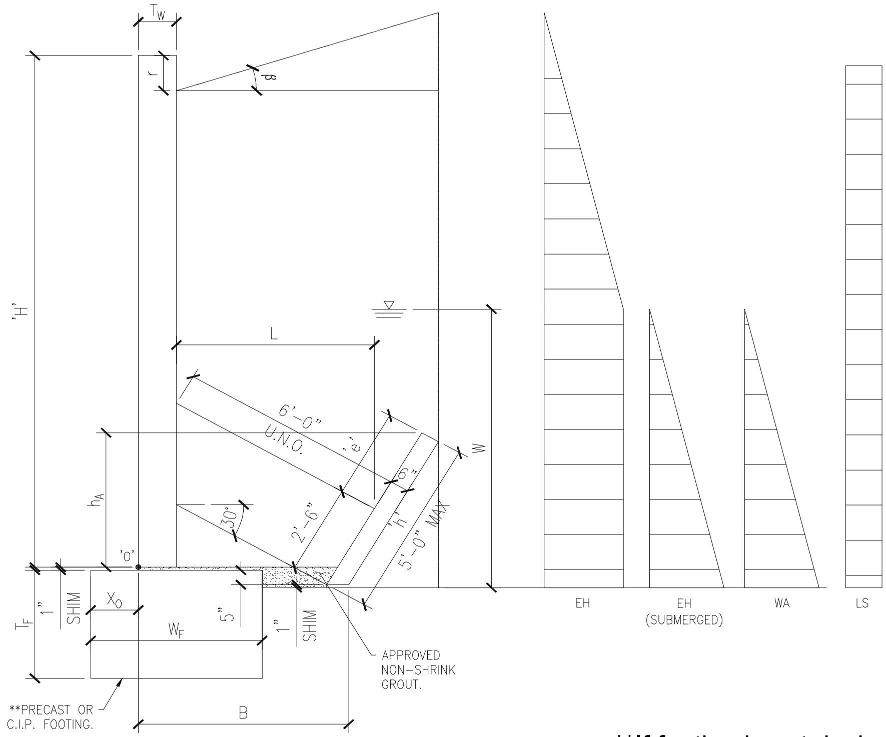

**If footing is not designed by FMC, T_F & W_F = 0

H =	11.660 ft	W_F =	0.000 ft
B =	6.678 ft	T_F =	0.000 ft
Individual Wall Segment =	11.000 ft	e =	2.500 ft
W =	0.000 ft	h =	5.000 ft (5'-0" max)
r =	0.500 ft	Xo =	0.750 ft
T_W =	1.000 ft	h_A =	3.830 ft
T_B =	0.000 ft	# anchors	2 per wall segment
$\beta =$	0.000 deg.	spacing	6.250 ft
$h_{eq} =$	2.000 ft	stem length	8.000 ft (6' min)
L =	6.928 ft	anchor width	4.750 ft

Wingwall Units will be designed as a segment with length shown above. Governing Unit: WW3

$$K_a = \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}\right]\cos\beta = \quad 0.283$$

$$K_p = \left[\frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}\right]\cos\beta = \quad 3.537$$

Load combinations/Load factors

	γ_{DC}	γ_{EV}	γ_{LS}	γ_{EH}	γ_{WA}
Service I	1.00	1.00	1.00	1.00	1.00
Strength Ia (min V, max H)	0.90	1.00	1.75	1.50	1.00
Strength Ib (max V, min H)	1.25	1.35	1.75	1.50	1.00

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Unfactored Vertical Forces/Moments about toe per wall segment:

	F_V (kips)	Arm (ft)	M_V (k-ft)
DC Wall (above water)	15.940	0.500	7.970
DC Wall (below water)	0.000	0.500	0.000
DC Anchor Stems	2.760	4.130	11.399
DC Anchor Faces	3.320	8.170	27.124
EV (above anchors)	50.060	5.080	254.305
EV (in anchor baskets)	28.910	4.580	132.408
LS (above anchor)	18.646	5.080	94.723
EH (uplift on soil anchor face)	0.000	7.928	0.000
EH (vertical component)	0.000	5.080	0.000
WA	0.000	5.080	0.000

Unfactored Horizontal Forces/Moments about toe per wall segment:

	F_H (kips)	Arm (ft)	M_H (k-ft)
EH (above water)	25.368	3.887	98.598
EH (below water, rectangular)	0.000	0.000	0.000
EH (below water, triangular)	0.000	0.000	0.000
LS	8.703	5.830	50.736
WA (triangular)	0.000	0.000	0.000

<u>Sliding Check</u>

(Articles 11.6.3.6 & 10.6.3.4)

Strength I-a Limit State: Max. horizontal force, min. vertical resistance

$$R_R = \phi R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep}$$

for C.I.P. footings (grout poured between walll and anchor):

$$\phi_\tau R_\tau = \phi_\tau \left(\sum F_V\right)\tan\delta = (1.0)\phi_\tau \left(\sum F_V\right)\tan\phi_{SG}$$

$\tan\phi_{SG} =$ 0.675

Factored horizontal sliding forces:

	F_H (kips)
EH	38.052
LS	15.230
WA	0.000
$\sum F_H =$	**53.282**

Factored vertical resistances:

	F_V (kips)
DC	19.818
EV	78.970
Net EH	0.000
WA	0.000
$(1.0)\phi_\tau\left(\sum F_V\right)\tan\phi_{SG} =$	**53.307** **OK**

Key Height: (no key) 0 ft

Passive Resistance $\phi_{ep} R_{ep} =$ 0.000 kips

Total Resistance = **53.307** **OK**

(no key)

Sliding is realistically not an issue for the precast wall due to the grouted anchor hook behind the CIP footing preventing it from translating.

<u>Overturn Check (Eccentricity)</u>
(Article 11.6.3.3)
Strength I-a Limit State: Max. horizontal force, min. vertical resistance
Assumes only vertical earth above anchor face will contribute to overturning resistance.

Allowable eccentricity: $e_{max} = B/3 =$ 2.226 ft

Resultant location: $X_0 = \dfrac{\sum M_V - \sum M_H}{\sum F_V}$

Actual eccentricity: $e_B = B/2 - X_0$

Factored forces/moments:

	Vertical Forces F_V (kips)	Vertical Moments M_V (kip-ft)	Horizontal Moments M_H (kip-ft)
DC	19.818	41.844	
EV	78.970	386.713	
EH	0.000	0.000	147.897
LS	--	--	88.789
WA	0.000	0.000	0.000
	$\sum F_V =$ 98.788	$\sum M_V =$ 428.556	$\sum M_H =$ 236.685

$X_0 =$ 1.942 ft
$e_B =$ **1.397 ft OK**

<u>Bearing Check on CIP Footing:</u>

There is no moment connection to the CIP footing at the base of the wall. Thus, the bearing stress under the CIP footing due to the wingwall above it is soley due to the dead weight of the wall and the soil directly on top of the footing. The wingwall does not create an eccentricity in the bearing stress, and thus it is a rectangular distribution.

C.I.P. Footing width on drawings: **3.000 ft (MIN)**

<u>Factored Vertical Bearing Forces:</u>

DC_{wall}	**1.811**	**k/ft**
EV	**2.361**	**k/ft**
$DC_{footing}$	**1.125**	**k/ft**
	5.298	**k/ft**

Calculated bearing stress: **1.766 ksf**

AASHTO LRFD Bridge Design Specifications - 8th Edition

Wingwall Stem Design

Member Geometry

Member Design Width b	12.00	in
Member Thickness t_s	12.00	in
Rebar clear cover	2.00	in
Design Bar Size	5	
A_b =	0.31	in^2
$d_e = t_s$-cover-$0.5d_b$ =	9.688	in

Unfactored Horizontal Forces/Moments above anchor at attachment location:

	F_H (kips)	Arm (ft)	M_H (k-ft)
EH (above water)	1.040	2.610	2.714
EH (below water, rectangular)	0.000	---	0.000
EH (below water, triangular)	0.000	---	0.000
LS	0.791	5.830	4.612
WA (triangular)	0.000	0.000	0.000

Controlling Forces: Strength Ib Limit State

Service Load Moment, M_a =	7.33	kip-ft	Resistance Factors (Art. 5.5.4.2.1)	
Design Ultimate Moment, M_u =	12.14	kip-ft	Flexure, φ_M =	0.9
Design Ultimate Shear, V_u =	2.94	kips	Shear, φ_V =	0.9
Ultimate Axial Force, N_u =	19.93	kips (compressive)		

Flexure

$$\rho = \left[1-\left(\sqrt{1-\frac{2M_u}{.85\phi b d^2 f'_c}}\right)\right] \bullet \frac{.85 f'_c}{f_y} = \quad 0.00244$$

A_s req'd = ρbd = 0.283 in^2/ft

Use #5's @ 6 in

A_s provided = **0.620 OK**

ρ provided = 0.00533

$$c = \frac{A_s f_y}{.85 f'_c \beta_1 b} = \quad 0.912 \text{ in.}$$

$$c/d_e = \quad 0.094 <0.003/(0.003+e), \text{ OK (Art. 5.6.2.1)}$$

$$a = \beta_1 c = 0.729 \text{ in.} \qquad 1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} = \quad 6.969 \text{ k-ft (Art. 5.6.3.3)}$$

$$I_g = \frac{b t_s^3}{12} = 1728 \text{ in}^4 \qquad \phi M_n = \phi A_s f_y (d_e - a/2) = \quad \textbf{26.011 k-ft}$$

>Mu, OK

$$y_t = t_s - c = \quad 11.088 \text{ in.}$$

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Wingwall Stem Design (cont'd)

Shear (Art. 5.7.3.4.2)

$d_v =$ 8.719 in. (Art. 5.8.2.9)

$s_x = \max(d_v, d_e - a/2) =$ 9.323 in. $s_{xe} = s_x \dfrac{1.38}{a_g + 0.63} =$ 9.323 in., <12 in., use 12 in.

$$\varepsilon_s = \frac{\dfrac{|M_u|}{d_v} - 0.5N_u + |V_u|}{E_s A_s} = \quad 0.000539 \qquad \beta = \frac{4.8}{(1+750\varepsilon_s)}\frac{51}{(39+s_{xe})} = \quad 3.418$$

$\phi V_n = \phi V_c = \phi 0.0316\beta\sqrt{f'_c}\,b_v d_v =$ **22.740 kips >Vu, OK**

Crack Control (Art 5.6.7)

$k = \sqrt{2\rho n + \rho n^2} - \rho n =$ 0.526 d_c= 2.31 in.

$j = 1 - \dfrac{k}{3} =$ 0.825 $\beta_s = 1 + \dfrac{d_c}{0.7(h - d_c)} =$ 1.341

Exposure γ_e = 0.75

$f_{ss} = \dfrac{M_a}{A_s\, jd} =$ 17.752 ksi $s \le \dfrac{700\gamma_e}{\beta_s f_{ss}} - 2d_c =$ **17.428 in. OK**

Wall Minimum Steel (Art. 12.14.5.8)

(Stem horizontal reinforcement)

$A_s \ge \dfrac{1.3bh}{2(b+h)f_y} =$ 0.065 in^2

$(0.11 \le A_s \le 0.60)$

Minimum As req'd, ea. mat: 0.11 in^2

Use # 5 @ 18 in. max. spacing

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Wingwall Anchor Design

Member Geometry

Member Design Width b	6.00	in (thickness of Twall stem)
Member Thickness t_s	30.00	in (depth of Twall stem)
Rebar clear cover	2.00	in
Design Bar Size	6	
A_b =	0.44	in^2
$d_e = t_s$-cover-$0.5d_b$ =	27.625	in
Tributary width per anchor	5.500	ft

Loads for stem design at anchor location above are multiplied by tributary width:

Controlling Forces: Strength Ib Limit State

Service Load Moment, M_a =	40.30	kip-ft
Design Ultimate Moment, M_u =	66.79	kip-ft

Resistance Factors (Art. 5.5.4.2.1)

Flexure, φ_M = 0.9

Flexure

$$\rho = \left[1 - \left(\sqrt{1 - \frac{2M_u}{.85\phi b d^2 f_c'}}\right)\right] \bullet \frac{.85 f_c'}{f_y} = \quad 0.00332$$

A_s req'd = ρbd = 0.550 in^2

Use #6's @ 6 in; i.e.: (2) #6 bars in stem

A_s provided = **0.880 OK**

ρ provided = 0.00531

$$c = \frac{A_s f_y}{.85 f_c' \beta_1 b} = \quad 2.588 \text{ in.}$$

$$c/d_e = \quad 0.094 \ \text{<0.003/(0.003+e), OK (Art. 5.6.2.1)}$$

$$a = \beta_1 c = 2.071 \text{ in.} \quad 1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} = \quad 22.025 \text{ k-ft (Art. 5.6.3.3)}$$

$$I_g = \frac{b t_s^3}{12} = 13500 \text{ in}^4 \qquad \phi M_n = \phi A_s f_y (d_e - a/2) = \quad \mathbf{105.295 \text{ k-ft}}$$

>Mu, OK

$$y_t = t_s - c = \quad 27.412 \text{ in.}$$

Crack Control (Art 5.6.7)

$$k = \sqrt{2\rho n + \rho n^2} - \rho n = \quad 0.525$$

d_c= 2.38 in.

$$j = 1 - \frac{k}{3} = \quad 0.825$$

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} = \quad 1.123$$

Exposure γ_e = 0.75

$$f_{ss} = \frac{M_a}{A_s j d} = \quad 24.112 \text{ ksi}$$

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c = \quad \mathbf{14.642 \text{ in. OK}}$$

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Development: (Art 5.10.8.2.4a)

Modification Factors:	M	
Epoxy Bar?	1.3	Yes
fy = 60	1.0	
Side Cover on hook	0.7	
Enclosed with stirrups?	1.0	No
As, req'd/As, prov	0.63	
Normal Weight Conc.	1.0	

$$l_{dh} = \frac{38.0 d_b}{\sqrt{f_c'}}(ModFactors) = \quad 7.251 \text{ in}$$

Cover on front face 2.00 in

ldh, prov = 10.000 in **OK**

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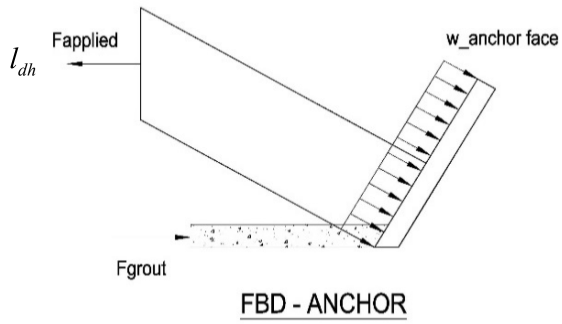
Anchor Design (continued)



Force on Anchor face is conservatively taken as the entire lateral load on the wall distributed over the face area of the Anchor. Check bar in Anchor face for critical section where cantilevered extended face of Anchor meets the stem. Check horizontal bar in face.
Force in grout is conservatively taken as equal to entire lateral load on the wall. Check strength of grout.

Calculate force on rear of Anchor face:

As req'd from stem bar calculation: 0.550 in^2/ft

Factored Lateral Loads (EH and Surcharge Loads, where applicable): 53.282 kip
Anchor face height = 5.000 ft
To calculate load on wall, distribute load over face height, and to design per unit width, distribute over
Total Anchor width = 9.500 ft
per unit width of wall $w_{anchor-face}$ = 1.122 kip/ft
e = 2.500 ft

$$M_{critical\ section} = (w_{anchor-face} \quad anc \quad or \quad extension\text{^}2)/3$$

$M_{critical\ section}$ = 2.337 kip*ft

Member Geometry (design of vertical reinforcing in Anchor face)

Member Design Width b	12.00	in (per unit width of stem face)
Member Thickness t_s	6.00	in (thickness of Anchor face)
Rebar clear cover	2.38	in
Design Bar Size	5	
A_b =	0.31	in^2
$d_e = t_s$-cover-$0.5d_b$ =	3.313	in

Use M_critial section calculated above.

Anchor Design (continued)

Flexure

$$\rho = \left[1-\left(\sqrt{1-\frac{2M_u}{.85\phi b d^2 f_c'}}\right)\right]\bullet\frac{.85 f_c'}{f_y} = \quad 0.00406$$

A_s req'd = ρbd = 0.161 in^2

Use #5's @ 8 in

A_s provided = **0.460 OK**

ρ provided = 0.01158

$$c = \frac{A_s f_y}{.85 f_c' \beta_1 b} = \quad 0.677 \text{ in.}$$

$$c/d_e = \quad 0.204$$ <0.003/(0.003+e), OK (Art. 5.6.2.1)

$$a = \beta_1 c = 0.541 \text{ in.}$$

$$1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} = \quad 1.815 \text{ k-ft (Art. 5.6.3.3)}$$

$$I_g = \frac{bt_s^3}{12} = \quad 216 \text{ in}^4$$

$$\phi M_n = \phi A_s f_y (d_e - a/2) = \quad \mathbf{6.299 \text{ k-ft}}$$

>Mu, OK

$$y_t = t_s - c = \quad 5.323 \text{ in.}$$

Member Geometry (design of horizontal reinforcing in Anchor face)

Member Design Width b	12.00	in (per unit width of stem face)
Member Thickness t_s	6.00	in (thickness of Anchor face)
Rebar clear cover	2.00	in
Design Bar Size	5	
A_b =	0.31	in^2
$d_e = t_s$-cover-0.5d_b =	3.688	in

Take load on entire height of Anchor (5') and distribute over horizontal cantilever beam.

cantilever length: 1.625 ft

$w_{anchor-face}$ = 2.243 kip/ft

$$M_{critical\ section} = (w_{anchor-face} \quad cantilever\ lengt \ \hat{}2)/3$$

$$M_{critical\ section} = \quad 1.975 \text{ kip*ft}$$

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Flexure

$$\rho = \left[1 - \left(\sqrt{1 - \frac{2M_u}{.85\phi b d^2 f_c'}}\right)\right] \bullet \frac{.85 f_c'}{f_y} = \quad 0.00274$$

A_s req'd = ρbd = 0.121 in^2

Use #5's @ 8 in

A_s provided = **0.460 OK**

ρ provided = 0.01040

$$c = \frac{A_s f_y}{.85 f_c' \beta_1 b} = \quad 0.677 \text{ in.}$$

$$c/d_e = \quad 0.184 < 0.003/(0.003+e), \text{ OK (Art. 5.6.2.1)}$$

$$a = \beta_1 c = 0.541 \text{ in.}$$

$$1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} = \quad 1.815 \text{ k-ft (Art. 5.6.3.3)}$$

$$I_g = \frac{b t_s^3}{12} = \quad 216 \text{ in}^4$$

$$\phi M_n = \phi A_s f_y (d_e - a/2) = \quad \mathbf{7.076 \text{ k-ft}}$$

>Mu, OK

$$y_t = t_s - c = \quad 5.323 \text{ in.}$$

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Wingwall Design Assumptions

Soil:

Backfill Unit Weight:	$\gamma_{BF} =$	120.00	pcf	Subgrade Unit Weight:	$\gamma_{SG} =$	120.00	pcf
Backfill Int. Friction Angle	$\phi_{BF} =$	34.00	deg.	Subgrade Int. Friction Angle	$\phi_{SG} =$	34.00	deg.
Unit Weight of Water:		62.40	pcf	Resistance Factor Passive Earth	$\phi_{ep} =$	0.50	
Backfill Submerged Unit Wgt.	$\gamma_{sub} =$	69.60	pcf				
Service Bearing Capacity:	$q_n =$	5.00	ksf (assumed)				
Resistance Factor for Bearing	$\phi_b =$	0.45					
Resistance Factor for Sliding	$\phi_\tau =$	0.80					

Concrete/Reinforcement:

Concrete Unit Weight	$w_c =$	150.00	pcf	Steel Elastic Modulus	$E_s =$	29000	ksi
Concrete 28-day Strength	$f_c' =$	5,000	psi	Conc. Elastic Mod.	$E_c = 33000 w_c^{1.5} \sqrt{f_c'} =$	4287	ksi
Steel Yield	$f_y =$	60,000	psi	Modular Ratio	$n = E_s / E_c =$	6.765	
Max. Aggregate Size	$a_g =$	0.75	in.	Stress Block	$\beta_1 =$	0.800	
				Conc. Rupture Modulus fr = 0.20√f'$_c$ =		0.447	ksi

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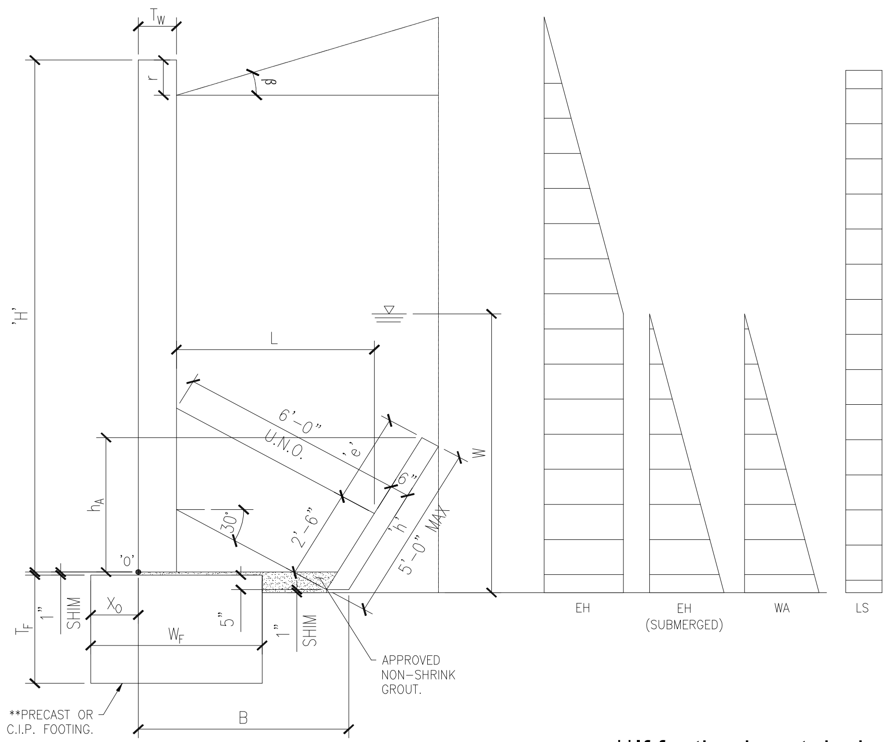

**If footing is not designed by FMC, T_F & W_F = 0

H =	13.210	ft	W_F =	0.000	ft
B =	6.678	ft	T_F =	0.000	ft
Individual Wall Segment =	7.500	ft	e =	2.500	ft
W =	0.000	ft	h =	5.000	ft (5'-0" max)
r =	0.500	ft	Xo =	0.750	ft
T_W =	1.000	ft	h_A =	3.830	ft
T_B =	0.000	ft	# anchors	2	per wall segment
$\beta =$	0.000	deg.	spacing	4.000	ft
$h_{eq} =$	2.000	ft	stem length	8.000	ft (6' min)
L =	6.928	ft	anchor width	3.500	ft

Wingwall Units will be designed as a segment with length shown above. Governing Unit: WW4

$$K_a = \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}\right]\cos\beta = \quad 0.283$$

$$K_p = \left[\frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}\right]\cos\beta = \quad 3.537$$

Load combinations/Load factors

	γ_{DC}	γ_{EV}	γ_{LS}	γ_{EH}	γ_{WA}
Service I	1.00	1.00	1.00	1.00	1.00
Strength Ia (min V, max H)	0.90	1.00	1.75	1.50	1.00
Strength Ib (max V, min H)	1.25	1.35	1.75	1.50	1.00

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Unfactored Vertical Forces/Moments about toe per wall segment:

	F_V (kips)	**Arm** (ft)	**M_V** (k-ft)
DC Wall (above water)	14.210	0.500	7.105
DC Wall (below water)	0.000	0.500	0.000
DC Anchor Stems	2.760	4.130	11.399
DC Anchor Faces	2.540	8.170	20.752
EV (above anchors)	48.620	5.080	246.990
EV (in anchor baskets)	21.310	4.580	97.600
LS (above anchor)	13.739	5.080	69.796
EH (uplift on soil anchor face)	0.000	7.928	0.000
EH (vertical component)	0.000	5.080	0.000
WA	0.000	5.080	0.000

Unfactored Horizontal Forces/Moments about toe per wall segment:

	F_H (kips)	**Arm** (ft)	**M_H** (k-ft)
EH (above water)	22.201	4.403	97.757
EH (below water, rectangular)	0.000	0.000	0.000
EH (below water, triangular)	0.000	0.000	0.000
LS	6.722	6.605	44.401
WA (triangular)	0.000	0.000	0.000

<u>Sliding Check</u> *

(Articles 11.6.3.6 & 10.6.3.4)

Strength I-a Limit State: Max. horizontal force, min. vertical resistance

$$R_R = \phi R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep}$$

for C.I.P. footings (grout poured between walll and anchor):

$$\phi_\tau R_\tau = \phi_\tau \left(\sum F_V\right) \tan\delta = (1.0)\phi_\tau \left(\sum F_V\right) \tan\phi_{SG}$$

$\tan\phi_{SG} =$ 0.675

Factored horizontal sliding forces:

	F_H (kips)
EH	33.301
LS	11.764
WA	0.000
$\sum F_H =$	**45.065**

Factored vertical resistances:

	F_V (kips)	
DC	17.559	
EV	69.930	
Net EH	0.000	
WA	0.000	
$(1.0)\phi_\tau \left(\sum F_V\right) \tan\phi_{SG} =$	**47.210**	**OK**

Key Height: (no key) 0 ft

Passive Resistance $\phi_{ep} R_{ep} =$ 0.000 kips

Total Resistance = **47.210** **OK**

(no key)

Sliding is realistically not an issue for the precast wall due to the grouted anchor hook behind the CIP footing preventing it from translating.

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Overturn Check (Eccentricity)

(Article 11.6.3.3)

Strength I-a Limit State: Max. horizontal force, min. vertical resistance

Assumes only vertical earth above anchor face will contribute to overturning resistance.

Allowable eccentricity: $e_{max} = B/3 =$ 2.226 ft

Resultant location: $X_0 = \dfrac{\sum M_V - \sum M_H}{\sum F_V}$

Actual eccentricity: $e_B = B/2 - X_0$

Factored forces/moments:

	Vertical Forces F_V (kips)	Vertical Moments M_V (kip-ft)	Horizontal Moments M_H (kip-ft)
DC	17.559	35.330	
EV	69.930	344.589	
EH	0.000	0.000	146.636
LS	--	--	77.702
WA	0.000	0.000	0.000
	$\sum F_V =$ 87.489	$\sum M_V =$ 379.919	$\sum M_H =$ 224.338

$X_0 =$ 1.778 ft

$e_B =$ **1.561 ft OK**

Bearing Check on CIP Footing:

There is no moment connection to the CIP footing at the base of the wall. Thus, the bearing stress under the CIP footing due to the wingwall above it is soley due to the dead weight of the wall and the soil directly on top of the footing. The wingwall does not create an eccentricity in the bearing stress, and thus it is a rectangular distribution.

C.I.P. Footing width on drawings: **3.000 ft (MIN)**

Factored Vertical Bearing Forces:

DC_{wall}	**2.368**	**k/ft**
EV	**2.675**	**k/ft**
$DC_{footing}$	**1.125**	**k/ft**
	6.168	**k/ft**

Calculated bearing stress: **2.056 ksf**

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Wingwall Stem Design

Member Geometry

Member Design Width b	12.00	in
Member Thickness t_s	12.00	in
Rebar clear cover	2.00	in
Design Bar Size	5	
A_b =	0.31	in^2
$d_e = t_s\text{-cover-}0.5d_b$ =	9.688	in

Unfactored Horizontal Forces/Moments above anchor at attachment location:

	F_H (kips)	Arm (ft)	M_H (k-ft)
EH (above water)	1.492	3.127	4.666
EH (below water, rectangular)	0.000	---	0.000
EH (below water, triangular)	0.000	---	0.000
LS	0.896	6.605	5.920
WA (triangular)	0.000	0.000	0.000

Controlling Forces: Strength Ib Limit State

Service Load Moment, M_a =	10.59	kip-ft
Design Ultimate Moment, M_u =	17.36	kip-ft
Design Ultimate Shear, V_u =	3.81	kips
Ultimate Axial Force, N_u =	17.76	kips (compressive)

Resistance Factors (Art. 5.5.4.2.1)

Flexure, φ_M = 0.9

Shear, φ_V = 0.9

Flexure

$$\rho = \left[1 - \left(\sqrt{1 - \frac{2M_u}{.85\phi b d^2 f_c'}}\right)\right] \bullet \frac{.85 f_c'}{f_y} = \quad 0.00351$$

A_s req'd = ρbd = 0.408 in^2/ft

Use #5's @ 6 in

A_s provided = **0.620 OK**

ρ provided = 0.00533

$$c = \frac{A_s f_y}{.85 f_c' \beta_1 b} = \quad 0.912 \text{ in.}$$

$$c/d_e = \quad 0.094 \text{ <0.003/(0.003+e), OK (Art. 5.6.2.1)}$$

$$a = \beta_1 c = 0.729 \text{ in.} \qquad 1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} = \quad 6.969 \text{ k-ft (Art. 5.6.3.3)}$$

$$I_g = \frac{b t_s^3}{12} = \quad 1728 \text{ in}^4 \qquad \phi M_n = \phi A_s f_y (d_e - a/2) = \quad \textbf{26.011 k-ft}$$

>Mu, OK

$$y_t = t_s - c = \quad 11.088 \text{ in.}$$

Wingwall Stem Design (cont'd)

Shear (Art. 5.7.3.4.2)

$d_v =$ 8.719 in. (Art. 5.8.2.9)

$s_x = \max(d_v, d_e - a/2) =$ 9.323 in.

$s_{xe} = s_x \dfrac{1.38}{a_g + 0.63} =$ 9.323 in., <12 in., use 12 in.

$\varepsilon_s = \dfrac{\dfrac{|M_u|}{d_v} - 0.5N_u + |V_u|}{E_s A_s} =$ 0.001047

$\beta = \dfrac{4.8}{(1+750\varepsilon_s)} \dfrac{51}{(39+s_{xe})} =$ 2.689

$\phi V_n = \phi V_c = \phi 0.0316 \beta \sqrt{f'_c} b_v d_v =$ **17.892 kips >Vu, OK**

Crack Control (Art 5.6.7)

$k = \sqrt{2\rho n + \rho n^2} - \rho n =$ 0.526

$d_c =$ 2.31 in.

$j = 1 - \dfrac{k}{3} =$ 0.825

$\beta_s = 1 + \dfrac{d_c}{0.7(h - d_c)} =$ 1.341

Exposure $\gamma_e =$ 0.75

$f_{ss} = \dfrac{M_a}{A_s\, jd} =$ 25.651 ksi

$s \le \dfrac{700\gamma_e}{\beta_s f_{ss}} - 2d_c =$ **10.638 in. OK**

Wall Minimum Steel (Art. 12.14.5.8)

(Stem horizontal reinforcement)

$A_s \ge \dfrac{1.3bh}{2(b+h)f_y} =$ 0.065 in^2

$(0.11 \le A_s \le 0.60)$

Minimum As req'd, ea. mat: 0.11 in^2

Use # 5 @ 18 in. max. spacing

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Wingwall Anchor Design

Member Geometry

Member Design Width b	6.00	in (thickness of Twall stem)
Member Thickness t_s	30.00	in (depth of Twall stem)
Rebar clear cover	2.00	in
Design Bar Size	6	
A_b =	0.44	in^2
$d_e = t_s$-cover-$0.5d_b$ =	27.625	in
Tributary width per anchor	3.750	ft

Loads for stem design at anchor location above are multiplied by tributary width:

Controlling Forces: Strength Ib Limit State

Service Load Moment, M_a =	39.70	kip-ft	Resistance Factors (Art. 5.5.4.2.1)
Design Ultimate Moment, M_u =	65.10	kip-ft	Flexure, φ_M = 0.9

Flexure

$$\rho = \left[1-\left(\sqrt{1-\frac{2M_u}{.85\phi b d^2 f_c'}}\right)\right]\bullet\frac{.85 f_c'}{f_y} = \quad 0.00323$$

A_s req'd = ρbd = 0.536 in^2

Use #6's @ 6 in; i.e.: (2) #6 bars in stem

A_s provided = **0.880 OK**

ρ provided = 0.00531

$$c = \frac{A_s f_y}{.85 f_c' \beta_1 b} = \quad 2.588 \text{ in.}$$

$$c/d_e = \quad 0.094 \text{ <0.003/(0.003+e), OK (Art. 5.6.2.1)}$$

$$a = \beta_1 c = 2.071 \text{ in.} \quad 1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} = \quad 22.025 \text{ k-ft (Art. 5.6.3.3)}$$

$$I_g = \frac{b t_s^3}{12} = 13500 \text{ in}^4 \qquad \phi M_n = \phi A_s f_y (d_e - a/2) = \quad \textbf{105.295 k-ft >Mu, OK}$$

$$y_t = t_s - c = \quad 27.412 \text{ in.}$$

Crack Control (Art 5.6.7)

$$k = \sqrt{2\rho n + \rho n^2} - \rho n = \quad 0.525$$

d_c= 2.38 in.

$$j = 1 - \frac{k}{3} = \quad 0.825$$

$$\beta_s = 1 + \frac{d_c}{0.7(h-d_c)} = \quad 1.123$$

Exposure γ_e = 0.75

$$f_{ss} = \frac{M_a}{A_s j d} = \quad 23.755 \text{ ksi}$$

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c = \quad \textbf{14.933 in. OK}$$

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Development: (Art 5.10.8.2.4a)

Modification Factors:	M	
Epoxy Bar?	1.3	Yes
fy = 60	1.0	
Side Cover on hook	0.7	
Enclosed with stirrups?	1.0	No
As, req'd/As, prov	0.61	
Normal Weight Conc.	1.0	

$$l_{dh} = \frac{38.0 d_b}{\sqrt{f_c'}}(ModFactors) =$$ 7.063 in

Cover on front face 2.00 in

ldh, prov = 10.000 in **OK**

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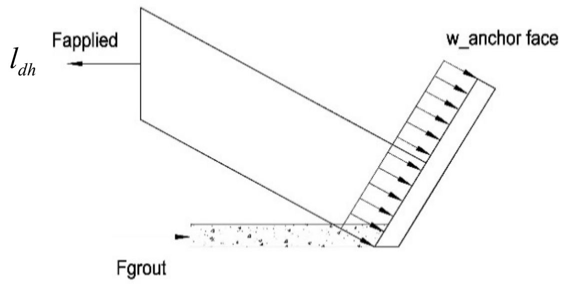
Anchor Design (continued)



FBD - ANCHOR

Force on Anchor face is conservatively taken as the entire lateral load on the wall distributed over the face area of the Anchor. Check bar in Anchor face for critical section where cantilevered extended face of Anchor meets the stem. Check horizontal bar in face.
Force in grout is conservatively taken as equal to entire lateral load on the wall. Check strength of grout.

Calculate force on rear of Anchor face:

As req'd from stem bar calculation: 0.536 in^2/ft

Factored Lateral Loads (EH and Surcharge Loads, where applicable): 45.065 kip
Anchor face height = 5.000 ft
To calculate load on wall, distribute load over face height, and to design per unit width, distribute over
Total Anchor width = 7.000 ft
per unit width of wall $w_{anchor-face}$ = 1.288 kip/ft
e = 2.500 ft

$M_{critical\ section} = (w_{anchor-face} \quad anc\ \ or\ \ extension\hat{}2)/3$

$M_{critical\ section}$ = 2.682 kip*ft

Member Geometry (design of vertical reinforcing in Anchor face)

Member Design Width b	12.00	in (per unit width of stem face)
Member Thickness t_s	6.00	in (thickness of Anchor face)
Rebar clear cover	2.38	in
Design Bar Size	5	
A_b =	0.31	in^2
$d_e = t_s$-cover-$0.5d_b$ =	3.313	in

Use M_critial section calculated above.

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Anchor Design (continued)

Flexure

$$\rho = \left[1 - \left(\sqrt{1 - \frac{2M_u}{.85\phi b d^2 f_c'}}\right)\right] \bullet \frac{.85 f_c'}{f_y} = \quad 0.00468$$

A_s req'd = ρbd = 0.186 in^2

Use #5's @ 8 in

A_s provided = **0.460 OK**

ρ provided = 0.01158

$$c = \frac{A_s f_y}{.85 f_c' \beta_1 b} = \quad 0.677 \text{ in.}$$

$$c/d_e = \quad 0.204 \ <0.003/(0.003+e), \text{ OK (Art. 5.6.2.1)}$$

$$a = \beta_1 c = 0.541 \text{ in.} \quad 1.2M_{cr} = \frac{1.2 f_r I_g}{y_t} = \quad 1.815 \text{ k-ft (Art. 5.6.3.3)}$$

$$I_g = \frac{bt_s^3}{12} = \quad 216 \text{ in}^4 \qquad \phi M_n = \phi A_s f_y (d_e - a/2) = \quad \mathbf{6.299 \text{ k-ft}}$$

>Mu, OK

$$y_t = t_s - c = \quad 5.323 \text{ in.}$$

Member Geometry (design of horizontal reinforcing in Anchor face)

Member Design Width b	12.00	in (per unit width of stem face)
Member Thickness t_s	6.00	in (thickness of Anchor face)
Rebar clear cover	2.00	in
Design Bar Size	5	
A_b =	0.31	in^2
$d_e = t_s$-cover-$0.5d_b$ =	3.688	in

Take load on entire height of Anchor (5') and distribute over horizontal cantilever beam.

cantilever length: 1.000 ft

$w_{anchor-face}$ = 2.575 kip/ft

$$M_{critical\ section} = (w_{anchor-face} \quad cantilever\ lengt \ \hat{}2)/3$$

$$M_{critical\ section} = \quad 0.858 \text{ kip*ft}$$

Flexure

$$\rho = \left[1-\left(\sqrt{1-\frac{2M_u}{.85\phi b d^2 f_c'}}\right)\right]\bullet\frac{.85f_c'}{f_y} =$$ 0.00118

A_s req'd = ρbd = 0.052 in^2

Use #5's @ 8 in

A_s provided = **0.460 OK**

ρ provided = 0.01040

$$c=\frac{A_s f_y}{.85f_c'\beta_1 b} =$$ 0.677 in.

$$c/d_e =$$ 0.184 <0.003/(0.003+e), OK (Art. 5.6.2.1)

$$a=\beta_1 c =$$ 0.541 in.

$$1.2M_{cr}=\frac{1.2f_r I_g}{y_t} =$$ 1.815 k-ft (Art. 5.6.3.3)

$$I_g=\frac{bt_s^3}{12} =$$ 216 in^4

$$\phi M_n = \phi A_s f_y (d_e - a/2) =$$ **7.076 k-ft**

>Mu, OK

$$y_t = t_s - c =$$ 5.323 in.

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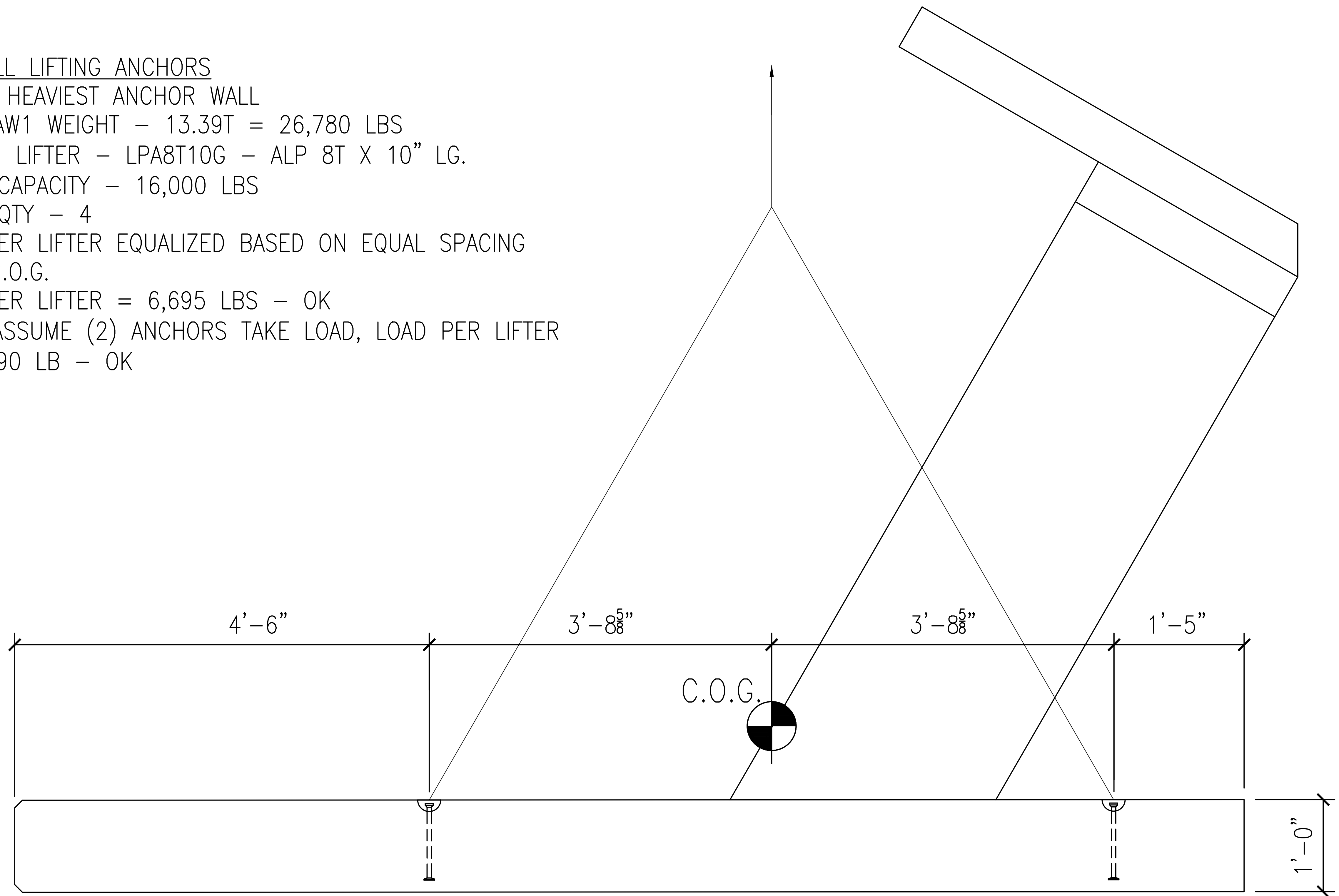
ANCHOR WALL LIFTING ANCHORS

- AW1 IS HEAVIEST ANCHOR WALL
- TOTAL AW1 WEIGHT – 13.39T = 26,780 LBS
- CHOSEN LIFTER – LPA8T10G – ALP 8T X 10" LG.
- LIFTER CAPACITY – 16,000 LBS
- LIFTER QTY – 4
- LOAD PER LIFTER EQUALIZED BASED ON EQUAL SPACING FROM C.O.G.
- LOAD PER LIFTER = 6,695 LBS – OK
- IF WE ASSUME (2) ANCHORS TAKE LOAD, LOAD PER LIFTER = 13,390 LB – OK



LIFTING VIEW

WINGWALL AW1

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ALP® LIFTING PIN ANCHORS

All ALP Lifting Pin Anchors are manufactured using high strength steel with forged ends. The head design provides uniform engagement with the Lifting Eye, and the large forged anchor foot is embedded in the concrete to create the lifting capacity. These Lifting Pin Anchors are designed to meet the OSHA requirements of a 4 to 1 Safety Factor. **Standard finish is hot-dipped galvanized.**

Safe Working Loads (SWL) displayed in the below chart apply to loading in any direction.

				Anchor Capacity in Concrete, 4:1 SWL					
Part #	Ton	Length	Weight (lbs)	1500 PSI (lbs)	2500 PSI (lbs)	3500 PSI (lbs)	5000 PSI (lbs)	Min. Edge Distances	
								Tension	Shear
LPA1T238G	1T	2-3/8”	0.14	1,045	1,350	1,600	1,910	8”	12”
LPA1T258G	1T	2-5/8”	0.14	1,160	1,500	1,770	2,000	8”	12”
LPA1T338G	1T	3-3/8”	0.17	1,900	2,000	2,000	2,000	8”	12”
LPA1T434G	1T	4-3/4”	0.22	2,000	2,000	2,000	2,000	10”	12”
LPA2T234G	2T	2-3/4”	0.31	1,375	1,775	2,100	2,510	8”	12”
LPA2T338G	2T	3-3/8”	0.35	2,000	2,700	3,250	3,900	8”	12”
LPA2T434G	2T	4-3/4”	0.44	3,250	4,000	4,000	4,000	10”	15”
LPA2T512G	2T	5-1/2”	0.49	4,000	4,000	4,000	4,000	11”	17”
LPA2T634G	2T	6-3/4”	0.57	4,000	4,000	4,000	4,000	11”	17”
LPA2T11G	2T	11”	0.85	4,000	4,000	4,000	4,000	11”	17”
LPA4T212G	4T	2-1/2”	0.67	1,400	1,810	2,150	2,560	8”	12”
LPA4T3G	4T	3”	0.74	1,960	2,530	2,990	3,570	8”	12”
LPA4T312G	4T	3-1/2”	0.82	2,275	2,935	3,475	4,150	8”	12”
LPA4T334G	4T	3-3/4”	0.82	2,550	3,250	3,950	4,700	8”	12”
LPA4T414G	4T	4-1/4”	0.89	3,000	3,850	4,550	5,450	9”	13”
LPA4T434G	4T	4-3/4”	0.95	3,650	4,700	5,600	6,700	10”	15”
LPA4T512G	4T	5-1/2”	1.05	4,550	5,850	6,950	8,000	11”	17”
LPA4T718G	4T	7-1/8”	1.26	6,900	8,000	8,000	8,000	15”	22”
LPA4T912G	4T	9-1/2”	1.57	8,000	8,000	8,000	8,000	17”	26”
LPA8T434G	8T	4-3/4”	1.98	4,050	5,200	6,200	7,450	10”	15”
LPA8T634G	8T	6-3/4”	2.50	7,000	9,000	10,750	12,850	14”	21”
LPA8T834G	8T	8-3/4”	3.07	12,940	16,000	16,000	16,000	18”	27”
LPA8T10G	8T	10”	3.45	14,790	16,000	16,000	16,000	20”	30”
LPA8T1338G	8T	13-3/8”	4.33	16,000	16,000	16,000	16,000	27”	41”
LPA8T2634G	8T	26-3/4”	7.87	16,000	16,000	16,000	16,000	27”	41”
LPA16T778G	16T	7-7/8”	6.54	7,000	9,000	10,750	12,850	14”	21”
LPA20T10G	20T	10”	7.73	11,750	15,150	17,950	21,500	20”	30”
LPA20T1934G	20T	19-3/4”	13.18	26,000	33,800	40,000	40,000	40”	48”

Shaded area indicates the capacity in concrete is limited by the mechanical capacity of the anchor

- Table is based on normal weight concrete (145-150PCF)
- Minimum anchor spacing is 2x the published edge distance
- Minimal reinforcement required to achieve above load values
- Capacities in radius sections are reduced - consult ALP Supply® Tech Support
- Above capacities are based upon mechanical testing, concrete testing and available industry data
- Loads are listed at varying concrete strengths (PSI) to accommodate varying conditions at time of loading
- Proper rigging and all lifting angle load magnifications are to be used to deter
- To achieve published values a minimum concrete cover of 1” below the foot of anchor is required
- For insulated wall panel and slab applications, the foot of the anchor can sit on or be recessed by the depth of the foot into the insulation, as long as full concrete bearing on the top of the foot is achieved
- For lifting and handling of thin slabs, design needs to ensure slab is properly reinforced to prevent flexural stresses from cracking slab

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