		PROJECT NO.		1122-1001-00	
	PROJECT CUY-271-0.00 (PID 80418) SUBJECT Reinforced Concrete Retaining Wall Design Wall WS1, Panels 1-3	COMP. BY CHECKED BY	ASP LNB	DATE DATE	3/17/2014 3/17/2014
Dimensions and Weights for Co	oncrete Design				
Footing width, w _{foot} =	9.50 ft = 114.00 in Concrete we	eight, w _c = 0.150 kc	f		

r ooting math, m _{toot}	3.50		-	114.00			0.100	KUI	
						Water weight, w _w =	0.062	kcf	
Footing heel width, w _{heel} =	6.00	ft				Saturated soil weight, w_{ss} =	0.130	kcf	
Footing heel height, h _{heel} =	1.50	ft	=	18.00	in	Buoyant soil weight, w _{sb} =	0.068	kcf	
		_						_	
Footing toe width, w _{toe} =	2.00	ft				Height of wall, h _w =	5.88	ft	top of heel to top of wall
Footing toe height, $h_{toe} =$	1.50	ft	=	18.00	in	Height of water, h _{water} =	0.00	ft	top of heel to water line
Wall width at top, t_{wt} =	1.50	ft	=	18.00	in	Height of soil, h _s =	5.88	ft	top of heel to ground line
Wall width at base, t_{wb} =	1.50	ft	=	18.00	in	Height of satur. soil, h_{ss} =	5.88	ft	height of satur. soil above top of heel
						Height of buoy. soil, h _{sb} =	0.00	ft	height of buoy. soil above top of heel
Concrete strength, fc' =	4.00	ksi							
Rebar strength, f _y =	60.00	ksi				Active pressure coeff., $K_a =$	0.280		
Steel mod. of elast., E _s =	29,000	ksi				LL surcharge soil ht., h_{LL} =	4.29	ft	

Optional Collision Loading for Barrier on Top of Wall

Per ODOT comments for I-70/71, use the transverse loading of 54 kips for a TL-4 test level railing [AASHTO, Table A13.2-1] distributed over the retaining wall's joint spacing.

у	(y or n)
24.43	ft
3.50	ft

Design Summary

Summary of Design Status

...... Footing Wall F Design Item Stem Heel Toe OK OK Shear OK Minimum Reinforcement OK OK OK Shrinkage & Temperature Crack Control OK OK OK v OK N/A N/A

For calculations related to each design item, see below.

Design Footing for Shear

Design footings to have adequate shear capacity without transverse reinforcement.

[5.13.3.6]

Determine d_v

Assume: For the he	#8 bars at #4 bars at	12.00 12.00		for the top t for the botto						0.79 i 0.20 i			0 in cover 0 in cover
		d ()		40.00		0.00	,	4 000			45 50		
	d _{sheel} = h - cover -	· d _{bar} / 2	=	18.00	-	2.00	- (1.000	/2)	=	15.50	in	
	$a_{heel} = \frac{A_s}{0.85}$	f _y f _c ' b	=	0.85	0.79 x	x 60 4.0 x	12			=	1.16	in	
[5.8.2.9]	d _{vheel} = d _{sheel} - a /	2	=	15.50	- (1.16	/2)			=	14.92	in	GOVERNS
	or d_{vheel} = 0.90 d_e		=	0.90	х	15.50				=	13.95	in	
	or d_{vheel} = 0.72 h		=	0.72	х	18.00				=	12.96	in	
For the toe	9:												
	d_{stoe} = h - cover -	d _{bar} / 2	=	18.00	-	3.00	- (0.500	/2)	=	14.75	in	
	$a_{toe} = \frac{A_s}{0.85}$	f _y f _c ' b	=	0.85	0.20 x	x 60 4.0 x	12			=	0.29	in	
[5.8.2.9]	d _{vtoe} = d _{stoe} - a / 2		=	14.75	- (0.29	/2)			=	14.60	in	GOVERNS
	or d_{vtoe} = 0.90 d_e		=	0.90	х	14.75				=	13.28	in	

Reinforcing Steel Summary

Footing:	Top transverse: Bottom transverse: Longitudinal:	#8 bars at 12.00 in c/c #4 bars at 12.00 in c/c #4 bars at 12.00 in c/c
Wall Stem:	Back face vertical: Front face vertical: Horizontal:	#6 bars at 12.00 in c/c #4 bars at 12.00 in c/c #4 bars at 12.00 in c/c

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or $d_{vtoe} = 0.72 h$ = 0.72 x 18.00 = 12.96 in

Check Heel for Shear

The critical shear section for the heel of the footing is located at the back face of the wall. The heel of the footing is assumed to carry its self weight and the rectangular soil block above it. This neglects the benefit of any upward soil pressure below the footing (conservative).

 $V_{u} = (\gamma_{EV} w_{ss} h_{ss} + \gamma_{EV} w_{sb} h_{sb} + \gamma_{DC} w_{c} h_{heel} + \gamma_{LL} w_{ss} h_{LL} + \gamma_{WA} w_{w} h_{water}) x w_{heel}$

V _u =	(1.35	х	0.130	х	5.88	+	1.35	х	0.068	х	0.00	+	1.25 x	0.15	50 x	1.50	+
	1.75	х	0.130	х	4.29	+	1.00	х	0.062	х	0.00) x	6.00	=	13.73	k/ft	

[5.8.3.3] Using β = 2.00 and assuming bars in the top mat as above:

 $\phi V_{c} = \phi 0.0316 \beta (f_{c})^{0.5} b_{v} d_{v}$ [5.8.3.4] [5.8.2.9] $\phi V_c =$ 0.0316 x (4.0)^{0.50} x 20.37 k 0.90 х 2.00 12.00 14.92 х х = 20.37 k > 13.73 k <u>OK</u>

Check Toe for Shear

The peak bearing stress is 2.36 ksf for the Extreme IIb load case.

The critical section for the toe of the footing is at dv from the front face of the wall. For a quick simplified check, try applying the peak bearing stress over the entire length of the toe (conservative).

 $V_u = \sigma_V w_{toe}$ = 2.36 x 2.00 = 4.72 k/ft

[5.8.3.3] Using β = 2.00 and assuming bars in the bottom mat as above:

 $\phi V_{c} = \phi 0.0316 \beta (f_{c})^{0.5} b_{v} d_{v}$ [5.8.3.4] [5.8.2.9] $\phi V_c =$ x (4.0)^{0.50} x 0.90 х 0.0316 х 2.00 12.00 х 14.60 = 19.93 k 19.93 4.72 <u>OK</u> k > k

Design Footing Reinforcement [5.13.3.4]

Each mat of reinforcement is checked to ensure that it has adequate capacity and that the maximum and minimum reinforcement checks are satisfied. The critical section for flexure in the footing is at the face of the wall.

Top Transverse Reinforcement

For

From the shear check of the heel, V_{μ} = 13.73 k/ft

 $M_u = V_u x (w_{heel} / 2) = 13.73 x (6.00 / 2) = 41.20 k-ft$

Set up the equation to solve for the required steel area:

$$\begin{split} M_{u} &= \phi M_{n} = \phi A_{s} f_{y} (d_{s} - a/2) \\ M_{u} &= 0.90 \quad x A_{s} x \quad 60 \quad (ds - \frac{As}{1.7 \ s} \frac{x \quad 60}{4.0 \ s} \frac{1}{1.7 \ s}) x (\frac{1}{12}) \\ 3.309 \quad A_{s}^{2} \quad - \quad 4.50 \quad d_{s} A_{s} + M_{u} = 0 \end{split}$$

the reinforcing steel assumed for the heel, $d_{s} = 15.50$ in

Substituting and solving for A_s , it is found that required $A_s = 0.61 \text{ in}^2/\text{ft}$

Try: #8 bars at 12.00 in c/c for the top transverse bars in the heel $As = 0.79 \text{ in}^2/\text{ft}$

Check Minimum Reinforcement [5.7.3.3.2]

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Determine the cracking moment:			
$f_r = 0.24 (f_c)^{0.5} = 0.24 \times (4.0)^{0.50} = 0.48 \text{ ksi}$. 4		
$l_g = (1/12) b h^3 = 0.0833 x 12.00 x (18.00)^3 = 5832.0 i$	n'		
$y_t = (1/2) h$ = 0.5000 x 18.00 = 9.00 in			
$M_{CR} = \gamma_1 \gamma_3 f_r l_g / y_t = 1.60 x 0.67 x 0.48 x 5832.0$	/(9.00 x	12.00) =	27.79 k-ft
The capacity of the section must be greater than or equal to the smaller of:			
$M_{CR} = 27.79 = 27.79 \text{ k-ft } GOVERNS$ (4/3) $M_u = 1.33 \text{ x} 41.20 = 54.94 \text{ k-ft}$			
The capacity of the top mat of reinforcement is:			
$M_r = \phi A_s f_y (d_s - a/2)$			
For the reinforcing steel used, $d_s = 18.00 - 2.00 - (1.000 / 2)$	= 15.50 in		
$M_r = 0.90 x 0.79 x 60 x (15.50 - \frac{0.79 x 60}{1.7 x 4.0 x 12}$	$) x (\frac{1}{12}) =$	53.04 k-ft	
53.04 k-ft > 27.79 k-ft <u>OK</u>	. 2	. 2	
Check minimum reinforcement for temperature and shrinkage (5.10.8) $A_s =$	0.79 in ² /ft >	0.17 in ² /ft	<u>OK</u>
Use #8 bars at 12.00 in c/c for top transverse reinforcement in the footing.			
Bottom Transverse Reinforcement			
From the shear check of the toe, $V_u = 4.72$ k/ft			
$M_u = V_u x (w_{toe} / 2) = 4.72 x (2.00 / 2) = 4.72 k-ft$			
Set up the equation to solve for the required steel area and again use:			
3.309 A_s^2 - 4.50 $d_s A_s + M_u = 0$			
For the reinforcing steel assumed for the heel, $d_s = 14.75$ in			
Substituting and solving for A_s , it is found that required $A_s = 0.07 \text{ in}^2/\text{ft}$			
Try: #4 bars at 12.00 in c/c for the bottom transverse bars in the toe As =	0.20 in ² /ft		
Check Minimum Reinforcement [5.7.3.3.2]			
Determine the cracking moment:			
$f_r = 0.24 (f_c)^{0.5} = 0.24 \times (4.0)^{0.50} = 0.48 \text{ ksi}$			
$I_g = (1/12) b h^3 = 0.0833 x 12.00 x (18.00)^3 = 5832.0 i$	in ⁴		
$y_t = (1/2) h$ = 0.5000 x 18.00 = 9.00 in			
$M_{CR} = \gamma_1 \gamma_3 f_r I_g / y_t$ = 1.60 x 0.67 x 0.48 x 5832.0	/(9.00 x	12.00) =	27.79 k-ft
The capacity of the section must be greater than or equal to the smaller of:			
M_{CR} = 27.79 = 27.79 k-ft (4/3) M_u = 1.33 x 4.72 = 6.29 k-ft <u>GOVERNS</u>			
The capacity of the bottom mat of reinforcement is:			
$M_r = \phi A_s f_y (d_s - a/2)$			

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For	the reinforcing stee	el used, d _s =	18	.00 -	3.00 -	(0.500 / 2) =	14.75 in				
	M _r = 0.90 x	0.20 x	60 x (14.75	- <u>0.20</u> 1.7 x	0 x 60 4.0 x 1	<u>-</u>)x(-	<u>1</u> 12) =	= 13.14	k-ft		
	13	.14 k-ft	> 6.	29 k-ft <u>Ol</u>	<u><</u>							
Che	eck minimum reinfo	rcement for t	emperature	and shrinkage	e (5.10.8)	As	= 0.20) in ² /ft :	> 0.17	in²/ft	<u>OK</u>	
<u>Use</u>	e #4 bars at 12.00 ir	n c/c for botto	m transvers	se reinforceme	nt in the footin	ng.						
Longitudi	inal Reinforcemer	ıt	[5.10.8]									
Prov	vide longitudinal rei	nforcement i	n the footing	g based on shr	inkage and te	mperature requi	rements.					
	$h_{max} = max(h_{heel},$	h _{toe}) =	18.00	in								
	Min. A _s =	1.30 w _{fe} 2 (w _{foot} +	h _{max} h _{max}) f _y	$=$ $\frac{1}{2}$	<u>30 x 11</u> x (114.00	14.00 x + 18.00	18.00) x	60 =	0.17 in	²/ft		
The	e maximum spacing	of reinforcer	nent is:									
	h _{min} = min(h _{heel} , h	n _{toe}) =	18.00	in								
	Max. spacing = 3 or Max. spacing : or Max. spacing :	= 18 in	alls and foo	tings ≥ 18 in th	= = iick) =	54.00 in 18.00 in 12.00 in	GOVE	<u>RNS</u>				
Try:	#4 bars at	12.00	in c/c for the	e top and botto	m longitudina	I bars A _s	= 0.20) in ² /ft :	> 0.17	in²/ft	<u>OK</u>	
	12 00 ir	o c/c for top a	nd bottom l	ongitudinal roj	oforcomont in	sp	11 < A acing =	A _s = 0.20 12.00 in	in ² /ft -	< 0.60 12.00 i	<u>OK</u> in <u>OK</u>	
Determine Loa	e #4 bars at 12.00 ir Ids for Wall Stem I s on the stem at the	Design				sp <u>the footing.</u>	acing =	-				
Determine Loa		Design top of the fo				sp <u>the footing.</u>	acing =	-				
Determine Loa	ids for Wall Stem I s on the stem at the	Design top of the for ure:		termined to ar	rive at the des	sp <u>the footing.</u>	acing = e wall.	-	=			
Determine Loa	nds for Wall Stem I s on the stem at the urated Earth Pressu	Design top of the for ure: K _a h _{ss} ²	oting are de = 0.5	termined to ar	rive at the des	sp <u>the footing.</u> sign forces for th	acing = e wall.	12.00 in	=	12.00 i		
Determine Loa The loads Satu	nds for Wall Stem I s on the stem at the urated Earth Pressu $P_{EH(S)} = (1/2) w_{ss}$	Design top of the for ure: K _a h _{ss} ² [(1/3) h _{ss} + h	oting are de = 0.5	termined to ar x 0.130	rive at the des	sp <u>the footing.</u> sign forces for th .280 x (acing = e wall. 5.88) [;]	12.00 in	= k	12.00 i	in <u>OK</u>	
Determine Loa The loads Satu	nds for Wall Stem I s on the stem at the urated Earth Pressu $P_{EH(S)} = (1/2) w_{ss}$ $M_{EH(S)} = P_{EH(S)} x$	Design top of the for ure: K _a h _{ss} ² [(1/3) h _{ss} + h e:	oting are de = 0.5 _{sb}]	termined to ar x 0.130	rive at the des	sp <u>the footing.</u> sign forces for th .280 x (acing = e wall. 5.88) [;]	12.00 in	= k	12.00 i	in <u>OK</u>	
Determine Loa The loads Satu	Inds for Wall Stem I s on the stem at the urated Earth Pressu $P_{EH(S)} = (1/2) w_{ss}$ $M_{EH(S)} = P_{EH(S)} x $ by ant Earth Pressur $P_{EH(B)} = K_a h_{sb} [w]$	Design top of the for ure: K _a h _{ss} ² [(1/3) h _{ss} + h e:	oting are de = 0.5 _{sb}]	termined to ar x 0.130	rive at the des	sp the footing. ign forces for th .280 x (.333 x	acing = e wall. 5.88) [;]	12.00 in	= k] =	12.00	in <u>OK</u>	k
Determine Loa The loads Satu	Inds for Wall Stem I s on the stem at the urated Earth Pressu $P_{EH(S)} = (1/2) w_{ss}$ $M_{EH(S)} = P_{EH(S)} x$ $M_{eh(S)} = P_{eh(S)} x$ $P_{eh(B)} = K_a h_{sb}$ [w	Design top of the for ure: $K_a h_{ss}^2$ [(1/3) $h_{ss} + h$ e: $_{ss} h_{ss} + (1/2)$ 280 x	oting are de = 0.5 _{sb}] w _{sb} h _{sb}] 0.00	termined to ar x 0.130 = 0.63 x [0.130	rive at the des	sp the footing. ign forces for th .280 x (.333 x	acing = e wall. 5.88) [;] 5.88	12.00 in ² = 0.63 + 0.00	= k] =	12.00	k-ft	k
Determine Loa The loads Satu	ads for Wall Stem I s on the stem at the urated Earth Pressu $P_{EH(S)} = (1/2) w_{ss}$ $M_{EH(S)} = P_{EH(S)} \times $ by ant Earth Pressur $P_{EH(B)} = K_a h_{sb} [w$ $P_{EH(B)} = 0.2$ $y_B = [h_{sb} (w_{ss} h_{ss})]$	Design top of the for ure: $K_a h_{ss}^2$ [(1/3) $h_{ss} + h$ e: $_{ss} h_{ss} + (1/2)$ 280 x	oting are de = 0.5 _{sb}] w _{sb} h _{sb}] 0.00	termined to ar x 0.130 = 0.63 x [0.130	rive at the des	sp the footing. ign forces for th .280 x (.333 x	acing = e wall. 5.88) [;] 5.88	12.00 in ² = 0.63 + 0.00 0.068 x x 0.00	= k] =	12.00	n <u>OK</u> k-ft 0.00	k
Determine Loa The loads Satu	ads for Wall Stem I s on the stem at the urated Earth Pressu $P_{EH(S)} = (1/2) w_{ss}$ $M_{EH(S)} = P_{EH(S)} \times $ by ant Earth Pressur $P_{EH(B)} = K_a h_{sb} [w$ $P_{EH(B)} = 0.2$ $y_B = [h_{sb} (w_{ss} h_{ss})]$	Design top of the for ure: $K_a h_{ss}^2$ [(1/3) $h_{ss} + h$ e: $s_{ss} h_{ss} + (1/2)$ 280 x $s_{s} + (1/3) w_{sb} h$ 00 x (2.0 x	oting are de = 0.5 sb] w _{sb} h _{sb}] 0.00 n _{sb})] / (2 w _s 0.130	termined to ar x = 0.130 = 0.63 x = 0.130 x = 0.130 x = 0.130 x = 0.130 x = 0.130 x = 0.130	rive at the des	sp <u>the footing.</u> sign forces for th .280 x (.333 x 5.88 + 0	acing = e wall. 5.88) [*] 5.88 .5 x 0.068	12.00 in ² = 0.63 + 0.00 0.068 x x 0.00	= k] = x 0.00	12.00 i 1.23] =	n <u>OK</u> k-ft 0.00	
Determine Loa The loads Satu Buo	Ads for Wall Stem Ia on the stem at theurated Earth Pressu $P_{EH(S)} = (1/2) w_{ss}$ $M_{EH(S)} = P_{EH(S)} \times $ oyant Earth Pressur $P_{EH(B)} = K_a h_{sb} [w$ $P_{EH(B)} = 0.1$ $y_B = [h_{sb} (w_{ss} h_{sc})$ $y_B = [0.1)$	Design top of the for ure: $K_a h_{ss}^2$ [(1/3) $h_{ss} + h$ e: $s_{ss} h_{ss} + (1/2)$ 280 x $s_{s} + (1/3) w_{sb} h$ 00 x (2.0 x	oting are de = 0.5 sb] 0.00 nsb)] / (2 ws 0.130 0.130	termined to ar x 0.130 = 0.63 x [0.130 x [0.130 $x s h_{ss} + w_{sb} h_{sb}$ x 5.88 x 5.88	rive at the des	sp <u>the footing.</u> sign forces for th .280 x (.333 x 5.88 + 0	acing = e wall. 5.88) [*] 5.88 .5 x 0.068	12.00 in ² = 0.63 + 0.00 0.068 x x 0.00	= k] = < 0.00)]	12.00 i 1.23] =	n <u>OK</u> k-ft 0.00	
Determine Loa The loads Satu Buo	Ads for Wall Stem Iis on the stem at theurated Earth Pressu $P_{EH(S)} = (1/2) w_{ss}$ $M_{EH(S)} = P_{EH(S)} \times $ oyant Earth Pressur $P_{EH(B)} = K_a h_{sb} [w]$ $P_{EH(B)} = 0.2$ $y_B = [h_{sb} (w_{ss} h_{ss})]$	Design top of the for ure: $K_a h_{ss}^2$ [(1/3) $h_{ss} + h$ e: ss $h_{ss} + (1/2)$ 280 x s + (1/3) $W_{sb} h$ 00 x (2.0 x y _B =	oting are de = 0.5 sb] 0.00 nsb)] / (2 ws 0.130 0.130	termined to ar x = 0.130 = 0.63 x = 0.130 x = 0.130 x = 0.130 x = 5.88 x = 5.88 x = 5.88 x = 0.00	rive at the des	sp the footing. ign forces for th .280 x (.333 x 5.88 + 0 .333 x .068 x	acing = e wall. 5.88) [*] 5.88 .5 x 0.068	12.00 in ² = 0.63 + 0.00 0.068 x x 0.00	= k] = < 0.00)]	12.00 i 1.23] =	n <u>OK</u> k-ft 0.00	
Determine Loa The loads Satu Buo	ands for Wall Stem I s on the stem at the urated Earth Pressu $P_{EH(S)} = (1/2) w_{ss}$ $M_{EH(S)} = P_{EH(S)} x $ by ant Earth Pressur $P_{EH(B)} = K_a h_{sb} [w$ $P_{EH(B)} = 0.2$ $y_B = [h_{sb} (w_{ss} h_{ss})$ $y_B = [0.2] / ($ $M_{EH(B)} = P_{EH(B)} x)$ ter Pressure:	Design top of the for ure: $K_a h_{ss}^2$ [(1/3) $h_{ss} + h$ e: $s_{ss} h_{ss} + (1/2)$ 280 x $s_s + (1/3) w_{sb} h$ 00 x (2.0 x y_B = water 2	$\begin{array}{rcl} \text{bing are de} \\ = & 0.5 \\ \text{sb} \end{array} \\ \\ \text{w}_{\text{sb}} \hspace{0.1cm} \text{h}_{\text{sb}} \\ \\ \text{0.00} \\ \\ \text{h}_{\text{sb}} \end{array})] / (2 \hspace{0.1cm} \text{w}_{\text{s}} \\ \\ \\ 0.130 \\ \\ 0.130 \\ \\ 0.00 \\ \end{array} \\ = & 0.5 \end{array}$	termined to ar x = 0.130 = 0.63 x = 0.130 x = 0.130 x = 0.130 x = 5.88 x = 5.88 x = 5.88 x = 0.00	rive at the des	sp the footing. ign forces for th .280 x (.333 x 5.88 + 0 .333 x .068 x	acing = e wall. 5.88) [*] 5.88 .5 x 0.068 0.00)	12.00 in 12.00 in $^2 = 0.63$ + 0.00 0.068 $^{\circ}$ x 0.00 = 0.00	= k] = (0.00)] k-ft	12.00 i 1.23] =	n <u>OK</u> k-ft 0.00	
Determine Loa The loads Satu Buo Wat	ads for Wall Stem I s on the stem at the urated Earth Pressu $P_{EH(S)} = (1/2) w_{ss}$ $M_{EH(S)} = P_{EH(S)} x $ by ant Earth Pressur $P_{EH(B)} = K_a h_{sb} [w$ $P_{EH(B)} = 0.2$ $y_B = [h_{sb} (w_{ss} h_{ss})]$ $y_B = [h_{sb} (w_{ss} h_{ss})]$ $y_B = [0.2] / (1)$ $M_{EH(B)} = P_{EH(B)} x + 1$ ter Pressure: $P_{WA} = (1/2) w_w h_s$	Design top of the for ure: $K_a h_{ss}^2$ [(1/3) $h_{ss} + h$ e: $s_{ss} h_{ss} + (1/2)$ 280 x $s_s + (1/3) w_{sb} h$ 00 x (2.0 x y_B = water 2	$\begin{array}{rcl} \text{bing are de} \\ = & 0.5 \\ \text{sb} \end{array} \\ \\ \text{w}_{\text{sb}} \hspace{0.1cm} \text{h}_{\text{sb}} \\ \\ \text{0.00} \\ \\ \text{h}_{\text{sb}} \end{array})] / (2 \hspace{0.1cm} \text{w}_{\text{s}} \\ \\ \\ 0.130 \\ \\ 0.130 \\ \\ 0.00 \\ \end{array} \\ = & 0.5 \end{array}$	termined to ar x 0.130 = 0.63 x [0.130 x [0.130 x 5.88 x 5.88 x 5.88 x 0.00 x 0.062	rive at the des	sp <u>the footing.</u> Sign forces for th .280 x (.333 x 5.88 + 0 .333 x .068 x 0.00) ²	acing = e wall. 5.88) [*] 5.88 .5 x 0.068 0.00)	12.00 in 12.00 in $^2 = 0.63$ + 0.00 0.068 : x 0.00 = 0.00 0.00	= k] = c 0.00)] k-ft k	12.00 i 1.23] =	n <u>OK</u> k-ft 0.00	

	T 7	CLIEN PROJE		T 271-0.00) (PID 80)418)						PROJE	CT NO.			1122-100	1-00	
D		SUBJE	CT Reinf	orced Co WS1, Pa	oncrete F	Retainir	ng Wal	l Design				COMP. CHECK		AS		DATE DATE		3/17/2014 3/17/2014
	M _{LS} = P _{LS} x (1	/2) h _s	=	0.92	х	0.5	500	x	5.88	=			2.70	k-ft				
Colli	sion Load at To	p of Parap	et:															
	Use a Live Lo	ad of	2210	lbs/ft app	plied at I	n _r =	3.5	ft above	the top	of the v	vall.							
	P _{CT} =												2.21	k				
	M _{CT} = P _{CT} x (h	ı _w + h _r)	=	2.21	х	(5.	88	+	3.50) =			20.73	k-ft				
Usin	g the Strength I	load comb	oination, th	e factore	d desigr	forces	s for the	e wall ste	em are:									
	H _u = 1.50 (P _{EF}	H(S) + P _{EH(B)}) + 1.00 P	_{wa} + 1.75	5 (P _{LS} +	P _{LL})												
	H _u = 1.50	x (0.6	3 +	0.00) +	1.00	х	0.00	+	1.75	х (0.92	+	0.00) =	2.55	k	
	M _u = 1.50 (M _E	H(S) + M _{EH(E}	₃₎) + 1.00 N	/I _{WA} + 1.7	′5 (M _{LS} +	- M _{LL})												
	M _u = 1.50	x (1.23	3 +	0.00) +	1.00	x	0.00	+	1.75	х (2.70	+	0.00) =	6.57	k-ft	
The Extrer	me Event II desi	gn forces f	or the wall	stem are	e:													
H _{ext}	= 1.50 (P _{EH(S)} +	P _{EH(B)}) + 1	.00 P _{CT} + (0.50 (P _{LS}	+ P _{LL})													
H _{ext}	= 1.50	x (0.63	3 +	0.00) +	1.00	x	2.21	+	0.50	х (0.92	+	0.00) =	3.61	k	
M _{ext}	= 1.50 (M _{EH(S)} +	M _{EH(B)}) + 7	1.00 M _{CT} +	0.50 (M _L	_{-S} + M _{LL})													
M _{ext}	= 1.50	x (1.23	3 +	0.00) +	1.00	x	20.73	+	0.50	х (2.70	+	0.00) =	23.93	k-ft	
The	service design f	forces for t	he wall ste	m are:														
	H _{serv} = 1.00 (P	P _{EH(S)} + P _{EH}	_{I(B)}) + 1.00	P _{WA} + 1.0	00 (P _{LS} -	+ P _{LL})												
	H _{serv} = 1.	00 x (0.63	+	0.00) +	1.00	x	0.00	+	1.00	х (0.92	+	0.00) =	1.55	k
	M _{serv} = 1.00 (N	M _{EH(S)} + M _E	_{H(B)}) + 1.00) M _{WA} + 1	1.00 (M _{LS}	₅ + M _{LL})											
	M _{serv} = 1.	00 x (1.23	+	0.00) +	1.00	х	0.00	+	1.00	х (2.70	+	0.00) =	3.93	k-ft
all Stem Desi	gn - Investigat	e Shear																
Shear typi	cally does not g	overn the c		etaining v	valls. If	shear o	does be	ecome a	in issue	, the thi	ckness	of the s	tem sho	ould be inc	reased	such		
Igno	ring the benefits	s of the she	ear key and	d axial co	mpressi	on, the	shear	capacity	of the	stem ca	in be s	hown to	be grea	iter than th	at requi	red.		
	$V_n = V_c + V_s +$	V _p			[5.8.	3.3-1]												
Reco	ognizing that V_s	and V_p are	e zero															
	$V_n = V_c$																	
	V_c = 0.0316 β	$(f_{c}')^{0.5} b_{v} d$	l _v		[5.8.	3.3-3]												
The	maximum effec	tive shear	depth is:															
	For 2.0 inc	ch clear co	ver and	#6 ba	ırs, d _s =	18	.00	-	2.00	- (0.7	50 / 2	:) =	15.63	in			
	$d_v = 0.9 d_e = 0$ $d_v = 0.72 h = 0$	0	0.90 0.72		15.63 18.00	= =	14. 12.		<u>GO'</u>	/ERNS								
Follo	ow the General I	Procedure	using the r	provisions	s of App	endix F	35 ası	nar Sact	ion [5 8	3 4 21.								
1 One			doing the p			0	50, ao j		1011 [0.0	.o.+. z].								

	CLIENT ODOT	PROJECT NO.		1122-1001-00	0
ODLZ	PROJECT CUY-271-0.00 (PID 80418) SUBJECT Reinforced Concrete Retaining Wall Design Wall WS1, Panels 1-3	COMP. BY CHECKED BY	ASP LNB	DATE DATE	3/17/2014 3/17/2014

19.31 k > 3.61 k <u>OK</u>

Wall Stem Design - Investigate Strength Limit State

Determine the area of back-face flexural reinforcement necessary to satisfy the design moment:

M_u = 23.93 k-ft

Again use the equation:

 $3.309 \quad A_s^2 \quad - \quad 4.50 \quad d_s A_s + M_u = 0$

Substituting and solving for A_s , it is found that required $A_s = 0.35$ in²/ft

Try: #6 bars at 12.00 in c/c $A_s = 0.44 \text{ in}^2/\text{ft} > 0.35 \text{ in}^2/\text{ft} \frac{OK}{S}$

Wall Stem Design - Investigate Service Limit State [5.7.3.4]

Check the crack control equations to ensure that the primary flexural reinforcement is well distributed. The service load bending moment is:

M_{serv} = 3.93 k-ft

Check the modulus of rupture for concrete:

$$\phi f_r = \phi \ 0.24 \ (f_c)^{0.5} = 0.80 \ x \ 0.24 \ x \ (4.0)^{0.50} = 0.384 \ \text{ksi}$$

$$S = \frac{b \ t_{wb}^2}{6} = \frac{12.00 \ x \ (18.00)^2}{6} = 648.00 \ \text{in}^3$$

$$f_{act} = \frac{M_{serv}}{S} = \frac{3.93 \ x \ 12}{648.00} = 0.073 \ \text{ksi} < 0.384 \ \text{ksi}$$

If modulus of rupture check is "NG", then check the spacing of the reinforcement. First, determine the modular ratio:

$$E_c = 33,000 w_c^{1.5} (f_c^{-1})^{0.5} = 33,000 x (0.150)^{1.5} x (4.00)^{0.5} = 3,834$$
 ksi
n = $E_s/E_c = 29,000$ / 3,834 = 7.56, use n = 8.00

For 2.0 inch clear cover and #6 bars, $d_s = 18.00 - 2.00 - (0.750 / 2) = 15.63$ in

Determine the location of the neutral axis:

0.5 b $x^2 = n A_s (d_s - x)$ 0.5 (12) $x^2 = 8 \times 0.44 \times (15.63 - x)$ solving, x = 2.75 in

Check the spacing of the reinforcement to control cracking:

$$y = d_{s} - x = 15.63 - 2.75 = 12.88 \text{ in}$$

$$l_{cr} = \frac{b x^{3}}{3} + n A_{s} (d_{s} - x)^{2}$$

$$l_{cr} = \frac{-12.00 \times (-2.75)^{3}}{3} + 8 \times 0.44 \times (-15.63 - 2.75)^{2} = 666.68 \text{ in}^{4}$$

$$\gamma_{e} = 1.00$$
For 2.0 inch clear cover and #6 bars, $d_{c} = 2.00 + (-0.750/2) = 2.38 \text{ in}$

$$\beta_{s} = 1.0 + \frac{d_{c}}{-0.7 (t_{wb} - d_{c})} = 1.0 + \frac{2.38}{-0.7 \times (-18.00 - 2.38)} = 1.22$$

$$f_{ss} = n \frac{M_{serv}y}{l_{cr}} = 8 \times \frac{3.93 \times 12.88 \times 12.00}{-666.68} = 7.29 \text{ ksi}$$

	T 7			-0.00 (PID 80						CT NO.		1122-10	
VD		SUBJEC	Reinforce Wall WS1	d Concrete F 1, Panels 1-3	Retaining	Wall Des	sign		COMP. CHECK		ASP LNB	DAT	
	_ 700 γ _e	204	_	700 x	1.00)	2.0	v 0.00	_	74.40	-		
S _{max}	$= \frac{\beta_{s} f_{e}}{\beta_{s} f_{ss}}$	— - 2.0 d _c	=	700 x 1.22 x	7.29) -	2.0	X 2.38	=	74.12	n		
S _{max}	_{ix} = 74.12	in >	12.00	in <u>OK</u>									
Wall Stem Des	sign - Check Reint	forcement	Limits										
Check Mi	linimum Reinforce	ement	[5.7.3.3.2]]									
Dete	termine the crackin	ng moment:											
	$f_r = 0.24 (f_c')^{0.5}$	=	0.24 x	(4.0) 0.50	=	0.48	ksi						
	l _g = (1/12) b h ³	= 0	.0833 x	12.00	х (18.00) ³ =	5832.0 in	4				
	$y_t = (1/2) h$	= 0	.5000 x	18.00	=	9.00	in						
M _{CF}	$_{R}$ = $\gamma_1 \gamma_3 f_r I_g / y_t$	=	0.67 x	1.6	x	0.48	x	5832.0	/(9.00	x	12.00)= 27.7	9 k-ft
The	e capacity of the se	ection must	be greater th	han or equal	to the sn	naller of:							
	M _{CR} = (4/3) M _u =	1.33		27.79 = 23.93 =	27.7 31.9		<u>GOVE</u>	RNS					
The	e capacity of the re	einforcemen ⁴	is:										
	$M_r = \phi A_s f_v (d_s \cdot$	- a/2)											
	M _r = 0.90 x	0.44 x	60 x	(15.63		0.44 1.7 x	× x 4.0	<u>60</u> x 12)	x (<u>1</u>)	=	30.30 I	k-ft	
	3	30.30 k-ft	> 2	27.79 k-ft	<u>OK</u>								
Che	eck minimum reinfo	orcement fo	r temperatu	re and shrink	age (5.1	0.8)		A _s =	0.44 in	²/ft >	0.16 i	in ² /ft <u>C</u>	<u> </u>
Use	e #6 bars at 12.00 i	in c/c for wa	II stem back	<u>k face vertica</u>	l reinford	ing.							
A minimu	sign - Shrinkage a um amount of reinfo e and temperature	orcement sh				5.10.8] concrete	elemen	ts to limit the	size of crac	ks associa	ted with cor	ncrete	
	h _{max} = max(t _{wt} , t	t _{wb}) =	18.00	in									
	h _w = 5.88	ft =	70.56	in									
	Min. A _s =	1.30 2 (h _w	h _w h _{max} + h _{max}) f _y	- = -2	1.30 x (x 70 70.56).56 +	x 18.00)) x 60	= 0.1	6 in²/ft		
The	e maximum spacing	ig of reinforc	ement is:										
	h _{min} = min(t _{wt} , t _w	_{vb}) =	18.00	in									
	Max. spacing = or Max. spacing or Max. spacing	g = 18 in	walls and fo	ootings ≥ 18 i	in thick)	= = =	54.0 18.0 12.0	0 in	<u>OVERNS</u>				
	1 0											2.0	_
Try:		12.00	in c/c					A _s =	0.20 in	²/ft >	0.16 i	in²/ft <u>C</u>	<u>DK</u>
Try:		12.00	in c/c					-	0.20 in < A _s =		0.16 n ² /ft <	-	<u> </u>

Use #4 bars at 12.00 in c/c for wall stem front face reinforcing and back face horizontal reinforcing.

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	PROJECT CUY-271-0.00 (PID 80418) SUBJECT Reinforced Concrete Retaining Wall Design Wall WS1, Panels 4-6	COMP. BY CHECKED BY	ASP LNB	DATE DATE	3/17/2014 3/17/2014
Dimensions and Weights for Co	oncrete Design				

Footing width, w _{foot} =	9.50	ft	=	114.00	in	Concrete weight, w _c =	0.150	kcf	
	0.00	n		114.00		Water weight, $w_w =$	0.062	kcf	
Footing heel width, wheel =	6.00	ft				Saturated soil weight, w _{ss} =		kcf	
Footing heel height, h _{heel} =	1.50	ft	=	18.00	in	Buoyant soil weight, w _{sb} =	0.068	kcf	
Footing toe width, wree =	2.00	ft				Height of wall, $h_w =$	6.92	ft	top of heel to top of wall
Footing toe height, h _{toe} =	1.50	ft	=	18.00	in	Height of water, h _{water} =	0.00	ft	top of heel to water line
Wall width at top, t_{wt} =	1.50	ft	=	18.00	in	Height of soil, h _s =	6.92	ft	top of heel to ground line
Wall width at base, t_{wb} =	1.50	ft	=	18.00	in	Height of satur. soil, h _{ss} =	6.92	ft	height of satur. soil above top of heel
						Height of buoy. soil, h _{sb} =	0.00	ft	height of buoy. soil above top of heel
Concrete strength, fc' =	4.00	ksi							
Rebar strength, f _y =	60.00	ksi				Active pressure coeff., $K_a =$	0.280		
Steel mod. of elast., E_s =	29,000	ksi				LL surcharge soil ht., h_{LL} =	3.97	ft	

Optional Collision Loading for Barrier on Top of Wall Per ODOT comments for I-70/71, use the transverse loading of 54 kips for a TL-4 test level railing [AASHTO, Table A13.2-1] distributed over the retaining wall's joint spacing.

Collision Loading =	У	(y or n)
Joint Spacing =	28.00	ft
Barrier Height =	3.50	ft

Design Summary

Summary of Design Status

Design Item	Foo	oting	Wall	Footing:	Top transverse:	#8 bars at ?
Designitem	Heel	Toe	Stem		Bottom transverse:	#4 bars at
Shear	OK	OK	OK		Longitudinal:	#4 bars at
Minimum Reinforcement	OK	OK	OK			
Shrinkage & Temperature	OK	OK	OK	Wall Stem:	Back face vertical:	#6 bars at 7
Crack Control	N/A	N/A	OK		Front face vertical:	#4 bars at

For calculations related to each design item, see below.

Design Footing for Shear

Design footings to have adequate shear capacity without transverse reinforcement.

[5.13.3.6]

Determine d_v

Assume: For the he	<pre>#8 bars at #4 bars at eel:</pre>	12.00 12.00	$A_s = 0.79 \text{ in}^2/\text{ft}$ $A_s = 0.20 \text{ in}^2/\text{ft}$			2.0 in cover 3.0 in cover							
	d _{sheel} = h - cover -	- d _{bar} / 2	=	18.00	-	2.00	- (1.000	/2)	=	15.50	in	
	$a_{heel} = \frac{A_s}{0.85}$	f _y f _c ' b	=	0.85	0.79 x	x 60 4.0 x	12			=	1.16	in	
[5.8.2.9]	$d_{vheel} = d_{sheel} - a /$ or $d_{vheel} = 0.90 d_{e}$ or $d_{vheel} = 0.72 h$		= = =	15.50 0.90 0.72	- (x x	1.16 15.50 18.00	/2)			= = =	14.92 13.95 12.96	in in in	<u>GOVERNS</u>
For the toe	e:												
	d _{stoe} = h - cover -	d _{bar} / 2	=	18.00	-	3.00	- (0.500	/2)	=	14.75	in	
	dtoo -	f _y f _c ' b	=	0.85		x 60 4.0 x	12			=	0.29	in	
[5.8.2.9]	$d_{vtoe} = d_{stoe} - a / 2$ or $d_{vtoe} = 0.90 d_e$		= =	14.75 0.90	- (x	0.29 14.75	/2)			= =	14.60 13.28	in in	<u>GOVERNS</u>

Reinforcing Steel Summary

Footing:	Top transverse: Bottom transverse: Longitudinal:	#8 bars at 12.00 in c/c #4 bars at 12.00 in c/c #4 bars at 12.00 in c/c
Vall Stem:	Back face vertical: Front face vertical: Horizontal:	#6 bars at 12.00 in c/c #4 bars at 12.00 in c/c #4 bars at 12.00 in c/c

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or $d_{vtoe} = 0.72 h$ = 0.72 x 18.00 = 12.96 in

Check Heel for Shear

The critical shear section for the heel of the footing is located at the back face of the wall. The heel of the footing is assumed to carry its self weight and the rectangular soil block above it. This neglects the benefit of any upward soil pressure below the footing (conservative).

 $V_{u} = (\gamma_{EV} w_{ss} h_{ss} + \gamma_{EV} w_{sb} h_{sb} + \gamma_{DC} w_{c} h_{heel} + \gamma_{LL} w_{ss} h_{LL} + \gamma_{WA} w_{w} h_{water}) x w_{heel}$

V _u =	(1.35	х	0.130	х	6.92	+	1.35	х	0.068	х	0.00	+	1.25 x	0.15	50 x	1.50	+
	1.75	х	0.130	х	3.97	+	1.00	х	0.062	х	0.00) x	6.00	=	14.39	k/ft	

[5.8.3.3] Using β = 2.00 and assuming bars in the top mat as above:

 $\phi V_{c} = \phi 0.0316 \beta (f_{c})^{0.5} b_{v} d_{v}$ [5.8.3.4] [5.8.2.9] $\phi V_c =$ 0.0316 x (4.0)^{0.50} x 12.00 20.37 k 0.90 х 2.00 14.92 х х = 20.37 k > 14.39 k <u>OK</u>

Check Toe for Shear

The peak bearing stress is 2.27 ksf for the Extreme IIb load case.

The critical section for the toe of the footing is at dv from the front face of the wall. For a quick simplified check, try applying the peak bearing stress over the entire length of the toe (conservative).

 $V_u = \sigma_V w_{toe}$ = 2.27 x 2.00 = 4.54 k/ft

[5.8.3.3] Using β = 2.00 and assuming bars in the bottom mat as above:

 $\phi V_{c} = \phi 0.0316 \beta (f_{c})^{0.5} b_{v} d_{v}$ [5.8.3.4] [5.8.2.9] $\phi V_c =$ x (4.0)^{0.50} x 0.90 х 0.0316 х 2.00 12.00 х 14.60 = 19.93 k 19.93 4.54 <u>OK</u> k > k

Design Footing Reinforcement [5.13.3.4]

Each mat of reinforcement is checked to ensure that it has adequate capacity and that the maximum and minimum reinforcement checks are satisfied. The critical section for flexure in the footing is at the face of the wall.

Top Transverse Reinforcement

For

From the shear check of the heel, V_{μ} = 14.39 k/ft

 $M_u = V_u x (w_{heel} / 2) = 14.39 x (6.00 / 2) = 43.18 k-ft$

Set up the equation to solve for the required steel area:

$$\begin{split} M_{u} &= \phi M_{n} = \phi A_{s} f_{y} (d_{s} - a/2) &= \phi A_{s} f_{y} (d_{s} - \frac{A_{s} f_{y}}{1.7 f_{c}' b}) \\ M_{u} &= 0.90 \quad x A_{s} x \quad 60 \quad (ds - \frac{As \quad x \quad 60}{1.7 \quad x \quad 4.0 \quad x \quad 12}) x (\frac{1}{12}) \\ 3.309 \quad A_{s}^{2} \quad - \quad 4.50 \quad d_{s} A_{s} + M_{u} = 0 \\ \end{split}$$
the reinforcing steel assumed for the heel, $d_{s} = 15.50$ in

Substituting and solving for A_s, it is found that required A_s = $0.64 \text{ in}^2/\text{ft}$

Try: #8 bars at 12.00 in c/c for the top transverse bars in the heel $As = 0.79 \text{ in}^2/\text{ft}$

Check Minimum Reinforcement [5.7.3.3.2]

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Determine the cracking moment:						
$f_r = 0.24 (f_c')^{0.5}$ = 0.24 x (4.0) ^{0.50} = 0.48 ksi						
$I_g = (1/12) b h^3 = 0.0833 x 12.00 x (18.00)^3 = 5832.0$	in ⁴					
$y_t = (1/2) h$ = 0.5000 x 18.00 = 9.00 in						
$M_{CR} = \gamma_1 \gamma_3 f_r I_g / y_t$ = 1.60 x 0.67 x 0.48 x 5832.0	/(9.00 x	12.00) =	27.79 k-ft			
The capacity of the section must be greater than or equal to the smaller of:						
M_{CR} = 27.79 = 27.79 k-ft <u>GOVERNS</u> (4/3) M_u = 1.33 x 43.18 = 57.57 k-ft						
The capacity of the top mat of reinforcement is:						
$M_r = \phi A_s f_y (d_s - a/2)$						
For the reinforcing steel used, $d_s = 18.00 - 2.00 - (1.000 / 2)$	= 15.50 in					
$M_{r} = 0.90 \times 0.79 \times 60 \times (15.50 - \frac{0.79 \times 60}{1.7 \times 4.0 \times 12})$	$(-) x (-\frac{1}{12}) =$	53.04 k-ft				
53.04 k-ft > 27.79 k-ft <u>OK</u>	0.79 in ² /ft >	0.17 in ² /ft				
Check minimum reinforcement for temperature and shrinkage (5.10.8) $A_s =$	0.79 in ⁻ /ft >	0.17 in ⁻ /ft	<u>OK</u>			
Use #8 bars at 12.00 in c/c for top transverse reinforcement in the footing.						
Bottom Transverse Reinforcement						
From the shear check of the toe, $V_u = 4.54$ k/ft						
$M_u = V_u x (w_{toe} / 2) = 4.54 x (2.00 / 2) = 4.54 k-ft$						
Set up the equation to solve for the required steel area and again use:						
$3.309 A_s^2 - 4.50 d_s A_s + M_u = 0$						
For the reinforcing steel assumed for the heel, $d_s = 14.75$ in						
Substituting and solving for A_s , it is found that required $A_s = 0.07 \text{ in}^2/\text{ft}$						
Try: #4 bars at 12.00 in c/c for the bottom transverse bars in the toe As =	0.20 in ² /ft					
Check Minimum Reinforcement [5.7.3.3.2]						
Determine the cracking moment:						
$f_r = 0.24 (f_c')^{0.5} = 0.24 \times (4.0)^{0.50} = 0.48 \text{ ksi}$						
$l_g = (1/12) b h^3 = 0.0833 x 12.00 x (18.00)^3 = 5832.0$	in ⁴					
$y_t = (1/2) h$ = 0.5000 x 18.00 = 9.00 in						
$M_{CR} = \gamma_1 \gamma_3 f_r I_g / y_t$ = 1.60 x 0.67 x 0.48 x 5832.0	/(9.00 x	12.00) =	27.79 k-ft			
The capacity of the section must be greater than or equal to the smaller of:						
M_{CR} = 27.79 = 27.79 k-ft (4/3) M_u = 1.33 x 4.54 = 6.05 k-ft <u>GOVERNS</u>						
The capacity of the bottom mat of reinforcement is:						
$M_r = \phi A_s f_y (d_s - a/2)$						

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PROJECT CUY-271-0.00 (PID 80418) SUBJECT Reinforced Concrete Retaining Wall Design Wall WS1, Panels 4-6	COMP. BYASPDATE3/17/2014CHECKED BYLNBDATE3/17/2014
For the reinforcing steel used, $d_s = 18.00 - 3.00 - (0.500 / 2)$	= 14.75 in
$M_r = 0.90 \times 0.20 \times 60 \times (14.75 - \frac{0.20 \times 60}{1.7 \times 4.0 \times 12})$	$) x \left(\frac{1}{12} \right) = 13.14$ k-ft
13.14 k-ft > 6.05 k-ft <u>OK</u>	
Check minimum reinforcement for temperature and shrinkage (5.10.8) $A_s =$	0.20 in ² /ft > 0.17 in ² /ft <u>OK</u>
Use #4 bars at 12.00 in c/c for bottom transverse reinforcement in the footing.	
Longitudinal Reinforcement [5.10.8]	
Provide longitudinal reinforcement in the footing based on shrinkage and temperature requirem	nents.
$h_{max} = max(h_{heel}, h_{toe}) = 18.00$ in	
Min. A _s = $\frac{1.30 \text{ w}_{foot} \text{ h}_{max}}{2 (\text{ w}_{foot} + \text{ h}_{max}) \text{ f}_y}$ = $\frac{1.30 \text{ x} 114.00 \text{ x} 18.00 \text{ x}}{2 \text{ x} (114.00 \text{ x} 18.00 \text{ x})}$	$\frac{00}{x 60} = 0.17 \text{ in}^2/\text{ft}$
The maximum spacing of reinforcement is:	
$h_{min} = min(h_{heel}, h_{toe}) = 18.00$ in	
Max. spacing = 3 h_{min} =54.00inor Max. spacing = 18 in=18.00inor Max. spacing = 12 in (for walls and footings \geq 18 in thick)=12.00in	GOVERNS
Try: #4 bars at 12.00 in c/c for the top and bottom longitudinal bars $A_s =$	0.20 in ² /ft > 0.17 in ² /ft <u>OK</u>
0.11	$< A_s = 0.20 \text{ in}^2/\text{ft} < 0.60 \text{ OK}$
spacir	ng = 12.00 in = 12.00 in <u>OK</u>
Use #4 bars at 12.00 in c/c for top and bottom longitudinal reinforcement in the footing.	
Determine Loads for Wall Stem Design	
The loads on the stem at the top of the footing are determined to arrive at the design forces for the w	all.
Saturated Earth Pressure:	
$P_{EH(S)} = (1/2) w_{ss} K_a h_{ss}^2 = 0.5 x 0.130 x 0.280 x (6.5)$	92) ² = 0.87 k
$M_{EH(S)} = P_{EH(S)} x [(1/3) h_{ss} + h_{sb}] = 0.87 x [0.333 x 6.9]$	92 + 0.00] = 2.01 k-ft
Buoyant Earth Pressure:	
$P_{EH(B)} = K_a h_{sb} [w_{ss} h_{ss} + (1/2) w_{sb} h_{sb}]$	
P _{EH(B)} = 0.280 x 0.00 x [0.130 x 6.92 + 0.5	x 0.068 x 0.00] = 0.00 k
y_{B} = [h_{sb} (w_{ss} h_{ss} + (1/3) w_{sb} h_{sb})] / (2 w_{ss} h_{ss} + w_{sb} h_{sb})	
$y_B = \begin{bmatrix} 0.00 & x & (0.130 & x & 6.92 & + & 0.333 & x & 0.0 \\ / & (2.0 & x & 0.130 & x & 6.92 & + & 0.068 & x & 0.02 \end{bmatrix}$	
$M_{EH(B)} = P_{EH(B)} \times y_B = 0.00 \times 0.00$	= 0.00 k-ft
Water Pressure:	
$P_{WA} = (1/2) w_w h_{water}^2 = 0.5 x 0.062 x (0.00)^2 =$	0.00 k
$M_{WA} = P_{WA} x (1/3) h_{water} = 0.00 x 0.333 x 0.00 =$	0.00 k-ft
Live Load Surcharge:	
$P_{LS} = w_{ss} K_a h_{LL} h_s$ = 0.130 x 0.280 x 3.97 x	6.92 = 1.00 k

D	1.7	PI	LIENT ROJEC UBJEC		271-0.00			n <u>g</u> Wal	l Design				PROJE(Comp.		A	SP		22-1001-00 DATE <u>3/17/201</u>		
		_			NS1, Pa								CHECK	ED BY	11	NB	DATE	3	3/17/20	
	$M_{LS} = P_{LS} x$	(1/2) h _a		=	1.00	x	0	500	x	6.92	=			3.46	k-ft					
Coll	lision Load at 1			:																
	Use a Live L	oad of		1929	lbs/ft ap	plied at	h _r =	3.5	ft above	the top	of the v	vall.								
	P _{CT} =													1.93	k					
	M _{CT} = P _{CT} x	(h _w + h _r	r)	=	1.93	x	(6	.92	+	3.50) =			20.10	k-ft					
Usir	ng the Strength	n I load	combin	nation, the	e factore	d desigi	n force:	s for the	e wall ste	em are:										
	H _u = 1.50 (P	е _{ЕН(S)} + Г	P _{EH(B)}) -	+ 1.00 P _v	_{VA} + 1.75	5 (P _{LS} +	P _{LL})													
	H _u = 1.50	х (0.87	+	0.00) +	1.00	x	0.00	+	1.75	x (1.00	+	0.00) =	3.06	k		
	M _u = 1.50 (N	И _{ЕН(S)} +	M _{EH(B)})	+ 1.00 N	1 _{WA} + 1.7	75 (M _{LS} ·	+ M _{LL})													
	M _u = 1.50	х (2.01	+	0.00) +	1.00	x	0.00	+	1.75	x (3.46	+	0.00) =	9.07	k-ft		
The Extre	me Event II de	sign for	rces for	r the wall	stem are	e:														
H _{ext}	= 1.50 (P _{EH(S)}	+ P _{EH(B}	s) + 1.0	0 P _{CT} + 0).50 (P _{LS}	+ P _{LL})														
H _{ext}	= 1.50	х (0.87	+	0.00) +	1.00	x	1.93	+	0.50	х (1.00	+	0.00) =	3.74	k		
M _{ext}	t = 1.50 (M _{EH(S)}	+ M _{EH(I}	_{B)}) + 1.0	00 M _{CT} +	0.50 (M _l	_s + M _{LL})													
M _{ext}	t = 1.50	х (2.01	+	0.00) +	1.00	x	20.10	+	0.50	х (3.46	+	0.00) =	24.84	k-ft		
The	service desig	n forces	s for the	e wall ste	m are:															
	H _{serv} = 1.00	(P _{EH(S)} -	+ P _{EH(B)}	₎) + 1.00	P _{WA} + 1.	00 (P _{LS}	+ P _{LL})													
	H _{serv} =	1.00	х (0.87	+	0.00) +	1.00	x	0.00	+	1.00	х (1.00	+	0.00) =	1.87	k	
	M _{serv} = 1.00	(M _{EH(S)}	+ M _{EH(E}	_{B)}) + 1.00	M _{WA} + 1	1.00 (M _L	s + M _{LL})												
	M _{serv} =	1.00	х (2.01	+	0.00) +	1.00	x	0.00	+	1.00	х (3.46	+	0.00) =	5.47	k-ft	
Stom Dos	ign - Investiga	ata Sha	ər																	
Shear typ	ically does not verse reinforce	govern	the de	0	etaining v	walls. If	shear	does be	ecome a	in issue	the thi	ckness	of the s	tem sho	ould be inc	creased	such			
Igno	oring the benef	its of th	e shear	r key and	axial co	mpress	ion, the	e shear	capacity	/ of the :	stem ca	in be s	hown to	be grea	iter than th	nat requi	red.			
	$V_n = V_c + V_s$	+ Vp				[5.8	.3.3-1]													
Rec	cognizing that \	/ $_{\rm s}$ and \	V _p are z	zero																
	$V_n = V_c$																			
	V _c = 0.0316	β (f _c ') ^{0.}	$^{.5}$ b _v d _v			[5.8.	.3.3-3]													
The	e maximum effe	ective sl	hear de	epth is:																
	For 2.0 i	inch cle	ar cove	er and	#6 ba	ars, d _s =	18	8.00	-	2.00	- (0.7	50 / 2	:) =	15.63	in in				
	d _v = 0.9 d _e = d _v = 0.72 h =	-		0.90 0.72		15.63 18.00	= =	14. 12.		<u>GO</u>	/ERNS									
Foll	ow the Genera	I Proce	dure us	sing the p	provision	s of App	endix l	B5, as	per Sect	ion [5.8	3.4.2]:									

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	COMP. BY	COMP. BY ASP	COMP. BY ASP DATE

18.90 k > 3.74 k <u>OK</u>

Wall Stem Design - Investigate Strength Limit State

Determine the area of back-face flexural reinforcement necessary to satisfy the design moment:

M_u = 24.84 k-ft

Again use the equation:

 $3.309 \quad A_s^2 \quad - \quad 4.50 \quad d_s A_s + M_u = 0$

Substituting and solving for A_s , it is found that required $A_s = 0.36$ in²/ft

Try: #6 bars at 12.00 in c/c $A_s = 0.44 \text{ in}^2/\text{ft} > 0.36 \text{ in}^2/\text{ft} \frac{OK}{S}$

Wall Stem Design - Investigate Service Limit State [5.7.3.4]

Check the crack control equations to ensure that the primary flexural reinforcement is well distributed. The service load bending moment is:

M_{serv} = 5.47 k-ft

Check the modulus of rupture for concrete:

$$\phi f_r = \phi \ 0.24 \ (f_c)^{0.5} = 0.80 \ x \ 0.24 \ x \ (4.0)^{0.50} = 0.384 \ \text{ksi}$$

$$S = \frac{b \ t_{wb}^2}{6} = \frac{12.00 \ x \ (18.00)^2}{6} = 648.00 \ \text{in}^3$$

$$f_{act} = \frac{M_{serv}}{S} = \frac{5.47 \ x \ 12}{648.00} = 0.101 \ \text{ksi} < 0.384 \ \text{ksi} \ OK$$

If modulus of rupture check is "NG", then check the spacing of the reinforcement. First, determine the modular ratio:

$$E_c = 33,000 w_c^{1.5} (f_c^{-1})^{0.5} = 33,000 x (0.150)^{1.5} x (4.00)^{0.5} = 3,834$$
 ksi
n = $E_s/E_c = 29,000$ / 3,834 = 7.56, use n = 8.00

For 2.0 inch clear cover and #6 bars, $d_s = 18.00 - 2.00 - (0.750 / 2) = 15.63$ in

Determine the location of the neutral axis:

0.5 b $x^2 = n A_s (d_s - x)$ 0.5 (12) $x^2 = 8 \times 0.44 \times (15.63 - x)$ solving, x = 2.75 in

Check the spacing of the reinforcement to control cracking:

$$y = d_{s} - x = 15.63 - 2.75 = 12.88 \text{ in}$$

$$l_{cr} = \frac{b x^{3}}{3} + n A_{s} (d_{s} - x)^{2}$$

$$l_{cr} = \frac{-12.00 \times (2.75)^{3}}{3} + 8 \times 0.44 \times (15.63 - 2.75)^{2} = 666.68 \text{ in}^{4}$$

$$\gamma_{e} = 1.00$$
For 2.0 inch clear cover and #6 bars, $d_{c} = 2.00 + (0.750/2) = 2.38 \text{ in}$

$$\beta_{s} = 1.0 + \frac{d_{c}}{0.7 (t_{wb} - d_{c})} = 1.0 + \frac{2.38}{0.7 \times (18.00 - 2.38)} = 1.22$$

$$f_{ss} = n \frac{M_{serv} y}{l_{cr}} = 8 \times \frac{5.47 \times 12.88 \times 12.00}{666.68} = 10.14 \text{ ksi}$$

	T 7	CLIENT PROJE	CT CUY-27	71-0.00 (PID	80418)								22-1001-0	
		SUBJE	CT <u>Reinfor</u> Wall W	ced Concret S1, Panels	e Retainin 1-6	g Wall De	sign		COMP CHEC	. BY KED BY	ASF LNE		DATE DATE	3/17/20 3/17/20
s	700 γ _e	200	4 <u> </u>	700 1.22	x 1.0	00	2.0	v 0.3	8 =	51.95	in			
Smax	$\beta_{s} = \frac{\beta_{s} \sigma_{f_{ss}}}{\beta_{s} f_{ss}}$	2.0 0	u _c – —	1.22	x 10.	14 -	2.0	X 2.3	D –	51.95				
S _{max}	, = 51.95	in	> 12.0	0 in <u>O</u>	K									
Wall Stem Desi	ign - Check Reinf	forcemen	nt Limits											
Check Mi	inimum Reinforce	ement	[5.7.3.3	.2]										
Dete	ermine the crackin	ıg momen	ıt:											
	$f_r = 0.24 (f_c')^{0.5}$	=	0.24	x (4.0)	^{0.50} =	0.48	ksi							
	$I_g = (1/12) b h^3$	=	0.0833	x 12.00) х (18.00) ³ =	5832.0 ii	1 ⁴					
	y _t = (1/2) h	=	0.5000	x 18.00) =	9.00	in							
M _{CR}	$_{\rm R}$ = $\gamma_1 \gamma_3 f_r I_g / y_t$	=	0.67	x 1.6	x	0.48	x	5832.0	/(9.0	x C	12.00) =	27.79	k-ft
The	capacity of the se	ection mus	st be greater	r than or equ	al to the s	maller of:								
	M _{CR} = (4/3) M _u =	1.33	x		= 27. = 33.		<u>GOVE</u>	<u>RNS</u>						
The	capacity of the rei	inforceme	ent is:											
	$M_r = \phi A_s f_y (d_s -$	- a/2)												
	M _r = 0.90 x	0.44	x 60	x (15.63	3	0.44 1.7 x	4 x 4.0	60 x 12) x (<u>1</u>)	=	30.30	k-ft		
	3'	80.30 k-	-ft >	27.79 k-	ft <u>OK</u>									
Che	eck minimum reinfo	orcement	for tempera	ture and shr	inkage (5.	10.8)		A _s =	0.44 ii	n²/ft >	0.16	in²/ft	<u>OK</u>	I
Use	#6 bars at 12.00 i	<u>in c/c for v</u>	wall stem ba	ick face vert	ical reinfor	cing.								
A minimur	ign - Shrinkage a m amount of reinfo and temperature	orcement	should be p			[5.10.8] of concrete	e elemen	ts to limit the	e size of cra	cks assoc	iated with c	oncrete		
	$h_{max} = max(t_{wt}, t_{wt})$	wb)	= 18.0	0 in										
	h _w = 6.92	ft	= 83.04	4 in										
	Min. A _s =	1.3 2 (h	30 h _w h _{max} n _w + h _{max}) f _y	=	1.30 2 x (x 8 83.04	3.04 +	x 18.0 18.00) x 60	= 0.	16 in²/f	t		
The	maximum spacing	g of reinfo	prcement is:											
	$h_{min} = min(t_{wt}, t_w)$	_{/b})	= 18.0	0 in										
	Max. spacing = or Max. spacing or Max. spacing	j = 18 in	or walls and	l footings ≥ ´	l8 in thick)	= = =	54.0 18.0 12.0	0 in	GOVERNS					
	#4 bars at	10.01) in c/c					A _s =	0.20 ii	∩²/ft >	0.16	in²/ft	OK	
Try:	m4 Dais at	12.00						3						
Try:	^{m4} Dais at	12.00						° 0.11	< A _s =		in²/ft <	0.60	<u>OK</u>	

Use #4 bars at 12.00 in c/c for wall stem front face reinforcing and back face horizontal reinforcing.

	CLIENT ODOT PROJECT CUY-271-0.00 (PID 80418)	PROJECT NO.		1122-1001-0	0
	SUBJECT Reinforced Concrete Retaining Wall Design Wall WS1, Panels 7-8	COMP. BY CHECKED BY	ASP LNB	DATE DATE	3/17/2014 3/17/2014
Dimensions and Weights for Co					

Footing width, w _{foot} =	9.50	ft	=	114.00	in	Concrete weight, w _c =	0.150	kcf	
						Water weight, w _w =	0.062	kcf	
Footing heel width, w _{heel} =	6.00	ft				Saturated soil weight, w_{ss} =	0.130	kcf	
Footing heel height, h _{heel} =	1.50	ft	=	18.00	in	Buoyant soil weight, w _{sb} =	0.068	kcf	
Footing toe width, $w_{toe} =$	2.00	ft				Height of wall, h _w =	7.82	ft	top of heel to top of wall
Footing toe height, h _{toe} =	1.50	ft	=	18.00	in	Height of water, h _{water} =	0.00	ft	top of heel to water line
Wall width at top, t_{wt} =	1.50	ft	=	18.00	in	Height of soil, h _s =	7.82	ft	top of heel to ground line
Wall width at base, t_{wb} =	1.50	ft	=	18.00	in	Height of satur. soil, h _{ss} =	7.82	ft	height of satur. soil above top of heel
						Height of buoy. soil, h _{sb} =	0.00	ft	height of buoy. soil above top of heel
Concrete strength, fc' =	4.00	ksi							
Rebar strength, f _y =	60.00	ksi				Active pressure coeff., $K_a =$	0.280		
Steel mod. of elast., E_s =	29,000	ksi				LL surcharge soil ht., h _{LL} =	3.70	ft	

Optional Collision Loading for Barrier on Top of Wall

Per ODOT comments for I-70/71, use the transverse loading of 54 kips for a TL-4 test level railing [AASHTO, Table A13.2-1] distributed over the retaining wall's joint spacing.

у	(y or n)
28.00	ft
3.50	ft

Design Summary

Summary of Design Status

Ton transverse: #8 bars at 12 00 in c/c Footing Wall Footing: Design Item Stem Heel Toe OK OK Shear OK Minimum Reinforcement OK OK OK Shrinkage & Temperature Crack Control OK OK OK W OK N/A N/A

For calculations related to each design item, see below.

Design Footing for Shear

Design footings to have adequate shear capacity without transverse reinforcement.

[5.13.3.6]

Determine d_v

Assume: For the he	#8 bars at #4 bars at	12.00 12.00		for the top t for the botto						0.79 i 0.20 i			0 in cover 0 in cover
		d ()		40.00		0.00	,	4 000			45 50		
	d _{sheel} = h - cover -	· d _{bar} / 2	=	18.00	-	2.00	- (1.000	/2)	=	15.50	in	
	$a_{heel} = \frac{A_s}{0.85}$	f _y f _c ' b	=	0.85	0.79 x	x 60 4.0 x	12			=	1.16	in	
[5.8.2.9]	d _{vheel} = d _{sheel} - a /	2	=	15.50	- (1.16	/2)			=	14.92	in	GOVERNS
	or d_{vheel} = 0.90 d_e		=	0.90	х	15.50				=	13.95	in	
	or d_{vheel} = 0.72 h		=	0.72	х	18.00				=	12.96	in	
For the toe	9:												
	d_{stoe} = h - cover -	d _{bar} / 2	=	18.00	-	3.00	- (0.500	/2)	=	14.75	in	
	$a_{toe} = \frac{A_s}{0.85}$	f _y f _c ' b	=	0.85	0.20 x	x 60 4.0 x	12			=	0.29	in	
[5.8.2.9]	d _{vtoe} = d _{stoe} - a / 2		=	14.75	- (0.29	/2)			=	14.60	in	GOVERNS
	or d_{vtoe} = 0.90 d_e		=	0.90	х	14.75				=	13.28	in	

Reinforcing Steel Summary

ooting.	Bottom transverse: Longitudinal:	#4 bars at 12.00 in c/c #4 bars at 12.00 in c/c
Wall Stem:	Back face vertical: Front face vertical: Horizontal:	#6 bars at 12.00 in c/c #4 bars at 12.00 in c/c #4 bars at 12.00 in c/c

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SUBJECT Reinforced Concrete Retaining Wall Design COMP. BY Wall WS1, Panels 7-8 CHECKED BY	ASP	DATE	3/17/2014
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or $d_{vtoe} = 0.72 h$ = 0.72 x 18.00 = 12.96 in

Check Heel for Shear

The critical shear section for the heel of the footing is located at the back face of the wall. The heel of the footing is assumed to carry its self weight and the rectangular soil block above it. This neglects the benefit of any upward soil pressure below the footing (conservative).

 $V_{u} = (\gamma_{EV} w_{ss} h_{ss} + \gamma_{EV} w_{sb} h_{sb} + \gamma_{DC} w_{c} h_{heel} + \gamma_{LL} w_{ss} h_{LL} + \gamma_{WA} w_{w} h_{water}) x w_{heel}$

V _u =	(1.35	х	0.130	х	7.82	+	1.35	х	0.068	х	0.00	+	1.25 x	0.15	0 x	1.50	+
	1.75	х	0.130	х	3.70	+	1.00	х	0.062	х	0.00) x	6.00	=	14.97	k/ft	

[5.8.3.3] Using β = 2.00 and assuming bars in the top mat as above:

 $\phi V_{c} = \phi 0.0316 \beta (f_{c})^{0.5} b_{v} d_{v}$ [5.8.3.4] [5.8.2.9] $\phi V_c =$ 0.0316 x (4.0)^{0.50} x 20.37 k 0.90 х 2.00 12.00 14.92 х х = 20.37 k > 14.97 k <u>OK</u>

Check Toe for Shear

The peak bearing stress is 2.42 ksf for the Extreme IIb load case.

The critical section for the toe of the footing is at dv from the front face of the wall. For a quick simplified check, try applying the peak bearing stress over the entire length of the toe (conservative).

 $V_u = \sigma_V w_{toe}$ = 2.42 x 2.00 = 4.84 k/ft

[5.8.3.3] Using β = 2.00 and assuming bars in the bottom mat as above:

 $\phi V_{c} = \phi 0.0316 \beta (f_{c})^{0.5} b_{v} d_{v}$ [5.8.3.4] [5.8.2.9] $\phi V_c =$ x (4.0)^{0.50} x 0.90 х 0.0316 х 2.00 12.00 х 14.60 = 19.93 k 19.93 4.84 <u>OK</u> k > k

Design Footing Reinforcement [5.13.3.4]

Each mat of reinforcement is checked to ensure that it has adequate capacity and that the maximum and minimum reinforcement checks are satisfied. The critical section for flexure in the footing is at the face of the wall.

Top Transverse Reinforcement

For

From the shear check of the heel, V_{μ} = 14.97 k/ft

 $M_u = V_u x (w_{heel} / 2) = 14.97 x (6.00 / 2) = 44.92 k-ft$

Set up the equation to solve for the required steel area:

$$\begin{split} M_{u} &= \phi M_{n} = \phi A_{s} f_{y} (d_{s} - a/2) &= \phi A_{s} f_{y} (d_{s} - \frac{A_{s} f_{y}}{1.7 f_{c}' b}) \\ M_{u} &= 0.90 \quad x A_{s} x \quad 60 \quad (ds - \frac{As \quad x \quad 60}{1.7 \quad x \quad 4.0 \quad x \quad 12}) x (\frac{1}{12}) \\ &3.309 \quad A_{s}^{2} \quad - \quad 4.50 \quad d_{s} A_{s} + M_{u} = 0 \\ \end{split}$$
the reinforcing steel assumed for the heel, $d_{s} = 15.50$ in

Substituting and solving for A_s , it is found that required $A_s = 0.66 \text{ in}^2/\text{ft}$

Try: #8 bars at 12.00 in c/c for the top transverse bars in the heel $As = 0.79 \text{ in}^2/\text{ft}$

Check Minimum Reinforcement [5.7.3.3.2]

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PROJECT CUY-271-0.00 (PID 80418) SUBJECT SUBJECT Reinforced Concrete Retaining Wall Design Wall WS1, Panels 7-8	COMP. BY	ASP LNB	DATE <u>3/17/2014</u> DATE <u>3/17/2014</u>		
Determine the cracking moment:					
$f_r = 0.24 (f_c)^{0.5} = 0.24 x (4.0)^{0.50} = 0.48 ksi$					
$I_g = (1/12) b h^3 = 0.0833 x 12.00 x (18.00)^3 = 5832.0 i$	n⁴				
$y_t = (1/2) h$ = 0.5000 x 18.00 = 9.00 in					
$M_{CR} = \gamma_1 \gamma_3 f_r I_g / y_t = 1.60 x 0.67 x 0.48 x 5832.0$	/(9.00 x	12.00) =	27.79 k-ft		
The capacity of the section must be greater than or equal to the smaller of:					
$\begin{array}{rclcrcl} M_{CR} & = & 27.79 & = & 27.79 & k-ft & \underline{GOVERNS} \\ (4/3) M_u & = & 1.33 & x & 44.92 & = & 59.89 & k-ft \end{array}$					
The capacity of the top mat of reinforcement is:					
$M_r = \phi A_s f_y (d_s - a/2)$					
For the reinforcing steel used, $d_s = 18.00 - 2.00 - (1.000 / 2)$	= 15.50 in				
$M_r = 0.90 x 0.79 x 60 x (15.50 - \frac{0.79 x 60}{1.7 x 4.0 x 12}$	$) x (\frac{1}{12}) =$	53.04 k-ft			
53.04 k-ft > 27.79 k-ft <u>OK</u>		2			
Check minimum reinforcement for temperature and shrinkage (5.10.8) $A_s =$	0.79 in ² /ft >	0.17 in ² /ft	<u>OK</u>		
Use #8 bars at 12.00 in c/c for top transverse reinforcement in the footing.					
Bottom Transverse Reinforcement					
From the shear check of the toe, $V_u = 4.84$ k/ft					
$M_u = V_u x (w_{toe} / 2) = 4.84 x (2.00 / 2) = 4.84 k-ft$					
Set up the equation to solve for the required steel area and again use:					
3.309 A_s^2 - 4.50 $d_s A_s + M_u = 0$					
For the reinforcing steel assumed for the heel, $d_s = 14.75$ in					
Substituting and solving for A_s , it is found that required $A_s = 0.07 \text{ in}^2/\text{ft}$					
Try: #4 bars at 12.00 in c/c for the bottom transverse bars in the toe As =	0.20 in ² /ft				
Check Minimum Reinforcement [5.7.3.3.2]					
Determine the cracking moment:					
$f_r = 0.24 (f_c)^{0.5} = 0.24 \times (4.0)^{0.50} = 0.48 \text{ ksi}$					
$l_g = (1/12) b h^3 = 0.0833 x 12.00 x (18.00)^3 = 5832.0 i$	n ⁴				
$y_t = (1/2) h$ = 0.5000 x 18.00 = 9.00 in					
$M_{CR} = \gamma_1 \gamma_3 f_r I_g / y_t$ = 1.60 x 0.67 x 0.48 x 5832.0	/(9.00 x	12.00) =	27.79 k-ft		
The capacity of the section must be greater than or equal to the smaller of:					
M_{CR} = 27.79 = 27.79 k-ft (4/3) M_u = 1.33 x 4.84 = 6.45 k-ft <u>GOVERNS</u>					
The capacity of the bottom mat of reinforcement is:					
$M_r = \phi A_s f_y (d_s - a/2)$					

			D 00440)			PROJECT NO		1122-1001-00	1
	ROJECT CUY UBJECT Rein	forced Concre	ete Retaining	Wall Design		COMP. BY	ASP	DATE	3/17/2014
	Wall	WS1, Panels	7-8			CHECKED BY	LNB	DATE	3/17/2014
For the reinforcing steel u	ised, d _s =	18.00	- 3.00	- (0.50	0 /2) =	14.75 in			
M _r = 0.90 x 0	.20 x 60	x (14.7	75 - 1	0.20 x .7 x 4.0	60 x 12) x	(<u>1</u>) =	= 13.14 k-ft		
13.1	4 k-ft >	6.45 I	k-ft <u>OK</u>						
Check minimum reinforce	ement for tempe	erature and sh	nrinkage (5.10	.8)	A _s =	0.20 in ² /ft >	> 0.17 in ² /fi	t <u>OK</u>	
<u>Use #4 bars at 12.00 in c</u>	/c for bottom tra	ansverse reinf	orcement in th	ne footing.					
Longitudinal Reinforcement	[5.10	0.8]							
Provide longitudinal reinfo	prcement in the	footing based	d on shrinkage	e and temperatu	e requirement	S.			
h _{max} = max(h _{heel} , h _{tt}	_{be}) = 18	3.00 in							
	1.30 w _{foot} h _m	iax	1.30	x 114.00	x 18.00		0.47		
Min. A _s = —	2 (w _{foot} + h _{max}	$(f_y) = -$	2 x (114.00 +	18.00) x	<u> </u>	0.17 in ² /ft		
The maximum spacing of	reinforcement	is:							
$h_{min} = min(h_{heel}, h_{tot})$.) = 18	3.00 in							
Max. spacing = 3 h or Max. spacing = 1 or Max. spacing = 1	8 in	and footings >	18 in thick)	= 54.0 = 18.0 = 12.0	0 in	VERNS			
		-				0.20 in ² /ft >	▶ 0.17 in ² /fi	OK	
Try: #4 bars at	12.00	tor the top a	nd bottom long	yiluumai bars	A _s =	0.20 111711 2		t <u>ОК</u>	
					0.11 <	A _s = 0.20	in ² /ft < 0.60) <u>OK</u>	
					spacing =	12.00 in	= 12.00	in <u>OK</u>	
<u>Use #4 bars at 12.00 in c</u>	/c for top and b	ottom longitud	dinal reinforce	ment in the footi	<u>ig.</u>				
Determine Loads for Wall Stem De The loads on the stem at the to	-	are determin	ed to arrive at	the desian force	s for the wall.				
Saturated Earth Pressure				Ū					
P _{EH(S)} = (1/2) w _{ss} K _a		0.5 x	0.130	x 0.280	x (7.82	$)^{2} = 1.11$	k		
М _{ЕН(S)} = Р _{ЕН(S)} х [(1		=		[0.333	x 7.82	+ 0.00] = 2.90	k-ft	
Buoyant Earth Pressure:				[0.000	x 1.02	0.00] 2.00	ĸ'n	
$P_{EH(B)} = K_a h_{sb} [w_{ss}]$	h + (1/2) w.h	. 1							
$P_{EH(B)} = 0.28$		-		x 7.82	+ 0.5 x	0.068 >	0.00] =	0.00 k	K
$y_B = [h_{sb} (w_{ss} h_{ss} +$	(1/3) w _{sb} h _{sb})]	/ (2 w _{ss} h _{ss} +	w _{sb} h _{sb})						
y _B = [0.00 / (2		130 x 130 x		+ 0.333 + 0.068	x 0.068 x 0.00	x 0.00))]	0.00 f	t
$M_{EH(B)} = P_{EH(B)} \times y_B$	= 0	.00 x	0.00			= 0.00	k-ft		
Water Pressure:									
P_{WA} = (1/2) $w_w h_{wate}$	er =	0.5 x	0.062	x (0.00)	=	0.00	k		
M _{WA} = P _{WA} x (1/3) h	n _{water} =	0.00	x 0.333	x 0.0) =	0.00	k-ft		
Live Load Surcharge:									
$P_{LS} = w_{ss} K_a h_{LL} h_s$	_	0 120	v 0.200	V 07		7 82 -	105 k		
Γ _{LS} – w _{ss} r _a n _{ll} n _s	=	0.130	x 0.280	x 3.7) x	7.82 =	1.05 k		

	[7		JECT CUY	′-271-0.00								PROJECT NO				1122-1001-00			
	\mathcal{L}	SUB.	JECT <u>Rein</u> Wall	WS1, Pa			ng Wall	Design				COMP. CHECK			SP NB	DATE DATE		3/17/20 3/17/20	
	M _{LS} = P _{LS} x (1	l/2) h _s	=	1.05	x	0.	500	x	7.82	=			4.12	k-ft					
Collisi	ion Load at To	op of Para	apet:																
	Use a Live Lo	ad of	1929) lbs/ft ap	plied at I	n _r =	3.5 f	t above	the top	of the v	vall.								
	P _{CT} =												1.93	k					
	M _{CT} = P _{CT} x (ł	n _w + h _r)	=	1.93	х	(7.	.82	+	3.50) =			21.83	k-ft					
Using	the Strength	I load cor	nbination, tl	he factore	ed desigr	n force	s for the	e wall st	em are:										
	H _u = 1.50 (P _E	_{H(S)} + P _{EH}	_(B)) + 1.00 F	P _{WA} + 1.7	5 (P _{LS} +	P _{LL})													
	H _u = 1.50	x (1.	.11 +	0.00) +	1.00	x	0.00	+	1.75	х (1.05	+	0.00) =	3.51	k		
	M _u = 1.50 (M _E	_{EH(S)} + M _{EI}	_{H(B)}) + 1.00	M _{WA} + 1.7	75 (M _{LS} +	⊦ M _{LL})													
	M _u = 1.50	x (2.	.90 +	0.00) +	1.00	x	0.00	+	1.75	x (4.12	+	0.00) =	11.56	i k-ft		
The Extrem	e Event II des	ign forces	s for the wa	ll stem ar	e:														
H _{ext} =	1.50 (P _{EH(S)} +	P _{EH(B)}) +	1.00 P _{CT} +	0.50 (P _{LS}	₅ + P _{LL})														
H _{ext} =	1.50	x (1.	.11 +	0.00) +	1.00	x	1.93	+	0.50	х (1.05	+	0.00) =	4.12	k		
M _{ext} =	1.50 (M _{EH(S)} +	⊢ M _{EH(B)}) +	+ 1.00 M _{CT} ·	+ 0.50 (M	_{LS} + M _{LL}))													
M _{ext} =	1.50	x (2.	.90 +	0.00) +	1.00	x	21.83	+	0.50	х (4.12	+	0.00) =	28.24	k-ft		
The s	ervice design	forces for	the wall st	em are:															
	H _{serv} = 1.00 (F	P _{EH(S)} + P	_{EH(B)}) + 1.00) P _{WA} + 1	.00 (P _{LS} ·	+ P _{LL})													
	H _{serv} = 1	.00 x (1.11	+	0.00) +	1.00	x	0.00	+	1.00	х (1.05	+	0.00) =	2.17	k	
	M _{serv} = 1.00 (f	M _{EH(S)} + N	1 _{ЕН(В)}) + 1.0	0 M _{WA} +	1.00 (M _{Ls}	s + M _{LL})												
	M _{serv} = 1	.00 x (2.90	+	0.00) +	1.00	x	0.00	+	1.00	х (4.12	+	0.00) =	7.02	k-ft	
-	n - Investigat									41 41-:	-1								
	ally does not g rse reinforcen		•	retaining	walls. If	snear	does be	ecome a	in issue,	the thi	ckness	s of the s	stem sno	onia pe inc	reased	sucn			
Ignori	ng the benefit	s of the s	hear key an	id axial co	ompressi	on, the	shear	capacity	/ of the s	stem ca	in be s	hown to	be grea	iter than th	nat requi	ired.			
	$V_n = V_c + V_s +$	+ V _p			[5.8.	3.3-1]													
Reco	gnizing that V_s	, and V_p a	ire zero																
	V _n = V _c																		
	V _c = 0.0316 ß	g (f _c ') ^{0.5} b _v	, d _v		[5.8.	3.3-3]													
The m	naximum effec	ctive shea	r depth is:																
	For 2.0 in	ch clear c	cover and	#6 ba	ars, d _s =	18	8.00	-	2.00	- (0.7	50 / 2	2) =	15.63	in				
	d _v = 0.9 d _e = 0 d _v = 0.72 h =	-) x 2 x	15.63 18.00	= =	14.0 12.9		<u>GO\</u>	/ERNS									
Follov	v the General	Procedur	e using the	provision	is of App	endix l	B5, as p	per Sect	ion [5.8.	3.4.2]:									

CLIENT	ODOT	PROJECT NO.		1122-1001-0	D
SUBJECT	CUY-271-0.00 (PID 80418) Reinforced Concrete Retaining Wall Design Wall WS1, Panels 7-8	COMP. BY CHECKED BY	ASP LNB	DATE DATE	3/17/2014 3/17/2014

17.52 k > 4.12 k <u>OK</u>

Wall Stem Design - Investigate Strength Limit State

Determine the area of back-face flexural reinforcement necessary to satisfy the design moment:

M_u = 28.24 k-ft

Again use the equation:

 $3.309 \quad A_s^2 \quad - \quad 4.50 \quad d_s A_s + M_u = 0$

Substituting and solving for A_s , it is found that required $A_s = 0.41 \text{ in}^2/\text{ft}$

Try: #6 bars at 12.00 in c/c $A_s = 0.44 \text{ in}^2/\text{ft} > 0.41 \text{ in}^2/\text{ft} \frac{OK}{S}$

Wall Stem Design - Investigate Service Limit State [5.7.3.4]

Check the crack control equations to ensure that the primary flexural reinforcement is well distributed. The service load bending moment is:

M_{serv} = 7.02 k-ft

Check the modulus of rupture for concrete:

$$\phi f_r = \phi \ 0.24 \ (f_c)^{0.5} = 0.80 \ x \ 0.24 \ x \ (4.0)^{0.50} = 0.384 \ \text{ksi}$$

$$S = \frac{b \ t_{wb}^2}{6} = \frac{12.00 \ x \ (18.00)^2}{6} = 648.00 \ \text{in}^3$$

$$f_{act} = \frac{M_{serv}}{S} = \frac{7.02 \ x \ 12}{648.00} = 0.130 \ \text{ksi} < 0.384 \ \text{ksi}$$

If modulus of rupture check is "NG", then check the spacing of the reinforcement. First, determine the modular ratio:

$$E_c = 33,000 w_c^{1.5} (f_c^{-1})^{0.5} = 33,000 x (0.150)^{1.5} x (4.00)^{0.5} = 3,834$$
 ksi
n = $E_s/E_c = 29,000$ / 3,834 = 7.56, use n = 8.00

For 2.0 inch clear cover and #6 bars, $d_s = 18.00 - 2.00 - (0.750 / 2) = 15.63$ in

Determine the location of the neutral axis:

0.5 b $x^2 = n A_s (d_s - x)$ 0.5 (12) $x^2 = 8 \times 0.44 \times (15.63 - x)$ solving, x = 2.75 in

Check the spacing of the reinforcement to control cracking:

$$y = d_{s} - x = 15.63 - 2.75 = 12.88 \text{ in}$$

$$l_{cr} = \frac{b x^{3}}{3} + n A_{s} (d_{s} - x)^{2}$$

$$l_{cr} = \frac{-12.00 \times (-2.75)^{3}}{3} + 8 \times 0.44 \times (-15.63 - 2.75)^{2} = 666.68 \text{ in}^{4}$$

$$\gamma_{e} = 1.00$$
For 2.0 inch clear cover and #6 bars, $d_{c} = 2.00 + (-0.750/2) = 2.38 \text{ in}$

$$\beta_{s} = 1.0 + \frac{d_{c}}{-0.7 (t_{wb} - d_{c})} = 1.0 + \frac{2.38}{-0.7 \times (-18.00 - 2.38)} = 1.22$$

$$f_{ss} = n \frac{M_{serv} y}{l_{cr}} = 8 \times \frac{7.02 \times 12.88 \times 12.00}{-666.68} = 13.01 \text{ ksi}$$

	17	CLIENT PROJEC	ODOT T <u>CUY-271</u>	-0.00 (PID 80)418)				PROJEC	-		1122-1001	
		SUBJEC	T Reinforce Wall WS	ed Concrete F 1, Panels 7-8	Retaining	Wall De	sign		COMP. E CHECKE		ASP LNB	DATE	<u>3/17/20</u> <u>3/17/20</u>
Sm	$hax = \frac{700 \gamma_e}{\beta_e f}$	— - 2.0 d.		700 x 1.22 x	1.0	0	20	x 2.38	=	39.44 i	in		
-11.	$\beta_s f_{ss}$			1.22 x	13.0)1	2.0	. 2.00					
s _m	_{hax} = 39.44	in >	• 12.00	in <u>OK</u>									
Wall Stem De	esign - Check Reinf	forcement	Limits										
Check N	Minimum Reinforce	ement	[5.7.3.3.2	2]									
De	etermine the crackin	ig moment:	1										
	$f_r = 0.24 (f_c')^{0.5}$	=	0.24 x	(4.0) ^{0.50}	=	0.48	ksi						
	l _g = (1/12) b h ³	= 0).0833 x	12.00	х (18.00) ³ =	5832.0 in ⁴					
	y _t = (1/2) h	= 0).5000 x	18.00	=	9.00	in						
Mc	$_{CR}$ = $\gamma_1 \gamma_3 f_r I_g / y_t$	=	0.67 ×	1.6	x	0.48	x	5832.0 /	(9.00	x	12.00) = 27.79	k-ft
Th	ne capacity of the se	ection must	be greater t	han or equal	to the sn	naller of:							
	M _{CR} = (4/3) M _u =	1.33		27.79 = 28.24 =	27.7 37.6		<u>GOVE</u>	RNS					
Th	ne capacity of the rei	inforcemen	it is:										
	$M_r = \phi A_s f_v (d_s -$	- a/2)											
	M _r = 0.90 x	0.44 x	60 x	(15.63		0.44 1.7 x	4 x 4.0	60 x 12) x	x (<u>1</u>)	=	30.30 k	-ft	
	3	0.30 k-ft	>	27.79 k-ft	<u>OK</u>								
Cr	neck minimum reinfo	orcement fo	or temperati	ire and shrink	age (5.1	0.8)		A _s =	0.44 in ²	/ft >	0.16 ir	n ² /ft <u>OK</u>	
<u>Us</u>	se #6 bars at 12.00 i	in c/c for wa	all stem bac	k face vertica	l reinford	<u>cing.</u>							
A minim	esign - Shrinkage a num amount of reinfo ge and temperature o	orcement sl				5.10.8] f concrete	elemen	ts to limit the s	size of crack	ks associa	ated with con	crete	
	$h_{max} = max(t_{wt}, t_{v})$	(wb) =	18.00	in									
	h _w = 7.82	ft =	93.84	in									
	Min. A _s =	1.30 2 (h _w	$\frac{h_w h_{max}}{h_{max} + h_{max}} f_y$	=2	1.30 x (x 9 93.84	3.84 +	x 18.00	x 60 =	= 0.1	6 in²/ft		
	ne maximum spacing	g of reinfor	cement is:										
Th	h - min/t t	_{/b}) =	18.00	in									
Th	$h_{min} = min(t_{wt}, t_{w})$					=	54.0 18.0						
Th	Max. spacing or Max. spacing or Max. spacing	ı = 18 in	r walls and f	$\overline{ootings} \ge 18$	in thick)	=	12.0	0 in <u>GC</u>	<u>OVERNS</u>				
Th	Max. spacing = a or Max. spacing or Max. spacing	= 18 in = 12 in (foi	r walls and f	ootings ≥ 18 i	in thick)			0 in <u>GC</u> A _s =		/ft >	0.16 ^{ir}	n²/ft <u>OK</u>	
	Max. spacing = or Max. spacing or Max. spacing	= 18 in = 12 in (foi	_	ootings ≥ 18 i	in thick)			A _s =	0.20 in ²	/ft > 0.20 i			

Use #4 bars at 12.00 in c/c for wall stem front face reinforcing and back face horizontal reinforcing.

	CLIENT PROJEC SUBJEC	r Rein	-271-0. forced		Retaining	Wall Design CO	DJECT NO. MP. BY ECKED BY		1122-1001-00 ASP DATE LNB DATE	3/17/2014 3/17/2014
Dimensions and Weights for Co	ncrete Des	sign								
Footing width, w _{foot} =	9.50	ft	=	114.00	in	Concrete weight, w_c =	0.150	kcf		
						Water weight, w _w =	0.062	kcf		
Footing heel width, w _{heel} =	6.00	ft				Saturated soil weight, w _{ss} =	0.130	kcf		
Footing heel height, h _{heel} =	1.50	ft	=	18.00	in	Buoyant soil weight, w _{sb} =	0.068	kcf		
		_								
Footing toe width, w_{toe} =	2.00	ft				Height of wall, h _w =	8.85	ft	top of heel to top of wall	
Footing toe height, h_{toe} =	1.50	ft	=	18.00	in	Height of water, h _{water} =	0.00	ft	top of heel to water line	

									-	
Wall width at top, t _{wt} =	1.50	ft	=	18.00	in	Height of soil, h _s =	8.85	ft	top of heel to ground line	
Wall width at base, t_{wb} =	1.50	ft	=	18.00	in	Height of satur. soil, h _{ss} =	8.85	ft	height of satur. soil above top of heel	
-						Height of buoy. soil, h _{sb} =	0.00	ft	height of buoy. soil above top of heel	
Concrete strength, f _c ' =	4.00	ksi								
Rebar strength, f _y =	60.00	ksi				Active pressure coeff., K _a =	0.280			
Steel mod. of elast., E _s =	29,000	ksi				LL surcharge soil ht., h_{LL} =	3.45	ft		

Optional Collision Loading for Barrier on Top of Wall

Per ODOT comments for I-70/71, use the transverse loading of 54 kips for a TL-4 test level railing [AASHTO, Table A13.2-1] distributed over the retaining wall's joint spacing.

Collision Loading =	У	(y or n)
Joint Spacing =	28.00	ft
Barrier Height =	3.50	ft

Design Summary

Summary of Design Status

Footing Wall Design Item Heel Stem Toe OK OK OK Shear Minimum Reinforcement OK OK OK Shrinkage & Temperature Crack Control OK OK OK OK N/A N/A

For calculations related to each design item, see below.

Design Footing for Shear

Design footings to have adequate shear capacity without transverse reinforcement.

[5.13.3.6]

Determine d_v

Assume: For the he	#4 bars at 1		for the top t for the botto						0.79 i 0.20 i			0 in cover 0 in cover
	d _{sheel} = h - cover - d _{bar}	/2 =	18.00	-	2.00	- (1.000	/2)	=	15.50	in	
	$a_{heel} = \frac{A_s f_y}{0.85 f_c' b}$	- =	0.85	0.79 x	x 60 4.0 x	12			=	1.16	in	
[5.8.2.9]	$d_{vheel} = d_{sheel} - a / 2$ or d _{vheel} = 0.90 d _e or d _{vheel} = 0.72 h	= = =	15.50 0.90 0.72	- (x x	1.16 15.50 18.00	/2)			= = =	14.92 13.95 12.96	in in in	<u>GOVERNS</u>
For the to	e:											
	$d_{stoe} = h - cover - d_{bar}$	/2 =	18.00	-	3.00	- (0.500	/2)	=	14.75	in	
	$a_{toe} = \frac{A_s f_y}{0.85 f_c' b}$		0.85	0.20 x	x 60 4.0 x	12			=	0.29	in	
[5.8.2.9]	d_{vtoe} = d_{stoe} - a / 2 or d_{vtoe} = 0.90 d_e	= =	14.75 0.90	- (x	0.29 14.75	/2)			= =	14.60 13.28	in in	<u>GOVERNS</u>

Reinforcing Steel Summary

Footing:	Top transverse: Bottom transverse: Longitudinal:	#8 bars at 12.00 in c/c #4 bars at 12.00 in c/c #4 bars at 12.00 in c/c
Wall Stem:	Back face vertical: Front face vertical: Horizontal:	#7 bars at 12.00 in c/c #4 bars at 12.00 in c/c #4 bars at 12.00 in c/c

CLIENT ODOT	PROJECT NO.		1122-1001-00)
PROJECT CUY-271-0.00 (PID 80418) SUBJECT Reinforced Concrete Retaining Wall Design Wall WS1, Panels 9-10	COMP. BY CHECKED BY	ASP LNB	DATE DATE	3/17/2014 3/17/2014

or $d_{vtoe} = 0.72 h$ = 0.72 x 18.00 = 12.96 in

Check Heel for Shear

The critical shear section for the heel of the footing is located at the back face of the wall. The heel of the footing is assumed to carry its self weight and the rectangular soil block above it. This neglects the benefit of any upward soil pressure below the footing (conservative).

 $V_{u} = (\gamma_{EV} w_{ss} h_{ss} + \gamma_{EV} w_{sb} h_{sb} + \gamma_{DC} w_{c} h_{heel} + \gamma_{LL} w_{ss} h_{LL} + \gamma_{WA} w_{w} h_{water}) x w_{heel}$

V _u =	(1.35	х	0.130	х	8.85	+	1.35	х	0.068	х	0.00	+	1.25 x	0.15	50 x	1.50	+
	1.75	х	0.130	х	3.45	+	1.00	х	0.062	х	0.00) x	6.00	=	15.72	k/ft	

[5.8.3.3] Using β = 2.00 and assuming bars in the top mat as above:

 $\phi V_{c} = \phi 0.0316 \beta (f_{c})^{0.5} b_{v} d_{v}$ [5.8.3.4] [5.8.2.9] $\phi V_c =$ 0.0316 x (4.0)^{0.50} x 12.00 20.37 k 0.90 х 2.00 14.92 х х = 20.37 k > 15.72 k <u>OK</u>

Check Toe for Shear

The peak bearing stress is 2.62 ksf for the Extreme IIb load case.

The critical section for the toe of the footing is at dv from the front face of the wall. For a quick simplified check, try applying the peak bearing stress over the entire length of the toe (conservative).

 $V_u = \sigma_V w_{toe}$ = 2.62 x 2.00 = 5.24 k/ft

[5.8.3.3] Using β = 2.00 and assuming bars in the bottom mat as above:

 $\phi V_{c} = \phi 0.0316 \beta (f_{c})^{0.5} b_{v} d_{v}$ [5.8.3.4] [5.8.2.9] $\phi V_c =$ x (4.0)^{0.50} x 0.90 х 0.0316 х 2.00 12.00 х 14.60 = 19.93 k 19.93 5.24 <u>OK</u> k > k

Design Footing Reinforcement [5.13.3.4]

Each mat of reinforcement is checked to ensure that it has adequate capacity and that the maximum and minimum reinforcement checks are satisfied. The critical section for flexure in the footing is at the face of the wall.

Top Transverse Reinforcement

From the shear check of the heel, V_{μ} = 15.72 k/ft

 $M_u = V_u x (w_{heel} / 2) = 15.72 x (6.00 / 2) = 47.15 k-ft$

Set up the equation to solve for the required steel area:

$$\begin{split} M_{u} &= \phi M_{n} = \phi A_{s} f_{y} (d_{s} - a/2) &= \phi A_{s} f_{y} (d_{s} - \frac{A_{s} f_{y}}{1.7 f_{c}^{+} b}) \\ M_{u} &= 0.90 \quad x A_{s} \ x \quad 60 \quad (ds - \frac{As}{1.7 \ x} \frac{60}{4.0 \ x} \frac{12}{12}) x (\frac{1}{12}) \\ &3.309 \quad A_{s}^{-2} - 4.50 \quad d_{s} A_{s} + M_{u} = 0 \end{split}$$

For the reinforcing steel assumed for the heel, $d_{s} = 15.50$ in
Substituting and solving for A_{s} , it is found that required $A_{s} = 0.70 \quad in^{2}/ft$

Try: #8 bars at 12.00 in c/c for the top transverse bars in the heel $As = 0.79 \text{ in}^2/\text{ft}$

Check Minimum Reinforcement [5.7.3.3.2]

	PROJECT NO.	22-1001-00	
PROJECT PROJECT CUY-271-0.00 (PID 80418) SUBJECT Reinforced Concrete Retaining Wall Design Wall WS1, Panels 9-10	COMP. BY CHECKED BY	ASP LNB	DATE <u>3/17/2014</u> DATE <u>3/17/2014</u>
Determine the cracking moment:			
$f_r = 0.24 (f_c')^{0.5} = 0.24 \times (4.0)^{0.50} = 0.48 \text{ ksi}$			
$l_g = (1/12) b h^3 = 0.0833 x 12.00 x (18.00)^3 = 5832.0$	in⁴		
$y_t = (1/2) h$ = 0.5000 x 18.00 = 9.00 in			
$M_{CR} = \gamma_1 \gamma_3 f_r l_g / y_t = 1.60 x 0.67 x 0.48 x 5832.0$	/(9.00 x	12.00) =	27.79 k-ft
The capacity of the section must be greater than or equal to the smaller of:			
M_{CR} = 27.79 = 27.79 k-ft <u>GOVERNS</u> (4/3) M_u = 1.33 x 47.15 = 62.86 k-ft			
The capacity of the top mat of reinforcement is:			
$M_r = \phi A_s f_y (d_s - a/2)$			
For the reinforcing steel used, d_s = 18.00 - 2.00 - (1.000 / 2)	= 15.50 in		
$M_r = 0.90 \times 0.79 \times 60 \times (15.50 - \frac{0.79 \times 60}{1.7 \times 4.0 \times 12})$	$(-) x (\frac{1}{12}) =$	53.04 k-ft	
53.04 k-ft > 27.79 k-ft <u>OK</u>	0.79 in ² /ft >	0.17 in ² /ft	24
Check minimum reinforcement for temperature and shrinkage (5.10.8) $A_s =$ Use #8 bars at 12.00 in c/c for top transverse reinforcement in the footing.	0.79 in²/ft >	0.17 in ⁻ /ft	<u>OK</u>
Use #0 bars at 12.00 in GC for top transverse reinforcement in the rooting.			
Bottom Transverse Reinforcement			
From the shear check of the toe, $V_u = 5.24$ k/ft			
$M_u = V_u x (w_{toe} / 2) = 5.24 x (2.00 / 2) = 5.24 k-ft$			
Set up the equation to solve for the required steel area and again use:			
$3.309 A_s^2 - 4.50 d_s A_s + M_u = 0$			
For the reinforcing steel assumed for the heel, $d_s = 14.75$ in			
Substituting and solving for A_s , it is found that required $A_s = 0.08 \text{ in}^2/\text{ft}$	0.20 in ² /ft		
Try: #4 bars at 12.00 in c/c for the bottom transverse bars in the toe As =	0.20 in /π		
Check Minimum Reinforcement [5.7.3.3.2]			
Determine the cracking moment:			
$f_r = 0.24 (f_c)^{0.5} = 0.24 \times (4.0)^{0.50} = 0.48 \text{ ksi}$. 1		
$l_g = (1/12) b h^3 = 0.0833 x 12.00 x (18.00)^3 = 5832.0$	in*		
$y_t = (1/2) h$ = 0.5000 x 18.00 = 9.00 in			
$M_{CR} = \gamma_1 \gamma_3 f_r I_g / y_t = 1.60 \times 0.67 \times 0.48 \times 5832.0$	/(9.00 x	12.00) =	27.79 k-ft
The capacity of the section must be greater than or equal to the smaller of:			
$M_{CR} = 27.79 = 27.79 \text{ k-ft}$ (4/3) $M_u = 1.33 \text{ x} 5.24 = 6.99 \text{ k-ft} GOVERNS$			
The capacity of the bottom mat of reinforcement is:			
$M_r = \phi A_s f_y (d_s - a/2)$			

-

	PROJECT NO. 1122-1001-00
PROJECT CUY-271-0.00 (PID 80418) SUBJECT Reinforced Concrete Retaining Wall Design Wall WS1, Panels 9-10	COMP. BYASPDATE3/17/2014CHECKED BYLNBDATE3/17/2014
For the reinforcing steel used, $d_s = 18.00 - 3.00 - (0.500 / 2)$	= 14.75 in
$M_r = 0.90 \times 0.20 \times 60 \times (14.75 - \frac{0.20 \times 60}{1.7 \times 4.0 \times 12})$	$) x (\frac{1}{12}) = 13.14$ k-ft
13.14 k-ft > 6.99 k-ft <u>OK</u>	
Check minimum reinforcement for temperature and shrinkage (5.10.8) $$A_{\rm s}$ = $A_{\rm s}$$	0.20 in ² /ft > 0.17 in ² /ft <u>OK</u>
Use #4 bars at 12.00 in c/c for bottom transverse reinforcement in the footing.	
Longitudinal Reinforcement [5.10.8]	
Provide longitudinal reinforcement in the footing based on shrinkage and temperature requirem	ients.
$h_{max} = max(h_{heel}, h_{toe}) = 18.00$ in	
Min. A _s = $\frac{1.30 \text{ w}_{foot} \text{ h}_{max}}{2 (\text{ w}_{foot} + \text{ h}_{max}) \text{ f}_{y}} = \frac{1.30 \text{ x} 114.00 \text{ x} 18.00}{2 \text{ x} (114.00 + 18.00)}$	$\frac{00}{x 60} = 0.17 \text{ in}^2/\text{ft}$
The maximum spacing of reinforcement is:	
$h_{min} = min(h_{heel}, h_{toe}) = 18.00$ in	
Max. spacing = 3 h_{min} =54.00inor Max. spacing = 18 in=18.00inor Max. spacing = 12 in (for walls and footings \geq 18 in thick)=12.00in	<u>GOVERNS</u>
Try: #4 bars at 12.00 in c/c for the top and bottom longitudinal bars $A_s =$	0.20 in ² /ft > 0.17 in ² /ft <u>OK</u>
0.11	$< A_{s} = 0.20 \text{ in}^{2}/\text{ft} < 0.60 \text{ OK}$
spacir	ng = 12.00 in = 12.00 in <u>OK</u>
Use #4 bars at 12.00 in c/c for top and bottom longitudinal reinforcement in the footing.	
Determine Loads for Wall Stem Design	
The loads on the stem at the top of the footing are determined to arrive at the design forces for the wa	all.
Saturated Earth Pressure:	
$P_{EH(S)} = (1/2) w_{ss} K_a h_{ss}^2 = 0.5 x 0.130 x 0.280 x (8.8)$	$(35)^2 = 1.43$ k
$M_{EH(S)} = P_{EH(S)} x [(1/3) h_{ss} + h_{sb}] = 1.43 x [0.333 x 8.8]$	85 + 0.00] = 4.21 k-ft
Buoyant Earth Pressure:	
$P_{EH(B)} = K_{a} h_{sb} [w_{ss} h_{ss} + (1/2) w_{sb} h_{sb}]$	
P _{EH(B)} = 0.280 x 0.00 x [0.130 x 8.85 + 0.5	x 0.068 x 0.00] = 0.00 k
$y_{\rm B} = [\ h_{\rm sb} \ (\ w_{\rm ss} \ h_{\rm ss} + (1/3) \ w_{\rm sb} \ h_{\rm sb} \) \] \ / \ (2 \ w_{\rm ss} \ h_{\rm ss} + w_{\rm sb} \ h_{\rm sb} \)$	
$y_{B} = \begin{bmatrix} 0.00 & x & (0.130 & x & 8.85 & + & 0.333 & x & 0.0 \\ / & (2.0 & x & 0.130 & x & 8.85 & + & 0.068 & x & 0.0 \end{bmatrix}$	
$M_{EH(B)} = P_{EH(B)} \times y_B = 0.00 \times 0.00$	= 0.00 k-ft
Water Pressure:	
$P_{WA} = (1/2) w_w h_{water}^2 = 0.5 x 0.062 x (0.00)^2 =$	0.00 k
$M_{WA} = P_{WA} x (1/3) h_{water} = 0.00 x 0.333 x 0.00 =$	0.00 k-ft
Live Load Surcharge:	
$P_{LS} = w_{ss} K_a h_{LL} h_s$ = 0.130 x 0.280 x 3.45 x	8.85 = 1.11 k

	T 7	CLIENT PROJECT	ODOT CUY-2	71-0.00	(PID 80)418)					•	PROJE	CT NO.			1122-100	1-00	
D		SUBJECT	Reinfor		ncrete F	Retainir	ng Wal	Design				COMP. CHECK			SP NB	DATE		3/17/201 3/17/201
	M _{LS} = P _{LS} x (1/2)	h _s	=	1.11	x	0.5	600	x	8.85	=			4.92	k-ft				
Col	Ilision Load at Top	of Parapet:																
	Use a Live Load	of	1929 lt	os/ft app	lied at h	ר _r =	3.5 1	ft above	the top	of the v	vall.							
	P _{CT} =												1.93	k				
	$M_{CT} = P_{CT} x (h_w -$	+ h _r)	=	1.93	x	(8.	85	+	3.50) =			23.82	k-ft				
Usi	ing the Strength I lo	ad combinat	ion, the	factored	d design	forces	for the	e wall ste	em are:									
	H _u = 1.50 (P _{EH(S)}	+ P _{EH(B)}) + 7	1.00 P _W	₄ + 1.75	(P _{LS} + I	P _{LL})												
	H _u = 1.50 x	(1.43	+	0.00) +	1.00	x	0.00	+	1.75	х (1.11	+	0.00) =	4.08	k	
	M_u = 1.50 ($M_{EH(S)}$) + M _{EH(B)}) +	1.00 M _v	_{VA} + 1.7	5 (M _{LS} +	- M _{LL})												
	M _u = 1.50 x	(4.21	+	0.00) +	1.00	x	0.00	+	1.75	х (4.92	+	0.00) =	14.91	k-ft	
The Extre	eme Event II design	forces for th	ne wall s	tem are	:													
H _{ex}	x_{t} = 1.50 (P _{EH(S)} + P _E	_{EH(B)}) + 1.00	Р _{ст} + 0.	50 (P _{LS}	+ P _{LL})													
H _{ex}	_{xt} = 1.50 x	(1.43	+	0.00) +	1.00	x	1.93	+	0.50	х (1.11	+	0.00) =	4.62	k	
M _{ex}	_{xt} = 1.50 (M _{EH(S)} + M	_{ЕН(В)}) + 1.00	M _{CT} + 0).50 (M _L	_s + M _{LL})													
M _{ex}	_{xt} = 1.50 x	(4.21	+	0.00) +	1.00	x	23.82	+	0.50	x (4.92	+	0.00) =	32.58	k-ft	
The	e service design for	ces for the w	all stem	are:														
	H_{serv} = 1.00 (P_{EH}	_(S) + P _{EH(B)}) ·	+ 1.00 P	_{WA} + 1.0	00 (P _{LS} ·	+ P _{LL})												
	H _{serv} = 1.00	x (1	.43	+	0.00) +	1.00	x	0.00	+	1.00	х (1.11	+	0.00) =	2.54	k
	M_{serv} = 1.00 (M_{EH}	H(S) + M _{EH(B)})	+ 1.00 N	M _{WA} + 1	.00 (M _{LS}	₅ + M _{LL})											
	M _{serv} = 1.00	x (4	.21	+	0.00) +	1.00	x	0.00	+	1.00	х (4.92	+	0.00) =	9.12	k-ft
	ainn Inventionte G																	
	sign - Investigate S		on of ret:	aining w	alls If	sheard	loes be	ecome a	n issue	the thi	ckness	s of the s	tem sho	ould be inc	reased	such		
	sverse reinforcemer		-	g														
Ign	noring the benefits o		ey and a	axial cor	mpressi	on, the	shear	capacity	of the s	stem ca	n be s	hown to	be grea	iter than th	nat requi	red.		
	$V_n = V_c + V_s + V_s$	p			[5.8.]	3.3-1]												
Red	cognizing that V _s ar	V_p are zer	0															
	$V_n = V_c$																	
	V_{c} = 0.0316 β (f	c') ^{0.5} b _v d _v			[5.8.	3.3-3]												
The	e maximum effectiv	e shear dept	th is:															
	For 2.0 inch	clear cover	and	#7 bar	rs, d _s =	18	.00	-	2.00	- (0.8	75 / 2	2) =	15.56	in			
	d _v = 0.9 d _e = 0.9 d _v = 0.72 h = 0.7	-	0.90 0.72		15.56 18.00	= =	14.0 12.9		<u>GO\</u>	/ERNS								
Fol	llow the General Pro	ocedure usin	ig the pro	ovisions	of App	endix E	35, as p	per Sect	ion [5.8	3.4.2]:								
	β = 2.02	20466		ε _s =	0.0	0187		S _{xe}	= 11.	42945		Max Ag	gregate	Size =	1 in	(BDM S	5.10.3.1	.1)
	φ V _c = 0	.90 x	0.031	16 x		.02	× (4.0) ⁰	50 v	10	.00	x	14.01	=	19.32	k		

CLIENT	ODOT	PROJECT NO.		1122-1001-00			
PROJECT SUBJECT	CUY-271-0.00 (PID 80418) Reinforced Concrete Retaining Wall Design Wall WS1, Panels 9-10	COMP. BY CHECKED BY	ASP LNB	DATE DATE	3/17/2014 3/17/2014		

19.32 k > 4.62 k <u>OK</u>

Wall Stem Design - Investigate Strength Limit State

Determine the area of back-face flexural reinforcement necessary to satisfy the design moment:

M_u = 32.58 k-ft

Again use the equation:

 $3.309 \quad A_s^2 \quad - \quad 4.50 \quad d_s A_s + M_u = 0$

Substituting and solving for A_s , it is found that required $A_s = 0.48 \text{ in}^2/\text{ft}$

Try: **#7** bars at **12.00** in c/c $A_s = 0.60 \text{ in}^2/\text{ft} > 0.48 \text{ in}^2/\text{ft} \frac{OK}{S}$

Wall Stem Design - Investigate Service Limit State [5.7.3.4]

Check the crack control equations to ensure that the primary flexural reinforcement is well distributed. The service load bending moment is:

M_{serv} = 9.12 k-ft

Check the modulus of rupture for concrete:

$$\phi f_r = \phi \ 0.24 \ (f_c')^{0.5} = 0.80 \ x \ 0.24 \ x \ (4.0)^{0.50} = 0.384 \ \text{ksi}$$

$$S = \frac{b \ t_{wb}^2}{6} = \frac{12.00 \ x \ (18.00)^2}{6} = 648.00 \ \text{in}^3$$

$$f_{act} = \frac{M_{serv}}{S} = \frac{9.12 \ x \ 12}{648.00} = 0.169 \ \text{ksi} < 0.384 \ \text{ksi} \ OK$$

If modulus of rupture check is "NG", then check the spacing of the reinforcement. First, determine the modular ratio:

$$E_c = 33,000 w_c^{1.5} (f_c^{-1})^{0.5} = 33,000 x (0.150)^{1.5} x (4.00)^{0.5} = 3,834$$
 ksi
n = $E_s/E_c = 29,000$ / 3,834 = 7.56, use n = 8.00

For 2.0 inch clear cover and #7 bars, $d_s = 18.00 - 2.00 - (0.875 / 2) = 15.56$ in

Determine the location of the neutral axis:

0.5 b $x^2 = n A_s (d_s - x)$ 0.5 (12) $x^2 = 8 \times 0.60 \times (15.56 - x)$ solving, x = 3.15 in

Check the spacing of the reinforcement to control cracking:

$$y = d_{s} - x = 15.56 - 3.15 = 12.41 \text{ in}$$

$$l_{cr} = \frac{b x^{3}}{3} + n A_{s} (d_{s} - x)^{2}$$

$$l_{cr} = \frac{12.00 \times (3.15)^{3}}{3} + 8 \times 0.60 \times (15.56 - 3.15)^{2} = 864.56 \text{ in}^{4}$$

$$\gamma_{e} = 1.00$$
For 2.0 inch clear cover and #7 bars, $d_{c} = 2.00 + (0.875/2) = 2.44$ in
$$\beta_{s} = 1.0 + \frac{d_{c}}{0.7 (t_{wb} - d_{c})} = 1.0 + \frac{2.44}{0.7 \times (18.00 - 2.44)} = 1.22$$

$$f_{ss} = n \frac{M_{serv} y}{l_{cr}} = 8 \times \frac{9.12 \times 12.41 \times 12.00}{864.56} = 12.57 \text{ ksi}$$

	LZ		ODOT CUY-271-0						_	ROJEC				122-1001-	
		SUBJECT	Reinforced Wall WS1,	Concrete R Panels 9-10	etaining)	Wall Des	sign			OMP. B		AS LNI		DATE DATE	3/17/20 3/17/20
S _{max}	=700 γ _e	— - 2.0 d _c	= $\frac{70}{1}$	00 x	1.00		2.0	x 2	.44	= 4	0.62	in			
				_	12.57	7									
S _{max}	= 40.62	in >	12.00	in <u>OK</u>											
Wall Stem Desi	ign - Check Reinf	forcement I	imits												
Check Mi	nimum Reinforce	ement	[5.7.3.3.2]												
Dete	ermine the crackin	ig moment:													
	$f_r = 0.24 (f_c')^{0.5}$	= ().24 x (4.0) ^{0.50}	=	0.48	ksi								
	$I_g = (1/12) b h^3$	= 0.	0833 x	12.00	х (18.00) ³ =	5832.0	in ⁴						
	y _t = (1/2) h	= 0.	5000 x	18.00	=	9.00	in								
M _{CR}	= $\gamma_1 \gamma_3 f_r I_g / y_t$	= ().67 x	1.6	x	0.48	x	5832.0	/ (9.00	x	12.00) =	27.79	k-ft
The	capacity of the se	ection must I	be greater that	in or equal t	o the sm	aller of:									
	M _{CR} = (4/3) M _u =	1.33		.79 = .58 =	27.79 43.45		<u>GOVEF</u>	<u>RNS</u>							
The	capacity of the rei	inforcement	is:												
	$M_r = \phi A_s f_y (d_s -$	- a/2)													
	M _r = 0.90 x	0.60 x	60 x (15.56		0.60 I.7 x	x 4.0	60 x 12	-) x (—	<u>1</u> 12	=	40.83	k-ft		
	4	0.83 k-ft	> 27	.79 k-ft	<u>OK</u>										
Che	ck minimum reinfo	orcement fo	r temperature	and shrinka	age (5.10	0.8)		A _s =	0.60) in ² /1	t>	0.17	in²/ft	<u>OK</u>	
Use	#7 bars at 12.00 i	in c/c for wa	II stem back f	ace vertical	reinforci	ing.									
A minimur	i gn - Shrinkage a m amount of reinfo and temperature o	prcement sh			-	.10.8] concrete	element	s to limit	the size	of cracks	s associ	ated with o	concrete		
	$h_{max} = max(t_{wt}, t_{v})$	(wb) =	18.00	in											
	h _w = 8.85	ft =	106.20	in											
	Min. A _s =	1.30 2 (h _w	h _w h _{max} + h _{max}) f _y	=2	1.30 x (x 10 106.20	6.20 +	x 18 18.00	3.00) x	=	0.1	17 in²/	ft		
The	maximum spacinę	g of reinforc	ement is:												
	$h_{min} = min(t_{wt}, t_{wt})$	_{'b}) =	18.00	in											
	Max. spacing = 3 or Max. spacing or Max. spacing	= 18 in	walls and foc	otings ≥ 18 ir	n thick)	= = =	54.00 18.00 12.00	0 in	<u>GOVEF</u>	RNS					
										im ² /	t>	0.47	:m ² /ft	014	
Try:	#4 bars at	12.00	in c/c					A _s =	0.20) 11171	ι >	0.17	in²/ft	<u>OK</u>	
Try:	#4 bars at	12.00	in c/c					A _s = 0.11					0.60		

Use #4 bars at 12.00 in c/c for wall stem front face reinforcing and back face horizontal reinforcing.

	CLIENT PROJECT SUBJECT	Reinf	-271-0. forced	00 (PID 80 Concrete I Panels 11-	Retainii	ng Wall Design COM	JECT NO. P. BY CKED BY		1122-1001-00 ASP DATE 3/17/2014 LNB DATE 3/17/2014	
Dimensions and Weights for Co	ncrete Des	ign								
Footing width, w _{foot} =	9.50	ft	=	114.00	in	Concrete weight, w _c =	0.150	kcf		
						Water weight, w _w =	0.062	kcf		
Footing heel width, w _{heel} =	6.00	ft				Saturated soil weight, w_{ss} =	0.130	kcf		
Footing heel height, $h_{heel} =$	1.50	ft	=	18.00	in	Buoyant soil weight, w_{sb} =	0.068	kcf		
Footing toe width, w _{toe} =	2.00	ft				Height of wall, h _w =	10.89	ft	top of heel to top of wall	
Footing toe height, h_{toe} =	1.50	ft	=	18.00	in	Height of water, h _{water} =	0.00	ft	top of heel to water line	
Wall width at top, t_{wt} =	1.50	ft	=	18.00	in	Height of soil, h _s =	10.89	ft	top of heel to ground line	
Wall width at base, t _{wb} =	1.50	ft	=	18.00	in	Height of satur. soil, h _{ss} =	10.89	ft	height of satur. soil above top of heel	
						Height of buoy. soil, h _{sb} =	0.00	ft	height of buoy. soil above top of heel	
Concrete strength, f_c ' =	4.00	ksi								
Rebar strength, f _y =	60.00	ksi				Active pressure coeff., K_a =	0.280			

Optional Collision Loading for Barrier on Top of Wall

Steel mod. of elast., Es =

Per ODOT comments for I-70/71, use the transverse loading of 54 kips for a TL-4 test level railing [AASHTO, Table A13.2-1] distributed over the retaining wall's joint spacing.

LL surcharge soil ht., h_{LL} =

у	(y or n)
28.00	ft
3.50	ft

Design Summary

Summary of Design Status

29,000 ksi

h tr otir Footing Wall Fc Design Item Heel Stem Toe OK OK OK Shear Minimum Reinforcement OK OK OK Shrinkage & Temperature Crack Control OK OK OK ۷ OK N/A N/A

For calculations related to each design item, see below.

Design Footing for Shear

Design footings to have adequate shear capacity without transverse reinforcement.

[5.13.3.6]

Determine d_v

		for the top tr					3	0.79 ii 0.20 ii			0 in cover 0 in cover
d _{sheel} = h - cover - d _{ba}	/2 =	18.00	-	2.00	- (1.000	/2)	=	15.50	in	
$a_{heel} = \frac{A_s f_y}{0.85 f_c' l}$	- =	0.85	0.79 x	x 60 4.0 x	12			=	1.16	in	
[5.8.2.9] d _{vheel} = d _{sheel} - a / 2	=	15.50	- (1.16	/2)			=	14.92	in	GOVERNS
or d_{vheel} = 0.90 d_e	=	0.90	х	15.50				=	13.95	in	
or $d_{vheel} = 0.72 h$	=	0.72	х	18.00				=	12.96	in	
For the toe:											
d_{stoe} = h - cover - d_{bar}	/2 =	18.00	-	3.00	- (0.500	/2)	=	14.75	in	
$a_{\text{toe}} = \frac{A_{\text{s}} f_{\text{y}}}{0.85 f_{\text{c}} I}$	- =		0.20 x	x 60 4.0 x	12			=	0.29	in	
[5.8.2.9] d _{vtoe} = d _{stoe} - a / 2	=	14.75	- (0.29	/2)			=	14.60	in	GOVERNS
or $d_{vtoe} = 0.90 d_e$	=	0.90	X	14.75				=	13.28	in	

Reinforcing Steel Summary

3.14

ft

Footing:	Top transverse: Bottom transverse: Longitudinal:	#8 bars at 12.00 in c/c #4 bars at 12.00 in c/c #4 bars at 12.00 in c/c
Wall Stem:	Back face vertical: Front face vertical: Horizontal:	#8 bars at 12.00 in c/c #4 bars at 12.00 in c/c #4 bars at 12.00 in c/c

CLIENT ODOT	PROJECT NO.		1122-1001-0	0
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or $d_{vtoe} = 0.72 h$ 0.72 18.00 12.96 in х

Check Heel for Shear

The critical shear section for the heel of the footing is located at the back face of the wall. The heel of the footing is assumed to carry its self weight and the rectangular soil block above it. This neglects the benefit of any upward soil pressure below the footing (conservative).

 $V_{u} = (\gamma_{EV} w_{ss} h_{ss} + \gamma_{EV} w_{sb} h_{sb} + \gamma_{DC} w_{c} h_{heel} + \gamma_{LL} w_{ss} h_{LL} + \gamma_{WA} w_{w} h_{water}) x w_{heel}$

V _u =	(1.35	х	0.130	х	10.89	+	1.35	х	0.068	х	0.00	+	1.25 x	0.1	50 x	1.50	+
	1.75	х	0.130	х	3.14	+	1.00	х	0.062	х	0.00) x	6.00	=	17.44	k/ft	

[5.8.3.3] Using β = 2.00 and assuming bars in the top mat as above:

 $\phi V_{c} = \phi 0.0316 \beta (f_{c})^{0.5} b_{v} d_{v}$ [5.8.3.4] [5.8.2.9] $\phi V_c =$ 0.0316 x (4.0)^{0.50} x 20.37 k 0.90 х 2.00 12.00 14.92 х х = 20.37 k > 17 44 k <u>OK</u>

Check Toe for Shear

The peak bearing stress is 3.13 ksf for the Extreme IIb load case.

The critical section for the toe of the footing is at dv from the front face of the wall. For a quick simplified check, try applying the peak bearing stress over the entire length of the toe (conservative).

 $V_{\mu} = \sigma_V W_{toe}$ = 3.13 х 2.00 6.26 k/ft

Using β = 2.00 and assuming bars in the bottom mat as above: [5.8.3.3]

 $\phi V_{c} = \phi 0.0316 \beta (f_{c})^{0.5} b_{v} d_{v}$ [5.8.3.4] [5.8.2.9] $\phi V_c =$ x (4.0)^{0.50} x 0.90 х 0.0316 х 2.00 12.00 х 14.60 = 19.93 k 19.93 6.26 <u>OK</u> k > k

Design Footing Reinforcement [5.13.3.4]

Each mat of reinforcement is checked to ensure that it has adequate capacity and that the maximum and minimum reinforcement checks are satisfied. The critical section for flexure in the footing is at the face of the wall.

Top Transverse Reinforcement

For

From the shear check of the heel, V_{μ} = 17 44 k/ft

 $M_u = V_u x (w_{heel} / 2)$ 17.44 x (6.00 /2) = 52.32 k-ft =

Set up the equation to solve for the required steel area:

$$M_{u} = \phi M_{n} = \phi A_{s} f_{y} (d_{s} - a/2) = \phi A_{s} f_{y} (d_{s} - \frac{A_{s} f_{y}}{1.7 f_{c}^{+} b})$$

$$M_{u} = 0.90 \quad x A_{s} x \quad 60 \quad (ds - \frac{As}{1.7 x} \frac{x \quad 60}{4.0 x} \frac{1}{12}) x (\frac{1}{12})$$

$$3.309 \quad A_{s}^{2} \quad - \quad 4.50 \quad d_{s} A_{s} + M_{u} = 0$$
For the reinforcing steel assumed for the heel, $d_{s} = 15.50$ in
Substituting and solving for A_{s} , it is found that required $A_{s} = 0.78 \text{ in}^{2}/\text{ft}$

 $As = 0.79 \text{ in}^2/\text{ft}$ 12.00 in c/c for the top transverse bars in the heel Try: #8 bars at

Check Minimum Reinforcement [5.7.3.3.2]

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Determine the cracking moment:					
$f_r = 0.24 (f_c')^{0.5} = 0.24 \times (4.0)^{0.50} = 0.48 \text{ ksi}$					
$l_g = (1/12) b h^3 = 0.0833 x 12.00 x (18.00)^3 = 5832.0$	in⁴				
$y_t = (1/2) h$ = 0.5000 x 18.00 = 9.00 in					
$M_{CR} = \gamma_1 \gamma_3 f_r l_g / y_t = 1.60 x 0.67 x 0.48 x 5832.0$	/(9.00 x	12.00) = 27.79	9 k-ft		
The capacity of the section must be greater than or equal to the smaller of:					
M_{CR} = 27.79 = 27.79 k-ft <u>GOVERNS</u> (4/3) M_u = 1.33 x 52.32 = 69.76 k-ft					
The capacity of the top mat of reinforcement is:					
$M_r = \phi A_s f_y (d_s - a/2)$					
For the reinforcing steel used, d_s = 18.00 - 2.00 - (1.000 / 2)	= 15.50 in				
$M_r = 0.90 \times 0.79 \times 60 \times (15.50 - \frac{0.79 \times 60}{1.7 \times 4.0 \times 12})$	$-) x (\frac{1}{12}) =$	53.04 k-ft			
53.04 k-ft > 27.79 k-ft <u>OK</u>					
Check minimum reinforcement for temperature and shrinkage (5.10.8) $A_s =$	0.79 in ² /ft >	0.17 in ² /ft <u>O</u>	<u>IK</u>		
Use #8 bars at 12.00 in c/c for top transverse reinforcement in the footing.					
Bottom Transverse Reinforcement					
From the shear check of the toe, $V_u = 6.26$ k/ft					
$M_u = V_u x (w_{toe} / 2) = 6.26 x (2.00 / 2) = 6.26 k-ft$					
Set up the equation to solve for the required steel area and again use:					
$3.309 A_s^2 - 4.50 d_s A_s + M_u = 0$					
For the reinforcing steel assumed for the heel, $d_s = 14.75$ in					
Substituting and solving for A_s , it is found that required $A_s = 0.09 \text{ in}^2/\text{ft}$					
Try: #4 bars at 12.00 in c/c for the bottom transverse bars in the toe As =	: 0.20 in ² /ft				
Check Minimum Reinforcement [5.7.3.3.2]					
Determine the cracking moment:					
$f_r = 0.24 (f_c')^{0.5} = 0.24 \times (4.0)^{0.50} = 0.48 \text{ ksi}$					
$l_g = (1/12) b h^3 = 0.0833 x 12.00 x (18.00)^3 = 5832.0$	in ⁴				
$y_t = (1/2) h$ = 0.5000 x 18.00 = 9.00 in					
$M_{CR} = \gamma_1 \gamma_3 f_r l_g / y_t$ = 1.60 x 0.67 x 0.48 x 5832.0	/(9.00 x	12.00) = 27.79	9 k-ft		
The capacity of the section must be greater than or equal to the smaller of:					
M_{CR} = 27.79 = 27.79 k-ft (4/3) M_u = 1.33 x 6.26 = 8.35 k-ft <u>GOVERNS</u>					
The capacity of the bottom mat of reinforcement is:					
$M_r = \phi A_s f_y (d_s - a/2)$					

-

$\frac{\text{PROJECT}}{\text{SUBJECT}} \underbrace{\frac{\text{CUY-271-0.00 (PID 80418)}}{\text{Reinforced Concrete Retaining Wall Design}}_{Wall WS1, Panels 11-13}$ For the reinforcing steel used, d _s = 18.00 - 3.00 - (0.500 / 2) M _r = 0.90 x 0.20 x 60 x (14.75 - $\frac{0.20 \text{ x } 60}{1.7 \text{ x } 4.0 \text{ x } 12}$ 13.14 k-ft > 8.35 k-ft OK	COMP. BY CHECKED BY		
$M_r = 0.90 \times 0.20 \times 60 \times (14.75 - \frac{0.20 \times 60}{1.7 \times 4.0 \times 12})$			DATE 3/17/2014 DATE 3/17/2014
) = 14.75 in		
	$\frac{1}{2}$) x ($\frac{1}{12}$) =	13.14 k-ft	
	. 12		
Check minimum reinforcement for temperature and shrinkage (5.10.8) $A_s =$	= 0.20 in ² /ft >	0.17 in ² /ft	<u>OK</u>
Use #4 bars at 12.00 in c/c for bottom transverse reinforcement in the footing.			
Longitudinal Reinforcement [5.10.8]			
Provide longitudinal reinforcement in the footing based on shrinkage and temperature require	ements.		
$h_{max} = max(h_{heel}, h_{toe}) = 18.00$ in			
Min. A _s = $\frac{1.30 \text{ w}_{\text{foot}} \text{ h}_{\text{max}}}{2 (\text{ w}_{\text{foot}} + \text{ h}_{\text{max}}) \text{ f}_{\text{y}}} = \frac{1.30 \text{ x} 114.00 \text{ x} 114.00 \text{ x}}{2 \text{ x} (114.00 \text{ + } 18.00 \text{ x})}$	18.00 = 0.17	/ in²/ft	
)x 60	in <i>n</i> c	
The maximum spacing of reinforcement is:			
$h_{min} = min(h_{heel}, h_{toe}) = 18.00$ in			
Max. spacing = 3 h_{min} =54.00inor Max. spacing = 18 in=18.00inor Max. spacing = 12 in (for walls and footings \geq 18 in thick)=12.00in	GOVERNS		
Try: #4 bars at 12.00 in c/c for the top and bottom longitudinal bars $A_s =$. 2.0	0.17 in ² /ft	OK
		² /ft < 0.60	OK
	U		
	acing = 12.00 in	= 12.00 in	<u>OK</u>
Use #4 bars at 12.00 in c/c for top and bottom longitudinal reinforcement in the footing.			
Determine Loads for Wall Stem Design			
The loads on the stem at the top of the footing are determined to arrive at the design forces for the	wall.		
Saturated Earth Pressure:			
$P_{EH(S)} = (1/2) w_{ss} K_a h_{ss}^2 = 0.5 x 0.130 x 0.280 x (1)$	$(10.89)^2 = 2.16$ k		
$P_{EH(S)} = (1/2) w_{ss} K_a h_{ss}^2 = 0.5 x 0.130 x 0.280 x (1)$ $M_{EH(S)} = P_{EH(S)} x [(1/3) h_{ss} + h_{sb}] = 2.16 x [0.333 x 1]$,	= 7.83 k-f	ft
$\begin{split} P_{EH(S)} &= (1/2) w_{ss} K_a h_{ss}^2 &= 0.5 x 0.130 x 0.280 x \ (\ 1 \\ & M_{EH(S)} = P_{EH(S)} x \ [\ (1/3) h_{ss} + h_{sb} \] &= 2.16 x \ [\ 0.333 x 1 \\ & Buoyant Earth Pressure: \end{split}$,	= 7.83 k-1	ft
$\begin{split} P_{EH(S)} &= (1/2) w_{ss} K_a h_{ss}^2 &= 0.5 x 0.130 x 0.280 x \ (\ 1 \\ M_{EH(S)} &= P_{EH(S)} x \ [\ (1/3) h_{ss} + h_{sb}] &= 2.16 x \ [\ 0.333 x 1 \\ \end{split}$ Buoyant Earth Pressure: $P_{EH(B)} &= K_a h_{sb} [w_{ss} h_{ss} + (1/2) w_{sb} h_{sb}]$, 10.89 + 0.00]		
$\begin{split} P_{EH(S)} &= (1/2) w_{ss} K_{a} h_{ss}^{2} &= 0.5 x 0.130 x 0.280 x \ (\ 1 \\ M_{EH(S)} &= P_{EH(S)} x \ [\ (1/3) h_{ss} + h_{sb} \] &= 2.16 x \ [\ 0.333 x 1 \\ \end{split}$ Buoyant Earth Pressure: $P_{EH(B)} &= K_{a} h_{sb} \ [w_{ss} h_{ss} + (1/2) w_{sb} h_{sb} \]$ $P_{EH(B)} &= 0.280 x 0.00 x \ [\ 0.130 x 10.89 + \ 0.5 \\ \end{split}$	10.89 + 0.00]	= 7.83 k-f	ft 0.00 k
$\begin{split} P_{EH(S)} &= (1/2) \ w_{ss} \ K_a \ h_{ss}^2 &= 0.5 \ x \ 0.130 \ x \ 0.280 \ x \ (\ 1 \ M_{EH(S)} &= P_{EH(S)} \ x \ [\ (1/3) \ h_{ss} + h_{sb} \] &= 2.16 \ x \ [\ 0.333 \ x \ 1 \ Buoyant \ Earth \ Pressure: \\ P_{EH(B)} &= K_a \ h_{sb} \ [w_{ss} \ h_{ss} + (1/2) \ w_{sb} \ h_{sb} \] \\ P_{EH(B)} &= 0.280 \ x \ 0.00 \ x \ [\ 0.130 \ x \ 10.89 \ + \ 0.5 \ y_B = [\ h_{sb} \ (\ w_{ss} \ h_{ss} + (1/3) \ w_{sb} \ h_{sb} \)] / \ (2 \ w_{ss} \ h_{ss} + w_{sb} \ h_{sb} \) \end{split}$	10.89 + 0.00] 5 x 0.068 x	0.00] =	
$\begin{split} P_{EH(S)} &= (1/2) \ w_{ss} \ K_{a} \ h_{ss}^{2} &= 0.5 \ x \ 0.130 \ x \ 0.280 \ x \ (\ 1 \ M_{EH(S)} = P_{EH(S)} \ x \ [\ (1/3) \ h_{ss} + h_{sb} \] &= 2.16 \ x \ [\ 0.333 \ x \ 1 \ M_{EH(S)} \\ \end{split}$ Buoyant Earth Pressure: $P_{EH(B)} &= K_{a} \ h_{sb} \ [w_{ss} \ h_{ss} + (1/2) \ w_{sb} \ h_{sb} \] \\ P_{EH(B)} &= 0.280 \ x \ 0.00 \ x \ [\ 0.130 \ x \ 10.89 \ + \ 0.5 \ M_{sb} \$	10.89 + 0.00] 5 x 0.068 x		
$\begin{split} P_{\text{EH}(\text{S})} &= (1/2) \ w_{\text{ss}} \ \text{K}_{\text{a}} \ h_{\text{ss}}^2 &= 0.5 \ \text{x} 0.130 \ \text{x} 0.280 \ \text{x} \ (\ 1 \ M_{\text{EH}(\text{S})} &= P_{\text{EH}(\text{S})} \ \text{x} \ [\ (1/3) \ h_{\text{ss}} + h_{\text{sb}} \] &= 2.16 \ \text{x} \ [\ 0.333 \ \text{x} \ 1 \ M_{\text{EH}(\text{S})} \\ \end{split}$ $\begin{split} \text{Buoyant Earth Pressure:} \\ P_{\text{EH}(\text{B})} &= \text{K}_{\text{a}} \ h_{\text{sb}} \ [w_{\text{ss}} \ h_{\text{ss}} + (1/2) \ w_{\text{sb}} \ h_{\text{sb}} \] \\ P_{\text{EH}(\text{B})} &= 0.280 \ \text{x} \ 0.00 \ \text{x} \ [\ 0.130 \ \text{x} \ 10.89 \ + \ 0.5 \ y_{\text{B}} \\ = \ [\ h_{\text{sb}} \ (w_{\text{ss}} \ h_{\text{ss}} + (1/3) \ w_{\text{sb}} \ h_{\text{sb}} \) \] / \ (2 \ w_{\text{ss}} \ h_{\text{ss}} + w_{\text{sb}} \ h_{\text{sb}} \) \\ y_{\text{B}} &= \ [\ 0.00 \ \text{x} \ (\ 0.130 \ \text{x} \ 10.89 \ + \ 0.333 \ \text{x} \ 0 \ \end{split}$	10.89 + 0.00] 5 x 0.068 x 0.068 x 0.00 0.00)	0.00] =	0.00 k
$\begin{split} P_{EH(S)} &= (1/2) \ w_{ss} \ K_{a} \ h_{ss}^{2} &= 0.5 \ x \ 0.130 \ x \ 0.280 \ x \ (\ 1 \ M_{EH(S)} = P_{EH(S)} \ x \ (\ 1/3) \ h_{ss} + h_{sb} \] &= 2.16 \ x \ [\ 0.333 \ x \ 1 \ M_{EH(S)} = M_{a} \ h_{sb} \ [w_{ss} \ h_{ss} + (1/2) \ w_{sb} \ h_{sb}] \\ \\ P_{EH(B)} &= K_{a} \ h_{sb} \ [w_{ss} \ h_{ss} + (1/2) \ w_{sb} \ h_{sb}] \\ P_{EH(B)} &= 0.280 \ x \ 0.00 \ x \ [\ 0.130 \ x \ 10.89 \ + \ 0.58 \ y_{B} = \ [\ h_{sb} \ (w_{ss} \ h_{ss} + (1/3) \ w_{sb} \ h_{sb} \) \] / \ (2 \ w_{ss} \ h_{ss} + w_{sb} \ h_{sb} \) \\ y_{B} &= \ [\ 0.00 \ x \ (\ 0.130 \ x \ 10.89 \ + \ 0.333 \ x \ 0 \ (\ 0.130 \ x \ 10.89 \ + \ 0.068 \ x \ 0 \) \end{split}$	10.89 + 0.00] 5 x 0.068 x 0.068 x 0.00 0.00)	0.00] =)] =	0.00 k
$\begin{split} P_{EH(S)} &= (1/2) w_{ss} K_{a} h_{ss}^{2} &= 0.5 \times 0.130 \times 0.280 \times (1100) \\ M_{EH(S)} &= P_{EH(S)} \times [(1/3) h_{ss} + h_{sb}] &= 2.16 \times [0.333 \times 100) \\ Buoyant Earth Pressure: \\ P_{EH(B)} &= K_{a} h_{sb} [w_{ss} h_{ss} + (1/2) w_{sb} h_{sb}] \\ P_{EH(B)} &= 0.280 \times 0.00 \times [0.130 \times 10.89 + 0.50) \\ y_{B} &= [h_{sb} (w_{ss} h_{ss} + (1/3) w_{sb} h_{sb})] / (2 w_{ss} h_{ss} + w_{sb} h_{sb}) \\ y_{B} &= [0.000 \times (0.130 \times 10.89 + 0.333 \times 00) / (0.200 \times 10.130 \times 10.89 + 0.068 \times 00) \\ M_{EH(B)} &= P_{EH(B)} \times y_{B} &= 0.00 \times 0.00 \end{split}$	10.89 + 0.00] 5 x 0.068 x 0.068 x 0.00 0.00) = 0.00 k	0.00] =)] = -ft	0.00 k
$\begin{split} P_{EH(S)} &= (1/2) w_{ss} K_{a} h_{ss}^{2} &= 0.5 x 0.130 x 0.280 x \ (\ 1 \\ M_{EH(S)} &= P_{EH(S)} x \ [\ (1/3) h_{ss} + h_{sb} \] &= 2.16 x \ [\ 0.333 x 1 \\ \end{split}$ Buoyant Earth Pressure: $\begin{split} P_{EH(B)} &= K_{a} h_{sb} \ [w_{ss} h_{ss} + (1/2) w_{sb} h_{sb} \] \\ P_{EH(B)} &= 0.280 x 0.00 x \ [\ 0.130 x 10.89 + 0.5 \\ y_{B} &= \ [h_{sb} \ (w_{ss} h_{ss} + (1/3) w_{sb} h_{sb} \) \] / \ (2 w_{ss} h_{ss} + w_{sb} h_{sb} \) \\ y_{B} &= \ [\ 0.00 x \ (\ 0.130 x 10.89 + 0.333 x 0 \\ &- \ (\ 2.0 x 0.130 x 10.89 + 0.068 x 0 \\ M_{EH(B)} &= P_{EH(B)} x y_{B} &= 0.00 x 0.00 \\ \end{split}$ Water Pressure:	10.89 + 0.00] 5 x 0.068 x 0.068 x 0.00 0.00) = 0.00 k	0.00] =)] = -ft	0.00 k
$\begin{split} P_{EH(S)} &= (1/2) w_{ss} K_{a} h_{ss}^{2} &= 0.5 \times 0.130 \times 0.280 \times (11) \\ M_{EH(S)} &= P_{EH(S)} \times [(1/3) h_{ss} + h_{sb}] &= 2.16 \times [0.333 \times 10) \\ Buoyant Earth Pressure: \\ P_{EH(B)} &= K_{a} h_{sb} [w_{ss} h_{ss} + (1/2) w_{sb} h_{sb}] \\ P_{EH(B)} &= 0.280 \times 0.00 \times [0.130 \times 10.89 + 0.5] \\ y_{B} &= [h_{sb} (w_{ss} h_{ss} + (1/3) w_{sb} h_{sb})] / (2 w_{ss} h_{ss} + w_{sb} h_{sb}) \\ y_{B} &= [0.00 \times (0.130 \times 10.89 + 0.333 \times 0)] / (0.130 \times 10.89 + 0.068 \times 0)] \\ M_{EH(B)} &= P_{EH(B)} \times y_{B} &= 0.00 \times 0.00 \\ \end{split}$	10.89 + 0.00] 5 x 0.068 x 0.068 x 0.00 0.00) = 0.00 k	0.00] =)] = -ft	0.00 k

Collisic L	$M_{LS} = P_{LS} \times ($ on Load at T Use a Live L	1/2) h _s		T Reinfo Wall V	orced Co WS1, Pa			ng Wall	Design				COMP. CHECK			SP NB	_ DATE DATE		3/17/201 3/17/201
Collisic L	on Load at T Use a Live L			=															
Collisic L	on Load at T Use a Live L			=															
L	Use a Live L	op of P			1.24	х	0.5	500	x	10.89	=			6.78	k-ft				
F			arapet:																
	Р _{ст}	oad of		1929	lbs/ft app	olied at I	h _r =	3.5 f	t above	the top	of the w	vall.							
Ν														1.93	k				
	M _{CT} = P _{CT} x ((h _w + h _r	,)	=	1.93	x	(10	.89	+	3.50) =			27.75	k-ft				
Using f	the Strength	I load	combin	ation, the	e factore	d desigr	n forces	for the	wall ste	em are:									
ŀ	H _u = 1.50 (P _i	_{EH(S)} + F	P _{EH(B)}) +	⊦ 1.00 P _v	_{va} + 1.75	(P _{LS} +	P _{LL})												
ŀ	H _u = 1.50	х (2.16	+	0.00) +	1.00	x	0.00	+	1.75	х (1.24	+	0.00) =	5.42	k	
Ν	M _u = 1.50 (M	EH(S) +	M _{EH(B)})	+ 1.00 M	I _{WA} + 1.7	5 (M _{LS} +	+ M _{LL})												
Ν	M _u = 1.50	х (7.83	+	0.00) +	1.00	x	0.00	+	1.75	х (6.78	+	0.00) =	23.61	k-ft	
The Extreme	e Event II de	sign for	rces for	the wall	stem are	e:													
H _{ext} = 1	1.50 (P _{EH(S)} ·	+ P _{EH(B)}	₎) + 1.00	0 P _{CT} + 0	.50 (P _{LS}	+ P _{LL})													
H _{ext} =	1.50	х (2.16	+	0.00) +	1.00	x	1.93	+	0.50	х (1.24	+	0.00) =	5.79	k	
$M_{ext} = 2$	1.50 (M _{EH(S)}	+ M _{EH(E}	_{B)}) + 1.0	00 M _{CT} +	0.50 (M _L	_s + M _{LL}))												
M _{ext} =	1.50	х (7.83	+	0.00) +	1.00	x	27.75	+	0.50	х (6.78	+	0.00) =	42.89	k-ft	
The se	ervice design	forces	for the	wall ster	m are:														
ŀ	H _{serv} = 1.00 (P _{EH(S)} +	+ P _{EH(B)}) + 1.00	P _{WA} + 1.0	00 (P _{LS}	+ P _{LL})												
ŀ	H _{serv} =	1.00	х (2.16	+	0.00) +	1.00	x	0.00	+	1.00	х (1.24	+	0.00) =	3.40	k
Ν	M _{serv} = 1.00 ((M _{EH(S)}	+ M _{EH(B}	₃₎) + 1.00	M _{WA} + 1	.00 (M _{Ls}	s + M _{LL})											
Ν	M _{serv} =	1.00	х (7.83	+	0.00) +	1.00	x	0.00	+	1.00	х (6.78	+	0.00) =	14.61	k-ft
Shear typical	Illy does not	govern	the des	-	etaining w	valls. If	shear o	loes be	ecome a	n issue,	, the thic	ckness	of the s	stem sho	ould be inc	reased	such		
Ignorin	ng the benefi	ts of the	e shear	· key and	axial co	mpressi	on, the	shear	capacity	of the	stem ca	n be s	hown to	be grea	ter than th	iat requi	red.		
١	$V_n = V_c + V_s$	+ V _p				[5.8.	3.3-1]												
Recog	nizing that V	$'_{\rm s}$ and ${ m V}$	/ _p are z	ero															
١	V _n = V _c																		
١	V _c = 0.0316	ර (f _c ') ^{0.8}	5 b _v d _v			[5.8.	3.3-3]												
The ma	aximum effe	ctive sh	hear de	pth is:															
F	For 2.0 in	nch clea	ar cove	r and	#8 ba	rs, d _s =	18	.00	-	2.00	- (1.0	00 / 2	2) =	15.50	in			
	d _v = 0.9 d _e = d _v = 0.72 h =	-		0.90 0.72		15.50 18.00	= =	13.9 12.9		<u>GO</u>	/ERNS								
Follow	the General	Proce	dure us	ing the p	orovisions	s of App	endix E	35, as p	er Sect	ion [5.8	.3.4.2]:								
	β =	2.0248	24		ε _s =	= 0.0	0186		S _{xe}	= 11.	42945		Max Ag	gregate	Size =	1 in	(BDM S5	5.10.3.1	.1)

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19.28 k > 5.79 k <u>OK</u>

Wall Stem Design - Investigate Strength Limit State

Determine the area of back-face flexural reinforcement necessary to satisfy the design moment:

M_u = 42.89 k-ft

Again use the equation:

 $3.309 \quad A_s^2 \quad - \quad 4.50 \quad d_s A_s + M_u = 0$

Substituting and solving for A_s , it is found that required $A_s = 0.63 \text{ in}^2/\text{ft}$

Try: #8 bars at 12.00 in c/c $A_s = 0.79 \text{ in}^2/\text{ft} > 0.63 \text{ in}^2/\text{ft} \frac{OK}{S}$

Wall Stem Design - Investigate Service Limit State [5.7.3.4]

Check the crack control equations to ensure that the primary flexural reinforcement is well distributed. The service load bending moment is:

M_{serv} = 14.61 k-ft

Check the modulus of rupture for concrete:

$$\phi f_r = \phi \ 0.24 \ (f_c)^{0.5} = 0.80 \ x \ 0.24 \ x \ (4.0)^{0.50} = 0.384 \ \text{ksi}$$

$$S = \frac{b \ t_{wb}^2}{6} = \frac{12.00 \ x \ (18.00)^2}{6} = 648.00 \ \text{in}^3$$

$$f_{act} = \frac{M_{serv}}{S} = \frac{14.61 \ x \ 12}{648.00} = 0.271 \ \text{ksi} < 0.384 \ \text{ksi} \ OK$$

If modulus of rupture check is "NG", then check the spacing of the reinforcement. First, determine the modular ratio:

$$E_c = 33,000 w_c^{1.5} (f_c)^{0.5} = 33,000 x (0.150)^{1.5} x (4.00)^{0.5} = 3,834$$
 ksi
n = $E_s/E_c = 29,000$ / 3,834 = 7.56, use n = 8.00

For 2.0 inch clear cover and #8 bars, $d_s = 18.00 - 2.00 - (1.000 / 2) = 15.50$ in

Determine the location of the neutral axis:

0.5 b $x^2 = n A_s (d_s - x)$ 0.5 (12) $x^2 = 8 \times 0.79 \times (15.50 - x)$ solving, x = 3.55 in

Check the spacing of the reinforcement to control cracking:

$$y = d_{s} - x = 15.50 - 3.55 = 11.95 \text{ in}$$

$$l_{cr} = \frac{b x^{3}}{3} + n A_{s} (d_{s} - x)^{2}$$

$$l_{cr} = \frac{-12.00 \times (-3.55)^{3}}{3} + 8 \times 0.79 \times (-15.50 - -3.55)^{2} = 1081.47 \text{ in}^{4}$$

$$\gamma_{e} = 1.00$$
For 2.0 inch clear cover and #8 bars, $d_{c} = 2.00 + (-1.000 / 2) = 2.50$ in
$$\beta_{s} = 1.0 + \frac{d_{c}}{0.7 (t_{wb} - d_{c})} = 1.0 + \frac{2.50}{0.7 \times (-18.00 - 2.50)} = 1.23$$

$$f_{ss} = n \frac{M_{serv}y}{l_{cr}} = 8 \times \frac{-14.61 \times 11.95 \times 12.00}{1081.47} = 15.50 \text{ ksi}$$

	T 7	CLIENT PROJE	CT CUY-2	71-0.00) (PID 804	118)				_	PROJE	CT NO.		1	122-1001-	-00
		SUBJE	CT Reinfo Wall V	rced Co VS1, Pa	oncrete R inels 11-1	etaining 3	Wall De	sign		_	COMP. CHECK		AS LN		DATE DATE	3/17/20 3/17/20
	700 γ _e			700	x	1.00)									
S _{max}	$= \frac{\beta_{s} f_{ss}}{\beta_{s} f_{ss}}$	2.0 0	d _c = -	1.23	х	15.5	<u>0</u> -	2.0	x 2	2.50	=	31.70	in			
s _{max}	= 31.70	in	> 12.0	00 in	<u>OK</u>											
Wall Stem Desi	ign - Check Reinf	forcemen	t Limits													
Check Mi	nimum Reinforce	ement	[5.7.3.	3.2]												
Dete	ermine the crackin	g momen	t:													
	$f_r = 0.24 (f_c')^{0.5}$	=	0.24	x (4	.0) ^{0.50}	=	0.48	ksi								
	l _g = (1/12) b h ³	=	0.0833	x	12.00	х (18.00) ³ =	5832.0	in⁴						
	y _t = (1/2) h	=	0.5000	x	18.00	=	9.00	in								
M _{CR}	$f_{g} = \gamma_1 \gamma_3 f_r I_g / y_t$	=	0.67	x	1.6	x	0.48	x	5832.0	/ (9.00	x	12.00) =	27.79	k-ft
The	capacity of the se	ction mus	t be greate	er than o	or equal t	o the sn	naller of:									
	M _{CR} = (4/3) M _u =	1.33	x	27.79 42.89		27.7 57.1		<u>GOVE</u>	<u>RNS</u>							
The	capacity of the rei	inforceme	nt is:													
	$M_r = \phi A_s f_y (d_s -$	- a/2)														
	M _r = 0.90 x	0.79	x 60	х (15.50		0.79 1.7 x	9 x 4.0	60 x 12	—) x (<u>1</u>)	=	53.04	k-ft		
	5	3.04 k-	ft >	27.79) k-ft	<u>OK</u>										
Che	ck minimum reinfo	orcement	for temper	ature ar	nd shrinka	age (5.1	0.8)		A _s =	0.	79 in	²/ft >	0.17	in²/ft	<u>OK</u>	
<u>Use</u>	#8 bars at 12.00 i	in c/c for v	vall stem b	ack fac	e vertical	reinford	ing.									
A minimur	ign - Shrinkage a m amount of reinfo and temperature o	orcement				-	5.10.8] f concrete	e elemer	nts to limit	the siz	e of crao	ks assoc	iated with	concrete	9	
	h _{max} = max(t _{wt} , t _v	wb)	= 18.0	00 in												
	h _w = 10.89	ft	= 130.	68 in												
	Min. A _s =	1.3 2 (h	30 h _w h _{max} n _w + h _{max}) 1	Fy	=2	1.30 x (x 13 130.68	30.68 +	x 1 18.00	8.00) x	60	= 0	.17 in ²	/ft		
The	maximum spacing	g of reinfo	rcement is	:												
	$h_{min} = min(t_{wt}, t_{wt})$	_b)	= 18.0	00 in												
	Max. spacing = 3 or Max. spacing or Max. spacing	= 18 in	or walls an	d footin	gs ≥ 18 ir	n thick)	= = =	54.0 18.0 12.0	00 in	GOV	ERNS					
Try:	#4 bars at	12.00) in c/c						A _s =	0.	20 in	²/ft >	0.17	in²/ft	<u>OK</u>	
									0.1	<	A _s =	0.20	in²/ft <	0.60	<u>OK</u>	

Use #4 bars at 12.00 in c/c for wall stem front face reinforcing and back face horizontal reinforcing.