US Army Corps of Engineers

MVP - Design Branch
Section: *(Structural/MECS/Civil)*
Calculation Cover Sheet and Design Check Documentation

Project Name:	Fargo Moorhead Metro Wild Rice River Control Structure	Date:	2-Oct-17
Project Location:	Fargo North Dakota	District/Customer:	St Paul
Project Manager:	Bonnie Greenleaf		

File Location:

Designer/Checker Information

Title of Calculations to be checked:	Control Building
Number of pages Including Cover Sheet	17
Assigned Checker:	Paul Muller, P.E.
Designer/Originator(of calculations):	C. Johnny Walker, P.E.

Additional Information:

Design Check Documentation

Check Box 1 or 2

1. [ ] All items have been checked in accordance with District QMP and found to be correct. Checker has no comments.

Checker's Signature: ______________________ Date: __________

OR

2. [X] Checker's comments have been provided on:

X	Calculations
X	Other Plans Attached

If box 2 is checked above, the section below to be completed after backcheck of any comments.

Check Box 3, **OR** go on to Box 4 **AND** Box 5

3. [X] Checker's comments have been adequately addressed by Designer/Originator and all issues have been resolved between Checker and Designer. The checker has backchecked all comments and reviewed all revised calculations to assure incorporation into final document.

OR

4. [ ] There are unresolved comments, and these have been submitted to the Section Chief or designee for resolution.

AND

5. [ ] Comments have been resolved by Section Chief or designee. The checker has backchecked comments and reviewed all revised calculations to assure that resolved comments have been incorporated into final document.

Checker's Signature: MULLER.PAUL.DANIEL.1291042232 2017.10.27 12:14:49 -05'00' Date: __________

QA Sign-Off

The Design/Calculation Check is complete and all comments have been resolved and closed out.

QA Signature: ______________________ Date: __________

Section Chief or Designee

US Army Corps of Engineers ®
Kansas City District

CALCULATION COVER SHEET

Project Name: Wild Rice River Sturcture **Labor Code:** 2D4D16
Calculation Title: Control Building
Total No. of Pages:
Prepared By: C. Johnny Walker, P.E. **Date:** 2-Oct-17
Checked By: Paul Muller, P.E. **Date:** 5-Oct-17

Design Basis/References/Assumptions:

IBC 2015
ASCE 7-10
ACI 318-14

Rev. No.	Description of Revision:	Prepared By:	Date:	Checked By:	Date:

Sheet Index:

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

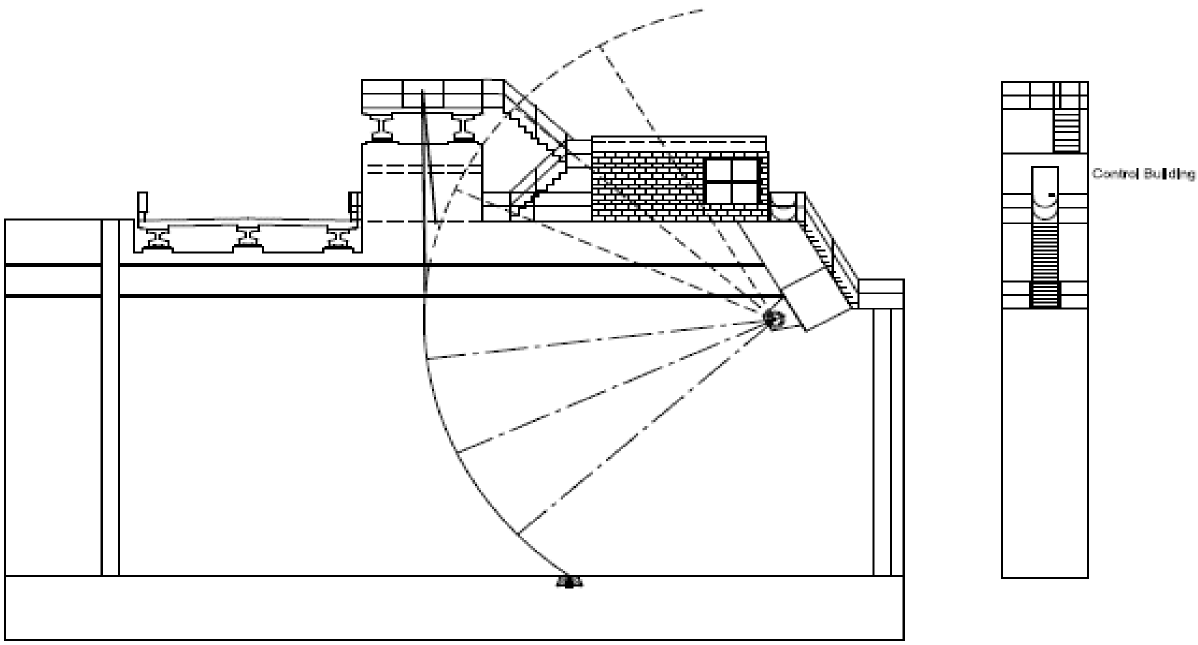
Project: Wild Rice River Structure

Subject: Design of Control Building

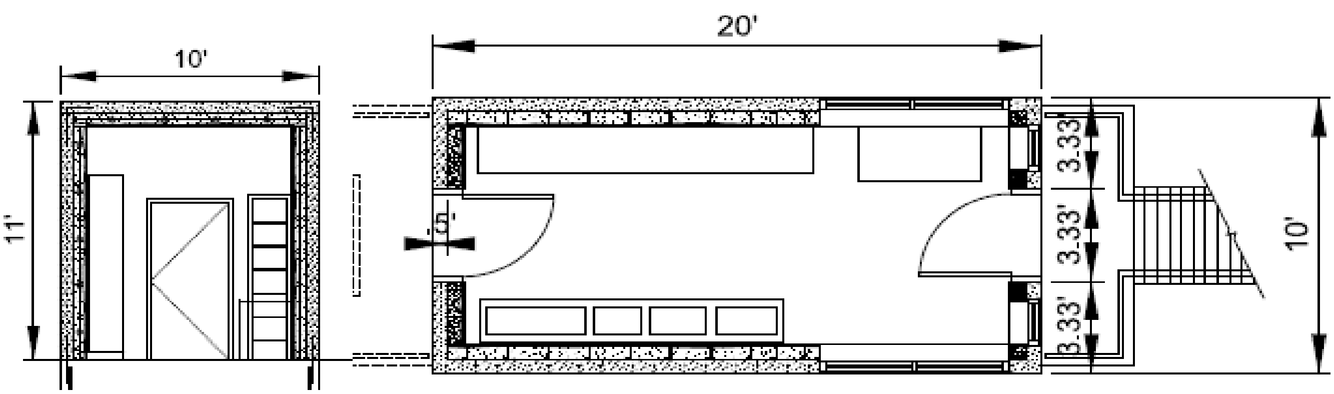
DOR: J. Walker **Date:** 2-Oct-17 **Checker:** P. Muller **Date:** 5-Oct-17

Control Building Plan and Elevations in relation to Control Structure

The control building will sit on the center pier of the control structure. Access to the control structure is provided from the service bridge and through a tunnel under the mechanical platform. The control building will have a rear door for access to the trunnion girders. The purpose of the control building is to house all the control panels for operation of the tainter gates. The control building will be semi-heated with operable windows and will have a 3" housekeeping pad as required. The building will consist of cast-in-place reinforced concrete walls with a CIP flat roof sloped to drain. The walls will be designed as a cantilever section on 3-sides. A moment frame will be provided on the trunnion girder side due to the door and window openings.

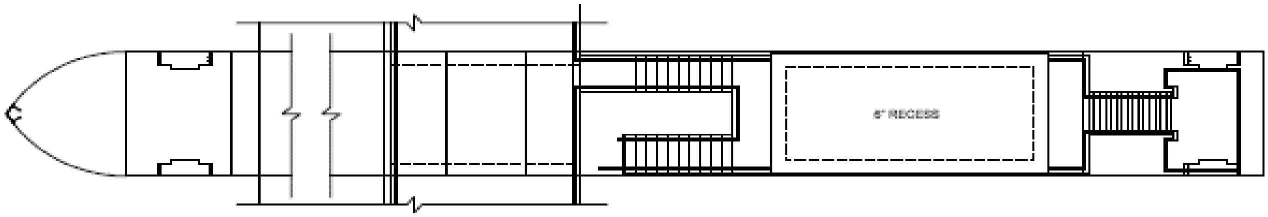


Architectural Floor Plan and Elevations



top of pier elev. =	931.50	
top of building elev. =	942.50	
top of water elev. =	890.60	(assumes 4' of water)

The following sketch shows the location of the recess required in the pier to accommodate 3" rigid insulation and 3" concrete topping.



Loads & Load Combinations

Basic Load Combinations (exempt for seismic):

1) 1.4D
2) 1.2D + 1.6L + $0.5L_r$
3) 1.2D + 1.6L + 0.5S
4) 1.2D + $1.6L_r$ + 0.5L
5) 1.2D + 1.6S + 0.5L
6) 1.2D + $1.6L_r$ + 0.5W
7) 1.2D + 1.6S + 0.5W
8) 1.2D + 1.0W + L + $0.5L_r$
9) 1.2D + 1.0W + L + 0.5S
10) 0.9D + 1.0W

The load combinations are derived from the basic load combinations shown in ASCE 7, section 2.3.2 for Strength Design. Based on the relative location of this project to the Diversion Inlet project and the seismic evaluation performed for that project, it is determined that this project is exempt from seismic design.

<u>Dead:</u>	Roof:	concrete slab:	75	psf	(assumes 6" thick)
		1/2" Gypsum:	2	psf	
		spray foam insulation:	0.3	psf	
		metal studs:	1.1	psf	
		lighting:	4	psf	(assumed)
			82.4	psf	
	Wall:	6" thick RC wall:	75	psf	(per foot height)

The metal stud wall with spray foam insulation is non-structural. The interior features such as the control panels will be supported by this wall. Therefore, the RC walls do not require design loads associated with those features.

<u>Live:</u>	roof:	125	psf	(ordinary flat)

The roof live load assumes the roof will be used for light storage in the future due to its relatively flat nature.

<u>Snow:</u>	Risk Category:	IV		(occupied during emergencies)
	Importance Factor, I_s:	1.2		
	ground snow load, p_g:	50	psf	
	flat roof snow load, p_f:	$0.7C_eC_tI_sp_g$		
	exposure factor, C_e:	0.9		(exposure C)
	thermal factor, C_t:	1.0		
	p_f:	37.8	psf	(greater than min. required)

Wind: (determined in accordance with ASCE 7-10, Chapter 27 Directional Procedure)

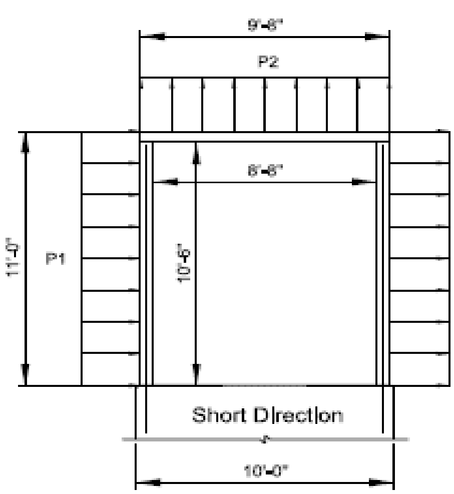
Risk Category:	IV	
Design wind speed, V:	120	mph
Wind directionality factor, K_d:	0.85	
Exposure Category:	C	
Topographic factor, K_{zt}:	1.0	
Gust effect factor, G:	0.85	
Enclosure classification:	Partially enclosed	
Internal pressure coefficient, GC_{pi}:	-0.55 0.55	
height above water surface:	51.90	ft
mean roof height, h:	11	ft
top of structure, z:	11	ft
Velocity pressure exposure coefficient, K_z:	1.09	
Velocity pressure exposure coefficient, K_h:	1.09	
$q = q_h = q_z =$	$0.00256K_zK_{zt}K_dV^2$	
$q = q_h = q_z =$	34.15	psf
building length:	20	ft
building width:	9.6667	ft

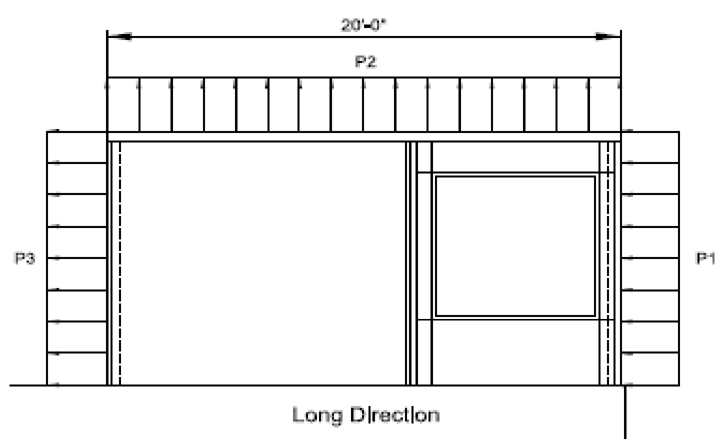
Compute external pressure coefficients:

Short Direction		Long Direction	
L/B =	0.48	L/B =	2.07
h/L =	1.14	h/L =	0.55
Windward, C_p:	0.8	Windward, C_p:	0.8
Leeward, C_p:	-0.5	Leeward, C_p:	-0.3
Sidewall, C_p:	-0.7	Sidewall, C_p:	-0.7
Roof, C_p:	-1.3	Roof, C_p:	-0.9
	-0.18		-0.18

Note: At 11 feet along the roof, the pressure coefficient is reduced to -0.5 which will result in a lower suction force. Therefore, it is conservative to use -0.9 for the entire roof length.

design wind pressure, $p = qGC_p - q_i(GC_{pi})$





Walls

		Short Direction			Long Direction		
Windward:	P1 =	42.01	psf	$(-GC_{pi})$	42.01	psf	$(-GC_{pi})$
		4.44	psf	$(+GC_{pi})$	4.44	psf	$(+GC_{pi})$
Leeward:	P3 =	4.27	psf	$(-GC_{pi})$	10.08	psf	$(-GC_{pi})$
		-33.30	psf	$(+GC_{pi})$	-27.49	psf	$(+GC_{pi})$
Sidewall:	P4 =	-1.54	psf	$(-GC_{pi})$	-1.54	psf	$(-GC_{pi})$
		-39.11	psf	$(+GC_{pi})$	-39.11	psf	$(+GC_{pi})$

Roof

		Short Direction			Long Direction		
Windward:	P2 =	-18.96	psf	$(-GC_{pi})$	-7.34	psf	$(-GC_{pi})$
		-56.53	psf	$(+GC_{pi})$	-44.91	psf	$(+GC_{pi})$
		13.56	psf	$(-GC_{pi})$	13.56	psf	$(-GC_{pi})$
		-24.01	psf	$(+GC_{pi})$	-24.01	psf	$(+GC_{pi})$

Project: Wild Rice River Structure

Subject: Design of Control Building

DOR: J. Walker **Date:** 2-Oct-17 **Checker:** P. Muller **Date:** 5-Oct-17

Design of Control Building Roof

Given:

Table 7.3.1.1—Minimum thickness of solid nonprestressed one-way slabs

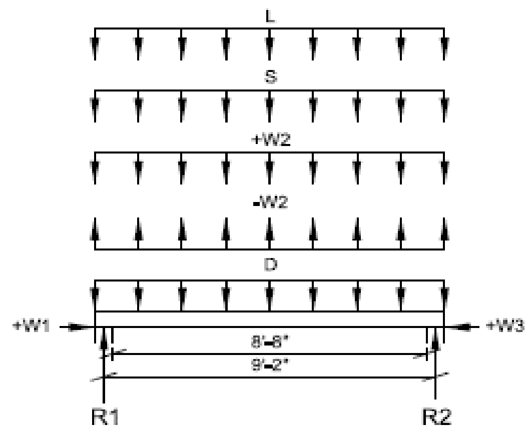
Support condition	Minimum h[1]
Simply supported	$\ell/20$
One end continuous	$\ell/24$
Both ends continuous	$\ell/28$
Cantilever	$\ell/10$

[1]Expression applicable for normalweight concrete and f_y = 60,000 psi. For other cases, minimum h shall be modified in accordance with 7.3.1.1.1 through 7.3.1.1.3, as appropriate.

Reference: ACI 318-14

clear span, L:	8.6667	ft
slab thickness, h:	5.2	in
use:	6	in
d =	3.5	in
f_y =	60000	psi
f'_c =	4500	psi
clear cover, c =	2	in
λ =	1.00	
(assumes 1 foot strip width)		
b =	12	in

dead load, D =	82.4	psf
live load, L =	125	psf
positiveroof wind load, $W2_{(+)}$ =	13.56	psf
negative roof wind load, $W2_{(-)}$ =	-56.53	psf
roof snow load, S =	37.80	psf
max windward wall pressure, W1 =	42.01	psf
max leeward wall pressure, W3 =	10.08	psf



Design Methodology:

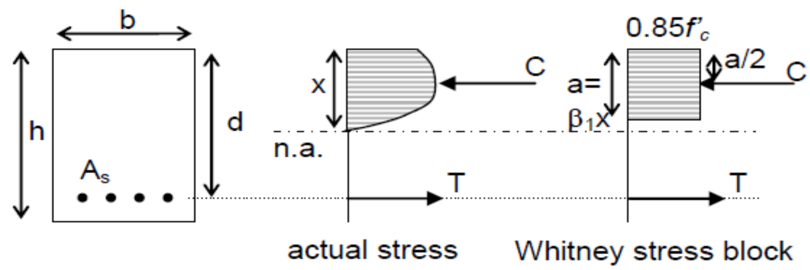
The roof slab is designed as simply supported, one-way. The primary reinforcement is located centered in the slab. An eccentric axial load is applied at the interface between the wall and slab.

LC#			
1)	1.4D =	115.36	psf
3)	1.2D + 1.6L + 0.5S =	317.78	psf
5)	1.2D + 1.6S + 0.5L =	221.86	psf
7a)	1.2D + 1.6S + 0.5W2 =	166.14	psf
7b)	1.2D + 1.6S - 0.5W2 =	131.10	psf
9a)	1.2D + 1.0W2 + L + 0.5S =	256.34	psf
9b)	1.2D - 1.0W2 + L + 0.5S =	186.25	psf
10a)	0.9D + 1.0W2 =	87.72	psf
10b)	0.9D - 1.0W2 =	17.63	psf

$M_u = wL^2/8$

M_u = 2984 lb-ft

Since the slab only sees positive moment, the axial load, when applied, would reduce the bending moment. As a result, the axial load will be negated for conservity.



$\beta_1 =$ 0.85

Let C = T, $A_s f_y = 0.85f'_c ba$

$A_s =$ 0.20 in^2

a = 0.2614 in

x = 0.3076 in

$M_n = A_s f_y (d-a/2)$

$M_n =$ 3,369 lb-ft

$\phi =$ 0.9

$\phi M_n =$ 3,032 lb-ft

check $\phi M_n \geq M_u$: **OK**

Bar No.	d_b (in)	area (in^2)	cover (in)	A_{st}/ft	
4	0.500	0.20	1.5	0.300	<--Try
5	0.625	0.31	1.5	0.465	
6	0.750	0.44	2.0	0.660	
7	0.875	0.6	2.0	0.900	
8	1.000	0.79	2.0	1.185	

Check Deflection:

$$\delta = \frac{5wl^4}{384E_c I_e}$$

$w_S =$ 37.8 psf

$w_W =$ 13.56 psf

$w_{D+L} =$ 207.40 psf

$w_L =$ 125 psf

$$E_c = 57{,}000\sqrt{f'_c} \text{ (in psi)} \qquad (19.2.2.1.b)$$

$E_c =$ 3,823,676 psi

$$f_r = 7.5\lambda\sqrt{f'_c} \qquad (19.2.3.1)$$

$f_r =$ 503.12 psi

$I_g =$ 216 in^4

$y_t =$ 3 in

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad (24.2.3.5b)$$

$M_{cr} =$ 36,224 lb-in

$M_a =$ 23,367 lb-in

$I_{cr} =$ 64 in^4

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad (24.2.3.5a)$$

$I_e =$ 630 in^4

					check
deflection limit:	l/240 =	0.4333 in	$\delta^{FEM}_{D+L} =$	0.0009 in	**OK**
	l/360 =	0.2889 in	$\delta^{FEM}_{L} =$	0.0005 in	**OK**

It should be noted that none of the load combinations investigated show the roof lifting off. As a result, the primary reinforcement will be located such that the maximum capacity can be achieved. See drawings for final "d".

Check $A_s > A_{s,min}$

Check 1: $A_{s,min} = 3(\sqrt{f'_c})b_w d/f_y$

$A_{s,min} =$ 0.141 in^2

Check 2: $A_{s,min} = 200 b_w d/f_y$

$A_{s,min} =$ 0.140 in^2

Since both the min. reinforcement values are less than provided, the design reinforcement is ok.

Check Shear:

$V_u =$ 1377 lbs

$\phi =$ 0.75

$$V_c = 2\lambda\sqrt{f'_c}b_w d \qquad (22.5.5.1)$$

$\lambda =$ 1

$V_c =$ 5635 lbs

Let $V_n = V_c$

$\phi V_n =$ 4226 lbs

The axial load is not included as it would increase the shear strength of the slab. Therefore, conservatively, the axial load has been neglected.

check $\phi V_n \geq V_u$: **OK**

Project: Wild Rice River Structure

Subject: Design of Control Building

DOR: J. Walker **Date:** 2-Oct-17 **Checker:** P. Muller **Date:** 5-Oct-17

Design of Control Building Walls

Given:

Table 11.3.1.1—Minimum wall thickness *h*

Wall type	Minimum thickness *h*		
Bearing[1]	Greater of:	4 in.	(a)
		1/25 the lesser of unsupported length and unsupported height	(b)
Nonbearing	Greater of:	4 in.	(c)
		1/30 the lesser of unsupported length and unsupported height	(d)
Exterior basement and foundation[1]	7.5 in.		(e)

[1]Only applies to walls designed in accordance with the simplified design method of 11.5.3.

Reference: ACI 318-14

effective wall height, L:	10.5	ft
wall thickness, t:	5.04	in
use:	6	in
d =	3	in
f_y =	60000	psi
f'_c =	4500	psi
clear cover, c =	2	in
max pressure, W =	42.01	psf
(assumes 1 foot strip width)		
b =	12	in

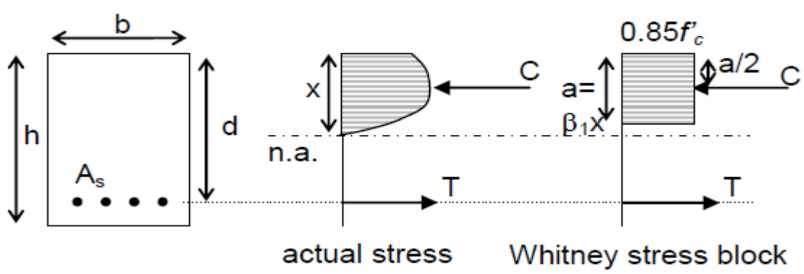
Design Methodology:

The walls are designed assuming simply supported and the base and at the roof diaphragm. Where there are openings, a moment frame will be designed.

$1.2D + 1.0W + L + 0.5L_r$

$1.2D + 1.0W + L + 0.5S$

$0.9D + 1.0W$



Bar No.	d_b (in)	area (in^2)	cover (in)	A_{st}/ft	
4	0.500	0.20	1.5	0.300	<--Try
5	0.625	0.31	1.5	0.465	
6	0.750	0.44	2.0	0.660	
7	0.875	0.6	2.0	0.900	
8	1.000	0.79	2.0	1.185	

$M_u = wL^2/8$

M_u =	579	lb-ft
β_1 =	0.85	

Let C = T, $A_s f_y = 0.85f'_c ba$

A_s =	0.20	in^2
a =	0.2614	in
x =	0.3076	in

$M_n = A_s f_y(d-a/2)$

M_n =	2,869	lb-ft
ϕ =	0.9	
ϕM_n =	2,582	lb-ft
V_u =	221	lbs
ϕ =	0.75	

$$V_c = 2\lambda\sqrt{f'_c}b_w d \qquad (22.5.5.1)$$

λ =	1	
V_c =	4830	lbs

Let $V_n = V_c$

ϕV_n =	3622	lbs

Check section @ opening:

width of door:	3.3333	ft
width of section to be considered:	0.6666667	ft
$w =$	98	plf
	$M_u = wL^2/8$	(assume full height - conservative)
$M_u =$	1,351	lb-ft

In order to minimize differential displacements, two #4 bars will be provided within the 12" section.

Check Shear:

$V_u =$	515	lbs
$b =$	8.0000	in
$V_c =$	3,220	lbs
Let $V_n = V_c$		
$\phi V_n =$	2,415	lbs

90 deg. Hooked Anchor Development Length:

greater of: (a) $\left(\frac{f_y \psi_e \psi_c \psi_r}{50\lambda\sqrt{f'_c}}\right) d_b$ with ψ_e, ψ_c, ψ_r, and λ given in 25.4.3.2.

(b) $8d_b$

(c) 6 in.

$\psi_e =$	1.0	*(uncoated)*
$\psi_r =$	1.0	*(not confined)*
$\psi_c =$	0.7	*(cover > 2.5" all sides)*
a)	6.3 in	<--- controls
b)	4.0 in	
c)	6.0 in	

The hook development length will start beyond the 6" recess depth. Thus total length below the top of concrete will be 13".

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Project: Wild Rice River Structure

Subject: Design of Control Building

DOR: J. Walker **Date:** 2-Oct-17 **Checker:** P. Muller **Date:** 5-Oct-17

Design of Control Building MWFRS

Reference: ACI 318-14

overall height, H: 11 ft

overall length, L: 20 ft

wall thickness, t: 6 in

d = 3 in

f_y = 60000 psi

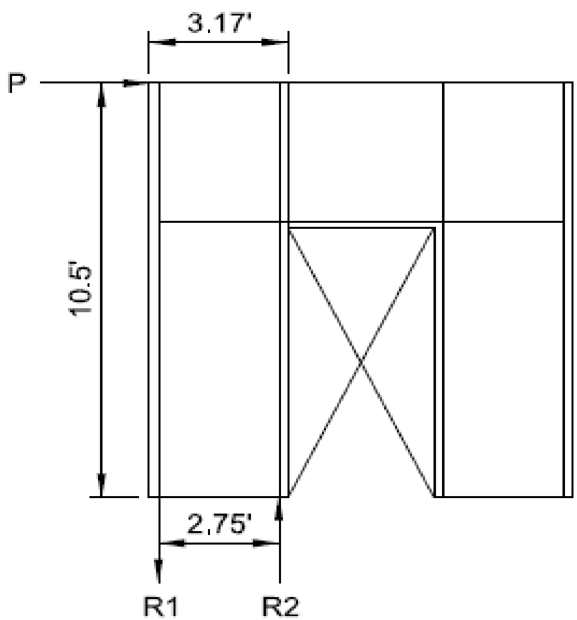
f'_c = 4500 psi

clear cover, c = 2 in

max pressure, w = 42.01 psf

horizontal load from roof diaphragm, P = 2311 lbs

Description of Load Path: The MWFRS will consist of a shear wall on the north side of the building and a moment frame on the south side. The wind load in transferred from the wall to the pier and roof diaphragm. The roof diaphragm transfers the load to the shear wall and moment frame then to the pier.



Conservatively, the weight of the wall and roof diaphragm are neglected.

By summing the moments about the reinforcement anchors you get:

$\sum M_{R1} = 0$ (+ clockwise)

P(10.5) - R2(2.75) = 0 (compression)

R2 = 8822 lbs

$\sum F_Y = 0$ (+ up)

-R1 + R2 = 0

R1 = 8822 lbs (tension)

Compute overturning moment:

M_u = P(10.5)

M_u = 24,261 lb-ft

β_1 = 0.85

Let C = T, $A_s f_y = 0.85 f'_c ba$ let b = t

Bar No.	d_b (in)	area (in^2)	cover (in)	A_{st}/ft
4	0.500	0.20	1.5	0.300
5	0.625	0.31	1.5	0.465
6	0.750	0.44	2.0	0.660
7	0.875	0.6	2.0	0.900
8	1.000	0.79	2.0	1.185

Try # 4

A_s = 0.20 in^2

a = 0.5229 in

x = 0.6151 in

$M_n = A_s f_y(d-a/2)$

M_n = 32,739 lb-ft

ϕ = 0.9

ϕM_n = 29,465 lb-ft

check $\phi M_n \geq M_u$: **OK**

Check Deflection: E_c = 3823676.2 psi Δ_{max} = 0.0224 in

I = 17,969 in^4

Check Shear:

$V_u =$ 2311 lbs

$\phi =$ 0.75

$$V_c = 2\lambda\sqrt{f_c'}b_w d \qquad (22.5.5.1)$$

$\lambda =$ 1

$V_c =$ 26,564 lbs

Let $V_n = V_c$

$\phi V_n =$ 19,923 lbs

check $\phi V_n \geq V_u$: **OK**

Design Summary: The shear wall w/ #4 bars as designed for the wall to resist the design lateral loads is adequate to also resist overturning provided the bar is fully developed into the foundation. Shear is also of no issue. Therefore it is not necessary to design a coupling beam and traditional reinforcement will be prescribed.

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Project: Wild Rice River Structure

Subject: Design of Control Building

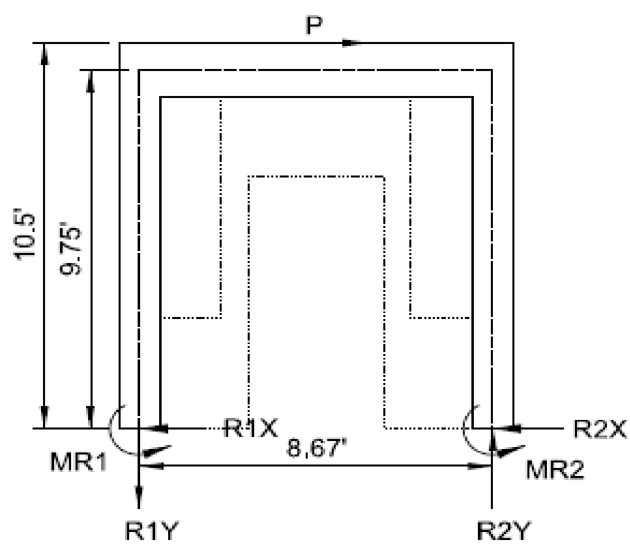
DOR: J. Walker **Date:** 2-Oct-17 **Checker:** P. Muller **Date:** 5-Oct-17

Design of Control Building MWFRS

Reference: ACI 318-14

overall height, H:	11	ft
overall length, L:	20	ft
column width, t:	7.5	in
d =	9.5	in
f_y =	60000	psi
f'_c =	4500	psi
clear cover, c =	2	in
max pressure, w =	42.01	psf
horizontal load from roof diaphragm, P =	2311	lbs

Description of Load Path: The MWFRS will consist of a shear wall on the north side of the building and a moment frame on the south side. The wind load in transferred from the wall to the pier and roof diaphragm. The roof diaphragm transfers the load to the shear wall and moment frame then to the pier.



Conservatively, the weight of the wall and roof diaphragm are neglected.

By summing the moments about the reinforcement anchors you get:

Using the Portal Method...

$\sum F_X = 0$ (assume R1X = R2X)

R1X = 1155 lbs

R2X = 1155 lbs

Assume the bending moment in the left column is equal to the bending moment in the right column.

Therefore: MR1 = MR2

MR1 = R1X(10.5/2)

MR1 = 6065 lb-ft

$\sum M_{R1} = 0$ (+ clockwise)

-MR1 - MR2 + P(10.5) - R2Y(8.67) = 0

R2Y = 1399 lbs (compression)

$\sum F_Y = 0$ (+ up)

-R1Y + R2Y = 0

R1Y = 1399 lbs (tension)

MR1 = MR2 = M_u = 6,065 lb-ft

β_1 = 0.85

Let C = T, $A_sf_y = 0.85f'_cba$ let b = t

Try # 4 qty. = 2

A_s = 0.40 in^2

a = 0.8366 in

x = 0.9842 in

$M_n = A_sf_y(d-a/2)$

M_n = 18,163 lb-ft

ϕ = 0.9

ϕM_n = 16,347 lb-ft

check $\phi M_n \geq M_u$: **OK**

Bar No.	d_b (in)	area (in^2)	cover (in)	A_{st}/ft
4	0.500	0.20	1.5	0.300
5	0.625	0.31	1.5	0.465
6	0.750	0.44	2.0	0.660
7	0.875	0.6	2.0	0.900
8	1.000	0.79	2.0	1.185

Check Deflection:

The below deflection calc assumes a cantilever column. The horizontal force is divided between the two columns. This is a very conservative quick check and depending on the results, a more detailed check can be performed based on a frame analysis with fixed end moments.

E_c = 3,823,676 psi

d = 9.5 in

b = 7.5 in

I = 536 in^4

Δ_{max} = 0.4323 in

If the gross moment of inertia of the frame is used with the full horizontal force, the deflection will be even smaller than shown left. Therefore by inspection, the differential deflection between the shear wall and moment frame is substantially small and will not cause any structural issues.

Check Shear:

V_u = 2311 lbs

ϕ = 0.75

$$V_c = 2\lambda\sqrt{f'_c}b_wd \qquad (22.5.5.1)$$

λ = 1

V_c = 9,559 lbs

Let $V_n = V_c$

ϕV_n = 7,169 lbs

check $\phi V_n \geq V_u$: **OK**

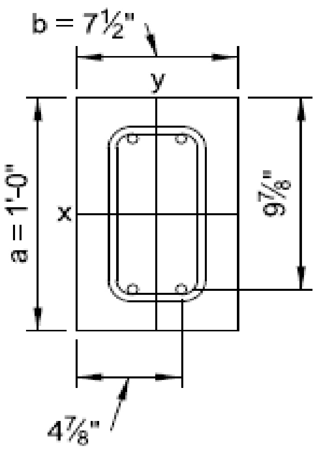
Design Summary: The moment frame will consists of four #4 bars. in the columns and the beams will have additional bars.

Check Slenderness: (ACI 318-14, Section 6.2.5)

In the transverse direction: column is braced against sway

(b) For columns braced against sidesway

$$\frac{k\ell_u}{r} \le 34 + 12(M_1/M_2) \qquad (6.2.5b)$$



d_b =	9.875	in
b =	7.5	in
d_a =	4.875	in
a =	12	in
column, I_x =	1080	in^4
column, I_y =	422	in^4
column, EI_x =	4,129,570	$k\text{-}in^2$
column, EI_y =	1,613,113	$k\text{-}in^2$
column length, l_c =	6.8333	ft
beam, d =	3.5	ft
beam, I =	46305	in^4
beam, EI =	177055328	$k\text{-}in^2$
beam length, l =	8.6667	ft
ψ_{Ax} =	50,361	
ψ_{Ay} =	19,672	
ψ_B =	1,702,449	
k =	0.84	
k =	2.0	
A_g =	72	in^2
r_x =	3.87	in
r_y=	2.42	in
kl/r_x =	17.78	
kl/r_y =	28.46	

Based on these values, slenderness effects can be ignored.

Table 20.6.1.3.1—Specified concrete cover for cast-in-place nonprestressed concrete members

Concrete exposure	Member	Reinforcement	Specified cover, in.
Cast against and permanently in contact with ground	All	All	3
Exposed to weather or in contact with ground	All	No. 6 through No. 18 bars	2
		No. 5 bar, W31 or D31 wire, and smaller	1-1/2
Not exposed to weather or in contact with ground	Slabs, joists, and walls	No. 14 and No. 18 bars	1-1/2
		No. 11 bar and smaller	3/4
	Beams, columns, pedestals, and tension ties	Primary reinforcement, stirrups, ties, spirals, and hoops	1-1/2

END OF CALCULATIONS