

# COMMUNITY OF MILNER WWTP IMPROVEMENT PROJECT MILNER, CO PROJECT NO. 2310219 (3)

STRUCTURAL CALCULATIONS

DANIEL MAZZEI, P.E., CWI, RRO ENGINEER OF RECORD

> wallace design collective, pc structural - civil - landscape - survey 9800 pyramid court, suite 350 englewood, colorado 80112 303.350.1690 - 800.364.5858



## TABLE OF CONTENTS

- 01 DESIGN LOADS
- 02 WALLS
- 03 LID
- 04 FOUNDATION SLAB
- 05 BUOYANCY

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## 01 DESIGN LOADS



123 north martin luther king jr. boulevard tulsa, oklahoma 74103 918.584.5858 · 800.364.5858 wallace.design



# CODE CHECK

DATE:	8/24/23										
TO:	Routt County Building Department Routt Count Courthouse Annex 136 6th St. Suite 201 Steamboat Springs, CO 80487										
PHONE:	970-870-5566 FAX: 970-870-5489										
ATTN:	Todd Carr, Building Official EMAIL: tcarr@co.routt.co.us										
PROJECT:	# 2310219 (	(8) Wastewa	ter Treatme	ent Tanks in Vari	ous Town	s Variou	us Towns,				
BY:	PHONE X	VISIT	OTHER		TIME:						
ITEM	DESCRIPTIO	Ν			RESPON	SE					
1. GOVER	NING CODE										
А. В. С. D. Е.	Local Buildin Local Ameno Do State Bui Observations Special Inspe- for Certificat	ng Code: Iments: Iding Code R s Required to ections Final the of Occupation	ode:     2018     IBC International Building Contents       hts:     Yes       g Code Requirements Differ?     Yes       quired to be performed by EOR?     No       ons Final Report Required     No		ernational Building Code						
2. ROOF LI	IVE LOAD										
Α.	Minimum Ro	of Live Load				20 psf					
3. SNOW L	OAD										
А. В.	Ground Snov Minimum Ro	w Load, Pg: of Snow Loa	d, Pf:			77 psf					
4. WIND LC	DAD										
A. B. 5. SEISMIC	Design Wind Risk Categoi CLOAD	l Speed: ry				112 mph III					
А. В.	Mapped Spe Mapped Spe	ectral Respon ectral Respon	se Acceler se Acceler	ation, Ss: ation, S1:		0.300 0.055	(short period, 0.2s) (long period, 1.0s)				
6. FROST I A.	DEPTH Minimum Be	aring Depth:				48 in.					
REMARKS: Milner, CO 2021 IBC is being adopted Effective January 1, 2024 Snow Load used is max between Routt County Snow Load Map tool and ASCE 7-16 CO Design Snow Loads equation based on K-value and elevation											

Please notify the undersigned if the above information is incorrect or incomplete.

FROM: Steve Jacob, P.E.

wallace design collective, pc

structural · civil · landscape · survey 123 north martin luther king jr. boulevard tulsa, oklahoma 74103 918.584.5858 · 800.364.5858 wallace.design

#### CC:



# ASCE 7 Hazards Report

Standard:

ASCE/SEI 7-16

Latitude:

**Risk Category:** III

Soil Class: C - Very Dense

40.482229 Longitude: -107.020721 Elevation: 0 ft (NAVD 88) Soil and Soft Rock



# Wind

### **Results:**

Wind Speed	112 Vmph
10-year MRI	76 Vmph
25-year MRI	83 Vmph
50-year MRI	87 Vmph
100-year MRI	92 Vmph

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1C and Figs. CC.2-	-1-CC.2-4, and Section 26.5.2
Date Accessed:	Thu Nov 09 2023	

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (annual exceedance probability = 0.000588, MRI = 1,700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.



#### Site Soil Class:

### **Results:**

S <sub>S</sub> :	0.581	S <sub>D1</sub> :	0.102
<b>S</b> <sub>1</sub> :	0.102	Τ <sub>L</sub> :	4
F <sub>a</sub> :	1.267	PGA :	0.405
<b>F</b> <sub>v</sub> :	1.5	PGA M :	0.486
S <sub>MS</sub> :	0.737	F <sub>PGA</sub> :	1.2
S <sub>M1</sub> :	0.153	l <sub>e</sub> :	1.25
S <sub>DS</sub> :	0.491	C <sub>v</sub> :	0.988







## Data Accessed:

Thu Nov 09 2023

### Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.



#### **Results:**

Mapped Elevation:

Data Source:

Date Accessed:

Thu Nov 09 2023

In "Case Study" areas, site-specific case studies are required to establish ground snow loads. Extreme local variations in ground snow loads in these areas preclude mapping at this scale.

Ground snow load determination for such sites shall be based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2 percent annual probability of being exceeded (50-year mean recurrence interval).

Statutory requirements of the Authority Having Jurisdiction are not included. Site is outside ASCE/SEI 7-16, Table 7.2-2 boundaries. For ground snow loads in this area, see SEAC Snow Load Committee. (2016). <u>Colorado Designative State State Transpegitors 6.5 Sine intersolution</u>. Pfield resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported 'elevation' and 'mapped elevation' differ significantly from each other.





A = 6.5 (thou/sandszoftfeetine/elevation)



The ASCE 7 Hazard Tool is provided for your convenience, for informational purposes only, and is provided "as is" and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE 7 standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

ASCE does not intend, nor should anyone interpret, the results provided by this Tool to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this Tool or the ASCE 7 standard.

In using this Tool, you expressly assume all risks associated with your use. Under no circumstances shall ASCE or its officers, directors, employees, members, affiliates, or agents be liable to you or any other person for any direct, indirect, special, incidental, or consequential damages arising from or related to your use of, or reliance on, the Tool or any information obtained therein. To the fullest extent permitted by law, you agree to release and hold harmless ASCE from any and all liability of any nature arising out of or resulting from any use of data provided by the ASCE 7 Hazard Tool.



#### **Results:**

Mapped Elevation:

Data Source:

Date Accessed:

Thu Nov 09 2023

In "Case Study" areas, site-specific case studies are required to establish ground snow loads. Extreme local variations in ground snow loads in these areas preclude mapping at this scale.

Ground snow load determination for such sites shall be based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2 percent annual probability of being exceeded (50-year mean recurrence interval).

Statutory requirements of the Authority Having Jurisdiction are not included. Site is outside ASCE/SEI 7-16, Table 7.2-2 boundaries. For ground snow loads in this area, see SEAC Snow Load Committee. (2016). <u>Colorado Designative state that between the mapped elevation and the site-specific</u> create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in class where the reported 'elevation' and 'mapped elevation' differ significantly from each other.

SEAC Snow Load Committee Equation:

NOTE: Routt County Snow Mapper

yields value of 77 psf for

snow load at this location

pq, site = max ( $k_{(site)}/100 * A^3$ )

= 22.5/1000 \* 6.5^3 = **61.8 psf** 

 $k_{(site)} = 22.5$  (see map below) A = 6.5 (thousands of feet in elevation)



Use 77 psf

https://asce7hazardtool.online/

	Date 11/6/2023		Sheet		of	
	Job Milner - 2310219		0.1001		0.	
	Subject Wind Loads (Container)					
WIND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD						
	1. Input					
windward Leon	Design Parameters					
Pressure Press	Basic Wind Speed, V =			115	mph (Section 26	i.5, Fig. 1A-2D)
	Exposure Category (B, C, or D) =			С	(Section 26.7)	
Roof	Building Risk Category (I, II, III, IV) =			ш	(Table 1.5-1)	
	Civil finished floor elevation (if unknown input 0) =			6500.00	feet (Sect. 26.9,	Table 26.9-1)
	Eave Height, He =	aval Hr =		9.50	feet	
	Max Building Height of Ridge Height above ground i	evei, ni –		9.50	feet	
	Building Width Perpendicular to Wind, B =			40.00	feet (max bldg d	im)
L L	Building Width Parallel to Wind, L =			8.00	feet	,
	Enclosure Classification =		Encl	osed Buildings	(Section 26.12)	
REFER TO FIGURE 27.3-1	Roof Configuration = Gable	ed, Hipped o	r Monoslop	e Roofs (ø ≤ 7)		
	Angle of Plane of Roof From Horizontal, $\theta$ =			0.50	degrees	
	la buildian an anna a bill sidan an anna 40			N		00.0
<b>A</b>	Is building on or near a fill, ridge, or escarpment?	in LI –		N 40.00	( I UI IN) (Section	1 20.0) 9 Eig 26 9 1)
	Here Diet Unwind to Deiet Where Elevel	и, п – 1 b –		10.00	feet (Section 26.	.u, i=iy. ∠u.ö-1)
x (upwind) x	Horiz, Dist, Upwind to Point where Elevation = H/2,	LI] =		10.00	feet (Section 26.	.o, ⊏ig. ∠0.8-1) 9 Eig. 26.9-4)
	DONZ. DISL. ITOM Crest to Building Site, X =			10.00		.o, i=iy. ∠o.ö-1)
	2D Ridge, 2D Escarpment, or Axisymmetrical Hill =	, ,		E	(R, E, OFH)	
	is the building site upwind or downwind of the crest:			DOWN	(up, down)	
$/ \longrightarrow I$	2 Calculations - Main Wind Force Resisting System					
	Equivalent Allowable Stress Design Wind Speed V	= bas		89.08	mph (IBC 2018	1609 3 1)
	Mean roof beight h =	100		9 50 1	feet	1000.0.1)
	Kz velocity pressure exposure coefficient at bz = 9	5ft =		0.85	Table 26 10-1	(use with az)
	$K_z$ , velocity pressure exposure coefficient at $h_z = 9$ .	5ft =		0.85	Table 26 10-1	(use with gh)
	Kz, velocity pressure exposure coefficient at hn = 9	5ft =		0.85	Table 26 10-1	(use with an)
2 D Ridge or Aviewmentrical Hill	Kzt topographic factor at $hz = 9.5ft =$	on		1.00	Figure 26.8-1	(use with gz)
2-D Ridge of Axisynmetrical Hill	Kzt.topographic factor at hh = 9.5ft =			1.00	Figure 26.8-1	(use with ah)
REFER TO FIGURE 26.8-1	Kzt topographic factor at hp = $9.5$ ft =			1.00	Figure 26.8-1	(use with ap)
	Kd. wind directionality factor =			0.85	Table 26.6-1	( 11)
	Ke, ground elevation factor at			0.79	Table 26.9-1	
	G, gust factor =			0.85	Section 26.11.4	
	-					
	qz, velocity pressure at hz = 9.5ft =			19.33	psf (Eq. 26.10-1	)
	qh, velocity pressure at hh = 9.5ft =			19.33	psf (Eq. 26.10-1	)
	qp, velocity pressure at np = 9.5it =			19.33	psi (Eq. 26.10-1	)
	Walls: P = g(GCpf-GCpi) Eqn. 27.3-1	az	GCp	GCpi	(1.0)P	(0.6)P
	Windward pressure	19.33	0.68		13.1 psf	7.9 psf
		qh	GCp	GCpi	(1.0)P	(0.6)P
	Leeward Pressure	19.33	-0.43	a 1-	-8.2 psf	-4.9 psf
	Sidewall pressure	19.33	-0.60	0.18	-15 psf	-9 psf
	Internal Pressure	19.33		0.18	3.5 psi	2.1 psi
	(1.0)W = (1.0)(Windward + Leeward Pressure) =		13.15 g	sf + 8.22 psf =	21.4 psf	
	(0.6)W =( 0.6)(Windward + Leeward Pressure) =		7.89 p	sf + 4.93 psf =	12.8 psf	
				-	-	
	Parapets: Pp =qp(GCpn) Eqn. 27.3-3	qp	GCpn	-	(1.0)Pp	(0.6)Pp
	Windward parapet pressure	19.33	1.5		29 psf	17.4 psf
	Leeward parapet pressure	19.33	-1.0		-19.3 pst	-11.6 pst
	Windward + Leeward Pressure	19.33	2 50		48.3 nsf	29 nsf
						10 PO.
	Roof Normal to Ridge (θ≥10 degrees)	qh	GCp	GCpi	(1.0)P	(0.6)P
	Windward Pressure case i	19.33	-1.11	0.18	-24.8 psf	-14.9 psf
	case ii	19.33	-0.15	0.18	-6.4 psf	-3.9 psf
	Leeward Pressure	19.33	-0.60	0.18	-15 pst	-9 pst
	Roof All Other Conditions	ah	GCn	GCni	(1.0)P	(0.6)P
	For 0 to $h/2 = 0$ ft to 4.75 ft	19.33	-1.09	0.18	-24.6 psf	-14.8 psf
	>h/2 ft	19.33	-0.60	0.18	-15 psf	-9 psf
		19.33			#VALUE!	#VALUE!
		19.33			#VALUE!	#VALUE!
						·
	Koot Overhangs Section 27.3.3	<b>qh</b>	GCp	GCpi	(1.0)P	(0.6)P
	maximum pressures	19.33	-1.11	0.08	-34.5 psr	-20.7 psf

	Date Job Subject	11/6/2023 Milner, C	3 O		Sheet		of	
WIND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD	Subject	WING LOS	105					
WIND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD ASCE 7-16, Chapters 27 and 30 3. Input - Component and Claddin Tributary Area for Wall Cor Tributary Area for Wall Cor Tributary Area for Parapet Tributary Area for Parapet Tributary Area for Roof Co Tributary Area for Overhan Tributary Area for Overhan				1 = 2 =		10.00 65.00 50.00 10.00 10.00 50.00 50.00	square feet square feet square feet square feet square feet square feet square feet	
GE GE ELEVATION	Kh, velocity Kh, velocity Kzt,topogra Kzt,topogra Kd, wind di Ke, ground G, gust fac	<ul> <li>/ pressure ex</li> <li>aphic factor a</li> <li>aphic factor a</li> <li>aphic factor a</li> <li>rectionality factor factor factor</li> <li>elevation factor factor factor</li> </ul>	posure coefficient at posure coefficient at th h = 9.5ft = actor = ctor at hh = 9.5ft =	t hh = 9.5ft = t hp = 9.5ft =		0.85 0.85 1.00 1.00 0.85 1.00 0.85 24 45	Table 26.10-1 Table 26.10-1 Figure 26.8-1 Figure 26.8-1 Table 26.6-1 Table 26.9-1 Section 26.11.4	(use with qh) (use with qp) (use with qh) (use with qp)
	qp, velocity	pressure at	hp = 9.5ft =			24.45	psf (Eq. 26.10-1	)
	Walls: trib. Are Zone 4 Zone 5 Zone 4 and	ea = 10 sq. f Interior Z End Zone	t. one e	<b>qh</b> 24.45 24.45 24.45	GCp -0.99 -1.26 0.90	GCpi 0.18 0.18 -0.18	(1.0)P -28.6 psf -35.2 psf 26.4 psf	(0.6)P -17.2 psf -21.1 psf 15.8 psf
0.2h	10/alla: 4:1 A.					00-1	(4.0)D	(0.0)D
	Zone 4	a = 65 sq. r Interior Z	t. one	<b>qn</b> 24.45	-0.86	0.18	(1.0)P -25.5 psf	(0.6)P -15.3 psf
	Zone 5	End Zone	9	24.45	-1.00	0.18	-28.9 psf	-17.3 psf
	Zone 4 and	15		24.45	0.77	-0.18	23.2 psf	13.9 psf
0.6h	Parapets: trib.	Area = 10 s	q. ft.	qp	GCp	GCpi	(1.0)P	(0.6)P
0.6h	Case A	Zone 4	Interior Zone	24.45	2.97	0.00	72.6 psf	43.6 psf
		Zone 5	End Zone	24.45	3.78	0.00	92.4 psf	55.5 psf
3	Case B	Zone 4	Interior Zone	24.45	1.89	0.00	46.2 psf	27.7 psf
		Zone 5	End Zone	24.45	2.16	0.00	52.8 psf	31.7 psf
PLAN	Paranets: trib	Area = 50 s	a ft	an	GCn	GCni	(1 0)P	(0.6)P
	Case A	Zone 4	Interior Zone	чр 24.45	2.53	0.00	61.8 psf	37.1 psf
		Zone 5	End Zone	24.45	3.00	0.00	73.4 psf	44 psf
	Case B	Zone 4	Interior Zone	24 45	1 67	0.00	40 8 nsf	24.5 nsf
	Case D	Zone 5	End Zone	24.45	1.83	0.00	44.7 psf	26.8 psf
>	Roofs: trib. Ar	ea = 10 sq. 1 ne (3)	ft.	<b>qh</b> 24 45	GCp -3 20	GCpi 0.18	(1.0)P -82.6 psf	(0.6)P -49.6 psf
	End Zone (	2)		24.45	-2.30	0.18	-60.6 psf	-36.4 psf
θ	Interior Zor	ne (1)		24.45	-1.70	0.18	-46 psf	-27.6 psf
	Interior Zor Positive (A	ie (1') Il Zones)		24.45 24.45	-0.90	0.18 -0.18	-26.4 pst 16 psf	-15.8 pst 9.6 psf
ELEVATION	Roofs: trib. Ar	ea = 100 sq.	ft.	qh	GCp	GCpi	(1.0)P	(0.6)P
ELE TATION	Corner Zor End Zone (	ie (3) 2)		24.45 24.45	-2.14 -1 77	0.18	-56.7 pst -47.7 psf	-34 pst -28.6 psf
REFER TO FIGURE 30.3-2A	Interior Zor	ne (1)		24.45	-1.29	0.18	-35.9 psf	-21.5 psf
	Interior Zor	ne (1') Il Zones)		24.45	-0.90	0.18 -0.18	-26.4 psf	-15.8 psf
	i ositive (A	1 201103/		24.40	0.20	-0.10	io pai	3.0 pai
	Overhangs: tri	b. Area = 10	) sq. ft.	<b>qh</b>	GCpn		(1.0)Pp	(0.6)Pp
	End Zone (	2)		24.45	-3.20		-76.2 psi	-33.7 psf
	Interior Zor	ne (1)		24.45	-1.70		-41.6 psf	-24.9 psf
	Interior Zor	ne (1')		24.45	-1.70 		-41.6 psf 	-24.9 psf 
	Overhander to	h Aroa - 52	) sa ft	ah	60		(1 0) 🕞	(0 E)P-
	Corner Zor	ie (3)	, oy. it.	24.45	-2.34		-57.1 psf	-34.3 psf
	End Zone (	2)		24.45	-1.81		-44.2 psf	-26.5 psf
	Interior Zor Interior Zor	ne (1) ne (1')		24.45 24.45	-1.63 -1.63		-39.9 psf -39.9 psf	-23.9 psf -23.9 psf

a, end zone width = Min. of 10% L and .4H but not < 4% L or 3' =

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Notes: 1. The gust factor of 0.85 is based on a building with a natural frequency of > 1 Hz. For other buildings, the gust factor must be calculated. 2. GCp for walls include a 10% reduction when angle of roof is 10 deg or less. (Figure 30.3-1, Footnote 5)

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WALLACE DESIGN PROGRAM						
Revised 12/27/18, Sheila Butcher Copyright ©						
		Date	11/9/2023	Sheet No.		of
		Job Subject	2310219 (3) M	lilner	· e	
		oubjeet	Celonilo Edua			
2018 IBC (Ch: 16) and ASCE 7-16 (Ch: 11 to 13	3)					
	,					
1. Input						
Spectral Response Acceleration for Short Peri	iods, Ss =		0.300			
Spectral Response Acceleration for 1-second	Periods, S1 =		0.055			
Risk Category =				(IBC Table 1604.5	& ASCE: Ta	ble 1-5-1)
				(AOOE 7 011.20 12	510 20.0-1)	
Basic Structural System		BEARING WA	ALL SYSTEMS	(Table 12.2-1)		
Lateral Force Resisting System	(Redundancy is	either 1.0 or 1.3	te Shear Walls	(Table 12.2-1)	2 2 4)	
ov redundancy in x-dir.=	(Redundancy is	either 1.0 or 1.3)	1.00	(ASCE 7 Section 1 (ASCE 7 Section 1	2.3.4)	
r, =1.0 for Seismic Design Category B and C,	RE: ASCE 7 Section 12.3.4.1 for additional exception	s.		(1002 1 0000011	2.0.1)	
In Structure regular with a pariod $< 5$ and $2$			Vac		7 Contine 12	9 1 2)
Is Structure regular with a period < .0 see	n?		No	(Yes or No. ASCE	7 Section 1	1.6)
Is Structure short period w/ non-rigid diaphrag	m & vertical elements of seismic force-resisting syster	m spaced at 40' o	No	(Yes or No, ASCE	7 Section 11	.6)
Does Structure have a flexible diaphragm?			No	(Yes or No, ASCE	7 Section 11	.6)
(For Wall anchorage requirements per Section	12.11.2.1)					
Span length of flexible diaphragm -x dir. =	· · · ,		50.083	feet (input 0 for r	igid diaphra	igm)
Span length of flexible diaphragm -y-dir. =			14.5	feet		
2 Determine Design Spectral Response Accel	erations and Seismic Design Category, Section 11	6.				
2. Determine Design opeen ar Response Acces	crations and ocisinic Design category, ecction in	.0.				
Response Modification Factor, R =			4	(Table 12.2-1)		
Overstrength Factor, Ωo = (refer to footnote to Deflection Amplification Factor, Cd =	o for .5 reduction for Flexible Diaphragms)		2.5	(Table 12.2-1)		
Defection Antpinication racio, ed -			-	(Table 12.2-1)		
Acceleration for Short Period						
Site Coefficient, Fa =	for Short Boriada, Sma -		1.30	(IBC Table 1613.2.	3(1), ASCE	7 Table 11.4-1)
Acceleration for 1-Second Period			0.390	(IBC Section 1013.	2.3, ASCE /	Seculo 11.4.4)
Site Coefficient, Fv =			1.50	(IBC Table 1613.2.	3(2), ASCE	7 Table 11.4-2)
Site Adjusted Spectral Response Acceleration	for 1-second Periods, Sm1 =		0.083	(IBC Section 1613.	2.3, ASCE 7	Section 11.4.4)
Design Spectral Response Acceleration for Sh	nort Periods. Sds =		0 260	(IBC Section 1613	2.4 and ASC	E 7 Section 11 4 5
Seismic Design Category based on short perio	od =		В	(120 0000011 1010.	2.1 4114 / 10 4	2 / 000001 / 1110
Design Spectral Response Acceleration for 1-	second Periods, Sd1 =		0.055	(IBC Section 1613.	2.4 and ASC	E 7 Section 11.4.5
Seismic Design Category based on 1-second	perioa =		A			
Design Response Spectrum, Ts =			0.212	seconds (Section 1	1.4.6)	
Approximate Fundamental Period, Ta =			0.500	seconds (Section 1	2.8.2.1)	
Fundamental Period, T, shall not exceed Ta *	Cu =		0.850	seconds (Section 1	2.8.2)	
Can the Seismic Design Category be based of	n the short period alone?		NO	(IBC Section 1613.	2.5.1, ASCE	7 Section 11.6)
Seismic Design Category =			В	(Most severe case	except as al	lowed by Sect 11.6
3 Salemic Base Shear for the Lateral Force Pe	sisting System using the Equivalent Lateral Force	Procedure Se	ction 12 8			
5. Seisinic Base Shear for the Laterari of the Re	sisting System using the Equivalent Lateral Force	e riocedule, de	20011 12.0.			
a. Calculation of Seismic Base Shear Coeffic	ient:					
Seismic Importance Factor, I <sub>e</sub> =			1.25	(ASCE 7 Table 1.5	-2)	
$Cs = (Sds/(R / I_e)) =$			0.081	(ASCE Equation 12	2.8-2, Sectio	n 12.8.1.3)
b. Seismic Base Shear, Section 12.8.1:		:	Strength (1.0E)	ASD ( 0.7E)		
V = Cs W =			0.081 W	0.057 W		
a Harizantal Salamia Load Soction 12 4 2 1-	_		Strongth (1 OE)			
For the X-direction: Eh=	-		0.081 W	0.057 W		
For the Y-direction: Eh=			0.081 W	0.057 W		
d Vertical Seismic Load Component Section	n 12 4 2 2·					
Ev = 0.2 Sds D =	11 12.7.2.2.		0.052 D	0.036 D		
For structures in SDC B and for the design of the second sec	foundations using ASD, Ev may be taken as zero. (Se	ection 12.4.2.2)				
e. Find the Design Seismic Shear for the Dia	phragm, Section 12.10.1.1:	9	Strength (1.0F)	ASD ( 0.7E)		
Force shall not be less than 0.2*Sds*le*wpx =	· · · · · · · · · · · · · · · · · · ·	,	0.065 W	0.046 W		
hut need not exceed 0 $4$ *le*Sde wox =			U.130 W	0.091 44		
For a one story building, Fpx =			0.081 W	0.057 W		
f For collector elements in Seismic Design (	Categories C through F. Section 12.10.2					
$Emh = \Omega \circ V =$			0.203 W	0.142 W		
Notos						
1. A building that is low rise (one or two story) bu	ilding with a short period is assumed for calculation of	f Seismic Respor	nse Coefficient,	Cs.		

A building that is low rise (one or two story) building with a short period is assumed for calculation of Seismic Response Coefficient, Cs.
 The values for design spectral response acceleration assume a regular structure of 5 stories or less with a period, T < 0.5 seconds</li>
 The values for design forces for the diaphragm assume no offsets or changes in the stiffness of the vertical components
 Section 1613.1 of 2018 IBC excludes the detailing requirements of Chapter 14 of ASCE 7.
 Per Section 1613.2.2 and 11.4.3, if site investigations performed per ASCE 7 Chpt 20 reveal rock conditions consistent with Site Class B, but site-specific velocity measurements are not made, Fa and Fv shall = 1.0.

WALLACE DESIGN PROGRAM Revised 12/27/18, Sheila Butcher Copyright © 8/24/2023 Sheet No. Date of Job Subject SEISMIC LOAD SUMMARY 2018 IBC (Ch: 16) and ASCE 7-16 (Ch: 11 to 13) 4. Minimum Continuous Load Path, Interconnection and Connection to supports, Section 12.1.3 and 12.1.4: a. Continuous Load Path and Interconnections, Section 12.1.3: Strength (1.0E) ASD ( 0.7E)  $F_p$ = 0.133  $S_{ds} W_p$  or .05  $W_p$  min. = 0.050 Wp 0.035 Wp (Section 12.1.3) b. Connection to Supports, Section 12.1.4 : Fp= .05 \* dead + live reaction = 0.050 Rd+I 0.035 Rd+I (Section 12.1.4) 5. Structural Walls and Anchorage, Section 12.11 Strength (1.0E) ASD (0.7E) a. Minimum Out-of-Plane Forces on Structural Walls, Section 12.11.1: Fp= 0.40 le Sds wp or .10 wp min = 0.130 Wp 0.091 Wp (Section 12.11.1) b. Minimum anchorage connection of structural walls to supporting construction, Section 12.11.2.1 and 12.11.2: Per 12.11.2.2, the strength For loading in the x-direction: Strength (1.0E) ASD ( 0.7E) design force for steel elements with the exception k<sub>a</sub>= 1.0+L<sub>f</sub>/100 or max 2.0 = 1.50 of anchor bolts and Fp= 0.4 S<sub>ds</sub> k<sub>a</sub> I<sub>e</sub> W<sub>p</sub> or .2 k<sub>a</sub> I<sub>e</sub> Wp min. = onnections at Flexible Diaphragms: 0.375 Wp 0.263 Wp reinforcing steel shall be  $F_p * 1.4$  for steel elements per 12.11.2.2.2 0.368 Wp 0.525 Wp increased by 1.4 times. k\_= 1.0 Fp= 0.4 S<sub>ds</sub>  $k_a I_e W_p$  or .2  $k_a I_e Wp$  min. = 0.175 Wp For Connections not at Flexible Diaphragms: 0.250 Wp Fp \* 1.4 for steel elements per 12.11.2.2.2 0.350 Wp 0.245 Wp For loading in the y-direction:  $k_{-}= 1.0 + L/100$  or max 2.0 = 1 15 Fp= 0.4 S<sub>ds</sub> k<sub>a</sub> l<sub>e</sub> W<sub>p</sub> or .2 k<sub>a</sub> l<sub>e</sub> Wp min. = For Connections at Flexible Diaphragms: 0.286 Wp 0.200 Wp Fp \* 1.4 for steel elements per 12.11.2.2.2 0.401 Wp 0.281 Wp k<sub>a</sub>= 1.0 Fp= 0.4  $S_{ds} k_a I_e W_p$  or .2  $k_a I_e Wp$  min. = For Connections not at Flexible Diaphragms: 0.250 Wp 0.175 Wp Fp \* 1.4 for steel elements per 12.11.2.2.2 0.350 Wp 0.245 Wp The minimum wall anchorage load for concrete or masonry walls is 0.2\* the wall weight or 5 psf per 1.4.4. 6. Horizontal Seismic Design Force on Nonstructural Architectural Components, Section 13.3: For Ip =1.0 For Ip=1.5 Fp max = 1.6 Sds lp Wp= 0.416 Wp 0.624 Wp (Equation 13.3-2) Fp min = 0.3 Sds lp Wp= 0.078 Wp 0.117 Wp (Equation 13.3-3) The Seismic Design Force is based on Equation 13.3-1, with the minimum and maximum limits noted above. Fp= 0.4 ap Sds Wp (1 + 2 z/h)/(Rp/lp) Seismic Design Force Summary on Architectural Components, Section 13.5: Rp= lp= z/h= trength (1.0E) ASD (0.7E) an= 1. Cantilevered (Unbraced) Parapets and Chimneys 2 50 2 50 1 00 1 00 0.312 Wp 0.218 Wp (Table 13.5-1) 2. Braced Interior Non-masonry walls and partitions Fp at floor= 0.078 Wp 1.00 2.50 1.00 0.00 0.055 Wp (Table 13.5-1) Fp at roof= 1.00 2.50 1.00 1.00 0.125 Wp 0.087 Wp (Table 13.5-1) Fp average at roof and floor 0.101 Wp 0.071 Wp 3. Braced Interior Unreinforced masonry walls and partitions Fp at floor= 1.00 1.50 1.00 0.00 0.078 Wp 0.055 Wp (Table 13.5-1) 0.146 Wp (Table 13.5-1) Fp at roof= 1.00 1.50 1.00 1.00 0.208 Wp Fp average at roof and floor: 0.143 Wp 0.100 Wp 4. Cantilevered (Unbraced) Interior Nonstructural walls 2 50 2 50 1.00 0.00 0.104 Wp 0.073 Wp (Table 13.5-1) 0.125 Wp 0.087 Wp (Table 13.5-1) 5. Braced Parapets and Chimneys 1.00 2.50 1.00 1.00 6. Exterior Nonstructural Wall Elements Fp at floor= 1.00 2.50 1.00 0.00 0.078 Wp 0.055 Wp (Table 13.5-1) Fp at roof= 1.00 2 50 1.00 1.00 0.125 Wp 0.087 Wp (Table 13.5-1) Fp average at roof and floor: 0.101 Wp 0.071 Wp For the Body of the Wall Panel Connection: Fp at floor= 1.00 2.50 1.00 0.00 0.078 Wp 0.055 Wp (Table 13.5-1) Fp at roof= 1.00 2.50 1.00 1.00 0.125 Wp 0.087 Wp (Table 13.5-1) For the fasteners of the connecting system: Fp at floor= 0.00 0.130 Wp 0.091 Wp (Table 13.5-1) 1.25 1.00 1.00 Fp at roof= 1.25 1.00 1.00 1.00 0.390 Wp 0.273 Wp (Table 13.5-1) 2.50 7. Appendages and Ornamentation 2.50 1.00 1.00 0.312 Wp 0.218 Wp (Table 13.5-1)

#### Notes:

1. Refer to Section 13.4.2 for additional requirements for anchors in concrete and masonry.

2. Section 1613.1 of 2018 IBC excludes the detailing requirements of Chapter 14 of ASCE 7.

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8/24/2023 Sheet No. of Date Job Subject

SEISMIC LOAD SUMMARY 2018 IBC (Ch: 16) and ASCE 7-16 (Ch: 11 to 13)

# Table 11.4-1 and IBC 1613.2.3(1) Site Coefficient, Fa

		Site Co	efficient, Fa				
Site	Mapped Sp	ectral Respo	nse Accelerat	ion at Short F	Periods (Ss)		Distance
Class	Ss<=0.25	0.5	0.75	1	1.25	Ss>=1.5	Value
А	0.80	0.80	0.80	0.80	0.80	0.80	0.80
В	0.90	0.90	0.90	0.90	0.90	0.90	0.90
С	1.30	1.30	1.20	1.20	1.20	1.20	1.30
D	1.60	1.40	1.20	1.10	1.00	1.00	1.56
E	2.40	1.70	1.30	1.20	1.20	1.20	2.26
-							

Minimum of 1.2 per Section 11.4.4 considered. Exceptions per Section 11.4.8 included.

# Table 11.4-2 and IBC 1613.2.3(2) Site Coefficient, Fv

		Site Co	efficient, Fv				
Site	Mapped Spec	ctral Respon	se Acceleratio	n at 1 Secon	d Period (S1	)	Distance
Class	S1<=0.1	0.2	0.3	0.4	0.5	S1>=0.6	Value
Α	0.80	0.80	0.80	0.80	0.80	0.80	0.8
В	0.80	0.80	0.80	0.80	0.80	0.80	0.8
С	1.50	1.50	1.50	1.50	1.50	1.40	1.5
D	2.40	2.20	2.00	1.90	1.80	1.70	2.4
Е	4.20	3.30	2.80	2.40	2.20	2.00	4.2
F							

# IBC Table 1613.2.5(1) and 11.6-1

IBC	able 1613	3.2.5(1) and	11.6-1	
ismic Design Catego	ory based or	Short Peri	od Response Acceleratio	
Value of	Occ	upancy Cate	gory	Design
Sds	l or ll	III	IV	Category Category
Sds <= 0.167	Α	А	A	A B
0.167 <= Sds < 0.33	В	В	С	В
0.33 <= Sds < 0.5	С	С	D	С
0.5 <= Sds	D	D	D	D
S1 >= 0.75	E	E	F	E

Table 1613.2.5(2) and 11.6-2

Ta	able 1613.2	2.5(2) and 1	1.6-2			
Seismic Design Cate	gory Based	on 1-Seco	nd Period Respo	nse Acceleration		
Value of	Occ	upancy Cate	gory		Design	
Sd1	l or ll	111	IV		Category	Category
Sd1 <= 0.067	А	А	A		А	Α
0.067 <= Sd1 < 0.133	В	В	С		В	
0.133 <= Sd1 < 0.2	С	С	D		С	
0.2 <= Sd1	D	D	D		D	
S1 >= 0.75	E	E	F		E	



#### WALLACE DESIGN PROGRAM

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	Date Project Subject	11/6/2023 Milner Snow Load	Sheet No. of
FLAT ROOF SNOW DRIFT - Joists Perpendicular to Wall			
	1. Input		
Wb1     Wb2       Wd     Wd       T.O.W.     hc       hr     hd	Dead Load = Roof Live Loa Pg, Ground S Drift for parap Risk Categoy Ce, Exposure Ct, Thermal F Use Pg minim	15 psf 20 psf 77 psf U (P), (PR) or (U) III Table 1.5-1 1.00 Table 7.3-1 1.00 Table 7.3-2 n (Y or N)	
J.B.E. td	T.O.W., Top of J.B.E., Joist E td, Thickness Wb1, length of Wb2, length of S, Joist Spaci L, Joist Span	of Upper Roof Elevation = Bearing Elevation = of Joist, Deck, and Insulation = If upper roof = If lower roof = ng = =	9.50 feet 0.00 feet 0.01 inches 8.00 feet 1000.00 feet 1.00 feet 12.00 feet
coniguration	2. Balanced Snov	v Load Check	
Assume very long lower "roof" is grade next to WWTP, where	Is, Importance Pf = 0.7 Ce C Pm = Is Pg = Rain on snow Pmin =	e Factor = t ls Pg = surcharge =	<ul> <li>1.10 Table 1.5-2</li> <li>59.29 psf (7.3-1)</li> <li>22.00 psf (7.3.4)</li> <li>0.00 psf (7.10)</li> <li>59.29 psf</li> </ul>
	3. Drifted Snow L	oad Check	
Use constant 228 psf snow load where drifts may occur	Pf = 0.7 Ce C D = 0.13 Pg + hb = Pf/D = Wb = hd = 0.75[0.4]	t Is Pg = • 14.0 ≤ 30 pcf = • 3 Wb2^1/3 (Pa+10)^1/4-1.5] Is/	59.29 psf 24.01 pcf 2.47 ft 1000.00 ft 2.13 ft
Wd	hd + hb = $hr =$ $hc = hr - hb =$ $Wd = 4 hd or$	4 [ hd^2/hc ] ≤ 8 hc =	11.60 ft 9.50 ft 7.03 ft 47.39 ft
Pmax Pd Pf	<ul> <li>Pmax = D (no</li> <li>Pd =D hd ≤ D</li> <li>4. Uniform Load \$</li> </ul>	hc = Bummary	168.78 psf
	Drifted Snow Lo R left = R right =	ad Snov 1283.0 1197.5	<ul> <li>Total</li> <li>1373.0 lbs</li> <li>1287.5 lbs</li> </ul>
	M max = w base = w drift =	3721.8 59.3 168.8	3         3991.8         ft-lbs           3         74.3         plf           3         183.8         plf
Snow Drift	w equiv =	213.8	3 228.8 plf *

Load Without Drift

	Snow	Total
w (Pmin = 59.29 psf	59.3	<b>74.3</b> plf

\* indicates controlling load (drifted vs. undrifted)

02 WALLS



123 north martin luther king jr. boulevard tulsa, oklahoma 74103 918.584.5858 · 800.364.5858 wallace.design

		Date:	11/9/2023	
		Job:	2310219(3) - Milne	r WWTP
		Subject:	Tank layout	
Inside dimensi 12.00 ft	on			
23.42 ft		Total length =	56.6667 ft	
18.00 ft				



<u>Plan</u>

Width

Length Tank Influent EQ Aeration Tank

Backfill height at perimeter of wall:	1	ft (above tar	nk slab elevation)
Height: of sludge above slab	7.5	ft (above tar	nk slab elevation)
Soil parameters:			
Allowable Soil Bearing Pressure:	3000	psf	See Geotechnical Report from NWCC, Inc
Modulus of Subgrade Reaction (Ks):	150	pci (assume	d) Job No 22-12814 Dated January 19, 2023
Coefficient of Friction for Sliding:	0.4		
Passive Lateral Pressure;	250	pcf	
Active Lateral Pressure:	60	pcf *	(on-site sands)
Equivalent Fluid Density	250	pcf	* 85-90 pcf for below water table. Ground-
Frost depth:	48	in	water found at 10 ft below grade

	Date: 11/0/2022	
	Job: 2310219(3) - 1	Vilner WWTP
	Subject: Tank lavout	
		Reference
nterior Tank Wall		ACI 350-06 ACI 318-
ypical wall		
Worst case condition - overflow one side of wall, other side of wal	l is empty	
assume overflow condition is completely full to bottom of lid (cons	servative)	
Lid Fluid Pressu digested slu p <sub>fluid</sub> =	re on wall from dge, anaerobic = 70 pcf	R 2.2
H = 7.50 ft (Height at low end of sloped Sludge Tank) Base H * p <sub>fuid</sub> =	Wmax Force at base of wall 525 plf	
vlinimum wall thickness = 12"	<- Controls	14.6.2
linimum thickness = 6" or $l/30$		14.6.1
id span, $\ell$ $\ell$ = 12.00 ft $\ell/30$ = 4.8	3 in	
Assume pinned condition top and fixed bottom. <u>(No backfill before l</u> Mmax = = 1753 lb ft / ft (triangular l Vmax = W = 1546 lbs/ft base, pinne (See RISA outp	i <b>a is poured)</b> oad, fixed id top) out)	
Modification factor, Sd		9.2.6
Modification factor, Sd $d = \phi Fy/(\gamma fs)$ $\phi = 0.9$ (modification factor for tension-co	ntrolled section)	9.2.6 B9.2.6
Modification factor, Sd $d = \phi Fy/(\gamma fs)$ $\phi = 0.9$ (modification factor for tension-co Fy = 60.000 psi (Steel yield strength)	ontrolled section)	9.2.6 R9.2.6
Modification factor, Sd         id = $\phi$ Fy/( $\gamma$ fs) $\phi =$ 0.9         Fy =       60,000 $\gamma =$ U factor         (Ultimate Load Factor (LRFD))	ontrolled section)	9.2.6 R9.2.6
Modification factor, Sd         Sd = $\phi$ Fy/( $\gamma$ fs) $\phi$ =       0.9         Fy =       60,000 $\gamma$ =       U factor         (Ultimate Load Factor (LRFD))         fs =       20,000         psi       (Steel direct hoop and tensile street)	ontrolled section) ngth)	9.2.6 R9.2.6
Modification factor, Sd         Sd = $\phi$ Fy/( $\gamma$ fs) $\phi$ =       0.9         Fy =       60,000 $\gamma$ =       U factor         fs =       20,000         psi       (Steel yield strength)         fs =       20,000         psi       (Steel direct hoop and tensile strength)	ontrolled section) ngth)	9.2.6 R9.2.6
Modification factor, Sd $d = \frac{\phi Fy/(\gamma fs)}{\phi = 0.9}$ (modification factor for tension-co Fy = 60,000 psi (Steel yield strength) $\gamma = U \text{ factor}$ (Ultimate Load Factor (LRFD)) fs = 20,000 psi (Steel direct hoop and tensile stre $fs_{max} = \frac{320}{P_{1}(x^{2} + d/2) + d/2} = 0$	ontrolled section) ngth)	9.2.6 R9.2.6 10.6.4.1
Modification factor, SdSd = $\phi$ Fy/( $\gamma$ fs) $\phi$ =0.9Fy =60,000 psiFy =60,000 psi $\gamma$ = U factor(Ultimate Load Factor (LRFD))fs =20,000 psifsmax =320B(s^2 + 4(2+db/2)^2)^0.5For "Nor	ontrolled section) ngth) mal <sup>"</sup> environment exposure:	9.2.6 R9.2.6 10.6.4.1
Modification factor, SdSd = $\phi$ Fy/( $\gamma$ fs) $\phi =$ 0.9Fy =60,000psi(Steel yield strength) $\gamma =$ U factorfs =20,000psi(Steel direct hoop and tensile strefsmax =320B(s^2 + 4(2+db/2)^2)^0.5B =1.35	ontrolled section) ngth) mal" environment exposure: lied to concrete surface, per in Williams Dura-Plate 6000	9.2.6 R9.2.6 10.6.4.1
Modification factor, SdSd = $\phi$ Fy/( $\gamma$ fs) $\phi$ =0.9Fy =60,000psi(Steel yield strength) $\gamma$ =U factorfs =20,000psi(Steel direct hoop and tensile stre)fs =320B(s^2 + 4(2+db/2)^2)^0.5B =1.35db =0.75in(#6 bar)	ontrolled section) ngth) mal" environment exposure: lied to concrete surface, per rin Williams Dura-Plate 6000. ore environment is NORMAL	9.2.6 R9.2.6 10.6.4.1
	ontrolled section) ngth) mal" environment exposure: lied to concrete surface, per in Williams Dura-Plate 6000. ore environment is NORMAL	9.2.6 R9.2.6 10.6.4.1
Modification factor, SdSd = $\phi$ Fy/( $\gamma$ fs) $\phi$ =0.9Fy =60,000Fy =60,000psi(Steel yield strength) $\gamma$ = U factor(Ultimate Load Factor (LRFD))fs =20,000psi(Steel direct hoop and tensile streetfsmax =320B(s^2 + 4(2+db/2)^2)^0.5B =1.35Aquaworks, Sherwdb =0.75s =12in spacing	ontrolled section) ngth) mal" environment exposure: lied to concrete surface, per rin Williams Dura-Plate 6000. ore environment is NORMAL	9.2.6 R9.2.6 10.6.4.1
$\begin{array}{l} \label{eq:modification factor, Sd} \hline \\ \hline Sd = & \phi Fy/(\gamma fs) \\ \phi = & 0.9 \\ Fy = & 60,000 \\ Fy = & 60,000$	ontrolled section) ngth) mal" environment exposure: lied to concrete surface, per rin Williams Dura-Plate 6000. ore environment is NORMAL	9.2.6 R9.2.6 10.6.4.1 10.6.4
Modification factor, Sd $Sd = \phi Fy/(\gamma fs)$ $\phi = 0.9$ $Fy = 60,000$ psi $Fy = 60,000$ psi $\gamma = U$ factor $(Ultimate Load Factor (LRFD))$ $fs = 20,000$ psi $fs_{max} = \frac{320}{B (s^2 + 4(2+db/2)^2)^{0.5}}$ $For "NorCoating to be appB = 1.35db = 0.75 ins = 12 in spacingfs_{max} = 20000 psi(for flexural stress)20000 psifor direct and hoop tensile stress$	ontrolled section) ngth) mal" environment exposure: lied to concrete surface, per rin Williams Dura-Plate 6000. ore environment is NORMAL in normal exposure)	9.2.6 R9.2.6 10.6.4.1 10.6.4 9.2.6.2
Modification factor, SdSd = $\phi$ Fy/( $\gamma$ fs) $\phi$ =0.9 $Fy$ =60,000 $Fy$ =60,000 $Fy$ =60,000 $Fy$ =60,000 $fs$ =20,000 $g$ =20,000 $fs$ =20,000 $g$ =320 $g$ = $fs$ = $fs_{max}$ =320 $g$ = $fs$ = $fs_{max}$ = $fs$ = $g$ = $fs$ = <tr< td=""><td>ontrolled section) ngth) mal" environment exposure: lied to concrete surface, per rin Williams Dura-Plate 6000. ore environment is NORMAL in normal exposure)</td><td>9.2.6 R9.2.6 10.6.4.1 10.6.4 9.2.6.2</td></tr<>	ontrolled section) ngth) mal" environment exposure: lied to concrete surface, per rin Williams Dura-Plate 6000. ore environment is NORMAL in normal exposure)	9.2.6 R9.2.6 10.6.4.1 10.6.4 9.2.6.2
Modification factor, SdSd = $\phi$ Fy/( $\gamma$ fs) $\phi$ =0.9Fy =60,000Fy =60,000psi(Steel yield strength) $\gamma$ = U factor(Ultimate Load Factor (LRFD))fs =20,000psi(Steel direct hoop and tensile streed)fsmax =320B (s^2 + 4(2+db/2)^2)^0.5For "NorCoating to be appB =1.35db =0.75in spacing(for flexural stress)20000 psi(for direct and hoop tensile stress)20000 psi(for direct and hoop tensile stress)U factorSdU factorSd </td <td>ontrolled section) ngth) mal" environment exposure: lied to concrete surface, per in Williams Dura-Plate 6000. ore environment is NORMAL in normal exposure) rt indicates lateral pressure</td> <td>9.2.6 R9.2.6 10.6.4.1 10.6.4 9.2.6.2</td>	ontrolled section) ngth) mal" environment exposure: lied to concrete surface, per in Williams Dura-Plate 6000. ore environment is NORMAL in normal exposure) rt indicates lateral pressure	9.2.6 R9.2.6 10.6.4.1 10.6.4 9.2.6.2
Modification factor, SdSd = $\phi$ Fy/( $\gamma$ fs) $\phi$ =0.9Fy =60,000Fy =60,000psi(Steel yield strength) $\gamma$ = U factor(Ultimate Load Factor (LRFD))fs =20,000psi(Steel direct hoop and tensile streetfsmax =320B (s^2 + 4(2+db/2)^2)^0.5For "NorCoating to be appB =1.35db =0.75in spacing(#6 bar)fsmax =20000 psifsmax =20000 psifor direct and hoop tensile stress20000 psi(for direct and hoop tensile stress)20000 psi(for direct and hoop tensile stress)20000 psi1.21.22.252.70exceeds 70 psf below the	ontrolled section) ngth) mal" environment exposure: lied to concrete surface, per in Williams Dura-Plate 6000. ore environment is NORMAL in normal exposure) rt indicates lateral pressure water table	9.2.6 R9.2.6 10.6.4.1 10.6.4 9.2.6.2
Modification factor, SdSd = $\phi$ Fy/( $\gamma$ fs) $\phi$ =0.9Fy =60,000Fy =60,000psi(Steel yield strength) $\gamma$ = U factor(Ultimate Load Factor (LRFD))fs =20,000psi(Steel direct hoop and tensile streetfsmax =320B (s^2 + 4(2+db/2)^2)^0.5For "NorCoating to be appB =1.35db =0.75s =12in spacingfsmax =20000 psifor flexural stress)20000 psi(for direct and hoop tensile stressSd TableU factorSd1.61.692.701.41.022.701.61.61.751.61.61.751.70	ontrolled section) ngth) mal" environment exposure: lied to concrete surface, per rin Williams Dura-Plate 6000. ore environment is NORMAL in normal exposure) rt indicates lateral pressure water table ater at 10 ft below grade	9.2.6 R9.2.6 10.6.4.1 10.6.4 9.2.6.2
Modification factor, SdSd = $\phi$ Fy/( $\gamma$ fs) $\phi$ =0.9Fy =60,000Fy =60,000psi(Steel yield strength) $\gamma$ = U factor(Ultimate Load Factor (LRFD))fs =20,000psi(Steel direct hoop and tensile streed)fsmax =320B (s^2 + 4(2+db/2)^2)^0.5For "NorCoating to be appB =1.35Aquaworks, Sherwdb =0.75in spacingfsmax =20000 psi(for flexural stress)20000 psi(for direct and hoop tensile stress)20000 psi1.22.252.701.61.692.704.41.932.704.44.7224.7224.7234.7234.7234.723	ontrolled section) ngth) mal" environment exposure: lied to concrete surface, per rin Williams Dura-Plate 6000. ore environment is NORMAL in normal exposure) t indicates lateral pressure water table ater at 10 ft below grade essure is not considered latiae walk	9.2.6 R9.2.6 10.6.4.1 10.6.4 9.2.6.2

	Date: 11/9/2023		
	Job: 2310219(3) -	Milner WWT	P
	Subject Wall design		
		Referer	nce
Wall design		ACI 350-06	ACI 318-14
Wall thickness = 12 in	As = 0.44 in^2 (#6 bar)		
Clear cover = 2.0 in	Fv = 60.000 psi at 12" spacing	7.7.1	
Bar diameter = 0.75 in	f'c = 4500 psi	Table 4.2.2	
d = 9.63 in	h = 12  in	10010 11212	
Check Wall Elexure Mn > Mu			
$0 \text{ min} = 0.00180 \text{ cm}^{-2} \text{ See the min}$	n calce below (For Slabs rhomin is T&S Steel)		Table 7 6 1 1
$\rho = \Delta s / (h^*d)$ 0.00100 < See 110 million	han a min, see below		14012 7.0.1.1
See Table 21.2.2. For Strain Boundaries when (at) is comp			
See Table 21.2.2 For Strain Boundaries when (ct) is comp	ession controlled, transition or tension controlled		
No. Compression Controller	l lf at a atu		Th. 01 0 0
			1 DI. 21.2.2
No Transition Zone If $\varepsilon_{ty} < 0$	Et < 0.005		
Yes Tension Controlled Since	$e(\epsilon t) = or > 0.005$		
			21.2.1
** Solve for phiMn Based on Whitney Stress Bl	ock		
a = stress block depth = As*Fy / (0.85*f'c*beff)	= 0.5752 in.		Fig R21.2.2a
c = depth to neutral axis $= a / \beta =$	0.6972 in.		
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12	= 20542 ftlb.		
φ=	0.9		21.2
φMn = Ultimate Moment Cap =	18488.1 ftlb.		
check against ρ min =	0.0033 <b>&lt;</b>		
check against ρ max =	0.0150 <== determine rho max eqn		
If below ρ min, multiply As x 1.33 =	0.5852 in^2		9.6.1.3
if below ρ min, multiply phiMn x 0.75 =	13866.07 lb-ft		
ir	$\phi$ Mn x 0.75 = 1 / 1.33 x $\phi$ Mn = reduction if $\phi < \phi$ min)		9613
	note: code says o min need not be satisfied if		0.0.1.0
(· •	s > 1.33 * As Required so if a min is not met can		
	manus advect concepts by 1/4 or multiply (0.75)		
s	10400 1 th A		
	18488.1 ID-JT		
Mu = Mmax (U) Sd= 4/33.1 lb ft <	fMn - Wall Flexure OK Utilization = 0.26		
Ν	lote:		
4	500 psi is minimum concrete strength for concrete	Table 4.3.1	
W	vith low permeability (concrete is coated		
i	s coated as directed by Aquaworks)		
Check $\rho$ balanced and $\epsilon$ t=.005 limits:			
β= Max of ((0.85-0.05*(f'c-4000)/1000,0.65)) =	0.8250	10.2.7.3	
ety = tension yield strain = fy / Es	0.0021 <== Es = 29,000,000 psi	R10.3.2	
ec = balanced concrete strain =	0.0030	R10.3.2	
$\epsilon t = actual tension strain = ((d-c) / c)*strain sc$	0.0384	R10.3.3	
0 temp = temp and shrink steel = 0.0018 <	== if Ev<60.000 psi 002 -or- 0018 * 60.000psi	Table 7 12 2 1	
$\Delta s(temp) = (0 temp * heff * h) = 0.2502 iv$	ii i y <00,000 psi, .002 oli .0010 - 00,000 psi /	10010 7.12.2.1	
$x_3(temp) = (p temp ben n) = 0.2352 n$	1 2		
	0 0211 halamand sharl	D10 2 2	
p bai = (0.05 ° C p/Fy) (0.003/(0.003+Ety) 0.75(a bal)=		R10.3.3	
(0.75)(p  bal)=	0.0233	R10.3.5	005
$pt = (.85^{+}p^{+}TC / Fy)^{+}(0.003/(0.008))$	0.019/ <== Max reint. Ratio, whe	en steel strai	in >= .005
As (max) = (ρt * bett * d) =	2.2780 in^2	R10.3.5	
ρ min (b) = 200/fy=	0.0033		9.6.1.2
ρ min (a) = 3*(f'c)^0.5/Fy=	0.0034	10.5.1	9.6.1.2
$\rho$ min.= greater of $\rho$ min(a) or $\rho$ min(b) =	0.0034 <== Min. reinf. Ratio		
As (min)= (ρ min * beff * d) =	0.3874 in.2 <== unless As > 1.33	*As reqd per	9.6.1.3

	Date:	11/9/2023		
	Job:	2310219(3) -	Milner WWT	Р
	Subject	Wall design		
			Referer	ice
			ACI 350-06	ACI 318-14
4. Check Shear Capacity				
$Vc = 2 * \lambda * (fc)^{.5} * b * d =$ 15496	5 lbs			11.2.1.1
φ = 0.75				21.2
φVc/2 = **For Beam Design (h > 10 in.) 5812	l lbs / ft			11.4.6.1
Check if shear capacity is OK without considering shear reinforcement	_			
(Envorinmental durability factor does not apply to steel shear capacity)			9.2.6	
Vu = Vmax (U) =2473.6 lbs> Vu - Wall Shear OK	Utilization	= 0.43		
U (Fluid Pressure) = 1.6 No Reinforcement Che	ck Needed		9.2.1	
Use 12 in wall w/ (#6 bar) @	12	in spacing		



[	Date:	11/9/2023		
<u>_</u>	lob:	2310219(3) -	Milner WW1	Р
<u> </u>	Subject	Wall design		
			Refere	ice
Tall wall			ACI 350-06	ACI 318-14
* Worst case condition - overflow one side of wall, other side of wall is	s empty			
- assume overflow condition is completely full to bottom of lid (conse	rvative)			
···· · · · · · · · · · · · · · · · · ·	,			
Lid Fluid Pressure	e on wall fi	om	R 2.2	
digested sludg	ze anaero	hic =		
	ncf	bic		
H - 7.75 It				
		<b>c</b> 11		
	-orce at ba	ase of wall		
Base H	H * p <sub>fluid</sub> =	10		
Wmax =	542.5	pir	4460	
Minimum wall thickness = 12"	<- Controls	5	14.6.2	
Minimum thickness = 6" or l/30			14.6.1	
Lid span, $\ell$ $\ell$ = 12.00 ft $\ell/30$ = 4.8 i	n			
Assume pinned condition top and fixed bottom. (No backfill before lid	is poured	1)		
Mmax = 2386  lb ft / ft  (triangular loss)	nd fixed	-		
Vmax = W = 1881 lbs/ft base ninned	ton)			
(See RISA output	top) t)			
Modification factor Sd	c)		926	
Sd = $\Phi Ev/(vfs)$			5.2.0	
$\phi = 0.9$ (modification factor for tension-con-	trolled ser	tion)	RQ 2 6	
$\varphi = \frac{1}{1000} \frac{1}{1000} \frac{1}{1000} \frac{1}{10000} \frac{1}{10000000000000000000000000000000000$	tioneu set		10.2.0	
y = 11 factor (1 RED))				
f = 0 factor (Ontinate Load Factor (Ent D))	rth)			
is – 20,000 psi (steel direct hoop and tensile streng	suij			
fs <sub>max</sub> = 320			10641	
1000000000000000000000000000000000000	environm	ent exposure	10.0.4.1	
D (3 2 + 4(2+0)/2) (2) 0.5    101	to concret	o curfaco por		
B = 125 Agupworks Charwin Mi	illiame Du	ra-Diato 6000		
B = 1.55 Aquaworks, Sherwill W		nt is NORMAL		
ab = 0.75  In  (# 0  bar) Therefore e	environme		•	
			10.5.5	
Ismax = 20000 psi (for flexural stress)			10.6.4	
20000 psi (for direct and hoop tensile stress in	normal e	xposure)	9.2.6.2	
U factor Sd U * Sd				
1.2 2.25 2.70				
1.6 1.69 2.70				
1.4 1.93 2.70				
Mu = Mmax (U) Sd= 6442 lb ft				

		Date:	11/9/2023		
		Job:	2310219(3) - I	Milner WWT	Р
		Subject	Wall design		
				Referer	ice
Wall design				ACI 350-06	ACI 318-14
Wall thickness = 12 in	As = 0.44	in^2	(#6 bar)		
Clear cover = 2.0 in	Fy = 60,000	psi	at 12" spacing	7.7.1	
Bar diameter = 0.75 in	f'c = 4500	psi		Table 4.2.2	
d = 9.63 in	b = 12	lin			
		4			
Check Wall Elexure Mn > Mu					
$o \min = 0.0033$	3 <== See o min calcs below (F	or Slabs omin	is T&S Steel)		
$p = As / (b^*d)$ 0.0038	$1 \le 1 \le 1$ fois less than the min set	e helow			
			ture II e. el		
See Table 21.2.2 For Strain Boundaries when (et) is co	ompression controlled, transition	or tension con	trolled		
No. Compression Contro	الممالة مفح مضر				Th. 01.0.0
					1 DI. 21.2.2
No $\Gamma$ I ransition Zone If $\epsilon_{t}$	$r_{\rm c} < c_{\rm t} < 0.005$				
Yes Tension Controlled S	Since ( $\epsilon$ t) = or > 0.005				
					21.2.1
** Solve for phiMn Based on Whitney Stress	Block				
a = stress block depth = As*Fy / (0.85*f'c*b	eff) = 0.5752	in.			Fig R21.2.2a
c = depth to neutral axis = a / $\beta$ =	0.6972	in.			
Mn = Nominal Moment Cap = As*Fy*(d-a/2)	/12 = 20542	ftlb.			
φ=	0.9				
φMn = Ultimate Moment Cap =	18488.1	ftlb.			
check against ρ min =	0.0033	>	φ min		
check against ρ max =	0.0150	<== determin	e rho max eqn		
If below $\rho$ min, multiply As x 1.33 =	0.5852	in^2			9613
if below o min multiply phiMn x 0 75 =	13866.07	lh.ft			0.0.1.0
	13000.07				0.04.0
	in. phiMn x $0.75 = 1/1.33 \text{ x p}$	nivin = reduct	ion if rno < rno min	)	9.0.1.3
	(note: code says rho min need	l not be satisfi	ed if		
	As > 1.33 * As Required, so if	rho min is not	met, can		
	simply reduce capacity by 1/4	or multiply x 0	.75)		
Mu = Mmax (U) Sd= 6442 lb ft	< fMn - Wall Flexure OK	Utilization =	0.46		
	Note:				
	4500 psi is minimum conc	rete strength	n for concrete	Table 4.3.1	
	with low permeability (co	ncrete is coa	ted		
	is coated as directed by A	quaworks)			
Check $\rho$ balanced and et=.005 limits:					
$\beta$ = Max of ((0.85-0.05*(f'c-4000)/1000.0.65	5)) = 0.8250	1		10.2.7.3	
ety = tension yield strain = fy / Fs	0.0021	<== Fs = 2	9.000.000 psi	R10.3.2	
ec = halanced concrete strain =	0.0030		s)000)000 po.	R10 3 2	
$ct = actual tension strain = ((d_c) / c)*strain$	0.0000			P10 3 3	
	30 0.0384			N10.5.5	
a tamp - tamp and chrink staal - 0.001	9 < if EuceCO 000 mail 00	22 ar 001	0 * CO 000mai	T	
p temp = temp and similar steer = 0.001	0 <== 11 FY<00,000 psi, .00	JZ -01001	a ' 60,000psi /	Table 7.12.2.1	
$As(temp) = (p temp \cdot ben \cdot n) = 0.259$	2 In^2				
$\rho$ bal = (0.85*f'c* $\beta$ /Fy)*(0.003/(0.003+ety)	0.0311	<== baland	ced steel ratio	R10.3.3	
0.75(ρ bal)=	0.0233			R10.3.5	
ρt = (.85*β*f'c / Fy)*(0.003/(0.008))	0.0197	<== Max r	einf. Ratio, whe	en steel strai	n >= .005
As (max) = (pt * beff * d) =	2.2780	in^2		R10.3.5	
ρ min (b) = 200/fy=	0.0033				9.6.1.2
ρ min (a) = 3*(fˈc)^0.5/Fy=	0.0034			10.5.1	9.6.1.2
$\rho$ min.= greater of $\rho$ min(a) or $\rho$ min(b) =	0.0034	<== Min. r	einf. Ratio		
As (min)= (o min * beff * d) =	0.3874	in.2 <== u	nless As > 1.33	*As read per	9.6.1.3
· · · · · · · /					



	Jop:	2310219(3) - Milr	ier WWTP
	Subject	Wall design	
	1/0/1900	1/0/1900	
			Reference
		AC	I 350-06 ACI 318-14
4. Check Shear Capacity			
**neglects Vs (shear reinforcement)			
$Vc = 2 * \lambda * (fc)^{.5} * b * d =$ 15496	5 lbs		11.2.1.1
φ = 0.75			9.3.2
ψVc/2 = **For Beam Design (h > 10 in.) 5812	1 lbs / ft		11.4.6.1
Check if shear capacity is OK without considering shear reinforcement			
(Envorinmental durability factor does not apply to concrete shear capacity)			9.2.6
Vu = Vmax (U) = 3817.6 lbs > Vu - Wall Shear OK	Utilization =	= 0.66	
U (Fluid Pressure) = 1.6 No Reinforcement Che	ck Needed		9.2.1
Use 12 in wall w/ (#6 bar) @	12	in spacing	

Note: Sludge pressure against wall is greater than backfill pressure against wall Also, groundwater - per Geotech Report, is below threshold that would make the pressure greater **Therefore, the above designs are applicable for interior and exterior walls** 

See below for short wall and tall wall locations



				Da	ite:	11/9/2023	8	
				Jol	b:	2310219(3) -	Milner WWT	Р
				Su	bject	Wall design		
Tank Wall with con	ncentrated load	d from LID BEAN	1 and/c	or CONTAINER				
Assume worst case loc	ation - Wall with t	ank pressure at full	wall hei	ght and supporting o	container			<controls< td=""></controls<>
concentrated load. Als	o, for case when t	ank is empty and so	il is back	filled to the top of t	he wall,			
the lateral soil pressur	e is LESS THAN the	tank sludge pressu	re, so th	e tank sludge pressu	ire control	S		
Find Container Con	ncentrated load	<u>t</u>			L	W	Н	
Large container:	(40 ft long	x 8 ft wide x 9.5	5 ft tall)		40	8	9.5	
Small container:	(10 ft long	x 8 ft wide x 9.5	5 ft tall)	)	10	8	9.5	
		Large Containe	r	Small				
(wet weight)	DL =	43975 lbs		10781 lbs	5		(provided b	y Aquaworks)
	LL =	60 psf						
	SL =	77 psf						
	RL =	20 psf						
	Winddown =	9.6 psf						
	Wind <sub>up</sub> =	-43.0 psf		(Average uplift	of corne	r and end zon	e pressures)	
Assume container	is supported a	t 4 corner and lo	ad is d	istributed equall	y to eacl	h corner*		
Corner Loads (Larg	<u>(e):</u>	At Center Beam, 2 co	ontainer c	orners at same point:		Corner Loads	s (small):	
DL = 10993	3.75 lbs	DL =	13689	lbs		DL =	2695.25	lbs
LL = 4	800 lbs	LL =	6000	lbs		LL =	1200	lbs
SL = 6	160 lbs	SL =	7700	lbs		SL =	1540	lbs
RL = 1	600 lbs	RL =	2000	lbs		RL =	400	lbs
Wind <sub>down</sub> =	768 lbs	Winddown =	960	lbs		Winddown =	192	lbs
Wind <sub>up</sub> = -3	440 lbs	Wind <sub>up</sub> =	-4300	lbs		Wind <sub>up</sub> =	-860	lbs
				* Actually, containe	er is shimn	ned along length	above	
Wind overturning	w =	21.1 psf		perpendicular walls	s, so this ca	alculation is cons	ervative	
Large Container	T = C =	2380 lbs		* Same value for sn	nall contai	ner due to heigh	t and width	
C C						-		
Т	otal Wside =	2380 lbs		(vertical load fro	om wind	overturning	on Container	s)
ASD Load Combina	ations	Large Only		Small Only		At (2) Contai	ners	
1 D + L		15794 lbs		3895		19689	) lbs	
2 D + S		17154 lbs		4235		21389	) lbs	
3 D + 0.7	5(L + S)	19214 lbs		4750		23964	l lbs	
4 D + W		11762 lbs		2887		14649	) lbs	
5 D + 0.7	5(L + S + W)	19790 lbs		4894		24684	l lbs	<controls< td=""></controls<>
6 0.6D - \	N	3156 lbs		757		3913	lbs	No Net Uplift
				1 cc	ontainer			
Maximum	Vertical Conce	ntrated Load fro	m lid 8	Containers=	19790	(Beam 1)		
Maximum	Vertical Conce	ntrated Load fro	m lid 8	Containers=	24684	(Beam 2)		
		(This	is point	load on beam) - see	beam des	ign for reaction	to wall	
		Rea	ction a	t Wall =	49441	lbs (max at 2 cont	ainer corners + be	am reaction)

	Date: 11/9/2023
	Job: 2310219(3) - Milner WWTP
	Subject Wall design
	Reference
	ACI 350-06 ACI 318-14
Wall with compression from beam bearing	
φPn = φ α [0.85 f'c (Ag - Ast) + fy Ast]	10.3.7.2
$\phi = 0.65$	
$\alpha = 0.80$	
Ast = 0.2Ag	
f'c = 4500 psi	
fy = 60,000 psi	
Pu = 49441 lbs Max container load	
Ag = 19.21 in^2	
Ast = 3.84 in^2 Number of #4 bars:	19.2
Number of #5 bars:	12.4
$\phi$ Pn = 150465.1 lbs Number of #6 bars:	8.7
Column size: 24 x 12 Use (8) #6 bars	
a 12 in	
b 24 in 💽	<b>∧</b> • 7 • 1
clear distance 2 in	/
bar diameter 0.75 in #6	•
#bars along long face4 bars	
stirrup diameter 0.375 in #3	
aggregate size 0.75 in	
#3 Tie spacing 12 in	25.7.2.1
Bar clear spacing (a)= 6.5 in	
Bar clear spacing (b)= 6 in ties needed for eve	ery other vertical bar 25.7.2.3
Use #3 stirrups @ 12	inches o.c.

03 LID



123 north martin luther king jr. boulevard tulsa, oklahoma 74103 918.584.5858 · 800.364.5858 wallace.design

	11/0/2023
Job: 231	10219(3) - Milner WWTP
Subject: Cor	ncrete Lid Design
	Reference
Lid Design	ACI 350-06 ACI 318-14
Typical span - away from openings and container support locations	
Thickness = 12 in	
Clear cover = 2 in	
bar area = 0.44 in^2 <b># 6 bar</b>	
bar diameter = 0.75 in <b># 6 bar</b>	
DL = 150 psf	
LL = 60 psf	
(For simplicity, assume LL and SL over full span, even though not under shipping container)	
SI = 84.7 psf -	
Winddown= 9.6 psf SEE BELOW FOR CALCULAT	TIONS
Use 12" wide strip for design	
Factored Loads * Sd (see below for Sd factor) wit	hout Sd 9.2.1
(9-2) 1.2DL+1.6LL+0.5SL 743.3 plf	318.35
(9-3) 1.2DL+1.6SL+1.0LL 743.3 plf	375.52
(9-4) 1.2DI +1.6W+1.0II +0.5SI 767.5 plf	297.71
(9-5) 1.2DI+1.0E+1.0II+0.2S 762.9 plf	264.74
0.052	
I se w = 7675  nlf	component
Madification Factor used in characlead calculations	
woundation Factor used in above load calculations	
Sd = $\phi Fy/(\gamma fs)$ $\phi = \frac{1}{2} \frac{1}{$	N P0.2.6
Sd = $\phi$ Fy/( $\gamma$ fs) $\phi$ = 0.9 (modification factor for tension-controlled section Fy = 60.000 pci (Steel yield strength)	) R9.2.6
Sd = $\phi$ Fy/( $\gamma$ fs) $\phi$ = 0.9 (modification factor for tension-controlled section) Fy = 60,000 psi (Steel yield strength) $\gamma$ = 11 factor (LEED))	) R9.2.6
Sd = $\phi$ Fy/( $\gamma$ fs) $\phi$ = 0.9 (modification factor for tension-controlled section Fy = 60,000 psi (Steel yield strength) $\gamma$ = U factor (Ultimate Load Factor (LRFD)) fr = 20,000 psi (Steel direct here and tensile strength)	) R9.2.6
Sd = $\phi$ Fy/( $\gamma$ fs) $\phi$ =0.9Fy =60,000Fy =60,000psi(Steel yield strength) $\gamma$ =U factor(Ultimate Load Factor (LRFD))fs =20,000psi(Steel direct hoop and tensile strength)	) R9.2.6
Sd = $\phi$ Fy/( $\gamma$ fs) $\phi = 0.9$ (modification factor for tension-controlled section Fy = 60,000 psi (Steel yield strength) $\gamma = U$ factor (Ultimate Load Factor (LRFD)) fs = 20,000 psi (Steel direct hoop and tensile strength)	) R9.2.6
$Sd = \frac{\phi Fy/(\gamma fs)}{\phi = 0.9}$ (modification factor for tension-controlled section Fy = 60,000 psi (Steel yield strength) $\gamma = U \text{ factor}$ (Ultimate Load Factor (LRFD)) fs = 20,000 psi (Steel direct hoop and tensile strength) $fs_{max} = \frac{320}{B (s02 + 4(2+db/2)02)00.5}$	) R9.2.6
$Sd = \frac{\phi Fy/(\gamma fs)}{\phi = 0.9}$ (modification factor for tension-controlled section $Fy = \frac{60,000}{60,000} psi$ (Steel yield strength) $\gamma = U \text{ factor}$ (Ultimate Load Factor (LRFD)) fs = 20,000 psi (Steel direct hoop and tensile strength) $fs_{max} = \frac{320}{B (s^2 + 4(2+db/2)^2)^{-0.5}}$ For "Normal" environment Coating to be applied to concrete s	) R9.2.6 nt exposure: 10.6.4.1 surface, per
$Sd = \frac{\phi Fy/(\gamma fs)}{\phi} = \underbrace{0.9}_{Fy = 60,000} \text{ psi} \qquad (\text{modification factor for tension-controlled section} \\ \gamma = \underbrace{0.9}_{Fy = 60,000} \text{ psi} \qquad (Steel yield strength) \\ \gamma = U \text{ factor} \qquad (Ultimate Load Factor (LRFD)) \\ \text{fs} = \underbrace{20,000}_{B} \text{ psi} \qquad (Steel direct hoop and tensile strength) \\ fs_{max} = \underbrace{320}_{B} \text{ (s}^2 + 4(2+db/2)^2)^{-0.5} \qquad For "Normal" environment Coating to be applied to concrete strength of the strength of the$	) R9.2.6 ht exposure: 10.6.4.1 surface, per -Plate 6000.
$Sd = \frac{\phi Fy/(\gamma fs)}{\phi}$ $\phi = \underbrace{0.9}{Fy = \underbrace{60,000}{psi}}$ $Fy = \underbrace{60,000}{bsi}$ $Fy = \underbrace{0.9}{60,000}$ $\phi = \underbrace{0.9}{Fy = \underbrace{60,000}{psi}}$ $(Ultimate Load Factor (LRFD))$ $fs = \underbrace{20,000}{psi}$ $(Steel direct hoop and tensile strength)$ $fs_{max} = \underbrace{320}{B (s^2 + 4(2+db/2)^2)^{2} \cdot 0.5}$ $For "Normal" environment Coating to be applied to concrete s Aquaworks, Sherwin Williams Dura-Therefore environment the factor of the$	) R9.2.6 nt exposure: 10.6.4.1 surface, per -Plate 6000. is NORMAL 10.6.4.4
$Sd = \frac{\phi Fy/(\gamma fs)}{\phi}$ $\phi = 0.9$ $Fy = 60,000$ $Fy = 0.9$ $Fy = 60,000$ $Fy = 0.9$ $F$	) R9.2.6 nt exposure: 10.6.4.1 surface, per -Plate 6000. is NORMAL 10.6.4.4
Sd = $\phi$ Fy/( $\gamma$ fs) $\phi$ = 0.9 Fy = 60,000 psi (Steel yield strength) $\gamma$ = U factor (Ultimate Load Factor (LRFD)) fs = 20,000 psi (Steel direct hoop and tensile strength) fsmax = 320 For "Normal" environment B = 1.35 B = 1.35 B = 1.35 B = 1.35 B = 1.35 B = 1.35 Coating to be applied to concrete s Aquaworks, Sherwin Williams Dura- Therefore environment $db = 0.75$ in # 6 bar s = 10 in spacing fsmax = 21411 prior (for flowural strengt)	) R9.2.6 ht exposure: 10.6.4.1 surface, per -Plate 6000. is NORMAL 10.6.4.4
$Sd = \frac{\varphi Fy/(\gamma fs)}{\varphi}$ $\varphi = \underbrace{0.9}{Fy = 60,000} psi  (modification factor for tension-controlled section for tension-control section for tension fo$	) R9.2.6 ht exposure: 10.6.4.1 surface, per -Plate 6000. is NORMAL 10.6.4.4 10.6.4
$Sd = \frac{\varphi Fy/(\gamma fs)}{\varphi}$ $\varphi = 0.9$ $Fy = 60,000$ $Fy = 60,000$ $Fy = 0.9$ $Fy = 20,000$ $Fy = 0  (Modification factor for tension-controlled section for tension-control section for tension-contrel section for tension-control section for tension-c$	) R9.2.6 ht exposure: 10.6.4.1 surface, per Plate 6000. t is NORMAL 10.6.4.4 10.6.4 sure) 9.2.6.2 0.2.5 4
$Sd = \frac{\varphi Fy/(\gamma fs)}{\varphi}$ $\frac{\varphi}{Fy} = \underbrace{0.9}{60,000} psi  (Steel yield strength)$ $\gamma = U factor  (Ultimate Load Factor (LRFD))$ $fs = \underbrace{20,000} psi  (Steel direct hoop and tensile strength)$ $fs_{max} = \underbrace{320}_{B}  For "Normal" environmer Coating to be applied to concrete s Aquaworks, Sherwin Williams Dura- B = 1.35  Therefore environment db = 0.75 in # 6 bar s = \underbrace{10}_{I} in spacing fs_{max} = \underbrace{21411 psi}_{20000 psi}  (for flexural stress) \\ 20000 psi  (for direct and hoop tensile stress in normal exposs 24000 psi  (for shear stress carried by shear reinforcement)$	) R9.2.6 ht exposure: 10.6.4.1 surface, per Plate 6000. is NORMAL 10.6.4.4 10.6.4 9.2.6.2 9.2.6.4
$Sd = \frac{\phi Fy/(\gamma fs)}{\varphi}$ $\phi = 0.9$ $Fy = 60,000$ $Fy = 0.9$ $Fy = 60,000$ $Fy = 0.9$ $F$	) R9.2.6 ht exposure: 10.6.4.1 surface, per -Plate 6000. is NORMAL 10.6.4.4 hure) 9.2.6.2 9.2.6.4
$\begin{aligned} Sd &= & \varphi Fy/(\gamma fs) \\ \varphi &= & 0.9 \\ Fy &= & 60,000 \\ Fy &= & 0.9 \\ Fy &= & 0.00 \\ Fy &= & 0.000 \\ Fy &= & 0.00 \\ Fy &= & 0.000 \\ Fy &= & 0.0000 \\$	) R9.2.6 ht exposure: 10.6.4.1 surface, per -Plate 6000. is NORMAL 10.6.4.4 hure) 9.2.6.2 9.2.6.4
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$\begin{array}{cccccccccccccccccccccccccccccccccccc$	) R9.2.6 nt exposure: 10.6.4.1 surface, per Plate 6000. is NORMAL 10.6.4.4 sure) 10.6.4 9.2.6.2 9.2.6.4
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	) R9.2.6 nt exposure: 10.6.4.1 surface, per Plate 6000. is NORMAL 10.6.4.4 nure) 10.6.4 9.2.6.2 9.2.6.4
$\begin{array}{c} Sd = & \varphi Fy/(\gamma fs) \\ \varphi = & 0.9 \\ Fy = & 60,000 \ psi & (Steel yield strength) \\ \gamma = & U \ factor & (Ultimate Load Factor(LRFD)) \\ fs = & 20,000 \ psi & (Steel direct hoop and tensile strength) \\ \hline fs = & 20,000 \ psi & (Steel direct hoop and tensile strength) \\ \hline fs = & 20,000 \ psi & (Steel direct hoop and tensile strength) \\ \hline fs = & 20,000 \ psi & (Steel direct hoop and tensile strength) \\ \hline fs = & 20,000 \ psi & (Steel direct hoop and tensile strength) \\ \hline fs = & 20,000 \ psi & (Steel direct hoop and tensile strength) \\ \hline fs = & 20,000 \ psi & (for flexural stress) \\ & Coating to be applied to concrete s Aquaworks, Sherwin Williams Durates and the stress of the stress of the stress in normal expose 20000 \ \mathsf{psi & (for flexural stress) \\ & 20000 \ psi & (for direct and hoop tensile stress in normal expose 24000 \ \mathsf{psi & (for shear stress carried by shear reinforcement) \\ \hline & & $Sd Factor - Bending \\ \hline & $U \ \mathtt{factor} & $Sd \\ \hline & 1.2 & 2.10 & 2.522 \\ \hline & 1.6 & 1.58 & 2.522 \\ \hline & 1.4 & 1.34 & 1.875 \\ \hline & 1.4 & 1.34 & 1.875 \\ \hline & 0.5 & 3.75 & 1.875 \\ \hline $	) R9.2.6 nt exposure: 10.6.4.1 surface, per Plate 6000. is NORMAL 10.6.4.4 nure) 10.6.4 9.2.6.2 9.2.6.4
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	) R9.2.6 at exposure: 10.6.4.1 surface, per Plate 6000. is NORMAL 10.6.4.4 sure) 10.6.4 9.2.6.2 9.2.6.4

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Subject: Concrete LID Design Span = (Use multi-span condition - 23:5° - 18:4° - 11'-3° (add the/deness of interior walks to span in RISA model) MULTI-SPAN (Assume no containers, no drift) SM u = <u>1073</u> [16] (See RISA output) Using St for in spacing) We = <u>1073</u> [16] (See RISA output) See RISA interior walk of the span and other and the/deness of interior walk output span and other Check simple span Member section forces: (Some interior walk output span and other Prof. Simple span Member section forces: (Some interior walk output span and other Ster will a span denter output span and other Ster will a span denter output span and other (Sae Risa Results) M S = <u>1174</u> 816 30 (bit Mores span) M S = <u>1174</u> 10 (See Risa Results) M S = <u>11734</u> 10 (See					Jop:	2310219(3) - N	/lilner WWTP		
$ \begin{array}{c c c c c c c c c c c c c c c c c c c $					Subject:	Concrete Lid D	esign		
Span = (Use multi-span continion = 23 $\times$ - 18 $\times$ - 11		- (i) iii					Reference	e I	
Inductions of multiply and to be plant in the final model         SM Will and the standard of the sta		Span = (Use multi-	span condition - 23'-	-5" - 18'-0" - 11'-3"			ACI 350-06	ACI 318-14	
$ \begin{array}{c} \begin{array}{c} \mbox{MUIT-SPAN (Assume no containers, no drift) \\ \mbox{Set } \mbox{Mu} = & \mbox{Mu} + & \mbox{Mu} $		ladd thickness of inter	for walls to span in F	RISA model)					
$ \begin{array}{c} \begin{array}{c} \text{Product} \\ \text{Soft } \text{W}_{2} & \frac{1}{2} $	1	ΜΙ ΙΙ ΤΙ-SPΔΝ (Δεςμπρ	no containers no di	-ift)					
Yu =       1373       Ibs       (See RISA output)       (Using Sd for 8in spacing)         Purper controlled schem, dtd author 2 set unding conducts beam, dtd author 2 set unding cond beam, dtd author 2 set undig conducts beam, dtd autho	<u>-</u>	Sd Mu =	45590 lb ft	(See RISA output)	(Using Sd for	8in spacing)			
Check simple span with our status indication and the second status indication is the second status indits is the second status indication is the second statu	,	Vu =	11731 lbs	(See RISA output)	(Using Sd for	8in spacing)			
Insiding conditions below: USE SMARE 59A4       Paint Clear span)       7.4.2         See RISA Results.       See RISA Results.         See RISA is white container in place and snow drifts:       (See Risa Results)         MS d = white and state distriction for the snow drifts:       (See Risa Results)         MS d = white and state distriction for the snow drifts:       (See Risa Results)         MS d = white and state distriction for the snow drifts:       (See Risa Results)         MS d = white and state distriction for the snow drifts:       (See Risa Results)         MS d = white and state distriction for the snow drifts:       (See Risa Results)         MS d = white and state distriction for the snow drifts:       (See Risa Results)         MS d = white and state distriction for the snow drifts:       (See Risa Results)         MS d = white and state distriction for the snow drifts:       (See Risa Results)         MS d = white and state distriction for the snow drifts:       (See Risa Results)         MS d = white and state distriction for the snow drifts:       (See Risa Results)         MS d = snow drifts:       (See Risa Results) <t< td=""><td>(</td><td>Compare continuous "bear</td><td>n" across multiple spans</td><td>with simple span and oth</td><td>her</td><td>on spacing,</td><td></td><td></td><td></td></t<>	(	Compare continuous "bear	n" across multiple spans	with simple span and oth	her	on spacing,			
Check signing span Member section forces:       (Span is clear span)         M Sd = wih2/8 = 13814.36 lb ft       More efficient to use Multi-span - detail accordingly         Por Simple span with container in place and snow drifts:       (See Risa Results)         M Sd = wih2/8 = 122 in       (See Risa Results)         M Sd = 112749 lbs       (Using 5d for 6in spacing)         Lid design       (Using 5d for 6in spacing)         M design       (Using 5d for 6in spacing)         Lid design       (Using 5d for 6in spacing)         Lid design       (Using 5d for 6in spacing)         See Table 21.22 For Strain Boundaries when (et) is compression controlled, transition or tension controlled       Table 7.6.1.1         P = As (D*1)       0.0033 <= space	1	loading conditions below. I	JSE SIMPLE SPAN						
Check simple span Member section forces:       Commit clear span)       7.4         M5d = wi/2/8 = 13814.36 lb ft       More efficient to use Multi-span - detail accordingly       See RISA Results.         S3d=wi/2 = 13814.36 lb ft       More efficient to use Multi-span - detail accordingly       See RISA Results.         S4d=wi/2/8 = 13814.36 lb ft       More efficient to use Multi-span - detail accordingly       See RISA Results.         M5d = 10000 psi       See RISA Results.       DRIFT LOAD = 228 psf       See RISA Results.         M5d = 11749 lbs       (Using S4 for 8in spacing)       Table 7.6.1.1       Table 4.2.2         M5d = 0.0333 <== 5ee nto min calcs below (For State fromin to 15 based)	L			I	<b>·</b>				
M 5d = wh2/8 = minited to be built accordingly       See RISA Results.         M 5d = wh2/8 = minited space and show drifts: V 5d = U 5d = See RISA Results.       See RISA Results.         M 5d = wh2/8 = minited space and show drifts: V 5d = U 5d = Clear cover = d = 9.63 in d = 9.64.13       7.7.1 7.7.	<u>(</u>	Check simple span Me	mber section forces	(Span is	clear span)			7.4.2	
Check w/2       Also and with contrainer in place and show drifts:       Clear Risa Results)       Design is based on this condition         M Sd =       48546 list in condition       Clear Cover = 228 psf       Clear Cover = 228 psf       Clear Cover = 21 in As = 0.528 in n2 ← if 6 bar       Table 7.5.1.1         M Sd =       42567 list in As =       0.528 in n2 ← if 6 bar       Table 7.5.1.1       Table 7.5.1.1         Check Lid Flexure       Mn > Mu       m pmin =       0.0033 the stat adduction to controlled in the spacing       Table 7.5.1.1         P min =       0.0033 the stat adduction to controlled in the spacing       In spacing       Table 7.5.1.1         Check Lid Flexure       Mn > Mu       m pmin =       0.0033 the stat adduction to the state of the state	I	M Sd = wl^2/8 =	13814.36 lb ft	More efficient to use N	1ulti-span - detail a	cordingly		See RISA	Results.
For Simple span with container in place and snow drifts       (See Rise Heartis)       (See Rise Heartis)       (See Rise Heartis)         M 5d =       4564 b ft       (Sing 5d for 8 in spacing)       (See Rise Results)       (See Rise Results)       (Mission See Table 5d for 1 ft)       (Sing 5d for 8 in spacing)         M 5d =       4567 l b ft       (Using 5d for 8 in spacing)       (Using 5d for 8 in spacing)       (See Rise Results)       7.7.1       Table 4.2.2         M 5d =       9.63 in       b =       12 in       in spacing)       10       in spacing)         Check Lid Flexure       M > Mu       0.00333 <== 50e nth on in cacks below (For Slabe showin is T&S Sleet)	$\bigcap$	VSdr WV27	~ 2253/lbs/~	$\sim$				- Design is	based on
M Sd =       Used is bit       DRFT LOAD = 228 psf         V Sd =       1298 ps       DRFT LOAD = 228 psf         M Sd =       4567 / b ft       (Using Sd or Sin spacing)         Uid design       (Using Sd or Sin spacing)       (See Risa Results)         M Sd =       12 in       As =       0.528 in^2 <		For Simple span with o	container in place an	d snow drifts:	(See Risa R	<del>esults</del> )		this condi	tion
$ \begin{array}{c} Vst = & 228 \mbox{ bit} \\ \mbox{ bit} Vst = & 45667 \mbox{ b} f \\ \mbox{ b} Vst = & 11749 \mbox{ b} t \\ \mbox{ b} Vst = & 11749 \mbox{ b} t \\ \mbox{ clear cover} = & 12 \mbox{ in } As = & 0.528 \mbox{ in } spacing \\ \mbox{ clear cover} = & 2 \mbox{ in } Fy = & 60,000 \mbox{ psi} \\ \mbox{ clear cover} = & 2 \mbox{ in } Fy = & 60,000 \mbox{ psi} \\ \mbox{ clear cover} = & 0.75 \mbox{ in } fc = & 4500 \mbox{ psi} \\ \mbox{ d} = & 9.63 \mbox{ in } b = & 12 \mbox{ in } spacing \\ \mbox{ clear cover} = & 0.00033 \mbox{ clear cover} = & 0.00033 \mbox{ clear cover} = & 0.0000 \mbox{ psi} \\ \mbox{ d} = & 9.63 \mbox{ in } b = & 12 \mbox{ in } spacing \\ \mbox{ clear cover} = & 0.00033 \mbox{ clear cover} = & 0.0000 \mbox{ clear cover} = & 0.0000 \mbox{ clear cover} = & 0.0000 \mbox{ clear cover} \\ \mbox{ d} = & 9.63 \mbox{ in } b = & 12 \mbox{ in } spacing \\ \mbox{ clear cover} = & 0.00033 \mbox{ clear cover} = & 0.0005 \\ \mbox{ Table 7.6.1.1} \\ \mbox{ Table 7.6.1.1} \\ \mbox{ Table 7.6.1.2} \\ \mbox{ Table 7.6.1.1} \\ \mbox{ Table 7.6.1.3} \\ \mbox{ Table 7.6.1.3} \\ \mbox{ Table 7.6.1.3} \\ \mbox{ Table 7.6.1.3} \\  Table$	1	M Sd =	18546 lb ft		FT LOAD = 22	8 psf			
Important Square Star statistic retriever Using Sd For 8in spacing)       (Using Sd For 8in spacing)         V Sd =       11749 lbs       (Using Sd For 8in spacing)         Lid design       Lid thickness =       12 in       As =       0.528 in^2        F 6 bar         Lid thickness =       12 in       As =       0.528 in^2        T 7.1.1       Table 4.2.2         Check Lid Flexure       Mn > Mu       in spacing       T 7.1.1       Table 4.2.2         Check Lid Flexure       Mn > Mu       in spacing       T 7.1.1       Table 7.6.1.1         P min =       0.0033 cm sets then pmin sets below       Set Table 7.6.1.1       Table 7.6.1.1         See Table 21.2.2 For Strain Boundaries when (gl is compression controlled, transition or tension controlled       T DI. 21.2.2         No       Compression Controlled Since (gl ) = or > 0.005       T Solve for philMn Based on Whitney Stress Block a a stress block depth = As*Fy / (0.86*Tc*beft) =       0.6002 in.       Fig R21.2.2a         ** Solve for philM based on Whitney Stress Block a a stress phock depth = As*Fy / (0.86*Tc*beft) =       0.0033 ≤ ρ       g.6.1.3         # main =       0.0033 ≤ ρ       0.00150        9.6.1.3       9.6.1.3         # beft bit neutral axis = a / β =       0.0033 ≤ ρ       g.6.1.3       9.6.1.3         # below p min, multiply As 1.33 =       0.7022 in²		V Sd =	5298 lbs						
$ \begin{array}{ccc} M & Sd = & 45667 \ lb \ t & (Using Sd for Sin spacing) \\ V Sd = & 11749 \ lbs & (Using Sd for Sin spacing) \\ \hline V Sd = & 11749 \ lbs & (Using Sd for Sin spacing) \\ \hline Ud \ design \\ Lid \ design \\ Lid \ design \\ Lid \ design \\ Check \ Lid \ flexure & Mn > Mu \\ \hline d = & 9.63 \ in & b = & 12 \ in & fv = & 60,000 \ psi \\ d = & 9.63 \ in & b = & 12 \ in & in \ spacing \\ \hline \hline Table 4.2.2 \\ \hline Table 4.2.2 \\ \hline Table 4.2.2 \\ \hline Table 7.6.1.1 \\ \hline Table 7.6.1.2 \\ \hline Table 7.6.1.2 \\ \hline Table 7.6.1.3 \\ \hline Table 7.6.1 \\ \hline T$	<u> </u>	For multi-Span slab ad	jacent to container	with snow drift	(See Risa R	esults)			
$ \begin{array}{c} V S d = \\ Lid design \\ d = 9.63 in \\ d = 9.00033 \\ d = 9.00033 \\ d = 9.00033 \\ S = 566 \ hom modes below (For Siabs chomin is TaS Sizel) \\ p = As / (b^{\circ}d) \\ 0.00457 <= If p is less than p min, see below \\ See Table 21.2 For Stain Boundaries when (et) is compression controlled that the elevity \\ No \\ Transition Zone If e_{y_{i}} < q_{i} < 0.0053 \\ ves \\ Tension Controlled If et < ety \\ No \\ c = depth to neutral axis = a / \beta \\ design = 0.0033 \\ check against p min = \\ chocs are advery by 14 or multiply vo 75) \\ \hline Lid design = \\ Lid design \\ d = 22040.05 \\ lb fr \\ Mu (Sd) = \\ Lid design \\ check against p min \\ check against p max \\ check against p min \\ check against p max \\ check against $		M Sd =	45667 lb ft	(Using Sd for 8in spacin	ng)				
$\frac{1}{200 \text{ sugn}} \frac{1}{100 \text{ cm}^2} \frac{1}{100 \text{ cm}^2} \frac{1}{100 \text{ m}^2} \frac{1}{10$		V Sd =	11/49 lbs	(Using Sd for 8in spacin	ıg)				
Lid (fitchess = 12 in $F_{y} = 60,000 \text{ psi}$ at a gradient = 0.75 in $f_{c} = 4500 \text{ psi}$ 10 Table 3.22 in $f_{c} = 4500 \text{ psi}$ 10 Table 4.2.2 in $f_{c} = 4500 \text{ psi}$ 10 Table 4.2.2 in $f_{c} = 4500 \text{ psi}$ 10 Table 4.2.2 in $f_{c} = 4500 \text{ psi}$ 10 Table 4.2.2 in $f_{c} = 4500 \text{ psi}$ 10 Table 4.2.2 in $f_{c} = 4500 \text{ psi}$ 10 Table 4.2.2 in $f_{c} = 4500 \text{ psi}$ 10 Table 4.2.2 in $f_{c} = 4500 \text{ psi}$ 10 Table 4.2.2 in $f_{c} = 4500 \text{ psi}$ 10 Table 4.2.2 in $f_{c} = 4500 \text{ psi}$ 10 Table 4.2.2 in $f_{c} = 4500 \text{ psi}$ 10 Table 4.2.2 in $f_{c} = 4500 \text{ psi}$ 10 Coulds 7 <= 10 f min, see below See Table 21.2.2 For Strain Boundaries when (at) is compression controlled, transition or tension controlled Table 21.2.2 For Strain Boundaries when (at) is compression controlled, transition or tension controlled Table 21.2.2 For Strain Boundaries when (at) is compression controlled for 0.005 Yes Tension Controlled Since (et) = or > 0.005 Yes Tension Controlled Since (et) = or > 0.005 Yes Tension Controlled Since (et) = 0.900 \text{ psi} 1.1 Since $f_{c} = 0.9306 \text{ in}$ . Mn = Nominal Moment Cap = 22040 ftlb. check against p min = 0.0033 < p 0.61.3 Point in . (whin x 0.75 = 1/1.133 x whith = reduction if p < p min) (rote: code says p min medicate braining the max eqn if below p min, multiply phiMn x 0.75 = 1/1.33 x whith = reduction if p < p min) (rote: code says p min medicate braining the max eqn if below p min, multiply phiMn x 0.75 = 1/2040 \text{ fb} - ft. Min + Uid Flexure 0 K Utilization = 0.34 Ft. Choose simple span with drift or multiply as a differed by Aquaworks) for the low premeability (concrete strength for concrete with how permeability (concrete strength for concrete with	-	Lid design	12 :-	4.5	F20 := 42 <				
$\begin{array}{c} \text{Litear over } = 2 \text{ in } \\ \text{Bar diameter } = 0.75 \text{ in } \\ \text{d} = 9.63 \text{ in } \\ \text{b} = 12 \text{ in } \\ \text{in spacing} \end{array} \begin{array}{c} \text{Table 4.2.2} \\ \text{Table 4.2.2} \end{array}$		Lid thickness =	12 In 2 in	As = 0	.528 In^2 <	- # 6 bar	774		
bar during $f = 0.5 \text{ in}$ $f = 4300 \text{ ps}$ $f = 12 \text{ in}$ $f = 3500 \text{ ps}$ $f = 12 \text{ in}$ $f = 3500 \text{ ps}$ $f = 12 \text{ in}$ $f = 3500 \text{ ps}$ $f = 12 \text{ in}$ $f = 3500 \text{ ps}$ $f = 12 \text{ in}$ $f = 3500 \text{ ps}$ $f = 12 \text{ in}$ $f = 3500 \text{ ps}$ $f = 12 \text{ in}$ $f = 3500 \text{ ps}$ $f = 12 \text{ in}$ $f = 3500 \text{ ps}$ $f = 12 \text{ in}$ $f = 3500 \text{ ps}$ $f = 1200 \text{ ps}$ $f = 1000 \text{ ps}$ $f = 10000 \text{ ps}$ $f = 1000 \text{ ps}$		Clear cover =	Z IN	Fy = 60	,000 psi	at	7.7.1 Table 4.2.2		
$\frac{d}{d} = 9.65 \text{ m} \qquad b = 12 \text{ m} \qquad [\text{in spacing}]$ $\frac{\text{Check Lid Flexure}}{\text{prim} = 0.00333 \iff \text{ses frem}} \text{ min calcs below (For Slabs thom in is T&S Steel)}$ $p = \text{As } / (b^*d) \qquad 0.00457 \iff \text{tr} \text{ is less than p min, see below}$ See Table 21.22 For Strain Boundaries when (ct) is compression controlled. It transition or tension controlled $\frac{\text{No}  \text{Compression Controlled If et < ety}}{\text{No}  \text{Transition Zone If } e_{y} < e_t < 0.005}$ $\frac{\text{Yes}  \text{Tension Controlled Since (et) = or > 0.005}$ $\frac{\text{Yes}  \text{Tension Controlled Since (et) = or > 0.005}$ $\frac{\text{Yes}  \text{Tension Controlled Since (et) = or > 0.005}$ $\frac{\text{Yes}  \text{Tension Controlled Since (et) = 0.6902 \text{ in.}}{\text{c} - depth to neutral axis = a / \beta = 0.8366 \text{ in.}}$ $\frac{\text{Mn} = \text{Nominal Moment Cap} = 22049 \text{ ftlb.}$ $\frac{\text{Check against p min = 0.0033} < \rho$ $\frac{\text{Ohm sum to Cap} = 22049 \text{ ftlb.}}{\text{Check against p min = 0.0033} < \rho}$ $\frac{\text{Ohm sum to p min, multiply As x 1.33 = 0.7022 \text{ in}^{2}}{\text{if below p min, multiply phiMn x 0.75 = 1} \frac{16337 \text{ kMn = reduction if } \rho  \frac{\text{As } 1.33 \cdot \text{As Requered, so if p m in is not met, can asimply reduce capaelity by 1.4 or multiply 0.75}{\text{Choose simple span with or without snow drift}} \frac{\text{As } 1.33 \cdot \text{As Requered, so if p m in is not met, can asimply reduce capaelity by 1.4 or multiply 0.75}{\text{Choose simple span with or without snow drift}} \frac{\text{As } 1.33 \cdot \text{As Requered, so if p m in is not met, can asimply reduce capaelity by 1.4 or multiply 0.75}{\text{Choose simple span with or without snow drift}} \frac{\text{Abel } 4.3.1}{\text{As } 1.30 \text{ or permeability (concrete is coated is is coated as directed by Aquaworks)}$		Bar diameter =	0.75 IN	rc= 2	12 in	10	Table 4.2.2		
Check Lid FlexureMn > MuTable 7.6.1.1 $\rho = As / (b^*d)$ 0.00437 <== tip is less than pmin, see below		u =	9.03 10	D =	12 10				
$\frac{p \text{ min}}{p \text{ min}} = 0.00333 \iff \text{See from} \text{ in calcs below (For Slabs thomin is T&S Steel)} \\ p = A_S / (b^{+}d) 0.00457 \iff \text{If } p \text{ is less than } p \text{ min, see below} \\ \text{See Table 21.2.2 For Strain Boundaries when (et) is compression controlled, transition or tension controlled \text{ Tbl. 21.2.2} \\ \text{No}  \text{Compression Controlled If et < ety} \\ \text{No}  \text{Transition Zone If } e_{y} < e_{i} < 0.005 \\ \text{Yes}  \text{Tensition Controlled Since (et) = or > 0.005} \\ \text{** Solve for philm Based on Whitney Stress Block} \\ a = stress block depth = As^{+}Fy / (0.85^{+}fc^{+}beff) = 0.6902 \text{ in.} \\ c = depth to neutral axis = a / \beta = 0.8366 \text{ in.} \\ \text{Mn = Nominal Moment Cap = 22049 ftlb.} \\ \phi^{=}  0.9 \\ \phi^{Mn} = \text{Ultimate Moment Cap = 3 s^{+}Fy^{*}(d-a/2)/12 = 24499 \text{ ftlb.} \\ check against p max = 0.0150 \iff \text{determine fho max eqn} \\ \text{if below p min, multiply As x 1.33 = 0.7022 in^{2} \text{ ct}^{-2} \text{ multiply x 0.75} \\ \text{function of the oright a xis a a / \beta = 0.5935 lb-ft \\ \text{in. } \phi \text{Mn x 0.75 = 1/1.33^{+}As \text{ Required, so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.75. \\ \text{function of the oright a xis not drift } Note: \\ 4500  psi s minimum concrete strength for concrete with low permeability (concrete strength for concrete with low permeability (concrete is coated dis is coated dis is coated dis is coated dis coa$		Check Lid Flexure	Mn > Mu						
$ \begin{array}{c} p = As / (b^*d) \\ 0.00457 <== If p is less than p min, see below \\ \hline \\ $	-	o min =	0.00333 <== See rh	 o min calcs below (For Sla	bs rhomin is T&S Si	eel)		Table 7.6.1.1	
See Table 21.2.2 For Strain Boundaries when (et) is compression controlled, transition or tension controlled No Compression Controlled If et < ety No Transition Zone If $\varepsilon_{b_1} < \varepsilon_i < < 0.005$ Yes Tension Controlled Since (et) = or > 0.005 ** Solve for phiMn Based on Whitney Stress Block a = stress block depth = $As^+Fy'(0.85^+tc^+beff) = 0.6902$ in. c = depth to neutral axis = a / $\beta = 0.8366$ in. Mn = Nominal Moment Cap = $22049$ ftlb. $\phi = 0.9$ $\phi = 0.0033 < \rho$ check against p min = $0.0033 < \rho$ check against p max = $0.0150 <== determine tho max eqn$ If below p min, multiply As x 1.33 = $0.7022$ in $h c_2$ if below p min, multiply phiMn x 0.75 = $1.6537$ lb-ft in. $\phi Mn x 0.75 = 1.133^{\circ} As$ Required, so if p min is not met, can simply reduce capacity by 14 or multiply x 0.75) $\phi Mn = Ultimate Moment Cap = 22049.05$ lb-ft $As > 1.33^{\circ} As$ Required, so if p min is not met, can simply reduce capacity by 14 or multiply x 0.75) $\phi Mn = Ultimate Moment Cap = 18546$ lb ft * Choose simple span with drift or multi span with or without snow drift Note: 4500 psi is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks)		p = As / (b*d)	0.00457 <== If p is l	ess than $\rho$ min, see below					
No Compression Controlled If $et < ety$ No Transition Zone If $e_{ty} < e_t < 0.005$ Yes Tension Controlled Since $(et) = or > 0.005$ ** Solve for phiMn Based on Whitney Stress Block a = stress block depth = As*Fy / (0.85*fc*beff) = 0.6902 in. c = depth to neutral axis = a / $\beta$ = 0.8366 in. Mn = Nominal Moment Cap = 0.9 $\emptyset$ Mn = Ultimate Moment Cap = 220499 ftlb. $\varphi$ = 0.9 $\emptyset$ Mn = Ultimate Moment Cap = 220499 ftlb. $\varphi$ = 0.0150 <== determine tho max eqn If below p min, multiply As x 1.33 = 0.7022 in^2 if below p min, multiply phiMn x 0.75 = 16537 lb.ft in. $\emptyset$ An x 0.75 = 1/1.33 x $\emptyset$ Mn = reduction if $p < p$ min) (note: code says p min sequence capacity by 1/4 or multiply x 0.75) $\emptyset$ Mn = Ultimate Moment Cap = 22049.05 [b.ft Mu (Sd) = 18546 [lb ft < fMn - Lid Flexure OK Utilization = 0.84 * Choose simple span with drift or multi span with or without snow drift Note: 4500 ps is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks)		See Table 21.2.2 For Strair	n Boundaries when (ɛt) is	compression controlled, tr	ansition or tension of	ontrolled		Tbl. 21.2.2	
No Compression Controlled If et < ety No Transition Zone If $e_{by} < e_t < 0.005$ Yes Tension Controlled Since (et) = or > 0.005 ** Solve for phiMn Based on Whitney Stress Block a = stress block depth = As*Fy / (0.85*fc*beff) = 0.6902 in. c = depth to neutral axis = a / $\beta$ = 0.8366 in. Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 = 24499 ftlb. $\phi$ = $\phi$ Mn = Ultimate Moment Cap = 22049 ftlb. check against p min = 0.0033 < p (note: code says p min ned not be satisfied if As > 1.33 * As Required, so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.75; $\phi$ Mn = Ultimate Moment Cap = 22049 ftlb. (note: code says p min ned not be satisfied if As > 1.33 * As Required, so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.75; $\phi$ Mn = Ultimate Moment Cap = 22049.05 [b-ft Mu (5d) = 18546] lb ft < fMn - Lid Flexure OK Utilization = 0.84 * Choose simple span with drift or multi span with or without snow drift Note: 4500 ps is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks)									
No Transition Zone If $e_{hy} < e_t < 0.005$ Yes Tension Controlled Since (et) = or > 0.005 ** Solve for phiMn Based on Whitney Stress Block a = stress block depth = As*Fy / (0.85*fc*beff) = 0.6902 in. c = depth to neutral axis = a / $\beta$ = 0.8366 in. Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 = 24499 ftlb. $\phi^{=r}$ 0.9 (21.2.1) $\phi^{=r}$ 0.9 (21.2.1) $\phi^{=r}$ 0.9 (21.2.1) $\phi^{=r}$ 0.9 (21.2.1) $\phi^{=r}$ 0.9 (21.2.1) $\phi^{=r}$ 0.0033  check against p min = 0.0033 $\phi^{=r}$ 0.0150 <== determine rho max eqn If below p min, multiply As x 1.33 = 0.7022 in^2 9.6.1.3 $\phi^{=r}$ 0.0150 <== determine rho max eqn If below p min, multiply phiMn x 0.75 = 1/1.33 x 0Mn = reduction if p  (note: code says p min need not be satisfied if As > 1.33 * As Required. so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.750 $\phi^{Mn} = Ultimate Moment Cap = 22049.05$ (b-ft Mu (5d) = 18546 (b ft < fMn - Lid Flexure OK Utilization = 0.84 * Choose simple span with or without snow drift Note: 4500 ps is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks)		No	Compression Contro	olled If ct < cty					
Yes Tension Controlled Since (et) = or > 0.005 ** Solve for phiMn Based on Whitney Stress Block a = stress block depth = As*Fy / (0.85*fc*beff) = 0.6902 in. c = depth to neutral axis = a / $\beta$ = 0.8366 in. Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 = 24499 ftlb. $\phi$ = 0.9 $\phi$ Mn = Ultimate Moment Cap = 22049 ftlb. check against p min = 0.0153 <= p check against p max = 0.0155 <== determine rho max eqn If below p min, multiply As x 1.33 = 0.7022 in^2 if below p min, multiply phiMn x 0.75 = 1.6537 lb-ft in. $\phi$ Mn x 0.75 = 1/1.33 x $\phi$ Mn = reduction if $\rho < p$ min) (note: code says p min ned not be satisfied if As > 1.33 * As Required, so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.75) $\phi$ Mn = Ultimate Moment Cap = 220409 ft he ft Mu (Sd) = 18546 lb ft * Choose simple span with drift or multi span with or without snow drift Note: 4500 ps is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks)		No	Transition Zone If $\varepsilon_{t_y}$	$c_t < \varepsilon_t < 0.005$					
** Solve for phiMn Based on Whitney Stress Block a = stress block depth = As*Fy / (0.85*fc*beff) = 0.6902 in. c = depth to neutral axis = a / $\beta$ = 0.8366 in. Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 = 24499 ftlb. $\phi$ = 0.9 $\phi$ Mn = Ultimate Moment Cap = 22049 ftlb. check against $\rho$ min = 0.0033 < $\rho$ check against $\rho$ min = 0.0033 < $\rho$ check against $\rho$ min, multiply As x 1.33 = 0.7022 in^2 if below $\rho$ min, multiply phiMn x 0.75 = 16537 lb-ft in. $\phi$ Mn = Ultimate Moment Cap = 0.9 $\phi$ Mn = Ultimate Moment Cap = 0.0150 <== determine tho max eqn if below $\rho$ min, multiply phiMn x 0.75 = 16537 lb-ft in. $\phi$ Mn x 0.75 = 1/1.33 x $\phi$ Mn = reduction if $\rho < \rho$ min (note: code says $\rho$ min is not met, can simply reduce capacity by 14 or multiply x 0.75) $\phi$ Mn = Ultimate Moment Cap = 22049.05 lb-ft Mu (Sd) = 18546 lb ft < fMn - Lid Flexure OK Utilization = 0.84 * Choose simple span with drift or multi span with or without snow drift Note: 4500 psi is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks)		Yes	Tension Controlled S	Since (ct) = or > 0.005	5				
** Solve for phiMn Based on Whitney Stress Block a = stress block depth = As*Fy / (0.85*fc*beff) = 0.6902 in. c = depth to neutral axis = $a/\beta$ = 0.8366 in. Mn = Nominal Moment Cap = 0.9 $\phi$ Mn = Ultimate Moment Cap = 22049 ftlb. check against p min = 0.0033 < p check against p max = 0.0150 <== determine rho max eqn If below p min, multiply As x 1.33 = 0.7022 in^2 if below p min, multiply phiMn x 0.75 = 16537 lb-ft in. $\phi$ Mn x 0.75 = 1/1.33 x $\phi$ Mn = reduction if $p < p$ min) (note: code says p min in need not be satisfied if As > 1.33 * As Required, so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.75) $\phi$ Mn = Ultimate Moment Cap = 22049.05 /b-ft Mu (Sd) = 18546 lb ft < fMn - Lid Flexure 0K Utilization = 0.84 * Choose simple span with drift or multi span with or without snow drift Note: 4500 psi is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks)									
a = stress block depth = As <sup>*</sup> Fy <sup>7</sup> (0.85 <sup>*</sup> fc <sup>*</sup> beff) = 0.6902 in. c = depth to neutral axis = a / $\beta$ = 0.8366 in. Mn = Nominal Moment Cap = As <sup>*</sup> Fy <sup>*</sup> (d-a/2)/12 = 24499 ftlb. $\phi$ = 0.9 $\phi$ Mn = Ultimate Moment Cap = 22049 ftlb. check against p min = 0.0033 < p check against p max = 0.0150 <== determine rho max eqn If below p min, multiply As x 1.33 = 0.7022 in^2 if below p min, multiply phiMn x 0.75 = 16537 lb-ft in. $\phi$ Mn x 0.75 = 1/1.33 × $\phi$ Mn = reduction if p  (note: code says p min need not be satisfied if As > 1.33 * As Required, so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.75) $\phi$ Mn = Ultimate Moment Cap = 22049.05 $\phi$ Mn = Ultimate Moment Cap = 22049.05 b-ft Mu (Sd) = 18546 lb ft < fMn - Lid Flexure OK Utilization = 0.84 * Choose simple span with drift or multi span with or without snow drift Note: 4500 psi is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks)	1	** Solve for phiMn Bas	ed on Whitney Stres	s Block				Fig R21.2.2a	
$c = depth to neutral axis = a / \beta = 0.8366 in.$ $Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 = 24499 ft.lb.$ $\phi^{=} 0.9$ $\phi Mn = Ultimate Moment Cap = 22049 ft.lb.$ $check against \rho min = 0.0033 < \rho$ $check against \rho max = 0.0150 <== determine rho max eqn$ If below p min, multiply As x 1.33 = 0.7022 in^2 if below p min, multiply phiMn x 0.75 = 16537 lb-ft in. $\phi Mn x 0.75 = 1/1.33 \times \phi Mn = reduction if \rho < \rho min)$ $(note: code says p min need not be satisfied if As > 1.33 * As Required, so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.75)$ $\phi Mn = Ultimate Moment Cap = 22049.05 lb-ft$ $Mu (Sd) = 18546 lb ft < fMn - Lid Flexure OK Utilization = 0.844$ * Choose simple span with drift or multi span with or without snow drift Note: 4500 psi is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks) $Table 4.3.1$	i	a = stress block depth	= As*Fy / (0.85*f'c*l	oeff) = 0.6	5902 in.				
$\begin{array}{c ccccc} Mn = Nominal Moment Cap = & 24499 \ \mathrm{ft.lb.} & & & & & & & & & & & & & & & & & & &$	(	c = depth to neutral ax	is = a / β =	3.0	3366 in.				
$ \phi^{=} \qquad 0.9 \qquad 21.2.1 \qquad 21.2$		Mn = Nominal Moment	t Cap = As*Fy*(d-a/2	2)/12 = 22	1499 ftlb.			04.0.4	
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	0	ψ– •Mn – I Iltimata Mama	at Can -	00	0.9		D10.2 F	21.2.1	
check against p min =       0.0033 < p		ebock against a min =	n Cap –	22	2049 1110.		R10.3.5	0612	
if below ρ min, multiply As x 1.33 =       0.702 in^2       9.6.1.3         if below ρ min, multiply phiMn x 0.75 =       16537 lb-ft       9.6.1.3         if below ρ min, multiply phiMn x 0.75 =       16537 lb-ft       9.6.1.3         in φMn x 0.75 = 1 / 1.33 x φMn = reduction if ρ < ρ min)		check against p min –		0.0	1033	p a rha may agn		9.0.1.3	
if below p min, multiply phiMn x 0.75 = $16537$ lb-ft in. $\phi$ Mn x 0.75 = $1/1.33 \times \phi$ Mn = reduction if $p < p$ min) (note: code says p min need not be satisfied if As > 1.33 * As Required, so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.75) $\phi$ Mn = Ultimate Moment Cap = $22049.05$ lb-ft Mu (Sd) = $18546$ lb ft * Choose simple span with drift or multi span with or without snow drift Note: 4500 psi is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks) Table 4.3.1	Ì	If below a min_multiply	As x 1 33 =	0.0	7022 in^2	e mo max eqn		9613	
$\frac{1}{1} \frac{1}{1} \frac{1}$	i	if below o min, multiply	$r_{\rm nbi}$ Mn x 0 75 =	0.7 16	537 lb-ft			3.0.1.5	
(note: code says p min need not be satisfied if As > 1.33 * As Required, so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.75) $\phi Mn = Ultimate Moment Cap = 22049.05 lb-ft$ Mu (Sd) = 18546 lb ft < fMn - Lid Flexure OK Utilization = 0.84 * Choose simple span with drift or multi span with or without snow drift Note: 4500 psi is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks)				in.	3 x  Mn = reduction	if $o < o \min$			
\$\phi Nn = Ultimate Moment Cap =       As > 1.33 * As Required, so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.75)         \$\phi Mn = Ultimate Moment Cap =       22049.05         Mu (Sd) =       18546         18546       Ib ft         < fMn - Lid Flexure OK				(note: code savs o min i	need not be satisfied	lif			
<pre>simply reduce capacity by 1/4 or multiply x 0.75) \$\phi Mn = Ultimate Moment Cap = Mu (Sd) = 18546] lb ft * Choose simple span with drift or multi span with or without snow drift Note: 4500 psi is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks) Table 4.3.1 Table 4.3.1 Table</pre>				As > 1.33 * As Required	l, so if ρ min is not n	net, can			
\$\phi Mn = Ultimate Moment Cap =       22049.05       lb-ft         Mu (Sd) =       18546       lb ft       < fMn - Lid Flexure OK				simply reduce capacity	by 1/4 or multiply x (	).75)			
Mu (Sd) =       18546 lb ft       < fMn - Lid Flexure OK		φMn = Ultimate Mom	ent Cap =	2204	9.05 lb-ft	,			
* Choose simple span with drift or multi span with or without snow drift Note: 4500 psi is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks)	I	Mu (Sd) =	18546 lb ft	< fMn - Lid Flexure O	K Utilization =	0.84			
or multi span with or without snow drift Note: 4500 psi is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks) Note: Table 4.3.1	:	* Choose simple span wi	th drift						
4500 psi is minimum concrete strength for concrete Table 4.3.1 with low permeability (concrete is coated is coated as directed by Aquaworks)	(	or multi span with or wit	hout snow drift	Note:					
with low permeability (concrete is coated is coated as directed by Aquaworks)				4500 psi is minimum	concrete strength	for concrete	Table 4.3.1		
is coated as directed by Aquaworks)				with low permeabilit	y (concrete is coa	ted			
				is coated as directed	l by Aquaworks)				

	Date: 11/6/2023	3			
	Job: 2310219(3) -	Milner WWTP			
	Subject: Concrete Lid	Design			
		Reference			
		ACI 350-06 ACI 318-14			
Check $\rho$ balanced and et=.005 limits:					
β= Max of ((0.85-0.05*(f'c-4000)/1000,0.65)) =	0.8250	10.2.7.3			
ety = tension yield strain = fy / Es	0.0021 <== Es = 29,000,000 psi	R10.3.2			
ec = balanced concrete strain =	0.0030	R10.3.2			
et = actual tension strain = ((d-c) / c)*strain sc	0.0315	R10.3.3			
$\rho$ temp = temp and shrink steel = 0.0018 <== if Fy< As(temp)= ( $\rho$ temp * beff * h) = 0.2592 in^2	:60,000 psi, .002 -or0018 * 60,000psi / Fy	Table 7.12.2.1			
ρ bal = (0.85*f'c*β/Fy)*(0.003/(0.003+εty)	0.0311 <== balanced steel ratio	R10.3.3			
0.75(ρ bal)=	0.0233	R10.3.5			
pt = (.85*β*f'c / Fy)*(0.003/(0.008))	0.0197 <== Max reinf. Ratio, whe	n steel strain >= .005			
As (max) = (ρt * beff * d) =	2.2780 in^2	R10.3.5			
ρ min (b) = 200/fy=	0.0033	9.6.1.2			
ρ min (a) = 3*(f'c)^0.5/Fy=	0.0034	10.5.1 9.6.1.2			
ρ min.= greater of ρ min(a) or ρ min(b) =	0.0034 <== Min. reinf. Ratio				
As (min)= (ρ min * beff * d) =	0.3874 in.2 <== unless As > 1.33*As	reqd per code 9.6.1.3			
Check Shear Capacity **neglects Vs (shear reinforcement) Vc = 2 * $\lambda$ * (fc) <sup>2</sup> .5 * b * d =	15496 lbs	22.5.5.1			
φ = 0.75 φV	c = 11622 lbs	21.2			
(Environmental durability factor does not apply to concrete sha		926			
$V_{\rm H} = V_{\rm max}(11) = 5298$ lbc $V_{\rm H}$ show the	Shoar OK	5.2.0			
See Belov	w for Reinforcement Check	9.2.1			
Vs required = Sd (Vu-fVc) = -15949.4 lbs No	shear reinforcement required	R9.2.6.4			
Use 12 in slab w/ #6 ba	r @ 10 in spacing				









#### Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	12	0	0	

#### Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]
1	N2	Reaction	Reaction	Reaction	
2	N1		Reaction	Reaction	Reaction

#### Member Distributed Loads (BLC 1 : DL)

	Member Label	Direction	Start Magnitude [lb/ft, F, psf, k-ft/ft]	End Magnitude [lb/ft, F, psf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-150	-150	0	%100

#### Member Distributed Loads (BLC 2 : LL)

	Member Label	Direction	Start Magnitude [lb/ft, F, psf, k-ft/ft]	End Magnitude [lb/ft, F, psf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-60	-60	7	12

#### Member Distributed Loads (BLC 3 : SL)

N	lember Label	Direction	Start Magnitude [lb/ft, F, psf, k-ft/ft]	End Magnitude [lb/ft, F, psf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1 Y -77.0		-77.0	-77.0	7	12

#### Member Distributed Loads (BLC 4 : SL - drift)

	Member Label	Direction	Start Magnitude [lb/ft, F, psf, k-ft/ft]	End Magnitude [lb/ft, F, psf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-228.5	-228.5	7	12

#### Member Distributed Loads (BLC 5 : W)

Member Label	Direction	Start Magnitude [lb/ft, F, psf, k-ft/ft]	End Magnitude [lb/ft, F, psf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1 M1	Y	-9.6	-9.6	7	12

#### **Basic Load Cases**

	BLC Description	Category	Distributed
1	DL	DL	1
2	LL	RLL	1
3	SL	SL	1
4	SL - drift	SL	1
5	W	WL	1

#### Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	Deflection 1		Y	1	1						
2	Deflection 2		Y	2	1						
3	Deflection 3		Y	1	1	2	1				
4	Deflection 4		Y	1	1	3	1				
5	Deflection 5		Y	1	1	4	1				



#### Load Combinations (Continued)

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
6	(9-2) DL + LL + SL		Y	1	1.2	2	1.6	3	0.5		
7	(9-2) DL + LL + SL(drift)		Y	1	1.2	2	1.6	4	0.5		
8	(9-3) DL + SL + LL		Y	1	1.2	3	1.6	2	1		
9	(9-3) DL + SL(drift) + LL		Y	1	1.2	4	1.6	2	1		
10	(9-3) DL + SL +W		Y	1	1.2	5	0.8	3	1.6		
11	(9-3) DL + SL(drift) + W		Y	1	1.2	5	0.8	4	1.6		
12	(9-4) DL +W + LL + SL		Y	1	1.2	2	1	3	0.5	5	1.6
13	(9-4) DL +W + LL + SL(drift)		Y	1	1.2	2	1	4	0.5	5	1.6
14	(9-5) DL +E+ LL + SL		Y	1	1.2	2	1	3	0.2	1	0.074
15	(9-5) DL +E+ LL + SL(drift)		Y	1	1.2	2	1	4	0.2	1	0.074
16	Deflection 1		Y	1	1						
17	Deflection 2		Y	2	1						
18	Deflection 3		Y	1	1	2	1				
19	Deflection 4		Y	1	1	3	1				
20	Deflection 5		Y	1	1	4	1				
21	(9-2) DL + LL + SL	Yes	Y	1	2.522	2	2.522	3	2.522		
22	(9-2) DL + LL + SL(drift)	Yes	Y	1	2.522	2	2.522	4	2.522		
23	(9-3) DL + SL + LL	Yes	Y	1	2.522	3	2.522	2	2.522		
24	(9-3) DL + SL(drift) + LL	Yes	Y	1	2.522	4	2.522	2	2.522		
25	(9-3) DL + SL +W	Yes	Y	1	2.522	5	2.522	3	2.522		
26	(9-3) DL + SL(drift) + W	Yes	Y	1	2.522	5	2.522	4	2.522		
27	(9-4) DL +W + LL + SL	Yes	Y	1	2.522	2	2.522	3	2.522	5	2.522
28	(9-4) DL +W + LL + SL(drift)	Yes	Y	1	2.522	2	2.522	4	2.522	5	2.522
29	(9-5) DL +E+ LL + SL	Yes	Y	1	2.522	2	2.522	3	2.522	1	2.522
30	(9-5) DL +E+ LL + SL(drift)	Yes	Y	1	2.522	2	2.522	4	2.522	1	2.522

#### Envelope Member Section Forces

	Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
1	M1	1	max	0	30	5.298	30	0	30	0	30	0	30	0	30
2			min	0	21	2.518	25	0	21	0	21	0	21	0	21
3		2	max	0	30	3.028	30	0	30	0	30	0	30	-5.85	25
4			min	0	21	1.383	25	0	21	0	21	0	21	-12.488	30
5		3	max	0	30	0.783	28	0	30	0	30	0	30	-8.296	25
6			min	0	21	0.248	25	0	21	0	21	0	21	-18.166	30
7		4	max	0	30	-1.363	25	0	30	0	30	0	30	-6.861	25
8			min	0	21	-2.967	30	0	21	0	21	0	21	-15.58	30
9		5	max	0	30	-3.211	25	0	30	0	30	0	30	0	30
10			min	0	21	-7.42	30	0	21	0	21	0	21	0	21

#### Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N2	max	0	30	7.42	30	0	30	0	30	0	30	0	30
2		min	0	21	3.211	25	0	21	0	21	0	21	0	21
3	N1	max	0	30	5.298	30	0	30	0	30	0	30	0	30
4		min	0	21	2.518	25	0	21	0	21	0	21	0	21
5	Totals:	max	0	30	12.717	30	0	30						
6		min	0	21	5.729	25	0	21						

	Date:	11/6/2023		
	Job:	2310219(3) - N	Ailner WWTP	
	Subject:	Concrete Lid D	esign	
			Reference	e
			ACI 350-06	ACI 318-14
Slab section near opening				
Slab Reinf is at in spacing				
Opening width = <b>5.5</b> ft (3 ft on each side of slab strip				
"beam" width = <b>2.75</b> ft (slab strip width between openi	ngs)			
Sd = 1				
Mu Sd = 40444 lb ft * (Multiply Typical lid Momen	nt by (1/2 open	ing+beam width)		
Vu = 64521 lbs * (Multiply Typical lid Shear b	y (1/2 opening	+ beam width)		
* Sd = 1.0 for 3.14	in spacing	of #6 bar		
# typical bars in "beam" width = 4				
Added bars next to opening = 6 (3 each opening =	ening)			
Can 10 bars fit in "beam"?				
bar spacing = 3.14 in	BAR SPACI	NG OK		
Minimum spacing = larger of: 1 in				25.2
db = 0.75 in				25.2
agg = 0.75 4/3 dagg = 1 in				25.2
				_
$\rho$ bal = (0.85*f'c*b/Fv)*(0.003/(0.003+etv) 0.0311	L <== balanc	ed steel ratio		
0.75(o bal)= 0.0233	}			
etv = tension vield strain = fv / Es 0.0021	<== Es = 29	j.000.000 psi		
Total Steel Area = As = $4.4 \text{ in}^2$ Less than A	As (max)	,,		
depth, $d = 9.63$ in Use	• 4.4(	) in^2		
width, $b = 33$ in	(See below	(for Max As)		
$F_{V} = 60.000 \text{ psi}$	(000 00.01			
$f'_{c} = 4500 \text{ psi}$				
Check Lid Flexure Mn > Mu				
$\rho$ min = 0.00333 <== See rho min calcs below (For Slabs rh	omin is T&S S	teel)		Table 7.6.1.1
$\rho = As / (b^*d)$ 0.01385 <== If $\rho$ is less than $\rho$ min, see below		,		
See Table 21.2.2 For Strain Boundaries when (ct) is compression controlled, transit	ion or tension o	controlled		Tbl. 21.2.2
No Compression Controlled If et < etv				
No Transition Zone If $\varepsilon_{tv} < \varepsilon_t < 0.005$				
Yes Tension Controlled Since (et) = or $> 0.005$				
** Solve for phiMn Based on Whitney Stress Block				Fig R21 2 2a
a = stress block depth = $As^*Fv / (0.85*fc^*beff) = 2.0915$	in.			· .g · .==.=
c = depth to neutral axis = a / $\beta$ = 25352	) in			
$Mn = Nominal Moment Can = As^*Fv^*(d-a/2)/12 = 188743$	 } ft -lb			
φ= 0.9	)			21 2 1
φMn = Ultimate Moment Cap = 169869	, ) ft -lh		R10 3 5	21.2.1
check against o min = 0.0033	}	0	11201010	9613
check against o max = 0.0233	, <== determin	P e rho max eqn		0.0.1.0
If below $\rho$ min. multiply As x 1.33 = 58520	) in^2	o mo max oqu		9613
if below $\rho$ min, multiply phiMn x 0.75 = 127402	Plb-ft			0.0.110
in. $\phi Mn \ge 0.75 = 1/1.33 \ge 0.12$	Mn = reduction	if $\rho < \rho$ min)		
(note: code savs o min need	not be satisfie	d if		
As > 1.33 * As Required, so	if o min is not n	net. can		
simply reduce capacity by 1/	4 or multiply v (	) 75)		
<i>Mn</i> = Ultimate Moment Cap =	Ih-ft			
$M_{\rm H} = \frac{40444}{1 < f_{\rm Mn} - 1 id Elevure OK}$	Litilization -	0.24		
	0111201011 =	0.24		
llse 12 in slah w/ 10 #6 bar	hetween	nenings		
	Serweeno	PC111165		I

		Date:	11/6/2023	}	
		Job:	2310219(3) -	Milner WWTP	
		Subject:	Concrete Lid	Design	
				Reference	2
				ACI 350-06	ACI 318-14
Check $\rho$ balanced and $\pi$ t=.005 limits:					
β= Max of ((0.85-0.05*(f'c-4000)/1000	,0.65)) =	0.8250		10.2.7.3	
ety = tension yield strain = fy / Es		0.0021 <== Es = 2	29,000,000 psi	R10.3.2	
εc = balanced concrete strain =		0.0030		R10.3.2	
et = actual tension strain = ((d-c) / c)*s	train sc	0.0084		R10.3.3	
$\rho$ temp = temp and shrink steel = 0 As(temp)= ( $\rho$ temp * beff * h) = 1	0050 9800 in^2			Table 7.12.2.1	
$\rho$ bal = (0.85*f'c* $\beta$ /Fy)*(0.003/(0.003+	-ety)	0.0311 <== balan	ced steel ratio	R10.3.3	
0.75(ρ bal)=		0.0233		R10.3.5	
ρt = (.85*β*f'c / Fy)*(0.003/(0.008))		0.0197 <== Max re	inf. Ratio, when ste	el strain >= .005	
As (max) = (pt * beff * d) =		6.26 in^2		R10.3.5	
		Use 14	# 6 bar		
ρ min (b) = 200/fy=		0.0033			9.6.1.2
ρ min (a) = 3*(f'c)^0.5/Fy=		0.0034		10.5.1	9.6.1.2
$\rho$ min.= greater of $\rho$ min(a) or $\rho$ min(b	) =	0.0034 <== Min. re	inf. Ratio		
As (min)= (ρ min * beff * d) =		1.0653 in^2 <== un	lless As > 1.33*As re	eqd per code	9.6.1.3
Check Shear Capacity **neglects Vs (shear reinforcement) Vc = 2 * $\lambda$ * (f'c)^.5 * b * d =		42614 lbs			22.5.5.1
φ = 0.75	φVc =	31960 lbs			21.2
Check if shear capacity is OK without cons	idering shear reinforce	ment			
(Environmental durability factor does not app	y to concrete shear capa	icity)		9.2.6	
Vu = Vmax (U) = 0 lbs	< Vc - Slab Shear	OK Utilization	= 0.00	)	
	See Below for R	einforcement Check		9.2.1	
Vs required = Sd (Vu-fVc) =	0 lbs - Shear rei	nforcement require	Ч	R9264	

Date:	11/6/2023
Job:	2310219(3) - Milner WWTP
Subject:	Concrete Lid Design





						Date:	11/6/	2023		
						Job:	2310219	(3) - N	/ilner WWTP	
						Subject:	Concrete	Lid D	esign	
									Reference	2
Modification	on Factor u	sed in abov	ve load calc	ulations					ACI 350-06	ACI 318-14
Sd =	φFy/(γfs)									
φ =	0.9		(modificati	ion factor fo	r tension-co	ntrolled se	ction)		R9.2.6	
Fy =	60,000	psi	(Steel yield	d strength)						
γ =	factored load/	unfactored loa	d	(Ultimate Lo	oad Factor (I	LRFD))				
fs =	20,000	psi	(Steel dire	ct hoop and	tensile strer	ngth)				
		-								
fs <sub>max</sub> =		320			For "Nor	rmal" enviro	onment expo	sure:	10.6.4.1	
-	B (s^2 + 4	(2+db/2)^2	2)^0.5	Coat	ing to be app	lied to conc	rete surface	, per		
				Aqua	works, Sherw	vin Williams	Dura-Plate	6000.		
B =	1.35	I			There	fore environ	nment is NOF	RMAL	10.6.4.4	
db =	0.75	in	#6 bar							
s =	3.85	in spacing								
fs <sub>max</sub> =	38767	psi	(for flexura	al stress)					10.6.4	
	20000	psi	(for direct	and hoop te	nsile stress i	in normal e	exposure)		9.2.6.2	
	24000	psi	(for shear	stress carried	d by <b>shear r</b> e	einforceme	ent)		9.2.6.4	
		1 -								
	Sd(min) = 2	1.0							9.2.6	
Sd Factor	- Bending			Sd Facto	r - Shear	1				
U factor	Sd	Sd*U		U factor	Sd	Sd*U				
1.2	1.16	1.393		1.2	1.88	2.25	0			
1.6	1.00	1.600	)	1.6	1.41	2.25	0			
1.4	1.00	1.400		1.4	1.61	2.25	0			
0.5	2.79	1.393		0.5	4.50	2.25	0			
1	1.39	1.393		1	2.25	2.25	0			
0.2	6.96	1.393		0.2	11.25	2.25	0			
Moment a	t mid-span		1			P				
M	u = Pab/L =	127375.9	lb ft			<u>ل</u>		a=	6.75	ft
Shear at su	upport			<u> </u>	a —	└── b ──		b=	5.25	ft
\	Vu = Pa/L =	39448	lbs	/	L			L=	12.00	ft
	, -		/		-		* *	-	0	
Fv =			60.000	psi						
f'c =			4.500	psi Beam	Reaction =	2475	2 lbs			
Check Lid I	Flexure	Mn > Mu	.,===							
$\rho$ min =		0.00333	<== See rho	- min calcs below	/ (For Slabs rho	omin is T&S S	Steel)			Table 7.6.1.1
ρ = As / (b	*d)	0.00830	<== If p is les	s than ρ min, se	e below		,			
See Table 21	, .2.2 For Strai	n Boundaries	when (ct) is co	ompression con	trolled, transition	on or tension	controlled	ŀ		Tbl. 21.2.2
	No	Compress	ion Controll	ed If et < etv						
	No	Transition	Zone If e. <	< c. < 0.005						
	Ves		ontrolled Si	$r_{ce}(et) = or$	> 0 005					
	163			100(00) = 01	0.000					

	Date: 11/6/2023
	Job: 2310219(3) - Milner WWTP
	Subject: Concrete Lid Design
** Solve for phiMp Peeed on Whitney Stress Plack	Fig P21 2 2a
Solve for prinkin based on writiney Stress block a = stress block depth = $As^*Ev / (0.85*fc*beff) =$	1 7255 in
c = denth to neutral axis = a / B =	2.0915 in
$Mn = Nominal Moment Can = Ac^*Ev^*(d_a/2)/12 =$	2.0913 m.
	0.9 21.2.1
∲ φMn = Ultimate Moment Cap =	147161 ft -lb R10 3 5
check against o min =	0.0033 < 0 9.61.3
check against o max =	$0.0000 \leq = determine rho max eqn$
If below ρ min. multiply As x 1.33 =	3 5112 in^2 9 6 1 3
if below ρ min, multiply φMn x 0.75 =	110370 lb-ft
in. øMn x 0.7	$5 = 1 / 1.33 \times \phi Mn = reduction if \rho < \rho min)$
(note: code s	ays $\rho$ min need not be satisfied if
As > 1.33 * A	is Required, so if $\rho$ min is not met, can
simply reduce	e capacity by 1/4 or multiply x 0.75)
<i>φ</i> Mn = Ultimate Moment Cap =	<b>147160.6</b> <i>lb-ft</i>
Mu = 127375.9 lb ft > fMn - Lid flexure NO C	OOD Utilization = 0.87
4. Check Shear Capacity **neglects Vs (shear reinforcement)	
Vc = 2 * \lambda * (fc)^.5 * b * d =	42664 lbs 22.5.5.1
φ = 0.75 φVc =	= 31998 lbs 21.2
$\phi$ VC/2 = **For Ream Design (h > 10 in )	15999 lbs / ft 11 4 6 1
· · · · · · · · · · · · · · · · · · ·	
Check if shear capacity is OK without considering shear reinf	orcement
(Environmental durability factor does not apply to concrete shear	capacity) 9.2.6
Vu = Vmax (U) = 39448 lbs > fVc - Consid	der Shear Reinforcement
See Below f	or Reinforcement Check 9.2.1
Vs required = Sd (Vu-fVc) = 10377.21 lbs Shear	r reinforcement required R9.2.6.4
Vs = Av fy d / s 29150 lbs	11.5.6.2
stirrup spacing = s = 6 in	
stirrup bar area = Av = 0.11 in^2	(2 bars per stirrup in Vs calculation)
minimum spacing = 6.625	11.5.4.1
Sd = 1.26803	
Vn = Vc + Vs/Sd = 65652.59	
φ Vn = 49239.45	Utilization = 0.80
Use 16 x 24 w/	6 #6 bar in beam
and #3 stirrups@	6 in o.c.

		Date:	11/6/2023		
		Job:	2310219(3) - 1	Milner WWTP	
		Subje	ct: Concrete Lid D	Design	
<u>Beam #2 Design - Over Infl</u>	uent EQ	<u>_</u>		Reference	
				ACI 350-06	ACI 318-14
Span =	12.00 ft			8.7.3	
* Doom chons o	s shown shows				
Beam spans a	- h - 24 in				
Beam boight	- 0 - 24 III				
bediti tielgiti	- 11 - 10 III	#6 hor			
bar ulanı bar a	roa = 0.44 in A2	#0 Dai			
stirrun har diam	otor - 0.375 in	#2			
stillup bal ulain	vor - 2 in	Solf Woight of hor	m		
	d = 12.25 in		1111		
Concentrated Load from co	u - 15.25 III	400 pil	ar abovo intorior walls		
	987 5 lbc	o shims support containe	er above interior Walls		
DL - 10	/1800 lbs	v 5 #6 hav	r in heam		
LL - CI _		y J #UUd bar spacing = 4	9125 in		
5L =		uai spacifig = 4	III C210.		
VV down =	708 IUS				
VVup -	-2912 IDS				
Factored Loads * Sd (see b	2500 IDS		without Sd	0.2.1	
$(0_2)$ 1 201+1 611+0	SI 31565	1 lbc	17387 5	9.2.1	
(9-2) 1.2DL+1.6LL+0.	31305. 31846	7 lbs	17387.5		
(9-3) 1.2DL+1.03L+1.0 (9-4) 1.2DL+1.6W/+1.0	)  _10 55  31700	9 lbs	18155 5		
(9-4) 1.2DL+1.0W+1.0 (9-5) 1.2DL+1.0E+1.0	11+0.25 31367	0 lbs	17958 85		
(55) 1.200, 1.00, 1.0		0 103	0.052		
Use P = 31	846.7 lbs	Seismic	vertical component		
		00.01110			
Modification Factor used in	above load calculations				
Sd = $\phi Ev/(\gamma fs)$					
$\phi = 0.9$	(modification factor f	for tension-controlle	d section)	R9.2.6	
Fy = 60.000 psi	(Steel vield strength)		,		
$\gamma = U$ factor	(Ultimate Load Facto	r (LRFD))			
fs = 20,000 psi	(Steel direct hoop an	d tensile strength)			
		0,			
fs <sub>max</sub> = 3	20	For "Normal" e	nvironment exposure:	10.6.4.1	
B (s^2 + 4(2+d	0/2)^2)^0.5 Co	ating to be applied to	concrete surface, per		
	Aqı	uaworks, Sherwin Will	iams Dura-Plate 6000.		
B = 1.35		Therefore en	vironment is NORMAL	10.6.4.4	
db = 0.75 in	#6 bar				
s = 4.8125 in sp	acing				
fs <sub>max</sub> = 35055 psi	(for flexural stress)			10.6.4	
20000 psi	(for direct and hoop	tensile stress in norr	nal exposure)	9.2.6.2	
24000 psi	(for shear stress carri	ied by <b>shear reinford</b>	ement)	9.2.6.4	

$ \begin{array}{c c c c c c c c c c c c c c c c c c c $							Date:	11/6/2	023		
Subject: Concrete Lid Design           Reference           Sd(min) = 1.0         Sd Factor - Shearing         ACI 330-06         ACI 318-14           U factor         Sd factor - Shearing $\frac{12}{120}$ Sd Factor - Shearing $\frac{12}{120}$							Job:	2310219(3	3) - N	/lilner WWTP	
$ \begin{array}{c c c c c c c c c c c c c c c c c c c $							Subject:	Concrete	Lid D	esign	
$\begin{array}{c c c c c c c c c c c c c c c c c c c $										Reference	2
$ \begin{array}{c c c c c c c c c c c c c c c c c c c $										ACI 350-06	ACI 318-14
$\frac{df = actor - Bending}{U = factor - Bending}}{U = factor - Bending}$ $\frac{df = actor - Shear}{U = factor - Shear}$ $\frac{df = actor - Shear}{V = factor - Sh$		Sd(min) = :	1.0				_			9.2.6	
$ U actor & Sd & Sd^*U \\ \hline 1.2 & 1.16 & 1.393 \\ \hline 1.6 & 1.00 & 1.600 \\ \hline 1.4 & 1.00 & 1.400 \\ \hline 1.4 & 1.00 & 1.400 \\ \hline 1.39 & 1.393 \\ \hline 0.2 & 6.96 & 1.393 \\ \hline 0.2 & 1.125 & 2.250 \\ \hline 0.2 & 11.25 & 2.250 \\ \hline 0.2 & 12.2 & 2.250 \\ $	Sd Factor	- Bending			Sd Facto	or - Shear					
$\begin{array}{c c c c c c c c c c c c c c c c c c c $	U factor	Sd	Sd*U		U factor	Sd	Sd*U				
$\begin{array}{c c c c c c c c c c c c c c c c c c c $	1.2	1.16	1.393		1.2	1.88	2.250				
$\begin{array}{c c c c c c c c c c c c c c c c c c c $	1.6	1.00	1.600		1.6	1.41	2.250				
$\begin{array}{c c c c c c c c c c c c c c c c c c c $	1.4	1.00	1.400		1.4	1.61	2.250				
$\begin{array}{c c c c c c c c c c c c c c c c c c c $	0.5	2.79	1.393		0.5	4.50	2.250				
0.26.961.3930.211.252.250Moment at mid-spanmuPa=6.75 ftShear at supportVu = Pa/L =32279 lbsa=6.75 ftVu = Pa/L =32279 lbsa=6.75 ftFy =60,000 psifc =12.00 ftFc =4,500 psiBeam Reaction =20314 lbsCheck Lid FlexureMn > Mu0.00333 <= see p min calcs below (For Slabs pmin is T&S Steel)	1	1.39	1.393		1	2.25	2.250				
Moment at mid-span Mu = Pab/L = 104076.4 lb ft Shear at support Vu = Pa/L = 32279 lbs Fy = 60,000 psi fc = 4,500 psi Beam Reaction = 20314 lbs <u>Check Lid Flexure</u> Mn > Mu $\rho$ min = 0.00333 <== See $\rho$ min calcs below (For Slabs pmin is T&S Steel) $\rho = As / (b^*d)$ 0.00692 <== if $\rho$ is less than $\rho$ min, see below See Table 21.2.2 For Strain Boundaries when (et) is compression controlled, transition or tension controlled No Compression Controlled If et < ety No Transition Zone If $e_{p_{i}} < e_{i} < 0.005$ Yes Tension Controlled Since (et) = or > 0.005 ** Solve for phiMn Based on Whitney Stress Block $a = stress block dept = As^*Fy' (0.85°tC*beff) = 1.4379$ in. $c = depth to neutral axis = a / \beta = 0.6875$ in. Mn = Nominal Moment Cap = 124057 ftlb. $\phi = 0.99$ $\phi Mn = Ultimate Moment Cap = 124057 (t-lb.)$ $check against \rho min = 0.0003 << \rho\rho endire JUltimate Moment Cap = 2.9260 in^2g = 3043$ lb-ft in. $\phi Mn x 0.75 = 1/1.33 \times M = 2.9260$ in^2 g = 3043 lb-ft $Mu = Ultimate Moment Cap = 2.9260$ in^2 g = 3043 lb-ft $Mu = Ultimate Moment Cap = 2.9260$ in^2 g = 3043 lb-ft $Mu = Ultimate Moment Cap = 2.9260$ in^2 g = 3043 lb-ft $Mu = Ultimate Moment Cap = 2.9260$ in^2 g = 3043 lb-ft $Mu = Ultimate Moment Cap = 2.9260$ in^2 g = 3043 lb-ft Mu = 104076.4 lb ft $< f(Mn - Lid Flexure OK$ Utilization = 0.844	0.2	6.96	1.393		0.2	11.25	2.250				
Mu = Pab/L = 104076.4 lb ft Shear at support Vu = Pa/L = 32279 lbs Fy = 60,000 psi f'c = 4,500 psi Beam Reaction = 20314 lbs Check Lid Flexure Mn > Mu p min = 0.00333 <= See p min calcs below (For Slabs pmin is T&S Steel) p = As / (b*d) 0.00692 <== If p is less than p min, see below See Table 21.2.2 For Strain Boundaries when (et) is compression controlled If et < ety No Compression Controlled If et < ety No Transition Zone If $e_{iy} < e_i < 0.005$ Yes Tension Controlled Since (et) = or > 0.005 ** Solve for phiMn Based on Whitney Stress Block a = stress block depth = As*Fy / (0.85*fc*beff) = 1.4379 in. c = depth to neutral axis = a / β = 0.6875 in. Mn = Nominal Moment Cap = 124057 ftlb. $\phi^{=}$ 0.9 $\phi$ Mn = Ultimate Moment Cap = 124057 ftlb. check against p min = 0.0033 < $\rho$ check against p min = 0.0033 < $\rho$ in. $\phi$ Mn x 0.75 = 1 / 1.33 x $\phi$ Mn = reduction if p  (note: code says p min need not be satisfied if As > 1.33 * As Required, so if p min is not met, can simply reduce capacity by 14 or multiply x 0.75) $\phi$ Mn = Ultimate Moment Cap = 124057.4 [b ft] Mu = 104076.4 lb ft < ftm - Lid Flexure 0K Mu = 104076.4 lb ft < ftm - Lid Flexure 0K	Moment at	t mid-span					P	-			
Shear at support Vu = Pa/L = 32279 lbs Fy = 60,000  psi f'c = 4,500  psi Beam Reaction = 20314 lbs Check Lid Flexure Mn > Mu $p \min = 0.00333 \iff ee p \min \text{ calcs below}$ (For Slabs pmin is T&S Steel) $p = As/(b^*d)$ 0.00692 $\iff fin \text{ is less than p min, see below}$ See Table 21.2.2 For Strain Boundaries when (et) is compression controlled If et < ety No Transition Zone If $e_{ty} < e_t < 0.005$ Yes Tension Controlled If et < ety No Transition Zone If $e_{ty} < e_t < 0.005$ Yes Tension Controlled Since (et) = or > 0.005 Yes Tension Controlled Since (et) = or > 0.005 Fig R21.2.2a $a = stress block depth = As^*Fy'(0.85° fc*beff) = 1.4379 in. c = depth to neutral axis = a / \beta = 0.6875 in.Mn = Nominal Moment Cap = 124057 ftlb.\phi = 0.0000 \iff 0.0033 \iff \rhocheck against p min = 0.0033 \iff \rhocheck against p max = 10.0000 \iff 0.0000 \iff 0.00000 \iff 0.0000 \iff 0.00000 \iff 0.000000 \iff 0.00000 \iff 0.000000 \iff 0.00000000$	Mu	u = Pab/L =	104076.4	lb ft			U		a=	6.75	ft
$Vu = Pa/L = 32279 \text{ lbs} \qquad L \qquad L = 12.00 \text{ ft}$ $Fy = 60,000 \text{ psi}$ $fc = 4,500 \text{ psi} \text{ Beam Reaction} = 20314 \text{ lbs}$ $\frac{Check \text{ Lid Flexure}  Mn > Mu}{p \text{ min} = 0.00333  see \text{ See p min calcs below (For Slabs pmin is T&S Steel)}$ $p = As / (b^*d) \qquad 0.00692  see \text{ lf p is less than p min, see below}$ See Table 21.2.2 For Strain Boundaries when (ct) is compression controlled, transition or tension controlled $No  Compression Controlled If et < ety$ $No  Transition Zone If e_{ty} < e_t < 0.005$ $Yes  Tension Controlled Since (et) = or > 0.005$ $** \text{ Solve for philMn Based on Whitney Stress Block}$ $a = stress block depth = As *Fy / (0.85 *fc*beff) = 1.4379 \text{ in.}$ $c = depth to neutral axis = a / \beta = 0.6875 \text{ in.}$ $Mn = Nominal Moment Cap = As *Fy*(d-a/2)/12 = 137842 \text{ ftlb.}$ $\phi^{=} \qquad 0.9$ $\phi(Mn = Ultimate Moment Cap = 124057 \text{ ftlb.}$ $check against p min = 0.0033 < p$ $check against p max = 0.00000  see determine p max eqn$ $If below p min, multiply As x 1.33 = 2.9260 \text{ in}^{-2}$ $g(Mn = Ultimate Moment Cap = 33^{+}A3 \text{ Required, so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.75)$ $f(Mn = Ultimate Moment Cap = 33^{+}A3 \text{ Required, so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.75)$ $f(Mn = Ultimate Moment Cap = 124057 \text{ if } M \text{ is for multiply x 0.75)}$ $f(Mn = Ultimate Moment Cap = 124057 \text{ if } M \text{ is for multiply x 0.75)}$ $f(Mn = Ultimate Moment Cap = 33^{+}A3 \text{ Required, so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.75)$ $f(Mn = Ultimate Moment Cap = 33^{+}A3 \text{ Required, so if p min is not met, can simply reduce capacity by 1/4 or multiply x 0.75)$ $I 2400576 \text{ lb ft} = (Mn - \text{ lid Flexure QK} Utilization = 0.84$	Shear at su	pport		Ĺ	<u> </u>	a —	ـــــــــــــــــــــــــــــــــــــ	$\bigcirc$	b=	5.25	ft
$Fy = 60,000 \text{ psi}$ $fc = 4,500 \text{ psi} \text{ Beam Reaction} = 20314 \text{ lbs}$ $Check \text{ Lid Flexure } Mn > Mu$ $p \text{ min} = 0.00333 \iff be p \text{ min calcs below (For Slabs pmin is T&S Steel)}$ $p = \text{As } / (b^*d) \qquad 0.00692 \iff \text{If } p \text{ is less than } p \text{ min, see below}$ See Table 21.2.2 For Strain Boundaries when (et) is compression controlled, transition or tension controlled $Mo  \text{Compression Controlled If et < ety}$ $No  \text{Transition Zone If } e_{iy} < e_i < 0.005$ $Yes  \text{Tension Controlled Since (et) = or > 0.005}$ ** Solve for phiMn Based on Whitney Stress Block $a = \text{stress block depth } = \text{As "Fy } / (0.85 \text{ fc}^+\text{beff}) = 1.4379 \text{ in.}$ $c = \text{depth to neutral axis } = a / \beta = 0.6875 \text{ in.}$ $Mn = \text{Nominal Moment Cap} = 124057 \text{ ftlb.}$ $(hck against p \text{ min} = 0.0033 < p$ $(hck against p \text{ min} = 0.0033 < p$ $(hck against p \text{ min} = 0.00000 \iff \text{etermine p max eqn}$ $If below p \text{ min, multiply As x 1.33 = 2.9260 \text{ in}^{2}$ $(i. e. 90.9 \text{ gainst } p \text{ min} = 0.0000 \iff \text{statified if}$ $As > 1.33 * As Required, so if p \text{ min is not met, can}$ $simply reduce capacity by 14 or multiply x 0.75)$ $\frac{124057.4 \text{ lb ft}}{194076.4 \text{ lb ft}} \ll (fm - \text{ lid Flexure QK}$	١	/u = Pa/L =	32279	lbs	/	L			L=	12.00	ft
$\begin{array}{cccccccccccccccccccccccccccccccccccc$											
f'c =4,500 psi Beam Reaction =20314 lbsCheck Lid FlexureMn > Mu $\rho$ min =0.00333 <== See ρ min calcs below (For Slabs pmin is T&S Steel) $\rho = As / (b^*d)$ Table 7.6.1.1 $\rho = As / (b^*d)$ 0.00692 <== If ρ is less than ρ min, see belowTable 7.6.1.1See Table 21.2.2 For Strain Boundaries when (et) is compression controlled. If et < ety NoTransition Zone If $e_{ty} < e_t < 0.005$ YesTension Controlled If et < ety NoTransition Zone If $e_{ty} < e_t < 0.005$ ** Solve for phiMn Based on Whitney Stress Block a = stress block depth = As*Fy / (0.85*f'c*beff) =1.4379 in. c. edepth to neutral axis = a / β =0.6875 in. Mn = Nominal Moment Cap =Fig R21.2.2a $\phi =$ $\phi(Mn = Ultimate Moment Cap =124057 ftlb.0.0003 <PR10.3.5check against ρ min =0.00033 <<0.0000 <== determine p max eqnin. eductori if ρ (note: code says ρ min ned not be satisfied ifAs > 1.33 * As Required, so if ρ min is not met, cansimply reduce capacity by 1/4 or multiply x0.75)9.6.1.3$\pmmodel{Mn} = Ultimate Moment Cap =1240574 lb.ftItilization =0.84$	Fy =			60,000	psi						
$\frac{\text{Check Lid Flexure}}{\text{pmin} =} \frac{\text{Mn} > \text{Mu}}{0.00333 <== \text{See pmin}} \text{ calcs below (For Slabs pmin is T&S Steel)} \\ p = \text{As } / (b^*d) 0.00692 <== If \rho \text{ is less than pmin, see below}} \\ \text{See Table 21.2.2 For Strain Boundaries when (et) is compression controlled, transition or tension controlled} \\ \text{No} Compression Controlled If et < ety \\ \text{No} Transition Zone If e_{1y} < e_1 < 0.005 \\ \text{Yes} Tension Controlled Since (et) = or > 0.005 \\ \text{** Solve for phiMn Based on Whitney Stress Block} \\ a = stress block depth = \text{As*Fy} / (0.85*fc*beff) = 1.4379 in. \\ c = depth to neutral axis = a / \beta = 0.6875 in. \\ Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 = 137842 ftlb. \\ \phi = 0.9 \\ \phi \text{Mn = Ultimate Moment Cap = 124057 ftlb. \\ check against \rho min = 0.0033 < \rho \\ check against p max = 0.0000 <== determine p max eqn \\ If below p min, multiply As x 1.33 = 2.9260 in^2 \\ is 0.0000 <== detormine p max eqn \\ is 0$	f'c =			4,500	psi Beam	Reaction =	20314	lbs			
$p \min =$ $0.00333 \iff$ See p min calcs below (For Slabs pmin is T&S Steel)Table 7.6.1.1 $p = As / (b^*d)$ $0.00692 \iff$ If p is less than p min, see belowTable 7.6.1.1See Table 21.2.2 For Strain Boundaries when (et) is compression controlled, transition or tension controlledNoCompression Controlled If et < ety	Check Lid F	lexure	Mn > Mu								
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See Table 21.2.2 For Strain Boundaries when (et) is compression controlled, transition or tension controlled No Compression Controlled If et < ety No Transition Zone If $e_{ty} < e_t < 0.005$ Yes Tension Controlled Since (et) = or > 0.005 ** Solve for phiMn Based on Whitney Stress Block a = stress block depth = As*Fy / (0.85*fc*beff) = 1.4379 in. c = depth to neutral axis = a / $\beta$ = 0.6875 in. Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 = 137842 ftlb. $\phi$ = 0.9 $\phi$ Mn = Ultimate Moment Cap = 124057 ftlb. check against $\rho$ min = 0.0033 < $\rho$ check against $\rho$ max = 0.0000 <== determine $\rho$ max eqn If below $\rho$ min, multiply As x 1.33 = 2.9260 in^2 if below $\rho$ min, multiply phiMn x 0.75 = 1/1.33 x $\phi$ Mn = reduction if $\rho < \rho$ min) (note: code says $\rho$ min ned not be satisfied if As > 1.33 * As Required, so if $\rho$ min is not met, can simply reduce capacity by 1/4 or multiply x 0.75) $\phi$ Mn = Ultimate Moment Cap = 124057.4 b-ft Mu = 104076.4 lb ft < fMn - Lid Flexure OK Utilization = 0.84	ρ = As / (b*	d)	0.00692	$\leq =  f \rho $ is les	s than ρ min, s	ee below					
No Compression Controlled If et < ety No Transition Zone If $e_{ry} < e_t < 0.005$ Yes Tension Controlled Since (et) = or > 0.005 *** Solve for phiMn Based on Whitney Stress Block a = stress block depth = As*Fy / (0.85*f°c*beff) = 1.4379 in. c = depth to neutral axis = a / $\beta$ = 0.6875 in. Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 = 137842 ftlb. $\phi^{=}$ 0.9 $\phi$ Mn = Ultimate Moment Cap = 124057 ftlb. check against $\rho$ min = 0.0033 < $\rho$ check against $\rho$ min = 0.0003 < $\rho$ check against $\rho$ min = 0.0000 <== determine $\rho$ max eqn If below $\rho$ min, multiply As x 1.33 = 2.9260 in^2 if below $\rho$ min, multiply phiMn x 0.75 = 93043 lb-ft in. $\phi$ Mn x 0.75 = 1 / 1.33 x $\phi$ Mn = reduction if $\rho < \rho$ min) (note: code says $\rho$ min need not be satisfied if As > 1.33 * As Required, so if $\rho$ min is not met, can simply reduce capacity by 1/4 or multiply x0.75) $\phi$ Mu = 104076.4 lb ft < fMn - Lid Flexure OK Utilization = 0.84	See Table 21	.2.2 For Strai	n Boundaries	when (et) is co	mpression cor	ntrolled, transiti	on or tension c	ontrolled			Tbl. 21.2.2
No Compression Controlled If $et < ety$ No Transition Zone If $e_{iy} < e_i < 0.005$ Yes Tension Controlled Since $(et) = or > 0.005$ *** Solve for phiMn Based on Whitney Stress Block a = stress block depth = As*Fy / $(0.85*fc*beff) = 1.4379$ in. c = depth to neutral axis = a / $\beta$ = 0.6875 in. Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 = 137842 ftlb. $\phi^{=}$ 0.9 $\phi$ Mn = Ultimate Moment Cap = 124057 ftlb. check against $\rho$ min = 0.0033 < $\rho$ check against $\rho$ max = 0.0000 <== determine $\rho$ max eqn If below $\rho$ min, multiply As x 1.33 = 2.9260 in^2 in. $\phi$ Mn x 0.75 = 1/1.33 x $\phi$ Mn = reduction if $\rho < \rho$ min) (note: code says $\rho$ min end not be satisfied if As > 1.33 * As Required, so if $\rho$ min is not met, can simply reduce capacity by 1/4 or multiply x 0.75) $\phi$ Mn = Ultimate Moment Cap = 124057.4 [b-ft] Mu = 124076.4 [b ft] < fMn - Lid Flexure OK Utilization = 0.84											
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$ \phi Mn = Ultimate Moment Cap = 124057 \text{ ftlb.} R10.3.5 $ $ check against \rho min = 0.0033 < \rho $ $ check against \rho max = 0.0000 <== determine \rho max eqn $ $ lf below \rho min, multiply As x 1.33 = 2.9260 \text{ in}^2 $ $ if below \rho min, multiply phiMn x 0.75 = 93043 \text{ lb-ft} $ $ in. \phi Mn x 0.75 = 1/1.33 x \phi Mn = reduction if \rho < \rho min ) $ $ (note: code says \rho min need not be satisfied if $ $ As > 1.33 * As Required, so if \rho min is not met, can $ $ simply reduce capacity by 1/4 \text{ or multiply x 0.75} $ $ \phi Mn = Ultimate Moment Cap = 124057.4 \text{ lb-ft} $ $ Mu = 104076.4 \text{ lb ft} < f Mn - Lid Flexure OK Utilization = 0.84 $	φ=		·	,		0.9					21.2.1
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in. $\phi$ Mn x 0.75 = 1 / 1.33 x $\phi$ Mn = reduction if $\rho < \rho$ min) (note: code says $\rho$ min need not be satisfied if As > 1.33 * As Required, so if $\rho$ min is not met, can simply reduce capacity by 1/4 or multiply x 0.75) $\phi$ Mn = Ultimate Moment Cap = 124057.4 <i> b-ft</i> Mu = 104076.4 lb ft < fMn - Lid Flexure OK Utilization = 0.84	if below ρ n	nin, multiply	y phiMn x 0	.75 =		93043	lb-ft				
$(note: code says \rho min need not be satisfied if As > 1.33 * As Required, so if ρ min is not met, can simply reduce capacity by 1/4 or multiply x 0.75) \phi Mn = Ultimate Moment Cap = 124057.4 \ lb-ft Mu = 104076.4 lb ft < fMn - Lid Flexure OK Utilization = 0.84$	-				in.	5 = 1 / 1.33 x ¢N	/In = reduction	ifρ<ρmin)			
$As > 1.33 * As Required, so if \rho min is not met, can$ simply reduce capacity by 1/4 or multiply x 0.75) $\phi Mn = Ultimate Moment Cap = 124057.4 \ lb ft$ $Mu = 104076.4 \ lb ft < fMn - Lid Flexure OK Utilization = 0.84$					(note: code sa	iys ρ min need	not be satisfied	l if			
simply reduce capacity by 1/4 or multiply x 0.75) $\phi$ Mn = Ultimate Moment Cap = 124057.4  b-ft Mu = 104076.4  b ft < fMn - Lid Flexure OK Utilization = 0.84					As > 1.33 * As	Required, so it	fρmin is not m	et, can			
\$\phi Mn = Ultimate Moment Cap =         124057.4         Ib-ft           Mu =         104076.4         Ib ft         < fMn - Lid Flexure OK					simply reduce	capacity by 1/4	l or multiply x 0	.75)			
Mu = 104076.4 lb ft < fMn - Lid Flexure OK Utilization = 0.84	φMn = Ulti	imate Mon	nent Cap =			124057.4	lb-ft				
	Mu =	104076.4	lb ft	< fMn - Lid F	lexure OK		Utilization =	(	).84		

		Date:	11/6/2023		
		Jop:	2310219(3) - N	/lilner WWTP	
		Subject:	Concrete Lid D	esign	
				Reference	
				ACI 350-06	ACI 318-14
4. Check Shear Capacity					
**neglects Vs (shear reinforcement)					
Vc = 2 * λ * (f'c)^.5 * b * d =	4266	4 lbs			22.5.5.1
φ = 0.75	φVc = 3199	8 lbs			21.2
$\phi$ Vc/2 = **For Beam Design (h > 10 in.)	1599	9 lbs / ft			11.4.6.1
Check if shear capacity is OK without consider	ing shear reinforcement				
(Environmental durability factor does not apply to	concrete shear capacity)			9.2.6	
Vu = Vmax (U) = 32279 lbs	< Vu - Beam Shear NO GOO	D			
	See Below for Reinforce	ment Check		9.2.1	
Vs required = Sd (Vu-tVc) = 391.713	1 lbs Shear reinforcem	ient required		R9.2.6.4	
Vs = Av fv d / s 2915	0 lbs			11.5.6.2	
stirrup spacing = s =	6 in				
stirrup bar area = Av = 0.1	.1 in^2 (2 bars per	stirrup in Vs ca	alculation)		
minimum spacing = 6.625				11.5.4.1	
Sd = 1.268809					
Vn = Vc + Vs/Sd = 65638.48					
φ Vn = 49228.86		Utilization =	0.66		
Use 16 x 24	w/ 5	#6 bar	in beam		
and #3	stirrups @	6 in o.c.			
	•		= '	·   ·	
* Make both beams the same design for	construction simplicity				

Date:	11/9/2023
Job:	2310219(3) - Milner WWTP
Subject:	Container Embeds

DL =	43975	lbs
LL =	60	psf
SL =	77	psf
RL =	20	psf
Winddown	9.6	psf
Windup =	-43	psf

length		width		height
	40		8	9.5
	10		8	9.5

(provided by Aquaworks) Use Lid depth = 10 in

Assume wind acts on single container for simplicity / consistency

Wind lateral pressure: **12.8** psf (factored)

Maximum lateral load at double support location: 1520 lbs ((large container length + small container length) x height x wind pressure / 4 supports)

Consider overturning for embed forces	
Overall container moment from wind:	11552 lb ft
couple distance (embed spacing):	7 ft (Assume 6" each side to CL of support)
T = C = (M/d) =	1650.286 lbs (up or down)
Total Factored Dead Load :	13689
0.6D - W =	6563.1 lbs down (no net uplift)
Maximum Container Gravity Load:	24684 lbs

See Hilti Profis Output for embed analysis



ununu hilti oom			
Company: Address: Phone I Fax: Design: Fastening point:	 Concrete - Jun 26, 2023	Page: Specifier: E-Mail: Date:	
Specifier's comments:			
1 Input data		001553	
Anchor type and diameter:	AWS D1.1 GR. B 1/2		
Item number:	not available		



1

Anchor type and diameter:	AWS D1.1 GR. B 1/2
Item number:	not available
Effective embedment depth:	h <sub>ef</sub> = 5.000 in.
Material:	
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- -
Proof:	Design Method ACI 318-14 / CIP
Stand-off installation:	e <sub>b</sub> = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate <sup>CBFEM</sup> :	I <sub>x</sub> x I <sub>y</sub> x t = 8.000 in. x 8.000 in. x 0.500 in.;
Profile:	Square HSS (AISC), HSS3X3X.250; (L x W x T) = 3.000 in. x 3.000 in. x 0.250 in.
Base material:	cracked concrete, Custom, $f_c$ ' = 4,500 psi; h = 10.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: none or < No. 4 bar

CBFEM - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

#### Geometry [in.] & Loading [lb, in.lb]



Input data and results must be checked for conformity with the existing conditions and for plausibility! PROFIS Engineering ( c ) 2003-2023 Hilti AG, FL-9494 Schaan Hilti is a registered Trademark of Hilti AG, Schaan



Company:		Page:	2
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	Concrete - Jun 26, 2023	Date:	11/6/2023
Fastening point:			

#### 1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -32,967; V <sub>x</sub> = 2,080; V <sub>y</sub> = 0;	no	7
		$M_x = 0; M_y = 0; M_z = 0;$		

Input data and results must be checked for conformity with the existing conditions and for plausibility! PROFIS Engineering ( c ) 2003-2023 Hilti AG, FL-9494 Schaan Hilti is a registered Trademark of Hilti AG, Schaan



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Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	Concrete - Jun 26, 2023	Date:	11/6/2023
Fastening point:			

# 2 Proof I Utilization (Governing Cases)

			Design	values [lb]	Utilization		
Loading	Proof		Load	Capacity	β <sub>N</sub> / β <sub>V</sub> [%]	Status	
Tension	-		-	-	- / -	N/A	
Shear	Steel Strength		527	8,281	- / 7	ОК	
Loading		β <sub>N</sub>	β <sub>v</sub>	ζ	Utilization β <sub>N,V</sub> [%]	Status	
Combined tension and shear loads		-	-	-	-	N/A	

## 3 Warnings

• Please consider all details and hints/warnings given in the detailed report!

## Fastening meets the design criteria!



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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	Concrete - Jun 26, 2023	Date:	11/6/2023
Fastening point:			

### 4 Remarks; Your Cooperation Duties

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  regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use
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04 FOUNDATION SLAB



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						Date:	11/9	<u>)/2023</u>		
						Job:	23102	19(3) -	Milner WWT	P
						Subject:	Found	ation		
Foundatio	n Design								Referen	ce
Total weig	ht of tanks	and contain	ers / Total sl	ab area					ACI 350-06	ACI 318-14
			,							
Tank dime	nsions									
W/	1	н	Volume						1	
22 /167	L 10	7 50	2107 5 4	f+^2						
23.4107	12	7.50	2107.31	11. J						
18	12	7.50	1620.01	rt^3						
11.25	12	7.50	1012.5 1	rt^3						
				_						
	Tota	al volume =	4740.0 1	ft^3						
Large Cont	tainer:			43975	lbs					
Small Cont	tainer:			10781	lbs					
Walls										
Interior	Length	height	thickness V	Weight						
			1	0	lbs					
	12	7.50	1	13500	lbs					
	12	7.50	1	13500	lbs	Тс	otal			
	12	7 50	1	13500	lhs	40	500 lhs			
Exterior		7.50		13500	105	40.	100 103			
LALEHOI	E6 667	7 50	1	62750	lbc					
	50.007	7.50	1	63750	IDS Ibe					
	56.667	7.50	1	63750		-				
	12	7.5	1	13500	1DS	IC	otal			
	12	7.50	1	13500	lbs	1545	501 lbs			
Lid	Length	Width	Thickness							
	56.667	14	12			1190	001 lbs			
Fnd										
Slab	Length	Width	Thickness	_	1.00	) ft projecti	ion outside	of wa	lls	
(ft)	58.667	16	1.25	_		1760	001 lbs		-	
	-							(A	ssume liquid	up to bottom
Liquid	Density:	70	pcf			3318	800 lbs	-	of lid (conse	rvative))
I	1								,	,,
Live Load =	=	containers	60	nsf x	40x8 =	: 192	200 lbs			
		lid	60	osf v remainde	ar of lid	28/	100 lbs			
		Total Live L	- heo	47600	lhc	Note: Tank I	liquid acts in	onnosin	a direction to th	o soil
Snowload	4 _		uau –	47000	103	NULE. TAIKT		oppositi	g unection to th	
Show Load	ב ג ג			64007	U	pressure aga	ainst the foul	ndation	for flexural desig	in of the
	SL =	//	psr	61087	IDS	foundation.	So liquid loa	d left ou	it of equation fo	e
Wind Dow	n:					flexural desi	ign			
Pressure =	:	9.6	pst	7616	lbs					
	-					-				
		Тс	otal Load =	992862	lbs		8	76559	Total DL	
				(Conservative	ely consid	er cumulati	ive load ins	stead o	of	
Slab area =	=	938.672	ft^2	ASD Load Co	mbinatior	s)				
	Bearing pr	essure =	1057.7 (	osf		DL Pressur	e =	933.8	psf	
							-	200.0	F	
Allowable	hearing pro		2000 /	OK						
Allowable	searing pre		5000 0							

						Date:	11/9/2023		
						Job:	2310219(3) -	Milner WWT	<u>)</u>
<u>Consider F</u>	lexure of fo	oundation sla	ab_			Subject:	Foundation		
								Referenc	ce
								ACI 350-06	ACI 318-14
From RISA	3D Model,	moment in	footing from	m	781.3	psf soil pressure a	and moments at		
yields a mo	oment force	e in the slab	of:	45000	lb ft	base of wall trans	sferring into slab, tir	nes Sd	
Slab	thickness =	15		Shear					
Clear co	over (top) =	3	in	11382	lb	per RISA model	l using multi-span		
bar	diameter =	0.875	in		Simple Sp	an moment =	14063	lb ft	
	d =	11.5625	in		Simple	Span shear =	4688	lb ft	
	f'c =	4500	psi						
Factored L	oads * Sd (s	see below fo	or Sd factor	)			without Sd	9.2.1	
(9-2)	1.2DL+1.6l	L+0.5SL		1616462	lbs		694707		
(9-3)	1.2DL+1.69	SL+1.0LL		1616462	lbs		733342		
(9-4)	1.2DL+1.6	W+1.0LL+0.5	5SL	1637026	lbs		678332		
(9-5)	1.2DL+1.0	E+1.0LL+0.2	5	1731843	lbs		690554		
						0.052	2		
	Use w =	781.3	psf	1844.9932		Seismic vertio	al component		
	Factored lo	oad, without	t Sd	with Sd					
Modificatio	on Factor u	sed in above	e load calcu	lations					
Sd =	φFy/(γfs)								
φ =	0.9		(modificati	ion factor for	tension-co	ontrolled section	ion)	R9.2.6	
Fy =	60,000	psi	(Steel yield	strength)					
γ =	U factor	•	(Ultimate I	Load Factor (	LRFD))				
fs =	20,000	psi	(Steel dire	ct hoop and t	tensile stre	ngth)			
		•							
fs <sub>max</sub> =		320		_	For "N	ormal" environ	ment exposure:	10.6.4.1	
-	B (s^2 + 4	(2+db/2)^2)	^0.5	Coa	ting to be ap	oplied to concre	ete surface, per		
As =	0.6	in^2		Aqua	aworks, Shei	rwin Williams D	Oura-Plate 6000.		
B =	1.35				Ther	efore environm	nent is NORMAL	10.6.4.4	
db =	0.875	in	As =	0.6	in^2	#7			
s =	12	in spacing							
fs <sub>max</sub> =	20000	psi	(for flexura	al stress)				10.6.4	
	20000	psi	(for direct	and hoop ter	nsile stress	in normal exp	oosure)	9.2.6.2	
	24000	psi	(for shear	stress carried	l by <b>shear i</b>	reinforcemen	t)	9.2.6.4	
Sd Factor	- Bending			Sd Factor	r - Shear				
U factor	Sd		_	U factor	Sd				
1.2	2.25	2.700		1.2	1.88	2.250	)		
1.6	1.69	2.700		1.6	1.41	2.250	D		
1.4	1.93	2.700		1.4	1.61	2.250	D		
0.5	5.40	2.700		0.5	4.50	2.250	)		
1	2.70	2.700		1	2.25	2.250	)		
0.2	13.50	2.700		0.2	11.25	2.250	D		

				Date:	11/6/2023		
				Job:	2310219(3) -	Milner WW1	ГР
				Subject:	Foundation	-	
Check Slab Flexur	e Mn > Mu						
ρ min =	0.00333 <=	= See rho min calc	s below (For Slabs rh	omin is T&S Ste	eel)		Table 7.6.1.1
ρ = As / (b*d)	0.00435 <=	= If ρ is less than ρ	min, see below				
See Table 21.2.2 For	Strain Boundaries when	n (ɛt) is compressio	n controlled, transition	n or tension con	trolled		Tbl. 21.2.2
No	Compression	Controlled If et	< ety				
No	Transition Zor	the If $\epsilon_{ty} < \epsilon_t < 0$ .	005				
Yes	s Tension Contr	olled Since (ct)	= or > 0.005				
** Cable for abilde		· Otress Dissi					
Solve for philvin	Based on Whitney	Stress Block	0 7042				FIG RZ I.Z.Za
a - stress DIOCK de	eptn = As"⊢y/(0.8 al avia = a / 8 =	оо п с "bem) =	0.7843	) IN. Zim			
	aiaxis −a/p =	*/-	0.9507	in.			
	ment Cap = As*Fy	(a-a/2)/12 =	33324	· ítið.			01.0.1
$\psi - \phi M p = 1$ litimate M	omont Con -		0.9	) 1		D40.25	21.2.1
wwwoutmate.We	oment Cap =		29991	πID.		R10.3.5	0.04.0
check against p m			0.0033	<	ρ		9.6.1.3
the law a min man			0.0150	<pre>0 &lt;== determine</pre>	e rho max eqn		
if below p min, mu	1000000000000000000000000000000000000	_	0.7980	) in/2			9.6.1.3
if below ρ min, mu	litipiy philvin x 0.75	=	22493	b lb-ft	(		
		In. φMr	1 x 0.75 = 1 / 1.33 x φl	Vin = reduction i	if ρ < ρ min)		
		(note: c	ode says ρ min need	not be satisfied	IT		
		As > 1.3	33 * As Required, so i	f ρ min is not m	et, can		
		simply ı	reduce capacity by 1/4	4 or multiply x 0	.75)		
$\phi$ IVIn = Ultimate I	Noment Cap =		<u>29991.2</u>	lb-ft			
Mu (Sd) = 19	039	< fMn	- Lid Flexure OK	Utilization =	0.63	-	
		Note:					
		4500 p	osi is minimum con	crete strength	for concrete	Table 4.3.1	
		with lo	w permeability (co	oncrete is coat	ted		
		is coat	ted as directed by <i>i</i>	Aquaworks)			
Check a balanced	and et- 005 limits						
$\beta = Max of (0.85-)$	0.05*(f'c-4000)/10	00 0 65)) =	በ	)		10 2 7 3	
etv = tension vielo	1 strain = fv / Fs		0.0230	, <== Fs = 79	i 000 000 nsi	R10 3 2	
ec = balanced con	crete strain =		0.0021	)	,000,000 p3i	R10 3 2	
et = actual tension	h strain = ((d-c) / c)	*strain sc	0.0055			R10 3 3	
		Strain Sc	0.0055			110.5.5	
ρ temp = temp an	nd shrink steel =	0.0020 <== if I	Fy<60,000 psi, .002	e-or0018 * 6	60,000psi / Fy	Table 7.12.2.	1
As(temp)= (ρ tem	p * beff * h) =	0.0608 in^2					
ρ bal = (0.85*f'c*l	B/Fy)*(0.003/(0.00	)3+etv)	0.0311	. <== balance	ed steel ratio	R10.3.3	
0.75(ρ bal)=	. ,,	,,	0.0233	}		R10.3.5	
$\rho t = (.85*B*f'c / F$	v)*(0.003/(0.008))	1	0.0197	<== Max reinf. I	Ratio, when steel stra	in >= .005	
As $(max) = (ot * b)$	eff * d) =		0.5991	. in^2	-,	R10.3.5	
· · · · · · · · · · · · · · · · · · ·	- /		0.0001	·· –		1	1

				Date:	11/9/2023		
				Job:	2310219(3) -	Milner WW1	ГР
				Subject:	Foundation		
Check Slab Flexure	e Mn > Mu						
ρ min =	0.00333 <=	= See rho min calcs	below (For Slabs rh	omin is T&S Ste	eel)		Table 7.6.1.1
$\rho$ = As / (b*d)	0.00432 <=	= If ρ is less than ρ i	min, see below				
See Table 21.2.2 For S	Strain Boundaries whe	ι (εt) is compressior	n controlled, transition	n or tension con	trolled		Tbl. 21.2.2
No	Compression	Controlled If ct <	< etv				
No	Transition Zor	e lf e₊. < e₊ < 0.(	005				
Yes	Tension Contr	olled Since (et)	= 0r > 0.005				
100			0.000				
** Solve for phiMn	Based on Whitney	/ Stress Block					Fig R21.2.2a
a = stress block de	epth = As*Fy / (0.8	35*f'c*beff) =	0.7843	l in.			
c = depth to neutra	alaxis =a/β=		0.9507	' in.			
Mn = Nominal Mor	ment Cap = As*Fy	ʻ(d-a/2)/12 =	33511	ftlb.			
φ=			0.9	)			21.2.1
φMn = Ultimate Mo	oment Cap =		30160	ftlb.		R10.3.5	
check against ρ m	in =		0.0033	<	ρ		9.6.1.3
check against ρ m	ax =		0.0150	<== determine	e rho max eqn		
If below ρ min, mu	Itiply As x 1.33 =		0.7980	) in^2			9.6.1.3
if below $\rho$ min, mu	ltiply phiMn x 0.75	=	22620	) lb-ft			
		in. φMn	x 0.75 = 1 / 1.33 x of	Vn = reduction i	if ρ < ρ min)		
		(note: co	ode says $\rho$ min need	not be satisfied	if		
		As > 1.3	3 * As Required, so i	f ρ min is not m	et, can		
		simply re	educe capacity by 1/2	4 or multiply x 0	.75)		
$\phi$ Mn = Ultimate N	Moment Cap =		<mark>30159.9</mark>	lb-ft			
Mu (Sd) = 140	063	< fMn -	Lid Flexure OK	Utilization =	0.47		
		Note:					
		4500 p <sup>2</sup>	si is minimum con	crete strenøth	for concrete	Table 4 3 1	
		with lo	w permeability (cc	oncrete is coat	red		
		is coat	ed as directed by A	Aquaworks)			
		10 0000		(quarterno)			
Check $\rho$ balanced	and et=.005 limits	:					
β= Max of ((0.85-0	0.05*(f'c-4000)/10	00,0.65)) =	0.8250	)		10.2.7.3	
ety = tension yield	strain = fy / Es		0.0021	. <== Es = 29	,000,000 psi	R10.3.2	
ec = balanced con	crete strain =		0.0030	)		R10.3.2	
et = actual tension	strain = ((d-c) / c)	*strain sc	0.0055	5		R10.3.3	
0 temp - temp an	d shrink steel -	0 0020 ~ ;f r		-or- 0010 * 4	50 000nci / Ev	Table 7 12 2	1
As(temp) = (o tem)	$a \sinh \pi k \operatorname{steel} =$	0.0608 in^2	y<00,000 psi, .002	010018 (	50,000psi / Fy		Ì
( F) (F -2)	/						
ρ bal = (0.85*f'c*β	3/Fy)*(0.003/(0.00	3+ety)	0.0311	. <== balance	ed steel ratio	R10.3.3	
0.75(ρ bal)=			0.0233	}		R10.3.5	
$\rho t = (.85^{*}\beta^{*}f'c / F')$	y)*(0.003/(0.008))		0.0197	<== Max reinf. I	Ratio, when steel stra	in >= .005	
As (max) = (ρt * beff * d) =			0.5991 in^2			R10.3.5	

		Date:	11/9/2023		
		Jop:	2310219(3) -	Milner WWT	Р
		Subject:	Foundation		
				Referen	се
				ACI 350-06	ACI 318-14
ρ min (b) = 200/fy=	0	.0033			9.6.1.2
ρ min (a) = 3*(f'c)^0.5/Fy=	0	.0034		10.5.1	9.6.1.2
$\rho$ min.= greater of $\rho$ min(a) or $\rho$ min(b) =	0	.0034 <== Min. re	einf. Ratio		
As (min)= ( $\rho$ min * beff * d) =		0.5 in.2 <== unless	As > 1.33*As reqd per	code	9.6.1.3
4. Check Shear Capacity					
**neglects Vs (shear reinforcement)					
Vc = 2 *	-	18615 lbs			22.5.5.1
φ = 0.75	φVc =	13961 lbs			21.2
	I				
Check if shear capacity is OK without considering	shear reinforcemen	t			
(Environmental durability factor does not apply to conc	rete shear capacity)			9.2.6	
Vu = Vmax (U) = 4688 lbs	< fVc - Slab Shear OK	Utilization =	0.34		
Vu = wl/2	See Below for Bein	forcement Check		921	
	See Delow for Rein			5.2.1	
Vs required = Sd (Vu-fVc) = $-20866.3$	lbs - No shear rei	inforcement requ	ired	R9264	
200003					
Use 18 in slab w/	#7	@ 12	in spacing		

Note: Calculation takes into account the sloping floor of the sludge tank, which, at its lowest point is 3" below the highest point Therefore, 'd' = 15 inches, but the FND slab thickness is 18 inches everywhere else - so the design is good for the entire slab

2	Sd Mu =	= 45000 ll	o ft				
¢	) Mn = A	sFy(d-a/2) =	402132 lb	ft	Slab F	lexure OK	
		As =	0.6 in	^2		0.6 #7 bars	
		Fy =	60,000 ps	i			_
		f'c =	4500 ps	i			
		b =	12 in				
		a =	0.784314 in				
Check Slab sh	ear	¢Vn>Vu+Vs∕S	Sd				
Vc = 2 * (f'c)^	(0.5) *b	)*d					
	φ =	0.75					
φVc =	13961	lbs	Sla	ab Shear	ОК		
Use	18	in slab w/		#7 bars	at	12	on center

05 BUOYANCY



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				Date	11/9/2023	Sheet No.	of
				Project	Steamboat Mour	ntain School	
				Subject	Bouyancy Calc		
CHECK CONCRETE BUOYANT DEADMAN FOR BURIED TANK							References
ASSUMPTIONS:							
TANK IS EMPTY			$\searrow$	_ / /	RE: PLAN FOR REINFORCEMENT		
TANK IS COMPLETELY SUBMERGED			$\rightarrow$ /	$\times //$	~9" THICK 이 아이 것		
NO SLAB						<u>↑ T.O.C. EL.</u>	
NO SLAD				el ··		50 	
		#5		$  \land \rangle$			
GIVEN INFO.	500000 11	10*	0.C. E.F.	$\boldsymbol{k}$	LONGITUDINAL (N-S)		
DRY WEIGHT OF TANK:	582333 Ibs	( cc			-RE: PLAN FOR		
ASSUMED WEIGHT OF SOIL:	110 pcf			T	REINFORCEMENT		
WEIGHT OF REINFORCED CONCRETE:	150 pcf		2" CLR	, 2" CLR			
DENSITY OF fluid:	62.4 pcf	#		+	#5_DOWELS AT	<u>م</u>	
DIMENSIONS OF TANK:					12" O.C. E.F. WITH 2'-6" HOOK INTO		
Length:	56.67 ft	1/2			FOOTING		
Round (R) or Square (S)	S			ri,			
Width:	14.00 ft	Ĺ			—6" WIDE RIBBED WATERSTOP, GREEN		
Height:	7.50 ft		/ 15 E		EQUIVALENT		
Diameter:	10.00 ft	/		K L		+ +	
	0.00 in	(1	1 23 5	J			
	0.00 in	(-		7 1	<u>/</u>		
DEPTH OF SUIL ABOVE DEADIVIAN:	0.00 m	```````````````````````````````````````			RE: PLAN FOR REINF.		
(used min. depth with no traffic)			TIP. 1'-0" 1'-0"		-3 1/2"x1 1/2" CONT. KEYWAY		
ASSUMED SLAB DEPTH:	0.00 in			1			
ASSUMED BUOYANT DEADMAN WIDTH:							
Assumed Deadman Width:	0.00 ft	EVEN IF TH	E SOIL IS COMPLETER	Y SATURA	TED TO THE TOP	OF THE TANK,	
DEPTH OF SOIL ABOVE WATER TABLE:	0.00 in	DEADMAN	NOT REQUIRED. WE	IGHT OF TA	ANK (FOOTING, W	/ALLS, LIDS,	
			DING SOIL, AND DL O	F METAL B	BUILDING) EXCEED	S WT OF EQUA	L.
			F WATER (CELL C55)				
DETERMINE VOLUMES:		//					
	0	* Insert "0" if tank volume is t	to be calculated from	"Given In	fo" above. Otherv	wise, insert kno	own volume
TANK + FOUNDATION: Calculated Value:	5908 79 ft <sup>3</sup>	*This cell can be overridden i	f tank volume is know	vn (should	l include foundatio	on volume)	
	5000.75 ft	inis cen can be overnodern		wii (Siloulu		on volume)	
SOIL: Tank + Fan Volume:	5908.793 π						
above tank:	0.00 ft <sup>3</sup>						
buoyant soil above deadman:	0.00 ft <sup>3</sup>						_
dry soil above deadman:	0.00						
4" SLAB:	,						
above tank:	0.00 ft <sup>3</sup>						
	0.00			_			
	F000 70 (4 <sup>3</sup>	—/					
TOTAL VOLOWE (TANK+FND):	5908.79 10	7358.04 IL (IHIS IS IF	TE TUTAL VOLUIVIE U	FSISIEIVI			
		RECHECK					
DETERMINE WEIGHTS:							
TANK:	582332.525 lbs						
SOIL (wt. of soil above tank x soil volume):	166616 lbs						
4" SLAB (wt. of concrete x volume):	0.00 lbs						
TOTAL WEIGHT:	748948.0843 lbs						
Distance from grade to water table:	3 ft						
WEIGHT OF EQUAL VOLUME OF WATER:							
(density of water x total volume)	367259.4332 lbs						
()							
DIFFERENCE OF BLIOYANT FORCES AND GRAVITY FORCES							
(wt. of ogual volume of water - total weight)	291699 6511 lbc						
Poquired Electric Sofety Eactor (SE):	1 25	[ACI 250 4P-04 2 1 2]					
A stud Cofety Faster (Tatal Meight (Ducurant Fanal))	1.25						
Actual Safety Factor (Total Weight/Buoyant Force):	2.04	OK, DEADMAN NOT REQUIR					
BUOYANT WEIGHT OF CONCRETE:	87.6 pcf						
(wt. of concrete - density of water)							
BUOYANT WEIGHT OF SOIL ABOVE DEADMAN:	47.6 pcf						
(wt. of soil - density of water)							
WEIGHT OF DRY SOIL ABOVE DEADMAN:	110 pcf						
(wt. of soil - density of water)	•						
WEIGHT OF BUOYANT + DRY SOIL ABOVE DEADMAN	0 lbs						
RESULTANT FORCE	-381688.6511 lbs						

(force difference-weight of buoyant soil)

#### WEIGHT CALCULATION (EMPTY TANK)

FOUNDATION:	THICKNESS LENGTH WIDTH EXTENSION AREA WEIGHT	18 IN 56.667 FT 14 FT 1 FT 938.672 SQ FT 150 PCF	211.2012 KIPS
WALLS	HEIGHT THICKNESS LENGTH	7.5 FT 12 IN 173.333 FT	194.9996 KIPS
CONC BEAMS	LENGTH DEPTH WIDTH	24 FT 4 IN 24 IN	2.4 KIPS
LIDS	THICKNESS AREA	12 IN 793.338 SQ FT	119.0007 KIPS
CONTAINERS	DL DL	43.95 KIPS 10.781 KIPS TOTAL	43.95 KIPS 10.781 KIPS 582.3325 KIPS

Soil weight Calculation.	Assu	Assume 2:1 soil slope acting on footing				
west wall: length	14					
height	7.5					
avg width	2.875	301.875 ft^3				
east wall length	32					
height	7.5					
avg width	2.875	711.5625 ft^3				
_						
north wall length	56.667					
height	7.5					
avg width	2.875	1221.8822 ft^3				
south wall length	56.667					
height	7.5					
avg width	2.875	1265.0072 ft^3				
	Tatal					
	TOTAL ADI	Ine = 3500.3269 It.3				
	_					
soil weight 110 pc	f	Total Weight - 385036 lbs				
huovant soil weight 47.6 pc	-1 -f	166615 6 lbs				
buoyant son weight 47.0 pt	.1	100013.0 105				



(using total soil weight) (using bouyant soil weight)