wallace
design
collective

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

STRUCTURAL CALCULATIONS

COLORADO LICENSED
DANIEL E. MAZZEI
PE.0058764
PROFESSIONAL ENGINEER

2023.11.13 16:48:19-06'00'

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

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TABLE OF CONTENTS

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01 DESIGN LOADS

wallace design collective

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) Wastewater Treatment Tanks in Various Towns -- Various Towns,

BY: PHONE X VISIT OTHER TIME:

ITEM	DESCRIPTION	RESPONSE	
1. GOVERNING CODE			
A.	Local Building Code:	2018	IBC -- International Building Code
B.	Local Amendments:	Yes	
C.	Do State Building Code Requirements Differ?	Yes	
D.	Observations Required to be performed by EOR?	No	
E.	Special Inspections Final Report Required for Certificate of Occupancy?	No	
2. ROOF LIVE LOAD			
A.	Minimum Roof Live Load:	20 psf	
3. SNOW LOAD			
A.	Ground Snow Load, Pg:	77 psf	
B.	Minimum Roof Snow Load, Pf:		
4. WIND LOAD			
A.	Design Wind Speed:	112 mph	
B.	Risk Category	III	
5. SEISMIC LOAD			
A.	Mapped Spectral Response Acceleration, Ss:	0.300	(short period, 0.2s)
B.	Mapped Spectral Response Acceleration, S1:	0.055	(long period, 1.0s)
6. FROST DEPTH			
A.	Minimum Bearing 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.

CC:

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tulsa, oklahoma 74103
918.584.5858 · 800.364.5858
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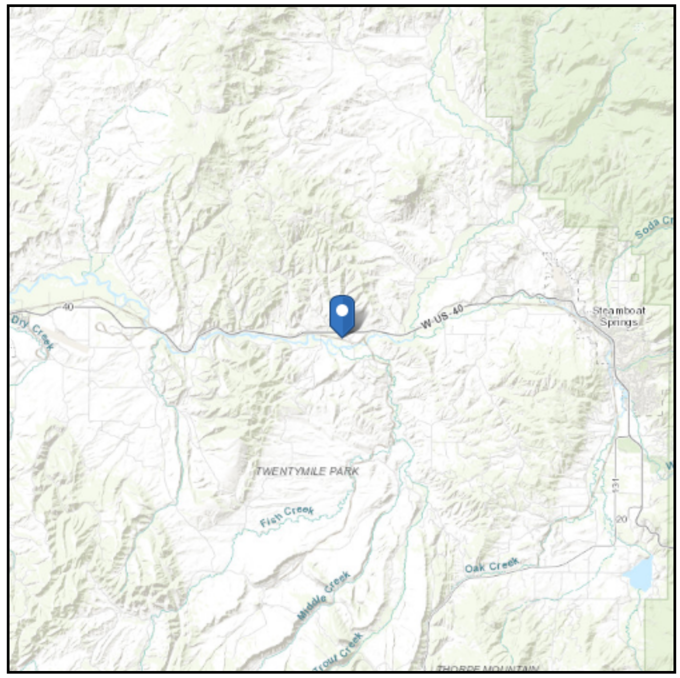
ASCE 7 Hazards Report

Address:
No Address at This Location

Standard:	ASCE/SEI 7-16	**Latitude:**	40.482229
Risk Category:	III	**Longitude:**	-107.020721
Soil Class:	C - Very Dense Soil and Soft Rock	**Elevation:**	0 ft (NAVD 88)



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).

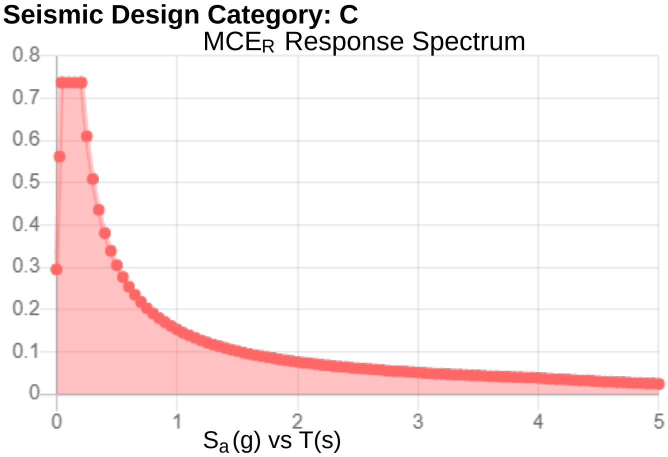
Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

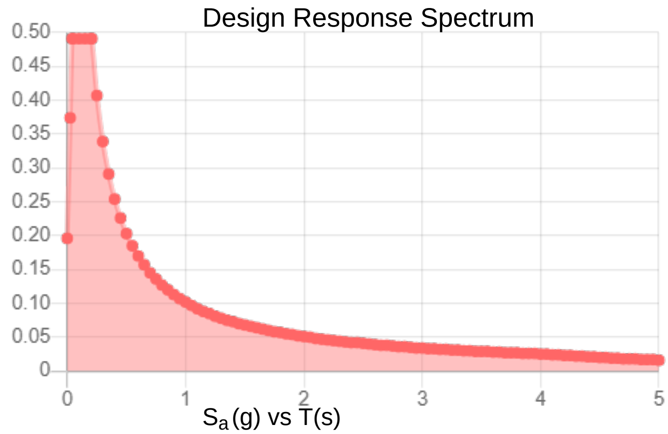
Site Soil Class:

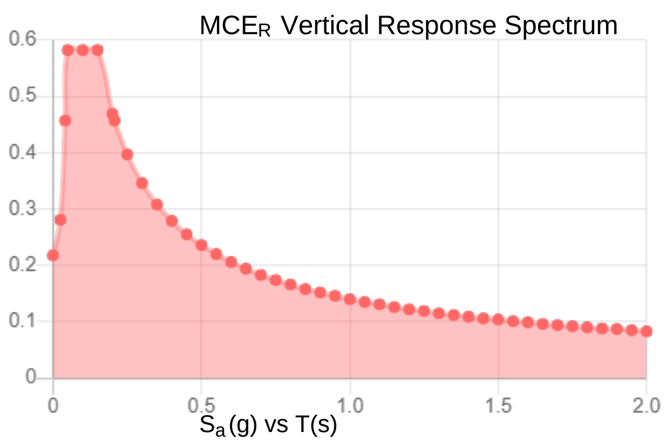
Results:

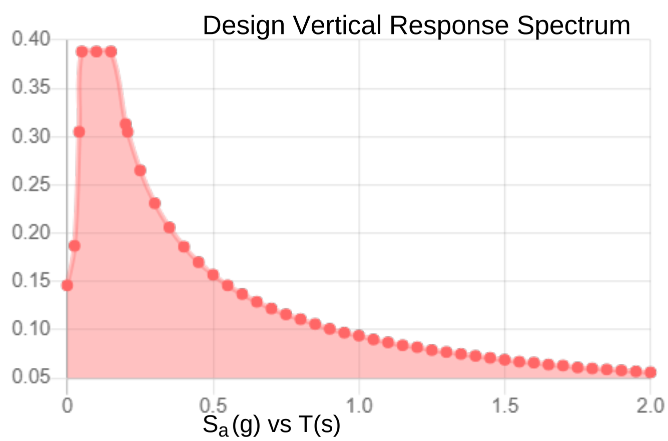
S_S :	0.581	S_{D1} :	0.102
S_1 :	0.102	T_L :	4
F_a :	1.267	PGA :	0.405
F_v :	1.5	PGA_M :	0.486
S_{MS} :	0.737	F_{PGA} :	1.2
S_{M1} :	0.153	I_e :	1.25
S_{DS} :	0.491	C_v :	0.988

Seismic Design Category: C









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.

Snow

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.

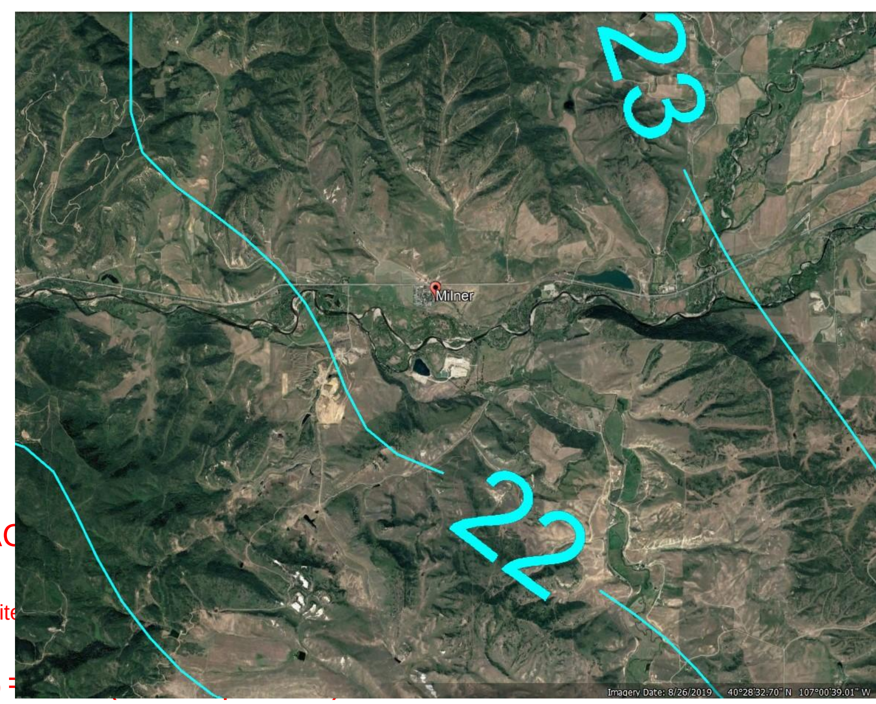
Ground snow load determination for such sites shall be based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2 percent annual probability of being exceeded (50-year mean recurrence interval).

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). [Colorado Design Snow Loads](). Structural Engineers Association of Colorado.

Snow load values are mapped to a 0.5 mile resolution. This 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.

NOTE:
Routt County Snow Mapper yields value of 77 psf for snow load at this location



Use 77 psf

SEAC

pg, site

.8 psf

k(site) =

A = 6.5 (thousands of feet in 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.

Snow

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).

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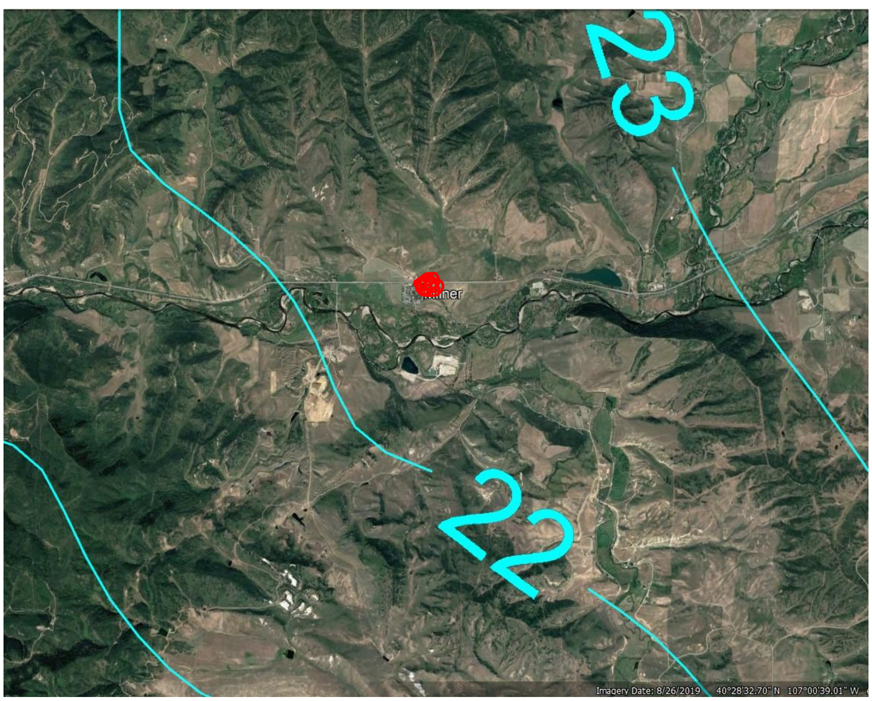
NOTE:
Routt County Snow Mapper yields value of 77 psf for snow load at this location

SEAC Snow Load Committee Equation:

$p_{g,\,site} = \max(k_{(site)}/100 * A^3)$ = $22.5/1000 * 6.5^3 =$ **61.8 psf**

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

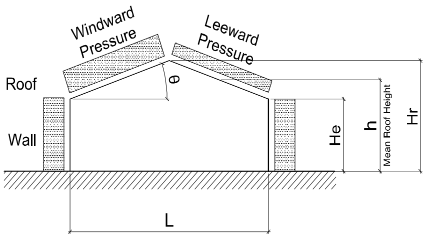


Use 77 psf

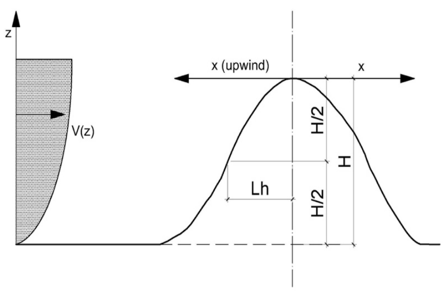
WALLACE DESIGN PROGRAM
Revised: 02/12/2019
Author: Katie Faulkner

Date	11/6/2023	Sheet	of
Job	Milner - 2310219		
Subject	Wind Loads (Container)		

WIND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD
ASCE 7-16, Chapters 26, 27 and 30



REFER TO FIGURE 27.3-1



2-D Ridge or Axisymmetrical Hill

REFER TO FIGURE 26.8-1

1. Input

Design Parameters

Basic Wind Speed, V =	**115**	mph (Section 26.5, Fig. 1A-2D)
Exposure Category (B, C, or D) =	**C**	(Section 26.7)
Building Risk Category (I, II, III, IV) =	**III**	(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 =	**9.50**	feet
Max Building Height or Ridge Height above ground level, Hr =	**9.50**	feet
Parapet Height above ground level, Hp =	**9.50**	feet
Building Width Perpendicular to Wind, B =	**40.00**	feet (max bldg dim)
Building Width Parallel to Wind, L =	**8.00**	feet
Enclosure Classification =	**Enclosed Buildings**	(Section 26.12)
Roof Configuration =	**Gabled, Hipped or Monoslope Roofs (ø ≤ 7)**	
Angle of Plane of Roof From Horizontal, θ =	**0.50**	degrees
Is building on or near a hill, ridge, or escarpment?	**N**	(Y or N) (Section 26.8)
Height of Hill or Escarpment relative to upwind terrain, H =	**10.00**	feet (Section 26.8, Fig. 26.8-1)
Horiz. Dist. Upwind to Point Where Elevation = H/2, Lh =	**10.00**	feet (Section 26.8, Fig. 26.8-1)
Horiz. Dist. from Crest to Building Site, x =	**10.00**	feet (Section 26.8, Fig. 26.8-1)
2D Ridge, 2D Escarpment, or Axisymmetrical Hill =	**E**	(R, E, or H)
Is the building site upwind or downwind of the crest?	**DOWN**	(up, down)

2. Calculations - Main Wind Force Resisting System

Equivalent Allowable Stress Design Wind Speed, Vasd =	**89.08**	mph (IBC 2018, 1609.3.1)	
Mean roof height, h =	**9.50**	feet	
Kz, velocity pressure exposure coefficient at hz = 9.5ft =	**0.85**	Table 26.10-1	(use with qz)
Kz, velocity pressure exposure coefficient at hh = 9.5ft =	**0.85**	Table 26.10-1	(use with qh)
Kz, velocity pressure exposure coefficient at hp = 9.5ft =	**0.85**	Table 26.10-1	(use with qp)
Kzt,topographic factor at hz = 9.5ft =	**1.00**	Figure 26.8-1	(use with qz)
Kzt,topographic factor at hh = 9.5ft =	**1.00**	Figure 26.8-1	(use with qh)
Kzt,topographic factor at hp = 9.5ft =	**1.00**	Figure 26.8-1	(use with qp)
Kd, wind directionality factor =	**0.85**	Table 26.6-1	
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 hp = 9.5ft =	**19.33**	psf (Eq. 26.10-1)	

Walls: P = q(GCpf-GCpi) Eqn. 27.3-1	**qz**	**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		-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 psf	2.1 psf

(1.0)W = (1.0)(Windward + Leeward Pressure) = **13.15 psf + 8.22 psf = 21.4 psf**
(0.6)W =(0.6)(Windward + Leeward Pressure) = **7.89 psf + 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 psf	-11.6 psf
Windward + Leeward Pressure	19.33	2.50	**48.3 psf**	**29 psf**

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 psf**	**-9 psf**

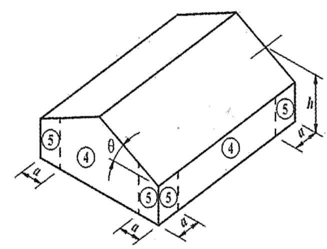
Roof All Other Conditions	**qh**	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
For 0 to h/2 = 0 ft to 4.75 ft	19.33	-1.09	0.18	**-24.6 psf**	**-14.8 psf**
>h/2 ft	19.33	-0.60	0.18	**-15 psf**	**-9 psf**
	19.33			**#VALUE!**	**#VALUE!**
	19.33			**#VALUE!**	**#VALUE!**

Roof Overhangs Section 27.3.3	**qh**	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
Maximum pressures	19.33	-1.11	0.68	**-34.5 psf**	**-20.7 psf**

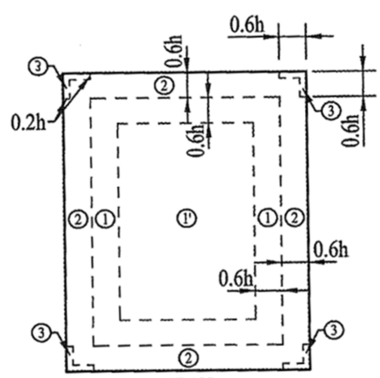
WALLACE DESIGN PROGRAM
Revised: 02/12/2019
Author: Katie Faulkner

Date	11/6/2023	Sheet	of
Job	Milner, CO		
Subject	Wind Loads		

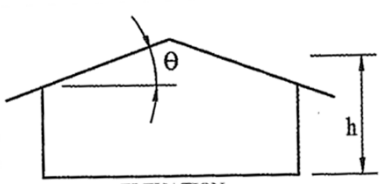
WIND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD
ASCE 7-16, Chapters 27 and 30



ELEVATION



PLAN



ELEVATION

REFER TO FIGURE 30.3-2A

3. Input - Component and Cladding Elements

Tributary Area for Wall Components, 1 =	**10.00** square feet
Tributary Area for Wall Components, 2 =	**65.00** square feet
Tributary Area for Parapet Components, 1 =	**10.00** square feet
Tributary Area for Parapet Components, 2 =	**50.00** square feet
Tributary Area for Roof Components, 1 =	**10.00** square feet
Tributary Area for Roof Components, 2 =	**100.00** square feet
Tributary Area for Overhangs or Canopies, 1 =	**10.00** square feet
Tributary Area for Overhangs or Canopies, 2 =	**50.00** square feet

4. Calculations - Component and Cladding Elements

Kh, velocity pressure exposure coefficient at hh = 9.5ft =	**0.85**	Table 26.10-1	(use with qh)
Kh, velocity pressure exposure coefficient at hp = 9.5ft =	**0.85**	Table 26.10-1	(use with qp)
Kzt,topographic factor at hh = 9.5ft =	**1.00**	Figure 26.8-1	(use with qh)
Kzt,topographic factor at hp = 9.5ft =	**1.00**	Figure 26.8-1	(use with qp)
Kd, wind directionality factor =	**0.85**	Table 26.6-1	
Ke, ground elevation factor at	**1.00**	Table 26.9-1	
G, gust factor =	**0.85**	Section 26.11.4	
qh, velocity pressure at hh = 9.5ft =	**24.45**	psf (Eq. 26.10-1)	
qp, velocity pressure at hp = 9.5ft =	**24.45**	psf (Eq. 26.10-1)	

Walls: trib. Area = 10 sq. ft.		qh	GCp	GCpi	(1.0)P	(0.6)P
Zone 4	Interior Zone	24.45	-0.99	0.18	**-28.6 psf**	**-17.2 psf**
Zone 5	End Zone	24.45	-1.26	0.18	**-35.2 psf**	**-21.1 psf**
Zone 4 and 5		24.45	0.90	-0.18	**26.4 psf**	**15.8 psf**

Walls: trib. Area = 65 sq. ft.		qh	GCp	GCpi	(1.0)P	(0.6)P
Zone 4	Interior Zone	24.45	-0.86	0.18	**-25.5 psf**	**-15.3 psf**
Zone 5	End Zone	24.45	-1.00	0.18	**-28.9 psf**	**-17.3 psf**
Zone 4 and 5		24.45	0.77	-0.18	**23.2 psf**	**13.9 psf**

Parapets: trib. Area = 10 sq. ft.			qp	GCp	GCpi	(1.0)P	(0.6)P
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**
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**

Parapets: trib. Area = 50 sq. ft.			qp	GCp	GCpi	(1.0)P	(0.6)P
Case A	Zone 4	Interior Zone	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 psf**	**24.5 psf**
	Zone 5	End Zone	24.45	1.83	0.00	**44.7 psf**	**26.8 psf**

Roofs: trib. Area = 10 sq. ft.	qh	GCp	GCpi	(1.0)P	(0.6)P
Corner Zone (3)	24.45	-3.20	0.18	**-82.6 psf**	**-49.6 psf**
End Zone (2)	24.45	-2.30	0.18	**-60.6 psf**	**-36.4 psf**
Interior Zone (1)	24.45	-1.70	0.18	**-46 psf**	**-27.6 psf**
Interior Zone (1')	24.45	-0.90	0.18	**-26.4 psf**	**-15.8 psf**
Positive (All Zones)	24.45	0.30	-0.18	**16 psf**	**9.6 psf**

Roofs: trib. Area = 100 sq. ft.	qh	GCp	GCpi	(1.0)P	(0.6)P
Corner Zone (3)	24.45	-2.14	0.18	**-56.7 psf**	**-34 psf**
End Zone (2)	24.45	-1.77	0.18	**-47.7 psf**	**-28.6 psf**
Interior Zone (1)	24.45	-1.29	0.18	**-35.9 psf**	**-21.5 psf**
Interior Zone (1')	24.45	-0.90	0.18	**-26.4 psf**	**-15.8 psf**
Positive (All Zones)	24.45	0.20	-0.18	**16 psf**	**9.6 psf**

Overhangs: trib. Area = 10 sq. ft.	qh	GCpn	(1.0)Pp	(0.6)Pp
Corner Zone (3)	24.45	-3.20	**-78.2 psf**	**-46.9 psf**
End Zone (2)	24.45	-2.30	**-56.2 psf**	**-33.7 psf**
Interior Zone (1)	24.45	-1.70	**-41.6 psf**	**-24.9 psf**
Interior Zone (1')	24.45	-1.70	**-41.6 psf**	**-24.9 psf**
---	---	---	**---**	**---**

Overhangs: trib. Area = 50 sq. ft.	qh	GCpn	(1.0)Pp	(0.6)Pp
Corner Zone (3)	24.45	-2.34	**-57.1 psf**	**-34.3 psf**
End Zone (2)	24.45	-1.81	**-44.2 psf**	**-26.5 psf**
Interior Zone (1)	24.45	-1.63	**-39.9 psf**	**-23.9 psf**
Interior Zone (1')	24.45	-1.63	**-39.9 psf**	**-23.9 psf**
---	---	---	**---**	**---**

a, end zone width = Min. of 10% L and .4H but not < 4% L or 3' = **3.0** feet (Fig. 30.3-1)

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)

WALLACE DESIGN PROGRAM
Revised 12/27/18, Sheila Butcher

Date	11/9/2023	Sheet No.	of
Job	2310219 (3) Milner		
Subject	Seismic Loads - Concrete Tanks		

SEISMIC LOAD SUMMARY

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

1. Input

Spectral Response Acceleration for Short Periods, Ss =	**0.300**	
Spectral Response Acceleration for 1-second Periods, S1 =	**0.055**	
Risk Category =	**III**	(IBC Table 1604.5 & ASCE: Table 1-5-1)
Site Classification (A,B,C,D,E,F) =	**C**	(ASCE 7 Ch.20 Table 20.3-1)

Basic Structural System	**BEARING WALL SYSTEMS**	(Table 12.2-1)
Lateral Force Resisting System	**Ordinary Reinforced Concrete Shear Walls**	(Table 12.2-1)
ρx , redundancy in x-dir.= (Redundancy is either 1.0 or 1.3)	**1.00**	(ASCE 7 Section 12.3.4)
ρy , redundancy in y-dir.= (Redundancy is either 1.0 or 1.3)	**1.00**	(ASCE 7 Section 12.3.4)

r, =1.0 for Seismic Design Category B and C, RE: ASCE 7 Section 12.3.4.1 for additional exceptions.

Is Structure regular with a period < .5 sec?	**Yes**	(Yes or No, ASCE 7 Section 12.8.1.3)
Is Structure short period with a rigid diaphragm?	**No**	(Yes or No, ASCE 7 Section 11.6)
Is Structure short period w/ non-rigid diaphragm & vertical elements of seismic force-resisting system 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 rigid diaphragm)
Span length of flexible diaphragm -y-dir. =	**14.5**	feet

2. Determine Design Spectral Response Accelerations and Seismic Design Category, Section 11.6:

Response Modification Factor, R =	**4**	(Table 12.2-1)
Overstrength Factor, Ωo = (refer to footnote b for .5 reduction for Flexible Diaphragms)	**2.5**	(Table 12.2-1)
Deflection Amplification Factor, Cd =	**4**	(Table 12.2-1)

Acceleration for Short Period		
Site Coefficient, Fa =	**1.30**	(IBC Table 1613.2.3(1), ASCE 7 Table 11.4-1)
Site Adjusted Spectral Response Acceleration for Short Periods, Sms =	**0.390**	(IBC Section 1613.2.3, ASCE 7 Section 11.4.4)
Acceleration for 1-Second Period		
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 Short Periods, Sds =	**0.260**	(IBC Section 1613.2.4 and ASCE 7 Section 11.4.5
Seismic Design Category based on short period =	**B**	
Design Spectral Response Acceleration for 1-second Periods, Sd1 =	**0.055**	(IBC Section 1613.2.4 and ASCE 7 Section 11.4.5
Seismic Design Category based on 1-second period =	**A**	

Design Response Spectrum, Ts =	**0.212**	seconds (Section 11.4.6)
Approximate Fundamental Period, Ta =	**0.500**	seconds (Section 12.8.2.1)
Fundamental Period, T, shall not exceed Ta * Cu =	**0.850**	seconds (Section 12.8.2)
Can the Seismic Design Category be based on the short period alone?	**No**	(IBC Section 1613.2.5.1, ASCE 7 Section 11.6)

Seismic Design Category =	**B**	(Most severe case except as allowed by Sect 11.6

3. Seismic Base Shear for the Lateral Force Resisting System using the Equivalent Lateral Force Procedure, Section 12.8:

a. Calculation of Seismic Base Shear Coefficient:

Seismic Importance Factor, I_e =	**1.25**	(ASCE 7 Table 1.5-2)
Cs = (Sds/(R / I_e)) =	**0.081**	(ASCE Equation 12.8-2, Section 12.8.1.3)

	Strength (1.0E)	ASD (0.7E)
b. Seismic Base Shear, Section 12.8.1:		
V = Cs W =	**0.081 W**	**0.057 W**

	Strength (1.0E)	ASD (0.7E)
c. Horizontal Seismic Load, Section 12.4.2.1=		
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 12.4.2.2:		
Ev = 0.2 Sds D =	**0.052 D**	**0.036 D**

For structures in SDC B and for the design of foundations using ASD, Ev may be taken as zero. (Section 12.4.2.2)

	Strength (1.0E)	ASD (0.7E)
e. Find the Design Seismic Shear for the Diaphragm, Section 12.10.1.1:		
Force shall not be less than 0.2*Sds*Ie*wpx =	**0.065 W**	**0.046 W**
but need not exceed 0.4*Ie*Sds wpx =	**0.130 W**	**0.091 W**
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 = Ωo V =	**0.203 W**	**0.142 W**

Notes:

1. A building that is low rise (one or two story) building with a short period is assumed for calculation of Seismic Response Coefficient, Cs.
2. The values for design spectral response acceleration assume a regular structure of 5 stories or less with a period, T < 0.5 seconds
3. The values for design forces for the diaphragm assume no offsets or changes in the stiffness of the vertical components
4. Section 1613.1 of 2018 IBC excludes the detailing requirements of Chapter 14 of ASCE 7.
5. 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

Date	8/24/2023	Sheet No.	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:

	Strength (1.0E)	ASD (0.7E)	
a. Continuous Load Path and Interconnections, Section 12.1.3:			
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 :			
F_p= .05 * dead + live reaction =	**0.050 Rd+l**	**0.035 Rd+l**	(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:			
F_p= 0.40 Ie 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 design force for steel elements with the exception of anchor bolts and reinforcing steel shall be increased by 1.4 times.

			Strength (1.0E)	ASD (0.7E)
For loading in the x-direction:				
k_a= 1.0+L_f/100 or max 2.0 =	**1.50**			
Fp= 0.4 S_{ds} k_a I_e W_p or .2 k_a I_e Wp min. =		onnections at Flexible Diaphragms:	**0.375 Wp**	**0.263 Wp**
F_p * 1.4 for steel elements per 12.11.2.2.2			**0.525 Wp**	**0.368 Wp**
k_a= 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**
For loading in the y-direction:				
k_a= 1.0+L_f/100 or max 2.0 =	**1.15**			
Fp= 0.4 S_{ds} k_a I_e W_p or .2 k_a I_e 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_a= 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 Ip Wp=	**0.416 Wp**	**0.624 Wp**	(Equation 13.3-2)
Fp min = 0.3 Sds Ip 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/Ip)

Seismic Design Force Summary on Architectural Components, Section 13.5:

	ap=	Rp=	Ip=	z/h=	;trength (1.0E)	ASD (0.7E)	
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=	**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**	
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)
Fp at roof=	**1.00**	**1.50**	**1.00**	**1.00**	**0.208 Wp**	**0.146 Wp**	(Table 13.5-1)
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)
5. Braced Parapets and Chimneys	**1.00**	**2.50**	**1.00**	**1.00**	**0.125 Wp**	**0.087 Wp**	(Table 13.5-1)
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=	**1.25**	**1.00**	**1.00**	**0.00**	**0.130 Wp**	**0.091 Wp**	(Table 13.5-1)
Fp at roof=	**1.25**	**1.00**	**1.00**	**1.00**	**0.390 Wp**	**0.273 Wp**	(Table 13.5-1)
7. Appendages and Ornamentation	**2.50**	**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.

Date	8/24/2023	Sheet No.	of
Job			
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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	Mapped Spectral Response Acceleration at Short Periods (Ss)						Distance
Class	Ss<=0.25	0.5	0.75	1	1.25	Ss>=1.5	Value
A	0.80	0.80	0.80	0.80	0.80	0.80	0.80
B	0.90	0.90	0.90	0.90	0.90	0.90	0.90
C	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
F	---	---	---	---	---	---	---

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

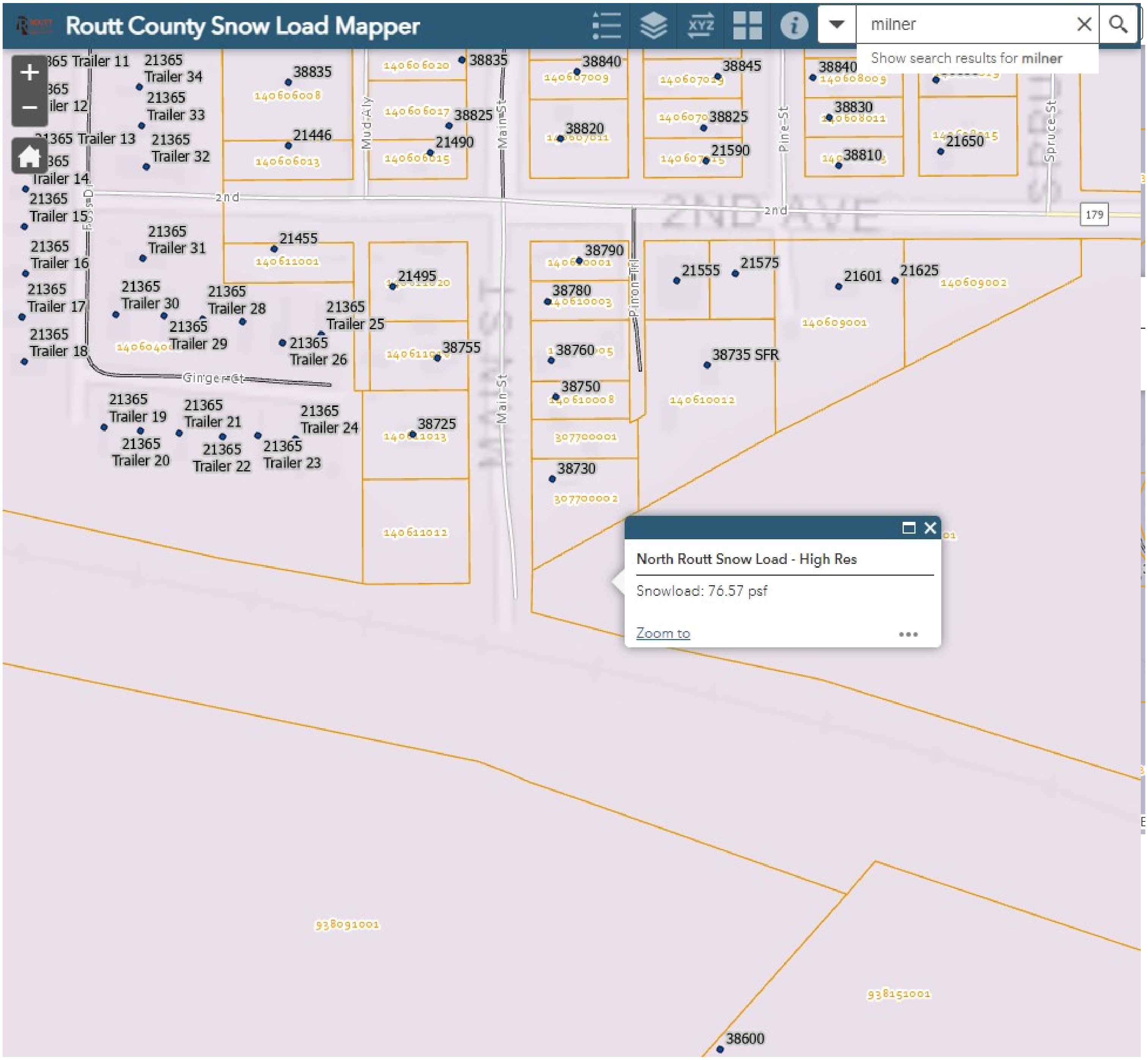
Site	Mapped Spectral Response Acceleration at 1 Second Period (S1)						Distance
Class	S1<=0.1	0.2	0.3	0.4	0.5	S1>=0.6	Value
A	0.80	0.80	0.80	0.80	0.80	0.80	0.8
B	0.80	0.80	0.80	0.80	0.80	0.80	0.8
C	1.50	1.50	1.50	1.50	1.50	1.40	1.5
D	2.40	2.20	2.00	1.90	1.80	1.70	2.4
E	4.20	3.30	2.80	2.40	2.20	2.00	4.2
F	---	---	---	---	---	---	---

IBC Table 1613.2.5(1) and 11.6-1
eismic Design Category based on Short Period Response Acceleratic

Value of	Occupancy Category			Design	
Sds	I or II	III	IV	Category	Category
Sds <= 0.167	A	A	A	A	B
0.167 <= Sds < 0.33	B	B	C	B	
0.33 <= Sds < 0.5	C	C	D	C	
0.5 <= Sds	D	D	D	D	
S1 >= 0.75	E	E	F	E	

Table 1613.2.5(2) and 11.6-2
Seismic Design Category Based on 1-Second Period Response Acceleration

Value of	Occupancy Category			Design	
Sd1	I or II	III	IV	Category	Category
Sd1 <= 0.067	A	A	A	A	A
0.067 <= Sd1 < 0.133	B	B	C	B	
0.133 <= Sd1 < 0.2	C	C	D	C	
0.2 <= Sd1	D	D	D	D	
S1 >= 0.75	E	E	F	E	

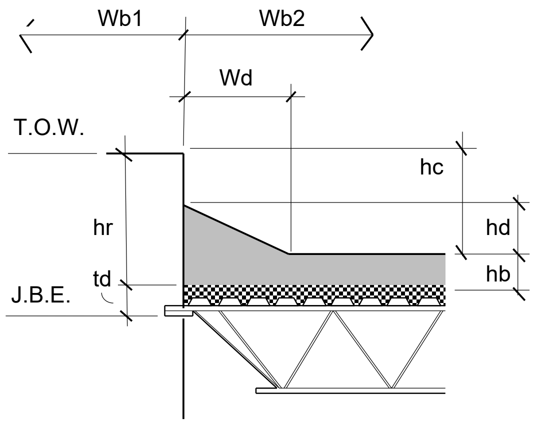


600ft
-106.819 40.538 Degrees

WALLACE DESIGN PROGRAM
Revised 05/20/19, Carrie Johnson

Date	11/6/2023	Sheet No.	of
Project	Milner		
Subject	Snow Load		

FLAT ROOF SNOW DRIFT - Joists Perpendicular to Wall
ASCE 7-16



Configuration

1. **Input**

Dead Load =	**15**	psf
Roof Live Load =	**20**	psf
Pg, Ground Snow Load =	**77**	psf
Drift for parapet, projection, or upper roof?	**U**	(P), (PR) or (U)
Risk Category (I, II, III, or IV) =	**III**	Table 1.5-1
Ce, Exposure Factor =	**1.00**	Table 7.3-1
Ct, Thermal Factor =	**1.00**	Table 7.3-2
Use Pg minimum for drift calc's (Pf = Pg)?	**n**	(Y or N)

Geometry

T.O.W., Top of Upper Roof Elevation =	**9.50**	feet
J.B.E., Joist Bearing Elevation =	**0.00**	feet
td, Thickness of Joist, Deck, and Insulation =	**0.01**	inches
Wb1, length of upper roof =	**8.00**	feet
Wb2, length of lower roof =	**1000.00**	feet
S, Joist Spacing =	**1.00**	feet
L, Joist Span =	**12.00**	feet

2. **Balanced Snow Load Check**

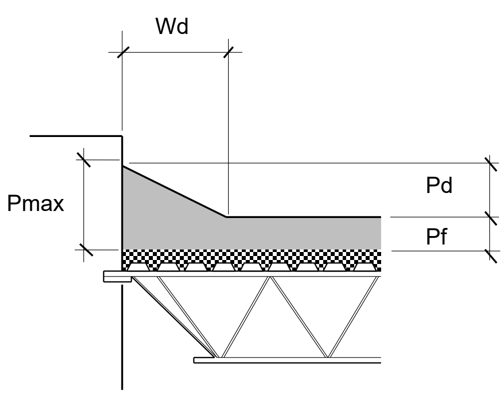
Is, Importance Factor =	**1.10**	Table 1.5-2
Pf = 0.7 Ce Ct Is Pg =	**59.29**	psf (7.3-1)
Pm = Is Pg =	**22.00**	psf (7.3.4)
Rain on snow surcharge =	**0.00**	psf (7.10)
Pmin =	**59.29**	psf

Assume very long lower "roof" is grade next to WWTP, where drift can build up

3. **Drifted Snow Load Check**

Pf = 0.7 Ce Ct Is Pg =	**59.29**	psf
D = 0.13 Pg + 14.0 ≤ 30 pcf =	**24.01**	pcf
hb = Pf/D =	**2.47**	ft
Wb =	**1000.00**	ft
hd = 0.75[0.43 Wb2^1/3 (Pg+10)^1/4-1.5] Is^2	**9.13**	ft
hd + hb =	**11.60**	ft
hr =	**9.50**	ft
hc = hr - hb =	**7.03**	ft
Wd = 4 hd or 4 [hd^2/hc] ≤ 8 hc =	**47.39**	ft
Pmax = D (hd + hb) ≤ D hr =	**228.07**	psf
Pd =D hd ≤ D hc =	**168.78**	psf

Use constant 228 psf snow load where drifts may occur



Snow Drift

4. **Uniform Load Summary**

Drifted Snow Load

	Snow	Total	
R left =	**1283.0**	**1373.0**	lbs
R right =	**1197.5**	**1287.5**	lbs
M max =	**3721.8**	**3991.8**	ft-lbs
w base =	**59.3**	**74.3**	plf
w drift =	**168.8**	**183.8**	plf
w equiv =	**213.8**	**228.8**	plf *

Load Without Drift

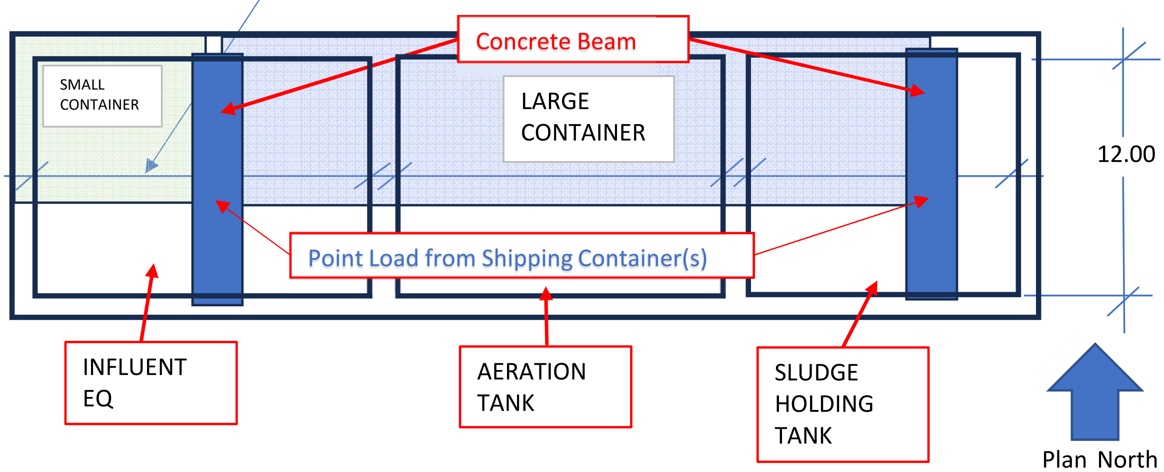
	Snow	Total	
w (Pmin = 59.29 psf	**59.3**	**74.3**	plf

* indicates controlling load (drifted vs. undrifted)

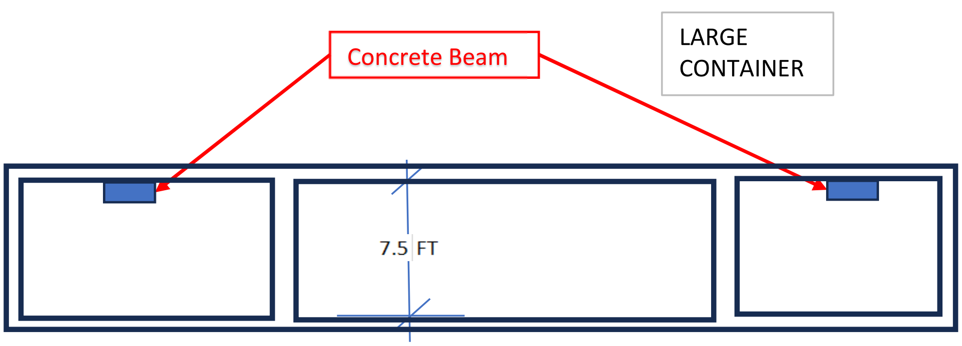
Date:	11/9/2023
Job:	2310219(3) - Milner WWTP
Subject:	Tank layout

Plan

	Inside dimension		
Width	12.00	ft	
Length			
Tank			
Influent EQ	23.42	ft	Total length = 56.6667 ft
Aeration Tank	18.00	ft	
Sludge Holding Tank	11.25	ft	Interior and exterior wall thickness = 12 in
			in



Tank Height 7.50 ft 7.75 ft at deeper tank



Backfill height at perimeter of wall:	1	ft (above tank slab elevation)
Height: of sludge above slab	7.5	ft (above tank slab elevation)
Soil parameters:		
Allowable Soil Bearing Pressure:	3000	psf
Modulus of Subgrade Reaction (Ks):	150	pci (assumed)
Coefficient of Friction for Sliding:	0.4	
Passive Lateral Pressure;	250	pcf
Active Lateral Pressure:	60	pcf *
Equivalent Fluid Density	250	pcf
Frost depth:	48	in

See Geotechnical Report from NWCC, Inc Job No 22-12814 Dated January 19, 2023

(on-site sands)

* 85-90 pcf for below water table. Ground-water found at 10 ft below grade

Date:	11/9/2023
Job:	2310219(3) - Milner WWTP
Subject:	Tank layout

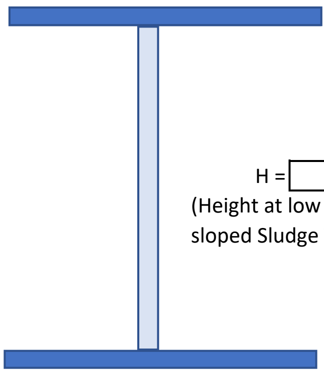
Reference

	ACI 350-06	ACI 318-14

Interior Tank Wall

Typical wall

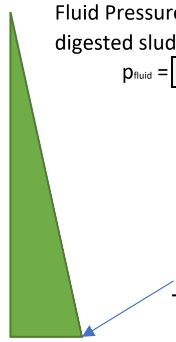
* Worst case condition - overflow one side of wall, other side of wall is empty
- assume overflow condition is completely full to bottom of lid (conservative)



Lid

H = 7.50 ft
(Height at low end of sloped Sludge Tank)

Base



Fluid Pressure on wall from digested sludge, anaerobic — R 2.2

p_{fluid} = 70 pcf

w_{max}

Force at base of wall

H * p_{fluid} = 525 plf

Minimum wall thickness = 12" — <- Controls — 14.6.2

Minimum thickness = 6" or ℓ/30 — 14.6.1

Lid span, ℓ — ℓ = 12.00 ft — ℓ/30 = 4.8 in

Assume pinned condition top and fixed bottom. **(No backfill before lid is poured)**

Mmax = = 1753 lb ft / ft (triangular load, fixed base, pinned top)

Vmax = W = 1546 lbs/ft

(See RISA output)

Modification factor, Sd — 9.2.6

Sd = ϕFy/(γfs)

ϕ =	0.9	(modification factor for tension-controlled section)	R9.2.6
Fy =	60,000 psi	(Steel yield strength)	
γ =	U factor	(Ultimate Load Factor (LRFD))	
fs =	20,000 psi	(Steel direct hoop and tensile strength)	

$$fs_{max} = \frac{320}{B\ (s^2 + 4(2+db/2)^2)^{0.5}}$$ — 10.6.4.1

For "Normal" environment exposure:
Coating to be applied to concrete surface, per
Aquaworks, Sherwin Williams Dura-Plate 6000.
Therefore environment is NORMAL

B = 1.35

db = 0.75 in (#6 bar)

s = 12 in spacing

fs_{max} = 20000 psi (for flexural stress) — 10.6.4

20000 psi (for direct and hoop tensile stress in normal exposure) — 9.2.6.2

Sd Table

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= 4733 lb ft

Note: Geotechnical Report indicates lateral pressure
exceeds 70 psf below the water table
Borings showed groundwater at 10 ft below grade
Therefore, this higher pressure is not considered
in the design of the foundation walls

Date:	11/9/2023
Job:	2310219(3) - Milner WWTP
Subject	Wall design

Wall design

								Reference 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	in				

Check Wall Flexure   Mn > Mu

ρ min =   0.00180 <== See rho min calcs below (For Slabs rhomin is T&S Steel)   Table 7.6.1.1

ρ = As / (b*d)   0.00381 <== If ρ is less than ρ min, see below

See Table 21.2.2 For Strain Boundaries when (et) is compression controlled, transition or tension controlled

No	Compression Controlled If $\epsilon t < \epsilon ty$	Tbl. 21.2.2
No	Transition Zone If $\epsilon_{ty} < \epsilon_t < 0.005$	
Yes	Tension Controlled Since (ϵt) = or > 0.005	
		21.2.1

** Solve for phiMn Based on Whitney Stress Block

a = stress block depth = As*Fy / (0.85*f'c*beff) =	0.5752	in.	Fig R21.2.2a
c = depth to neutral axis = a / β =	0.6972	in.	
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 =	20542	ft.-lb.	
ϕ=	0.9		21.2
ϕMn = Ultimate Moment Cap =	**18488.1**	ft.-lb.	
check against ρ min =	0.0033	< ϕ	
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	

in. ϕMn x 0.75 = 1 / 1.33 x ϕMn = reduction if ρ < ρ min)   9.6.1.3

(note: code says ρ 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)

ϕMn = Ultimate Moment Cap =   ***18488.1*** *lb-ft*

Mu = Mmax (U) Sd=   4733.1 lb ft   < fMn - Wall Flexure OK   Utilization = 0.26

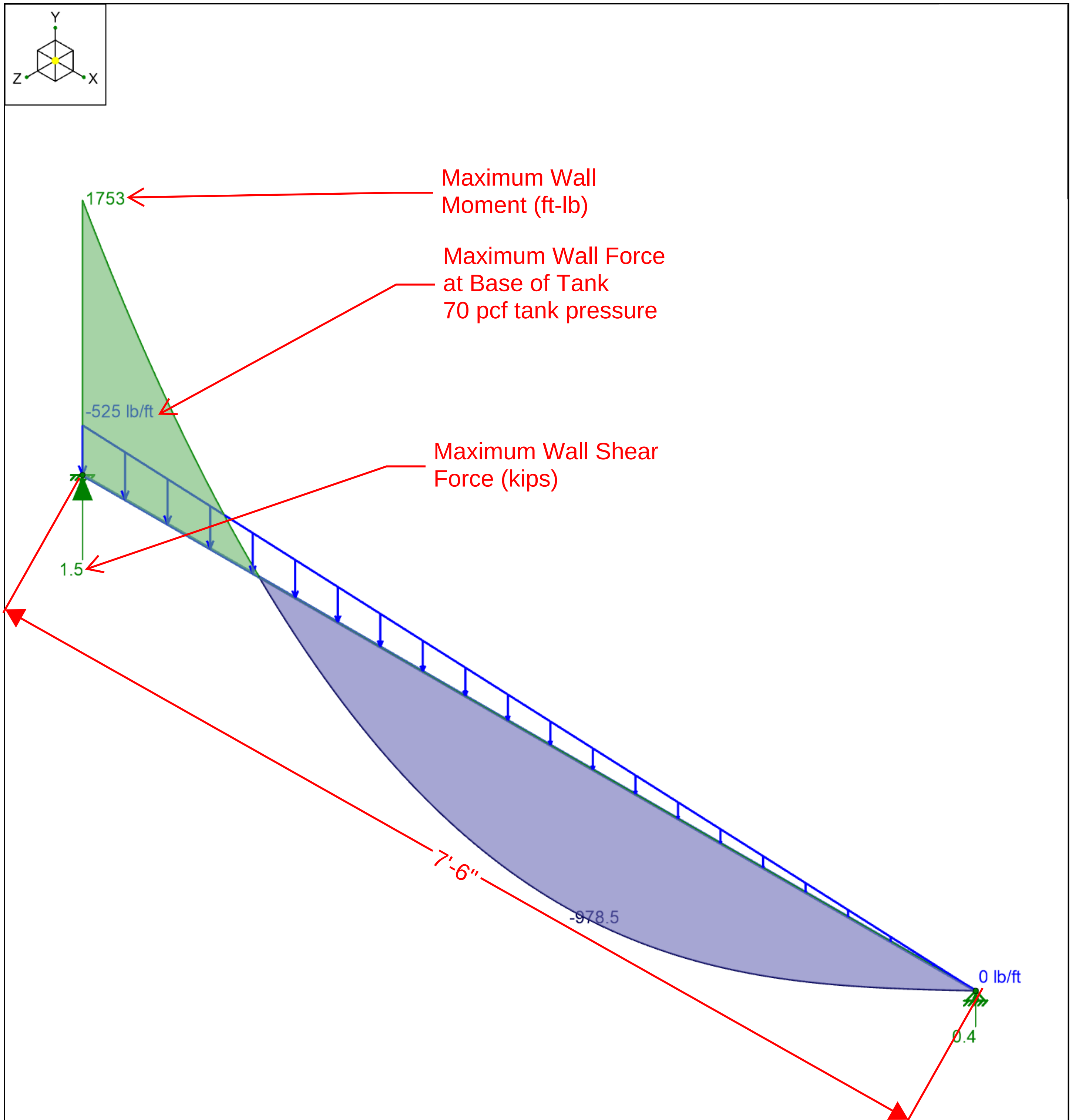
Note:
4500 psi is minimum concrete strength for concrete   Table 4.3.1
with low permeability (concrete is coated
is coated as directed by Aquaworks)

Check ρ balanced and ϵt=.005 limits:

			ACI 350-06	ACI 318-14
β= Max of ((0.85-0.05*(f'c-4000)/1000,0.65)) =	0.8250		10.2.7.3	
ϵty = tension yield strain = fy / Es	0.0021	<== Es = 29,000,000 psi	R10.3.2	
ϵc = balanced concrete strain =	0.0030		R10.3.2	
ϵt = actual tension strain = ((d-c) / c)*strain sc	0.0384		R10.3.3	
ρ temp = temp and shrink steel =	0.0018	<== if Fy<60,000 psi, .002 -or- .0018 * 60,000psi /	Table 7.12.2.1	
As(temp)= (ρ temp * beff * h) =	0.2592	in^2		
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	
ρt = (.85*β*f'c / Fy)*(0.003/(0.008))	0.0197	<== Max reinf. Ratio, when 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		9.6.1.3

				Reference ACI 350-06	Reference ACI 318-14
4. Check Shear Capacity					
**neglects Vs (shear reinforcement)					
Vc = 2 * λ * (f'c)^.5 * b * d =		15496 lbs			11.2.1.1
ϕ =	0.75				21.2
ϕVc/2 =	**For Beam Design (h > 10 in.)	5811 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 Check Needed		9.2.1	

Use 12 in wall w/ (#6 bar) @ 12 in spacing



Loads: LC 1, D
Results for LC 1, D
Member z Bending Moments (lb-ft)
Y-direction Reaction Units are kips and lb-ft

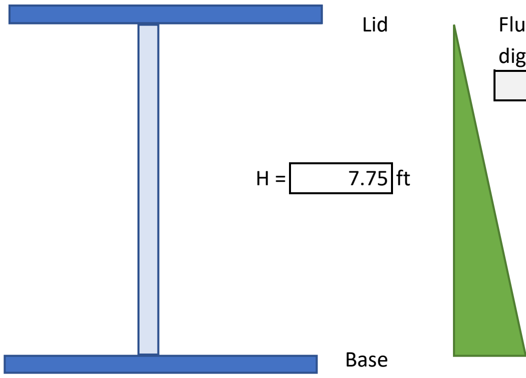
Wallace Engineering	Interior Tank Wall (7.5 ft) - 1 ft section typ	SK-1
S Jacob		Nov 06, 2023
2310219 (3) - Milner WWTP		Short interior wall.r3d

Date:	11/9/2023
Job:	2310219(3) - Milner WWTP
Subject	Wall design

Reference: ACI 350-06 | ACI 318-14

Tall wall

* Worst case condition - overflow one side of wall, other side of wall is empty
 - assume overflow condition is completely full to bottom of lid (conservative)



Fluid Pressure on wall from digested sludge, anaerobic = 70 pcf (R 2.2)

Force at base of wall
$H * p_{fluid}$ =
w_{max} = 542.5 plf

Minimum wall thickness = 12" <- Controls (14.6.2)

Minimum thickness = 6" or ℓ/30 (14.6.1)

Lid span, ℓ ℓ = 12.00 ft ℓ/30 = 4.8 in

Assume pinned condition top and fixed bottom. **(No backfill before lid is poured)**

Mmax = = 2386 lb ft / ft (triangular load, fixed base, pinned top)
Vmax = W = 1881 lbs/ft
(See RISA output)

Modification factor, Sd (9.2.6)

Sd = ϕFy/(γfs)

ϕ =	0.9	(modification factor for tension-controlled section)	R9.2.6
Fy =	60,000 psi	(Steel yield strength)	
γ =	U factor	(Ultimate Load Factor (LRFD))	
fs =	20,000 psi	(Steel direct hoop and tensile strength)	

$$f_{s\,max} = \frac{320}{B\ (s^2 + 4(2+db/2)^2)^{0.5}}$$ (10.6.4.1)

For "Normal" environment exposure:
Coating to be applied to concrete surface, per Aquaworks, Sherwin Williams Dura-Plate 6000.
Therefore environment is NORMAL

B = 1.35
db = 0.75 in (#6 bar)
s = 12 in spacing

$f_{s\,max}$ = 20000 psi (for flexural stress) (10.6.4)
20000 psi (for direct and hoop tensile stress in normal exposure) (9.2.6.2)

Sd Table

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) - Milner WWTP
Subject	Wall design

Reference: ACI 350-06 | ACI 318-14

Wall design

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	in			

Check Wall Flexure Mn > Mu

ρ min = 0.00333 <== See ρ min calcs below (For Slabs ρmin is T&S Steel)

ρ = As / (b*d) 0.00381 <== If ρis less than rho min, see below

See Table 21.2.2 For Strain Boundaries when (εt) is compression controlled, transition or tension controlled

No	Compression Controlled If εt < εty	Tbl. 21.2.2
No	Transition Zone If $\epsilon_{ty} < \epsilon_t < 0.005$	
Yes	Tension Controlled Since (εt) = or > 0.005	

21.2.1

** Solve for phiMn Based on Whitney Stress Block

a = stress block depth = As*Fy / (0.85*f'c*beff) =	0.5752	in.	Fig R21.2.2a
c = depth to neutral axis = a / β =	0.6972	in.	
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 =	20542	ft.-lb.	
ϕ=	0.9		
ϕMn = Ultimate Moment Cap =	**18488.1**	ft.-lb.	
check against ρ min =	0.0033	> ϕ min	
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	

in. phiMn x 0.75 = 1 / 1.33 x phiMn = reduction if rho < rho min) 9.6.1.3
(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)

Mu = Mmax (U) Sd= 6442 lb ft < fMn - Wall Flexure OK Utilization = 0.46

Note:
4500 psi is minimum concrete strength for concrete Table 4.3.1
with low permeability (concrete is coated
is coated as directed by Aquaworks)

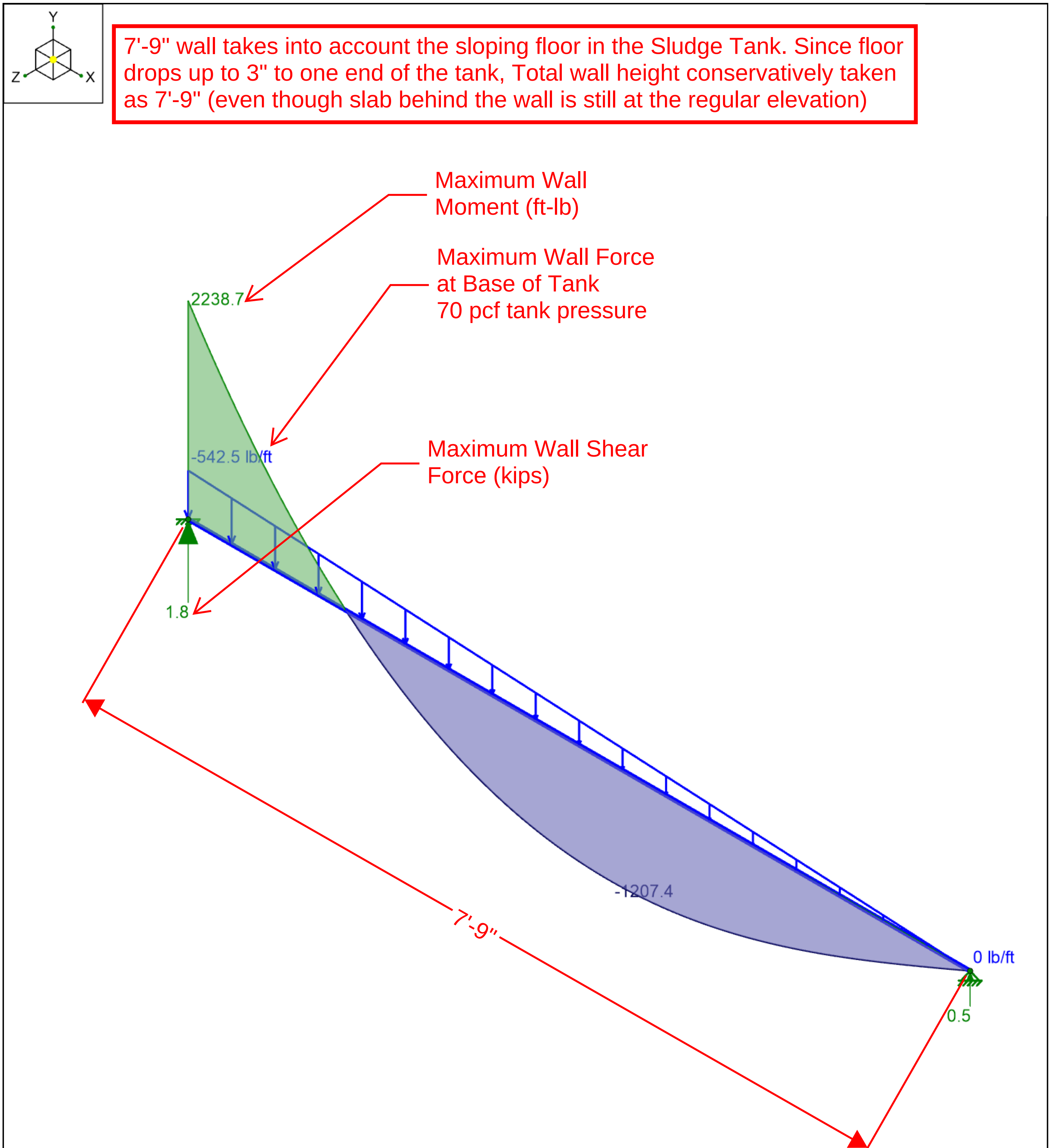
Check ρ balanced and εt=.005 limits:

β= Max of ((0.85-0.05*(f'c-4000)/1000,0.65)) =	0.8250		10.2.7.3
εty = tension yield strain = fy / Es	0.0021	<== Es = 29,000,000 psi	R10.3.2
εc = balanced concrete strain =	0.0030		R10.3.2
εt = actual tension strain = ((d-c) / c)*strain sc	0.0384		R10.3.3

ρ temp = temp and shrink steel = 0.0018 <== if Fy<60,000 psi, .002 -or- .0018 * 60,000psi / Table 7.12.2.1

As(temp)= (ρ temp * beff * h) = 0.2592 in^2

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	
ρt = (.85*β*f'c / Fy)*(0.003/(0.008))	0.0197	<== Max reinf. Ratio, when 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		9.6.1.3



Loads: LC 1, D
Results for LC 1, D
Member z Bending Moments (lb-ft)
Y-direction Reaction Units are kips and lb-ft

Wallace Engineering	Interior Tank Wall (7.5 ft) - 1 ft section typ	SK-3
S Jacob		Nov 06, 2023
2310219 (3) - Milner WWTP		Short interior wall.r3d

Job:	2310219(3) - Milner WWTP
Subject	Wall design
1/0/1900	1/0/1900

Reference

ACI 350-06	ACI 318-14

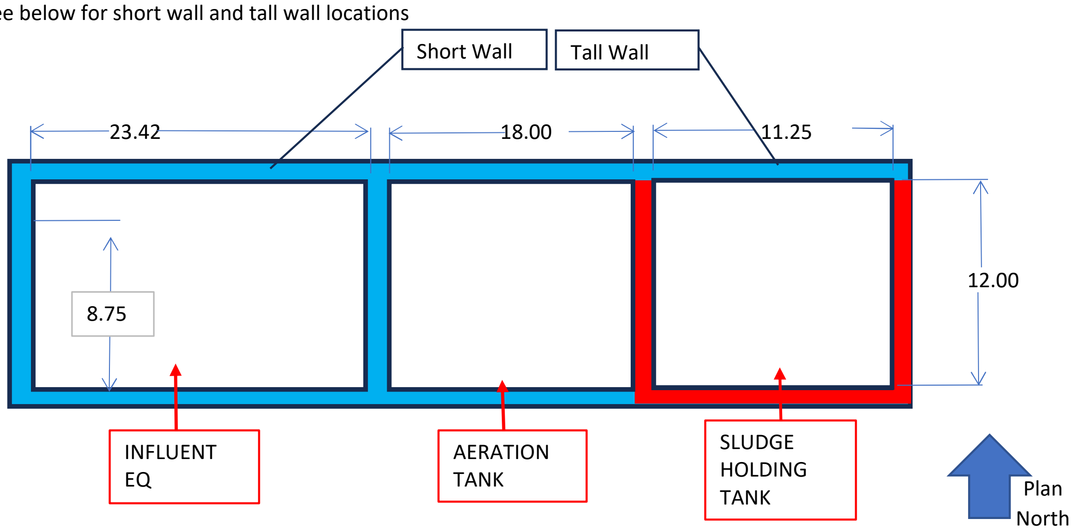
4. Check Shear Capacity

**neglects Vs (shear reinforcement)

			ACI 350-06	ACI 318-14
Vc = 2 * λ * (f'c)^.5 * b * d =		15496 lbs		11.2.1.1
ϕ =	0.75			9.3.2
ϕVc/2 = **For Beam Design (h > 10 in.)		5811 lbs / ft		11.4.6.1
Check if shear capacity is OK without considering shear reinforcement				
(Evorinmental 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 Check 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



Date:	11/9/2023
Job:	2310219(3) - Milner WWTP
Subject	Wall design

Tank Wall with concentrated load from LID BEAM and/or CONTAINER

Assume worst case location - Wall with tank pressure at full wall height and supporting container concentrated load. Also, for case when tank is empty and soil is backfilled to the top of the wall, the lateral soil pressure is LESS THAN the tank sludge pressure, so the tank sludge pressure controls <---CONTROLS

Find Container Concentrated load		L	W	H
Large container:	(40 ft long x 8 ft wide x 9.5 ft tall)	40	8	9.5
Small container:	(10 ft long x 8 ft wide x 9.5 ft tall)	10	8	9.5

		Large Container	Small	
(wet weight)	DL =	43975 lbs	10781 lbs	(provided by Aquaworks)
	LL =	60 psf		
	SL =	77 psf		
	RL =	20 psf		
	$Wind_{down}$ =	9.6 psf		
	$Wind_{up}$ =	-43.0 psf	(Average uplift of corner and end zone pressures)	

Assume container is supported at 4 corner and load is distributed equally to each corner*

Corner Loads (Large):		At Center Beam, 2 container corners at same point:		Corner Loads (small):	
DL =	10993.75 lbs	DL =	13689 lbs	DL =	2695.25 lbs
LL =	4800 lbs	LL =	6000 lbs	LL =	1200 lbs
SL =	6160 lbs	SL =	7700 lbs	SL =	1540 lbs
RL =	1600 lbs	RL =	2000 lbs	RL =	400 lbs
$Wind_{down}$ =	768 lbs	$Wind_{down}$ =	960 lbs	$Wind_{down}$ =	192 lbs
$Wind_{up}$ =	-3440 lbs	$Wind_{up}$ =	-4300 lbs	$Wind_{up}$ =	-860 lbs

* Actually, container is shimmed along length above perpendicular walls, so this calculation is conservative

Wind overturning	w =	21.1 psf
Large Container	T = C =	2380 lbs

* Same value for small container due to height and width

Total Wside = 2380 lbs (vertical load from wind overturning on Containers)

ASD Load Combinations	Large Only	Small Only	At (2) Containers	
1 D + L	15794 lbs	3895	19689 lbs	
2 D + S	17154 lbs	4235	21389 lbs	
3 D + 0.75(L + S)	19214 lbs	4750	23964 lbs	
4 D + W	11762 lbs	2887	14649 lbs	
5 D + 0.75(L + S + W)	19790 lbs	4894	24684 lbs	<---CONTROLS
6 0.6D - W	3156 lbs	757	3913 lbs	No Net Uplift

	1 container	
Maximum Vertical Concentrated Load from lid & Containers=	19790	(Beam 1)
Maximum Vertical Concentrated Load from lid & Containers=	24684	(Beam 2)

(This is point load on beam) - see beam design for reaction to wall

Reaction at Wall = 49441 lbs (max at 2 container corners + beam reaction)

Date:	11/9/2023
Job:	2310219(3) - Milner WWTP
Subject	Wall design

Reference	
ACI 350-06	ACI 318-14

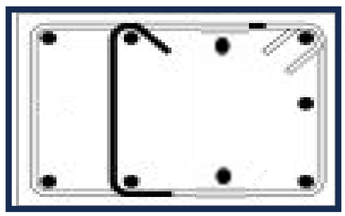
Wall with compression from beam bearing

$\phi Pn = \phi\ \alpha\ [0.85\ f'c\ (Ag - Ast) + fy\ Ast]$ — 10.3.7.2

ϕ =	0.65			
α =	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
ϕPn =	150465.1	lbs	Number of #6 bars:	8.7

Column size: 24 x 12 — **Use (8) #6 bars**

a	12	in	
b	24	in	
clear distance	2	in	
bar diameter	0.75	in	#6
#bars along long face	4	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 every other vertical bar	25.7.2.3

Use #3 stirrups @ 12 inches o.c.

wallace
design
collective

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

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

Reference: ACI 350-06 | ACI 318-14

Lid Design

Typical span - away from openings and container support locations

Thickness = 12 in
Clear cover = 2 in
bar area = 0.44 in^2 **# 6 bar**
bar diameter = 0.75 in **# 6 bar**
DL = 150 psf
LL = 60 psf
(For simplicity, assume LL and SL over full span, even though not under shipping container)
SL = 84.7 psf
$Wind_{down}$= 9.6 psf

SEE BELOW FOR CALCULATIONS USING DRIFT LOADING

Use 12" wide strip for design

Factored Loads * Sd (see below for Sd factor)

				without Sd	ACI 350-06
					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)	1.2DL+1.6W+1.0LL+0.5SL	767.5	plf	297.71	
(9-5)	1.2DL+1.0E+1.0LL+0.2S	762.9	plf	264.74	

0.052
Seismic vertical component

Use w = 767.5 plf

Modification Factor used in above load calculations

Sd = $\phi Fy/(\gamma fs)$

ϕ = 0.9 (modification factor for tension-controlled section) — R9.2.6
Fy = 60,000 psi (Steel yield strength)
γ = U factor (Ultimate Load Factor (LRFD))
fs = 20,000 psi (Steel direct hoop and tensile strength)

$$fs_{max} = \frac{320}{B\ (s^2 + 4(2+db/2)^2)^{0.5}}$$ — 10.6.4.1

For "Normal" environment exposure: Coating to be applied to concrete surface, per Aquaworks, Sherwin Williams Dura-Plate 6000. Therefore environment is NORMAL — 10.6.4.4

B = 1.35
db = 0.75 in # 6 bar
s = 10 in spacing
fs_{max} = 21411 psi (for flexural stress) — 10.6.4
20000 psi (for direct and hoop tensile stress in normal exposure) — 9.2.6.2
24000 psi (for shear stress carried by **shear reinforcement**) — 9.2.6.4

Sd Factor - Bending		
U factor	Sd	
1.2	2.10	2.522
1.6	1.58	2.522
1.4	1.80	2.522
0.5	5.04	2.522
1	2.52	2.522
0.2	12.61	2.522

Sd Factor - Shear		
U factor	Sd	
1.2	1.56	1.875
1.6	1.17	1.875
1.4	1.34	1.875
0.5	3.75	1.875
1	1.88	1.875
0.2	9.38	1.875

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

Reference: ACI 350-06 | ACI 318-14

Span = (Use multi-span condition - 23'-5" - 18'-0" - 11'-3"

(add thickness of interior walls to span in RISA model)

MULTI-SPAN (Assume no containers, no drift)

Sd Mu =	45590	lb ft	(See RISA output)	(Using Sd for 8in spacing)
Vu =	11731	lbs	(See RISA output)	(Using Sd for 8in spacing)

Compare continuous "beam" across multiple spans with simple span and other loading conditions below. USE SIMPLE SPAN

Check simple span Member section forces: (Span is clear span) — 7.4.2

M Sd = wl^2/8 = 13814.36 lb ft — More efficient to use Multi-span - detail accordingly

V Sd = wl/2 = 2253 lbs

For Simple span with container in place and snow drifts: (See Risa Results)

M Sd = 18546 lb ft

V Sd = 5298 lbs

DRIFT LOAD = 228 psf

See RISA Results. Design is based on this condition

For multi-Span slab adjacent to container with snow drift (See Risa Results)

M Sd = 45667 lb ft (Using Sd for 8in spacing)

V Sd = 11749 lbs (Using Sd for 8in spacing)

Lid design

							ACI 350-06
Lid thickness =	12	in	As =	0.528	in^2	<----- # 6 bar	
Clear cover =	2	in	Fy =	60,000	psi	at	7.7.1
Bar diameter =	0.75	in	f'c =	4500	psi	10	Table 4.2.2
d =	9.63	in	b =	12	in	in spacing	

Check Lid Flexure Mn > Mu

ρ min = 0.00333 <== See rho min calcs below (For Slabs rhomin is T&S Steel) — Table 7.6.1.1

ρ = As / (b*d) 0.00457 <== If ρ is less than ρ min, see below

See Table 21.2.2 For Strain Boundaries when (єt) is compression controlled, transition or tension controlled — Tbl. 21.2.2

No	Compression Controlled If єt < єty
No	Transition Zone If $\epsilon_{ty} < \epsilon_t < 0.005$
Yes	Tension Controlled Since (єt) = or > 0.005

** Solve for phiMn Based on Whitney Stress Block — Fig R21.2.2a

			ACI 350-06	ACI 318-14
a = stress block depth = As*Fy / (0.85*f'c*beff) =	0.6902	in.		
c = depth to neutral axis = a / β =	0.8366	in.		
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 =	24499	ft.-lb.		
ϕ=	0.9			21.2.1
ϕMn = Ultimate Moment Cap =	22049	ft.-lb.	R10.3.5	
check against ρ min =	0.0033	< ρ		9.6.1.3
check against ρ max =	0.0150	<== determine rho max eqn		
If below ρ min, multiply As x 1.33 =	0.7022	in^2		9.6.1.3
if below ρ min, multiply phiMn x 0.75 =	16537	lb-ft		

in. ϕMn x 0.75 = 1 / 1.33 x ϕMn = reduction if ρ < ρ min)
(note: code says ρ 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)

ϕMn = 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) — Table 4.3.1

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

			Reference ACI 350-06	ACI 318-14
Check ρ balanced and εt=.005 limits:				
β= Max of ((0.85-0.05*(f'c-4000)/1000,0.65)) =	0.8250		10.2.7.3	
εty = tension yield strain = fy / Es	0.0021	<== Es = 29,000,000 psi	R10.3.2	
εc = balanced concrete strain =	0.0030		R10.3.2	
εt = actual tension strain = ((d-c) / c)*strain sc	0.0315		R10.3.3	
ρ temp = temp and shrink steel =	0.0018	<== if Fy<60,000 psi, .002 -or- .0018 * 60,000psi / Fy	Table 7.12.2.1	
As(temp)= (ρ temp * beff * h) =	0.2592	in^2		
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	
ρt = (.85*β*f'c / Fy)*(0.003/(0.008))	0.0197	<== Max reinf. Ratio, when 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 * λ * (f'c)^.5 * b * d =		15496 lbs		22.5.5.1
ϕ = 0.75	ϕVc =	11622 lbs		21.2

Check if shear capacity is OK without considering shear reinforcement

(Environmental durability factor does not apply to concrete shear capacity) — 9.2.6

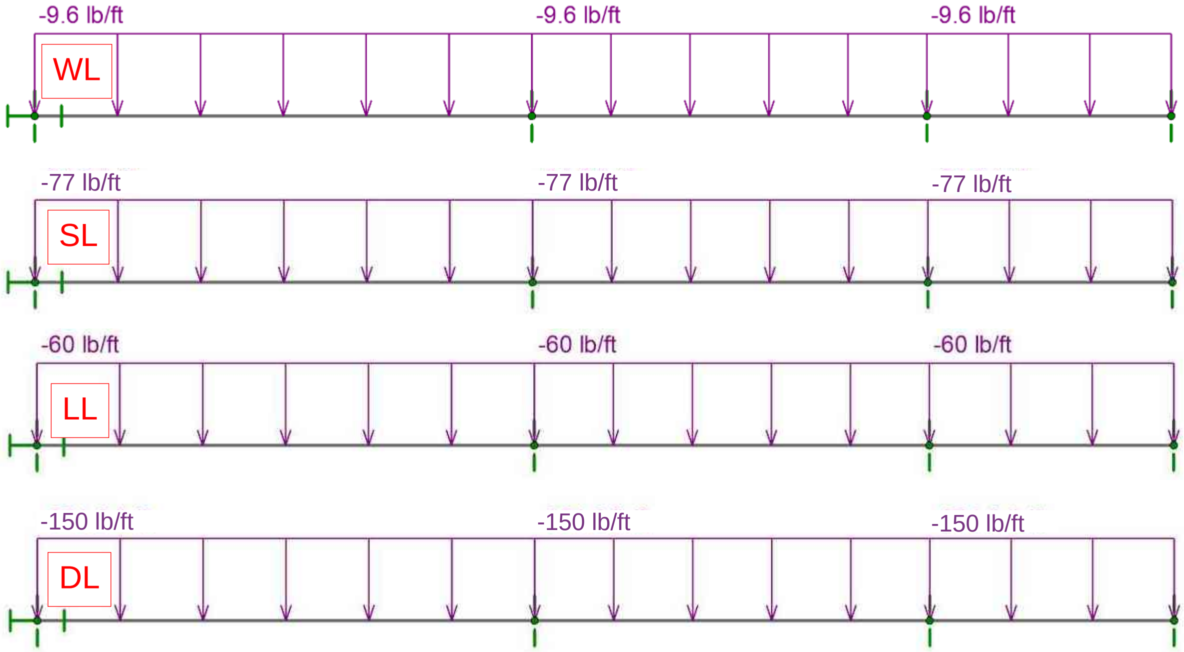
Vu = Vmax (U) = 5298 lbs — < Vc - Slab Shear OK — Utilization = 0.46

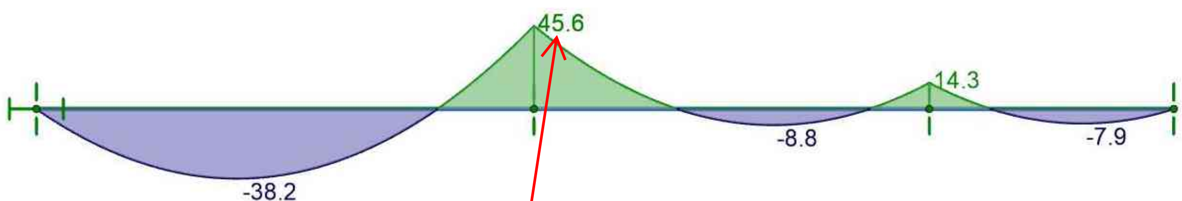
See Below for Reinforcement Check — 9.2.1

Vs required = Sd (Vu-fVc) = -15949.4 lbs. - No shear reinforcement required — R9.2.6.4

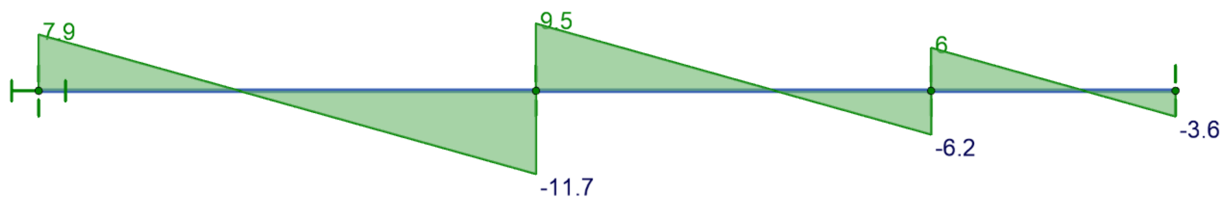
Use 12 in slab w/ # 6 bar @ 10 in spacing

See Spreadsheet for Load Combinations



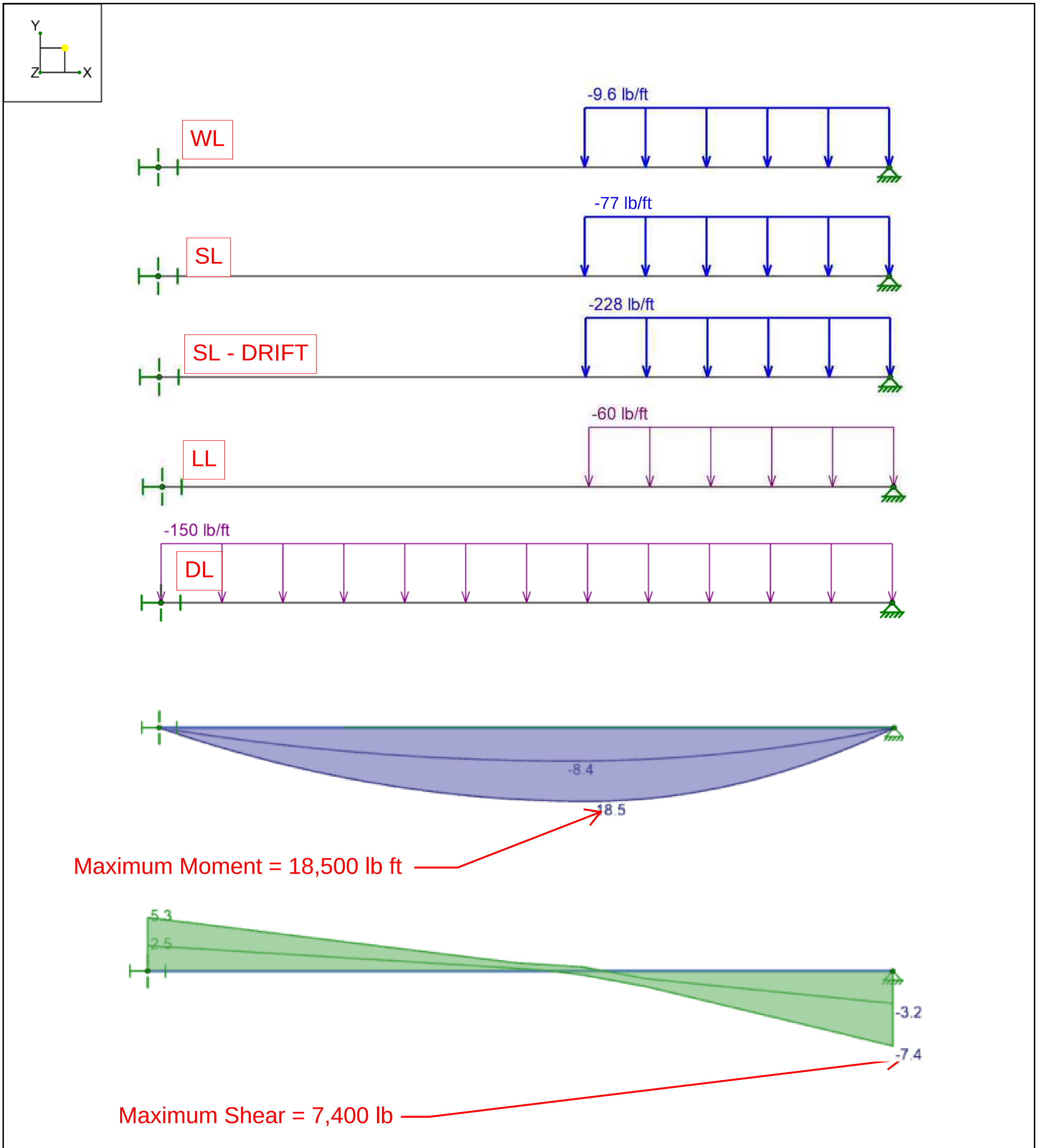


Maximum Moment = 45600 lb ft



Maximum Shear = 11,700 lb

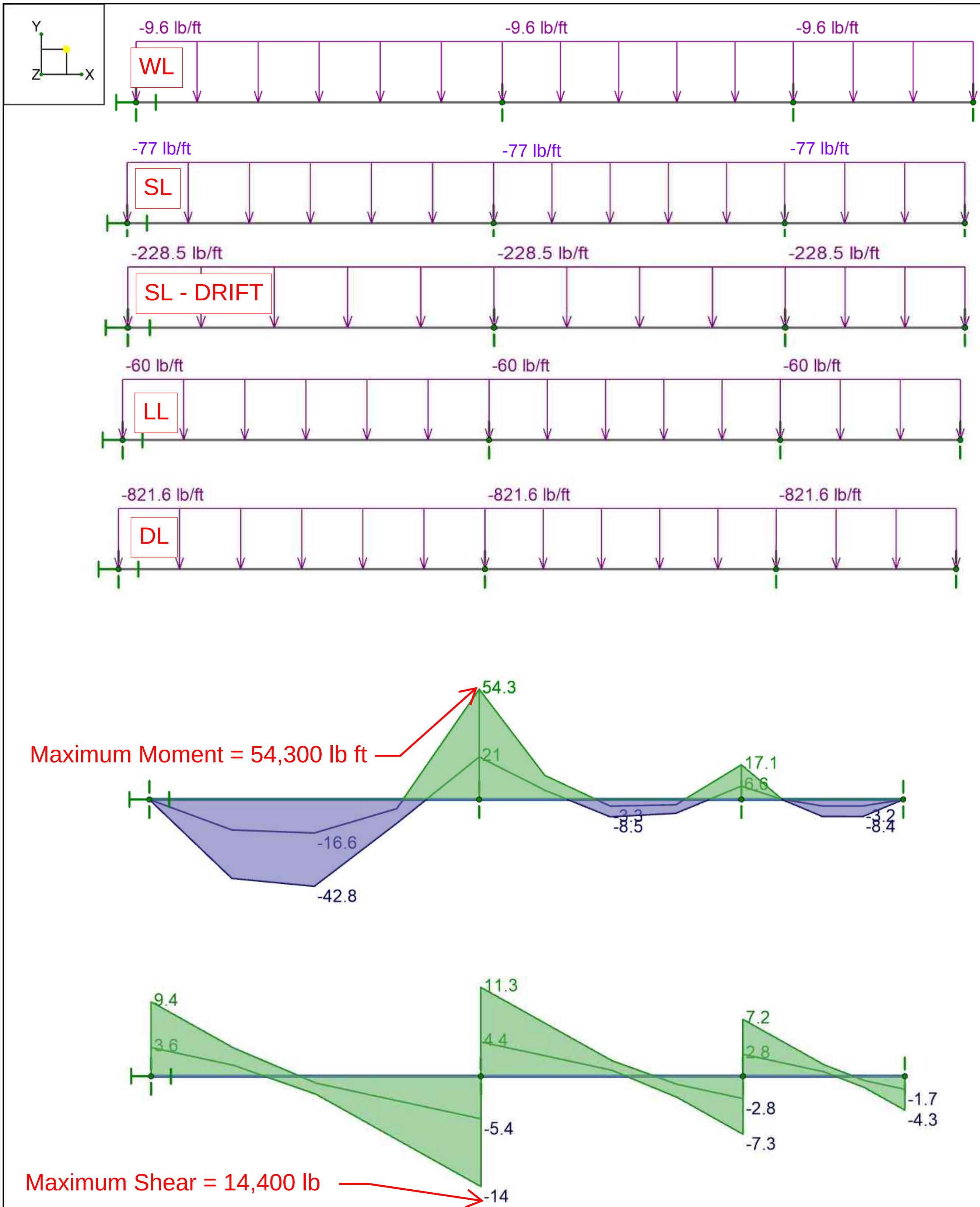
Wallace Design Collective	Lid Design	SK-1
S Jacob		Nov 06, 2023
2310219 (3) Milner WWTP		Lid - multispan.r3d



Design Lid for this condition: Top-most and bottom most steel shall span the short-direction of the tank (Plan N-S)

Loads: BLC 1, DL
Envelope Only Solution

Wallace Engineering	Milner WWTP	SK-7
S Jacob		Nov 06, 2023
2310219		Lid Span with drift.r3d



Envelope Only Solution
Member z Bending Moments (kip-ft) (Enveloped)

Wallace Design Collective	Lid Design	SK-13
S Jacob		Nov 06, 2023
2310219 (3) Milner WWTP		Lid - multispan with drift along cont…

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)

	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	-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) - Milner WWTP
Subject:	Concrete Lid Design

Reference: ACI 350-06 | ACI 318-14

Slab section near opening

Slab Reinf is at 10 in spacing

Opening width = 5.5 ft (3 ft on each side of slab strip

"beam" width = 2.75 ft (slab strip width between openings)

Sd = 1

Mu Sd = 40444 lb ft * (Multiply Typical lid Moment by (1/2 opening+beam width)

Vu = 64521 lbs * (Multiply Typical lid Shear by (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)

Can 10 bars fit in "beam"?

bar spacing = 3.14 in BAR SPACING OK

			ACI 318-14
Minimum spacing = larger of:	1	in	25.2
db =	0.75	in	25.2
agg = 0.75 4/3 dagg =	1	in	25.2

ρ bal = (0.85*f'c*b/Fy)*(0.003/(0.003+ϵty)		0.0311	<== balanced steel ratio
0.75(ρ bal)=		0.0233	
ϵty = tension yield strain = fy / Es		0.0021	<== Es = 29,000,000 psi
Total Steel Area = As =	4.4 in^2	Less than As (max)	
depth, d =	9.63 in	**Use**	4.40 in^2
width, b =	33 in		(See below for Max As)
Fy =	60,000 psi		
f'c =	4,500 psi		

Check Lid Flexure Mn > Mu

			ACI 318-14
ρ min =	0.00333	<== See rho min calcs below (For Slabs rhomin is T&S Steel)	Table 7.6.1.1
ρ = As / (b*d)	0.01385	<== If ρ is less than ρ min, see below	

See Table 21.2.2 For Strain Boundaries when (ϵt) is compression controlled, transition or tension controlled — Tbl. 21.2.2

No	Compression Controlled If ϵt < ϵty
No	Transition Zone If $\epsilon_{ty} < \epsilon_t < 0.005$
Yes	Tension Controlled Since (ϵt) = or > 0.005

			ACI 350-06	ACI 318-14
** Solve for phiMn Based on Whitney Stress Block				Fig R21.2.2a
a = stress block depth = As*Fy / (0.85*f'c*beff) =	2.0915	in.		
c = depth to neutral axis = a / β =	2.5352	in.		
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 =	188743	ft.-lb.		
ϕ=	0.9			21.2.1
ϕMn = Ultimate Moment Cap =	169869	ft.-lb.	R10.3.5	
check against ρ min =	0.0033	< ρ		9.6.1.3
check against ρ max =	0.0233	<== determine rho max eqn		
If below ρ min, multiply As x 1.33 =	5.8520	in^2		9.6.1.3
if below ρ min, multiply phiMn x 0.75 =	127402	lb-ft		

in. ϕMn x 0.75 = 1 / 1.33 x ϕMn = reduction if ρ < ρ min)
(note: code says ρ 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)

ϕMn = Ultimate Moment Cap = ***169869.1*** *lb-ft*

Mu = 40444 1 < fMn - Lid Flexure OK Utilization = 0.24

Use 12 in slab w/ 10 # 6 bar between openings

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

			Reference	
			ACI 350-06	ACI 318-14
Check ρ balanced and πt=.005 limits:				
β= Max of ((0.85-0.05*(f'c-4000)/1000,0.65)) =	0.8250		10.2.7.3	
εty = tension yield strain = fy / Es	0.0021	<== Es = 29,000,000 psi	R10.3.2	
εc = balanced concrete strain =	0.0030		R10.3.2	
εt = actual tension strain = ((d-c) / c)*strain sc	0.0084		R10.3.3	
ρ temp = temp and shrink steel = 0.0050			Table 7.12.2.1	
As(temp)= (ρ temp * beff * h) = 1.9800 in^2				
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	
ρt = (.85*β*f'c / Fy)*(0.003/(0.008))	0.0197	<== Max reinf. Ratio, when steel strain >= .005		
As (max) = (ρt * 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
ρ min.= greater of ρ min(a) or ρ min(b) =	0.0034	<== Min. reinf. Ratio		
As (min)= (ρ min * beff * d) =	1.0653	in^2 <== unless As > 1.33*As reqd per code		9.6.1.3

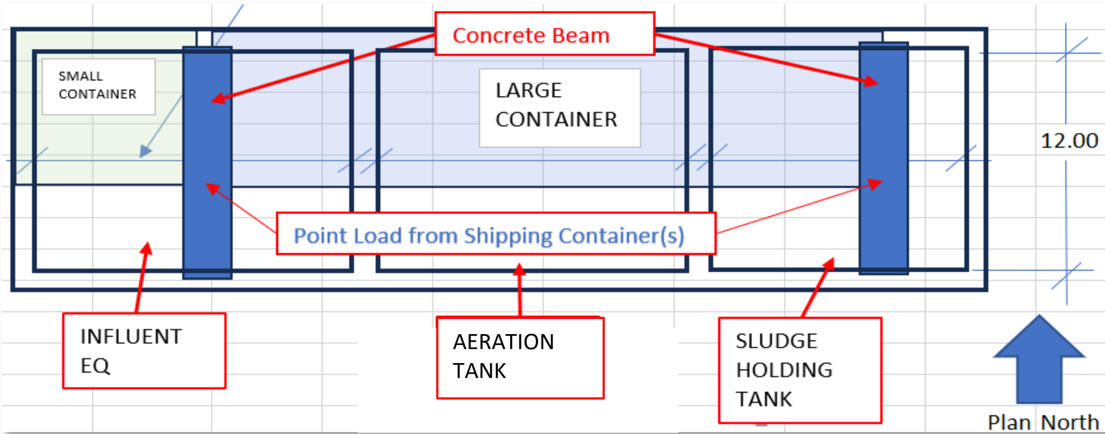
Check Shear Capacity

**neglects Vs (shear reinforcement)				
Vc = 2 * λ * (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 considering shear reinforcement

(Environmental durability factor does not apply to concrete shear capacity)			9.2.6	
Vu = Vmax (U) = 0 lbs	< Vc - Slab Shear OK	Utilization = 0.00		
	See Below for Reinforcement Check		9.2.1	
Vs required = Sd (Vu-fVc) = 0	lbs. - Shear reinforcement required		R9.2.6.4	

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



Beam #1 Design - Over Sludge Holding Tank

			Reference ACI 350-06	ACI 318-14
Span =	12.00 ft		8.7.3	9.4.2

* Beam spans as shown above

Beam width = b =	24	in	
Beam height = h =	16	in	
bar diameter =	0.75	in	**#6 bar**
bar area =	0.44	in^2	
stirrup bar diameter =	0.375	in	#3
clear cover =	2	in	Self Weight of beam
d =	13.25	in	400 plf

Concentrated Load from container: * Assume no shims support container above interior walls

DL =	13683	lbs
LL =	6000	lbs
SL =	7700	lbs
RL =	2000	lbs
W_{down} =	960	lbs
W_{up} =	-3640	lbs
Wind (overturning) =	2380	lbs

Try 6 #6 bar in beam

bar spacing = 3.85 in

Factored Loads * Sd (see below for Sd factor)

			without Sd	Reference
(9-2)	1.2DL+1.6LL+0.5SL	39384.5 lbs	21682.75	9.2.1
(9-3)	1.2DL+1.6SL+1.0LL	39736.5 lbs	21682.75	
(9-4)	1.2DL+1.6W+1.0LL+0.5SL	39678.0 lbs	22642.75	
(9-5)	1.2DL+1.0E+1.0LL+0.2S	39133.1 lbs	22394.253	

0.052
Seismic vertical component

Use P = 39736.5 lbs

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

Reference: ACI 350-06 | ACI 318-14

Modification Factor used in above load calculations

Sd = $\phi Fy/(\gamma fs)$

ϕ = 0.9 (modification factor for tension-controlled section) — R9.2.6

Fy = 60,000 psi (Steel yield strength)

γ = factored load/unfactored load (Ultimate Load Factor (LRFD))

fs = 20,000 psi (Steel direct hoop and tensile strength)

$$fs_{max} = \frac{320}{B\ (s^2 + 4(2+db/2)^2)^{0.5}}$$ — 10.6.4.1

For "Normal" environment exposure: Coating to be applied to concrete surface, per Aquaworks, Sherwin Williams Dura-Plate 6000. Therefore environment is NORMAL — 10.6.4.4

B = 1.35

db = 0.75 in #6 bar

s = 3.85 in spacing

fs_{max} = 38767 psi (for flexural stress) — 10.6.4

20000 psi (for direct and hoop tensile stress in normal exposure) — 9.2.6.2

24000 psi (for shear stress carried by **shear reinforcement**) — 9.2.6.4

Sd(min) = 1.0 — 9.2.6

Sd Factor - Bending		
U factor	Sd	Sd*U
1.2	1.16	1.393
1.6	1.00	1.600
1.4	1.00	1.400
0.5	2.79	1.393
1	1.39	1.393
0.2	6.96	1.393

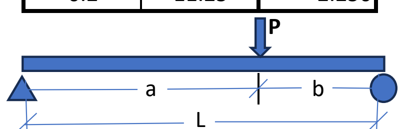
Sd Factor - Shear		
U factor	Sd	Sd*U
1.2	1.88	2.250
1.6	1.41	2.250
1.4	1.61	2.250
0.5	4.50	2.250
1	2.25	2.250
0.2	11.25	2.250

Moment at mid-span

Mu = Pab/L = 127375.9 lb ft

Shear at support

Vu = Pa/L = 39448 lbs



a= 6.75 ft

b= 5.25 ft

L= 12.00 ft

Fy = 60,000 psi

f'c = 4,500 psi Beam Reaction = 24752 lbs

Check Lid Flexure Mn > Mu

ρ min = 0.00333 <== See rho min calcs below (For Slabs rhomin is T&S Steel) — Table 7.6.1.1

ρ = As / (b*d) 0.00830 <== If ρ is less than ρ min, see below

See Table 21.2.2 For Strain Boundaries when (et) is compression controlled, transition or tension controlled — Tbl. 21.2.2

No	Compression Controlled If $\epsilon t < \epsilon ty$
No	Transition Zone If $\epsilon_{ty} < \epsilon_t < 0.005$
Yes	Tension Controlled Since (ϵt) = or > 0.005

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

** Solve for phiMn Based on Whitney Stress Block				Fig R21.2.2a
a = stress block depth = As*Fy / (0.85*f'c*beff) =	1.7255	in.		
c = depth to neutral axis = a / β =	2.0915	in.		
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 =	163512	ft.-lb.		
ϕ=	0.9			21.2.1
ϕMn = Ultimate Moment Cap =	147161	ft.-lb.	R10.3.5	
check against ρ min =	0.0033	< ρ		9.6.1.3
check against ρ max =	0.0000	<== 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. ϕMn x 0.75 = 1 / 1.33 x ϕMn = reduction if ρ < ρ min)
(note: code says ρ 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)

ϕMn = Ultimate Moment Cap = ***147160.6*** *lb-ft*

Mu = $\boxed{127375.9}$ lb ft > fMn - Lid flexure NO GOOD — Utilization = 0.87

4. Check Shear Capacity

**neglects Vs (shear reinforcement)

Vc = 2 * λ * (f'c)^.5 * b * d =		42664	lbs	22.5.5.1
ϕ = 0.75	ϕVc =	$\boxed{31998}$	lbs	21.2
ϕVc/2 = **For Beam Design (h > 10 in.)		15999	lbs / ft	11.4.6.1

Check if shear capacity is OK without considering shear reinforcement
(Environmental durability factor does not apply to concrete shear capacity) — 9.2.6

Vu = Vmax (U) = $\boxed{39448}$ lbs > fVc - Consider Shear Reinforcement
See Below for Reinforcement Check — 9.2.1

Vs required = Sd (Vu-fVc) = 10377.21 lbs. - Shear 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) - Milner WWTP
Subject:	Concrete Lid Design

Beam #2 Design - Over Influent EQ

Reference: ACI 350-06 | ACI 318-14

Span = 12.00 ft — 8.7.3

* Beam spans as shown above

Beam width = b =	24	in	
Beam height = h =	16	in	
bar diameter =	0.75	in	**#6 bar**
bar area =	0.44	in^2	
stirrup bar diameter =	0.375	in	**#3**
clear cover =	2	in	Self Weight of beam
d =	13.25	in	400 plf

Concentrated Load from container:

* Assume no shims support container above interior walls

DL =	10987.5	lbs
LL =	4800	lbs
SL =	6160	lbs
RL =	1600	lbs
W_{down} =	768	lbs
W_{up} =	-2912	lbs
Wind $_{(overturning)}$ =	2380	lbs

Try 5 #6 bar in beam

bar spacing = 4.8125 in

Factored Loads * Sd (see below for Sd factor) — 9.2.1

				without Sd
(9-2)	1.2DL+1.6LL+0.5SL	31565.1	lbs	17387.5
(9-3)	1.2DL+1.6SL+1.0LL	31846.7	lbs	17387.5
(9-4)	1.2DL+1.6W+1.0LL+0.5SL	31799.9	lbs	18155.5
(9-5)	1.2DL+1.0E+1.0LL+0.2S	31367.0	lbs	17958.85

0.052
Seismic vertical component

Use P = 31846.7 lbs

Modification Factor used in above load calculations

Sd = $\phi Fy/(\gamma fs)$

ϕ =	0.9		(modification factor for tension-controlled section)	R9.2.6
Fy =	60,000	psi	(Steel yield strength)	
γ =	U factor		(Ultimate Load Factor (LRFD))	
fs =	20,000	psi	(Steel direct hoop and tensile strength)	

$$fs_{max} = \frac{320}{B\ (s^2 + 4(2+db/2)^2)^{0.5}}$$ — 10.6.4.1

For "Normal" environment exposure: Coating to be applied to concrete surface, per Aquaworks, Sherwin Williams Dura-Plate 6000. Therefore environment is NORMAL — 10.6.4.4

B =	1.35			
db =	0.75	in	#6 bar	
s =	4.8125	in spacing		
fs_{max} =	35055	psi	(for flexural stress)	10.6.4
	20000	psi	(for direct and hoop tensile stress in normal exposure)	9.2.6.2
	24000	psi	(for shear stress carried by **shear reinforcement**)	9.2.6.4

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

Reference

ACI 350-06	ACI 318-14
9.2.6	

Sd(min) = 1.0

Sd Factor - Bending		
U factor	Sd	Sd*U
1.2	1.16	1.393
1.6	1.00	1.600
1.4	1.00	1.400
0.5	2.79	1.393
1	1.39	1.393
0.2	6.96	1.393

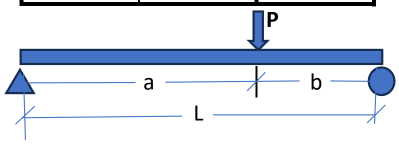
Sd Factor - Shear		
U factor	Sd	Sd*U
1.2	1.88	2.250
1.6	1.41	2.250
1.4	1.61	2.250
0.5	4.50	2.250
1	2.25	2.250
0.2	11.25	2.250

Moment at mid-span

Mu = Pab/L = 104076.4 lb ft

Shear at support

Vu = Pa/L = 32279 lbs



a= 6.75 ft

b= 5.25 ft

L= 12.00 ft

Fy = 60,000 psi

f'c = 4,500 psi   Beam Reaction = 20314 lbs

Check Lid Flexure   Mn > Mu

ρ min =	0.00333	<== See ρ min calcs below (For Slabs ρmin is T&S Steel)	Table 7.6.1.1
ρ = As / (b*d)	0.00692	<== If ρ is less than ρ min, see below	

See Table 21.2.2 For Strain Boundaries when (εt) is compression controlled, transition or tension controlled — Tbl. 21.2.2

No   Compression Controlled If εt < εty

No   Transition Zone If $\epsilon_{ty} < \epsilon_t < 0.005$

Yes   Tension Controlled Since (εt) = or > 0.005

				ACI 350-06	ACI 318-14
** Solve for phiMn Based on Whitney Stress Block					Fig R21.2.2a
a = stress block depth = As*Fy / (0.85*f'c*beff) =	1.4379	in.			
c = depth to neutral axis = a / β =	0.6875	in.			
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 =	137842	ft.-lb.			
φ=	0.9				21.2.1
φMn = Ultimate Moment Cap =	124057	ft.-lb.		R10.3.5	
check against ρ min =	0.0033	<	ρ		9.6.1.3
check against ρ max =	0.0000	<== determine ρ max eqn			
If below ρ min, multiply As x 1.33 =	2.9260	in^2			9.6.1.3
if below ρ min, multiply phiMn x 0.75 =	93043	lb-ft			

in. φMn x 0.75 = 1 / 1.33 x φMn = reduction if ρ < ρ min)

(note: code says ρ 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)

φMn = Ultimate Moment Cap = ***124057.4*** *lb-ft*

Mu = 104076.4 lb ft   < fMn - Lid Flexure OK   Utilization = 0.84

		Reference ACI 350-06	Reference ACI 318-14
4. Check Shear Capacity			
**neglects Vs (shear reinforcement)			
Vc = 2 * λ * (f'c)^.5 * b * d =	42664 lbs		22.5.5.1
ϕ = 0.75	ϕVc = 31998 lbs		21.2
ϕVc/2 = **For Beam Design (h > 10 in.)	15999 lbs / ft		11.4.6.1
Check if shear capacity is OK without considering shear reinforcement			
(Environmental durability factor does not apply to concrete shear capacity)		9.2.6	
Vu = Vmax (U) = 32279 lbs	< Vu - Beam Shear NO GOOD		
	See Below for Reinforcement Check	9.2.1	
Vs required = Sd (Vu-fVc) = 391.7131 lbs. - Shear 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 =	1.268809		
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

Container Dimensions:
(40 ft long x 8 ft wide x 9.5 ft tall)
(10 ft long x 8 ft wide x 9.5 ft tall)

length	width	height
40	8	9.5
10	8	9.5

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

(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 www.hilti.com

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Jun 26, 2023	Date:	11/6/2023
Fastening point:			

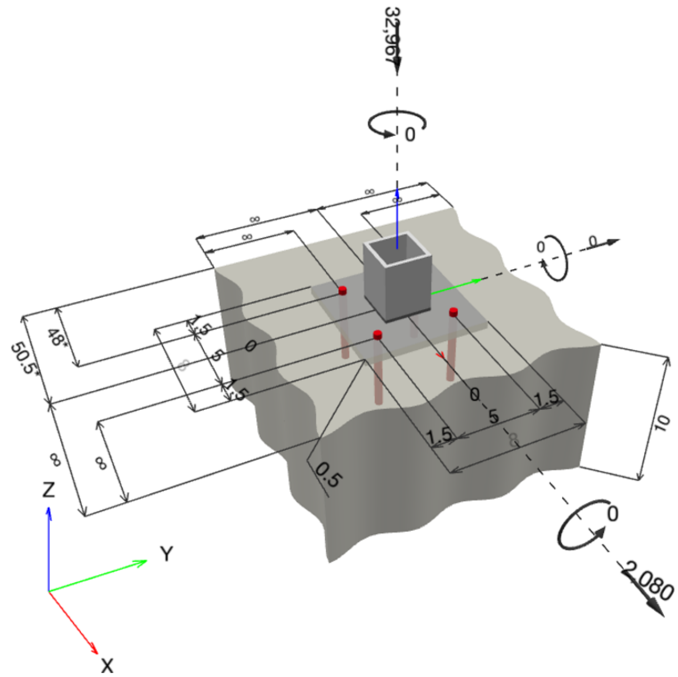
Specifier's comments:

1 Input data

Anchor type and diameter:	**AWS D1.1 GR. B 1/2**
Item number:	not available
Effective embedment depth:	h_{ef} = 5.000 in.
Material:	
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-14 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[CBFEM] :	l_x x l_y 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]



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	$N = -32{,}967$; $V_x = 2{,}080$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$;	no	7

2 Proof I Utilization (Governing Cases)

Loading	Proof	Design values [lb] Load	Capacity	Utilization β_N / β_V [%]	Status
Tension	-	-	-	- / -	N/A
Shear	Steel Strength	527	8,281	- / 7	OK

Loading	β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	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!

4 Remarks; Your Cooperation Duties

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

Reference

ACI 350-06	ACI 318-14

Foundation Design

Total weight of tanks and containers / Total slab area

Tank dimensions

W	L	H	Volume	
23.4167	12	7.50	2107.5	ft^3
18	12	7.50	1620.0	ft^3
11.25	12	7.50	1012.5	ft^3

Total volume = 4740.0 ft^3

Large Container: 43975 lbs

Small Container: 10781 lbs

Walls

	Length	height	thickness	Weight			
Interior			1	0	lbs		
	12	7.50	1	13500	lbs		
	12	7.50	1	13500	lbs	Total	
	12	7.50	1	13500	lbs	40500	lbs
Exterior							
	56.667	7.50	1	63750	lbs		
	56.667	7.50	1	63750	lbs		
	12	7.5	1	13500	lbs	Total	
	12	7.50	1	13500	lbs	154501	lbs

Lid	Length	Width	Thickness		
	56.667	14	12	119001	lbs

Fnd

Slab	Length	Width	Thickness		
(ft)	58.667	16	1.25	176001	lbs

1.00 ft projection outside of walls

Liquid Density: 70 pcf 331800 lbs (Assume liquid up to bottom of lid (conservative))

Live Load = containers 60 psf x 40x8 = 19200 lbs

lid 60 psf x remainder of lid 28400 lbs

Total Live Load = 47600 lbs

Snow Load =

SL = 77 psf 61087 lbs

Wind Down:

Pressure = 9.6 psf 7616 lbs

Note: Tank liquid acts in opposing direction to the soil pressure against the foundation for flexural design of the foundation. So liquid load left out of equation for flexural design

Total Load = 992862 lbs

876559 Total DL

(Conservatively consider cumulative load instead of ASD Load Combinations)

Slab area = 938.672 ft^2

Bearing pressure = 1057.7 psf

DL Pressure = 933.8 psf

Allowable bearing pressure = 3000 OK

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

Consider Flexure of foundation slab

Reference: ACI 350-06 | ACI 318-14

From RISA 3D Model, moment in footing from 781.3 psf soil pressure and moments at base of wall transferring into slab, times Sd
yields a moment force in the slab of: 45000 lb ft

Slab thickness = 15
Shear
Clear cover (top) = 3 in
11382 lb per RISA model using multi-span
bar diameter = 0.875 in
Simple Span moment = 14063 lb ft
d = 11.5625 in
Simple Span shear = 4688 lb ft
f'c = 4500 psi

Factored Loads * Sd (see below for Sd factor) — 9.2.1

				without Sd
(9-2)	1.2DL+1.6LL+0.5SL	1616462	lbs	694707
(9-3)	1.2DL+1.6SL+1.0LL	1616462	lbs	733342
(9-4)	1.2DL+1.6W+1.0LL+0.5SL	1637026	lbs	678332
(9-5)	1.2DL+1.0E+1.0LL+0.2S	1731843	lbs	690554

0.052 Seismic vertical component

Use w = 781.3 psf — 1844.9932
Factored load, without Sd — with Sd

Modification Factor used in above load calculations

Sd = $\phi Fy/(\gamma fs)$

ϕ = 0.9 (modification factor for tension-controlled section) — R9.2.6
Fy = 60,000 psi (Steel yield strength)
γ = U factor (Ultimate Load Factor (LRFD))
fs = 20,000 psi (Steel direct hoop and tensile strength)

$$fs_{max} = \frac{320}{B\ (s^2 + 4(2+db/2)^2)^{0.5}}$$ — 10.6.4.1

For "Normal" environment exposure: Coating to be applied to concrete surface, per Aquaworks, Sherwin Williams Dura-Plate 6000. Therefore environment is NORMAL — 10.6.4.4

As = 0.6 in^2
B = 1.35
db = 0.875 in — As = 0.6 in^2 #7
s = 12 in spacing

fs_{max} = 20000 psi (for flexural stress) — 10.6.4
20000 psi (for direct and hoop tensile stress in normal exposure) — 9.2.6.2
24000 psi (for shear stress carried by **shear reinforcement**) — 9.2.6.4

Sd Factor - Bending		
U factor	Sd	
1.2	2.25	2.700
1.6	1.69	2.700
1.4	1.93	2.700
0.5	5.40	2.700
1	2.70	2.700
0.2	13.50	2.700

Sd Factor - Shear		
U factor	Sd	
1.2	1.88	2.250
1.6	1.41	2.250
1.4	1.61	2.250
0.5	4.50	2.250
1	2.25	2.250
0.2	11.25	2.250

Date:	11/6/2023
Job:	2310219(3) - Milner WWTP
Subject:	Foundation

Check Slab Flexure   Mn > Mu

ρ min =	0.00333	<== See rho min calcs below (For Slabs rhomin is T&S Steel)	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 (εt) is compression controlled, transition or tension controlled   Tbl. 21.2.2

No	Compression Controlled If εt < εty
No	Transition Zone If $\epsilon_{ty} < \epsilon_t < 0.005$
Yes	Tension Controlled Since (εt) = or > 0.005

** Solve for phiMn Based on Whitney Stress Block   Fig R21.2.2a

a = stress block depth = As*Fy / (0.85*f'c*beff) =	0.7843	in.		
c = depth to neutral axis = a / β =	0.9507	in.		
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 =	33324	ft.-lb.		
φ=	0.9			21.2.1
φMn = Ultimate Moment Cap =	29991	ft.-lb.	R10.3.5	
check against ρ min =	0.0033	< ρ		9.6.1.3
check against ρ max =	0.0150	<== determine rho max eqn		
If below ρ min, multiply As x 1.33 =	0.7980	in^2		9.6.1.3
if below ρ min, multiply phiMn x 0.75 =	22493	lb-ft		

in. φMn x 0.75 = 1 / 1.33 x φMn = reduction if ρ < ρ min)
(note: code says ρ 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)

φMn = Ultimate Moment Cap = **29991.2** *lb-ft*

Mu (Sd) = 19039   < fMn - Lid Flexure OK   Utilization = 0.63

Note:
4500 psi is minimum concrete strength for concrete   Table 4.3.1
with low permeability (concrete is coated
is coated as directed by Aquaworks)

Check ρ balanced and εt=.005 limits:

β= Max of ((0.85-0.05*(f'c-4000)/1000,0.65)) =	0.8250		10.2.7.3
εty = tension yield strain = fy / Es	0.0021	<== Es = 29,000,000 psi	R10.3.2
εc = balanced concrete strain =	0.0030		R10.3.2
εt = actual tension strain = ((d-c) / c)*strain sc	0.0055		R10.3.3

ρ temp = temp and shrink steel =	0.0020	<== if Fy<60,000 psi, .002 -or- .0018 * 60,000psi / Fy	Table 7.12.2.1
As(temp)= (ρ temp * beff * h) =	0.0608	in^2	

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
ρt = (.85*β*f'c / Fy)*(0.003/(0.008))	0.0197	<== Max reinf. Ratio, when steel strain >= .005	
As (max) = (ρt * beff * d) =	0.5991	in^2	R10.3.5

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

Check Slab Flexure   Mn > Mu

ρ min =	0.00333	<== See rho min calcs below (For Slabs rhomin is T&S Steel)		Table 7.6.1.1
ρ = As / (b*d)	0.00432	<== If ρ is less than ρ min, see below		

See Table 21.2.2 For Strain Boundaries when (εt) is compression controlled, transition or tension controlled   Tbl. 21.2.2

No	Compression Controlled If εt < εty
No	Transition Zone If $\epsilon_{ty} < \epsilon_t < 0.005$
Yes	Tension Controlled Since (εt) = or > 0.005

** Solve for phiMn Based on Whitney Stress Block				Fig R21.2.2a
a = stress block depth = As*Fy / (0.85*f'c*beff) =	0.7843	in.		
c = depth to neutral axis = a / β =	0.9507	in.		
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 =	33511	ft.-lb.		
ϕ=	0.9			21.2.1
ϕMn = Ultimate Moment Cap =	30160	ft.-lb.	R10.3.5	
check against ρ min =	0.0033	< ρ		9.6.1.3
check against ρ max =	0.0150	<== determine rho max eqn		
If below ρ min, multiply As x 1.33 =	0.7980	in^2		9.6.1.3
if below ρ min, multiply phiMn x 0.75 =	22620	lb-ft		

in. ϕMn x 0.75 = 1 / 1.33 x ϕMn = reduction if ρ < ρ min)
(note: code says ρ 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)

ϕMn = Ultimate Moment Cap =	***30159.9***	*lb-ft*	
Mu (Sd) = 14063	< fMn - Lid Flexure OK	Utilization =	0.47

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

Check ρ balanced and εt=.005 limits:

β= Max of ((0.85-0.05*(f'c-4000)/1000,0.65)) =	0.8250		10.2.7.3
εty = tension yield strain = fy / Es	0.0021	<== Es = 29,000,000 psi	R10.3.2
εc = balanced concrete strain =	0.0030		R10.3.2
εt = actual tension strain = ((d-c) / c)*strain sc	0.0055		R10.3.3

ρ temp = temp and shrink steel =	0.0020	<== if Fy<60,000 psi, .002 -or- .0018 * 60,000psi / Fy	Table 7.12.2.1
As(temp)= (ρ temp * beff * h) =	0.0608	in^2	

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
ρt = (.85*β*f'c / Fy)*(0.003/(0.008))	0.0197	<== Max reinf. Ratio, when steel strain >= .005	
As (max) = (ρt * beff * d) =	0.5991	in^2	R10.3.5

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

			Reference 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
ρ min.= greater of ρ min(a) or ρ min(b) =	0.0034	<== Min. reinf. Ratio		
As (min)= (ρ 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 * λ * (f'c)^.5 * b * d =			18615 lbs		22.5.5.1
ϕ =	0.75	ϕVc =	13961 lbs		21.2

Check if shear capacity is OK without considering shear reinforcement

(Environmental durability factor does not apply to concrete shear capacity) — 9.2.6

Vu = Vmax (U) = 4688 lbs — < fVc - Slab Shear OK — Utilization = 0.34

Vu = wl/2 — See Below for Reinforcement Check — 9.2.1

Vs required = Sd (Vu-fVc) = -20866.3 lbs. - No shear reinforcement required — R9.2.6.4

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

Sd Mu =	45000	lb ft		
ϕMn = AsFy(d-a/2) =	402132	lb ft	Slab Flexure OK	
As =	0.6	in^2	0.6	#7 bars
Fy =	60,000	psi		
f'c =	4500	psi		
b =	12	in		
a =	0.784314	in		

Check Slab shear — ϕVn>Vu+Vs/Sd

Vc = 2 * (f'c)^(0.5) *b*d

ϕ = 0.75

ϕVc = 13961 lbs — Slab Shear OK

Use 18 in slab w/ #7 bars at 12 on center

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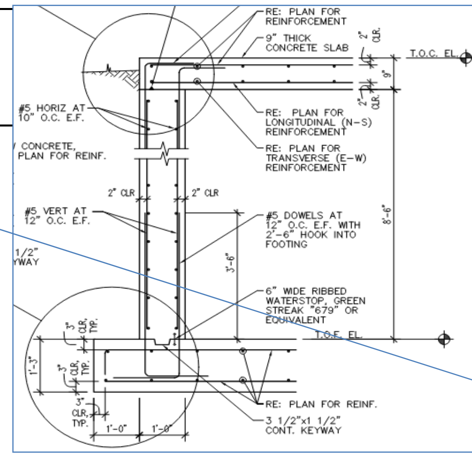
Created 8/29/11

Date	11/9/2023	Sheet No.	of
Project	Steamboat Mountain School		
Subject	Bouyancy Calc		

CHECK CONCRETE BUOYANT DEADMAN FOR BURIED TANK References

ASSUMPTIONS:

TANK IS EMPTY
TANK IS COMPLETELY SUBMERGED
NO SLAB



GIVEN INFO:

DRY WEIGHT OF TANK:		582333	lbs
ASSUMED WEIGHT OF SOIL:		110	pcf
WEIGHT OF REINFORCED CONCRETE:		150	pcf
DENSITY OF fluid:		62.4	pcf
DIMENSIONS OF TANK:			
	Length:	56.67	ft
	Round (R) or Square (S)	S	
	Width:	14.00	ft
	Height:	7.50	ft
	Diameter:	10.00	ft
DEPTH OF SOIL ABOVE TANK:		0.00	in
DEPTH OF SOIL ABOVE DEADMAN:		0.00	in
(used min. depth with no traffic)			
ASSUMED SLAB DEPTH:		0.00	in
ASSUMED BUOYANT DEADMAN WIDTH:			
	Assumed Deadman Width:	0.00	ft
DEPTH OF SOIL ABOVE WATER TABLE:		0.00	in

EVEN IF THE SOIL IS COMPLETELY SATURATED TO THE TOP OF THE TANK, DEADMAN NOT REQUIRED. WEIGHT OF TANK (FOOTING, WALLS, LIDS, SURROUNDING SOIL, AND DL OF METAL BUILDING) EXCEEDS WT OF EQUAL VOLUME OF WATER (CELL C55)

DETERMINE VOLUMES:

		0	* Insert "0" if tank volume is to be calculated from "Given Info" above. Otherwise, insert known volume
TANK + FOUNDATION:	Calculated Value:	5908.79 ft^3	*This cell can be overridden if tank volume is known (should include foundation volume)
SOIL:	Tank + Fdn Volume:	5908.793 ft^3	
above tank:		0.00 ft^3	
buoyant soil above deadman:		0.00 ft^3	
dry soil above deadman:		0.00	
4" SLAB:			
above tank:		0.00 ft^3	
TOTAL VOLUME (TANK+FND):		**5908.79 ft^3**	7358.04 ft^3 (THIS IS THE TOTAL VOLUME OF SYSTEM)
			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 BUOYANT FORCES AND GRAVITY FORCES:

(wt. of equal volume of water - total weight)	**-381688.6511 lbs**	OK WITHOUT SF
Required Flotation Safety Factor (SF):	1.25	[ACI 350.4R-04 3.1.2]
Actual Safety Factor (Total Weight/Buoyant Force):	**2.04**	**OK, DEADMAN NOT REQUIRED**

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)	

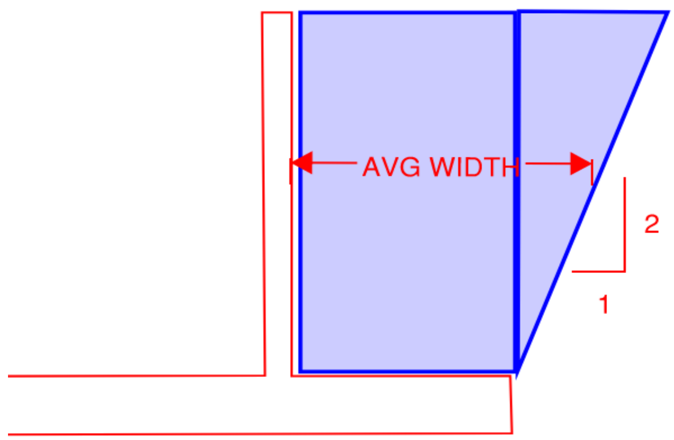
FOUNDATION CONCRETE VOLUME: 1408.008

WEIGHT CALCULATION (EMPTY TANK)

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

Soil weight Calculation. **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
	Total Volume =	3500.3269	ft^3



soil weight	110 pcf	Total Weight =	385036 lbs	(using total soil weight)
buoyant soil weight	47.6 pcf		166615.6 lbs	(using bouyant soil weight)