wallace design collective

COMMUNITY OF PHIPPSBURG
WWTP IMPROVEMENT PROJECT
PHIPPSBURG, CO
PROJECT NO. 2310219 (4)

STRUCTURAL CALCULATIONS

COLORADO LICENSED
DANIEL E. MAZZEI
PE.0058764
PROFESSIONAL ENGINEER

2023.11.13 16:56:10-06'00'

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

wallace design collective

TABLE OF CONTENTS

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

01 DESIGN LOADS

wallace design collective

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

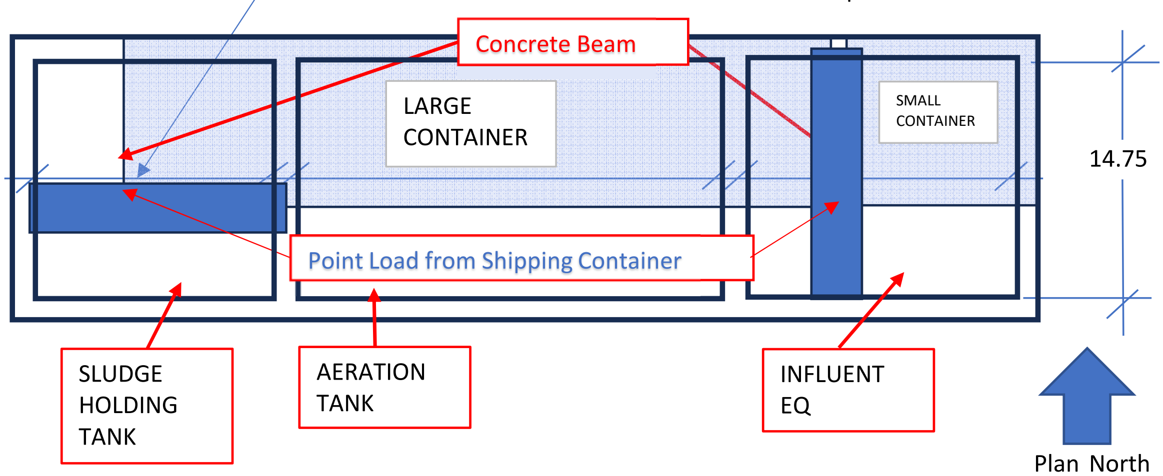
Date:	11/6/2023
Job:	2310219(4) - Phippsburg WWTP
Subject:	Tank layout

Plan

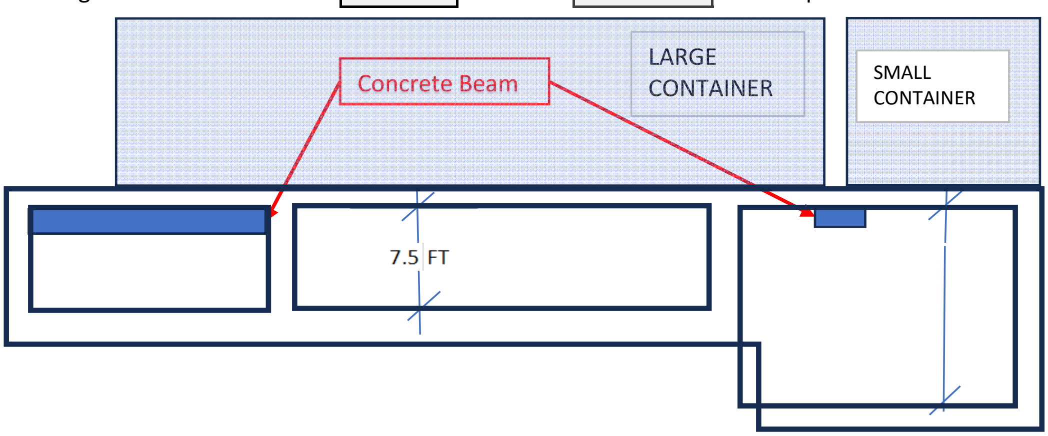
	Inside dimension		Main structural system for Lid:
Width	14.75	ft	(One-way slab spans Plan N-S)

Length

Tank				
Sludge Holding Tank	13.17	ft	Total length =	57.667 ft
Aeration Tank	22.00	ft		
Influent EQ	17.50	ft	Interior and exterior wall thickness =	12 in
			at deeper tanks =	12 in



Tank Height	7.50	ft	11.8109	ft at deeper tank



Backfill height at perimeter of wall:	12.8109	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	See Geotechnical Report from NWCC, Inc
Modulus of Subgrade Reaction (Ks):	150	pci (assumed)	Job No 22-12813 Dated January 17, 2023
Coefficient of Friction for Sliding:	0.4		5,000 w/ 700 min DL pressure if bearing on
Passive Lateral Pressure;	250	pcf	claystone shale bedrock or struct fill
Active Lateral Pressure:	50	pcf	(granular backfill, 70 for on-site soils)
Equivalent Fluid Density	250	pcf	(ALP = 85 / 97 if below groundwater table)
Frost depth:	48	in	

CODE CHECK

DATE:	7/12/23		
TO:	Routt County Building Department Routt County 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 - Seismic provisions
	C.	Do State Building Code Requirements Differ?	Yes - Seismic provisions
	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:	71 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:	0.524 (short period, 0.2s)
	B.	Mapped Spectral Response Acceleration, S1:	0.097 (long period, 1.0s)
6. FROST DEPTH			
	A.	Minimum Bearing Depth:	48 in.

REMARKS: Phippsburg, CO
2021 IBC is being adopted Effective January 1, 2024
Snow Load used is max between Routt County Snow Load Map tool and ASCE 2016 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:

ASCE 7 Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: III
Soil Class: B - Rock

Latitude: 40.240691
Longitude: -106.942071
Elevation: 7404.36006946248 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:	Mon Oct 30 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.

![ASCE American Society of Civil Engineers]

Seismic

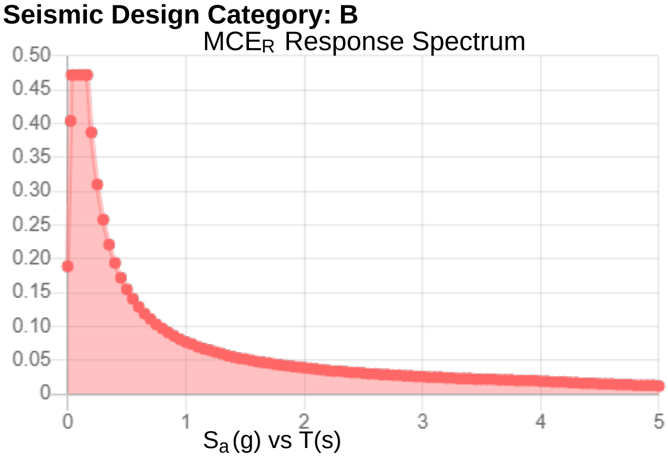
B - Rock

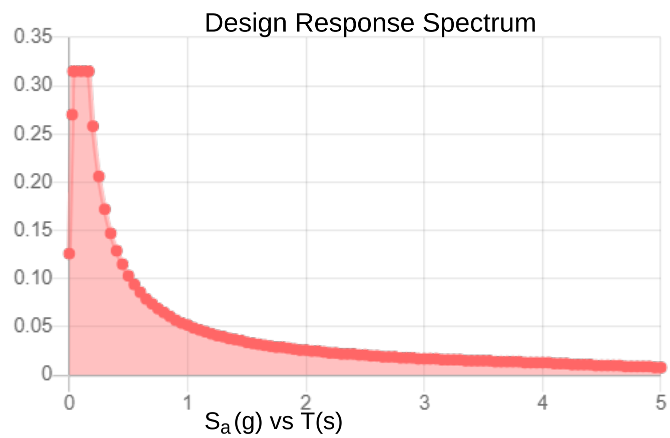
Site Soil Class:

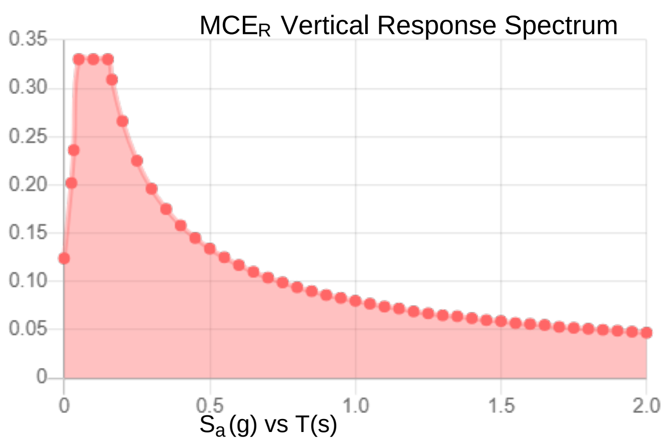
Results:

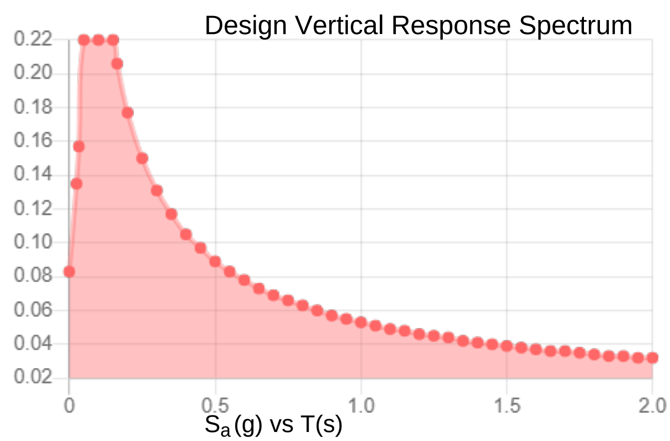
S_S :	0.524	S_{D1} :	0.052
S_1 :	0.097	T_L :	4
F_a :	0.9	PGA :	0.356
F_v :	0.8	PGA_M :	0.32
S_{MS} :	0.472	F_{PGA} :	0.9
S_{M1} :	0.077	I_e :	1.25
S_{DS} :	0.315	C_v :	0.875

Seismic Design Category: B









Data Accessed: **Mon Oct 30 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.

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Snow

Results:

Mapped Elevation:

Data Source:

Date Accessed: Mon Oct 30 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.

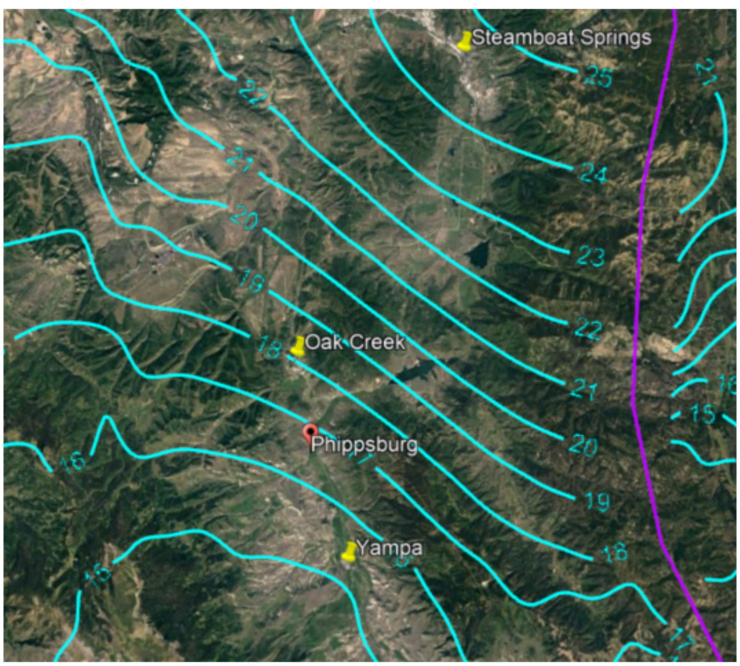
NOTE:
Routt County Snow Mapper yields value of 71 psf for snow load at this location

SEAC Snow Load Committee Equation:

$p_{g,\ site} = \max(k_{(site)}/100 * A^3) = 16.8/100 * 7.4^3 =$ **68.1 psf**

$k_{(site)} = 16.8$ (see map below)

A = 7.4 (thousands of feet in elevation)



Use 71 psf

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.

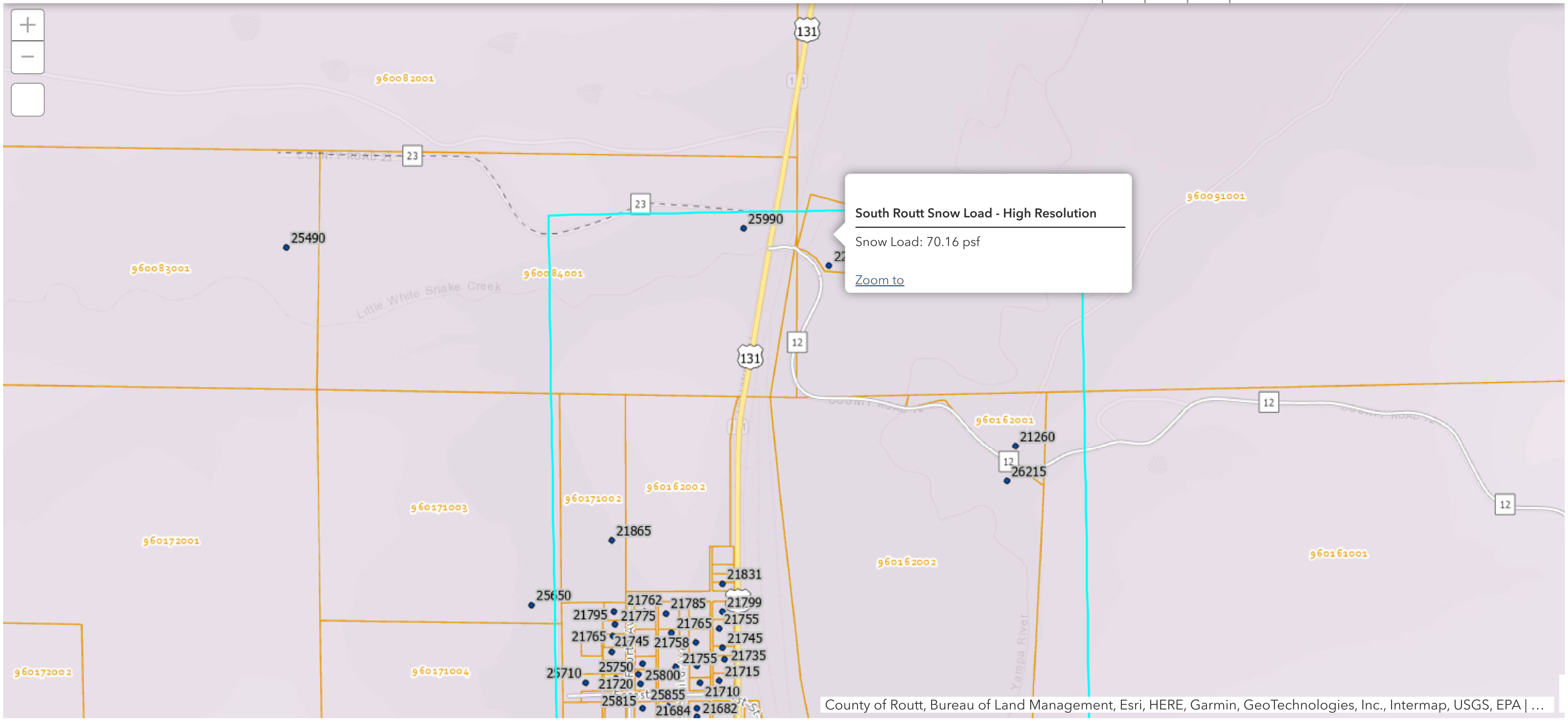
ASCE does not intend, nor should anyone interpret, the results provided by this Tool to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting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

Routt County Snow Load Mapper

Routt County Building Department

phippsburg



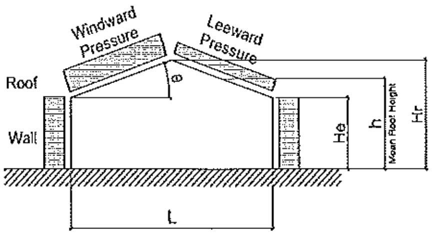
600ft

-106.940 40.243 Degrees

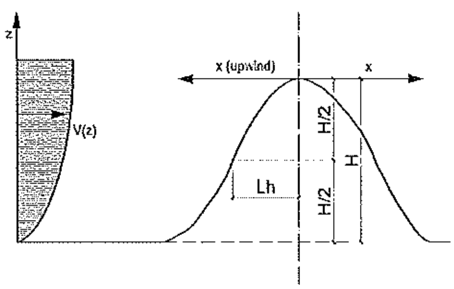
Date	10/30/2023	Sheet	of
Job	Phippsburg, CO		
Subject	Wind Loads		

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 (θ ≤ 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**

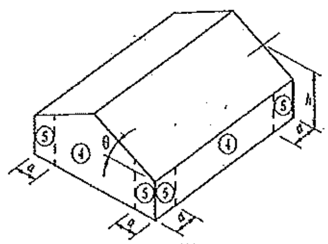
Roof All Other Conditions	**qh**	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
For 0 to h/2 = 0 ft to 4.75 ft	19.33	-1.09	0.18	**-24.6 psf**	**-14.8 psf**
>h/2 ft	19.33	-0.60	0.18	**-15 psf**	**-9 psf**
	19.33			**#VALUE!**	**#VALUE!**
	19.33			**#VALUE!**	**#VALUE!**

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**

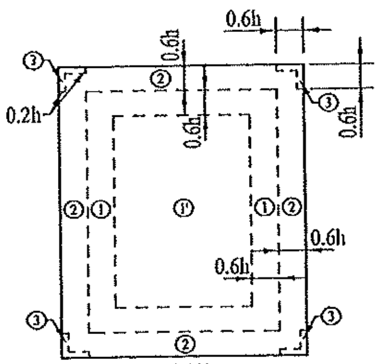
WALLACE DESIGN PROGRAM
Revised: 02/12/2019
Author: Katie Faulkner

Date	10/30/2023	Sheet	of
Job	Phippsburg, CO		
Subject	Wind Loads		

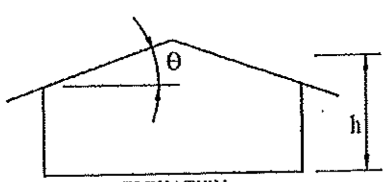
WIND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD
ASCE 7-16, Chapters 27 and 30



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 =	200.00	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	**0.79**	Table 26.9-1	
G, gust factor =	**0.85**	Section 26.11.4	
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: trib. Area = 10 sq. ft.		qh	GCp	GCpi	(1.0)P	(0.6)P
Zone 4	Interior Zone	19.33	-0.99	0.18	**-22.6 psf**	**-13.6 psf**
Zone 5	End Zone	19.33	-1.26	0.18	**-27.8 psf**	**-16.7 psf**
Zone 4 and 5		19.33	0.90	-0.18	**20.9 psf**	**12.5 psf**

Walls: trib. Area = 200 sq. ft.		qh	GCp	GCpi	(1.0)P	(0.6)P
Zone 4	Interior Zone	19.33	-0.78	0.18	**-18.6 psf**	**-11.2 psf**
Zone 5	End Zone	19.33	-0.85	0.18	**-19.8 psf**	**-11.9 psf**
Zone 4 and 5		19.33	0.69	-0.18	**16.9 psf**	**10.1 psf**

Parapets: trib. Area = 10 sq. ft.			qp	GCp	GCpi	(1.0)P	(0.6)P
Case A	Zone 4	Interior Zone	19.33	2.97	0.00	**57.4 psf**	**34.5 psf**
	Zone 5	End Zone	19.33	3.78	0.00	**73.1 psf**	**43.8 psf**
Case B	Zone 4	Interior Zone	19.33	1.89	0.00	**36.5 psf**	**21.9 psf**
	Zone 5	End Zone	19.33	2.16	0.00	**41.8 psf**	**25.1 psf**

Parapets: trib. Area = 50 sq. ft.			qp	GCp	GCpi	(1.0)P	(0.6)P
Case A	Zone 4	Interior Zone	19.33	2.53	0.00	**48.8 psf**	**29.3 psf**
	Zone 5	End Zone	19.33	3.00	0.00	**58 psf**	**34.8 psf**
Case B	Zone 4	Interior Zone	19.33	1.67	0.00	**32.2 psf**	**19.3 psf**
	Zone 5	End Zone	19.33	1.83	0.00	**35.3 psf**	**21.2 psf**

Roofs: trib. Area = 10 sq. ft.	qh	GCp	GCpi	(1.0)P	(0.6)P
Corner Zone (3)	19.33	-3.20	0.18	**-65.3 psf**	**-39.2 psf**
End Zone (2)	19.33	-2.30	0.18	**-47.9 psf**	**-28.8 psf**
Interior Zone (1)	19.33	-1.70	0.18	**-36.3 psf**	**-21.8 psf**
Interior Zone (1')	19.33	-0.90	0.18	**-20.9 psf**	**-12.6 psf**
Positive (All Zones)	19.33	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)	19.33	-2.14	0.18	**-44.9 psf**	**-26.9 psf**
End Zone (2)	19.33	-1.77	0.18	**-37.7 psf**	**-22.6 psf**
Interior Zone (1)	19.33	-1.29	0.18	**-28.4 psf**	**-17 psf**
Interior Zone (1')	19.33	-0.90	0.18	**-20.9 psf**	**-12.5 psf**
Positive (All Zones)	19.33	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)	19.33	-3.20	**-61.9 psf**	**-37.1 psf**
End Zone (2)	19.33	-2.30	**-44.5 psf**	**-26.7 psf**
Interior Zone (1)	19.33	-1.70	**-32.9 psf**	**-19.7 psf**
Interior Zone (1')	19.33	-1.70	**-32.9 psf**	**-19.7 psf**
---	---	---	---	---

Overhangs: trib. Area = 50 sq. ft.	qh	GCpn	(1.0)Pp	(0.6)Pp
Corner Zone (3)	19.33	-2.34	**-45.2 psf**	**-27.1 psf**
End Zone (2)	19.33	-1.81	**-34.9 psf**	**-21 psf**
Interior Zone (1)	19.33	-1.63	**-31.5 psf**	**-18.9 psf**
Interior Zone (1')	19.33	-1.63	**-31.5 psf**	**-18.9 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)
3. If a parapet equal to 3 ft or higher is provided around the perimeter of a roof with a slope of ≤ 7°, the roof corner zones may be treated as end zones. (Fig. 30.3-2A, Footnote 5)

Date	11/9/2023	Sheet No.	of
Job	Phippsburg, CO WWTP		
Subject	Seismic Loads		

SEISMIC LOAD SUMMARY

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

1. Input

Spectral Response Acceleration for Short Periods, Ss =		**0.524**	
Spectral Response Acceleration for 1-second Periods, S1 =		**0.097**	
Risk Category =		**III**	(IBC Table 1604.5 & ASCE: Table 1-5-1)
Site Classification (A,B,C,D,E,F) =		**B**	(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' c	**No**	(Yes or No, ASCE 7 Section 11.6)
Does Structure have a flexible diaphragm?	**Yes**	(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. =	**56.667**	feet (input 0 for rigid diaphragm)
Span length of flexible diaphragm -y-dir. =	**14**	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**	(Table 12.2-1)
Deflection Amplification Factor, Cd =	**4**	(Table 12.2-1)

Acceleration for Short Period

Site Coefficient, Fa =	**0.90**	(IBC Table 1613.2.3(1), ASCE 7 Table 11.4-1)
Site Adjusted Spectral Response Acceleration for Short Periods, Sms =	**0.472**	(IBC Section 1613.2.3, ASCE 7 Section 11.4.4)

Acceleration for 1-Second Period

Site Coefficient, Fv =	**0.80**	(IBC Table 1613.2.3(2), ASCE 7 Table 11.4-2)
Site Adjusted Spectral Response Acceleration for 1-second Periods, Sm1 =	**0.078**	(IBC Section 1613.2.3, ASCE 7 Section 11.4.4)
Design Spectral Response Acceleration for Short Periods, Sds =	**0.314**	(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 =	**0.052**	(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 =	**0.165**	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)) =	**0.098**	(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 =	**0.098 W**	**0.069 W**
c. Horizontal Seismic Load, Section 12.4.2.1=		
For the X-direction: Eh=	**0.098 W**	**0.069 W**
For the Y-direction: Eh=	**0.098 W**	**0.069 W**
d. Vertical Seismic Load Component, Section 12.4.2.2:		
Ev = 0.2 Sds D =	**0.063 D**	**0.044 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 =	**0.079 W**	**0.055 W**
but need not exceed 0.4*Ie*Sds wpx =	**0.157 W**	**0.110 W**
For a one story building, Fpx =	**0.098 W**	**0.069 W**
f. For collector elements in Seismic Design Categories C through F, Section 12.10.2		
Emh = Ωo V =	**0.197 W**	**0.138 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	10/30/2023	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. =	**0.052 Wp**	**0.037 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 =	**0.157 Wp**	**0.110 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 =	**1.14**			
Fp= 0.4 S_{ds} k_a I_e W_p or .2 k_a I_e Wp min. =		For Connections at Flexible Diaphragms:	**0.285 Wp**	**0.200 Wp**
Fp * 1.4 for steel elements per 12.11.2.2.2			**0.399 Wp**	**0.279 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=	**0.503 Wp**	**0.755 Wp**	(Equation 13.3-2)
Fp min = 0.3 Sds Ip Wp=	**0.094 Wp**	**0.141 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**	**0.377 Wp**	**0.264 Wp**	(Table 13.5-1)
2. Braced Interior Non-masonry walls and partitions							
Fp at floor=	**1.00**	**2.50**	**1.00**	**0.00**	**0.094 Wp**	**0.066 Wp**	(Table 13.5-1)
Fp at roof=	**1.00**	**2.50**	**1.00**	**1.00**	**0.151 Wp**	**0.106 Wp**	(Table 13.5-1)
Fp average at roof and floor:					**0.123 Wp**	**0.086 Wp**	
3. Braced Interior Unreinforced masonry walls and partitions							
Fp at floor=	**1.00**	**1.50**	**1.00**	**0.00**	**0.094 Wp**	**0.066 Wp**	(Table 13.5-1)
Fp at roof=	**1.00**	**1.50**	**1.00**	**1.00**	**0.252 Wp**	**0.176 Wp**	(Table 13.5-1)
Fp average at roof and floor:					**0.173 Wp**	**0.121 Wp**	
4. Cantilevered (Unbraced) Interior Nonstructural walls	**2.50**	**2.50**	**1.00**	**0.00**	**0.126 Wp**	**0.088 Wp**	(Table 13.5-1)
5. Braced Parapets and Chimneys	**1.00**	**2.50**	**1.00**	**1.00**	**0.151 Wp**	**0.106 Wp**	(Table 13.5-1)
6. Exterior Nonstructural Wall Elements							
Fp at floor=	**1.00**	**2.50**	**1.00**	**0.00**	**0.094 Wp**	**0.066 Wp**	(Table 13.5-1)
Fp at roof=	**1.00**	**2.50**	**1.00**	**1.00**	**0.151 Wp**	**0.106 Wp**	(Table 13.5-1)
Fp average at roof and floor:					**0.123 Wp**	**0.086 Wp**	
For the Body of the Wall Panel Connection:							
Fp at floor=	**1.00**	**2.50**	**1.00**	**0.00**	**0.094 Wp**	**0.066 Wp**	(Table 13.5-1)
Fp at roof=	**1.00**	**2.50**	**1.00**	**1.00**	**0.151 Wp**	**0.106 Wp**	(Table 13.5-1)
For the fasteners of the connecting system:							
Fp at floor=	**1.25**	**1.00**	**1.00**	**0.00**	**0.157 Wp**	**0.110 Wp**	(Table 13.5-1)
Fp at roof=	**1.25**	**1.00**	**1.00**	**1.00**	**0.472 Wp**	**0.330 Wp**	(Table 13.5-1)
7. Appendages and Ornamentation	**2.50**	**2.50**	**1.00**	**1.00**	**0.377 Wp**	**0.264 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.

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

Date	10/30/2023	Sheet No.	of
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	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	1.29
D	1.60	1.40	1.20	1.10	1.00	1.00	1.38
E	2.40	1.70	1.30	1.20	1.20	1.20	1.66
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	0.80
B	0.80	0.80	0.80	0.80	0.80	0.80	0.80
C	1.50	1.50	1.50	1.50	1.50	1.40	1.50
D	2.40	2.20	2.00	1.90	1.80	1.70	2.40
E	4.20	3.30	2.80	2.40	2.20	2.00	4.20
F	---	---	---	---	---	---	---

IBC Table 1613.2.5(1) and 11.6-1
ismic 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	

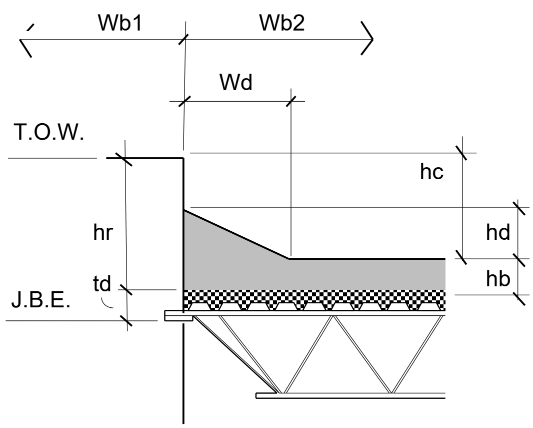
Table 1613.2.5(2) and 11.6-2
Seismic Design Category Based on 1-Second Period Response Acceleration

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	

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

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

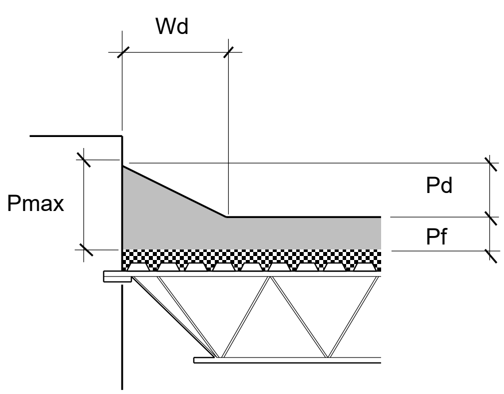
FLAT ROOF SNOW DRIFT - Joists Perpendicular to Wall
ASCE 7-16



Configuration

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

Use constant 220 psf snow load where drifts may occur



Snow Drift

1. **Input**

Dead Load =	**15**	psf
Roof Live Load =	**20**	psf
Pg, Ground Snow Load =	**70.16**	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)?	**y**	(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 =	**40.00**	feet
Wb2, length of lower roof =	**2000.00**	feet
S, Joist Spacing =	**1.00**	feet
L, Joist Span =	**22.50**	feet

2. **Balanced Snow Load Check**

Is, Importance Factor =	**1.10**	Table 1.5-2
Pf = 0.7 Ce Ct Is Pg =	**54.02**	psf (7.3-1)
Pm = Is Pg =	**22.00**	psf (7.3.4)
Rain on snow surcharge =	**0.00**	psf (7.10)
Pmin =	**54.02**	psf

3. **Drifted Snow Load Check**

Pf = Pg =	**77.18**	psf
D = 0.13 Pg + 14.0 ≤ 30 pcf =	**23.12**	pcf
hb = Pf/D =	**3.34**	ft
Wb =	**2000.00**	ft
hd = 0.75[0.43 Wb2^1/3 (Pg+10)^1/4-1.5] Is^2	**11.54**	ft
hd + hb =	**14.88**	ft
hr =	**9.50**	ft
hc = hr - hb =	**6.16**	ft
Wd = 4 hd or 4 [hd^2/hc] ≤ 8 hc =	**49.29**	ft
Pmax = D (hd + hb) ≤ D hr =	**219.63**	psf
Pd =D hd ≤ D hc =	**142.45**	psf

4. **Uniform Load Summary**

Drifted Snow Load

	Snow	Total	
R left =	**2227.0**	**2395.7**	lbs
R right =	**1983.1**	**2151.9**	lbs
M max =	**11850.8**	**12799.3**	ft-lbs
w base =	**70.2**	**85.2**	plf
w drift =	**149.5**	**164.5**	plf
w equiv =	**198.0**	**213.0**	plf *

Load Without Drift

	Snow	Total	
w (Pmin = 54.0232 ¡	**54.0**	**69.0**	plf

* indicates controlling load (drifted vs. undrifted)

wallace
design
collective

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

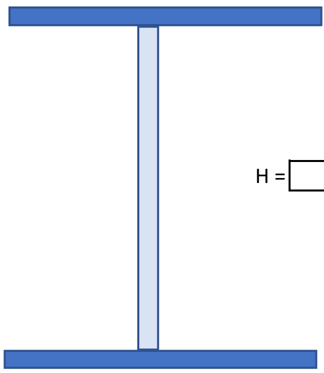
Date:	11/6/2023
Job:	2310219(4) - Phippsburg WWTP
Subject:	Tank Wall Design

Reference: ACI 350-06 | ACI 318-14

Interior Tank Wall

Short wall

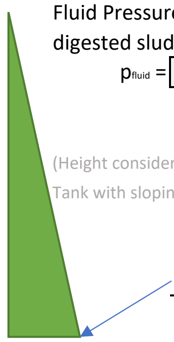
* Worst case condition - overflow one side of wall, other side of wall is empty
 - assume overflow condition is completely full to bottom of lid (conservative)



Lid

H = 7.81 ft

Base



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

p_{fluid} = 70 pcf

(Height considers low point of Sludge Tank with sloping slab)

w_{max}

Force at base of wall

H * p_{fluid} = 546.875 plf

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

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

Lid span, ℓ ℓ = 14.75 ft ℓ/30 = 5.9 in

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

Mmax =	=	1753 lb ft / ft	(triangular load, fixed
Vmax = W	=	1546 lbs/ft	base, pinned top)
			(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 =	0.5	in	(**#4** 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:	11/6/2023
Job:	2310219(4) - Phippsburg WWTP
Subject	Wall design

Reference: ACI 350-06 | ACI 318-14

Wall design

								ACI 350-06	ACI 318-14
Wall thickness =	12	in	As =	0.2	in^2	(#4 bar)			
Clear cover =	2.0	in	Fy =	60,000	psi	at 12" spacing		7.7.1	
Bar diameter =	0.5	in	f'c =	4500	psi			Table 4.2.2	
d =	9.75	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) 0.00171 <== 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

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) =	0.2614	in.	Fig R21.2.2a
c = depth to neutral axis = a / β =	0.3169	in.	
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 =	9619	ft.-lb.	
ϕ=	0.9		21.2
ϕMn = Ultimate Moment Cap =	**8657.4**	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 =	0.2660	in^2	9.6.1.3
if below ρ min, multiply phiMn x 0.75 =	6493.01	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 =		***6493.0147***	*lb-ft*
Mu = Mmax (U) Sd=	4733.1 lb ft	< fMn - Wall Flexure OK	Utilization = 0.73

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	0.0893		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) =	2.3076	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) =	0.3924	in.2 <== unless As > 1.33*As reqd per code		9.6.1.3

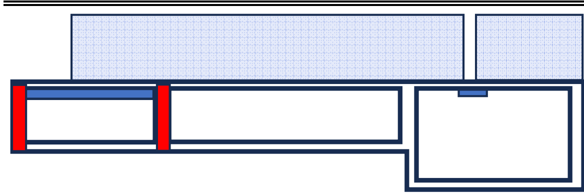
Date:	11/6/2023	
Job:	2310219(4) - Phippsburg WWTP	
Subject	Wall design	
	Reference	
	ACI 350-06	ACI 318-14

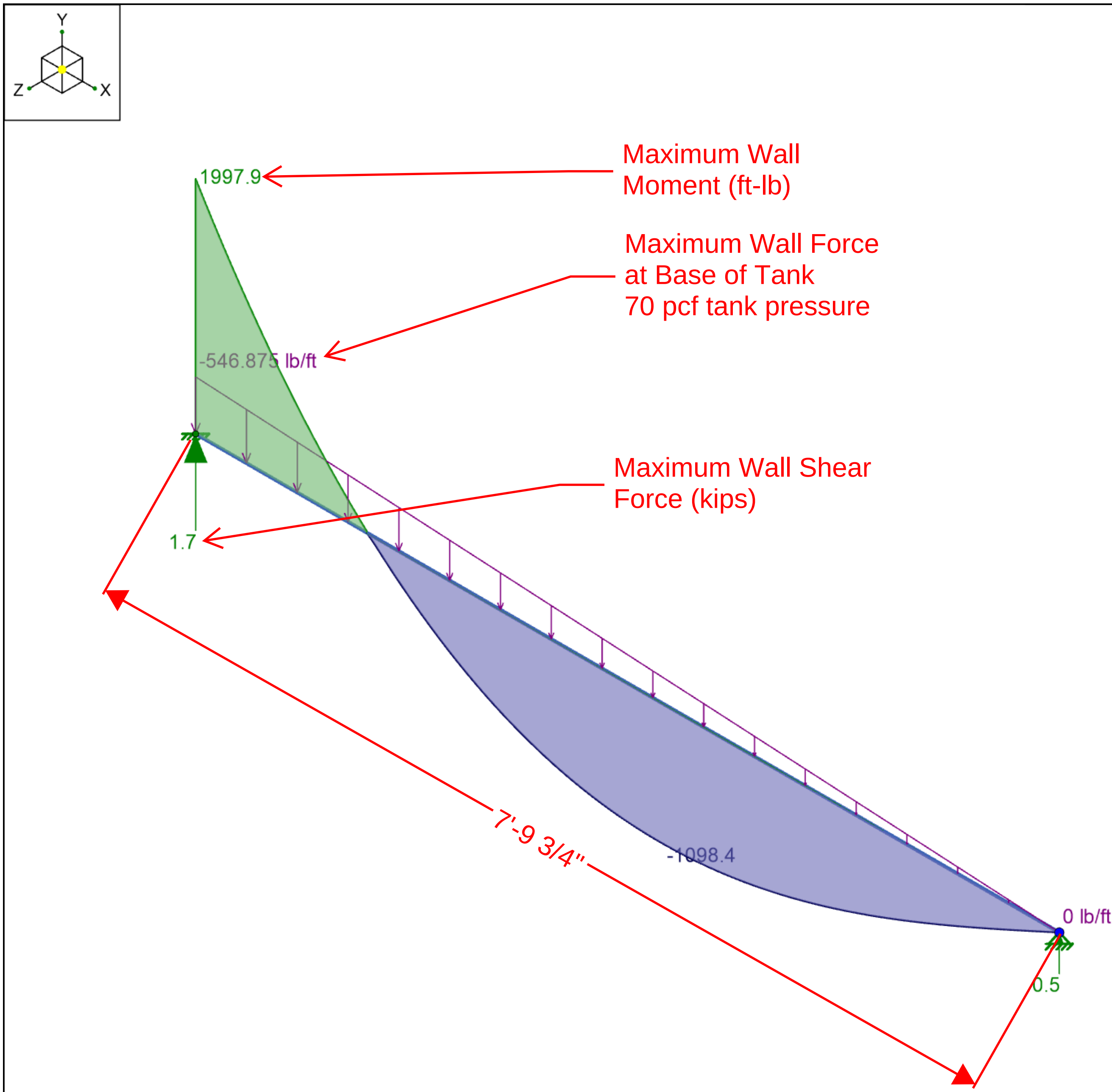
4. Check Shear Capacity

**neglects Vs (shear reinforcement)

				ACI 350-06	ACI 318-14
Vc = 2 * λ * (f'c)^.5 * b * d =		15697 lbs			11.2.1.1
ϕ =	0.75				21.2
ϕVc/2 =	**For Beam Design (h > 10 in.)	$\boxed{5886}$ 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) =	$\boxed{2473.6}$ lbs	> Vu - Wall Shear OK	Utilization = 0.42		
U (Fluid Pressure) =	1.6	No Reinforcement Check Needed		9.2.1	

Use 12 in wall w/ (#4 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

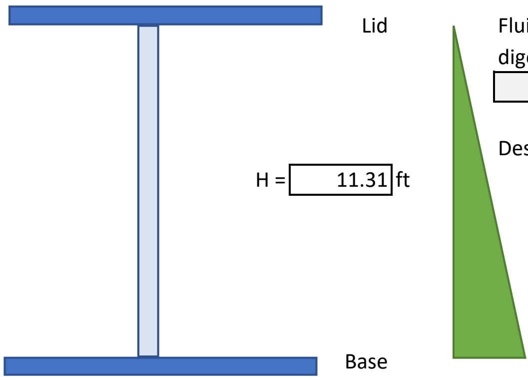
Wallace Engineering	Interior Tank Wall (7.5 ft) - 1 ft section typ	SK-17
S Jacob		Nov 06, 2023
2310219 (3)		Short interior wall.r3d

Date:	11/6/2023
Job:	2310219(4) - Phippsburg 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

Design is based on liquid height 6" below lid

Force at base of wall
H * p_{fluid} =
w_{max} = 791.763 plf

Minimum wall thickness = 12" <- Controls — 14.6.2
Minimum thickness = 6" or ℓ/30 — 14.6.1
Lid span, ℓ   ℓ = 14.75 ft   ℓ/30 = 5.9 in

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

Mmax = = 6694 lb ft / ft (triangular load, fixed
Vmax = W = 3601 lbs/ft base, pinned top)
(See RISA output)

Modification factor, Sd — 9.2.6

Sd = $\phi F_y/(\gamma f_s)$

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 = 9 in spacing

$f_{s\,max}$ = 23292 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	1.93	2.32
1.6	1.45	2.32
1.4	1.66	2.32

Mu = Mmax (U) Sd= 15519 lb ft

Date:	11/6/2023
Job:	2310219(4) - Phippsburg WWTP
Subject	Wall design

Reference: ACI 350-06 | ACI 318-14

Wall design

							ACI 350-06	ACI 318-14
Wall thickness =	12	in	As =	0.587	in^2	(#4 bar)		
Clear cover =	2.0	in	Fy =	60,000	psi	at 9" 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) 0.00508 <== If $\rho$is 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 $\epsilon t < \epsilon ty$	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) =	0.7669	in.	Fig R21.2.2a
c = depth to neutral axis = a / β =	0.9296	in.	
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 =	27109	ft.-lb.	
ϕ=	0.9		
ϕMn = Ultimate Moment Cap =	**24397.7**	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 =	0.7803	in^2	9.6.1.3
if below ρ min, multiply phiMn x 0.75 =	**18298.29**	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= 15519 lb ft — < fMn - Wall Flexure OK — Utilization = 0.85

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:

			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	0.0281		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

Date:	11/6/2023
Job:	2310219(4) - Phippsburg WWTP
Subject	Wall design

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	11621.963			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

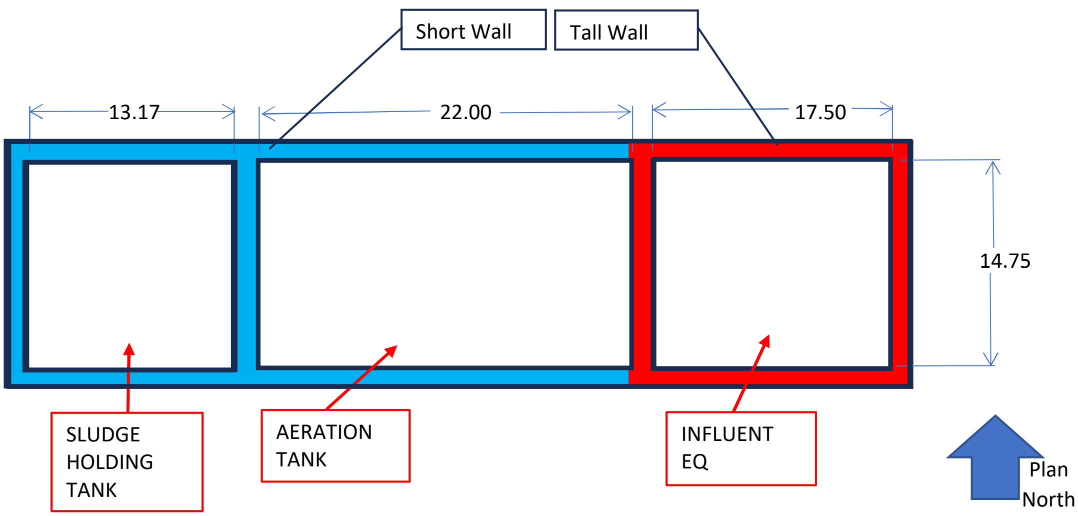
(Evorinmental durability factor does not apply to concrete shear capacity) — 9.2.6

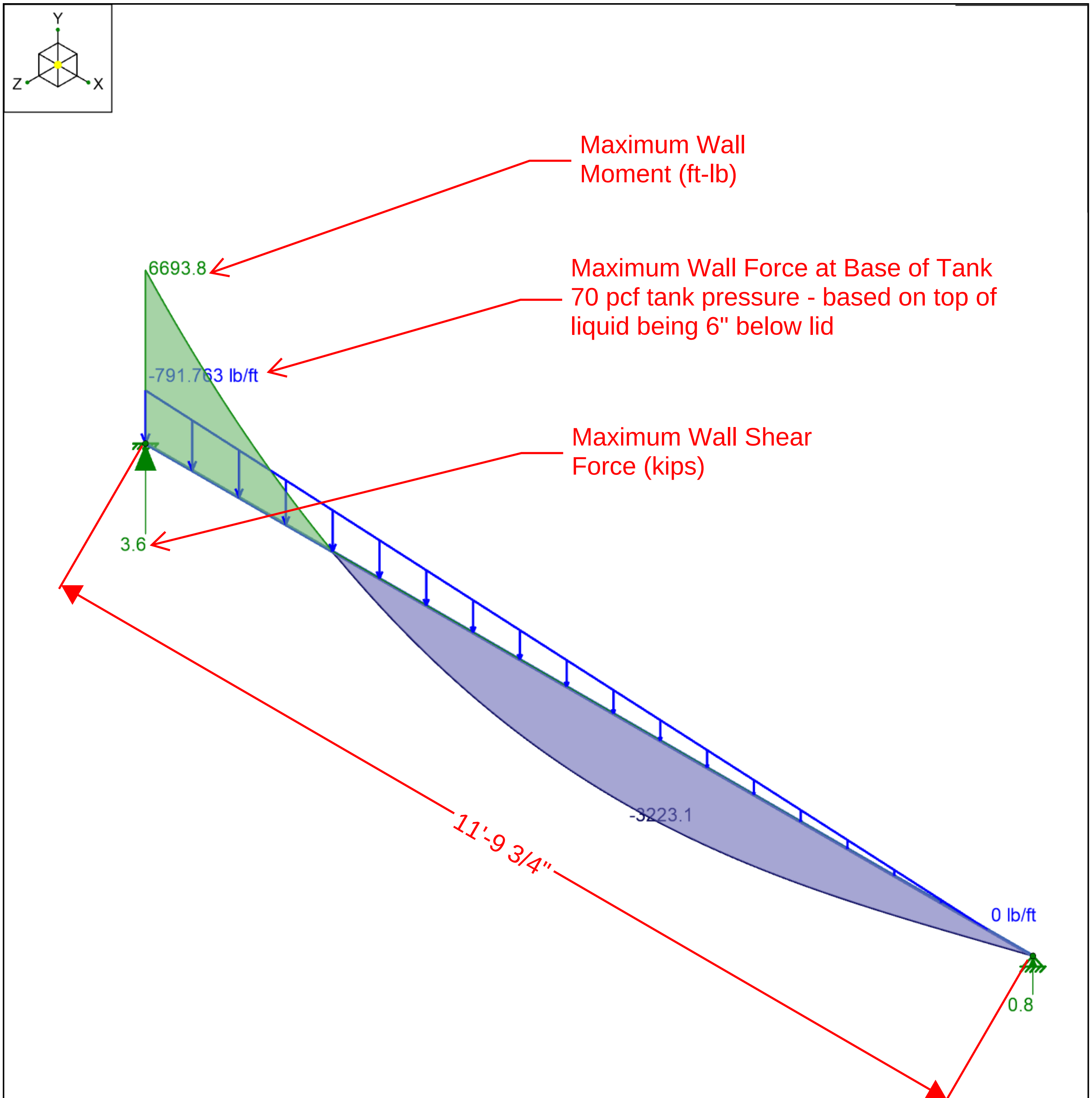
Vu = Vmax (U) =	10710.4	lbs	Utilization = 0.92	
U (Fluid Pressure) =	1.6		< Vu - Wall Shear NO GOOD	9.2.1

See Below for Reinforcement Check

Use 12 in wall w/ (#6 bar) @ 9 in spacing

Note: Sludge pressure against wall is greater than backfill pressure against wall
Therefore, the above designs are applicable for interior and exterior walls
See below for short wall and tall wall locations





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 (11'-9 3/4") - 1 ft section typ	SK-19
S Jacob		Nov 06, 2023
2310219 (4) - Phippsburg		Tall interior wall.r3d

Date:	11/6/2023
Job:	2310219(4) - Phippsburg 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

Large container: (40 ft long x 8 ft wide x 9.5 ft tall) — Small container: (10 ft long, 8 ft wide, 9.5 ft tall)

L	W	H	L	W	H
40	8	9.5	10	8	9.5

Large container			Small container		
(wet weight)	DL =	43975 lbs	DL =	10781 lbs
(provided by Aquaworks)	LL =	60 psf	LL =	60 psf
	SL =	77 psf	SL =	77 psf
	RL =	20 psf	RL =	20 psf
	$Wind_{down}$ =	9.6 psf	$Wind_{down}$ =	9.6 psf
	$Wind_{up}$ =	-36.4 psf	$Wind_{up}$ =	-36.4 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:		At Beam, 2 container corners at same point:	
DL =	10993.75 lbs	DL =	13689 lbs
LL =	4800 lbs	LL =	6000 lbs
SL =	6160 lbs	SL =	7700 lbs
RL =	1600 lbs	RL =	2000 lbs
$Wind_{down}$ =	768 lbs	$Wind_{down}$ =	960 lbs
$Wind_{up}$ =	-2912 lbs	$Wind_{up}$ =	-3640 lbs

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

Wind overturning	w =	21.1 psf
Large Container	T = C =	2380 lbs
Small Container	T = C =	595

Due to container location, do not double load at center beam. Just use single container overturning load

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

ASD Load Combinations

				If (2) Containers	
1	D + L	15794 lbs		19689 lbs	
2	D + S	17154 lbs		21389 lbs	
3	D + 0.75(L + S)	19214 lbs		23964 lbs	
4	D + W	11762 lbs		14649 lbs	
5	D + 0.75(L + S + W)	19790 lbs	<---CONTROLS	24684 lbs	<---CONTROLS
6	0.6D - W	3684 lbs	No Net Uplift	4573 lbs	No Net Uplift

	1 container	
Maximum Vertical Concentrated Load from lid & Containers=	19790	24684

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

Reaction at Wall = 49866 lbs

Date:	11/6/2023
Job:	2310219(4) - Phippsburg WWTP
Subject	Wall design

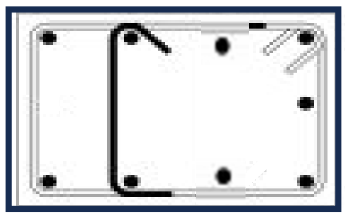
Reference

	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 =	49866	lbs		
Ag =	19.38	in^2		
Ast =	3.88	in^2	Number of #4 bars:	19.4
			Number of #5 bars:	12.5
ϕPn =	151760.2	lbs	Number of #6 bars:	8.8

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.

03 LID

Date:	11/7/2023
Job:	2310219(4) - Phippsburg 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 = 0.6 in^2 **# 7 bar**
bar diameter = 0.875 in **# 7 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 (Non-drift)
$Wind_{down}$ = 9.6 psf
Use 12" wide strip for design

Factored Loads * Sd (see below for Sd factor) — ACI 350-06: 9.2.1

				without Sd
(9-2)	1.2DL+1.6LL+0.5SL	687.2	plf	318.35
(9-3)	1.2DL+1.6SL+1.0LL	687.2	plf	375.52
(9-4)	1.2DL+1.6W+1.0LL+0.5SL	709.6	plf	297.71
(9-5)	1.2DL+1.0E+1.0LL+0.2S	705.4	plf	264.74

0.052 Seismic vertical component

Use w = 709.6 plf

Modification Factor used in above load calculations

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)

$fs_{max} = \dfrac{320}{B\ (s^2 + 4(2+db/2)^2)^{0.5}}$ — 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 = 0.875 in # 7 bar
s = 9 in spacing
fs_{max} = 23158 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	1.94	2.332
1.6	1.46	2.332
1.4	1.67	2.332
0.5	4.66	2.332
1	2.33	2.332
0.2	11.66	2.332

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:	11/7/2023
Job:	2310219(4) - Phippsburg WWTP
Subject:	Concrete Lid Design

Reference: ACI 350-06 | ACI 318-14

Span = (Use multi-span condition - 13'-2" - 22-0" - 17'-6")
(add thickness of interior walls to span in RISA model)
Single-Span (N-S) = 15.25

Typical multi-span

Sd Mu = wl^2/8 = 31369 lb ft (See RISA output)
Vu = wl/2 = 8375 lbs (See RISA output)

Compare continuous "beam" across multiple spans with simple span and other loading conditions below. Choose largest values for moment and shear

Check simple span Member section forces: (Span is clear span + half the wall thickness)

M Sd = wl^2/8 = 20627 lb ft More efficient to use Simple-span - detail accordingly
V Sd = wl/2 = 2863 lbs

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

M Sd = 31887 lb ft
V Sd = 9327 lbs

For 3-Span slab adjacent to container with snow drift (See Risa Results)

M Sd = 43790 lb ft
V Sd = 11673 lbs

Lid design

							ACI 350-06
Lid thickness =	12 in	As =	0.8 in^2	<-----	**# 7 bar**		
Clear cover =	2 in	Fy =	60,000 psi		at		7.7.1
Bar diameter =	0.875 in	f'c =	4500 psi		9		Table 4.2.2
d =	9.56 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) — ACI 318-14: Table 7.6.1.1
ρ = As / (b*d) 0.00697 <== 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 — ACI 318-14: Tbl. 21.2.2

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

** Solve for phiMn Based on Whitney Stress Block — ACI 318-14: Fig R21.2.2a

			ACI 350-06	ACI 318-14
a = stress block depth = As*Fy / (0.85*f'c*beff) =	1.0458	in.		
c = depth to neutral axis = a / β =	1.2676	in.		
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 =	36158	ft.-lb.		
ϕ=	0.9			21.2.1
ϕMn = Ultimate Moment Cap =	32543	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 =	1.0640	in^2		9.6.1.3
if below ρ min, multiply phiMn x 0.75 =	24407	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 = ***32542.65*** *lb-ft*

Mu (Sd) = 31887 lb ft < fMn - Lid Flexure OK Utilization = 0.98

Note:
4500 psi is minimum concrete strength for concrete with low permeability (concrete is coated is coated as directed by Aquaworks) — ACI 350-06: Table 4.3.1

Date:	11/7/2023
Job:	2310219(4) - Phippsburg WWTP
Subject:	Concrete Lid Design

			Reference ACI 350-06	Reference 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	0.0196		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) =	2.2632	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) =	0.3849	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 =	15395	lbs		22.5.5.1
φ = 0.75 φVc =	11546	lbs		21.2
φVc/2 = **For Beam Design (h > 10 in.)	5773	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

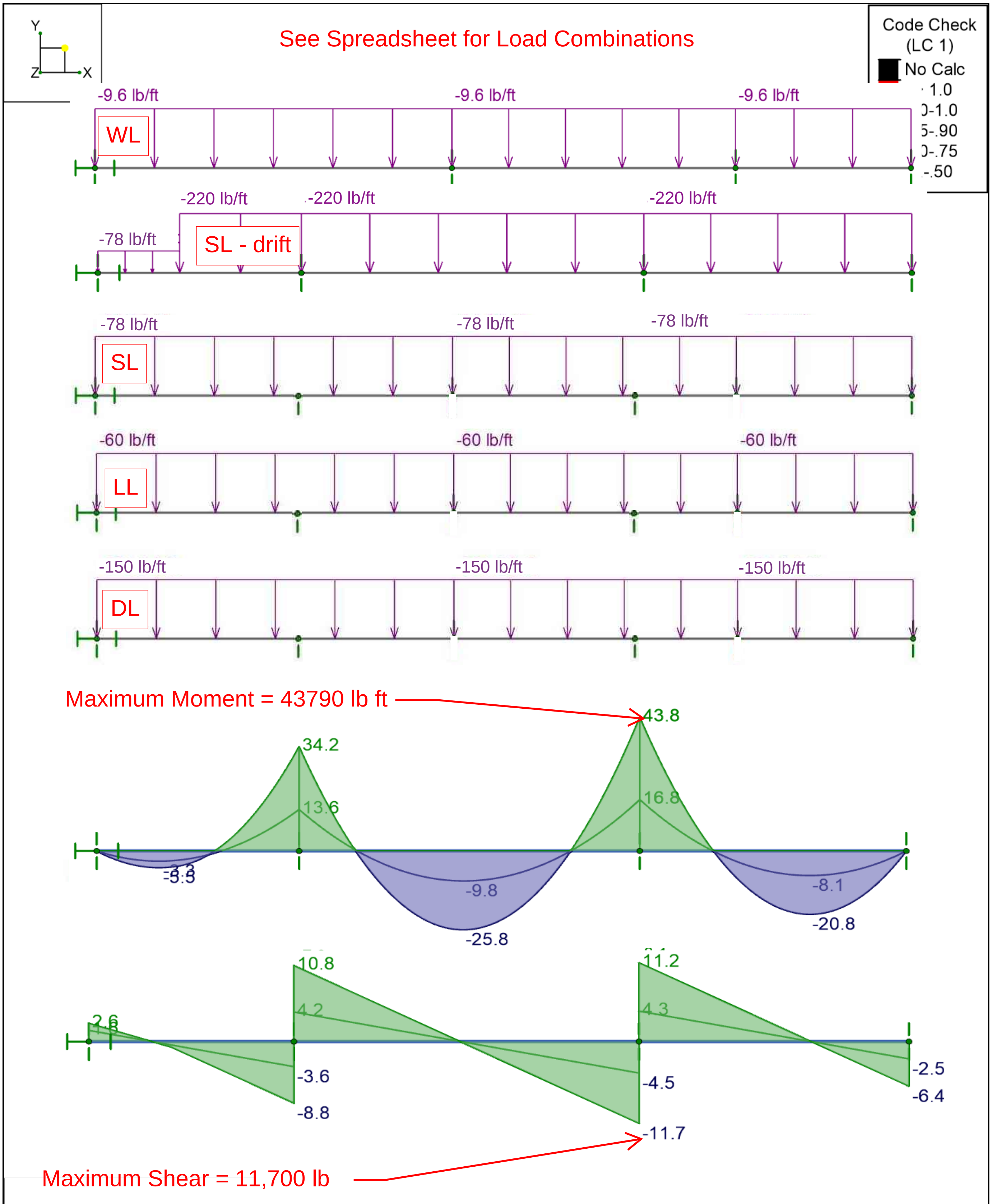
Vu = Vmax (U) = 9327 lbs < Vu - Slab Shear NO GOOD Utilization = 0.81

See Below for Reinforcement Check — 9.2.1

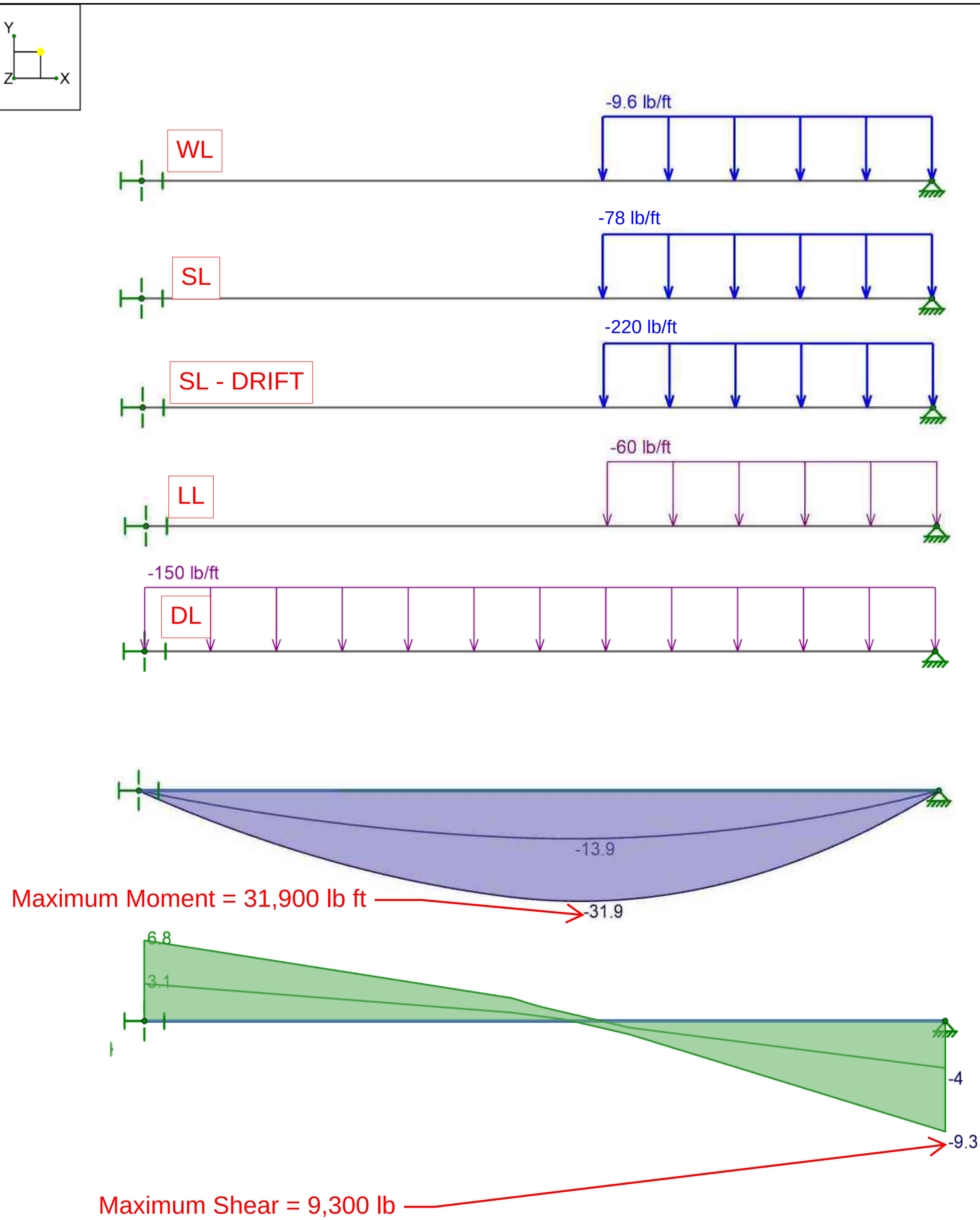
Vs required = Sd (Vu-fVc) = -5175.37 lbs. - No shear reinforcement required — R9.2.6.4

Use 12 in slab w/ # 7 bar @ 9 in spacing

See Spreadsheet for Load Combinations



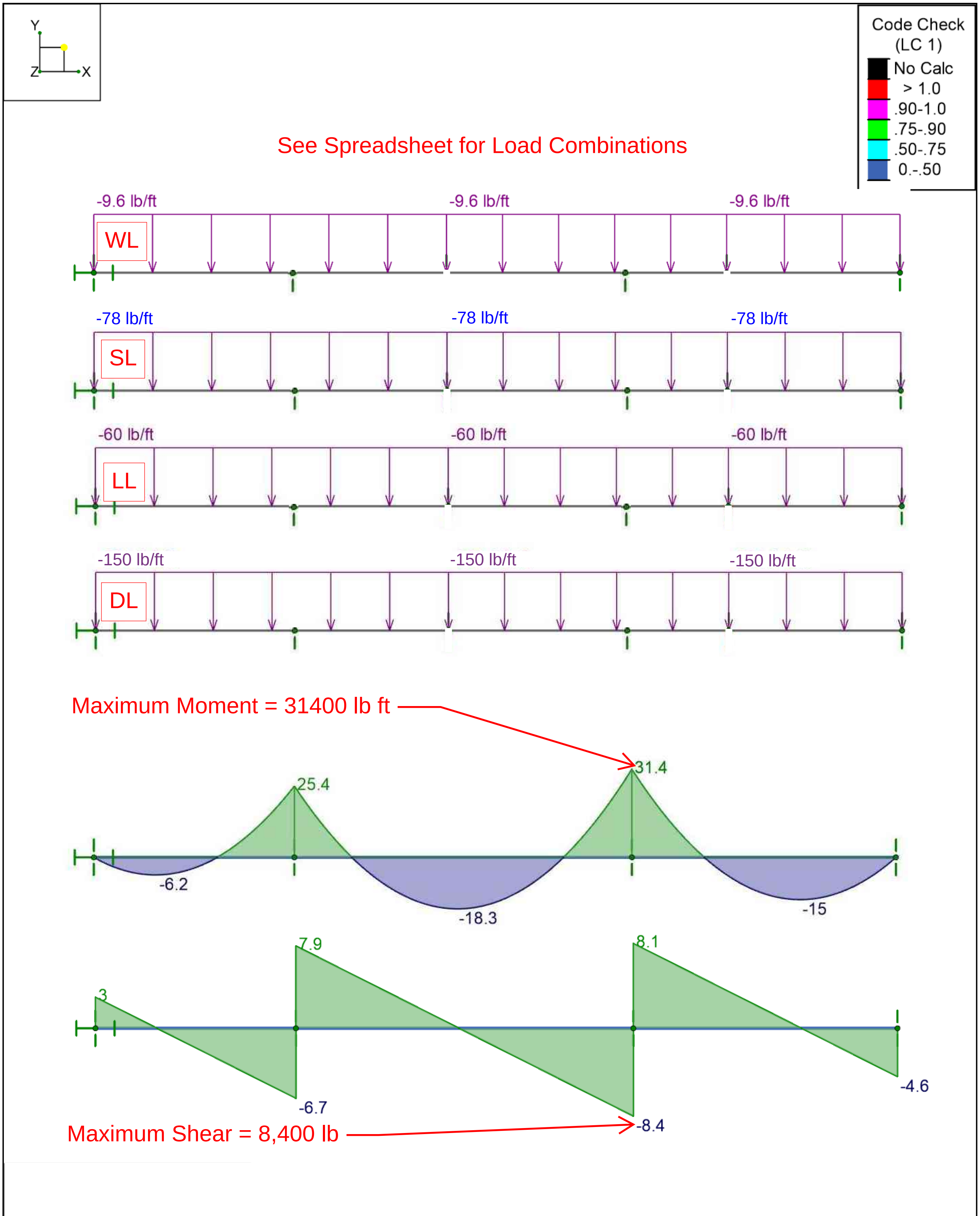
Wallace Design Collective	Lid Design	SK-20
ɔb		Nov 07, 2023
19 (4) Phippsburg WWTP		Lid - multispan.r3d



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



Wallace Design Collective	Lid Design	SK-20
›b		Nov 07, 2023
19 (4) Phippsburg WWTP		Lid - multispan.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	Yes
2	N2	13.667	0	0	
3	N3	36.667	0	0	
4	N4	54.667	0	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]
1	N4		Reaction	Reaction	
2	N2		Reaction	Reaction	
3	N3		Reaction	Reaction	
4	N1	Reaction	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
2	M2	Y	-150	-150	0	%100
3	M3	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	0	0	0	%100
2	M2	Y	0	0	0	%100
3	M3	Y	0	0	0	%100

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	-220.0	-220.0	5.5	%100
2	M2	Y	-220.0	-220.0	0	%100
3	M3	Y	-220.0	-220.0	0	%100
4	M1	Y	-78.0	-78.0	0	5.5

Member Distributed Loads (BLC 4 : Wind)

	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	0	%100
2	M2	Y	-9.6	-9.6	0	%100
3	M3	Y	-9.6	-9.6	0	%100

Basic Load Cases

	BLC Description	Category	Distributed
1	DL	DL	3
2	LL	LL	3
3	SL	SL	4
4	Wind	WL	3

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	DL Only	Yes	Y	DL	2.522								
2	1.2DL + 1.6LL + 0.5SL	Yes	Y	DL	2.522	LL	2.522	SL	2.522				
3	1.2DL + 1.6SL + 1.0LL	Yes	Y	DL	2.522	LL	2.522	SL	2.522				
4	1.2DL + 1.6LL + 0.5W(down)	Yes	Y	DL	2.522	LL	2.522	WL	2.522				
5	1.2DL + 1.6SL + 0.5W(down)	Yes	Y	DL	2.522	SL	2.522	SLN		WL	2.522		
6	without Sd1			DL	1.2	SL	1.6	WL	0.5				
7	1.2DL + 1.0W(down) + 0.5LL	Yes	Y	DL	2.522	WL	2.522	LL	2.522				
8	1.2DL + 1.0W(down) + 0.5LL + 0.5SL	Yes	Y	DL	2.522	WL	2.522	LL	2.522			SL	2.522
9	without Sd			DL	1.2	WL	1	LL	0.5			SL	0.5
10	0.9DL + 1.0W(down)	Yes	Y	DL	2.522	WL	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	10	2.594	8	0	10	0	10	0	10	0	10
2			min	0	1	1.59	1	0	1	0	1	0	1	0	1
3		2	max	0	10	0.488	8	0	10	0	10	0	10	-3.225	1
4			min	0	1	0.298	1	0	1	0	1	0	1	-5.265	5
5		3	max	0	10	-0.995	1	0	10	0	10	0	10	-2.034	1
6			min	0	1	-2.1	5	0	1	0	1	0	1	-3.015	5
7		4	max	0	10	-2.287	1	0	10	0	10	0	10	9.874	8
8			min	0	1	-5.445	5	0	1	0	1	0	1	3.573	1
9		5	max	0	10	-3.58	1	0	10	0	10	0	10	34.191	8
10			min	0	1	-8.789	5	0	1	0	1	0	1	13.597	1
11	M2	1	max	0	10	10.839	8	0	10	0	10	0	10	34.191	8
12			min	0	1	4.21	1	0	1	0	1	0	1	13.597	1
13		2	max	0	10	5.211	8	0	10	0	10	0	10	-4.359	1
14			min	0	1	2.035	1	0	1	0	1	0	1	-11.951	5
15		3	max	0	10	-0.14	1	0	10	0	10	0	10	-9.806	1
16			min	0	1	-0.417	5	0	1	0	1	0	1	-25.732	5
17		4	max	0	10	-2.315	1	0	10	0	10	0	10	-2.747	1
18			min	0	1	-6.045	5	0	1	0	1	0	1	-7.152	5
19		5	max	0	10	-4.491	1	0	10	0	10	0	10	43.79	8
20			min	0	1	-11.673	5	0	1	0	1	0	1	16.821	1
21	M3	1	max	0	10	11.242	8	0	10	0	10	0	10	43.79	8
22			min	0	1	4.339	1	0	1	0	1	0	1	16.821	1
23		2	max	0	10	6.837	8	0	10	0	10	0	10	3.112	8
24			min	0	1	2.637	1	0	1	0	1	0	1	1.125	1
25		3	max	0	10	2.433	8	0	10	0	10	0	10	-6.911	1
26			min	0	1	0.934	1	0	1	0	1	0	1	-17.746	5
27		4	max	0	10	-0.768	1	0	10	0	10	0	10	-7.286	1
28			min	0	1	-1.972	5	0	1	0	1	0	1	-18.783	5
29		5	max	0	10	-2.47	1	0	10	0	10	0	10	0	10
30			min	0	1	-6.376	5	0	1	0	1	0	1	0	1

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	N4	max	0	10	6.376	8	0	10	0	10	0	10	0	10
2		min	0	1	2.47	1	0	1	0	1	0	1	0	1
3	N2	max	0	10	19.628	8	0	10	0	10	0	10	0	10
4		min	0	1	7.79	1	0	1	0	1	0	1	0	1
5	N3	max	0	10	22.915	8	0	10	0	10	0	10	0	10
6		min	0	1	8.83	1	0	1	0	1	0	1	0	1

Envelope Node Reactions (Continued)

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
7	N1	max	0	10	2.594	8	0	10	0	10	0	10	0	10
8		min	0	1	1.59	1	0	1	0	1	0	1	0	1
9	Totals:	max	0	10	51.513	8	0	10						
10		min	0	1	20.681	1	0	1						

Date:	11/7/2023
Job:	2310219(4) - Phippsburg WWTP
Subject:	Concrete Lid Design

Reference: ACI 350-06 | ACI 318-14

Slab section near opening

Slab Reinf is at 9 in spacing
Opening width = 6 ft (3 ft on each side of slab strip
"beam" width = 2.25 ft (slab strip width between openings)
Sd = 1
Mu Sd = 71794 lb ft * (Multiply Typical lid Moment by (1/2 opening+beam width)
Vu = 48967 lbs * (Multiply Typical lid Shear by (1/2 opening + beam width)
* Sd = 1.0 for 2.46 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 = 2.46 in BAR SPACING OK
Minimum spacing = larger of: 1 in — 25.2
db = 0.875 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 = 6 in^2 Exceeds As (max)
depth, d = 9.56 in **Use** 4.80 in^2
width, b = 27 in (See below for Max As)
Fy = 60,000 psi
f'c = 4,500 psi

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) 0.02324 <== 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) = 2.7887 in.
c = depth to neutral axis = a / β = 3.3802 in.
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 = 196036 ft.-lb.
ϕ= 0.9 — 21.2.1
ϕMn = Ultimate Moment Cap = 176432 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 = 6.3840 in^2 — 9.6.1.3
if below ρ min, multiply phiMn x 0.75 = 132324 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)

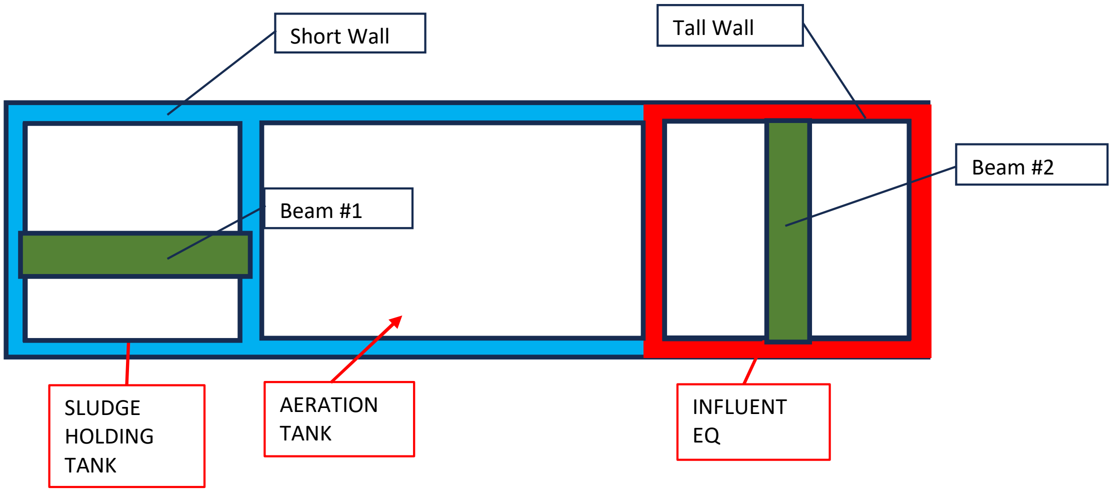
ϕMn = Ultimate Moment Cap = ***176432.4*** *lb-ft*
Mu = 71794 1 < fMn - Lid Flexure OK Utilization = 0.41

Use 12 in slab w/ 10 # 7 bar between openings

Date:	11/7/2023	
Job:	2310219(4) - Phippsburg 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	0.0055		R10.3.3	
ρ temp = temp and shrink steel = 0.0050			Table 7.12.2.1	
As(temp)= (ρ temp * beff * h) = 1.6200 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) =	**5.09**	in^2	R10.3.5	
	Use	8 # 7 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) =	0.8660	in^2 <== unless As > 1.33*As reqd per code		9.6.1.3

Date:	11/7/2023
Job:	2310219(4) - Phippsburg WWTP
Subject:	Concrete Lid Design



Beam #1 Design - Over Sludge Holding Tank

Reference: ACI 350-06 8.7.3; ACI 318-14

Span = 13.17 ft

* 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 =	10993.75	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) — ACI 350-06 9.2.1

				without Sd
(9-2)	1.2DL+1.6LL+0.5SL	34104.2	lbs	17393.75
(9-3)	1.2DL+1.6SL+1.0LL	34185.2	lbs	17393.75
(9-4)	1.2DL+1.6W+1.0LL+0.5SL	35047.1	lbs	18161.75
(9-5)	1.2DL+1.0E+1.0LL+0.2S	34698.9	lbs	17965.425

0.052 Seismic vertical component

Use P = 35047.1 lbs

Date:	11/7/2023
Job:	2310219(4) - Phippsburg 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

B = 1.35 — 10.6.4.4

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

Sd(min) = 1.0 — 9.2.6

Sd Factor - Bending		
U factor	Sd	Sd*U
1.2	1.28	1.540
1.6	1.00	1.600
1.4	1.10	1.540
0.5	3.08	1.540
1	1.54	1.540
0.2	7.70	1.540

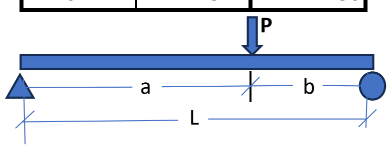
Sd Factor - Shear		
U factor	Sd	Sd*U
1.2	1.88	2.250
1.6	1.41	2.250
1.4	1.61	2.250
0.5	4.50	2.250
1	2.25	2.250
0.2	11.25	2.250

Moment at mid-span

Mu = Pab/L = 123986.4 lb ft

Shear at support

Vu = Pa/L = 24468 lbs



a= 5.25 ft

b= 7.92 ft

L= 13.17 ft

Fy = 60,000 psi

f'c = 4,500 psi Beam Reaction = 16608 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) 0.00692 <== 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 et < ety
No	Transition Zone If $\epsilon_{ty} < \epsilon_t < 0.005$
Yes	Tension Controlled Since (et) = or > 0.005

Date:	11/7/2023
Job:	2310219(4) - Phippsburg 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) =	1.4379	in.		
c = depth to neutral axis = a / β =	1.7429	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 rho max eqn		
If below ρ min, multiply As x 1.33 =	2.9260	in^2		9.6.1.3
if below ρ min, multiply ϕMn 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 = 123986.4 lb ft > fMn - Lid flexure NO GOOD Utilization = 1.00

Note Point Load at Container Corner is MUCH smaller than this
Calculation uses

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) = 24468 lbs < Vu - Beam Shear NO GOOD

See Below for Reinforcement Check 9.2.1

Vs required = Sd (Vu-fVc) = -11600.3 lbs. - No 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 =	1.153368	
Vn = Vc + Vs/Sd =	67937.99	
ϕ Vn =	50953.5	Utilization = 0.48

Use	16	x	24	w/	5	#6 bar	in beam
		and	#3	stirrups @	6	in o.c.	

Note: T-beam analysis would yield higher capacity, but was not used unless it was needed for more precision

Date:	11/7/2023		
Job:	2310219(4) - Phippsburg WWTP		
Subject:	Concrete Lid Design		

Beam #2 Design - Over Influent EQ

Reference: ACI 350-06 | ACI 318-14

Span = **14.75** ft — 8.7.3

* Beam spans as shown above

Beam width = b =	**24**	in	
Beam height = h =	**18**	in	
bar diameter =	**0.875**	in	**#7 bar**
bar area =	**0.6**	in^2	
stirrup bar diameter =	**0.375**	in	**#3**
clear cover =	**2**	in	Self Weight of beam
d =	**15.1875**	in	450 plf

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

DL =	13689	lbs
LL =	6000	lbs
SL =	7700	lbs
RL =	2000	lbs
W_{down} =	960	lbs
W_{up} =	-3640	lbs
Wind $_{(overturning)}$ =	2975	lbs

Try 5 #7 bar in beam

bar spacing = 4.78125 in

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

				without Sd
(9-2)	1.2DL+1.6LL+0.5SL	42548.4	lbs	21689
(9-3)	1.2DL+1.6SL+1.0LL	42649.6	lbs	21689
(9-4)	1.2DL+1.6W+1.0LL+0.5SL	43727.0	lbs	22649
(9-5)	1.2DL+1.0E+1.0LL+0.2S	43287.5	lbs	22400.828

0.052 Seismic vertical component

Use P = 43727.0 lbs

Modification Factor used in above load calculations

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 — 10.6.4.4

B =	1.35			
db =	0.875	in	#7 bar	
s =	4.78125	in spacing		
fs_{max} =	34714	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:	11/7/2023
Job:	2310219(4) - Phippsburg WWTP
Subject:	Concrete Lid Design

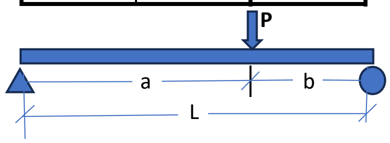
Reference

ACI 350-06	ACI 318-14
9.2.6	

Sd(min) = 1.0

Sd Factor - Bending		
U factor	Sd	Sd*U
1.2	1.28	1.540
1.6	1.00	1.600
1.4	1.10	1.540
0.5	3.08	1.540
1	1.54	1.540
0.2	7.70	1.540

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 = 180094.9 lb ft

Shear at support

Vu = Pa/L = 37047 lbs

(Values include self-weight)

a= 7.375 ft
b= 7.375 ft
L= 14.75 ft

Fy = 60,000 psi

f'c = 4,500 psi Beam Reaction = 25182 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) 0.00823 <== 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) =	1.9608 in.		
c = depth to neutral axis = a / β =	1.1250 in.		
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 =	213107 ft.-lb.		
φ=	0.9		21.2.1
φMn = Ultimate Moment Cap =	191796 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 =	3.9900 in^2		9.6.1.3
if below ρ min, multiply phiMn x 0.75 =	143847 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 = ***191796*** *lb-ft*

Mu = 180094.9 lb ft < fMn - Lid Flexure OK Utilization = 0.94

Date:	11/7/2023
Job:	2310219(4) - Phippsburg WWTP
Subject:	Concrete Lid Design

			Reference: ACI 350-06	ACI 318-14
4. Check Shear Capacity				
**neglects Vs (shear reinforcement)				
Vc = 2 * λ * (f'c)^.5 * b * d =	48903	lbs		22.5.5.1
φ = 0.75	φVc = 36677	lbs		21.2
φVc/2 = **For Beam Design (h > 10 in.)	18339	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) = 37047 lbs		< Vu - Beam Shear NO GOOD		
		See Below for Reinforcement Check	9.2.1	
Vs required = Sd (Vu-fVc) =	569.3428	lbs. - Shear reinforcement required	R9.2.6.4	
Vs = Av fy d / s	33412.5	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 =	7.59375		11.5.4.1	
Sd =	1.152649			
Vn = Vc + Vs/Sd =	77890.37			
φ Vn =	58417.78	Utilization = 0.63		

Use	18	x	24	w/	5	#7 bar	in beam
	and	#3	stirrups @	6	in o.c.		

Date:	11/9/2023
Job:	2310219(4) - Phippsburg WWTP
Subject:	Container Embeds

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

length	width	height
40	8	10

DL =	43975	lbs
LL =	60	psf
SL =	40	psf
RL =	20	psf
Winddown	9.6	psf
Windup =	23.075	psf

(provided by Aquaworks)

Use Lid depth = 10 in

Assume wind acts on single container
for simplicity / consistency

Wind lateral pressure: **20.8** psf (factorered)

Maximum lateral load at corner support: 2080 lbs
(container length x height x wind pressure / 4 supports)

Consider overturning for embed forces

Overall container moment from wind:	20800	lb ft
couple distance (embed spacing):	7	ft (Assume 6" each side to CL of support)
T = C = (M/d) =	2971.429	lbs (up or down)
Total Factored Dead Load :	10987.5	
0.6D - W =	3621.071	lbs down (no net uplift)
Maximum Load:	32967	lbs

See Hilti Profis Output for embed analysis

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

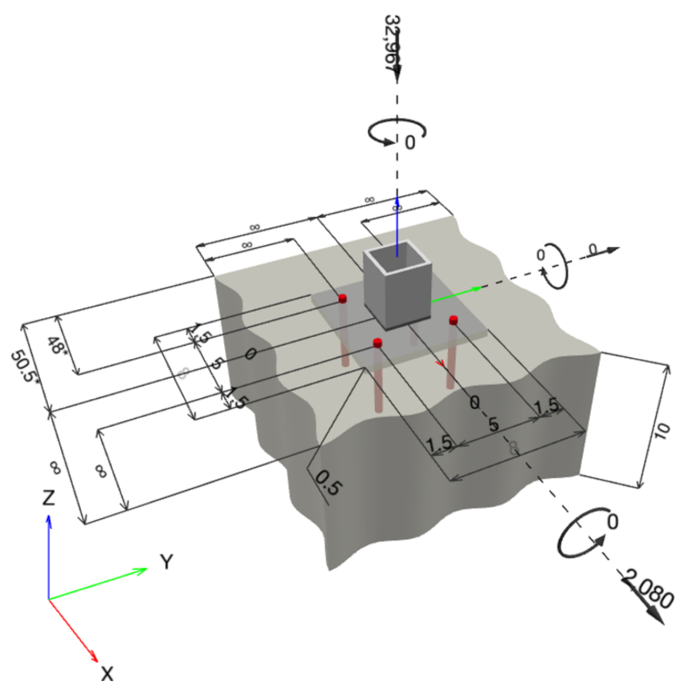
Specifier's comments:

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

Company:		Page:	3
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_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

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the 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 the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

04 FOUNDATION SLAB

wallace design collective

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

Date:	11/9/2023
Job:	2310219(4) - Phippsburg WWTP
Subject:	Foundation

Reference

ACI 350-06	ACI 318-14

Foundation Design

Total weight of tanks and containers

Tank dimensions

W	L	H	Volume	
13.167	14.75	7.75	1505	ft^3
22	14.75	7.75	2515	ft^3
17.5	14.75	11.81	3089	ft^3

Total volume = 7109 ft^3

Large Container: 43975 lbs

Small Container: 10781 lbs

Walls

	Length	height	thickness	Weight			
Interior	14.75	7.75	1	17146.875 lbs			
	14.75	11.81	1	26135.156 lbs			
				0 lbs		Total	
				0 lbs		43282.0313	lbs
Exterior							
	12.75	12.75	1	24384 lbs			
	12.75	12.75	1	24384 lbs			
	37.167	19.5	1	108713.48 lbs		Total	
	37.167	19.5	1	108713.48 lbs		266195.7	lbs

Lid	Length	Width	Thickness		
	56.667	14.75	12	125376	lbs

Fnd Slab	Length	Width	Thickness	1 ft projection outside of walls	
(ft)	58.667	16.75	1.1875	221101	lbs

Thickness considers minimum depth at sloping slab

Liquid Density: 70 pcf — 497662 lbs (Assume liquid up to bottom of lid (conservative))

Live Load = containers 60 psf x 50x8 = 24000 lbs

lid 60 psf x remainder of lid 26150 lbs

Total Live Load = 50150 lbs

Snow Load =

SL = 78.1 psf 65278.967 lbs (Assume full SL over entire lid area)

Wind Down:

Pressure = 9.6 psf 8024.0472 lbs

Total Load = 1331826 lbs — 710711 lbs Total DL

(Conservatively consider cumulative load instead of ASD Load Combinations)

Slab area = 983 ft^2

Bearing pressure = 1355.3 psf — 723 DL Pressure (empty tanks)

Allowable bearing pressure = 3000 OK — Min DL Bearing Pressure = 700 OK

Date:	11/9/2023
Job:	2310219(4) - Phippsburg WWTP
Subject:	Foundation

Consider Flexure of foundation slab

Reference: ACI 350-06 | ACI 318-14

From RISA 3D Model, moment in footing from 1355.3 psf soil pressure and moments at base of wall transferring into slab, times Sd
yields a moment force in the slab of: 42590 lb ft

Slab thickness = 14.25
Clear cover (top) = 3 in — 19039 lb ft per RISA model using multi-span
bar diameter = 1 in
d = 10.75 in
f'c = 4500 psi

Factored Loads * Sd (see below for Sd factor)

				without Sd	9.2.1
(9-2)	1.2DL+1.6LL+0.5SL	3232287.7	lbs	1497220.37	
(9-3)	1.2DL+1.6SL+1.0LL	3232287.7	lbs	1538937.06	
(9-4)	1.2DL+1.6W+1.0LL+0.5SL	3252725.2	lbs	1479968.67	
(9-5)	1.2DL+1.0E+1.0LL+0.2S	3385078.7	lbs	1507534.59	

0.052 Seismic vertical component

Use w = 1566.1 psf
Factored load, without Sd

Modification Factor used in above load calculations

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 — 10.6.4.4

As = 0.79 in^2
B = 1.35
db = 1 in — As = 0.948 in^2 #9
s = 10 in spacing

fs_{max} = 21201 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	2.12	2.547
1.6	1.59	2.547
1.4	1.82	2.547
0.5	5.09	2.547
1	2.55	2.547
0.2	12.74	2.547

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:	11/9/2023
Job:	2310219(4) - Phippsburg WWTP
Subject:	Foundation

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)	0.00735	<== 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) =	1.2392	in.		
c = depth to neutral axis = a / β =	1.5021	in.		
Mn = Nominal Moment Cap = As*Fy*(d-a/2)/12 =	48018	ft.-lb.		
ϕ=	0.9			21.2.1
ϕMn = Ultimate Moment Cap =	43216	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 =	1.2608	in^2		9.6.1.3
if below ρ min, multiply phiMn x 0.75 =	32412	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 = ***43216.3*** *lb-ft*

Mu (Sd) = 42590   < fMn - Lid Flexure OK   Utilization = 0.99

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	0.0021		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) =	0.0573	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) =	0.5651	in^2	R10.3.5

Date:	11/9/2023
Job:	2310219(4) - Phippsburg 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) =	0.4	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 =			17307 lbs		22.5.5.1
ϕ =	0.75	ϕVc =	12980 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) =	12603 lbs	> fVc - Slab Shear OK	Utilization = 0.97		
Shear at 'd' away from support		See Below for Reinforcement Check		9.2.1	
See RISA file					
Vs required = Sd (Vu-fVc) =	-849.093	lbs. - No shear reinforcement required		R9.2.6.4	

Use 18 in slab w/ #9 @ 10 in spacing

Note: Calculation takes into account the sloping floor of the sludge tank, which, at its lowest point is 3.75" below the highest point
Therefore, 'd' = 14.25 inches, but the FND slab thickness is 18 inches everywhere else - so the design is good for the entire slab

Sd Mu =	42590	lb ft		
ϕMn = AsFy(d-a/2) =	480181	lb ft	Slab Flexure OK	
As =	0.79	in^2	1.795455	#8 bars
Fy =	60,000	psi		
f'c =	4500	psi		
b =	10	in		
a =	1.239216	in		

Check Wall shear ϕVn>Vu+Vs/Sd

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

ϕ = 0.75

ϕVc = 10817 lbs Slab Shear OK

Use 14.25 in slab w/ 1.795455 #8 bars at 10 on center

05 BUOYANCY

wallace design collective

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

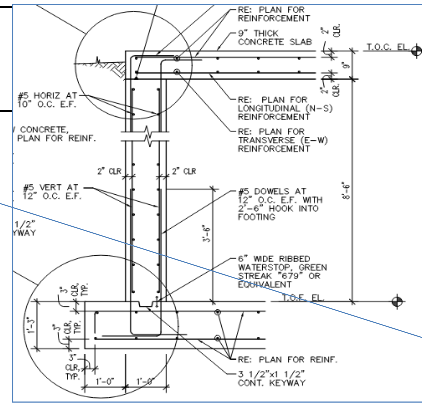
Created 8/29/11

Date	11/9/2023	Sheet No.	of
Project	Phippsburg WWTP		
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:	660788	lbs
ASSUMED WEIGHT OF SOIL:	110	pcf
WEIGHT OF REINFORCED CONCRETE:	150	pcf
DENSITY OF fluid:	62.4	pcf
DIMENSIONS OF TANK:		
Length:	58.67	ft
Round (R) or Square (S)	S	
Width:	16.75	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:	7394.80	ft^3	*This cell can be overridden if tank volume is known (should include foundation volume)
SOIL:	Tank + Fdn Volume:	7394.80025	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):		**7394.80**	**ft^3**	8844.05 ft^3 (THIS IS THE TOTAL VOLUME OF SYSTEM)
				RECHECK

DETERMINE WEIGHTS:

TANK:	660787.7063	lbs
SOIL (wt. of soil above tank x soil volume):	173544	lbs
4" SLAB (wt. of concrete x volume):	0.00	lbs
TOTAL WEIGHT:	**834331.2968**	**lbs**
Distance from grade to water table:	3	ft
WEIGHT OF EQUAL VOLUME OF WATER:		
(density of water x total volume)	459986.2856	lbs

DIFFERENCE OF BUOYANT FORCES AND GRAVITY FORCES:

(wt. of equal volume of water - total weight)	**-374345.0112**	**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):	**1.81**		**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:	**-374345.0112**	**lbs**
(force difference-weight of buoyant soil)		

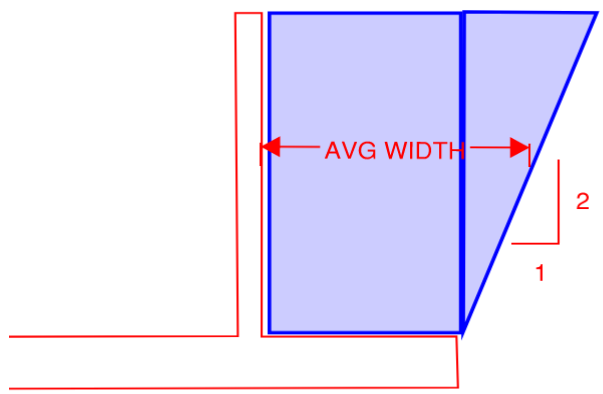
FOUNDATION CONCRETE VOLUME: 1474.0084

WEIGHT CALCULATION (EMPTY TANK)

FOUNDATION:		THICKNESS	18	IN	221.1013	KIPS
		LENGTH	58.667	FT		
		WIDTH	16.75	FT		
		EXTENSION	1	FT		
		AREA	982.6723	SQ FT		
		WEIGHT	150	PCF		
	Tall Walls		Short Walls			
WALLS	11.8125	HEIGHT	7.5	FT	238.1856	KIPS
	12	THICKNESS	12	IN		
	68.5	LENGTH	103.833	FT		
CONC BEAMS		LENGTH	27.917	FT	4.18755	KIPS
		DEPTH	6	IN		
		WIDTH	24	IN		
LIDS		THICKNESS	12	IN	142.3758	KIPS
		AREA	949.1723	SQ FT		
CONTAINERS		DL	43.95	KIPS	54.9375	KIPS
			10.9875			
				TOTAL	660.7877	KIPS

Soil weight Calculation. **Assume 2:1 soil slope acting on footing**

west wall:	length	16.75		
	height	7.5		
	avg width	2.875	361.17188	ft^3
east wall	length	32		
	height	7.5		
	avg width	2.875	711.5625	ft^3
north wall	length	58.667		
	height	7.5		
	avg width	2.875	1265.0072	ft^3
south wall	length	58.667		
	height	7.5		
	avg width	2.875	1308.1322	ft^3
		Total Volume =	3645.8738	ft^3



soil weight	110	pcf	Total Weight =	401046.1	lbs	(using total soil weight)
buoyant soil weight	47.6	pcf		173543.6	lbs	(using bouyant soil weight)