Appendix D
SEISMIC EVALUATION

City of Wilsonville
Wastewater Treatment Plant Master Plan

Technical Memorandum 1
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FINAL | July 2022

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Wastewater Treatment Plant Master Plan

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Abbreviations

AACEI	Association for the Advancement of Cost Engineering
ACI	American Concrete Institute
ACCU	Air Cooled Condensing Unit
AISC	American Institute of Steel Construction
ASCE	American Society of Civil Engineers
ASCE 41-17	ASCE Standard Seismic Evaluation and Retrofit of Existing Buildings
ASTM	American Society for Testing and Materials
BPOE	Basic Performance Objective for Existing Buildings
BSE-1E	Basic Safety Earthquake-1 for use with existing buildings
BSE-2E	Basic Safety Earthquake-2 for use with existing buildings
C	Soil Site Class Type
Carollo	Carollo Engineers, Inc.
CMU	Concrete Masonry Wall
CSZ	Cascadia Subduction Zone
D	Soil Site Class Type
DCR	Demand to Capacity Ratio
E	East
f'_c	Concrete Compressive Strength
F_a	Factor to Adjust Spectral Acceleration in the short period range for Site Class
F_v	Factor to Adjust Spectral Acceleration at 1 Second for Site Class
f_y	Yield Strength of Rebar
F_y	Yield Strength of Steel
g	acceleration due to Gravity
M9.0	Magnitude 9.0
N	North
OSSC	Oregon Structural Specialty Code
pcf	pounds per cubic foot
Plant	Wilsonville Wastewater Treatment Plant
psf	pounds per square foot
psi	pounds per square inch
RAS	Return Activated Sludge
S	South
S_1	Spectral Response Acceleration Parameter at a 1 Second Period for any Seismic Hazard Level and any Damping, Adjusted for Site Class
$S_{1,\ 5/50}$	Spectral Response Acceleration Parameter at a 1 Second Period for any Seismic Hazard Level and any Damping, Adjusted for Site Class

$S_{1,\ 20/50}$	Spectral Response Acceleration Parameter at a 1 Second Period for any Seismic Hazard Level and any Damping, Adjusted for Site Class
S_S	Spectral Response Acceleration Parameter at Short Periods for CSZ Seismic Hazard Level and any Damping, Adjusted for Site Class
$S_{S,\ 5/50}$	Spectral Response Acceleration Parameter at Short Periods for BSE-1E Seismic Hazard Level and any Damping, Adjusted for Site Class
$S_{S,\ 20/50}$	Spectral Response Acceleration Parameter at Short Periods for any Seismic Hazard Level and any Damping, Adjusted for Site Class
$S_{X1,\ BSE\text{-}1E}$	Spectral Response Acceleration Parameter at a 1 Second Period for BSE-1E Seismic Hazard Level and any Damping, Adjusted for Site Class
$S_{X1,\ BSE\text{-}2E}$	Spectral Response Acceleration Parameter at a 1 Second Period for BSE-2E Seismic Hazard Level and any Damping, Adjusted for Site Class
$S_{X1,\ CSZ}$	Spectral Response Acceleration Parameter at a 1 Second Period for CSZ Seismic Hazard Level and any Damping, Adjusted for Site Class
$S_{XS,\ BSE\text{-}1E}$	Spectral Response Acceleration Parameter at Short Periods for BSE-1E Seismic Hazard Level and any Damping, Adjusted for Site Class
$S_{XS,\ BSE\text{-}2E}$	Spectral Response Acceleration Parameter at Short Periods for BSE-2E Seismic Hazard Level and any Damping, Adjusted for Site Class
$S_{XS,\ CSZ}$	Spectral Response Acceleration Parameter at Short Periods for CSZ Seismic Hazard Level and any Damping, Adjusted for Site Class
USGS	United States Geological Survey
W	West

Technical Memorandum 1

SEISMIC EVALUATION

1.1 Introduction

The City of Wilsonville retained Carollo Engineers, Inc. (Carollo) to perform a seismic evaluation of five of their existing structures located at the Wastewater Treatment Plant (Plant) in Wilsonville, Oregon.

The Plant is comprised of several buildings and process structures that include water-bearing basins, channels, and clarifiers. The scope of work, however, is limited to the evaluation of five structures. Much of the Plant was recently upgraded and expanded in 2014 and included new buildings such as the Headworks and Solids Drying Building. These newer facilities were designed in accordance with the 2010 Oregon Structural Specialty Code (OSSC) and should meet modern seismic design and detailing. These newer structures were also designed to a higher performance level than typical commercial facilities. Therefore, these structures were not included in the scope of work and should have a relatively low risk of poor seismic performance.

The Plant does have some older structures that were considered for inclusion in this evaluation, but by the nature of their design are considered to be inherently resilient. After a review of the record drawings for the various facilities and a site visit to the Plant, five existing structures were selected for inclusion in this seismic evaluation. The list of existing structures evaluated in this study are shown in Table 1.1. An aerial view of these structures is shown on Figure 1.1.

Table 1.1 List of Structures Evaluated

Structure Name	Type	Approximate Date Built
Operations Building	Building	1995
Process Gallery	Building	1995
Workshop	Building	1979
Aeration Basins and Stabilization Basins	Water-bearing Basin	1993
Sludge Storage Basins and Biofilter	Water-bearing Basin	1979

The purpose of the evaluation was to identify seismic vulnerabilities and deficiencies to consider for enhancement of plant resiliency. The seismic evaluation was performed using the procedures established by American Society of Civil Engineers (ASCE) Standard: Seismic Evaluation and Retrofit of Existing Buildings 41-17 (ASCE 41-17). The standard prescribes a three-tiered approach for the seismic evaluation: Tier 1 - Screening, Tier 2 - Deficiency-based evaluation and retrofit and Tier 3 - Systematic evaluation and retrofit. For this evaluation, analysis procedures were limited to the Tier 1 level and Tier 2 level as required. The balance of this report presents background information, a description of seismic evaluation criteria and procedures used, findings, mitigation recommendations, including conceptual level mitigation cost estimates.

Geo-seismic site hazards were assessed as part of this study and findings are summarized in a technical memorandum prepared by Northwest Geotech, Inc. that is presented in Appendix D. Non-structural components were also evaluated in accordance with the Tier 1 procedures set forth in ASCE 41-17 for each of the five structures as well as the overall plant as part of this study.

Figure 1.1 Aerial View of the Structures Evaluated

1.2 Background Information

The required information for this evaluation was obtained by reviewing the existing record drawings and by performing a site visit. The typical structures are one-story tall except for the process gallery, which has a below grade basement. The structural systems consisted of reinforced concrete masonry (CMU) shear walls, cast-in-place concrete shear walls, or wood framed shear walls with wood or metal deck roof diaphragms. Table 1.2 provides detailed information about the structural systems for the structures that were evaluated.

Table 1.2 Detailed Structural Information for Structures Evaluated

Detailed Structural Information for Structures Evaluated	
Operations Building	
No. of Stories	One-Story
Vertical Seismic System	Reinforced CMU Shear Walls
Vertical Gravity System	Reinforced CMU Walls and Tube Steel Columns
Roof Gravity System	Steel Open-Web Joists and Steel Beams
Roof Diaphragm	Steel Roof Decking
Foundation System	Shallow Spread and Wall Footings
Process Gallery	
No. of Stories	One-story above grade + basement
Vertical Seismic System	Reinforced CMU Shear Walls + Cast-in-Place Concrete Shear Walls
Vertical Gravity System	Reinforced CMU Walls, Concrete Walls, and Concrete Columns
Gravity System	Steel Beams at Roof, concrete slab and concrete beams at grade level
Roof Diaphragm	Steel Roof Decking
Foundation System	Mat Slab
Workshop	
No. of Stories	One-Story
Vertical Seismic System	Wood Framed Shear Walls
Vertical Gravity System	Wood Framed Walls and Tube Steel Columns
Gravity System	Wood Joists and Glulam Beams
Roof Diaphragm	Plywood Sheathing
Foundation System	Shallow Spread and Wall Footings
Aeration and Stabilization Basins	
No. of Stories	One-Story (partially buried)
Vertical Seismic System	Cantilevered Concrete Walls
Vertical Gravity System	Concrete Walls
Roof Gravity System	N/A
Roof Diaphragm	N/A
Foundation System	Mat Slab
Solids Storage and Biofilter Basins	
No. of Stories	One- Story (mostly buried)
Vertical Seismic System	Cantilevered Concrete Walls
Vertical Gravity System	Concrete Walls
Roof Gravity System	N/A
Diaphragms	N/A
Foundation System	Mat Slab

Notes:
Abbreviations: No. - number; N/A - not applicable.

Modifications were made to some of the structures after their original construction. The aeration basins were modified in 2014 by adding a third basin to the east of basin No. 1 as well as two stabilization basins to the west of basin No. 2 and a blower canopy. The sludge storage and biofilter basins were modified in 2014 to include a concrete wall within the biofilter.

1.3 Seismic Evaluation Criteria

Seismic evaluation of the buildings was performed using the ASCE 41-17 prescribed Tier 1 screening evaluation. The purpose of Tier 1 screening is to efficiently identify potential deficiencies or identify the need for additional investigation. Tier 1 screening is performed using checklists and quick procedure calculations. The Tier 1 evaluation requires selection of a Performance Objective using the Structural Performance Levels and Seismic Hazard Levels defined within ASCE 41-17.

The results obtained from the above two-stage analysis for the performance objective selected is presented in this report. After Tier 1 evaluation, there are two additional evaluations: Tier 2 - Deficiency based and Tier 3 - Systematic Evaluation. In the Tier 2 evaluation, the deficiencies observed in Tier 1 can be further evaluated by performing more detailed analysis and calculations, while the Tier 3 evaluation involves performing detailed linear and non-linear finite element mathematical models for the buildings. The Tier 2 evaluation was performed for items deemed deficient from Tier 1 while Tier 3 evaluation was not considered as part of this study.

Since a portion of the structures included in the scope of work are non-building structures with structural systems and load paths that are not similar to buildings, for the seismic evaluation we chose to apply the relevant design standard, which is American Concrete Institute (ACI) 350.3-06, "Seismic Design of Liquid-Containing Concrete Structures and Commentary," recognizing that no relevant seismic evaluation guides or standards are available for existing concrete tanks.

1.3.1 Performance Objective

The performance objective is typically a two-fold objective that establishes building performance levels for different seismic hazards. For example, a typical performance objective for a non-essential building might be meeting the life safety performance level when subjected to an earthquake having a return period of 225 years and meeting the collapse prevention performance level when subjected to an earthquake having a return period of 975 years.

Structures that are considered to have an elevated or essential function to society are expected to have relatively higher structural performance levels. To address this need, the 2019 OSSC classifies structures into Risk Categories. Essential facilities, such as fire stations, emergency response centers, reservoirs, pump stations and intake structures are typically classified as Risk Category IV structures and are evaluated with stringent performance objectives, since an interruption in the operation of these facilities can result in a significant and immediate hazard to the general public. Risk Category III structures are generally considered to serve an important role, but their structural performance requirements after a major seismic event are less stringent than that of a Risk Category IV structure but higher than that of a Risk Category II structure.

The structures evaluated in this study were classified as Risk Category III based on the functionality. As prescribed by ASCE 41-17 a performance objective of Basic Performance Objective for Existing Buildings (BPOE) was selected for these Risk Category III structures. The concrete tanks were also considered to be Risk Category III structures and an importance factor of 1.25 was used in the evaluation.

1.3.2 Performance Level

Building performance levels include both structural and non-structural performance levels. The structural performance levels defined in ASCE 41-17 are as follows:

- S-1: Immediate Occupancy.
- S-2: Damage Control.
- S-3: Life Safety.
- S-4: Limited Safety.
- S-5: Collapse Prevention.
- S-6: Not Considered.

Non-structural performance levels defined in ASCE 41-17 are as follows:

- N-A: Operational.
- N-B: Position Retention.
- N-C: Life Safety.

The performance level of a structure can be described in terms of:

1. Safety of the building occupants during and after a seismic event.
2. Cost of restoring the building to its pre-event condition.
3. Length of time the building is removed from service, i.e., not occupiable.

To help provide some perspective, the definitions of the S-1 and S-3 structural performance levels are as followS:

S-1: Immediate Occupancy: "Immediate Occupancy" refers to the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical- and lateral-force resisting systems of the building retain almost all their pre-earthquake strength and stiffness. The risk of life-threatening injury from structural damage is very low, and although some minor structural repairs might be appropriate, these repairs would generally not be required before re-occupancy. Continued use of the building is not limited by its structural condition but might be limited by damage or disruption to nonstructural elements of the building, furnishings, or equipment and availability of external utility services.

S-3: Life Safety: "Life Safety" refers to the post-earthquake damage state in which significant damage to the structure has occurred but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this damage has not resulted in large falling debris hazards, either inside or outside the building. Injuries might occur during the earthquake, however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure, however, for economic reasons, this repair might not be practical. Although the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing before re-occupancy.

The tanks do not have an associated structural performance level but are expected to sustain similar damage levels as those described for buildings.

1.3.3 Seismic Hazard Level

The seismic hazard level quantifies the magnitudes of spectral response accelerations the structures will experience in an earthquake. Per ASCE 41-17 two levels of seismic hazards should be considered when using the BPOE performance objective defined above for existing buildings. These are Basic Safety Earthquake-1 for use with existing buildings (BSE-1E) and Basic Safety Earthquake-2 for use with existing buildings (BSE-2E). These seismic hazards quantify the probabilistic magnitude of ground shaking that might occur at the project site. The BSE-1E and BSE-2E are defined as follows:

- BSE-1E: Taken as a seismic hazard with a 20 percent probability of exceedance in 50 years. This ground motion has an approximate return period of 225 years.
- BSE-2E: Taken as a seismic hazard with a 5 percent probability of exceedance in 50 years. This ground motion has an approximate return period of 975 years.

The BSE-1E and BSE-2E seismic hazards result in smaller forces in the structures than those compared to the forces obtained when designing a new building per the ASCE 7-16 load criteria. The reduced seismic hazard used in this evaluation is justified for existing buildings because the remaining service life for the existing building is less than that for new buildings and thus the magnitude of earthquakes experienced by existing buildings are likely to be smaller, given the reduced exposure period.

This evaluation also considered a single seismic hazard associated with a magnitude 9.0 (M9.0) scenario earthquake originating on the Cascadia Subduction Zone (CSZ). A geotechnical memorandum was prepared (Northwest Geotech, Inc., 2021) that provided estimates of the spectral acceleration and geologic hazards associated with the M9.0 CSZ scenario earthquake. Refer to Appendix D for the geotechnical memorandum.

The ASCE 41-17 BSE-1E and the BSE-2E seismic hazards used in this seismic evaluation are summarized in Tables 1. 3 and 1.4. The ground motion is based upon the seismic data obtained from the United States Geological Survey (USGS). The CSZ seismic hazard used in the seismic evaluation is summarized in Table 1.5.

Table 1.3 BSE-1E Seismic Parameters

Parameter	Value
Latitude	45.29 N
Longitude	122.77 W
$S_{S,\ 20/50}$	0.223 g
$S_{1,\ 20/50}$	0.082 g
Site Class	C
F_a	1.3
F_v	1.5
$S_{XS,\ BSE\text{-}1E}$	0.291 g
$S_{X1,\ BSE\text{-}1E}$	0.123 g

Notes:
Abbreviations: N – north; W – west; g – acceleration due to Gravity; C – Soil Site Class Type; $S_{S,\ 20/50}$ – Spectral Response Acceleration Parameter at Short Periods for any Seismic Hazard Level and any Damping, Adjusted for Site Class;
$S_{1,\ 20/50}$ – Spectral Response Acceleration Parameter at a 1 Second Period for any Seismic Hazard Level and any Damping, Adjusted for Site Class;
F_a – Factor to Adjust Spectral Acceleration in the short period range for Site Class;
F_v – Factor to Adjust Spectral Acceleration at 1 Second for Site Class;
$S_{XS,\ BSE\text{-}1E}$ – Spectral Response Acceleration Parameter at Short Periods for BSE-1E Seismic Hazard Level and any Damping, Adjusted for Site Class;
$S_{X1,\ BSE\text{-}1E}$ – Spectral Response Acceleration Parameter at a 1 Second Period for BSE-1E Seismic Hazard Level and any Damping, Adjusted for Site Class.

Table 1.4 BSE-2E Seismic Parameters

Parameter	Value
Latitude	45.29 N
Longitude	122.77 W
$S_{S,\ 5/50}$	0.598 g
$S_{1,\ 5/50}$	0.27 g
Site Class	C
F_a	1.265
F_v	1.5
$S_{XS,\ BSE\text{-}2E}$	0.744 g
$S_{X1,\ BSE\text{-}2E}$	0.405 g

Notes:
Abbreviations: $S_{S,\ 5/50}$ – Spectral Response Acceleration Parameter at Short Periods for any Seismic Hazard Level and any Damping, Adjusted for Site Class;
$S_{1,\ 5/50}$ – Spectral Response Acceleration Parameter at a 1 Second Period for any Seismic Hazard Level and any Damping, Adjusted for Site Class;
$S_{XS,\ BSE\text{-}2E}$ – Spectral Response Acceleration Parameter at Short Periods for BSE-2E Seismic Hazard Level and any Damping, Adjusted for Site Class;
$S_{X1,\ BSE\text{-}2E}$ – Spectral Response Acceleration Parameter at a 1 Second Period for BSE-2E Seismic Hazard Level and any Damping, Adjusted for Site Class.

Table 1.5 CSZ Seismic Parameters

Parameter	Value
Latitude	45.29 N
Longitude	122.77 W
S_S	0.343 g
S_1	0.221 g
Site Class	C
F_a	1.3
F_v	1.5
$S_{XS,\ CSZ}$	0.446 g
$S_{X1,\ CSZ}$	0.332 g

Notes:
Abbreviations: S_S – Spectral Response Acceleration Parameter at Short Periods for any Seismic Hazard Level and any Damping, Adjusted for Site Class;
S_1 – Spectral Response Acceleration Parameter at a 1 Second Period for any Seismic Hazard Level and any Damping, Adjusted for Site Class;
$S_{XS,\ CSZ}$ – Spectral Response Acceleration Parameter at Short Periods for CSZ Seismic Hazard Level and any Damping, Adjusted for Site Class;
$S_{X1,\ CSZ}$ – Spectral Response Acceleration Parameter at a 1 Second Period for CSZ Seismic Hazard Level and any Damping, Adjusted for Site Class.

1.3.4 Selection of Performance Objectives

Taking into account the performance levels and seismic hazards described above, per ASCE 41-17 Risk Category III existing structures were evaluated for Damage Control (S-2) at the BSE-1E seismic hazard level and Limited Safety (S-4) at the BSE-2E seismic hazard level. From Table 1.5 above, the CSZ seismic hazard is greater than those for the BSE-1E seismic hazard level. Our analysis replaced the BSE-1E seismic hazard for the higher seismic ground motions associated with the CSZ seismic hazard. For Tier 1 analyses the performance objectives are deemed to be satisfied if the analysis is performed for BSE-2E using Limited Safety Structural Performance parameters and Position Retention Non-Structural Performance parameters provided in ASCE 41-17. However, because the CSZ hazard exceeds that of the BSE-2E, the Tier 1 checks were explicitly evaluated at the CSZ hazard level.

1.4 Seismic Evaluation and Analysis

The buildings were evaluated using the Tier 1 and Tier 2 procedures set forth in ASCE 41-17. The seismic evaluation and analysis comprised of data collection and review, a site visit, completion of Tier 1 checklists and calculations, and Tier 2 calculations based on the deficiencies found in the Tier 1 evaluation.

Similarly, the basins were evaluated using ACI 350-06 and ACI 350.3-06 with the same seismic hazard levels as the buildings, but the following adjustments were made to adapt the use of ACI 350-06 and ACI 350.3-06 for seismic evaluation, rather than design:

- Importance factor = 1.25.
- Load factors were limited to 1.0 for load combinations.
- Capacity-reduction factors were set equal to 1.0.

1.4.1 Data Collection and Review

To obtain data and information necessary for performing the seismic evaluation, the following construction documents and reports were reviewed:

- City of Wilsonville, Oregon Sewage Treatment Plant Phase III Expansion, prepared by CH2M Hill, dated August 1979.
- City of Wilsonville Wastewater Treatment Plant, prepared by CH2M Hill, dated December 1995.
- City of Wilsonville Wastewater Treatment Plant Improvements DBO, prepared by CH2M Hill, dated June 2012.
- Technical Memorandum, prepared by Northwest Geotech, Inc., dated June 2021.

The material properties used in this evaluation are listed in Table 1.6. These properties were determined based on information shown in the record drawings. For the properties that could not be obtained from the record drawings, the values were obtained from the default historical material properties prescribed in ASCE 41-17, Chapter 4.

Table 1.6 Material Properties

Material and Mechanical Property	Value
Concrete Compressive Strength	f'_c = 4,000 psi
Reinforcing Steel (ASTM A615 G60) Yield Strength	f_y = 60,000 psi
Steel Framing (ASTM A36) Yield Strength	F_y = 36,000 psi
Corrugated Steel Roof Deck (ASTM A446) Yield Strength	F_y = 50,000 psi

Notes:
Abbreviations: ASTM – ASTM International; f'_c – Concrete Compressive Strength; f_y – Yield Strength of Rebar; F_y – Yield Strength of Steel; psi - pounds per square inch.

1.4.2 Site Visit

A site visit was conducted by James Doering, S.E., and Brian Stuetzel, E.I.T., of Carollo on June 16 and 17, 2021. The site visit included review of both interior and exterior spaces with access gained to the basement and ground floor levels. Access to the roof levels was not available. The buildings were in full operation at the time of the site visit.

The site visit objectives included:

- Verification that the structures are generally configured and constructed in accordance with the record drawings.
- Completion of Tier 1 checklist items that required visual verification.
- Identification of additional loads that are to be included in the seismic analysis, such as equipment, piping, and ceilings.
- Structural condition assessment of visual portions of the structures.
- Non-Structural items within the structures and around the plant.

Observations made during the site visits were collected using an Apple iPhone and a digital camera. Photographs collected during the site visit are included in Appendix A for reference.

1.4.3 Analysis Procedures

Analysis procedures followed those set forth in ASCE 41-17, ACI 350-06, and ACI 350.3-06. For the most part, the structures did not have any significant irregularities.

The analysis included the structure above grade and those portions of the structure below grade that are within the seismic load path for lateral load resistance. Loads applied to the structure include dead loads, live loads, inertial seismic loads, and hydrodynamic loads where applicable. Load combinations analyzed were limited to those that include seismic loads. Load intensities and material unit weights assumed for the evaluation are presented in Table 1.7.

Refer to Appendix B for the Tier 1 checklists and calculations.

Table 1.7 Load Intensities and Material Unit Weights

Load/Material	Value
Unit Weight of Concrete	150 pcf
Unit Weight of Steel	490 pcf
Roof Live Load	20 psf
Roof Snow Load	25 psf
Floor Live Load	250 psf

Notes:
Abbreviations: pcf - pounds per cubic foot; psf - pounds per square foot.

1.4.4 Acceptance Criteria

The analysis involves the estimation of seismic load and deformation demands placed upon structural members. These demands are compared against their estimated capacity, which is a function of the member proportions, material properties, and desired performance level. The metric used in this evaluation to quantify the degree of distress of an existing member or connection is referred to as the demand to capacity ratio (DCR):

$$DCR = \frac{Load\ Demand}{Available\ Capacity}$$

DCR values that exceed 1.0 are typically considered to have exceeded their capacity for the evaluated performance level and are considered deficient.

The estimated capacity is a function of the material properties. For this evaluation, the material properties have been obtained from the record construction documents. For Tier 2 investigations and beyond, ASCE 41-17 requires that a knowledge factor be applied to the material property depending on which type of construction documents served as the source for the material information. Since the information for materials is provided on the construction documents, the knowledge factor has been assumed to be 0.90.

1.5 Evaluation Findings

The results from review of the record drawings, site visit and structural calculations are presented in this section. The structural members and connection capacities were checked against the demands imposed by the prescribed seismic loads as described in previous sections to obtain corresponding DCRs. DCR values that exceed 1.0 indicate a deficiency with respect to the evaluated performance level. The members and connections that were found to be deficient for each building are listed in Table 1.8.

Table 1.8 List of Deficiencies

No.	Deficiency	Description
Operations Building		
S1	Load path / Transfer to Shear Walls	No drag connections to transfer diaphragm forces into shear walls where those walls are discontinuous within the plan of the building.
S2	Plan Irregularities	No diaphragm ties in the N-S direction to transfer diaphragm forces into shear walls.
NS1	Edge Clearance	The ceiling edges do not have a sufficient gap between the enclosing walls and this could cause damage due to restraint.
NS2	Lens Covers	Lens covers over lights lack safety device.
NS3	Overhead Glazing	Windows above entrance appear to lack proper restraint in frame if cracked or damaged.
NS4	Tall Narrow Contents	Storage racks lack restraint to structure. Also, the refrigerator in laboratory appears to lack restraint if wheels are locked.
NS5	Fall-Prone Contents / Suspended Equipment	The laboratory hoods could not be determined if adequate lateral bracing is attached back to structure. In addition, the air handler unit lacks anchorage to support structure.
Process Gallery		
S1	Load path / transfer to shear walls	The roof beam aligned with interior shear wall lacks ability to transfer seismic loads into the shear wall.
NS1	In-line Equipment	Air handling unit lacks anchorage along channel support. Also, the aeration blower pumps in basement lack proper anchorage back to structure.
NS2	Fluid and Gas Piping	Multiple pipes lack restraint to Unistrut support below. In addition, the compression struts for RAS piping lack diagonal bracing back to structure.
Workshop		
S1	Narrow Wood Shear Walls	The shear wall segments along the east elevation cannot develop the overturning forces due to a lack of holdowns at the ends of each shear wall segment.
S2	Narrow Wood Shear Walls	The shear wall segments along the east elevation do not have sufficient shear capacity to resisting the in-plane seismic loads.
S3	Narrow Wood Shear Walls	The shear wall segments along the east elevation do not have adequate sill bolt anchorage for resisting the in-plane seismic loads.
NS1	Tall Narrow Contents	The storage racks within building lack restraint back to the structure. In addition, the shelving unit along south elevation lacks anchorage across entire length.

No.	Deficiency	Description
Stabilization Basins		
S1	Freeboard	The longitudinal sloshing direction results in a freeboard deficit of about 1.2 feet. The aluminum covers can be damaged by sloshing water.
Sludge Storage Basins		
S1	Freeboard	The longitudinal sloshing direction results in a freeboard deficit of about 1.6 feet. The membrane covers can be damaged by sloshing water.
Overall Plant Structures		
NS1	Tall Narrow Contents	Storage racks within the Headworks building lack anchorage back to structure.
NS2	In-Line Equipment	Recirculation pump at Disk Filters lacks restraint against overturning.
NS3	Heavy Equipment	ACCU units near the aeration basins lack anchorage to structural pads.

Notes:
Abbreviations: ACCU - air cooled condensing unit; RAS - return activated sludge.

Based on the observed deficiencies, mitigation methods are outlined in the following section along with a planning level cost estimates to mitigate these deficiencies.

1.6 Recommendations for Mitigation

In this section mitigation measures to address the structural deficiencies are presented. The observed structural deficiencies can be mitigated by performing reasonable retrofit and strengthening of the existing buildings. Following is a detailed discussion for each of the observed structural deficiencies and potential mitigation strategies.

1.6.1 Load Path / Transfer to Shear Walls

There needs to be a direct load path for the seismic forces to be transferred from the roof level down to the foundation level. The loads will eventually need to be transferred into the shear walls, and one way to ensure transfer into the shear wall system is through collector beams. To mitigate this issue, adding collector beams and associated connections to the shear walls will allow for the seismic loads to transfer into the lateral load resisting system.

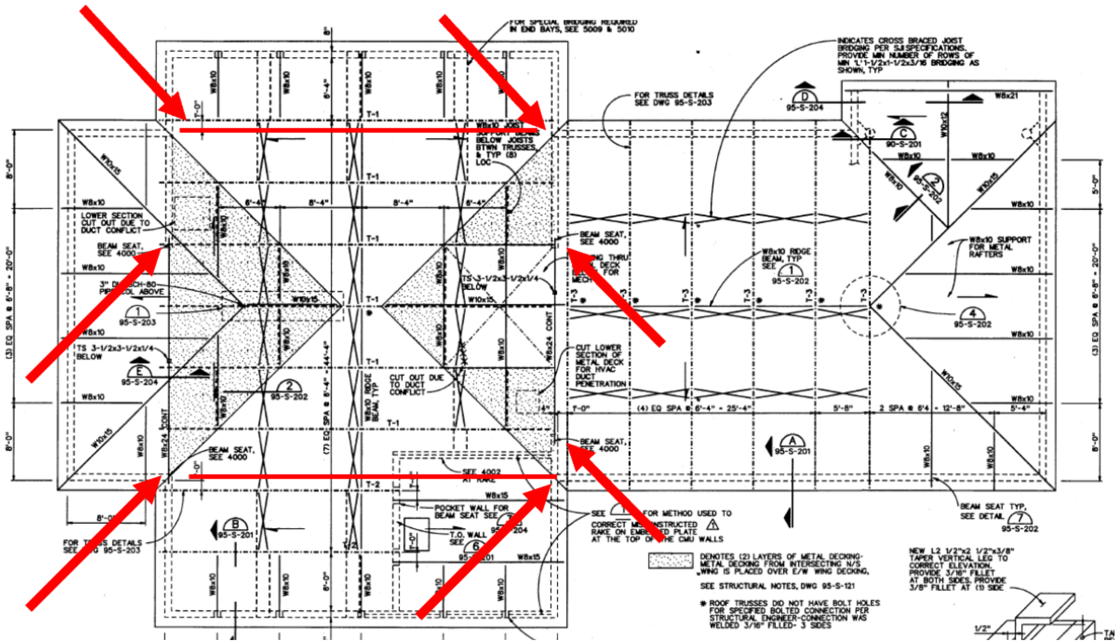

Figure 1.2 Operations Building - Collector Beam Locations and Anchorage Deficiencies

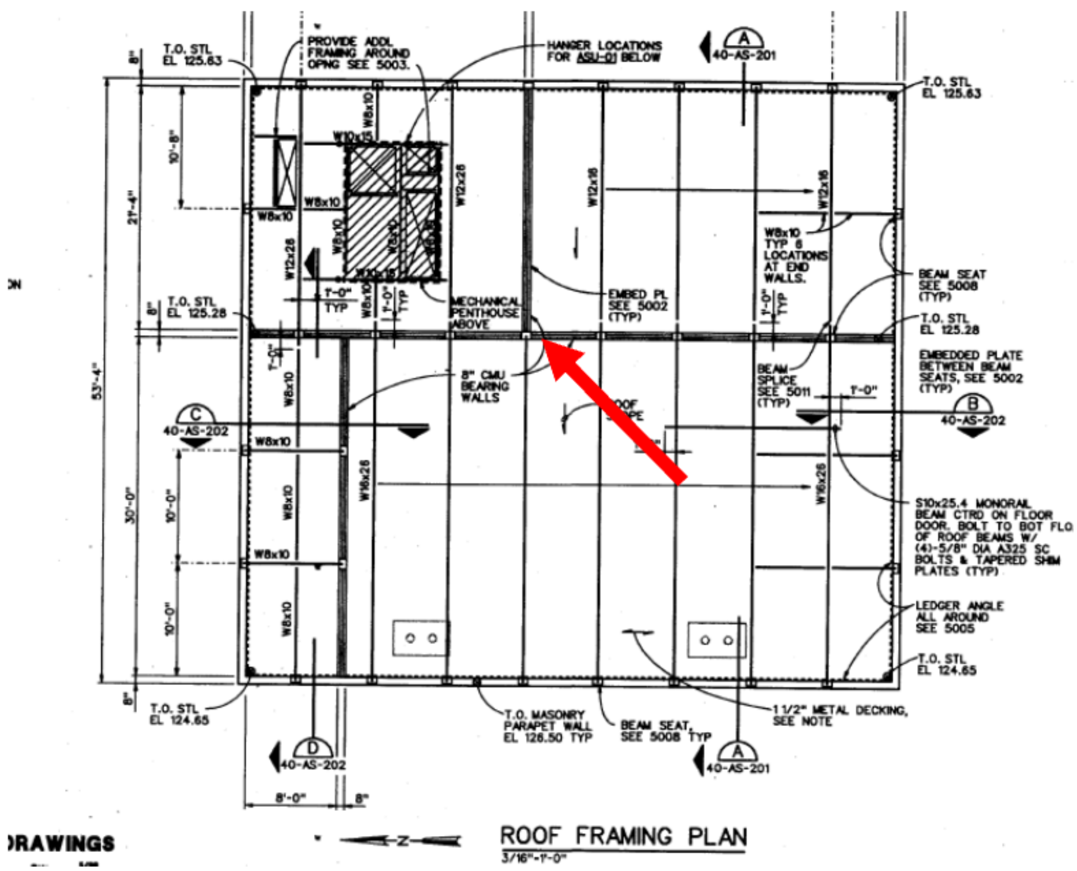

Figure 1.3 Process Gallery - Collector Beam Location and Anchorage Deficiency

1.6.2 Narrow Wood Shear Walls

Narrow wood shear walls tend to have reduced shear strength and overturning capacity to resist lateral forces. Since the wall height tends to be significantly larger than the wall length, the overturning forces on the wall can cause damage at the wall base and render it less effective at resisting shear forces. A Tier 2 evaluation for the east shear wall segments was performed and the overturning capacity, shear capacity, and shear anchorage to the foundation were found to be deficient. The following mitigation is recommended for the east wall of the workshop:

- Provide holdown anchorage for each end of the middle (2) shear wall segments. Holdown anchorage is typically comprised of a prefabricated metal connector with a post-installed anchor into the building foundation/footing.
- Enhance the shear strength of the middle (2) shear wall segments by providing a plywood overlay at the interior side of the wall.
- Enhance the shear anchorage capacity at the sill plate connection to the foundation/footing by providing additional sill plate anchors to reduce the overall spacing of sill bolts to no more than 24 inches. Sill plate anchorage should be comprised of a post-installed anchor into the foundation/footing and associated plate washer on top of the sill plate.
- Provide top plate straps where splices occur in the stud wall top plates. Because only the middle (2) shear wall segments are considered to be effective in resisting in-plane seismic loads, the top plates will be required to transmit diaphragm loads to the middle (2) shear walls and the configuration of the top plates may require supplemental ties to ensure seismic loads can be effectively transmitted to the middle (2) shear walls.

1.7 Cost Estimates

To assist the City of Wilsonville with their planning efforts to improve reliability of these buildings, we have developed a rough order of magnitude of cost associated with mitigating the seismic deficiencies identified. The construction cost estimate includes direct and indirect costs. Direct costs include materials, labor, and construction equipment required for the retrofit. This cost also includes removal and re-installation of the interior finishes to allow access to perform recommended mitigation. Various indirect cost and non-construction cost factors that have been included in the total estimated project cost are identified in Appendix C.

Cost estimates provided in this evaluation/study are considered to be a Class 5 estimate as defined in "Recommended Practice 18R-97 Cost Estimate Classification System for the Process Industries," published by the Association for the Advancement of Cost Engineering (AACEI). These costs are anticipated to have an accuracy range of +50 percent to -30 percent and are intended for planning purposes. The unit costs in Appendix C are derived from RS Means and Carollo's cost estimate database.

A summary of the cost estimate is provided in Table 1.9. The detailed breakdown of the cost estimate is provided in Appendix C. The total estimated cost for mitigating all the seismic deficiencies identified is $810,400.

Table 1.9 Summary of Retrofit Cost Estimate

Structure	Cost Estimate
Operations Building	$646,900
Process Gallery	$44,800
Workshop	$114,400
Overall Plant (Non-Structural)	$4,300
TOTAL	**$810,400**

1.8 Conclusion

The goal of the seismic evaluation was to identify specific seismic vulnerabilities and deficient structural conditions for the purpose of improving the overall reliability of the subject buildings. Our findings presented in this report identify numerous seismic vulnerabilities and deficient conditions that warrant retrofit. Mitigation strategies for retrofit were developed and presented in this report along with cost estimates.

1.9 References

1.9.1 Standards

American Society of Civil Engineers (ASCE). (2017). "*Seismic Evaluation and Retrofit of Existing Buildings.*" ASCE/SEI 41-17.

American Society of Civil Engineers (ASCE). (2016). "*Minimum Design Loads for Buildings and other Structures,*" ASCE/SEI 7-16.

American Concrete Institute (ACI). (2014), "*Building Code Requirements for Structural Concrete.*" ACI 318-14, Farmington Hills, MI.

AISC (2005), *"Specification for Structural Steel Buildings."* American Institute of Steel Construction, Inc., Chicago, IL.

American Forest & Paper Association/American Wood Council (2018), ANSI/AF&PA National Design Specification for Wood Construction.

2019 Oregon Structural Specialty Code.

2018 International Building Code.

1.9.2 Reports

Technical Memorandum, prepared by Northwest Geotech, Inc., dated June 25, 2021.

Appendix A
SITE VISIT PHOTOGRAPHS

Figure 1 Operations Building - East Elevation View

Figure 2 Operations Building - Roof Joist Framing

Figure 3 Operations Building - Roof Steel Framing

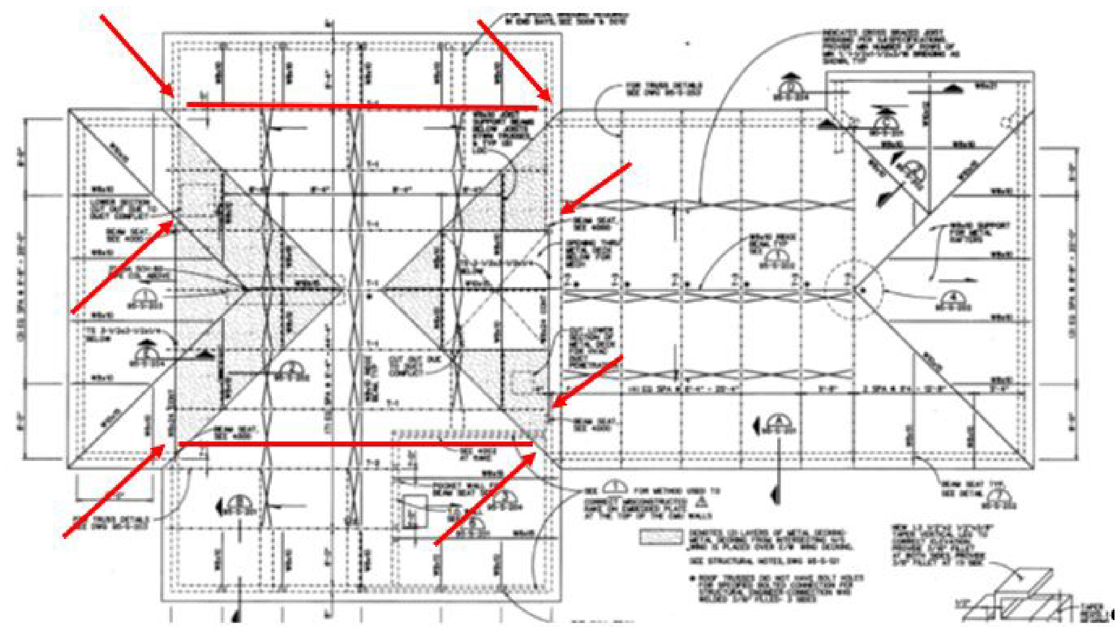

Figure 4 Operations Building - Drag Connection Deficiency Locations

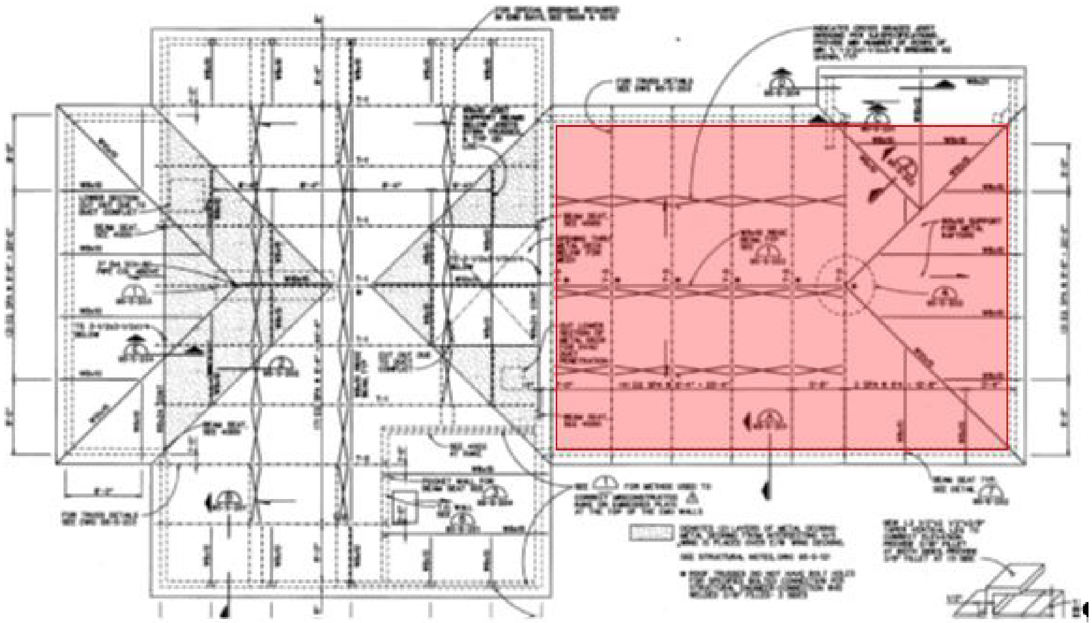

Figure 5 Operations Building - Diaphragm Span Location Exceeding 40 feet Deficiency

Figure 6 Operations Building - Ceiling Clearance to Wall

Figure 7 Operations Building - Lens Cover Lacks Safety Device

Figure 8 Operations Building - Window Above Entrance Appears to Lack Special Treatment to Limit Damage

Figure 9 Operations Building - Collector Beam Connection to CMU Wall

Figure 10 Process Gallery - South Elevation

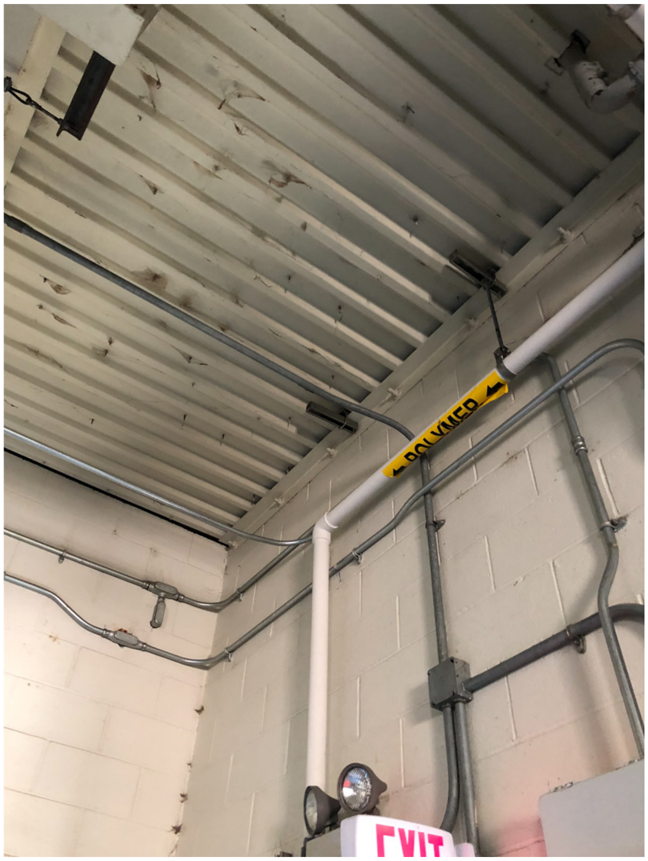

Figure 11 Process Gallery - Wall Anchorage

Figure 12 Process Gallery - Beam Anchorage

Figure 13 Process Gallery - Basement Interior View

Figure 14 Process Gallery - Air Handling Unit Lacking Anchorage

Figure 15 Process Gallery - Blower Equipment with Missing Nuts

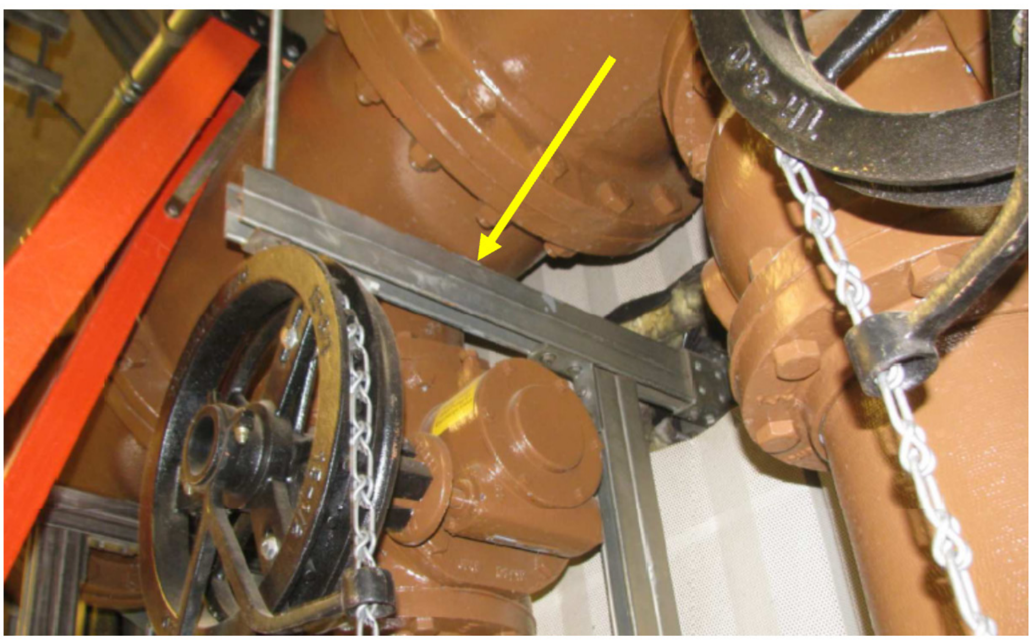

Figure 16 Process Gallery - Piping Lacks Connection Back to Supports Below

Figure 17 Process Gallery - RAS Piping Lacks Lateral Bracing Along Length at Compression Struts

Figure 18 Workshop - North Elevation

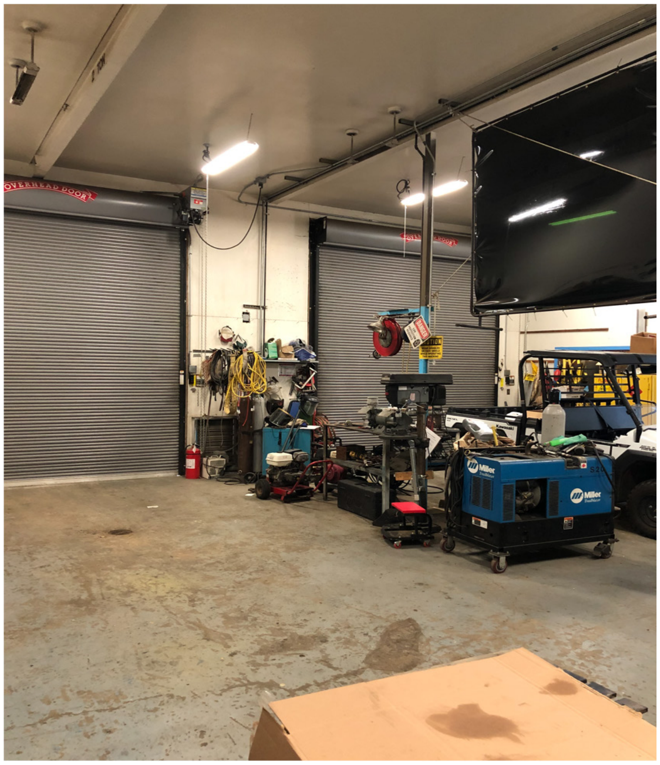

Figure 19 Workshop - Interior View

Figure 20 Workshop - Storage Room Interior View

Figure 21 Workshop - East Elevation Shear Walls are Considered Narrow and Could Limit Strength

Figure 22 Workshop - Storage Shelving Lacks Restraint to Structure

Figure 23 Workshop - Storage Shelves on South Wall Missing Anchorage

Figure 24 Aeration Basin - Top View

Figure 25 Stabilization Basin - Top View

Figure 26 Stabilization Basin - Walkway with Piping and Support

Figure 27 Sludge Storage and Biofilter Basins - Top View

Figure 28 Sludge Storage and Biofilter Basins - Pump Equipment

Figure 29 Headworks Building - Shelving Lacks Anchorage to Structure

Figure 30 Disk Filters - Recirculation Pump Lacking Resistance to Overturning

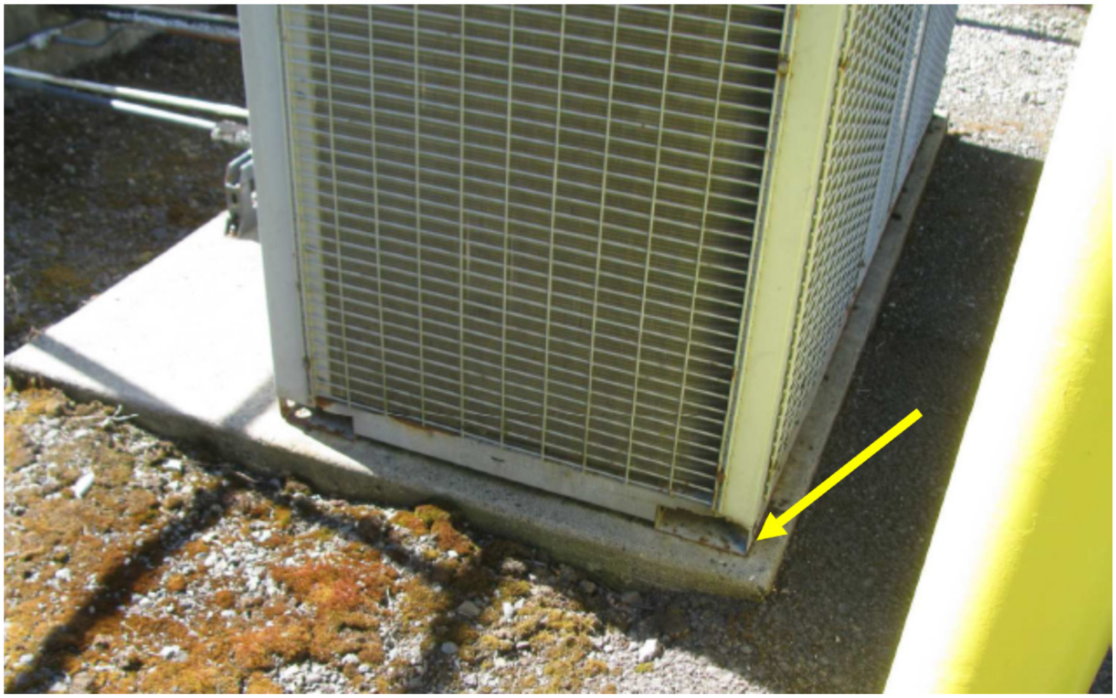

Figure 31 Aeration Basins Canopy - ACCU Units Lacking Anchorage To Concrete Pad

Appendix B

ASCE 41-17 TIER 1 CHECKLISTS AND CALCULATIONS / TIER 2 CALCULATIONS

City of Wilsonville
Wastewater Treatment Plant
Structural Checklists & Calculations

Table of Contents

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Operations Building
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	07/01/21
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Op

Project Number 11962A.00

Table 17-2. Collapse Prevention Basic Configuration Checklist

Very Low Seismicity

BSE-2E Seismic Check at Limited Safety

Structural Components

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☒	N/A ☐	U ☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	There are no diaphragm ties in the N-S direction where perimeter CMU shear walls stop. The diaphragm at these locations will lack the ability to transfer diaphragm forces into the shear walls.
C ☒	NC ☐	N/A ☐	U ☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps taht are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	Roof joist bearing anchorage DCR = 0.23(OK) E-W beam bearing anchorage DCR = 0.55 (OK) N-S beam bearing anchorage DCR = 0.24 (OK)

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Op
Project Number 11962A.00

Low Seismicity

Building System

General

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☒	N/A ☐	U ☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	There are no diaphragm ties in the N-S direction where perimeter CMU shear walls stop. The diaphragm at these locations will lack the ability to transfer diaphragm forces into the shear walls.
C ☐	NC ☐	N/A ☒	U ☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement need not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	
C ☐	NC ☐	N/A ☒	U ☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name	City of Wilsonville - Op
Project Number	11962A.00

Building Configuration

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1)	Building is a one-story structure.
C ☐	NC ☐	N/A ☒	U ☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	Building is a one-story structure.
C ☒	NC ☐	N/A ☐	U ☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
C ☐	NC ☐	N/A ☒	U ☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	Building is a one-story structure.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Op

Project Number 11962A.00

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	Building is a one-story structure.
☐	☐	☒	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	Torsion check applies for structures with rigid diaphragms, not for flexible diaphragms.

Moderate Seismicity

Geologic Site Hazards

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)	Liquefaction has been determined to not be an issue per NGI technical memorandum.
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)	Slope failure has been determined to not be an issue per NGI technical memorandum.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C ☒	NC ☐	N/A ☐	U ☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)	Surface fault rupture has been determined to not be an issue per NGI technical memorandum.

High Seismicity

Foundation Configuration

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)	Height = 10.25ft Base = 58ft Sa = 0.744 B/H = 58ft / 10.25ft = 5.66 0.6*Sa = 0.6 * 0.744 = 0.45 5.66 > 0.45 (OK)
C ☒	NC ☐	N/A ☐	U ☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Operations Building
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	07/01/21
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Project Name	City of Wilsonville - Op
Project Number	11962A.00

Table 17-33. Collapse Prevention Structural Checklist for Building Types RM1 and RM2

Low and Moderate Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	DESCRIPTION	COMMENTS
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than 70 lb/in.^{2}. (Commentary: Sec. A.3.2.4.1. Tier 2: Sec. 5.5.3.1.1)	West wall line DCR = 0.06 (OK) East wall line DCR = 0.08 (OK) North wall line DCR = 0.46 (OK) South wall line DCR = 0.45 (OK)
☐	☒	☐	☐	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in., and all vertical bars extend to the top of the walls. (Commentary: Sec. A.3.2.4.2. Tier 2: Sec. 5.5.3.1.3)	Horiz steel = #5@48" Vert steel = #6@32" Horiz ratio = 0.31/ (7.625*48) = 0.0008 > 0.0007 (OK) Vert ratio = 0.44 / (7.625*32) = 0.0018 > 0.0007 (OK) Combined = 0.0018 + 0.0008 = 0.0026 > 0.002 (OK) Horizontal reinforcing is spaced at 48in, and this is not less than 48in spacing so NC.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Stiff Diaphragms

C	NC	N/A	U	DESCRIPTION	COMMENTS
☐	☐	☒	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	

Connections

C	NC	N/A	U	DESCRIPTION	COMMENTS
☒	☐	☐	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps taht are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	Roof joist bearing anchorage DCR = 0.23 (OK) E-W beam bearing anchorage DCR = 0.55 (OK) N-S beam bearing anchorage DCR = 0.24 (OK)
☐	☐	☒	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	
☐	☒	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)	There are no diaphragm ties in the N-S direction where perimeter CMU shear walls stop. The diaphragm at these locations will lack the ability to transfer diaphragm forces into the shear walls.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2: Sec. 5.7.2)	
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
☒	☐	☐	☐	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	

High Seismicity

Stiff Diaphragms

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)	

Flexible Diaphragms

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
☐	☐	☒	☐	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Statement	Comments
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm shall not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Connections

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Administration and Operations Building
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	07/01/2021
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

Table 17-38. Nonstructural Checklist

The Performance Level is designated LS for Life Safety or PR for Position Retention. The level of seismicity is designated as "not required" or by L, M, or H, for Low, Moderate, and High.

All Seismicity Levels

For BSE-1E Tier 1, use PR (Position Retention)

Life Safety Systems

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. FIRE SUPPRESSION PIPING: Fire suppression piping is anchored and braced in accordance with NFPA-13. (Commentary: Sec. A.7.13.1. Tier 2: Sec. 13.7.4)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Fire suppression piping has flexible couplings in accordance with NFPA-13. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.4)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. EMERGENCY POWER: Equipment used to power or control life safety systems is anchored or braced. (Commentary: Sec. A.7.12.1. Tier 2: Sec. 13.7.7)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. STAIR AND SMOKE DUCTS: Stair pressurization and smoke control ducts are braced and have flexible connections at seismic joints. (Commentary: Sec. A.7.14.1. Tier 2: Sec. 13.7.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-MH; PR-MH. SPRINKLER CEILING CLEARANCE: Penetrations through panelized ceilings for fire suppression devices provide clearances in accordance with NFPA-13. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.4)	
☐	☐	☒	☐	LS-not required; PR-LMH. EMERGENCY LIGHTING: Emergency and egress lighting equipment is anchored or braced. (Commentary: Sec. A.7.3.1. Tier 2: Sec. 13.7.9)	

Hazardous Materials

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-LMH; PR-LMH. HAZARDOUS MATERIAL EQUIPMENT: Equipment mounted on vibration isolators and containing hazardous material is equipped with restraints or snubbers. (Commentary: Sec. A.7.12.2. Tier 2: 13.7.1)	
☐	☐	☒	☐	LS-LMH; PR-LMH. HAZARDOUS MATERIAL STORAGE: Breakable containers that hold hazardous material, including gas cylinders, are restrained by latched doors, shelf lips, wires, or other methods. (Commentary: Sec. A.7.15.1. Tier 2: Sec. 13.8.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

C	NC	N/A	U	Statement	Comments
☐	☐	☒	☐	LS-MH; PR-MH. HAZARDOUS MATERIAL DISTRIBUTION: Piping or ductwork conveying hazardous materials is braced or otherwise protected from damage that would allow hazardous material release. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)	
☐	☐	☒	☐	LS-MH; PR-MH. SHUT-OFF VALVES: Piping containing hazardous material, including natural gas, has shut-off valves or other devices to limit spills or leaks. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.3 and 13.7.5)	
☐	☐	☒	☐	LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Hazardous material ductwork and piping, including natural gas piping, has flexible couplings. (Commentary: Sec. A.7.15.4, Tier 2: Sec.13.7.3 and 13.7.5)	
☐	☐	☒	☐	LS-MH; PR-MH. PIPING OR DUCTS CROSSING SEISMIC JOINTS: Piping or ductwork carrying hazardous material that either crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.13.6. Tier 2: Sec.13.7.3, 13.7.5, and 13.7.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

Partitions

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. UNREINFORCED MASONRY: Unreinforced masonry or hollow-clay tile partitions are braced at a spacing of at most 10 ft in Low or Moderate Seismicity, or at most 6 ft in High Seismicity. (Commentary: Sec. A.7.1.1. Tier 2: Sec. 13.6.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. HEAVY PARTITIONS SUPPORTED BY CEILINGS: The tops of masonry or hollow-clay tile partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. DRIFT: Rigid cementitious partitions are detailed to accommodate the following drift ratios: in steel moment frame, concrete moment frame, and wood frame buildings, 0.02; in other buildings, 0.005. (Commentary A.7.1.2 Tier 2: Sec. 13.6.2)	
C ☒	NC ☐	N/A ☐	U ☐	LS-not required; PR-MH. LIGHT PARTITIONS SUPPORTED BY CEILINGS: The tops of gypsum board partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-MH. STRUCTURAL SEPARATIONS: Partitions that cross structural separations have seismic or control joints. (Commentary: Sec. A.7.1.3. Tier 2. Sec. 13.6.2)	
C ☒	NC ☐	N/A ☐	U ☐	LS-not required; PR-MH. TOPS: The tops of ceiling-high framed or panelized partitions have lateral bracing to the structure at a spacing equal to or less than 6 ft. (Commentary: Sec. A.7.1.4. Tier 2. Sec. 13.6.2)	Partition walls appear to be anchored to be attached to bottom chord of truss members.

Ceilings

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	LS-MH; PR-LMH. SUSPENDED LATH AND PLASTER: Suspended lath and plaster ceilings have attachments that resist seismic forces for every 12 ft^2 of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)	
C ☒	NC ☐	N/A ☐	U ☐	LS-MH; PR-LMH. SUSPENDED GYPSUM BOARD: Suspended gypsum board ceilings have attachments that resist seismic forces for every 12 ft^2 of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Item	Comments
☒	☐	☐	☐	LS-not required; PR-MH. INTEGRATED CEILINGS: Integrated suspended ceilings with continuous areas greater than 144 ft^2, and ceilings of smaller areas that are not surrounded by restraining partitions, are laterally restrained at a spacing no greater than 12 ft with members attached to the structure above. Each restraint location has a minimum of four diagonal wires and compression struts, or diagonal members capable of resisting compression. (Commentary: Sec. A.7.2.2. Tier 2: Sec. 13.6.4)	
☐	☒	☐	☐	LS-not required; PR-MH. EDGE CLEARANCE: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 have clearances from the enclosing wall or partition of at least the following: in Moderate Seismicity, 1/2 in.; in High Seismicity, 3/4 in. (Commentary: Sec. A.7.2.4. Tier 2: Sec. 13.6.4)	Ceiling edges are placed next to partitions and exterior framing. No gap is provided between.
☐	☐	☒	☐	LS-not required; PR-MH. CONTINUITY ACROSS STRUCTURE JOINTS: The ceiling system does not cross any seismic joint and is not attached to multiple independent structures. (Commentary: Sec. A.7.2.5. Tier 2: Sec. 13.6.4)	
☐	☐	☒	☐	LS-not required; PR-H. EDGE SUPPORT: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 are supported by closure angles or channels not less than 2 in. wide. (Commentary: Sec. A.7.2.6. Tier 2: Sec. 13.6.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Ceiling supported off wall. There is no gap between ceiling channel edge and wall.

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-not required; PR-H. SEISMIC JOINTS: Acoustical tile or lay-in panel ceilings have seismic separation joints such that each continuous portion of the ceiling is no more than 2500 ft^2 and has a ratio of long-to-short dimension no more than 4-to-1. (Commentary: Sec. A.7.2.7. Tier 2: 13.6.4)	

Light Fixtures

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	LS-MH; PR-MH. INDEPENDENT SUPPORT: Light fixtures that weigh more per square foot than the ceiling they penetrate are supported independent of the grid ceiling suspension system by a minimum of two wires at diagonally opposite corners of each fixture. (Commentary: Sec. A.7.3.2. Tier 2: Sec. 13.6.4 and 13.7.9)	
☐	☐	☒	☐	LS-not required; PR-H PENDANT SUPPORTS: Light fixtures on pendant supports are attached at a spacing equal to or less than 6 ft. Unbraced suspended fixtures are free to allow a 360-degree range of motion at an angle not less than 45 degrees from horizontal without contacting adjacent components. Alternatively, if rigid supported and/or braced, they are free to move with the structure to which they are attached without damaging adjoining components. Additionally, the connection to the structure is capable of accommodating the movement without failure. (Commentary: Sec. A.7.3.3. Tier 2: Sec. 13.7.9)	
☐	☒	☐	☐	LS-not required; PR-H. LENS COVERS: Lens covers on light fixtures are attached with safety devices. (Commentary: Sec. A.7.3.4. Tier 2: Sec. 13.7.9)	There appears to be a lack of safety devices on lights.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Lens covers within building lack safety devices. The covers currently can swing open without a latch.

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Cladding and Glazing

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. CLADDING ANCHORS: Cladding components weighing more than 10 lb/ft^2 are mechanically anchored to the structure at a spacing equal to or less than the following: for Life Safety in Moderate Seismicity, 6 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 ft. (Commentary: Sec. A.7.4.1. Tier 2: Sec. 13.6.1)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH CLADDING ISOLATION: For steel or concrete moment-frame buildings, panel connections are detailed to accommodate a story drift ratio by the use of rods attached to framing with oversize holes or slotted holes of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02, and the rods have a length-to-diameter ratio of 4.0 or less. (Commentary: Sec. A.7.4.3. Tier 2: Sec. 13.6.1)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH MULTI-STORY PANELS: For multi-story panels attahed at more than one floor level panel connections are detailed to accommodate a story drift ratio by the use of rods attached to framing with oversize holes or slotted holes of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02, and the rods have a length-to-diameter ratio of 4.0 or less. (Commentary: Sec. A.7.4.4. Tier 2: Sec. 13.6.1)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH THREADED RODS: Threaded rods for panel connections detailed to accommodate drift by bending of the rod have a length-to-diameter ratio greater than 0.06 times the story height in inches for Life Safety in Moderate Seismicity and 0.12 times the story height in inches for Life Safety in High Seismicity and for Position Retention in any seismicity. (Commentary: Sec. A.7.4.9. Tier 2: Sec. 13.6.1)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U		
☐	☐	☒	☐	LS-MH; PR-MH. PANEL CONNECTIONS: Cladding panels are anchored out-of-plane with a minimum number of connections for each wall panel, as follows: for Life Safety in Moderate Seismicity, 2 connections; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 connections. (Commentary: Sec. A.7.4.5. Tier 2: Sec. 13.6.1.4)	

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C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-MH; PR-MH. BEARING CONNECTIONS: Where bearing connections are used, there is a minimum of two bearing connections for each cladding panel. (Commentary: Sec. A.7.4.6. Tier 2: Sec. 13.6.1.4)	
☐	☐	☒	☐	LS-MH; PR-MH. INSERTS: Where concrete cladding components use inserts, the inserts have positive anchorage or are anchored to reinforcing steel. (Commentary: Sec. A.7.4.7. Tier 2: Sec. 13.6.1.4)	
☐	☒	☐	☐	LS-MH; PR-MH. OVERHEAD GLAZING: Glazing panes of any size in curtain walls and individual interior or exterior panes over 16 ft^2 in area are laminated annealed or laminated heat-strengthened glass and are detailed to remain in the frame when cracked. (Commentary: Sec. A.7.4.8: Tier 2: Sec. 13.6.1.5)	Windows are assumed to lack proper restraint in frame if cracked or damaged.

Masonry Veneer

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-LMH; PR-LMH. TIES: Masonry veneer is connected to the backup with corrosion-resistant ties. There is a minimum of one tie for every 2-2/3 ft^2, and the ties have spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 36 in.; for Life Safety in High Seismicity and for Position Retention in any seismicity, 24 in. (Commentary: Sec. A.7.5.1. Tier 2: Sec. 13.6.1.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Exterior windows do not appear to meet requirements of heat laminated or have restraint to remain within frame if cracked.

C	NC	N/A	U		
☐	☐	☒	☐	LS-LMH; PR-LMH. SHELF ANGLES: Masonry veneer is supported by shelf angles or other elements at each floor above the ground floor. (Commentary: Sec. A.7.5.2. Tier 2: Sec. 13.6.1.2)	
☐	☐	☒	☐	LS-LMH; PR-LMH. WEAKENED PLANES: Masonry veneer is anchored to the backup adjacent to weakened planes, such as at the locations of flashing. (Commentary: Sec. A.7.5.3. Tier 2: Sec. 13.6.1.2)	
☐	☐	☒	☐	LS-LMH; PR-LMH. UNREINFORCED MASONRY BACKUP: There is no unreinforced masonry backup. (Commentary: Sec. A.7.7.2. Tier 2: Section 13.6.1.1 and 13.6.1.2)	
☐	☐	☒	☐	LS-MH; PR-MH STUD TRACKS: For veneer with cold-formed steel stud backup, stud tracks are fastened to the structure at a spacing equal to or less than 24 in. on center. (Commentary: Sec. A.7.6.1. Tier 2: Sec. 13.6.1.1 and 13.6.1.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-MH; PR-MH. ANCHORAGE: For veneer with concrete block or masonry backup, the backup is positively anchored to the structure at a horizontal spacing equal to or less than 4 ft along the floors and roof. (Commentary: Sec. A.7.7.1. Tier 2: Section 13.6.1.1 and 13.6.1.2)	
☐	☐	☒	☐	LS-not required; PR-MH. WEEP HOLES: In veneer anchored to stud walls, the veneer has functioning weep holes and base flashing. (Commentary: Sec. A.7.5.6. Tier 2: Section 13.6.1.2)	
☐	☐	☒	☐	LS-not required; PR-MH OPENINGS: For veneer with cold-formed -steel stud backup, steel studs frame window and door openings. (Commentary: Sec. A.7.6.2. Tier 2: Sec. 13.6.1.1 and 13.6.1.2)	

Parapets, Cornices, Ornamentation, and Appendages

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. URM PARAPETS OR CORNICES: Laterally unsupported unreinforced masonry parapets or cornices have height-to-thickness ratios no greater than the following: for Life Safety in Low or Moderate Seismicity, 2.5; for Life Safety in High Seismicity and for Position Retention in any seismicity, 1.5. (Commentary: Sec. A.7.8.1. Tier 2: Sec. 13.6.5)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-LMH; PR-LMH. CANOPIES: Canopies at building exits are anchored to the structure at a spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 10 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 6 ft. (Commentary: Sec. A.7.8.2. Tier 2: Sec. 13.6.6)	
☐	☐	☒	☐	LS-MH; PR-LMH. CONCRETE PARAPETS: Concrete parapets with height-to-thickness ratios greater than 2.5 have vertical reinforcement. (Commentary: Sec. A.7.8.3. Tier 2: Sec. 13.6.5)	
☒	☐	☐	☐	LS-MH; PR-LMH. APPENDAGES: Cornices, parapets, signs, and other ornamentation or appendages that extend above the highest point of anchorage to the structure or cantilever from components are reinforced and anchored to the structural system at a spacing equal to or less than 6 ft. This checklist item does not apply to parapets or cornices covered by other checklist items. (Commentary: Sec. A.7.8.4. Tier 2: Sec. 13.6.6)	Vent stact for the laboratory hoods on roof is restrained with (3) cables.

Masonry Chimneys

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-LMH; PR-LMH. URM CHIMNEYS: Unreinforced masonry chimneys extend above the roof surface no more than the following: for Life Safety in Low or Moderate Seismicity, 3 times the least dimension of the chimney; for Life Safety in High Seismicity and for Position Retention in any seismicity, 2 times the least dimension of the chimney. (Commentary: Sec. A.7.9.1. Tier 2: 13.6.7)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-LMH; PR-LMH. ANCHORAGE: Masonry chimneys are anchored at each floor level, at the topmost ceiling level, and at the roof. (Commentary: Sec. A.7.9.2. Tier 2: 13.6.7)	

Stairs

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. STAIR ENCLOSURES: Hollow-clay tile or unreinforced masonry walls around stair enclosures are restrained out-of-plane and have height-to-thickness ratios not greater than the following: for Life Safety in Low or Moderate Seismicity, 15-to-1; for Life Safety in High Seismicity and for Position Retention in any seismicity, 12-to-1. (Commentary: Sec. A.7.10.1. Tier 2: Sec. 13.6.2 and 13.6.8)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH STAIR DETAILS: The connection between the stairs and the structure does not rely on post-installed anchors in concrete or masonry, and the stair details are capable of accommodating the drift calculated using the Quick Check procedure of Section 4.4.3.1 for moment-frame structures or 0.5 in. for all other structures without including any lateral stiffness contribution from the stairs. (Commentary: Sec. A.7.10.2. Tier 2: Sec. 13.6.8)	

Contents and Furnishings

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. INDUSTRIAL STORAGE RACKS: Industrial storage racks or pallet racks more than 12 ft high meet the requirements of ANSI/MH 16.1 as modified by ASCE 7 Chapter 15. (Commentary: Sec. A.7.11.1. Tier 2: Sec. 13.8.1)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Statement	Comment
☐	☒	☐	☐	LS-H; PR-MH. TALL NARROW CONTENTS: Contents more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 are anchored to the structure or to each other. (Commentary: Sec. A.7.11.2. Tier 2: Sec. 13.8.2)	Storage racks lack restraint to structure in storage room. Refrigerators in laboratory are on rollers which can be locked in place. If set in locked position, there is potential for it to overturn.
☐	☒	☐	☐	LS-H; PR-H. FALL-PRONE CONTENTS: Equipment, stored items, or other contents weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level are braced or otherwise restrained. (Commentary: Sec. A.7.11.3. Tier 2: Sec. 13.8.2)	We could not determine/identify whether the laboratory hoods are laterally braced to structure. The equipment is suspended from the roof framing above the ceiling. Our assumption is there is no lateral bracing to support these equipment.
☐	☐	☒	☐	LS-not required; PR-MH. ACCESS FLOORS: Access floors more than 9 in. high are braced. (Commentary: Sec. A.7.11.4. Tier 2: Sec. 13.8.3)	
☐	☐	☒	☐	LS-not required; PR-MH. EQUIPMENT ON ACCESS FLOORS: Equipment and other contents supported by access floor systems are anchored or braced to the structure independent of the access floor. (Commentary: Sec. A.7.11.5. Tier 2: Sec. 13.7.7 and 13.8.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Storage racks in storage room don't appear to be restrained laterally.

Refrigerator doesn't appear to be restrained laterally. There are rollers, but if set in lock position, there is potential for overturning.

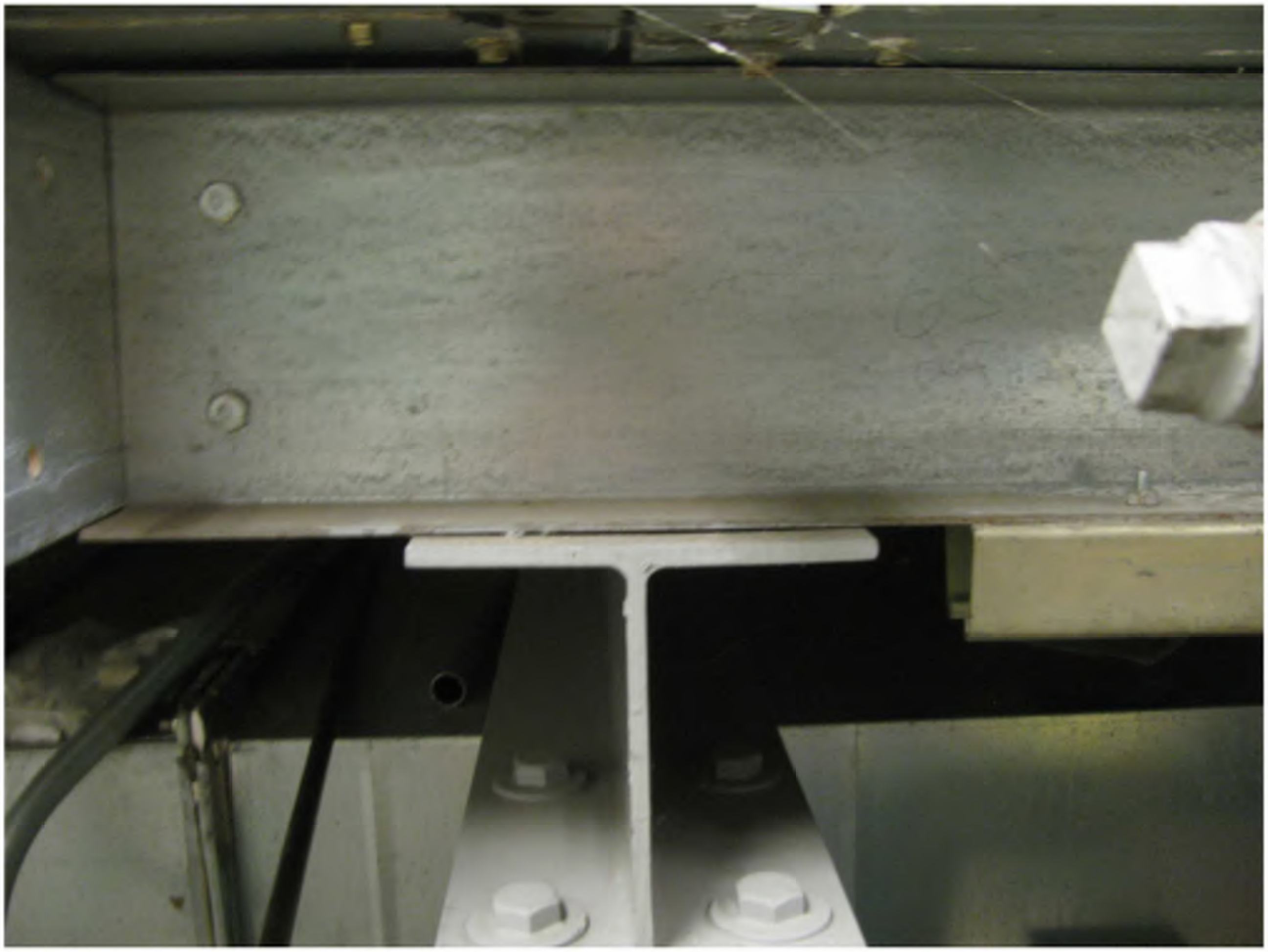
The laboratory hood equipment is assumed to be seismically unbraced.

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☒	N/A ☐	U ☐	LS-not required; PR-H. SUSPENDED CONTENTS: Items suspended without lateral bracing are free to swing from or move with the structure from which they are suspended without damaging themselves or adjoining components. (Commentary. A.7.11.6. Tier 2: Sec. 13.8.2)	There is a gap between the laboratory hoods and ceiling to allow for some movement, but gap might not be adequate.

Mechanical and Electrical Equipment

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☒	N/A ☐	U ☐	LS-H; PR-H. FALL-PRONE EQUIPMENT: Equipment weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level, and which is not in-line equipment, is braced. (Commentary: A.7.12.4. Tier 2: 13.7.1 and 13.7.7)	We could not determine/identify whether the laboratory hoods are laterally braced to structure. The equipment is suspended from the roof framing above the ceiling. Our assumption is there is no lateral bracing to support these equipment. In addition, the air handlers in mechanical room lack anchorage to the support structure below.
C ☒	NC ☐	N/A ☐	U ☐	LS-H; PR-H. IN-LINE EQUIPMENT: Equipment installed in-line with a duct or piping system, with an operating weight more than 75 lb, is supported and laterally braced independent of the duct or piping system. (Commentary: Sec. A.7.12.5. Tier 2: Sec. 13.7.1)	
C ☐	NC ☒	N/A ☐	U ☐	LS-H; PR-MH. TALL NARROW EQUIPMENT: Equipment more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 is anchored to the floor slab or adjacent structural walls. (Commentary: Sec. A.7.12.6. Tier 2: Sec. 13.7.1 and 13.7.7)	The air handlers in mechanical room lack anchorage to the support structure below.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown



The HVAC equipment in Mechanical Room doesn't appear to be anchored to supporting structure framing below.

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C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-MH. MECHANICAL DOORS: Mechanically operated doors are detailed to operate at a story drift ratio of 0.01. (Commentary: Sec. A.7.12.7. Tier 2: Sec. 13.6.9)	
☐	☒	☐	☐	LS-not required; PR-H. SUSPENDED EQUIPMENT: Equipment suspended without lateral bracing is free to swing from or move with the structure from which it is suspended without damaging itself or adjoining components. (Commentary: Sec. A.7.12.8. Tier 2: Sec. 13.7.1 and 13.7.7)	We could not determine/identify whether the laboratory hoods are laterally braced to structure. The equipment is suspended from the roof framing above the ceiling. Our assumption is there is no lateral bracing to support these equipment.
☐	☐	☒	☐	LS-not required; PR-H. VIBRATION ISOLATORS: Equipment mounted on vibration isolators is equipped with horizontal restraints or snubbers and with vertical restraints to resist overturning. (Commentary: Sec. A.7.12.9. Tier 2: Sec. 13.7.1)	
☐	☒	☐	☐	LS-not required; PR-H. HEAVY EQUIPMENT: Floor-supported or platform-supported equipment weighing more than 400 lb is anchored to the structure. (Commentary: Sec. A.7.12.10. Tier 2: 13.7.1 and 13.7.7)	The air handling unit within mechanical room lacks connection to the supporting frame structure it is located on.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	LS-not required; PR-H. ELECTRICAL EQUIPMENT: Electrical equipment is laterally braced to the structure. (Commentary: Sec. A.7.12.11. Tier 2: 13.7.7)	
☐	☐	☒	☐	LS-not required; PR-H. CONDUIT COUPLINGS: Conduit greater than 2.5 in. trade size that is attached to panels, cabinets, or other equipment and is subject to relative seismic displacement has flexible couplings or connections. (Commentary: Sec. A.7.12.12. Tier 2: 13.7.8)	

Piping

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. FLEXIBLE COUPLINGS: Fluid and gas piping has flexible couplings. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.3 and 13.7.5)	
☐	☐	☒	☐	LS-not required; PR-H. FLUID AND GAS PIPING: Fluid and gas piping is anchored and braced to the structure to limit spills or leaks. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-not required; PR-H. C-CLAMPS: One-sided C-clamps that support piping larger than 2.5 in. in diameter are restrained. (Commentary: Sec. A.7.13.5. Tier 2: Sec. 13.7.3 and 13.7.5)	
☐	☐	☒	☐	LS-not required; PR-H. PIPING CROSSING SEISMIC JOINTS: Piping that crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A7.13.6. Tier 2: Sec.13.7.3 and Sec. 13.7.5)	

Ducts

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-H. DUCT BRACING: Rectangular ductwork larger than 6 ft^2 in cross-sectional area and round ducts larger than 28 in. in diameter are braced. The maximum spacing of transverse bracing does not exceed 30 ft. The maximum spacing of longitudinal bracing does not exceed 60 ft. (Commentary: Sec. A.7.14.2. Tier 2: Sec. 13.7.6)	
C ☒	NC ☐	N/A ☐	U ☐	LS-not required; PR-H. DUCT SUPPORT: Ducts are not supported by piping or electrical conduit. (Commentary: Sec. A.7.14.3. Tier 2: Sec. 13.7.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-not required; PR-H. DUCTS CROSSING SEISMIC JOINTS: Ducts that cross seismic joints or isolation planes or are connected to independent structures have couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.14.5. Tier 2: Sec. 13.7.6)	

Elevators

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-H; PR-H. RETAINER GUARDS: Sheaves and drums have cable retainer guards. (Commentary: Sec. A.7.16.1. Tier 2: 13.8.6)	
C ☐	NC ☐	N/A ☒	U ☐	LS-H; PR-H. RETAINER PLATE: A retainer plate is present at the top and bottom of both car and counterweight. (Commentary: Sec. A.7.16.2. Tier 2: 13.8.6)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-H. ELEVATOR EQUIPMENT: Equipment, piping, and other components that are part of the elevator system are anchored. (Commentary: Sec. A.7.16.3. Tier 2: 13.8.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

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C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. SEISMIC SWITCH: Elevators capable of operating at speeds of 150 ft/min or faster are equipped with seismic switches that meet the requirements of ASME A17.1 or have trigger levels set to 20% of the acceleration of gravity at the base of the structure and 50% of the acceleration of gravity in other locations. (Commentary: Sec. A.7.16.4. Tier 2: 13.8.6)	
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. SHAFT WALLS: Elevator shaft walls are anchored and reinforced to prevent toppling into the shaft during strong shaking. (Commentary: Sec. A.7.16.5. Tier 2: 13.8.6)	
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. COUNTERWEIGHT RAILS: All counterweight rails and divider beams are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.6. Tier 2: 13.8.6)	
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. BRACKETS: The brackets that tie the car rails and the counterweight rail to the structure are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.7. Tier 2: 13.8.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. SPREADER BRACKET: Spreader brackets are not used to resist seismic forces. (Commentary: Sec. A.7.16.8. Tier 2: 13.8.6)	
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. GO-SLOW ELEVATORS: The building has a go-slow elevator system. (Commentary: Sec. A.7.16.9. Tier 2: 13.8.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

City of Wilsonville

Operations Building Tier 1 Structural Calculations

Latitude, Longitude: 45.294444, -122.77167

Date	6/28/2021, 11:18:38 AM
Design Code Reference Document	ASCE41-17
Custom Probability	
Site Class	C - Very Dense Soil and Soft Rock

Type	Description	Value
Hazard Level		BSE-2N
S_S	spectral response (0.2 s)	0.813
S_1	spectral response (1.0 s)	0.381
S_{XS}	site-modified spectral response (0.2 s)	0.976
S_{X1}	site-modified spectral response (1.0 s)	0.571
F_a	site amplification factor (0.2 s)	1.2
F_v	site amplification factor (1.0 s)	1.5
ssuh	max direction uniform hazard (0.2 s)	0.92
crs	coefficient of risk (0.2 s)	0.884
ssrt	risk-targeted hazard (0.2 s)	0.813
ssd	deterministic hazard (0.2 s)	1.5
s1uh	max direction uniform hazard (1.0 s)	0.441
cr1	coefficient of risk (1.0 s)	0.863
s1rt	risk-targeted hazard (1.0 s)	0.381
s1d	deterministic hazard (1.0 s)	0.6

Type	Description	Value
Hazard Level		BSE-1N
S_{XS}	site-modified spectral response (0.2 s)	0.651
S_{X1}	site-modified spectral response (1.0 s)	0.381

Type	Description	Value
Hazard Level		BSE-2E
S_S	spectral response (0.2 s)	0.589
S_1	spectral response (1.0 s)	0.27
S_{XS}	site-modified spectral response (0.2 s)	0.744
S_{X1}	site-modified spectral response (1.0 s)	0.405
f_a	site amplification factor (0.2 s)	1.265
f_v	site amplification factor (1.0 s)	1.5

Type	Description	Value
Hazard Level		BSE-1E
S_S	spectral response (0.2 s)	0.223
S_1	spectral response (1.0 s)	0.082
S_{XS}	site-modified spectral response (0.2 s)	0.291
S_{X1}	site-modified spectral response (1.0 s)	0.123
F_a	site amplification factor (0.2 s)	1.3
F_v	site amplification factor (1.0 s)	1.5

Type	Description	Value
Hazard Level		TL Data
T-Sub-L	Long-period transition period in seconds	16

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BY: BS	DATE Jul-21	CLIENT City of Wilsonville	SHEET
CHKD BY	DESCRIPTION Operations Building		JOB NO. 11962A.00
DESIGN TASK	Operations Building Seismic Weight		

Roof Loads

Roof EL 125.63

Description	Load		
1-1/2"x20ga metal deck	2.5	psf	
Rigid insulation w/ metal sheet roofing	4.5		
Steel beam	1.8		
Steel truss	2.5		
Suspended accoustical ceiling	3.5		
Miscellaneous	5.0		
Dead Load for Gravity Design	**19.8**	psf	
Roof Live Load	**20.0**	psf	(Assumed)
Snow Load	**25.0**	psf	

Notes

1. The roof deck is set at a slope of 5/12, so the deck and truss members will have the unit weight increased by a factor of 1.08 to account for a projected unit horizontal weight.

Wall Loads

Wall Loads

Description	Load	
8" CMU wall (partial grouted @ 24")	47.0	psf
5/8" GWB w/ insulation	3.7	
5/8" GWB w/ insulation double sided	7.4	
3-5/8"x20ga studs @ 16"	4.0	
Plastic veneer finish	7.5	
8" CMU Wall w/ GWB 1-side for Seismic Load	**58.2**	psf
8" CMU Wall w/ GWB 2-sides for Seismic Load	**54.4**	psf
8" CMU Wall w/ metal studs for Seismic Load	**62.2**	psf

Seismic Weight

Roof Weight

Roof Area	4888.0	ft^2
Roof Seismic Weight	**96.8**	**kip**

Wall Weight

Wall Height to Roof	10.17	ft
8" CMU Wall w/ GWB 1-side Length	254.67	ft
8" CMU Wall w/ GWB 2-sides Length	25.33	ft
8" CMU Wall w/ metal studs Length	37.00	ft
Roof Wall Seismic Weight	**94.1**	**kip**

Total Seismic Weight	**190.9**	**kip**

Notes

1. Wall seismic weight assumes half of the wall height associated with each level.
2. Per Section 4.5.2.1, the effective seismic weight includes the total dead load, 25% of the live load when area is used as storage, and 20% of the roof snow live load if greater than 30 psf (otherwise assume zero).

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BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Operations Building	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 1 Screening (BSE-2E Level)		

SEISMIC BASE SHEAR

4.4.2.1 Pseudo Seismic Force. The pseudo seismic force, in a given horizontal direction of a building, shall be calculated in accordance with Eq. (4-1).

$$V = CS_aW \qquad (4\text{-}1)$$

where

V = Pseudo seismic force;
C = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response; C shall be taken from Table 4-7;
S_a = Response spectral acceleration at the fundamental period of the building in the direction under consideration. The value of S_a shall be calculated in accordance with the procedures in Section 4.4.2.3; and
W = Effective seismic weight of the building, including the total dead load and applicable portions of other gravity loads listed below:

Table 4-7. Modification Factor, *C*

	Number of Stories			
Building Type[a]	**1**	**2**	**3**	**≥4**
Wood and cold-formed steel shear wall (W1, W1a, W2, CFS1) Moment frame (S1, S3, C1, PC2a)	1.3	1.1	1.0	1.0
Shear wall (S4, S5, C2, C3, PC1a, PC2, RM2, URMa) Braced frame (S2) Cold-formed steel strap-brace wall (CFS2)	1.4	1.2	1.1	1.0
Unreinforced masonry (URM) Flexible diaphragms (S1a, S2a, S5a, C2a, C3a, PC1, RM1)	1.0	1.0	1.0	1.0

[a] Defined in Table 3-1.

Process Gallery

Modification Factor, C =	1.0	
S_{X1} =	0.405	(BSE-2E seismic hazard)
T =	0.114	s
S_{XS} =	0.744	(BSE-2E seismic hazard)
Spectral Acceleration, S_a =	0.744	
Seismic Weight, W =	190.9	kip
Seismic Force, V =	142.0	kip

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BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Operations Building	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 1 Screening (BSE-2E Level)		

WALL SHEAR STRESS CHECK

4.4.3.3 Shear Stress in Shear Walls. The average shear stress in shear walls, v_j^{avg}, shall be calculated in accordance with Eq. (4-8).

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \quad (4\text{-}8)$$

where

V_j = Story shear at level j computed in accordance with Section 4.4.2.2;

A_w = Summation of the horizontal cross-sectional area of all shear walls in the direction of loading. Openings shall be taken into consideration where computing A_w. For masonry walls, the net area shall be used. For wood-framed walls, the length shall be used rather than the area; and

M_s = System modification factor; M_s shall be taken from Table 4-8.

Table 4-8. M_s Factors for Shear Walls

	Level of Performance		
Wall Type	**CP[a]**	**LS[a]**	**IO[a]**
Reinforced concrete, precast concrete, wood, reinforced masonry, and cold-formed steel	4.5	3.0	1.5
Unreinforced masonry	1.75	1.25	1.0

[a] CP = Collapse Prevention, LS = Life Safety, IO = Immediate Occupancy.

CMU wall thickness, t = 7.625 in
Roof Story Base Shear, V_{roof} = 142.0 kips
System Modification Factor, M_s = 3.75 (Interpolated between LS & CP)

Roof Level

Shear Wall in N-S Direction

West Elevation Wall Line

Total length of exterior 8" CMU walls = 84.00 ft
Grout spacing = 32 in
total net area of shear walls = 4611.6 in^2
average shear stress, $v_{avg,NS}$ = 4.1 psi < *70.0* ***Shear Stress OK***
DCR = 0.06

East Elevation Wall Line

Total length of exterior 8" CMU walls = 60.67 ft
Grout spacing = 32 in
total net area of shear walls = 3330.8 in^2
average shear stress, $v_{avg,NS}$ = 5.7 psi < *70.0* ***Shear Stress OK***
DCR = 0.08

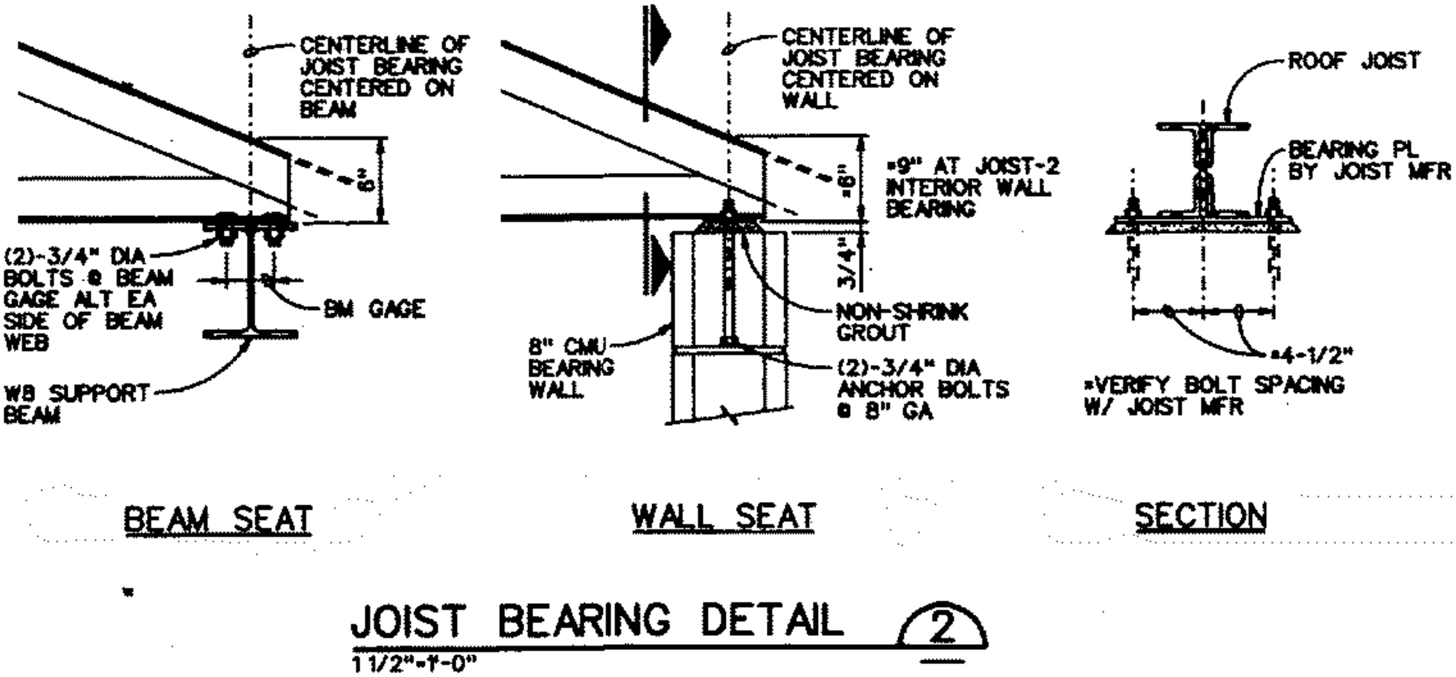
Shear Wall in E-W Direction

North Elevation Wall Line

Total length of exterior 8" CMU walls = 21.33 ft
Grout spacing = 32 in
total net area of shear walls = 1171.0 in^2
average shear stress, $v_{avg,NS}$ = 32.3 psi < *70.0* ***Shear Stress OK***
DCR = 0.46

South Elevation Wall Line

Total length of exterior 8" CMU walls = 22.00 ft
Grout spacing = 32 in
total net area of shear walls = 1207.8 in^2
average shear stress, $v_{avg,NS}$ = 31.4 psi < *70.0* ***Shear Stress OK***
DCR = 0.45



ROOF JOIST BEARING CONNECTION TO CMU WALL

Engineers... Working Wonders With Water ™

BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Operations Building	**JOB NO.**	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 1 Screening (BSE-2E Level)			

WALL ANCHORAGE FORCE

Operations Building: Roof Joist Bearing Anchorage into 8" CMU Wall along East and West Wall Elevations

4.4.3.7 Flexible Diaphragm Connection Forces. The horizontal seismic forces associated with the connection of a flexible diaphragm to either concrete or masonry walls, T_c, shall be calculated in accordance with Eq. (4-12).

$$T_c = \psi S_{XS} w_p A_p \qquad (4\text{-}12)$$

where

w_p = Unit weight of the wall;
A_p = Area of wall tributary to the connection;
ψ = 1.0 for Collapse Prevention Performance Level, 1.3 for Life Safety Performance Level, and 1.8 for Immediate Occupancy Performance Level; and
S_{XS} = Value specified in Section 4.4.2.3.

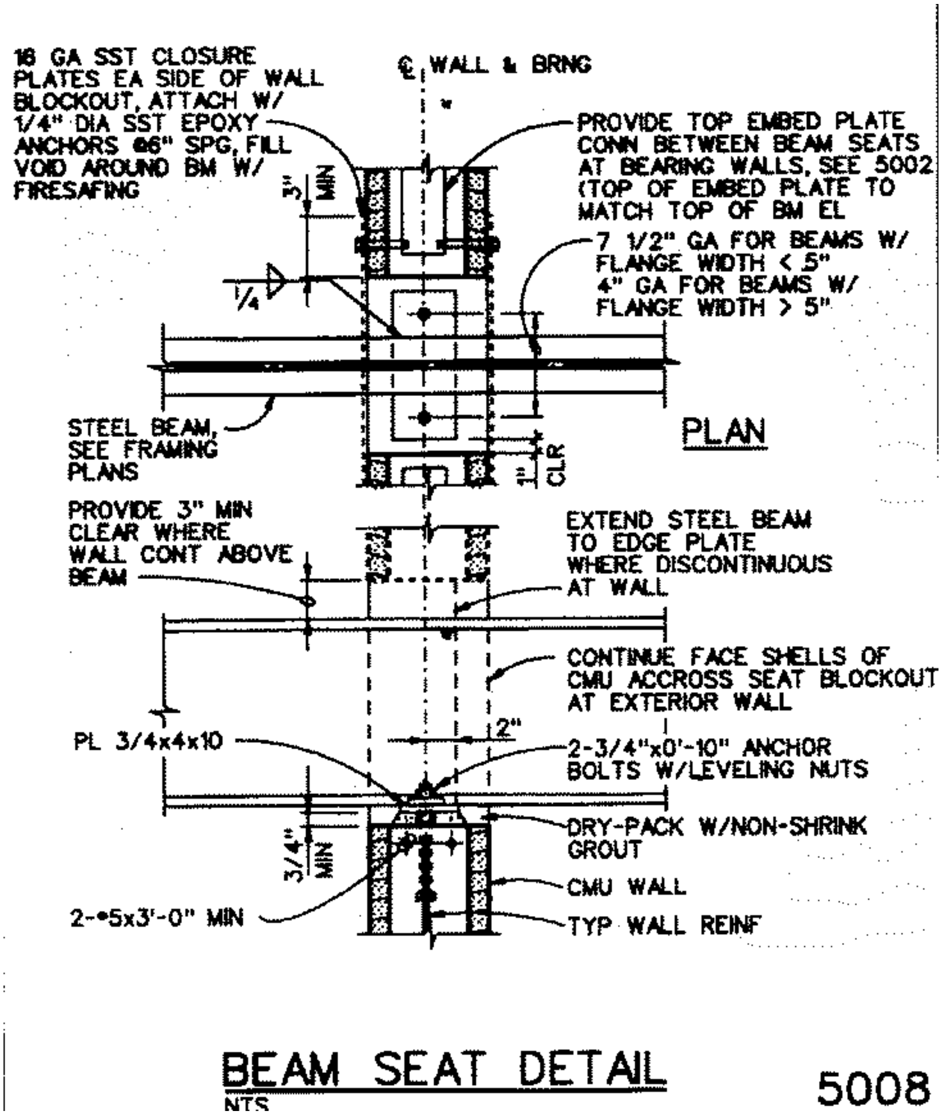
wall height to diaphragm, h_w =	10.17	ft	
unit weight of wall, w_p =	58.20	psf	(partial grout for wall)
Ψ =	1.15		(Interpolated between LS & CP)
S_{XS} =	0.744	g	
wall out-of-plane load =	253.2	lbs/ft	
roof joist spacing =	6.33	ft	
wall anchorage force, T_c =	1602.8	lbs	

Masonry & Steel Strength

anchor bolt size =	0.750	in
anchor bolt embed, l_b =	7.00	in
anchor bolt location from face, l_{be} =	3.81	in
anchor bolt yield stress, f_y =	36.00	ksi
masonry compressive strength, f_m =	1500	psi
projected area of anchor bolt in tension, A_{pt} =	101.1	in^2
projected area of each anchor bolt in shear, A_{pvbolt} =	22.80	in^2
cross section area of anchor bolt, A_b =	0.44	in^2
estimated overlap of projected area, $A_{ptoverlap}$ =	43.20	in^2
net projected area of anchor bolt in tension, A_{ptnet} =	180.70	in^2
estimated overlap of projected area, $A_{pvoverlap}$ =	0.00	in^2
net projected area of anchor bolt in shear, A_{pvnet} =	45.60	in^2

$\phi B_{vnb} = 4{*}A_{pvnet}{*}(f'_m)^{0.5}$ =	7064.9	lbs	group masonry breakout shear strength
$\phi B_{vnc} = 1050{*}(f'_m{*}A_b)^{0.25}$ =	10654.8	lbs	group masonry crushing shear strength
$\phi B_{vnpry} = 8{*}A_{ptnet}{*}(f'_m)^{0.5}$ =	55987.6	lbs	group anchor pryout shear strength
$\phi B_{vns} = 0.60{*}A_b{*}f_y$ =	19085.2	lbs	group steel yielding strength

Masonry breakout strength DCR =	*0.23*	***OK***
Masonry crushing strength DCR =	*0.15*	***OK***
Anchor pryout DCR =	*0.03*	***OK***
Steel yielding DCR =	*0.08*	***OK***



BEAM BEARING CONNECTION TO CMU WALL

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Engineers... Working Wonders With Water ®

BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Operations Building	**JOB NO.**	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 1 Screening (BSE-2E Level)			

WALL ANCHORAGE FORCE

Operations Building: Beam Anchorage into 8" CMU Wall along East and West Wall Elevations

4.4.3.7 Flexible Diaphragm Connection Forces. The horizontal seismic forces associated with the connection of a flexible diaphragm to either concrete or masonry walls, T_c, shall be calculated in accordance with Eq. (4-12).

$$T_c = \psi S_{XS} w_p A_p \qquad (4\text{-}12)$$

where

w_p = Unit weight of the wall;
A_p = Area of wall tributary to the connection;
ψ = 1.0 for Collapse Prevention Performance Level, 1.3 for Life Safety Performance Level, and 1.8 for Immediate Occupancy Performance Level; and
S_{XS} = Value specified in Section 4.4.2.3.

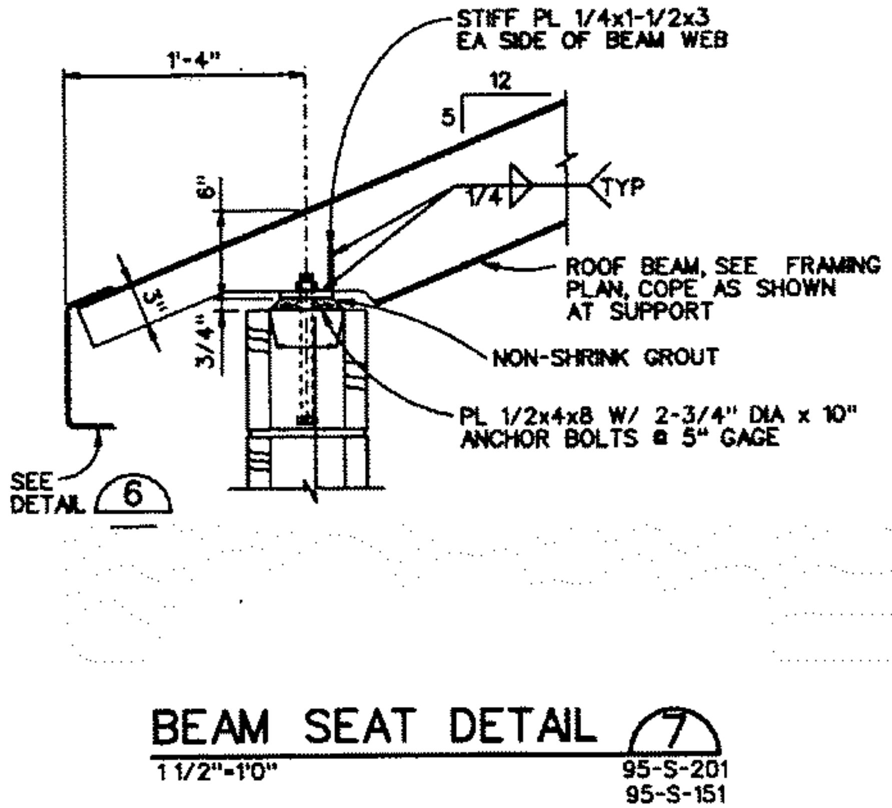
wall height to diaphragm, h_w =	18.36	ft	
unit weight of wall, w_p =	58.20	psf	(partial grout for exterior walls [CMU + venee
Ψ =	1.15		(Interpolated between LS & CP)
S_{XS} =	0.744	g	
wall out-of-plane load =	457.1	lbs/ft	
beam spacing =	8.33	ft	
wall anchorage force, T_c =	3807.9	lbs	

Masonry & Steel Strength

anchor bolt size =	0.750	in
anchor bolt embed, l_b =	7.00	in
anchor bolt location from face, l_{be} =	3.81	in
anchor bolt yield stress, f_y =	36.00	ksi
masonry compressive strength, f_m =	1500	psi
projected area of single anchor bolt in tension, A_{pt} =	101.1	in^2
projected area of single anchor bolt in shear, A_{pvbolt} =	22.80	in^2
cross section area of single anchor bolt, A_b =	0.44	in^2
estimated overlap of projected area, $A_{ptoverlap}$ =	43.20	in^2
net projected area of anchor bolt in tension, A_{ptnet} =	180.70	in^2
estimated overlap of projected area, $A_{pvoverlap}$ =	1.25	in^2
net projected area of anchor bolt in shear, A_{pvnet} =	44.98	in^2

$\phi B_{vnb} = 4*A_{pvnet}*(f'_m)^{0.5}$ =	6968.1	lbs	group masonry breakout shear strength
$\phi B_{vnc} = 1050*(f'_m*A_b)^{0.25}$ =	10654.8	lbs	group masonry crushing shear strength
$\phi B_{vnpry} = 8*A_{ptnet}*(f'_m)^{0.5}$ =	55987.6	lbs	group anchor pryout shear strength
$\phi B_{vns} = 0.60*A_b*f_y$ =	19085.2	lbs	group steel yielding strength

Masonry breakout strength DCR =	*0.55*	***OK***
Masonry crushing strength DCR =	*0.36*	***OK***
Anchor pryout DCR =	*0.07*	***OK***
Steel yielding DCR =	*0.20*	***OK***



SLOPED BEAM BEARING CONNECTION TO CMU WALLS

Carollo Engineers... Working Wonders With Water ™

BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Operations Building	**JOB NO.**	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 1 Screening (BSE-2E Level)		

WALL ANCHORAGE FORCE

Operations Building: Beam Anchorage into 8" CMU Wall along North and South Wall Elevations

4.4.3.7 Flexible Diaphragm Connection Forces. The horizontal seismic forces associated with the connection of a flexible diaphragm to either concrete or masonry walls, T_c, shall be calculated in accordance with Eq. (4-12).

$$T_c = \psi S_{XS} w_p A_p \qquad (4\text{-}12)$$

where

w_p = Unit weight of the wall;
A_p = Area of wall tributary to the connection;
ψ = 1.0 for Collapse Prevention Performance Level, 1.3 for Life Safety Performance Level, and 1.8 for Immediate Occupancy Performance Level; and
S_{XS} = Value specified in Section 4.4.2.3.

wall height to diaphragm, h_w =	10.17	ft	
unit weight of wall, w_p =	58.20	psf	(partial grout for exterior walls [CMU + venee
Ψ =	1.15		(Interpolated between LS & CP)
S_{XS} =	0.744	g	
wall out-of-plane load =	253.2	lbs/ft	
beam spacing =	6.67	ft	
wall anchorage force, T_c =	1688.9	lbs	

Masonry & Steel Strength

anchor bolt size =	0.750	in
anchor bolt embed, l_b =	7.00	in
anchor bolt location from face, l_{be} =	3.81	in
anchor bolt yield stress, f_y =	36.00	ksi
masonry compressive strength, f_m =	1500	psi
projected area of single anchor bolt in tension, A_{pt} =	101.1	in^2
projected area of single anchor bolt in shear, A_{pvbolt} =	22.80	in^2
cross section area of single anchor bolt, A_b =	0.44	in^2
estimated overlap of projected area, $A_{ptoverlap}$ =	63.60	in^2
net projected area of anchor bolt in tension, A_{ptnet} =	170.50	in^2
estimated overlap of projected area, $A_{pvoverlap}$ =	2.20	in^2
net projected area of anchor bolt in shear, A_{pvnet} =	44.50	in^2

$\phi B_{vnb} = 4 * A_{pvnet} * (f'_m)^{0.5}$ =	6894.5	lbs	group masonry breakout shear strength
$\phi B_{vnc} = 1050 * (f'_m * A_b)^{0.25}$ =	10654.8	lbs	group masonry crushing shear strength
$\phi B_{vnpry} = 8 * A_{ptnet} * (f'_m)^{0.5}$ =	52827.2	lbs	group anchor pryout shear strength
$\phi B_{vns} = 0.60 * A_b * f_y$ =	19085.2	lbs	group steel yielding strength

Masonry breakout strength DCR =	*0.24*	***OK***
Masonry crushing strength DCR =	*0.16*	***OK***
Anchor pryout DCR =	*0.03*	***OK***
Steel yielding DCR =	*0.09*	***OK***

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Operations Building
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	07/01/21
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Op

Project Number 11962A.00

Table 17-3. Immediate Occupancy Basic Configuration Checklist

Very Low Seismicity

CSZ Seismic Check at Damage Control

Structural Components

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☒	N/A ☐	U ☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	There are no diaphragm ties in the N-S direction where perimeter CMU shear walls stop. The diaphragm at these locations will lack the ability to transfer diaphragm forces into the shear walls.
C ☒	NC ☐	N/A ☐	U ☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps taht are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	Roof joist bearing anchorage DCR = 0.18 (OK) E-W beam bearing anchorage DCR = 0.44 (OK) N-S beam bearing anchorage DCR = 0.20 (OK)

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Op
Project Number 11962A.00

Very Low Seismicity

Building System

General

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☒	N/A ☐	U ☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	There are no diaphragm ties in the N-S direction where perimeter CMU shear walls stop. The diaphragm at these locations will lack the ability to transfer diaphragm forces into the shear walls.
C ☐	NC ☐	N/A ☒	U ☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement need not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	
C ☐	NC ☐	N/A ☒	U ☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Op
Project Number 11962A.00

Building Configuration

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction shall not be less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	Building is a one-story structure.
C ☐	NC ☐	N/A ☒	U ☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story shall not be less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	Building is a one-story structure.
C ☒	NC ☐	N/A ☐	U ☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
C ☐	NC ☐	N/A ☒	U ☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	Building is a one-story structure.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U		
☐	☐	☒	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	Building is a one-story structure.
C	NC	N/A	U		
☐	☐	☒	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	Torsion check applies for structures with rigid diaphragms, not for flexible diaphragms.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Op
Project Number 11962A.00

Low Seismicity

Geologic Site Hazards

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)	Liquefaction has been determined to not be an issue per NGI technical memorandum.
C ☒	NC ☐	N/A ☐	U ☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)	Slope failure has been determined to not be an issue per NGI technical memorandum.
C ☒	NC ☐	N/A ☐	U ☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)	Surface fault rupture has been determined to not be an issue per NGI technical memorandum.

Moderate and High Seismicity

Foundation Configuration

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)	Height = 10.25ft Base = 58ft Sa = 0.446 B/H = 58ft / 10.25ft = 5.66 0.6*Sa = 0.6 * 0.446 = 0.27 5.66 > 0.27 (OK)

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name ____________________

Project Number ____________________

C	NC	N/A	U		
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Operations Building
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	07/01/21
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Table 17-35. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2

Very Low Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	DESCRIPTION	COMMENTS
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than 70 lb/in.^{2}. (Commentary: Sec. A.3.2.4.1. Tier 2: Sec. 5.5.3.1.1)	West wall line DCR = 0.06 (OK) East wall line DCR = 0.08 (OK) North wall line DCR = 0.46 (OK) South wall line DCR = 0.45 (OK)
☐	☒	☐	☐	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in., and all vertical bars extend to the top of the walls. (Commentary: Sec. A.3.2.4.2. Tier 2: Sec. 5.5.3.1.3)	Horiz steel = #5@48" Vert steel = #6@32" Horiz ratio = 0.31/(7.625*48) = 0.0008 > 0.0007 (OK) Vert ratio = 0.44/(7.625*32) = 0.0018 > 0.0007 (OK) Combined = 0.0018+0.0008 = 0.0026 > 0.002 (OK) Horizontal reinforcing is spaced at 48in, but this is not less than 48in spacing, so NC.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Connections

C	NC	N/A	U	DESCRIPTION	COMMENTS
☐	☐	☒	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	
☒	☐	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)	Anchorage to CMU connection DCR = 0.48 (OK) Puddle weld connection DCR = 0.84 (OK)
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
☒	☐	☐	☐	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U		
☒	☐	☐	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps taht are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	Roof joist bearing anchorage DCR = 0.18 (OK) E-W beam bearing anchorage DCR = 0.44 (OK) N-S beam bearing anchorage DCR = 0.20 (OK)

Stiff Diaphragms

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	
C	NC	N/A	U		
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2: Sec. 5.7.2)	

Foundation System

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	DEEP FOUNDATIONS: Piles and piers are capable of transferring the seismic forces between the structure and the soil. (Commentary: Sec. A.6.2.3.)	No deep foundations present.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	DESCRIPTION	COMMENTS
☒	☐	☐	☐	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story high. (Commentary: Sec. A.6.2.4)	

Low, Moderate, and High Seismicity

Seismic-Force-Resisting System

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	REINFORCING AT WALL OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides. (Commentary: Sec. A.3.2.4.3. Tier 2: Sec. 5.5.3.1.5)	
☒	☐	☐	☐	PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than 30. (Commentary: Sec. A.3.2.4.4. Tier 2: Sec. 5.5.3.1.2)	Height = 10.17ft Thickness = 7.625in H/t = 10.17*12/7.625 = 16.0 < 30 (OK)

Diaphragms (Flexible or Stiff)

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 4 ft long. (Commentary: A.4.1.6. Tier 2: Sec. 5.6.1.3)	
☐	☒	☐	☐	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities. (Commentary: Sec. A.4.1.7. Tier 2: Sec. 5.6.1.4)	There are no diaphragm ties in the N-S direction where perimeter CMU shear walls stop. The diaphragm at these locations will lack the ability to transfer forces into the shear walls.
☐	☐	☒	☐	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. (Commentary: Sec. A.4.1.8. Tier 2: Sec. 5.6.1.5)	

Flexible Diaphragms

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Item	Comments
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 12 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft and aspect ratios less than or equal to 3-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☒	☐	☐	NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft and have aspect ratios less than 4-to-1. (Commentary: Sec. A.4.3.1. Tier 2: Sec. 5.6.3)	Span 1 = 40ft x 58ft Span 2 = 50ft x 36ft Span 1 ratio = 58/40 = 1.45 < 4 (OK) Span 2 ratio = 50/36 = 1.39 < 4 (OK) The aspect ratio is less than the 4-to-1 requirement, but the diaphragm spans between shear walls is greater than 40ft.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name	City of Wilsonville - Op
Project Number	11962A.00

C	NC	N/A	U		
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

Connections

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

City of Wilsonville

Operations Building Tier 1 Structural Calculations

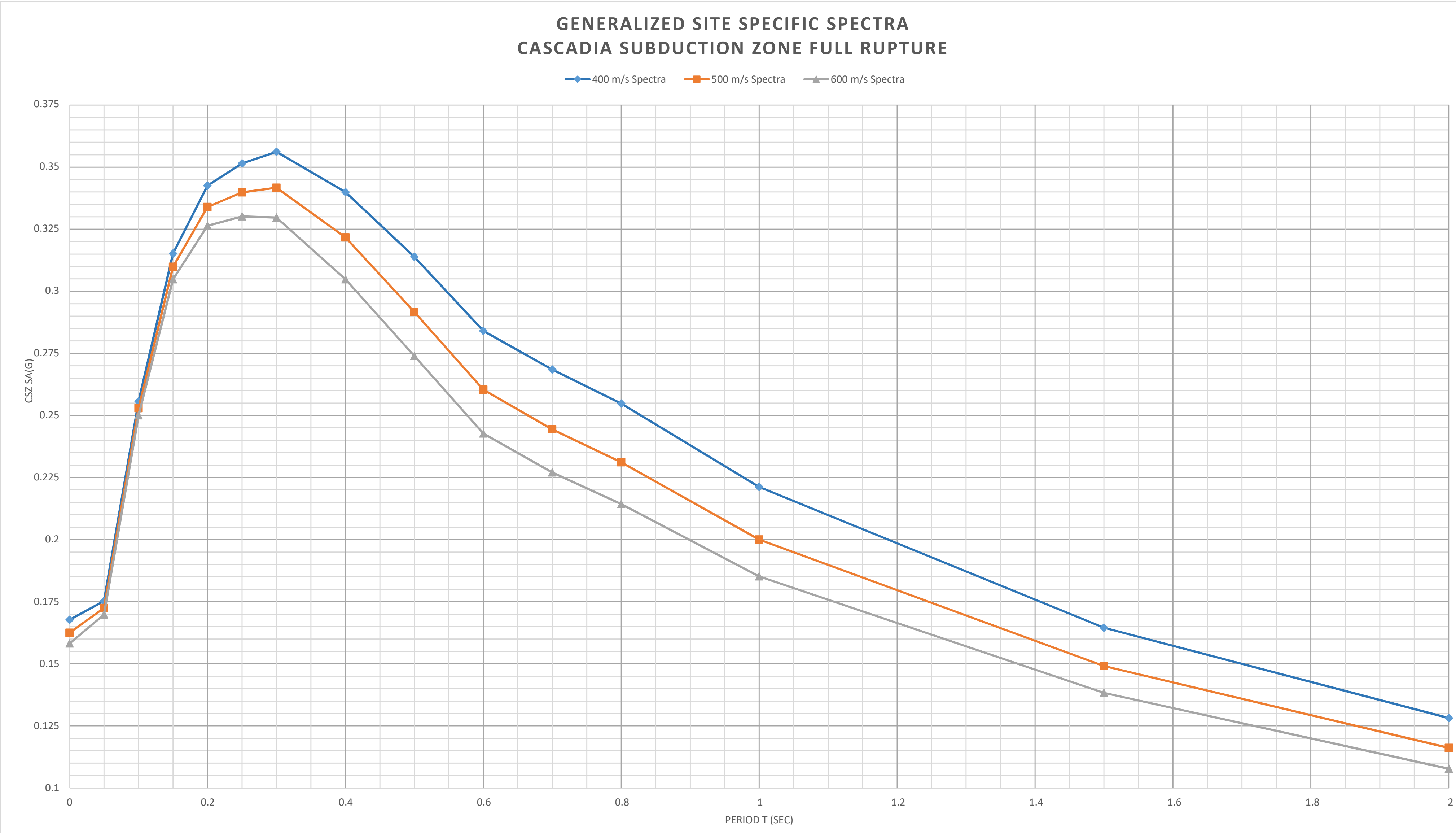

GENERALIZED SITE SPECIFIC SPECTRA
CASCADIA SUBDUCTION ZONE FULL RUPTURE
400 m/s Spectra
500 m/s Spectra
600 m/s Spectra
CSZ SA(G)
0.375
0.35
0.325
0.3
0.275
0.25
0.225
0.2
0.175
0.15
0.125
0.1
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
PERIOD T (SEC)

Figure No. 3

Ss @ T=0.20 sec

Table 2: CSZ Generalized Response Spectra Ordinates					
Latitude 45.295155 degrees Longitude -122.771810 degrees					
Vs30 = 400 m/s		Vs30 = 500 m/s		Vs30 = 600 m/s	
Period T(sec)	CSZ Sa(g)	Period T (sec)	CSZ Sa (g)	Period T (sec)	CSZ Sa (g)
0	0.168	0	0.163	0	0.158
0.05	0.175	0.05	0.172	0.05	0.170
0.1	0.256	0.1	0.253	0.1	0.250
0.15	0.315	0.15	0.310	0.15	0.305
0.2	0.343	0.2	0.334	0.2	0.326
0.25	0.352	0.25	0.340	0.25	0.330
0.3	0.356	0.3	0.342	0.3	0.330
0.4	0.340	0.4	0.322	0.4	0.305
0.5	0.314	0.5	0.292	0.5	0.274
0.6	0.284	0.6	0.260	0.6	0.243
0.7	0.269	0.7	0.244	0.7	0.227
0.8	0.255	0.8	0.231	0.8	0.214
1	0.221	1	0.200	1	0.185
1.5	0.165	1.5	0.149	1.5	0.138
2	0.128	2	0.116	2	0.108
2.5	0.104	2.5	0.094	2.5	0.087
3	0.085	3	0.077	3	0.071

Engineers...Working Wonders With Water™

BY: BS	DATE Jul-21	CLIENT City of Wilsonville	SHEET
CHKD BY	DESCRIPTION Operations Building		JOB NO. 11962A.00
DESIGN TASK	Operations Building Seismic Weight		

Roof Loads

Roof EL 125.63

Description	Load		
1-1/2"x20ga metal deck	2.5	psf	
Rigid insulation w/ metal sheet roofing	4.5		
Steel beam	1.8		
Steel truss	2.5		
Suspended accoustical ceiling	3.5		
Miscellaneous	5.0		
Dead Load for Gravity Design	**19.8**	psf	
Roof Live Load	**20.0**	psf	(Assumed)
Snow Load	**25.0**	psf	

Notes

1. The roof deck is set at a slope of 5/12, so the deck and truss members will have the unit weight increased by a factor of 1.08 to account for a projected unit horizontal weight.

Wall Loads

Wall Loads

Description	Load	
8" CMU wall (partial grouted @ 24")	47.0	psf
5/8" GWB w/ insulation	3.7	
5/8" GWB w/ insulation double sided	7.4	
3-5/8"x20ga studs @ 16"	4.0	
Plastic veneer finish	7.5	
8" CMU Wall w/ GWB 1-side for Seismic Load	**58.2**	psf
8" CMU Wall w/ GWB 2-sides for Seismic Load	**54.4**	psf
8" CMU Wall w/ metal studs for Seismic Load	**62.2**	psf

Seismic Weight

Roof Weight

Roof Area	4888.0	ft^2
Roof Seismic Weight	**96.8**	**kip**

Wall Weight

Wall Height to Roof	10.17	ft
8" CMU Wall w/ GWB 1-side Length	254.67	ft
8" CMU Wall w/ GWB 2-sides Length	25.33	ft
8" CMU Wall w/ metal studs Length	37.00	ft
Roof Wall Seismic Weight	**94.1**	**kip**

Total Seismic Weight **190.9** **kip**

Notes

1. Wall seismic weight assumes half of the wall height associated with each level.
2. Per Section 4.5.2.1, the effective seismic weight includes the total dead load, 25% of the live load when area is used as storage, and 20% of the roof snow live load if greater than 30 psf (otherwise assume zero).

Engineers... Working Wonders With Water ™

BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Operations Building	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)		

SEISMIC BASE SHEAR

4.4.2.1 Pseudo Seismic Force. The pseudo seismic force, in a given horizontal direction of a building, shall be calculated in accordance with Eq. (4-1).

$$V = CS_aW \quad (4\text{-}1)$$

where

V = Pseudo seismic force;
C = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response; C shall be taken from Table 4-7;
S_a = Response spectral acceleration at the fundamental period of the building in the direction under consideration. The value of S_a shall be calculated in accordance with the procedures in Section 4.4.2.3; and
W = Effective seismic weight of the building, including the total dead load and applicable portions of other gravity loads listed below:

Table 4-7. Modification Factor, C

Building Type[a]	Number of Stories 1	2	3	≥4
Wood and cold-formed steel shear wall (W1, W1a, W2, CFS1) Moment frame (S1, S3, C1, PC2a)	1.3	1.1	1.0	1.0
Shear wall (S4, S5, C2, C3, PC1a, PC2, RM2, URMa) Braced frame (S2) Cold-formed steel strap-brace wall (CFS2)	1.4	1.2	1.1	1.0
Unreinforced masonry (URM) Flexible diaphragms (S1a, S2a, S5a, C2a, C3a, PC1, RM1)	1.0	1.0	1.0	1.0

[a] Defined in Table 3-1.

Process Gallery

Modification Factor, C =	1.0	
S_s =	0.343	(CSZ spectral response)
S_1 =	0.221	(CSZ spectral response)
F_a =	1.3	(Site amplication factor per ASCE 7-16)
F_v =	1.5	(Site amplication factor per ASCE 7-16)
$S_{X1} = S_1 {*} F_v$ =	0.332	(CSZ seismic hazard)
T =	0.114	s
$S_{Xs} = S_s {*} F_a$ =	0.446	(CSZ seismic hazard)
Spectral Acceleration, S_a =	0.446	
Seismic Weight, W =	190.9	kip
Seismic Force, V =	85.1	kip

Carollo Engineers. Working Wonders With Water ™

BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Operations Building	**JOB NO.**	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)			

WALL SHEAR STRESS CHECK

4.4.3.3 Shear Stress in Shear Walls. The average shear stress in shear walls, v_j^{avg}, shall be calculated in accordance with Eq. (4-8).

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \quad (4\text{-}8)$$

where

V_j = Story shear at level j computed in accordance with Section 4.4.2.2;
A_w = Summation of the horizontal cross-sectional area of all shear walls in the direction of loading. Openings shall be taken into consideration where computing A_w. For masonry walls, the net area shall be used. For wood-framed walls, the length shall be used rather than the area; and
M_s = System modification factor; M_s shall be taken from Table 4-8.

Table 4-8. M_s Factors for Shear Walls

	Level of Performance		
Wall Type	**CP[a]**	**LS[a]**	**IO[a]**
Reinforced concrete, precast concrete, wood, reinforced masonry, and cold-formed steel	4.5	3.0	1.5
Unreinforced masonry	1.75	1.25	1.0

[a] CP = Collapse Prevention, LS = Life Safety, IO = Immediate Occupancy.

CMU wall thickness, t = 7.625 in
Roof Story Base Shear, V_{roof} = 85.1 kips
System Modification Factor, M_s = 2.25 (Interpolated between LS & IO)

Roof Level

Shear Wall in N-S Direction

West Elevation Wall Line

Total length of exterior 8" CMU walls = 84.00 ft
Grout spacing = 32 in
total net area of shear walls = 4611.6 in^2
average shear stress, $v_{avg,NS}$ = 4.1 psi < 70.0 ***Shear Stress OK***
DCR = 0.06

East Elevation Wall Line

Total length of exterior 8" CMU walls = 60.67 ft
Grout spacing = 32 in
total net area of shear walls = 3330.8 in^2
average shear stress, $v_{avg,NS}$ = 5.7 psi < 70.0 ***Shear Stress OK***
DCR = 0.08

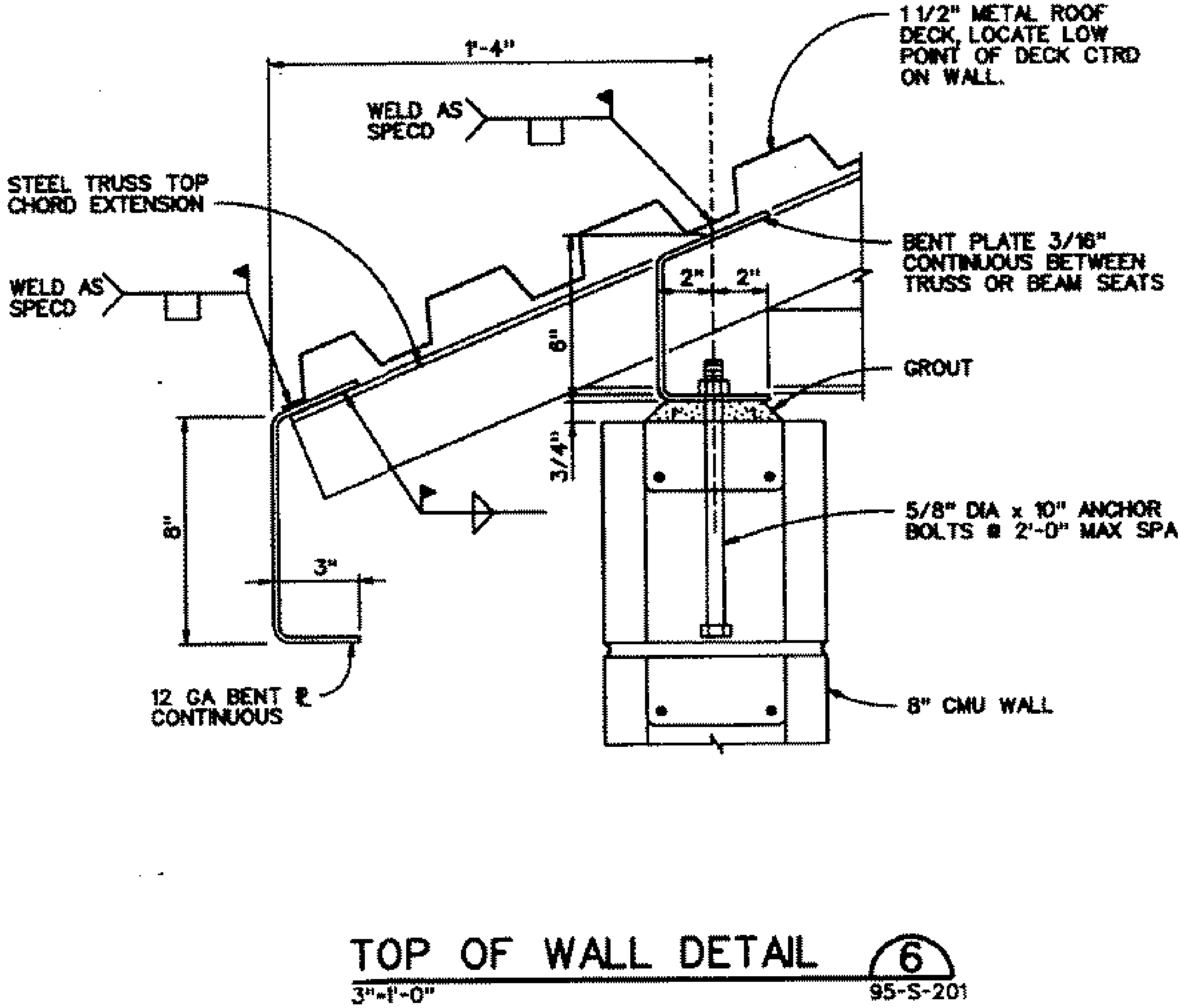
Shear Wall in E-W Direction

North Elevation Wall Line

Total length of exterior 8" CMU walls = 21.33 ft
Grout spacing = 32 in
total net area of shear walls = 1171.0 in^2
average shear stress, $v_{avg,NS}$ = 32.3 psi < 70.0 ***Shear Stress OK***
DCR = 0.46

South Elevation Wall Line

Total length of exterior 8" CMU walls = 22.00 ft
Grout spacing = 32 in
total net area of shear walls = 1207.8 in^2
average shear stress, $v_{avg,NS}$ = 31.3 psi < 70.0 ***Shear Stress OK***
DCR = 0.45



Transfer to shear wall connection

Engineers... Working Wonders With Water

BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Operations Building	JOB NO.	11962A.00
DESIGN TASK			ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)				

TRANSFER TO SHEAR WALLS

Top of Wall Connection into CMU Walls (Detail 6/95-S-202)

diaphragm shear strength, q_{ult} =	1170	lbs/ft	(assumed less than wall shear strength)
anchor bolt spacing =	24	in	
diaphragm shear strength =	2340.0	lbs	

Masonry & Steel Strength (Assuming ϕ = 1.0 for Tier 1)

anchor bolt size =	0.625	in
anchor bolt embed, l_b =	7.00	in
anchor bolt yield stress, f_y =	36.00	ksi
masonry compressive strength, f_m =	1500	psi
projected area of anchor bolt in tension, A_{pt} =	153.94	in^2
cross section area of anchor bolt, A_b =	0.31	in^2

$B_{vnc} = 1050*(f'_m*A_b)^{0.25}$ =	4863.2	lbs	masonry crushing shear strength
$B_{vnpry} = 8*A_{pt}*(f'_m)^{0.5}$ =	47696.0	lbs	anchor pryout shear strength
$B_{vns} = 0.60*A_b*f_y$ =	6626.8	lbs	steel yielding strength

Masonry crushing strength DCR =	*0.48*	***OK***
Anchor pryout DCR =	*0.05*	***OK***
Steel yielding DCR =	*0.35*	***OK***

Puddle Weld Strength

deck thickness = 0.0359 in

N-S Wall Elevations - Deck welded to support with puddle weld at 18"

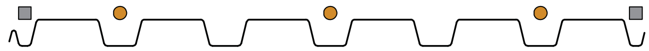
effective puddle weld diameter =	0.625	in
puddle weld spacing =	18.00	in
load at puddle weld =	1755.0	lbs /weld
strength of puddle weld =	2093.7	lbs /weld
Puddle weld strength DCR =	*0.84*	***OK***

E-W Wall Elevations - Deck welded to support with puddle weld at 12"

effective puddle weld diameter =	0.625	in
puddle weld spacing =	12.00	in
load at puddle weld =	1170.0	lbs /weld
strength of puddle weld =	2093.7	lbs /weld
Puddle weld strength DCR =	*0.56*	***OK***

Type HSB®-36

- **36/5 Weld Pattern at Supports**
- **Sidelaps connected with Button Punch or 1½" Top Seam Weld**



Allowable Diaphragm Shear Strength, q (plf) and Flexibility Factors, F ((in./lb)x10⁶)

DECK GAGE	SIDELAP ATTACHMENT		SPAN (ft-in.) 4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
22	BP @ 24"	q	369	303	248	218	187	174	156		
		F	1.5+187R	5.7+148R	9.5+122R	12.3+103R	15.2+88R	17.3+77R	19.9+68R		
	BP @ 12"	q	405	332	284	249	223	205	192		
		F	0.8+188R	5+149R	8.2+123R	10.8+104R	13.1+90R	15.1+79R	16.9+70R		
	TSW @ 24"	q	714	724	623	644	575	598	545		
		F	-4.6+191R	-2.5+153R	-0.4+127R	0.5+109R	1.8+95R	2.2+85R	3.1+76R		
	TSW @ 18"	q	857	839	731	736	739	672	682		
		F	-5.4+191R	-3.1+153R	-1.1+127R	0+109R	0.8+95R	1.7+85R	2.2+76R		
	TSW @ 12"	q	977	939	913	894	879	867	857		
		F	-5.9+191R	-3.5+153R	-1.8+127R	-0.7+109R	0.2+96R	0.9+85R	1.4+76R		
	TSW @ 6"	q	1275	1258	1246	1237	1231	1225	1001		
		F	-6.7+191R	-4.3+153R	-2.7+128R	-1.6+109R	-0.8+96R	-0.1+85R	0.4+77R		
20	BP @ 24"	q	524	433	356	315	271	249	224	213	195
		F	4.3+117R	7.5+92R	10.5+75R	12.7+63R	15.2+54R	17+47R	19.2+40R	20.6+36R	22.7+31R
	BP @ 12"	q	576	475	407	359	323	295	275	260	247
		F	3.7+118R	6.8+93R	9.3+76R	11.4+64R	13.3+55R	14.9+48R	16.4+42R	17.8+38R	19+34R
	TSW @ 24"	q	944	951	819	843	752	779	711	737	683
		F	-1.2+121R	0+96R	1.5+80R	2+69R	3+60R	3.2+53R	3.8+48R	3.9+44R	4.4+40R
	TSW @ 18"	q	1125	1097	956	959	962	874	885	894	832
		F	-2+121R	-0.5+97R	0.9+80R	1.5+69R	2+60R	2.7+54R	3+48R	3.2+44R	3.6+40R
	TSW @ 12"	q	1276	1224	1188	1160	1139	1123	1109	1085	912
		F	-2.4+121R	-0.8+97R	0.2+81R	0.9+69R	1.5+60R	1.9+54R	2.3+48R	2.5+44R	2.8+40R
	TSW @ 6"	q	1655	1631	1615	1602	1593	1585	1313	1085	912
		F	-3.1+121R	-1.6+97R	-0.6+81R	0.1+69R	0.6+61R	1+54R	1.4+48R	1.7+44R	1.9+40R
18	BP @ 24"	q	909	757	624	556	482	444	396	375	343
		F	6.2+56R	8.4+44R	10.6+35R	12.2+29R	14.2+24R	15.6+20R	17.3+17R	18.5+14R	20.2+12R
	BP @ 12"	q	989	830	716	634	573	525	487	458	435
		F	5.6+56R	7.8+44R	9.6+36R	11.1+29R	12.5+25R	13.8+21R	15+18R	16+16R	17+14R
	TSW @ 24"	q	1479	1472	1269	1295	1155	1190	1085	1120	1037
		F	1.3+59R	1.9+47R	2.8+39R	3+33R	3.6+29R	3.6+26R	4+23R	4+21R	4.4+19R
	TSW @ 18"	q	1739	1685	1468	1465	1462	1329	1341	1351	1257
		F	0.7+59R	1.4+47R	2.3+39R	2.5+34R	2.7+29R	3.2+26R	3.3+23R	3.4+21R	3.7+20R
	TSW @ 12"	q	1958	1871	1808	1762	1725	1697	1673	1654	1394
		F	0.3+59R	1.1+47R	1.6+39R	2+34R	2.3+29R	2.5+26R	2.7+24R	2.8+21R	3+20R
	TSW @ 6"	q	2520	2479	2449	2427	2410	2397	2007	1659	1394
		F	-0.3+59R	0.4+47R	0.9+39R	1.3+34R	1.6+30R	1.8+26R	1.9+24R	2.1+22R	2.2+20R
16	BP @ 24"	q	1161	984	812	731	634	591	527	501	457
		F	6.4+31R	8.2+24R	10+18R	11.4+15R	13+12R	14.2+9R	15.7+7R	16.7+6R	18.2+4R
	BP @ 12"	q	1285	1098	955	854	777	718	670	631	600
		F	6+31R	7.6+24R	9.1+19R	10.4+15R	11.5+13R	12.6+11R	13.6+9R	14.5+7R	15.4+6R
	TSW @ 24"	q	1904	1907	1647	1687	1508	1557	1422	1471	1363
		F	2.1+33R	2.4+27R	3.1+22R	3.1+19R	3.5+17R	3.5+15R	3.8+13R	3.8+12R	4+11R
	TSW @ 18"	q	2246	2185	1909	1910	1911	1741	1759	1774	1652
		F	1.6+34R	2+27R	2.6+22R	2.7+19R	2.8+17R	3.1+15R	3.2+13R	3.2+12R	3.4+11R
	TSW @ 12"	q	2529	2424	2350	2295	2252	2218	2190	2167	1941
		F	1.2+34R	1.7+27R	2+22R	2.2+19R	2.4+17R	2.5+15R	2.6+13R	2.7+12R	2.8+11R
	TSW @ 6"	q	3232	3185	3152	3127	3108	3093	2795	2310	1941
		F	0.7+34R	1.1+27R	1.4+23R	1.6+19R	1.7+17R	1.9+15R	2+14R	2+12R	2.1+11R

See footnotes on page 28.

Deck Span = 6'-8"
q = 1170 psf (interpolated)

Engineers... Working Wonders With Water

BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Operations Building	JOB NO.	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)			

FOUNDATION DOWELS

Wall Shear Strength

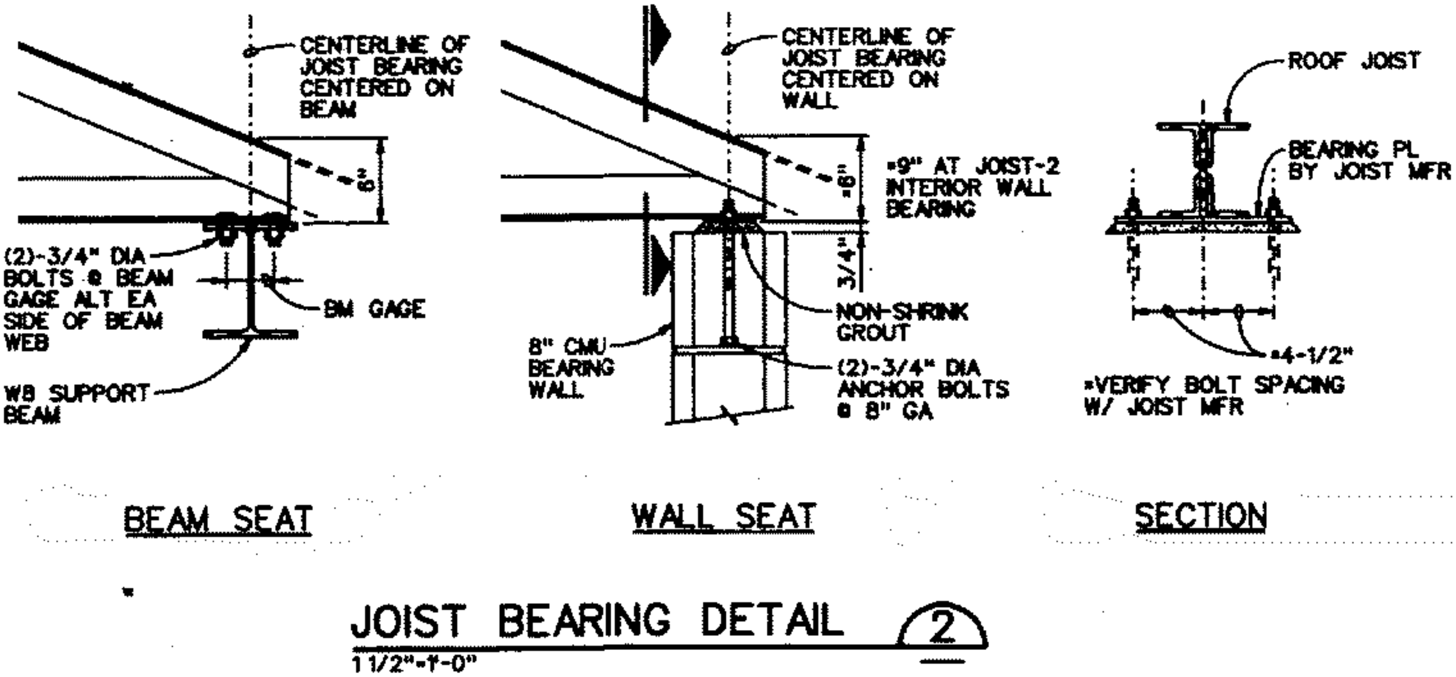
steel yield strength, f_y =	60000	psi	
Seismic unit shear, Vu =	0.43	kip/ft	
Seismic unit moment, Mu =	4.3	ft*kip/ft	
unit depth, dv =	12.00	in	
Mu/(Vu*dv) =	10.07		
Wall area, A_{nv} =	91.5	in^2	
masonry strength, f'_m =	1500	psi	
Reinforcement area, A_v =	0.44	in^2	
reinforcement spacing, s =	32.0	in	
Nominal reinforcement shear strength, V_{ns} =	4.95	kip	
γ_g =	0.75		
Nominal Unit Wall Shear, V_n =	10.63	kip/ft	ACI 530-13 Eq. 9-23

Shear Friction between wall and slab

Dowels into foundation are #6@32"

Reinforcement area, A_{vf} =	0.44	in^2
μ =	1.0	
Unit Shear Friction, V_n =	26.40	kip/ft

Dowels can develop wall strength



ROOF JOIST BEARING CONNECTION TO CMU WALL

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BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Operations Building	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)		

WALL ANCHORAGE FORCE

Operations Building: Roof Joist Bearing Anchorage into 8" CMU Wall along East and West Wall Elevations

4.4.3.7 Flexible Diaphragm Connection Forces. The horizontal seismic forces associated with the connection of a flexible diaphragm to either concrete or masonry walls, T_c, shall be calculated in accordance with Eq. (4-12).

$$T_c = \psi S_{XS} w_p A_p \qquad (4\text{-}12)$$

where

w_p = Unit weight of the wall;
A_p = Area of wall tributary to the connection;
ψ = 1.0 for Collapse Prevention Performance Level, 1.3 for Life Safety Performance Level, and 1.8 for Immediate Occupancy Performance Level; and
S_{XS} = Value specified in Section 4.4.2.3.

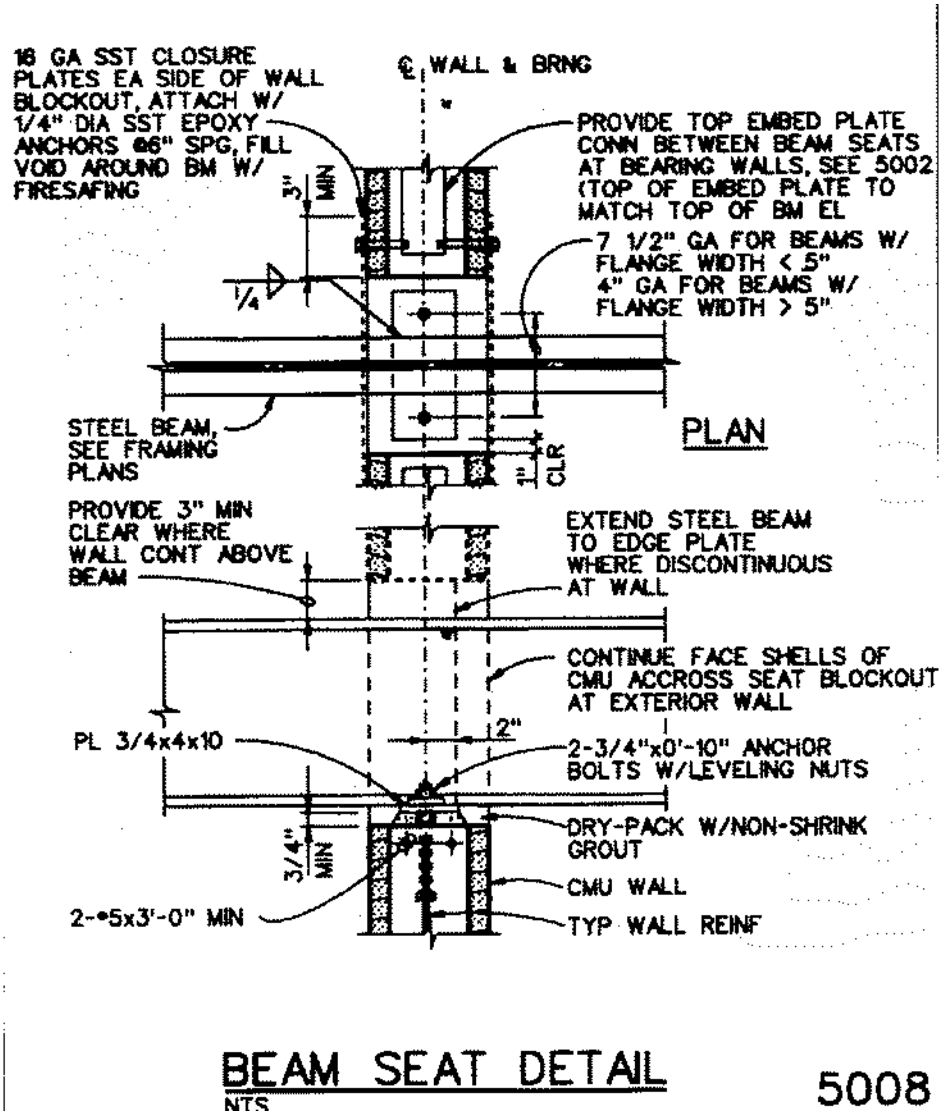
wall height to diaphragm, h_w =	10.17	ft	
unit weight of wall, w_p =	58.20	psf	(partial grout for wall)
Ψ =	1.55		(Interpolated between LS & IO)
S_{XS} =	0.446	g	
wall out-of-plane load =	204.6	lbs/ft	
roof joist spacing =	6.33	ft	
wall anchorage force, T_c =	1295.0	lbs	

Masonry & Steel Strength

anchor bolt size =	0.750	in
anchor bolt embed, l_b =	7.00	in
anchor bolt location from face, l_{be} =	3.81	in
anchor bolt yield stress, f_y =	36.00	ksi
masonry compressive strength, f_m =	1500	psi
projected area of anchor bolt in tension, A_{pt} =	101.1	in^2
projected area of each anchor bolt in shear, A_{pvbolt} =	22.80	in^2
cross section area of anchor bolt, A_b =	0.44	in^2
estimated overlap of projected area, $A_{ptoverlap}$ =	43.20	in^2
net projected area of anchor bolt in tension, A_{ptnet} =	180.70	in^2
estimated overlap of projected area, $A_{pvoverlap}$ =	0.00	in^2
net projected area of anchor bolt in shear, A_{pvnet} =	45.60	in^2

$\phi B_{vnb} = 4 * A_{pvnet} * (f'_m)^{0.5}$ =	7064.9	lbs	group masonry breakout shear strength
$\phi B_{vnc} = 1050 * (f'_m * A_b)^{0.25}$ =	10654.8	lbs	group masonry crushing shear strength
$\phi B_{vnpry} = 8 * A_{ptnet} * (f'_m)^{0.5}$ =	55987.6	lbs	group anchor pryout shear strength
$\phi B_{vns} = 0.60 * A_b * f_y$ =	19085.2	lbs	group steel yielding strength

Masonry breakout strength DCR =	*0.18*	***OK***
Masonry crushing strength DCR =	*0.12*	***OK***
Anchor pryout DCR =	*0.02*	***OK***
Steel yielding DCR =	*0.07*	***OK***



BEAM BEARING CONNECTION TO CMU WALL

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BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Operations Building	**JOB NO.**	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)			

WALL ANCHORAGE FORCE

Operations Building: Beam Anchorage into 8" CMU Wall along East and West Wall Elevations

4.4.3.7 Flexible Diaphragm Connection Forces. The horizontal seismic forces associated with the connection of a flexible diaphragm to either concrete or masonry walls, T_c, shall be calculated in accordance with Eq. (4-12).

$$T_c = \psi S_{XS} w_p A_p \qquad (4\text{-}12)$$

where

w_p = Unit weight of the wall;
A_p = Area of wall tributary to the connection;
ψ = 1.0 for Collapse Prevention Performance Level, 1.3 for Life Safety Performance Level, and 1.8 for Immediate Occupancy Performance Level; and
S_{XS} = Value specified in Section 4.4.2.3.

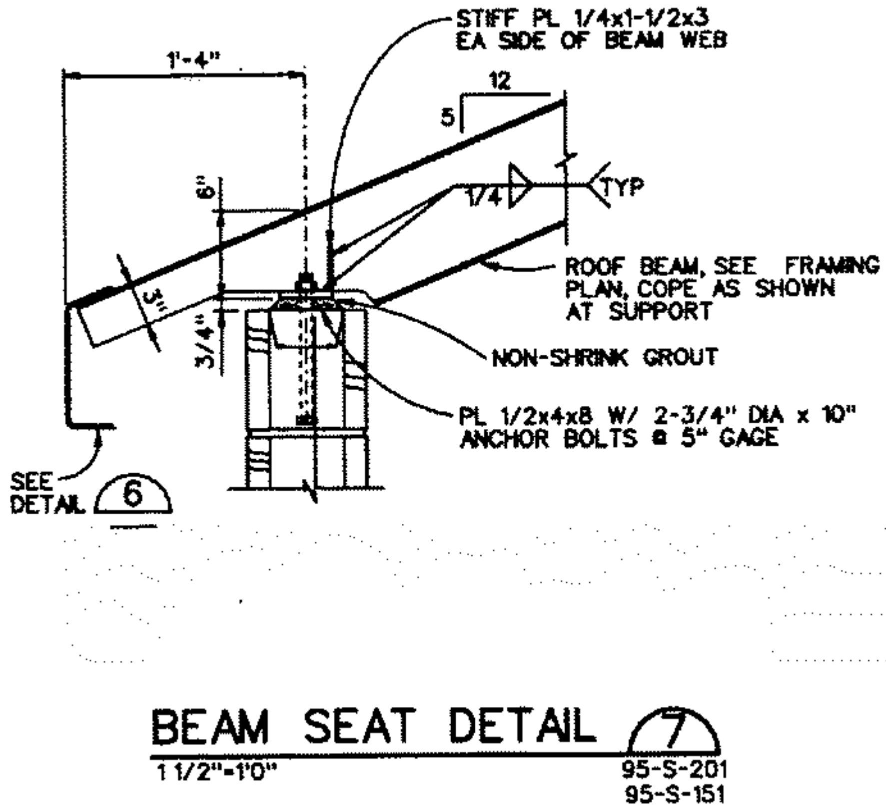
wall height to diaphragm, h_w =	18.36	ft	
unit weight of wall, w_p =	58.20	psf	(partial grout for exterior walls [CMU + venee
Ψ =	1.55		(Interpolated between LS & IO)
S_{XS} =	0.446	g	
wall out-of-plane load =	369.3	lbs/ft	
beam spacing =	8.33	ft	
wall anchorage force, T_c =	3076.6	lbs	

Masonry & Steel Strength

anchor bolt size =	0.750	in
anchor bolt embed, l_b =	7.00	in
anchor bolt location from face, l_{be} =	3.81	in
anchor bolt yield stress, f_y =	36.00	ksi
masonry compressive strength, f_m =	1500	psi
projected area of single anchor bolt in tension, A_{pt} =	101.1	in^2
projected area of single anchor bolt in shear, A_{pvbolt} =	22.80	in^2
cross section area of single anchor bolt, A_b =	0.44	in^2
estimated overlap of projected area, $A_{ptoverlap}$ =	43.20	in^2
net projected area of anchor bolt in tension, A_{ptnet} =	180.70	in^2
estimated overlap of projected area, $A_{pvoverlap}$ =	1.25	in^2
net projected area of anchor bolt in shear, A_{pvnet} =	44.98	in^2

$\phi B_{vnb} = 4 * A_{pvnet} * (f'_m)^{0.5}$ =	6968.1	lbs	group masonry breakout shear strength
$\phi B_{vnc} = 1050 * (f'_m * A_b)^{0.25}$ =	10654.8	lbs	group masonry crushing shear strength
$\phi B_{vnpry} = 8 * A_{ptnet} * (f'_m)^{0.5}$ =	55987.6	lbs	group anchor pryout shear strength
$\phi B_{vns} = 0.60 * A_b * f_y$ =	19085.2	lbs	group steel yielding strength

Masonry breakout strength DCR =	*0.44*	***OK***
Masonry crushing strength DCR =	*0.29*	***OK***
Anchor pryout DCR =	*0.05*	***OK***
Steel yielding DCR =	*0.16*	***OK***



SLOPED BEAM BEARING CONNECTION TO CMU WALLS

BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Operations Building	**JOB NO.**	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)			

WALL ANCHORAGE FORCE

Operations Building: Beam Anchorage into 8" CMU Wall along North and South Wall Elevations

4.4.3.7 Flexible Diaphragm Connection Forces. The horizontal seismic forces associated with the connection of a flexible diaphragm to either concrete or masonry walls, T_c, shall be calculated in accordance with Eq. (4-12).

$$T_c = \psi S_{XS} w_p A_p \qquad (4\text{-}12)$$

where

w_p = Unit weight of the wall;
A_p = Area of wall tributary to the connection;
ψ = 1.0 for Collapse Prevention Performance Level, 1.3 for Life Safety Performance Level, and 1.8 for Immediate Occupancy Performance Level; and
S_{XS} = Value specified in Section 4.4.2.3.

wall height to diaphragm, h_w =	10.17	ft	
unit weight of wall, w_p =	58.20	psf	(partial grout for exterior walls [CMU + venee
Ψ =	1.55		(Interpolated between LS & IO)
S_{XS} =	0.446	g	
wall out-of-plane load =	204.6	lbs/ft	
beam spacing =	6.67	ft	
wall anchorage force, T_c =	1364.6	lbs	

Masonry & Steel Strength

anchor bolt size =	0.750	in
anchor bolt embed, l_b =	7.00	in
anchor bolt location from face, l_{be} =	3.81	in
anchor bolt yield stress, f_y =	36.00	ksi
masonry compressive strength, f_m =	1500	psi
projected area of single anchor bolt in tension, A_{pt} =	101.1	in^2
projected area of single anchor bolt in shear, A_{pvbolt} =	22.80	in^2
cross section area of single anchor bolt, A_b =	0.44	in^2
estimated overlap of projected area, $A_{ptoverlap}$ =	63.60	in^2
net projected area of anchor bolt in tension, A_{ptnet} =	170.50	in^2
estimated overlap of projected area, $A_{pvoverlap}$ =	2.20	in^2
net projected area of anchor bolt in shear, A_{pvnet} =	44.50	in^2

$\phi B_{vnb} = 4 * A_{pvnet} * (f'_m)^{0.5}$ =	6894.5	lbs	group masonry breakout shear strength
$\phi B_{vnc} = 1050 * (f'_m * A_b)^{0.25}$ =	10654.8	lbs	group masonry crushing shear strength
$\phi B_{vnpry} = 8 * A_{ptnet} * (f'_m)^{0.5}$ =	52827.2	lbs	group anchor pryout shear strength
$\phi B_{vns} = 0.60 * A_b * f_y$ =	19085.2	lbs	group steel yielding strength

Masonry breakout strength DCR =	*0.20*	***OK***
Masonry crushing strength DCR =	*0.13*	***OK***
Anchor pryout DCR =	*0.03*	***OK***
Steel yielding DCR =	*0.07*	***OK***

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Process Gallery
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	07/06/21
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Pro
Project Number 11962A.00

Table 17-2. Collapse Prevention Basic Configuration Checklist

Very Low Seismicity BSE-2E Seismic Level at Limited Safety

Structural Components

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☒	N/A ☐	U ☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	There is a roof beam aligned with an interior shear wall but no connection to the wall for seismic transfer. The diaphragm at this location will lack the ability to transfer diaphragm forces into the shear wall.
C ☒	NC ☐	N/A ☐	U ☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps taht are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	- Ledger anchorage steel yielding DCR = 0.05 (OK) - Interior wall bearing anchorage masonry breakout strength DCR = 0.11 (OK) - Beam anchorage masonry breakout strength DCR = 0.39 (OK)

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name	City of Wilsonville - Pro
Project Number	11962A.00

Low Seismicity

Building System

General

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☒	N/A ☐	U ☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	There is a roof beam aligned with an interior shear wall but no connection to the wall for seismic transfer. The diaphragm at this location will lack the ability to transfer diaphragm forces into the shear wall.
C ☐	NC ☐	N/A ☒	U ☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement need not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	
C ☐	NC ☐	N/A ☒	U ☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Pro
Project Number 11962A.00

Building Configuration

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1)	
C ☒	NC ☐	N/A ☐	U ☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
C ☐	NC ☒	N/A ☐	U ☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	The center interior CMU shear walls don't continue down into the basement level. These walls are supported by concrete beams.
C ☒	NC ☐	N/A ☐	U ☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☐	☐	☒	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	Building roof is considered flexible and check is not required.

Moderate Seismicity

Geologic Site Hazards

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)	Liquefaction has been determined to not be an issue per NGI technical memorandum.
☐	☐	☒	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)	There are no slopes nearby structure.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U		
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)	Liquefaction has been determined to not be an issue per NGI technical memorandum.

High Seismicity

Foundation Configuration

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)	Height = 19.50 ft Length = 56.42 ft Sa = 0.744 L/H = 56.42 / 19.50 = 2.89 0.6*Sa = 0.6 * 0.744 = 0.45 2.89 > 0.45 (ok)
C	NC	N/A	U		
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Process Gallery
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	06/23/21
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Pro
Project Number 11962A.00

Table 17-33. Collapse Prevention Structural Checklist for Building Types RM1 and RM2

Low and Moderate Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	DESCRIPTION	COMMENTS
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than 70 lb/in.^{2}. (Commentary: Sec. A.3.2.4.1. Tier 2: Sec. 5.5.3.1.1)	Roof Level N-S direction DCR = 0.15 (OK) E-W direction DCR = 0.13 (OK) 1st Floor N-S direction DCR = 0.12 (OK) E-W direction DCR = 0.13 (OK)
☐	☒	☐	☐	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in., and all vertical bars extend to the top of the walls. (Commentary: Sec. A.3.2.4.2. Tier 2: Sec. 5.5.3.1.3)	Horiz steel = #6@48" Vert steel = #6@24" (ext) & #6@32" (int) Horiz ratio = 0.44 / (7.625*48) = 0.0012 > 0.0007 (OK) Vert ratio = 0.44 / (7.625*24) = 0.0024 > 0.0007 (OK) 0.44 / (7.625*32) = 0.0018 > 0.0007 (OK) Horizontal reinforcing is specified at 48" but this is less than 48in required. Reinforcing is non-compliant.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Stiff Diaphragms

C	NC	N/A	U	DESCRIPTION	COMMENTS
☐	☐	☒	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	

Connections

C	NC	N/A	U	DESCRIPTION	COMMENTS
☒	☐	☐	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps taht are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	- Ledger anchorage steel yielding DCR = 0.05 (OK) - Interior wall bearing anchorage masonry breakout strength DCR = 0.11 (OK) - Beam anchorage masonry breakout strength DCR = 0.39 (OK)
☐	☐	☒	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	
☐	☒	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)	There is a roof beam aligned with an interior shear wall but no connection to the wall for seismic transfer.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	DESCRIPTION	COMMENTS
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2: Sec. 5.7.2)	
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
☐	☐	☒	☐	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	

High Seismicity

Stiff Diaphragms

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Pro
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C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)	

Flexible Diaphragms

C	NC	N/A	U	DESCRIPTION	COMMENTS
☒	☐	☐	☐	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
☐	☐	☒	☐	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Statement	Comments
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm shall not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name	City of Wilsonville - Pro
Project Number	11962A.00

Connections

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Process Gallery
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	07/06/2021
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

Table 17-38. Nonstructural Checklist

The Performance Level is designated LS for Life Safety or PR for Position Retention. The level of seismicity is designated as "not required" or by L, M, or H, for Low, Moderate, and High.

All Seismicity Levels

For BSE-1E Tier 1, use PR (Position Retention)

Life Safety Systems

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. FIRE SUPPRESSION PIPING: Fire suppression piping is anchored and braced in accordance with NFPA-13. (Commentary: Sec. A.7.13.1. Tier 2: Sec. 13.7.4)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Fire suppression piping has flexible couplings in accordance with NFPA-13. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.4)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. EMERGENCY POWER: Equipment used to power or control life safety systems is anchored or braced. (Commentary: Sec. A.7.12.1. Tier 2: Sec. 13.7.7)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. STAIR AND SMOKE DUCTS: Stair pressurization and smoke control ducts are braced and have flexible connections at seismic joints. (Commentary: Sec. A.7.14.1. Tier 2: Sec. 13.7.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-MH; PR-MH. SPRINKLER CEILING CLEARANCE: Penetrations through panelized ceilings for fire suppression devices provide clearances in accordance with NFPA-13. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.4)	
☐	☐	☒	☐	LS-not required; PR-LMH. EMERGENCY LIGHTING: Emergency and egress lighting equipment is anchored or braced. (Commentary: Sec. A.7.3.1. Tier 2: Sec. 13.7.9)	

Hazardous Materials

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-LMH; PR-LMH. HAZARDOUS MATERIAL EQUIPMENT: Equipment mounted on vibration isolators and containing hazardous material is equipped with restraints or snubbers. (Commentary: Sec. A.7.12.2. Tier 2: 13.7.1)	
☐	☐	☒	☐	LS-LMH; PR-LMH. HAZARDOUS MATERIAL STORAGE: Breakable containers that hold hazardous material, including gas cylinders, are restrained by latched doors, shelf lips, wires, or other methods. (Commentary: Sec. A.7.15.1. Tier 2: Sec. 13.8.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Statement	Comments
☐	☐	☒	☐	LS-MH; PR-MH. HAZARDOUS MATERIAL DISTRIBUTION: Piping or ductwork conveying hazardous materials is braced or otherwise protected from damage that would allow hazardous material release. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)	
☐	☐	☒	☐	LS-MH; PR-MH. SHUT-OFF VALVES: Piping containing hazardous material, including natural gas, has shut-off valves or other devices to limit spills or leaks. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.3 and 13.7.5)	
☐	☐	☒	☐	LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Hazardous material ductwork and piping, including natural gas piping, has flexible couplings. (Commentary: Sec. A.7.15.4, Tier 2: Sec.13.7.3 and 13.7.5)	
☐	☐	☒	☐	LS-MH; PR-MH. PIPING OR DUCTS CROSSING SEISMIC JOINTS: Piping or ductwork carrying hazardous material that either crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.13.6. Tier 2: Sec.13.7.3, 13.7.5, and 13.7.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

Partitions

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. UNREINFORCED MASONRY: Unreinforced masonry or hollow-clay tile partitions are braced at a spacing of at most 10 ft in Low or Moderate Seismicity, or at most 6 ft in High Seismicity. (Commentary: Sec. A.7.1.1. Tier 2: Sec. 13.6.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. HEAVY PARTITIONS SUPPORTED BY CEILINGS: The tops of masonry or hollow-clay tile partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. DRIFT: Rigid cementitious partitions are detailed to accommodate the following drift ratios: in steel moment frame, concrete moment frame, and wood frame buildings, 0.02; in other buildings, 0.005. (Commentary A.7.1.2 Tier 2: Sec. 13.6.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-MH. LIGHT PARTITIONS SUPPORTED BY CEILINGS: The tops of gypsum board partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-MH. STRUCTURAL SEPARATIONS: Partitions that cross structural separations have seismic or control joints. (Commentary: Sec. A.7.1.3. Tier 2. Sec. 13.6.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-MH. TOPS: The tops of ceiling-high framed or panelized partitions have lateral bracing to the structure at a spacing equal to or less than 6 ft. (Commentary: Sec. A.7.1.4. Tier 2. Sec. 13.6.2)	

Ceilings

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-LMH. SUSPENDED LATH AND PLASTER: Suspended lath and plaster ceilings have attachments that resist seismic forces for every 12 ft^2 of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-LMH. SUSPENDED GYPSUM BOARD: Suspended gypsum board ceilings have attachments that resist seismic forces for every 12 ft^2 of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Item	Comments
☐	☐	☒	☐	LS-not required; PR-MH. INTEGRATED CEILINGS: Integrated suspended ceilings with continuous areas greater than 144 ft^2, and ceilings of smaller areas that are not surrounded by restraining partitions, are laterally restrained at a spacing no greater than 12 ft with members attached to the structure above. Each restraint location has a minimum of four diagonal wires and compression struts, or diagonal members capable of resisting compression. (Commentary: Sec. A.7.2.2. Tier 2: Sec. 13.6.4)	
☐	☐	☒	☐	LS-not required; PR-MH. EDGE CLEARANCE: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 have clearances from the enclosing wall or partition of at least the following: in Moderate Seismicity, 1/2 in.; in High Seismicity, 3/4 in. (Commentary: Sec. A.7.2.4. Tier 2: Sec. 13.6.4)	
☐	☐	☒	☐	LS-not required; PR-MH. CONTINUITY ACROSS STRUCTURE JOINTS: The ceiling system does not cross any seismic joint and is not attached to multiple independent structures. (Commentary: Sec. A.7.2.5. Tier 2: Sec. 13.6.4)	
☐	☐	☒	☐	LS-not required; PR-H. EDGE SUPPORT: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 are supported by closure angles or channels not less than 2 in. wide. (Commentary: Sec. A.7.2.6. Tier 2: Sec. 13.6.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-not required; PR-H. SEISMIC JOINTS: Acoustical tile or lay-in panel ceilings have seismic separation joints such that each continuous portion of the ceiling is no more than 2500 ft^2 and has a ratio of long-to-short dimension no more than 4-to-1. (Commentary: Sec. A.7.2.7. Tier 2: 13.6.4)	

Light Fixtures

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-MH; PR-MH. INDEPENDENT SUPPORT: Light fixtures that weigh more per square foot than the ceiling they penetrate are supported independent of the grid ceiling suspension system by a minimum of two wires at diagonally opposite corners of each fixture. (Commentary: Sec. A.7.3.2. Tier 2: Sec. 13.6.4 and 13.7.9)	
C	NC	N/A	U		
☒	☐	☐	☐	LS-not required; PR-H PENDANT SUPPORTS: Light fixtures on pendant supports are attached at a spacing equal to or less than 6 ft. Unbraced suspended fixtures are free to allow a 360-degree range of motion at an angle not less than 45 degrees from horizontal without contacting adjacent components. Alternatively, if rigid supported and/or braced, they are free to move with the structure to which they are attached without damaging adjoining components. Additionally, the connection to the structure is capable of accommodating the movement without failure. (Commentary: Sec. A.7.3.3. Tier 2: Sec. 13.7.9)	
C	NC	N/A	U		
☒	☐	☐	☐	LS-not required; PR-H. LENS COVERS: Lens covers on light fixtures are attached with safety devices. (Commentary: Sec. A.7.3.4. Tier 2: Sec. 13.7.9)	Rooms with lens cover present do have safety devices.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

Cladding and Glazing

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. CLADDING ANCHORS: Cladding components weighing more than 10 lb/ft^2 are mechanically anchored to the structure at a spacing equal to or less than the following: for Life Safety in Moderate Seismicity, 6 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 ft. (Commentary: Sec. A.7.4.1. Tier 2: Sec. 13.6.1)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH CLADDING ISOLATION: For steel or concrete moment-frame buildings, panel connections are detailed to accommodate a story drift ratio by the use of rods attached to framing with oversize holes or slotted holes of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02, and the rods have a length-to-diameter ratio of 4.0 or less. (Commentary: Sec. A.7.4.3. Tier 2: Sec. 13.6.1)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH MULTI-STORY PANELS: For multi-story panels attahed at more than one floor level panel connections are detailed to accommodate a story drift ratio by the use of rods attached to framing with oversize holes or slotted holes of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02, and the rods have a length-to-diameter ratio of 4.0 or less. (Commentary: Sec. A.7.4.4. Tier 2: Sec. 13.6.1)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH THREADED RODS: Threaded rods for panel connections detailed to accommodate drift by bending of the rod have a length-to-diameter ratio greater than 0.06 times the story height in inches for Life Safety in Moderate Seismicity and 0.12 times the story height in inches for Life Safety in High Seismicity and for Position Retention in any seismicity. (Commentary: Sec. A.7.4.9. Tier 2: Sec. 13.6.1)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

C	NC	N/A	U		
☐	☐	☒	☐	LS-MH; PR-MH. PANEL CONNECTIONS: Cladding panels are anchored out-of-plane with a minimum number of connections for each wall panel, as follows: for Life Safety in Moderate Seismicity, 2 connections; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 connections. (Commentary: Sec. A.7.4.5. Tier 2: Sec. 13.6.1.4)	

Project Name City of Wilsonville
Project Number 11962A.00

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-MH; PR-MH. BEARING CONNECTIONS: Where bearing connections are used, there is a minimum of two bearing connections for each cladding panel. (Commentary: Sec. A.7.4.6. Tier 2: Sec. 13.6.1.4)	
☐	☐	☒	☐	LS-MH; PR-MH. INSERTS: Where concrete cladding components use inserts, the inserts have positive anchorage or are anchored to reinforcing steel. (Commentary: Sec. A.7.4.7. Tier 2: Sec. 13.6.1.4)	
☐	☐	☒	☐	LS-MH; PR-MH. OVERHEAD GLAZING: Glazing panes of any size in curtain walls and individual interior or exterior panes over 16 ft^2 in area are laminated annealed or laminated heat-strengthened glass and are detailed to remain in the frame when cracked. (Commentary: Sec. A.7.4.8: Tier 2: Sec. 13.6.1.5)	

Masonry Veneer

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. TIES: Masonry veneer is connected to the backup with corrosion-resistant ties. There is a minimum of one tie for every 2-2/3 ft^2, and the ties have spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 36 in.; for Life Safety in High Seismicity and for Position Retention in any seismicity, 24 in. (Commentary: Sec. A.7.5.1. Tier 2: Sec. 13.6.1.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Item	
☐	☐	☒	☐	LS-LMH; PR-LMH. SHELF ANGLES: Masonry veneer is supported by shelf angles or other elements at each floor above the ground floor. (Commentary: Sec. A.7.5.2. Tier 2: Sec. 13.6.1.2)	
☐	☐	☒	☐	LS-LMH; PR-LMH. WEAKENED PLANES: Masonry veneer is anchored to the backup adjacent to weakened planes, such as at the locations of flashing. (Commentary: Sec. A.7.5.3. Tier 2: Sec. 13.6.1.2)	
☐	☐	☒	☐	LS-LMH; PR-LMH. UNREINFORCED MASONRY BACKUP: There is no unreinforced masonry backup. (Commentary: Sec. A.7.7.2. Tier 2: Section 13.6.1.1 and 13.6.1.2)	
☐	☐	☒	☐	LS-MH; PR-MH STUD TRACKS: For veneer with cold-formed steel stud backup, stud tracks are fastened to the structure at a spacing equal to or less than 24 in. on center. (Commentary: Sec. A.7.6.1. Tier 2: Sec. 13.6.1.1 and 13.6.1.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

Rating				Description	Comments
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. ANCHORAGE: For veneer with concrete block or masonry backup, the backup is positively anchored to the structure at a horizontal spacing equal to or less than 4 ft along the floors and roof. (Commentary: Sec. A.7.7.1. Tier 2: Section 13.6.1.1 and 13.6.1.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-MH. WEEP HOLES: In veneer anchored to stud walls, the veneer has functioning weep holes and base flashing. (Commentary: Sec. A.7.5.6. Tier 2: Section 13.6.1.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-MH OPENINGS: For veneer with cold-formed -steel stud backup, steel studs frame window and door openings. (Commentary: Sec. A.7.6.2. Tier 2: Sec. 13.6.1.1 and 13.6.1.2)	

Parapets, Cornices, Ornamentation, and Appendages

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. URM PARAPETS OR CORNICES: Laterally unsupported unreinforced masonry parapets or cornices have height-to-thickness ratios no greater than the following: for Life Safety in Low or Moderate Seismicity, 2.5; for Life Safety in High Seismicity and for Position Retention in any seismicity, 1.5. (Commentary: Sec. A.7.8.1. Tier 2: Sec. 13.6.5)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-LMH; PR-LMH. CANOPIES: Canopies at building exits are anchored to the structure at a spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 10 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 6 ft. (Commentary: Sec. A.7.8.2. Tier 2: Sec. 13.6.6)	
☐	☐	☒	☐	LS-MH; PR-LMH. CONCRETE PARAPETS: Concrete parapets with height-to-thickness ratios greater than 2.5 have vertical reinforcement. (Commentary: Sec. A.7.8.3. Tier 2: Sec. 13.6.5)	
☐	☐	☒	☐	LS-MH; PR-LMH. APPENDAGES: Cornices, parapets, signs, and other ornamentation or appendages that extend above the highest point of anchorage to the structure or cantilever from components are reinforced and anchored to the structural system at a spacing equal to or less than 6 ft. This checklist item does not apply to parapets or cornices covered by other checklist items. (Commentary: Sec. A.7.8.4. Tier 2: Sec. 13.6.6)	

Masonry Chimneys

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-LMH; PR-LMH. URM CHIMNEYS: Unreinforced masonry chimneys extend above the roof surface no more than the following: for Life Safety in Low or Moderate Seismicity, 3 times the least dimension of the chimney; for Life Safety in High Seismicity and for Position Retention in any seismicity, 2 times the least dimension of the chimney. (Commentary: Sec. A.7.9.1. Tier 2: 13.6.7)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-LMH; PR-LMH. ANCHORAGE: Masonry chimneys are anchored at each floor level, at the topmost ceiling level, and at the roof. (Commentary: Sec. A.7.9.2. Tier 2: 13.6.7)	

Stairs

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-LMH; PR-LMH. STAIR ENCLOSURES: Hollow-clay tile or unreinforced masonry walls around stair enclosures are restrained out-of-plane and have height-to-thickness ratios not greater than the following: for Life Safety in Low or Moderate Seismicity, 15-to-1; for Life Safety in High Seismicity and for Position Retention in any seismicity, 12-to-1. (Commentary: Sec. A.7.10.1. Tier 2: Sec. 13.6.2 and 13.6.8)	
☒	☐	☐	☐	LS-LMH; PR-LMH STAIR DETAILS: The connection between the stairs and the structure does not rely on post-installed anchors in concrete or masonry, and the stair details are capable of accommodating the drift calculated using the Quick Check procedure of Section 4.4.3.1 for moment-frame structures or 0.5 in. for all other structures without including any lateral stiffness contribution from the stairs. (Commentary: Sec. A.7.10.2. Tier 2: Sec. 13.6.8)	

Contents and Furnishings

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-MH; PR-MH. INDUSTRIAL STORAGE RACKS: Industrial storage racks or pallet racks more than 12 ft high meet the requirements of ANSI/MH 16.1 as modified by ASCE 7 Chapter 15. (Commentary: Sec. A.7.11.1. Tier 2: Sec. 13.8.1)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
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C	NC	N/A	U	Item	Comments
☐	☐	☒	☐	LS-H; PR-MH. TALL NARROW CONTENTS: Contents more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 are anchored to the structure or to each other. (Commentary: Sec. A.7.11.2. Tier 2: Sec. 13.8.2)	
☐	☐	☒	☐	LS-H; PR-H. FALL-PRONE CONTENTS: Equipment, stored items, or other contents weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level are braced or otherwise restrained. (Commentary: Sec. A.7.11.3. Tier 2: Sec. 13.8.2)	
☐	☐	☒	☐	LS-not required; PR-MH. ACCESS FLOORS: Access floors more than 9 in. high are braced. (Commentary: Sec. A.7.11.4. Tier 2: Sec. 13.8.3)	
☐	☐	☒	☐	LS-not required; PR-MH. EQUIPMENT ON ACCESS FLOORS: Equipment and other contents supported by access floor systems are anchored or braced to the structure independent of the access floor. (Commentary: Sec. A.7.11.5. Tier 2: Sec. 13.7.7 and 13.8.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	DESCRIPTION	COMMENTS
☒	☐	☐	☐	LS-not required; PR-H. SUSPENDED CONTENTS: Items suspended without lateral bracing are free to swing from or move with the structure from which they are suspended without damaging themselves or adjoining components. (Commentary. A.7.11.6. Tier 2: Sec. 13.8.2)	

Mechanical and Electrical Equipment

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-H; PR-H. FALL-PRONE EQUIPMENT: Equipment weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level, and which is not in-line equipment, is braced. (Commentary: A.7.12.4. Tier 2: 13.7.1 and 13.7.7)	
☐	☒	☐	☐	LS-H; PR-H. IN-LINE EQUIPMENT: Equipment installed in-line with a duct or piping system, with an operating weight more than 75 lb, is supported and laterally braced independent of the duct or piping system. (Commentary: Sec. A.7.12.5. Tier 2: Sec. 13.7.1)	Equipment 40-ASU-02 lacks anchorage along backside channel supports. There are (2) anchors to the front, but missing in back. See photo on next page. Aeration blower pumps do have anchor rods but the nuts appear to be backing off or missing completely.
☐	☐	☒	☐	LS-H; PR-MH. TALL NARROW EQUIPMENT: Equipment more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 is anchored to the floor slab or adjacent structural walls. (Commentary: Sec. A.7.12.6. Tier 2: Sec. 13.7.1 and 13.7.7)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

HVAC equipment is unanchored to structure along backside.

Aeration blower pump lacks anchorage to structure. Nuts are missing or backing off from threaded rods.

Project Name City of Wilsonville
Project Number 11962A.00

C	NC	N/A	U	Statement	Comments
☐	☐	☒	☐	LS-not required; PR-MH. MECHANICAL DOORS: Mechanically operated doors are detailed to operate at a story drift ratio of 0.01. (Commentary: Sec. A.7.12.7. Tier 2: Sec. 13.6.9)	
☒	☐	☐	☐	LS-not required; PR-H. SUSPENDED EQUIPMENT: Equipment suspended without lateral bracing is free to swing from or move with the structure from which it is suspended without damaging itself or adjoining components. (Commentary: Sec. A.7.12.8. Tier 2: Sec. 13.7.1 and 13.7.7)	
☐	☐	☒	☐	LS-not required; PR-H. VIBRATION ISOLATORS: Equipment mounted on vibration isolators is equipped with horizontal restraints or snubbers and with vertical restraints to resist overturning. (Commentary: Sec. A.7.12.9. Tier 2: Sec. 13.7.1)	
☐	☐	☒	☐	LS-not required; PR-H. HEAVY EQUIPMENT: Floor-supported or platform-supported equipment weighing more than 400 lb is anchored to the structure. (Commentary: Sec. A.7.12.10. Tier 2: 13.7.1 and 13.7.7)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	LS-not required; PR-H. ELECTRICAL EQUIPMENT: Electrical equipment is laterally braced to the structure. (Commentary: Sec. A.7.12.11. Tier 2: 13.7.7)	
☐	☐	☒	☐	LS-not required; PR-H. CONDUIT COUPLINGS: Conduit greater than 2.5 in. trade size that is attached to panels, cabinets, or other equipment and is subject to relative seismic displacement has flexible couplings or connections. (Commentary: Sec. A.7.12.12. Tier 2: 13.7.8)	

Piping

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	LS-not required; PR-H. FLEXIBLE COUPLINGS: Fluid and gas piping has flexible couplings. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.3 and 13.7.5)	
☐	☒	☐	☐	LS-not required; PR-H. FLUID AND GAS PIPING: Fluid and gas piping is anchored and braced to the structure to limit spills or leaks. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)	Multiple pipes lack restraint to unistrut supports as these pipes are sitting on supports. Compression strut supports lack diagonal bracing back to structure.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Piping lacks restraint to unistrut support. Pipe shown is sitting on unistrut.

Compression strut lacks diagonal bracing.

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-not required; PR-H. C-CLAMPS: One-sided C-clamps that support piping larger than 2.5 in. in diameter are restrained. (Commentary: Sec. A.7.13.5. Tier 2: Sec. 13.7.3 and 13.7.5)	
☐	☐	☒	☐	LS-not required; PR-H. PIPING CROSSING SEISMIC JOINTS: Piping that crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A7.13.6. Tier 2: Sec.13.7.3 and Sec. 13.7.5)	

Ducts

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. DUCT BRACING: Rectangular ductwork larger than 6 ft^2 in cross-sectional area and round ducts larger than 28 in. in diameter are braced. The maximum spacing of transverse bracing does not exceed 30 ft. The maximum spacing of longitudinal bracing does not exceed 60 ft. (Commentary: Sec. A.7.14.2. Tier 2: Sec. 13.7.6)	
☒	☐	☐	☐	LS-not required; PR-H. DUCT SUPPORT: Ducts are not supported by piping or electrical conduit. (Commentary: Sec. A.7.14.3. Tier 2: Sec. 13.7.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. DUCTS CROSSING SEISMIC JOINTS: Ducts that cross seismic joints or isolation planes or are connected to independent structures have couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.14.5. Tier 2: Sec. 13.7.6)	

Elevators

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-H; PR-H. RETAINER GUARDS: Sheaves and drums have cable retainer guards. (Commentary: Sec. A.7.16.1. Tier 2: 13.8.6)	
C	NC	N/A	U		
☐	☐	☒	☐	LS-H; PR-H. RETAINER PLATE: A retainer plate is present at the top and bottom of both car and counterweight. (Commentary: Sec. A.7.16.2. Tier 2: 13.8.6)	
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. ELEVATOR EQUIPMENT: Equipment, piping, and other components that are part of the elevator system are anchored. (Commentary: Sec. A.7.16.3. Tier 2: 13.8.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. SEISMIC SWITCH: Elevators capable of operating at speeds of 150 ft/min or faster are equipped with seismic switches that meet the requirements of ASME A17.1 or have trigger levels set to 20% of the acceleration of gravity at the base of the structure and 50% of the acceleration of gravity in other locations. (Commentary: Sec. A.7.16.4. Tier 2: 13.8.6)	
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. SHAFT WALLS: Elevator shaft walls are anchored and reinforced to prevent toppling into the shaft during strong shaking. (Commentary: Sec. A.7.16.5. Tier 2: 13.8.6)	
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. COUNTERWEIGHT RAILS: All counterweight rails and divider beams are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.6. Tier 2: 13.8.6)	
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. BRACKETS: The brackets that tie the car rails and the counterweight rail to the structure are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.7. Tier 2: 13.8.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. SPREADER BRACKET: Spreader brackets are not used to resist seismic forces. (Commentary: Sec. A.7.16.8. Tier 2: 13.8.6)	
☐	☐	☒	☐	LS-not required; PR-H. GO-SLOW ELEVATORS: The building has a go-slow elevator system. (Commentary: Sec. A.7.16.9. Tier 2: 13.8.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

City of Wilsonville

Process Gallery Building Tier 1 Structural Calculations

OSHPD

Latitude, Longitude: 45.294444, -122.77167

Date	6/28/2021, 11:18:38 AM
Design Code Reference Document	ASCE41-17
Custom Probability	
Site Class	C - Very Dense Soil and Soft Rock

Type	**Description**	**Value**
Hazard Level		BSE-2N
S_S	spectral response (0.2 s)	0.813
S_1	spectral response (1.0 s)	0.381
S_{XS}	site-modified spectral response (0.2 s)	0.976
S_{X1}	site-modified spectral response (1.0 s)	0.571
F_a	site amplification factor (0.2 s)	1.2
F_v	site amplification factor (1.0 s)	1.5
ssuh	max direction uniform hazard (0.2 s)	0.92
crs	coefficient of risk (0.2 s)	0.884
ssrt	risk-targeted hazard (0.2 s)	0.813
ssd	deterministic hazard (0.2 s)	1.5
s1uh	max direction uniform hazard (1.0 s)	0.441
cr1	coefficient of risk (1.0 s)	0.863
s1rt	risk-targeted hazard (1.0 s)	0.381
s1d	deterministic hazard (1.0 s)	0.6

Type	**Description**	**Value**
Hazard Level		BSE-1N
S_{XS}	site-modified spectral response (0.2 s)	0.651
S_{X1}	site-modified spectral response (1.0 s)	0.381

Type	Description	Value
Hazard Level		BSE-2E
S_S	spectral response (0.2 s)	0.589
S_1	spectral response (1.0 s)	0.27
S_{XS}	site-modified spectral response (0.2 s)	0.744
S_{X1}	site-modified spectral response (1.0 s)	0.405
f_a	site amplification factor (0.2 s)	1.265
f_v	site amplification factor (1.0 s)	1.5

Type	Description	Value
Hazard Level		BSE-1E
S_S	spectral response (0.2 s)	0.223
S_1	spectral response (1.0 s)	0.082
S_{XS}	site-modified spectral response (0.2 s)	0.291
S_{X1}	site-modified spectral response (1.0 s)	0.123
F_a	site amplification factor (0.2 s)	1.3
F_v	site amplification factor (1.0 s)	1.5

Type	Description	Value
Hazard Level		TL Data
T-Sub-L	Long-period transition period in seconds	16

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BY: BS	DATE Jul-21	CLIENT City of Wilsonville	SHEET
CHKD BY	DESCRIPTION Process Gallery Building		JOB NO. 11962A.00
DESIGN TASK	Process Gallery Building Seismic Weight		

Roof Loads

Roof EL 125.63

Description	Load		
1-1/2"x20ga metal deck	2.3	psf	
Rigid insulation w/ sheet roofing	4.5		
Steel beam	3.5		
Miscellaneous	5.0		
Dead Load for Gravity Design	**15.3**	psf	
Roof Live Load	**20.0**	psf	(Assumed)
Snow Load	**25.0**	psf	

Notes

1. Roof beam self weight assumed total beam weight, 9831.0 lb, divided by total roof area, 3093.1 ft^2 which is 9831.0lb/3093.1ft^2 = 3.18 lb/ft^2. Assume 3.5 psf.

Floor Loads

Floor EL 111.00

Description	Load	
8" concrete slab	100.0	psf
Concrete beam	73.0	
Miscellaneous	10.0	
Dead Load for Gravity Design	**183.0**	psf
Floor Live Load	**200.0**	psf

Notes

1. Floor beam self weight assumed total beam weight, 226925.0 lb, divided by total floor area, 3093.1 ft^2 which is 226925.0lb/3093.1ft^2 = 73.4 lb/ft^2. Assume 73.5 psf.

Wall Loads

Wall Loads

Description	Load	
8" CMU wall (partial grouted @ 24")	51.0	psf
8" CMU wall (partial grouted @ 32")	47.0	
8" Concrete wall	100.0	
14" Concrete wall	175.0	
Exterior Plastic Veneer Finish	7.5	
8" Exterior CMU Wall Load for Seismic	**58.5**	psf
8" Interior CMU Wall Load for Seismic	**47.0**	psf
8" Concrete Wall Load for Seismic	**100.0**	psf
14" Concrete Wall Load for Seismic	**175.0**	psf

Seismic Weight

Roof Weight

Roof Area	3093.1	ft^2
Roof Seismic Weight	**47.3**	**kip**

Floor Weight

Dry Chemical Storage Area	3093.1	ft^2
Floor Seismic Weight	**566.0**	**kip**

Wall Weight

Wall Height to Roof	14.63	ft
Wall Height to 2nd Level	18.00	ft
Parapet Height	0.87	ft
8" Exterior CMU Wall Length (1st floor)	220.00	ft
8" Interior CMU Wall Length (1st floor)	108.00	ft
8" Concrete Wall Length (basement)	37.42	ft
14" Concrete Wall Length (basement)	220.00	ft
Roof Wall Seismic Weight	**142.5**	**kip**
Basement Wall Seismic Weight	**511.5**	**kip**
Combined Roof Seismic Weight	**189.8**	**kip**
Combined Base Level Seismic Weight	**1077.5**	**kip**

Total Seismic Weight	**1267.3**	**kip**

Notes

1. Wall seismic weight assumes half of the wall height associated with each level.
2. Per Section 4.5.2.1, the effective seismic weight includes the total dead load, 25% of the live load when area is used as storage, and 20% of the roof snow live load if greater than 30 psf (otherwise assume zero).

Carollo Engineers. Working Wonders With Water

BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Process Gallery	JOB NO.	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 1 Screening (BSE-2E Seismic Level)			

SEISMIC BASE SHEAR

4.4.2.1 Pseudo Seismic Force. The pseudo seismic force, in a given horizontal direction of a building, shall be calculated in accordance with Eq. (4-1).

$$V = CS_aW \quad (4\text{-}1)$$

where

V = Pseudo seismic force;

C = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response; C shall be taken from Table 4-7;

S_a = Response spectral acceleration at the fundamental period of the building in the direction under consideration. The value of S_a shall be calculated in accordance with the procedures in Section 4.4.2.3; and

W = Effective seismic weight of the building, including the total dead load and applicable portions of other gravity loads listed below:

Table 4-7. Modification Factor, C

Building Type[a]	Number of Stories 1	2	3	≥4
Wood and cold-formed steel shear wall (W1, W1a, W2, CFS1) Moment frame (S1, S3, C1, PC2a)	1.3	1.1	1.0	1.0
Shear wall (S4, S5, C2, C3, PC1a, PC2, RM2, URMa) Braced frame (S2) Cold-formed steel strap-brace wall (CFS2)	1.4	1.2	1.1	1.0
Unreinforced masonry (URM) Flexible diaphragms (S1a, S2a, S5a, C2a, C3a, PC1, RM1)	1.0	1.0	1.0	1.0

[a] Defined in Table 3-1.

Process Gallery

Modification Factor, C = 1.2

S_{X1} = 0.405 (BSE-2E seismic hazard)

T = 0.149 s

S_{XS} = 0.744 (BSE-2E seismic hazard)

Spectral Acceleration, S_a = 0.744

Seismic Weight, W = 1267.3 kip

Seismic Force, V = 1131.4 kip

Story	Weight, w_x (kip)	Floor Height, h_x (ft)	k factor	$w_xh_x^k$ (kip*ft^2)	C_{vx}	Force on Level, F_x (kip)	Story Force, V_j (kip)
Roof	189.8	32.63	1.0	6193.2	0.242	273.8	273.8
1st	1077.5	18.00	1.0	19395.0	0.758	857.6	1131.4

$\Sigma w_xh_x^k$ = 25588.2

Carollo Engineers. Working Wonders With Water

BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Process Gallery	JOB NO.	11962A.00
DESIGN TASK			ASCE 41-17 - Tier 1 Screening (BSE-2E Seismic Level)				

WALL SHEAR STRESS CHECK

4.4.3.3 Shear Stress in Shear Walls. The average shear stress in shear walls, v_j^{avg}, shall be calculated in accordance with Eq. (4-8).

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \quad (4\text{-}8)$$

where

V_j = Story shear at level j computed in accordance with Section 4.4.2.2;

A_w = Summation of the horizontal cross-sectional area of all shear walls in the direction of loading. Openings shall be taken into consideration where computing A_w. For masonry walls, the net area shall be used. For wood-framed walls, the length shall be used rather than the area; and

M_s = System modification factor; M_s shall be taken from Table 4-8.

Table 4-8. M_s Factors for Shear Walls

	Level of Performance		
Wall Type	CP[a]	LS[a]	IO[a]
Reinforced concrete, precast concrete, wood, reinforced masonry, and cold-formed steel	4.5	3.0	1.5
Unreinforced masonry	1.75	1.25	1.0

[a] CP = Collapse Prevention, LS = Life Safety, IO = Immediate Occupancy.

CMU wall thickness, t =	7.625	in
Concrete wall thickness, t =	14	in
Concrete strength, f'_c =	4000	psi
Roof Story Base Shear, V_{roof} =	273.8	kips
1st Floor Story Base Shear, V_{1st} =	1131.4	kips
System Modification Factor, M_s =	3.75	(Interpolated between LS & CP)

Roof Level

Shear Wall in N-S Direction

Total length of exterior 8" CMU walls =	74.00	ft			
Grout spacing =	24	in			
Total length of interior 8" CMU walls =	49.42	ft			
Grout spacing =	32	in			
total net area of shear walls =	6997.1	in^2			
average shear stress, $v_{avg,NS}$ =	10.4	psi	*<*	*70.0*	*Shear Stress OK*
			DCR =	*0.15*	

Shear Wall in E-W Direction

Total length of exterior 8" CMU walls =	88.00	ft			
Grout spacing =	24	in			
Total length of interior 8" CMU walls =	50.00	ft			
Grout spacing =	32	in			
total net area of shear walls =	8280.8	in^2			
average shear stress, $v_{avg,NS}$ =	8.8	psi	*<*	*70.0*	*Shear Stress OK*
			DCR =	*0.13*	

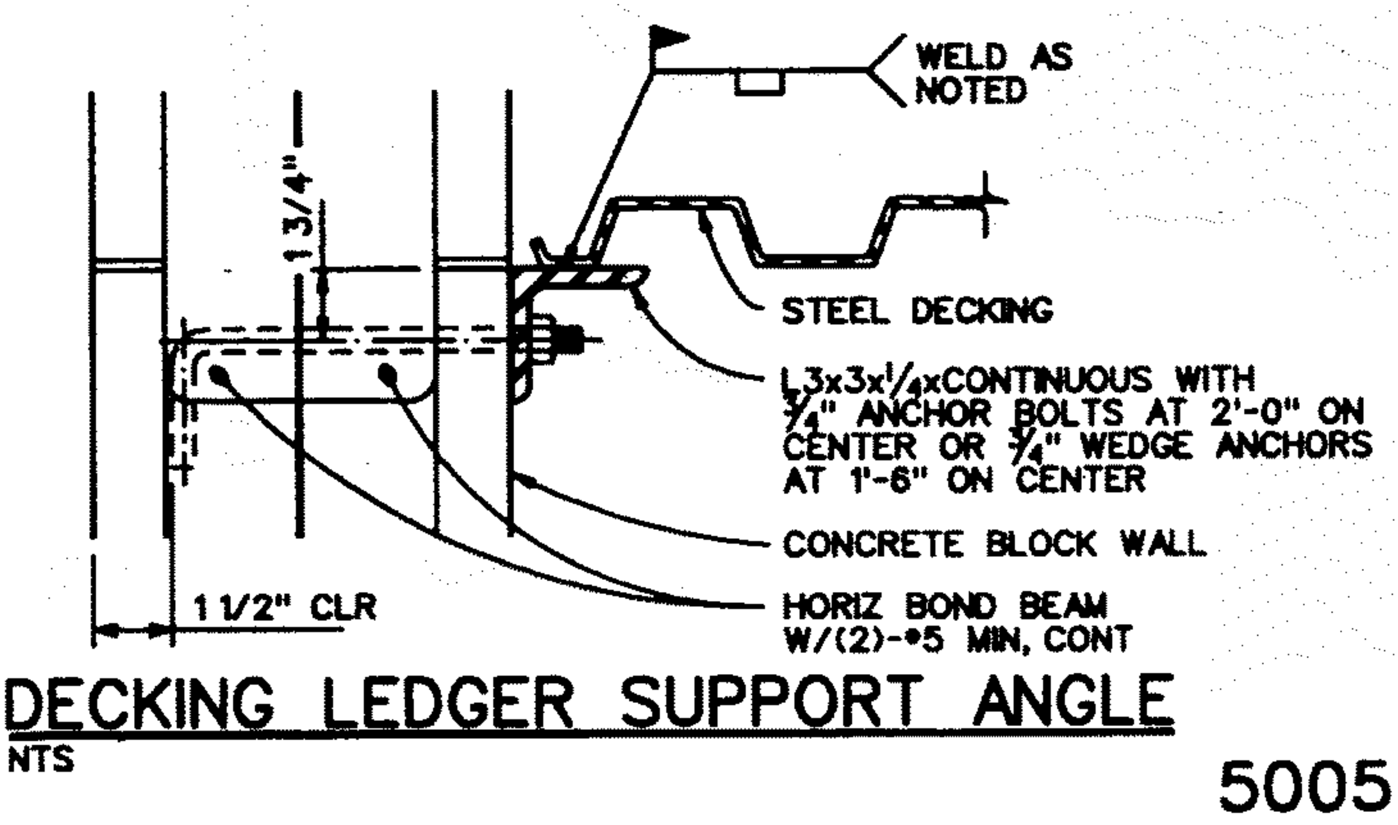
1st Level

Shear Wall in N-S Direction

Total length of 14" concrete walls =	118.00	ft			
total net area of shear walls =	19824.0	in^2			
average shear stress, $v_{avg,NS}$ =	15.2	psi	*<*	*126.5*	*Shear Stress OK*
			DCR =	*0.12*	

Shear Wall in E-W Direction

Total length of 14" concrete walls =	109.50	ft			
total net area of shear walls =	18396.0	in^2			
average shear stress, $v_{avg,NS}$ =	16.4	psi	*<*	*126.5*	*Shear Stress OK*
			DCR =	*0.13*	



WALL ANCHORAGE CONNECTION DETAIL ALONG NORTH AND SOUTH WALL ELEVATIONS

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BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Process Gallery	**JOB NO.**	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 1 Screening (BSE-2E Seismic Level)			

WALL ANCHORAGE FORCE

Process Gallery: Ledger Angle Anchorage into 8" CMU Wall along North and South Wall Elevations

4.4.3.7 Flexible Diaphragm Connection Forces. The horizontal seismic forces associated with the connection of a flexible diaphragm to either concrete or masonry walls, T_c, shall be calculated in accordance with Eq. (4-12).

$$T_c = \psi S_{XS} w_p A_p \tag{4-12}$$

where

w_p = Unit weight of the wall;
A_p = Area of wall tributary to the connection;
ψ = 1.0 for Collapse Prevention Performance Level, 1.3 for Life Safety Performance Level, and 1.8 for Immediate Occupancy Performance Level; and
S_{XS} = Value specified in Section 4.4.2.3.

wall height to diaphragm, h_w =	14.63	ft	
parapet height, h_p =	0.87	ft	
unit weight of wall, w_p =	58.50	psf	(partial grout for exterior walls [CMU + veneer])
Ψ =	1.15		(Interpolated between LS & CP)
S_{XS} =	0.744	g	
wall out-of-plane load =	409.7	lbs/ft	
anchor bolt spacing =	24.00	in	
wall anchorage force, T_c =	819.4	lbs	

Masonry & Steel Strength

anchor bolt size =	0.750	in	
anchor bolt embed, l_b =	6.00	in	
anchor bolt yield stress, f_y =	36.00	ksi	
masonry compressive strength, f'_m =	1500	psi	
projected area of anchor bolt in tension, A_{pt} =	113.10	in^2	
cross section area of anchor bolt, A_b =	0.44	in^2	
$\phi B_{anb} = 4 * A_{pt} * (f'_m)^{0.5}$ =	17521.0	lbs	masonry breakout tensile strength
$\phi B_{ans} = A_b * f_y$ =	15904.3	lbs	steel yielding strength
Masonry breakout strength DCR =	*0.05*	**OK**	
Steel yielding DCR =	*0.05*	**OK**	

Puddle Weld Shear Strength

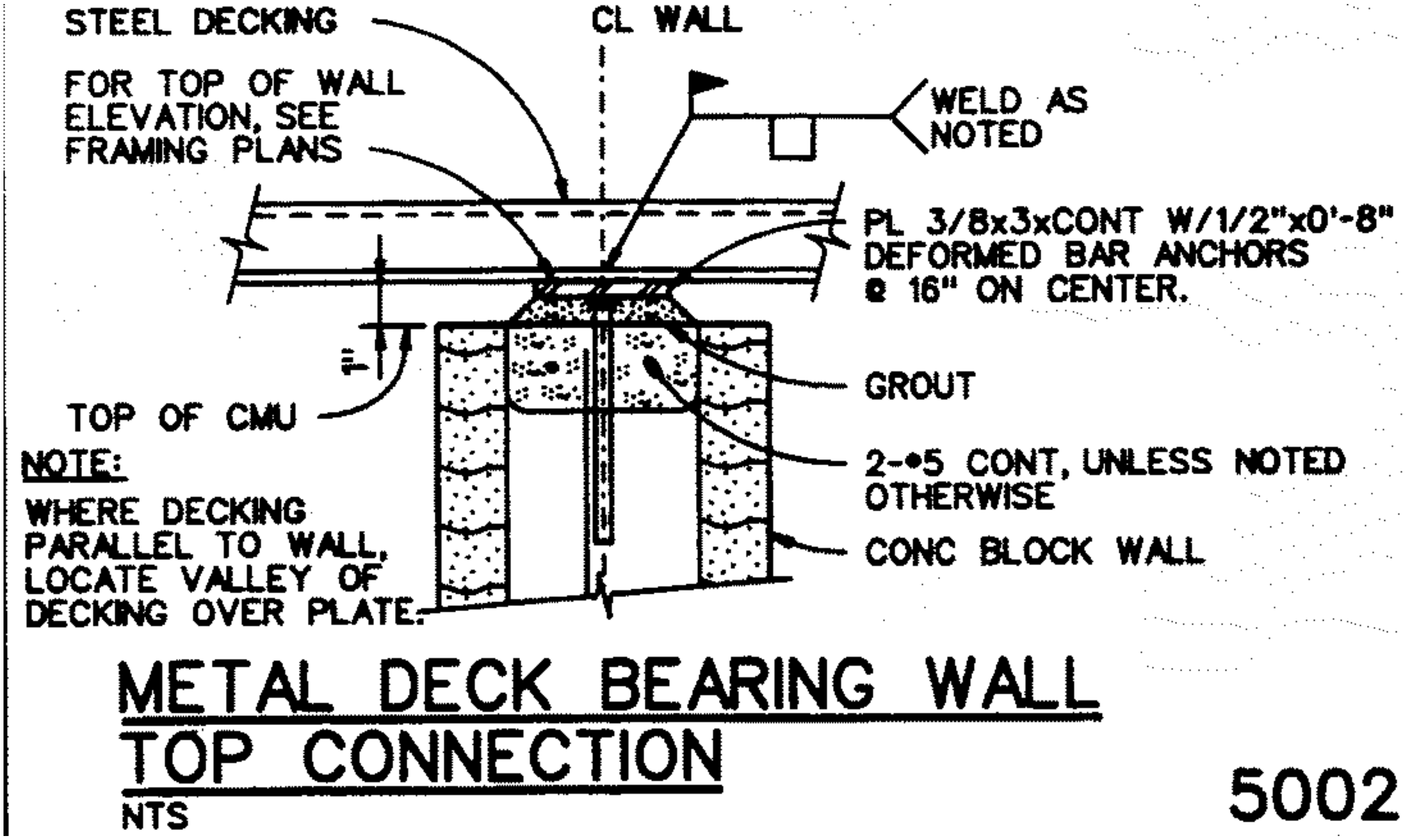
Table 4: Allowable Shear Strength (lbs/connection) for Arc Spot Welds, Arc Seam Welds, Hilti Fasteners, Pneutek Fasteners and SDI Recognized Screws for Verco Deck Panel Support Connections

Deck Gage	Profile	BMT	ARC SPOT WELD	ARC SEAM WELD	HILTI X-ENDK22 or X-HSN 24	HILTI X-ENP-19	PNEUTEK SDK61	PNEUTEK SDK63	PNEUTEK K64	PNEUTEK K66	SDI RECOGNIZED SCREWS
		(in.)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)
22	B & N	0.0299	703	1231	603	650	618	691	684	736	561
20	B & N	0.0359	1091	1491	720	776	733	791	886	903	673
18	B & N	0.0478	1850	2017	947	1020	951	967	1204	1253	896
16	B & N	0.0598	2309	2564	1169	1259	1158	1125	1474	1630	1121

Engineers... Working Wonders With Water ™

BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Process Gallery	**JOB NO.**	11962A.00
DESIGN TASK			ASCE 41-17 - Tier 1 Screening (BSE-2E Seismic Level)				

deck thickness =	0.0359	in	
weld spacing =	6.00	in	
load at weld =	204.8	lbs / weld	
allowable strength of screw from chart=	1091.0	lbs / weld	*ASCE 41-17 Section 9.10.1.3 allows for 2*
strength level of screw in shear =	2182.0	lbs / weld	*times allowable strength for strength level.*
Puddle weld strength DCR =	*0.09*	***OK***	



WALL ANCHORAGE CONNECTION DETAIL ALONG INTERIOR WALL ELEVATIONS

Carollo
Engineers... Working Wonders With Water ™

BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Process Gallery	**JOB NO.**	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 1 Screening (BSE-2E Seismic Level)			

WALL ANCHORAGE FORCE

Process Gallery: Bearing Anchorage into 8" Interior CMU Wall

4.4.3.7 Flexible Diaphragm Connection Forces. The horizontal seismic forces associated with the connection of a flexible diaphragm to either concrete or masonry walls, T_c, shall be calculated in accordance with Eq. (4-12).

$$T_c = \psi S_{XS} w_p A_p \qquad (4\text{-}12)$$

where

w_p = Unit weight of the wall;
A_p = Area of wall tributary to the connection;
ψ = 1.0 for Collapse Prevention Performance Level, 1.3 for Life Safety Performance Level, and 1.8 for Immediate Occupancy Performance Level; and
S_{XS} = Value specified in Section 4.4.2.3.

wall height to diaphragm, h_w =	14.63	ft	
unit weight of wall, w_p =	47.00	psf	(partial grout for interior walls)
Ψ =	1.15		(Interpolated between LS & CP)
S_{XS} =	0.744	g	
wall out-of-plane load =	294.2	lbs/ft	
anchor bolt spacing =	16.00	in	
wall anchorage force, T_c =	392.2	lbs	

Masonry & Steel Strength

anchor bolt size =	0.750	in
anchor bolt embed, l_b =	6.00	in
anchor bolt location from face, l_{be} =	3.81	in
anchor bolt yield stress, f_y =	36.00	ksi
masonry compressive strength, f_m =	1500	psi
projected area of anchor bolt in tension, A_{pt} =	113.10	in^2
projected area of each anchor bolt in shear, A_{pvbolt} =	22.80	in^2
cross section area of anchor bolt, A_b =	0.44	in^2

$\phi B_{vnb} = 4*A_{pv}*(f'_m)^{0.5}$ =	3532.4	lbs	masonry breakout shear strength
$\phi B_{vnc} = 1050*(f'_m*A_b)^{0.25}$ =	5327.4	lbs	masonry crushing shear strength
$\phi B_{vnpry} = 8*A_{pt}*(f'_m)^{0.5}$ =	35041.9	lbs	anchor pryout shear strength
$\phi B_{vns} = 0.60*A_b*f_y$ =	9542.6	lbs	steel yielding strength

Masonry breakout strength DCR =	*0.11*	***OK***
Masonry crushing strength DCR =	*0.07*	***OK***
Anchor pryout DCR =	*0.01*	***OK***
Steel yielding DCR =	*0.04*	***OK***

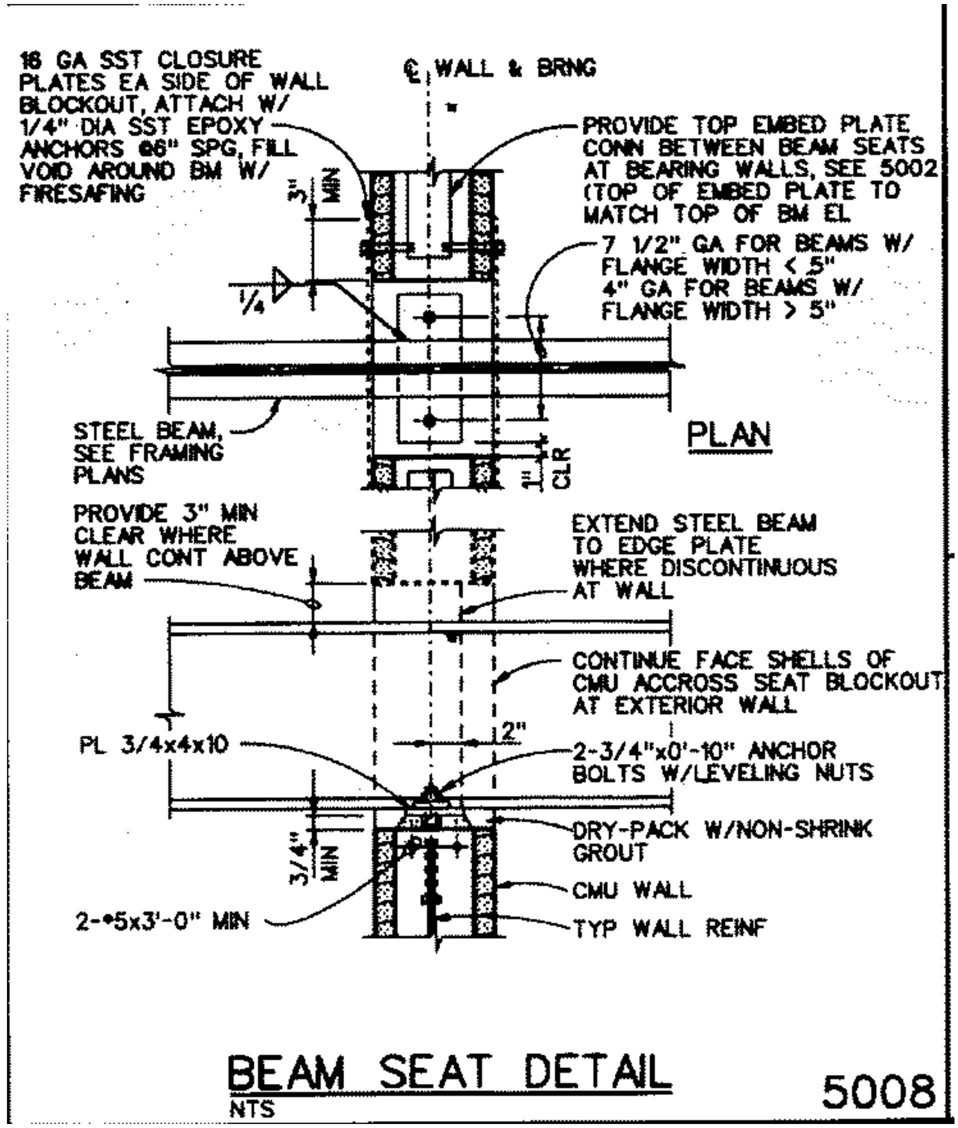
<u>Puddle Weld Shear Strength</u>

Table 4: Allowable Shear Strength (lbs/connection) for Arc Spot Welds, Arc Seam Welds, Hilti Fasteners, Pneutek Fasteners and SDI Recognized Screws for Verco Deck Panel Support Connections

Deck Gage	Profile	BMT	ARC SPOT WELD	ARC SEAM WELD	HILTI X-ENDK22 or X-HSN 24	HILTI X-ENP-19	PNEUTEK SDK61	PNEUTEK SDK63	PNEUTEK K64	PNEUTEK K66	SDI RECOGNIZED SCREWS
		(in.)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)
22	B & N	0.0299	763	1231	603	650	618	691	694	736	561
20	B & N	0.0359	1091	1491	720	775	733	791	886	903	673
18	B & N	0.0478	1850	2017	947	1020	951	987	1204	1253	896
16	B & N	0.0598	2309	2564	1169	1259	1158	1125	1474	1630	1121

deck thickness =	0.0359	in	
weld spacing =	6.00	in	
load at weld =	147.1	lbs / weld	
allowable strength of screw from chart=	1091.0	lbs / weld	*ASCE 41-17 Section 9.10.1.3 allows for 2*
strength level of screw in shear =	2182.0	lbs / weld	*times allowable strength for strength level.*

Puddle weld strength DCR = *0.07* ***OK***



BEAM ANCHORAGE CONNECTION DETAIL ALONG EAST AND WEST WALL ELEVATIONS

Engineers... Working Wonders With Water ®

BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Process Gallery	**JOB NO.**	11962A.00
DESIGN TASK			ASCE 41-17 - Tier 1 Screening (BSE-2E Seismic Level)				

WALL ANCHORAGE FORCE

Process Gallery: Beam Anchorage into 8" CMU Wall along East and West Wall Elevations

4.4.3.7 Flexible Diaphragm Connection Forces. The horizontal seismic forces associated with the connection of a flexible diaphragm to either concrete or masonry walls, T_c, shall be calculated in accordance with Eq. (4-12).

$$T_c = \psi S_{XS} w_p A_p \tag{4-12}$$

where

w_p = Unit weight of the wall;
A_p = Area of wall tributary to the connection;
ψ = 1.0 for Collapse Prevention Performance Level, 1.3 for Life Safety Performance Level, and 1.8 for Immediate Occupancy Performance Level; and
S_{XS} = Value specified in Section 4.4.2.3.

wall height to diaphragm, h_w =	14.63	ft	
parapet height, h_p =	0.87	ft	
unit weight of wall, w_p =	58.50	psf	(partial grout for exterior walls [CMU + veneer])
Ψ =	1.15		(Interpolated between LS & CP)
S_{XS} =	0.744	g	
wall out-of-plane load =	409.7	lbs/ft	
beam spacing =	6.67	ft	
wall anchorage force, T_c =	2732.6	lbs	

Masonry & Steel Strength

anchor bolt size =	0.750	in
anchor bolt embed, l_b =	8.00	in
anchor bolt location from face, l_{be} =	3.81	in
anchor bolt yield stress, f_y =	36.00	ksi
masonry compressive strength, f'_m =	1500	psi
projected area of single anchor bolt in tension, A_{pt} =	201.06	in^2
projected area of single anchor bolt in shear, A_{pvbolt} =	22.80	in^2
cross section area of single anchor bolt, A_b =	0.44	in^2
estimated overlap of projected area, $A_{ptoverlap}$ =	2.50	in^2
net projected area of anchor bolt in tension, A_{ptnet} =	400.87	in^2
estimated overlap of projected area, $A_{pvoverlap}$ =	1.25	in^2
net projected area of anchor bolt in shear, A_{pvnet} =	44.98	in^2

$\phi B_{vnb} = 4*A_{pvnet}*(f'_m)^{0.5}$ =	6968.1	lbs	group masonry breakout shear strength
$\phi B_{vnc} = 1050*(f'_m*A_b)^{0.25}$ =	10654.8	lbs	group masonry crushing shear strength
$\phi B_{vnpry} = 8*A_{ptnet}*(f'_m)^{0.5}$ =	124206.2	lbs	group anchor pryout shear strength
$\phi B_{vns} = 0.60*A_b*f_y$ =	19085.2	lbs	group steel yielding strength

Masonry breakout strength DCR =	*0.39*	***OK***
Masonry crushing strength DCR =	*0.26*	***OK***
Anchor pryout DCR =	*0.02*	***OK***
Steel yielding DCR =	*0.14*	***OK***

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Process Gallery
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	07/06/21
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Pro
Project Number 11962A.00

Table 17-3. Immediate Occupancy Basic Configuration Checklist

Very Low Seismicity

CSZ Seismic Level at Damage Control

Structural Components

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☒	N/A ☐	U ☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	There is a roof beam aligned with an interior shear wall but no connection to the wall for seismic transfer. The diaphragm at this location will lack the ability to transfer diaphragm forces into the shear wall.
C ☒	NC ☐	N/A ☐	U ☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	- Ledger anchorage steel yielding DCR = 0.04 (OK) - Interior wall bearing anchorage masonry breakout strength DCR = 0.09 (OK) - Beam anchorage masonry breakout strength DCR = 0.32 (OK)

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Very Low Seismicity

Building System

General

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☒	N/A ☐	U ☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	There is a roof beam aligned with an interior shear wall but no connection to the wall for seismic transfer. The diaphragm at this location will lack the ability to transfer diaphragm forces into the shear wall.
C ☐	NC ☐	N/A ☒	U ☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement need not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	
C ☐	NC ☐	N/A ☒	U ☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Pro
Project Number 11962A.00

Building Configuration

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction shall not be less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
C ☒	NC ☐	N/A ☐	U ☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story shall not be less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
C ☐	NC ☒	N/A ☐	U ☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	The center interior CMU shear walls don't continue down into the basement level. These walls are supported by concrete beams.
C ☒	NC ☐	N/A ☐	U ☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Pro

Project Number 11962A.00

C	NC	N/A	U		
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
C	NC	N/A	U		
☐	☐	☒	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	Building roof is considered flexible and check is not required.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Pro
Project Number 11962A.00

Low Seismicity

Geologic Site Hazards

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)	Liquefaction has been determined to not be an issue per NGI technical memorandum.
C ☐	NC ☐	N/A ☒	U ☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)	There are no slopes nearby structure.
C ☒	NC ☐	N/A ☐	U ☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)	Surface fault rupture has been determined to not be an issue per NGI technical memorandum.

Moderate and High Seismicity

Foundation Configuration

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)	Height = 19.50 ft Length = 56.42 ft Sa = 0.446 L/H = 56.42 / 19.50 = 2.89 0.6*Sa = 0.6 * 0.446 = 0.27 2.89 > 0.27 (OK)

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name ______________

Project Number ______________

C	NC	N/A	U		
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Process Gallery
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	07/07/21
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Pro
Project Number 11962A.00

Table 17-35. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2

Very Low Seismicity

Seismic-Force-Resisting System

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
C ☒	NC ☐	N/A ☐	U ☐	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than 70 lb/in.^{2}. (Commentary: Sec. A.3.2.4.1. Tier 2: Sec. 5.5.3.1.1)	Roof Level N-S direction DCR = 0.15 (OK) E-W direction DCR = 0.13 (OK) 1st Floor N-S direction DCR = 0.12 (OK) E-W direction DCR = 0.13 (OK)
C ☐	NC ☒	N/A ☐	U ☐	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in., and all vertical bars extend to the top of the walls. (Commentary: Sec. A.3.2.4.2. Tier 2: Sec. 5.5.3.1.3)	Horiz steel = #6@48" Vert steel = #6@24" (ext) & #6@32" (int) Horiz ratio = 0.44 / (7.625*48) = 0.0012 > 0.0007 (OK) Vert ratio = 0.44 / (7.625*24) = 0.0024 > 0.0007 (OK) 0.44 / (7.625*32) = 0.0018 > 0.0007 (OK) Horizontal reinforcing is specified at 48" but this is not less than 48in required. Reinforcing is non-compliant.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Connections

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	
C ☐	NC ☒	N/A ☐	U ☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)	Ledger Connection: Anchorage connection DCR = 0.48 (OK) Deck weld connection DCR = 0.61 (OK) Collector Beam: Anchorage connection DCR = 5.51 (NG)
C ☒	NC ☐	N/A ☐	U ☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	CMU wall dowel DCR = 0.40 (OK) Concrete wall dowel DCR = 0.34 (OK)
C ☐	NC ☐	N/A ☒	U ☐	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	- Ledger anchorage steel yielding DCR = 0.04 (OK) - Interior wall bearing anchorage masonry breakout strength DCR = 0.09 (OK) - Beam anchorage masonry breakout strength DCR = 0.32 (OK)

Stiff Diaphragms

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2: Sec. 5.7.2)	

Foundation System

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	DEEP FOUNDATIONS: Piles and piers are capable of transferring the seismic forces between the structure and the soil. (Commentary: Sec. A.6.2.3.)	Structure is not located on a deep foundation system.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story high. (Commentary: Sec. A.6.2.4)	

Low, Moderate, and High Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	DESCRIPTION	COMMENTS
☒	☐	☐	☐	REINFORCING AT WALL OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides. (Commentary: Sec. A.3.2.4.3. Tier 2: Sec. 5.5.3.1.5)	
☒	☐	☐	☐	PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than 30. (Commentary: Sec. A.3.2.4.4. Tier 2: Sec. 5.5.3.1.2)	Height = 14.63 ft Thickness = 7.625 in H/t = 14.63 * 12 / 7.625 = 23.0 < 30 (OK)

Diaphragms (Flexible or Stiff)

C	NC	N/A	U	DESCRIPTION	COMMENTS
☐	☐	☒	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 4 ft long. (Commentary: A.4.1.6. Tier 2: Sec. 5.6.1.3)	
☐	☐	☒	☐	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities. (Commentary: Sec. A.4.1.7. Tier 2: Sec. 5.6.1.4)	
☐	☐	☒	☐	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. (Commentary: Sec. A.4.1.8. Tier 2: Sec. 5.6.1.5)	

Flexible Diaphragms

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Item	Comments
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 12 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft and aspect ratios less than or equal to 3-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☒	☐	☐	NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft and have aspect ratios less than 4-to-1. (Commentary: Sec. A.4.3.1. Tier 2: Sec. 5.6.3)	Span 1 = 24ft x 21.33ft Span 2 = 32ft x 21.33ft Span 3 = 56.67ft x 30ft (NG). Span exceeds 40ft requirement. Span 1 ratio = 24/21.33 = 1.13 < 4 (OK) Span 2 ratio = 32/21.33 = 1.50 < 4 (OK) Span 3 ratio = 56.67/30 = 1.89 < 4 (OK)

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	DESCRIPTION	COMMENTS
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

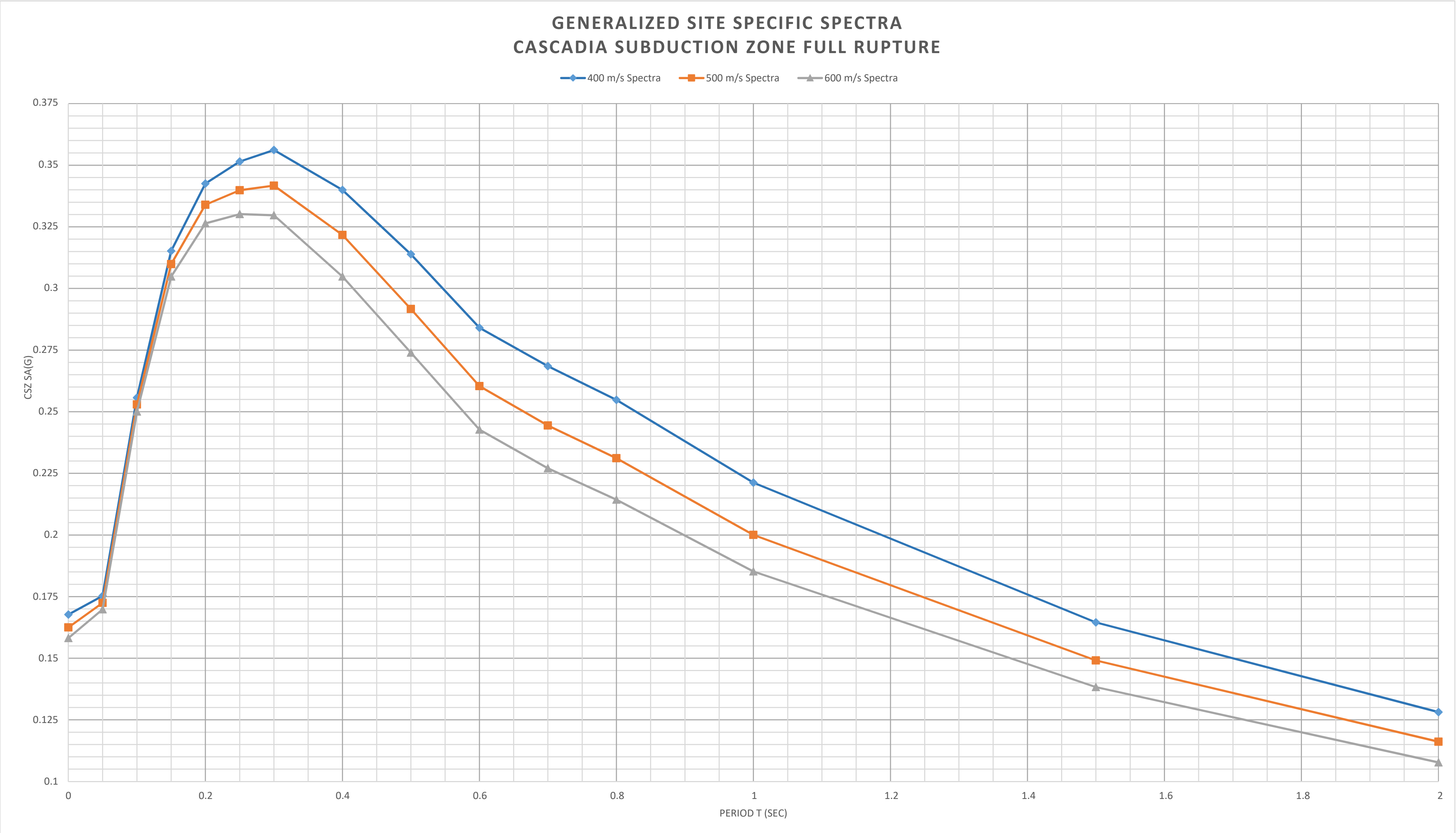
Connections

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

City of Wilsonville

Process Gallery Building Tier 1 Structural Calculations



Northwest Geotech, Inc.

Figure No. 3

Ss @ T=0.20 sec

Table 2: CSZ Generalized Response Spectra Ordinates					
Latitude 45.295155 degrees Longitude -122.771810 degrees					
Vs30 = 400 m/s		Vs30 = 500 m/s		Vs30 = 600 m/s	
Period T(sec)	CSZ Sa(g)	Period T (sec)	CSZ Sa (g)	Period T (sec)	CSZ Sa (g)
0	0.168	0	0.163	0	0.158
0.05	0.175	0.05	0.172	0.05	0.170
0.1	0.256	0.1	0.253	0.1	0.250
0.15	0.315	0.15	0.310	0.15	0.305
0.2	0.343	0.2	0.334	0.2	0.326
0.25	0.352	0.25	0.340	0.25	0.330
0.3	0.356	0.3	0.342	0.3	0.330
0.4	0.340	0.4	0.322	0.4	0.305
0.5	0.314	0.5	0.292	0.5	0.274
0.6	0.284	0.6	0.260	0.6	0.243
0.7	0.269	0.7	0.244	0.7	0.227
0.8	0.255	0.8	0.231	0.8	0.214
1	0.221	1	0.200	1	0.185
1.5	0.165	1.5	0.149	1.5	0.138
2	0.128	2	0.116	2	0.108
2.5	0.104	2.5	0.094	2.5	0.087
3	0.085	3	0.077	3	0.071

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BY: BS	DATE Jul-21	CLIENT City of Wilsonville	SHEET
CHKD BY	DESCRIPTION Process Gallery Building		JOB NO. 11962A.00
DESIGN TASK	Process Gallery Building Seismic Weight		

Roof Loads

Roof EL 125.63

Description	Load		
1-1/2"x20ga metal deck	2.3	psf	
Rigid insulation w/ sheet roofing	4.5		
Steel beam	3.5		
Miscellaneous	5.0		
Dead Load for Gravity Design	**15.3**	psf	
Roof Live Load	**20.0**	psf	(Assumed)
Snow Load	**25.0**	psf	

Notes

1. Roof beam self weight assumed total beam weight, 9831.0 lb, divided by total roof area, 3093.1 ft ^{2} which is 9831.0lb/3093.1ft^2 = 3.18 lb/ft^2. Assume 3.5 psf.

Floor Loads

Floor EL 111.00

Description	Load	
8" concrete slab	100.0	psf
Concrete beam	73.0	
Miscellaneous	10.0	
Dead Load for Gravity Design	**183.0**	psf
Floor Live Load	**200.0**	psf

Notes

1. Floor beam self weight assumed total beam weight, 226925.0 lb, divided by total floor area, 3093.1 ft ^{2} which is 226925.0lb/3093.1ft^2 = 73.4 lb/ft^2. Assume 73.5 psf.

Wall Loads

Wall Loads

Description	Load	
8" CMU wall (partial grouted @ 24")	51.0	psf
8" CMU wall (partial grouted @ 32")	47.0	
8" Concrete wall	100.0	
14" Concrete wall	175.0	
Exterior Plastic Veneer Finish	7.5	
8" Exterior CMU Wall Load for Seismic	**58.5**	psf
8" Interior CMU Wall Load for Seismic	**47.0**	psf
8" Concrete Wall Load for Seismic	**100.0**	psf
14" Concrete Wall Load for Seismic	**175.0**	psf

Seismic Weight

Roof Weight

Roof Area	3093.1	ft^2
Roof Seismic Weight	**47.3**	**kip**

Floor Weight

Dry Chemical Storage Area	3093.1	ft^2
Floor Seismic Weight	**566.0**	**kip**

Wall Weight

Wall Height to Roof	14.63	ft
Wall Height to 2nd Level	18.00	ft
Parapet Height	0.87	ft
8" Exterior CMU Wall Length (1st floor)	220.00	ft
8" Interior CMU Wall Length (1st floor)	108.00	ft
8" Concrete Wall Length (basement)	37.42	ft
14" Concrete Wall Length (basement)	220.00	ft
Roof Wall Seismic Weight	**142.5**	**kip**
Basement Wall Seismic Weight	**511.5**	**kip**
Combined Roof Seismic Weight	**189.8**	**kip**
Combined Base Level Seismic Weight	**1077.5**	**kip**

Total Seismic Weight **1267.3 kip**

Notes

1. Wall seismic weight assumes half of the wall height associated with each level.

2. Per Section 4.5.2.1, the effective seismic weight includes the total dead load, 25% of the live load when area is used as storage, and 20% of the roof snow live load if greater than 30 psf (otherwise assume zero).

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BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Process Gallery	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)		

SEISMIC BASE SHEAR

4.4.2.1 Pseudo Seismic Force. The pseudo seismic force, in a given horizontal direction of a building, shall be calculated in accordance with Eq. (4-1).

$$V = CS_aW \quad (4\text{-}1)$$

where

V = Pseudo seismic force;
C = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response; C shall be taken from Table 4-7;
S_a = Response spectral acceleration at the fundamental period of the building in the direction under consideration. The value of S_a shall be calculated in accordance with the procedures in Section 4.4.2.3; and
W = Effective seismic weight of the building, including the total dead load and applicable portions of other gravity loads listed below:

Table 4-7. Modification Factor, *C*

	Number of Stories			
Building Type[a]	1	2	3	≥4
Wood and cold-formed steel shear wall (W1, W1a, W2, CFS1) Moment frame (S1, S3, C1, PC2a)	1.3	1.1	1.0	1.0
Shear wall (S4, S5, C2, C3, PC1a, PC2, RM2, URMa) Braced frame (S2) Cold-formed steel strap-brace wall (CFS2)	1.4	1.2	1.1	1.0
Unreinforced masonry (URM) Flexible diaphragms (S1a, S2a, S5a, C2a, C3a, PC1, RM1)	1.0	1.0	1.0	1.0

[a] Defined in Table 3-1.

Process Gallery

Modification Factor, C = 1.2
S_s = 0.343 (CSZ spectral response)
S_1 = 0.221 (CSZ spectral response)
F_a = 1.3 (Site amplication factor per ASCE 7-16)
F_v = 1.5 (Site amplication factor per ASCE 7-16)
$S_{X1} = S_1*F_v$ = 0.332 (CSZ seismic hazard)
T = 0.149 s
$S_{XS} = S_s*F_a$ = 0.446 (CSZ seismic hazard)
Spectral Acceleration, S_a = 0.446
Seismic Weight, W = 1267.3 kip

Seismic Force, V = 678.1 kip

Story	Weight, w_x (kip)	Floor Height, h_x (ft)	k factor	$w_xh_x^k$ (kip*ft^2)	C_{vx}	Force on Level, F_x (kip)	Story Force, V_j (kip)
Roof	189.8	32.63	1.0	6193.2	0.242	164.1	164.1
1st	1077.5	18.00	1.0	19395.0	0.758	514.0	678.1

$\Sigma w_xh_x^k$ = 25588.2

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BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Process Gallery	**JOB NO.**	11962A.00
DESIGN TASK		ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)					

WALL SHEAR STRESS CHECK

4.4.3.3 Shear Stress in Shear Walls. The average shear stress in shear walls, v_j^{avg}, shall be calculated in accordance with Eq. (4-8).

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \quad (4\text{-}8)$$

where

V_j = Story shear at level j computed in accordance with Section 4.4.2.2;

A_w = Summation of the horizontal cross-sectional area of all shear walls in the direction of loading. Openings shall be taken into consideration where computing A_w. For masonry walls, the net area shall be used. For wood-framed walls, the length shall be used rather than the area; and

M_s = System modification factor; M_s shall be taken from Table 4-8.

Table 4-8. M_s Factors for Shear Walls

	Level of Performance		
Wall Type	**CP[a]**	**LS[a]**	**IO[a]**
Reinforced concrete, precast concrete, wood, reinforced masonry, and cold-formed steel	4.5	3.0	1.5
Unreinforced masonry	1.75	1.25	1.0

[a] CP = Collapse Prevention, LS = Life Safety, IO = Immediate Occupancy.

CMU wall thickness, t =	7.625	in	
Concrete wall thickness, t =	14	in	
Concrete strength, f'_c =	4000	psi	
Roof Story Base Shear, V_{roof} =	164.1	kips	
1st Floor Story Base Shear, V_{1st} =	678.1	kips	
System Modification Factor, M_s =	2.25		(Interpolated between LS & IO)

Roof Level

Shear Wall in N-S Direction

Total length of exterior 8" CMU walls =	74.00	ft			
Grout spacing =	24	in			
Total length of interior 8" CMU walls =	49.42	ft			
Grout spacing =	32	in			
total net area of shear walls =	6997.1	in^2			
average shear stress, $v_{avg,NS}$ =	10.4	psi	<	70.0	Shear Stress OK
			DCR =	*0.15*	

Shear Wall in E-W Direction

Total length of exterior 8" CMU walls =	88.00	ft			
Grout spacing =	24	in			
Total length of interior 8" CMU walls =	50.00	ft			
Grout spacing =	32	in			
total net area of shear walls =	8280.8	in^2			
average shear stress, $v_{avg,NS}$ =	8.8	psi	<	70.0	Shear Stress OK
			DCR =	*0.13*	

1st Level

Shear Wall in N-S Direction

Total length of 14" concrete walls =	118.00	ft			
total net area of shear walls =	19824.0	in^2			
average shear stress, $v_{avg,NS}$ =	15.2	psi	<	126.5	Shear Stress OK
			DCR =	*0.12*	

Shear Wall in E-W Direction

Total length of 14" concrete walls =	109.50	ft			
total net area of shear walls =	18396.0	in^2			
average shear stress, $v_{avg,NS}$ =	16.4	psi	<	126.5	Shear Stress OK
			DCR =	*0.13*	

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BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Process Gallery	**JOB NO.**	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)			

TRANSFER TO SHEAR WALLS

Beam Connection to CMU Walls (Detail 5008)

diaphragm shear strength, q_{ult} =	1280	lbs/ft	(assumed less than wall shear strength)
beam length =	30	ft	
diaphragm shear strength =	38400.0	lbs	

W16x26 Beam Tensile Strength (Assuming ϕ = 1.0 for Tier 1)

beam area, A =	7.68	in^2	
steel yield stress, F_y =	50	ksi	
steel tensile stress, F_u =	65	ksi	
$\phi B_t = \min(F_y{*}A, F_u{*}A)$ =	384.0	kip	
Masonry breakout strength DCR =	*0.10*	***OK***	

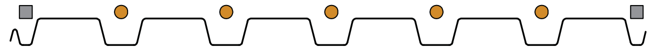
Masonry & Steel Strength (Assuming ϕ = 1.0 for Tier 1)

anchor bolt size =	0.750	in	
anchor bolt embed, l_b =	8.00	in	
anchor bolt location from face, l_{be} =	3.81	in	
anchor bolt yield stress, f_y =	36.00	ksi	
masonry compressive strength, f_m =	1500	psi	
projected area of single anchor bolt in tension, A_{pt} =	201.06	in^2	
projected area of single anchor bolt in shear, A_{pvbolt} =	22.80	in^2	
cross section area of single anchor bolt, A_b =	0.44	in^2	
estimated overlap of projected area, $A_{ptoverlap}$ =	2.50	in^2	
net projected area of anchor bolt in tension, A_{ptnet} =	400.87	in^2	
estimated overlap of projected area, $A_{pvoverlap}$ =	1.25	in^2	
net projected area of anchor bolt in shear, A_{pvnet} =	44.98	in^2	
$\phi B_{vnb} = 4{*}A_{pvnet}{*}(f'_m)^{0.5}$ =	6968.1	lbs	group masonry breakout shear strength
$\phi B_{vnc} = 1050{*}(f'_m{*}A_b)^{0.25}$ =	10654.8	lbs	group masonry crushing shear strength
$\phi B_{vnpry} = 8{*}A_{ptnet}{*}(f'_m)^{0.5}$ =	124206.2	lbs	group anchor pryout shear strength
$\phi B_{vns} = 0.60{*}A_b{*}f_y$ =	19085.2	lbs	group steel yielding strength

Masonry breakout strength DCR =	*5.51*	***NG***
Masonry crushing strength DCR =	*3.60*	***NG***
Anchor pryout DCR =	*0.31*	***OK***
Steel yielding DCR =	*2.01*	***NG***

Type HSB®-36

- **36/7 Weld Pattern at Supports**
- **Sidelaps connected with Button Punch or 1½" Top Seam Weld**



Allowable Diaphragm Shear Strength, q (plf) and Flexibility Factors, F ((in./lb)x10^6)

DECK GAGE	SIDELAP ATTACHMENT		SPAN (ft-in.)								
			4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
22	BP @ 24"	q	398	327	267	235	202	186	168		
		F	9.9+27R	11.9+20R	14+15R	15.7+12R	17.6+9R	19.1+7R	20.9+5R		
	BP @ 12"	q	434	355	303	266	238	218	204		
		F	9.5+27R	11.4+21R	13.1+16R	14.6+13R	16.1+10R	17.4+8R	18.7+6R		
	TSW @ 24"	q	770	777	661	682	601	627	566		
		F	5.1+30R	5.3+24R	6.1+20R	6.1+17R	6.6+15R	6.5+13R	7+12R		
	TSW @ 18"	q	937	911	781	785	787	710	720		
		F	4.4+30R	4.8+24R	5.4+20R	5.5+17R	5.6+15R	6+13R	6.1+12R		
	TSW @ 12"	q	1084	1033	998	972	952	936	924		
		F	4+30R	4.4+24R	4.7+20R	4.9+17R	5.1+15R	5.2+13R	5.3+12R		
	TSW @ 6"	q	1504	1476	1456	1442	1430	1236	1001		
		F	3.2+30R	3.6+24R	3.8+20R	4+17R	4.2+15R	4.3+14R	4.4+12R		
20	BP @ 24"	q	564	466	383	338	292	267	240	228	209
		F	9.3+16R	11+12R	12.8+8R	14.3+6R	16+4R	17.3+2R	19+1R	20.1+0R	21.7-1R
	BP @ 12"	q	616	507	434	382	343	313	292	274	260
		F	8.8+16R	10.5+12R	12+9R	13.3+7R	14.6+5R	15.8+3R	16.9+2R	18+1R	19+0R
	TSW @ 24"	q	1024	1026	873	896	793	821	742	772	711
		F	4.8+19R	4.9+15R	5.6+12R	5.5+11R	6+9R	5.8+8R	6.2+7R	6.1+7R	6.4+6R
	TSW @ 18"	q	1236	1197	1026	1027	1028	926	937	946	875
		F	4.2+19R	4.4+15R	5+13R	5+11R	5.1+9R	5.4+8R	5.4+7R	5.4+7R	5.7+6R
	TSW @ 12"	q	1425	1354	1304	1267	1239	1217	1200	1085	912
		F	3.8+19R	4.1+15R	4.3+13R	4.5+11R	4.6+10R	4.7+8R	4.7+8R	4.8+7R	4.8+6R
	TSW @ 6"	q	1970	1930	1901	1880	1864	1621	1313	1085	912
		F	3.1+19R	3.4+15R	3.5+13R	3.7+11R	3.7+10R	3.8+9R	3.9+8R	3.9+7R	4+6R
18	BP @ 24"	q	979	812	670	595	516	475	423	400	366
		F	8.1+7R	9.5+4R	11+2R	12.2+1R	13.7+0R	14.8-1R	16.2-2R	17.2-3R	18.6-4R
	BP @ 12"	q	1070	885	761	673	607	556	515	483	458
		F	7.7+7R	9+5R	10.3+3R	11.4+2R	12.5+1R	13.5+0R	14.4-1R	15.3-2R	16.2-2R
	TSW @ 24"	q	1617	1598	1359	1383	1224	1258	1141	1177	1085
		F	4.3+9R	4.2+7R	4.7+6R	4.6+5R	5+4R	4.9+4R	5.2+3R	5+3R	5.3+3R
	TSW @ 18"	q	1928	1851	1586	1577	1570	1414	1425	1434	1326
		F	3.7+9R	3.8+7R	4.2+6R	4.2+5R	4.2+4R	4.5+4R	4.5+4R	4.4+3R	4.7+3R
	TSW @ 12"	q	2208	2084	1998	1935	1886	1848	1817	1659	1394
		F	3.4+9R	3.5+7R	3.6+6R	3.7+5R	3.8+5R	3.8+4R	3.9+4R	3.9+3R	3.9+3R
	TSW @ 6"	q	3036	2962	2910	2872	2842	2478	2007	1659	1394
		F	2.7+9R	2.9+8R	3+6R	3+5R	3.1+5R	3.1+4R	3.1+4R	3.2+3R	3.2+3R
16	BP @ 24"	q	1255	1052	869	780	677	629	561	532	485
		F	7.2+3R	8.4+1R	9.7+0R	10.8-1R	12.1-2R	13.1-2R	14.3-3R	15.2-4R	16.4-4R
	BP @ 12"	q	1395	1167	1013	902	820	756	704	662	628
		F	6.8+3R	8+2R	9.1+1R	10.1+0R	11-1R	11.9-2R	12.7-2R	13.5-3R	14.3-3R
	TSW @ 24"	q	2083	2073	1766	1805	1599	1649	1497	1548	1428
		F	3.7+5R	3.7+4R	4.1+3R	4+3R	4.3+2R	4.2+2R	4.5+2R	4.3+2R	4.6+1R
	TSW @ 18"	q	2496	2408	2067	2062	2058	1856	1874	1889	1748
		F	3.2+5R	3.3+4R	3.7+3R	3.6+3R	3.6+2R	3.9+2R	3.8+2R	3.8+2R	4+2R
	TSW @ 12"	q	2862	2713	2609	2532	2473	2427	2389	2310	1941
		F	2.9+5R	3.1+4R	3.1+3R	3.2+3R	3.2+3R	3.3+2R	3.3+2R	3.3+2R	3.4+2R
	TSW @ 6"	q	3918	3833	3773	3729	3695	3451	2795	2310	1941
		F	2.4+5R	2.5+4R	2.5+4R	2.6+3R	2.6+3R	2.6+2R	2.6+2R	2.7+2R	2.7+2R

See footnotes on page 28.

Deck span = 6'-8"
q = 1280 lb/ft (interpolated)

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BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Process Gallery	**JOB NO.**	11962A.00
DESIGN TASK			ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)				

TRANSFER TO SHEAR WALLS

Ledger Angle Connection into CMU Walls

diaphragm shear strength, q_{ult} =	1280	lbs/ft	(assumed less than wall shear strength)
anchor bolt spacing =	24	in	
diaphragm shear strength =	2560.0	lbs	

Masonry & Steel Strength (Assuming ϕ = 1.0 for Tier 1)

anchor bolt size =	0.750	in	
anchor bolt embed, l_b =	6.00	in	
anchor bolt location from face, l_{be} =	13.50	in	
anchor bolt yield stress, f_y =	36.00	ksi	
masonry compressive strength, f_m =	1500	psi	
projected area of anchor bolt in tension, A_{pt} =	113.10	in^2	
projected area of each anchor bolt in shear, A_{pv} =	286.28	in^2	
cross section area of anchor bolt, A_b =	0.44	in^2	
$\phi B_{vnb} = 4*A_{pv}*(f'_m)^{0.5}$ =	44349.9	lbs	masonry breakout shear strength
$B_{vnc} = 1050*(f'_m*A_b)^{0.25}$ =	5327.4	lbs	masonry crushing shear strength
$B_{vnpry} = 8*A_{pt}*(f'_m)^{0.5}$ =	35041.9	lbs	anchor pryout shear strength
$B_{vns} = 0.60*A_b*f_y$ =	9542.6	lbs	steel yielding strength

Masonry breakout strength DCR =	*0.06*	***OK***
Masonry crushing strength DCR =	*0.48*	***OK***
Anchor pryout DCR =	*0.07*	***OK***
Steel yielding DCR =	*0.27*	***OK***

Puddle Weld Strength

deck thickness = 0.0359 in

N-S Wall Elevations - Deck welded to support with puddle weld at 6"

effective puddle weld diameter =	0.625	in
puddle weld spacing =	6.00	in
load at puddle weld =	640.0	lbs /weld
strength of puddle weld =	2093.7	lbs /weld

Puddle weld strength DCR = *0.31* ***OK***

E-W Wall Elevations - Deck welded to support with puddle weld at 12"

effective puddle weld diameter =	0.625	in
puddle weld spacing =	12.00	in
load at puddle weld =	1280.0	lbs /weld
strength of puddle weld =	2093.7	lbs /weld

Puddle weld strength DCR = *0.61* ***OK***

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BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Process Gallery	JOB NO.	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)			

FOUNDATION DOWELS

CMU Wall Shear Strength

steel yield strength, f_y =	60000	psi	
Seismic unit shear, Vu =	1.54	kip/ft	
Seismic unit moment, Mu =	22.5	ft*kip/ft	
unit depth, dv =	12.00	in	
Mu/(Vu*dv) =	14.62		
Wall area, A_{nv} =	91.5	in^2	
masonry strength, f'_m =	1500	psi	
Reinforcement area, A_v =	0.44	in^2	
reinforcement spacing, s =	32.0	in	
Nominal reinforcement shear strength, V_{ns} =	4.95	kip	
Υ_g =	0.75		
Nominal Unit Wall Shear, V_n =	10.63	kip/ft	ACI 530-13 Eq. 9-23

Shear Friction between wall and slab

Dowels into concrete walls below are #6@32'

Reinforcement area, A_{vf} =	0.44	in^2	
μ =	1.0		
Unit Shear Friction, V_n =	26.40	kip/ft	
Dowel DCR =	*0.40*	***Dowels can develop wall strength***	

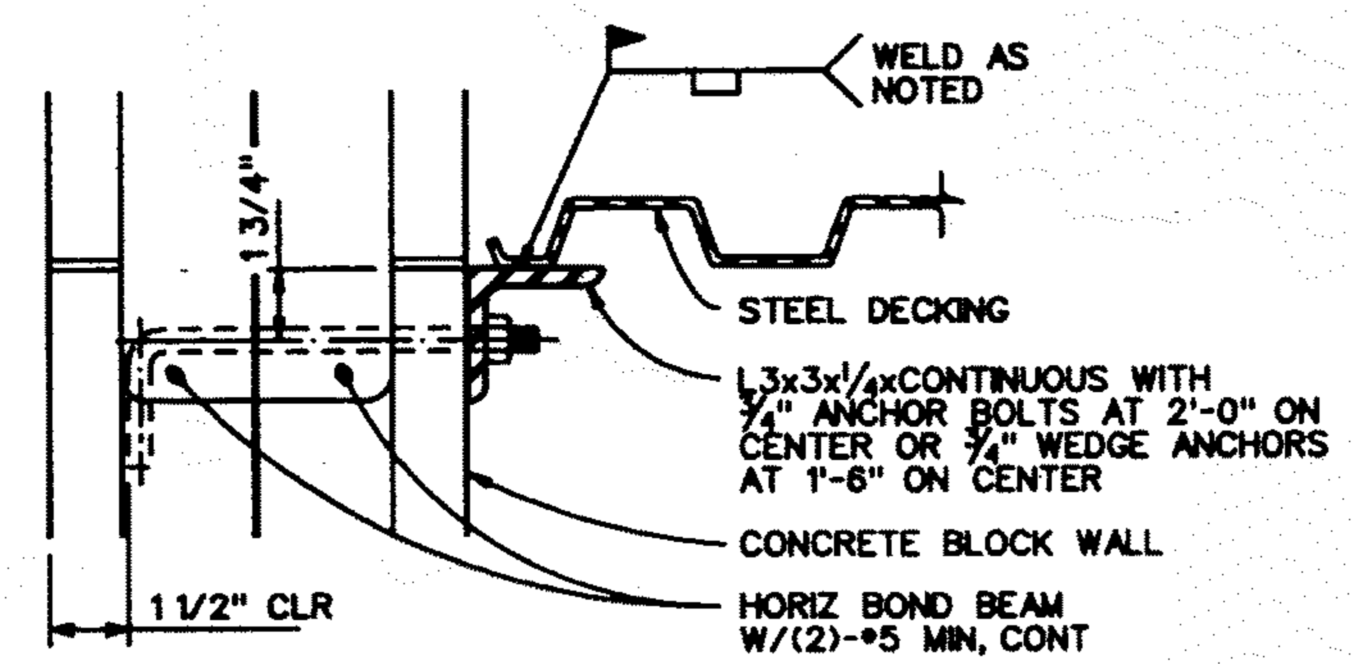
Concrete Wall Shear Strength

Concrete wall thickness, t =	14	in	
Concrete strength, f'_c =	4000	psi	
Seismic unit shear, Vu =	6.4	kips/ft	
Axial unit load on wall, Nu =	10.1	kips/ft	
Shear strength, $V_{c1} = 2*\lambda*(f'_c)^{0.5}*h*dv$ =	17.0	kips/ft	(ACI 318-14 Table 11.5.4.6)
Shear strength, $V_{c2} = 3.3*(f'_c)^{0.5}*h*dv+[(Nu*dv)/(4*L)]$ =	28.1	kips/ft	(ACI 318-14 Table 11.5.4.6)
Shear strength, $V_{cmax} = 10*(f'_c)^{0.5}*h*dv$ =	85.0	kips/ft	(ACI 318-14 Section 11.5.4.3)
Conc shear capacity, $V_c = \min[\max(V_{c1},V_{c2}),V_{cmax}]$ =	28.1	kips/ft	

Shear Friction between wall and slab

Dowels into foundation are #7@12" & #8@12" alternating (effective 6" spacing)

Reinforcement area, A_{vf} =	1.39	in^2	
steel yield strength, f_y =	60000	psi	
μ =	1.0		
Unit Shear Friction, V_n =	83.40	kip/ft	
Dowel DCR =	*0.34*	***Dowels can develop wall strength***	



DECKING LEDGER SUPPORT ANGLE

NTS

5005

WALL ANCHORAGE CONNECTION DETAIL ALONG NORTH AND SOUTH WALL ELEVATIONS

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BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Process Gallery	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)		

WALL ANCHORAGE FORCE

Process Gallery: Ledger Angle Anchorage into 8" CMU Wall along North and South Wall Elevations

4.4.3.7 Flexible Diaphragm Connection Forces. The horizontal seismic forces associated with the connection of a flexible diaphragm to either concrete or masonry walls, T_c, shall be calculated in accordance with Eq. (4-12).

$$T_c = \psi S_{XS} w_p A_p \qquad (4\text{-}12)$$

where

w_p = Unit weight of the wall;
A_p = Area of wall tributary to the connection;
ψ = 1.0 for Collapse Prevention Performance Level, 1.3 for Life Safety Performance Level, and 1.8 for Immediate Occupancy Performance Level; and
S_{XS} = Value specified in Section 4.4.2.3.

wall height to diaphragm, h_w =	14.63	ft	
parapet height, h_p =	0.87	ft	
unit weight of wall, w_p =	58.50	psf	(partial grout for exterior walls [CMU + veneer])
Ψ =	1.55		(Interpolated between LS & IO)
S_{XS} =	0.446	g	
wall out-of-plane load =	331.0	lbs/ft	
anchor bolt spacing =	24.00	in	
wall anchorage force, T_c =	662.0	lbs	

Masonry & Steel Strength

anchor bolt size =	0.750	in	
anchor bolt embed, l_b =	6.00	in	
anchor bolt yield stress, f_y =	36.00	ksi	
masonry compressive strength, f'_m =	1500	psi	
projected area of anchor bolt in tension, A_{pt} =	113.10	in^2	
cross section area of anchor bolt, A_b =	0.44	in^2	
$\phi B_{anb} = 4 \cdot A_{pt} \cdot (f'_m)^{0.5}$ =	17521.0	lbs	masonry breakout tensile strength
$\phi B_{ans} = A_b \cdot f_y$ =	15904.3	lbs	steel yielding strength

Masonry breakout strength DCR =	*0.04*	**OK**
Steel yielding DCR =	*0.04*	**OK**

Puddle Weld Shear Strength

Table 4: Allowable Shear Strength (lbs/connection) for Arc Spot Welds, Arc Seam Welds, Hilti Fasteners, Pneutek Fasteners and SDI Recognized Screws for Verco Deck Panel Support Connections

Deck Gage	Profile	BMT (in.)	ARC SPOT WELD (lbs)	ARC SEAM WELD (lbs)	HILTI X-ENDK22 or X-HSN 24 (lbs)	HILTI X-ENP-19 (lbs)	PNEUTEK SDK61 (lbs)	PNEUTEK SDK63 (lbs)	PNEUTEK K64 (lbs)	PNEUTEK K66 (lbs)	SDI RECOGNIZED SCREWS (lbs)
22	B & N	0.0299	703	1231	603	650	618	691	684	736	561
20	B & N	0.0359	1091	1491	720	776	733	791	886	903	673
18	B & N	0.0478	1850	2017	947	1020	951	967	1204	1253	896
16	B & N	0.0598	2309	2564	1169	1259	1158	1125	1474	1630	1121

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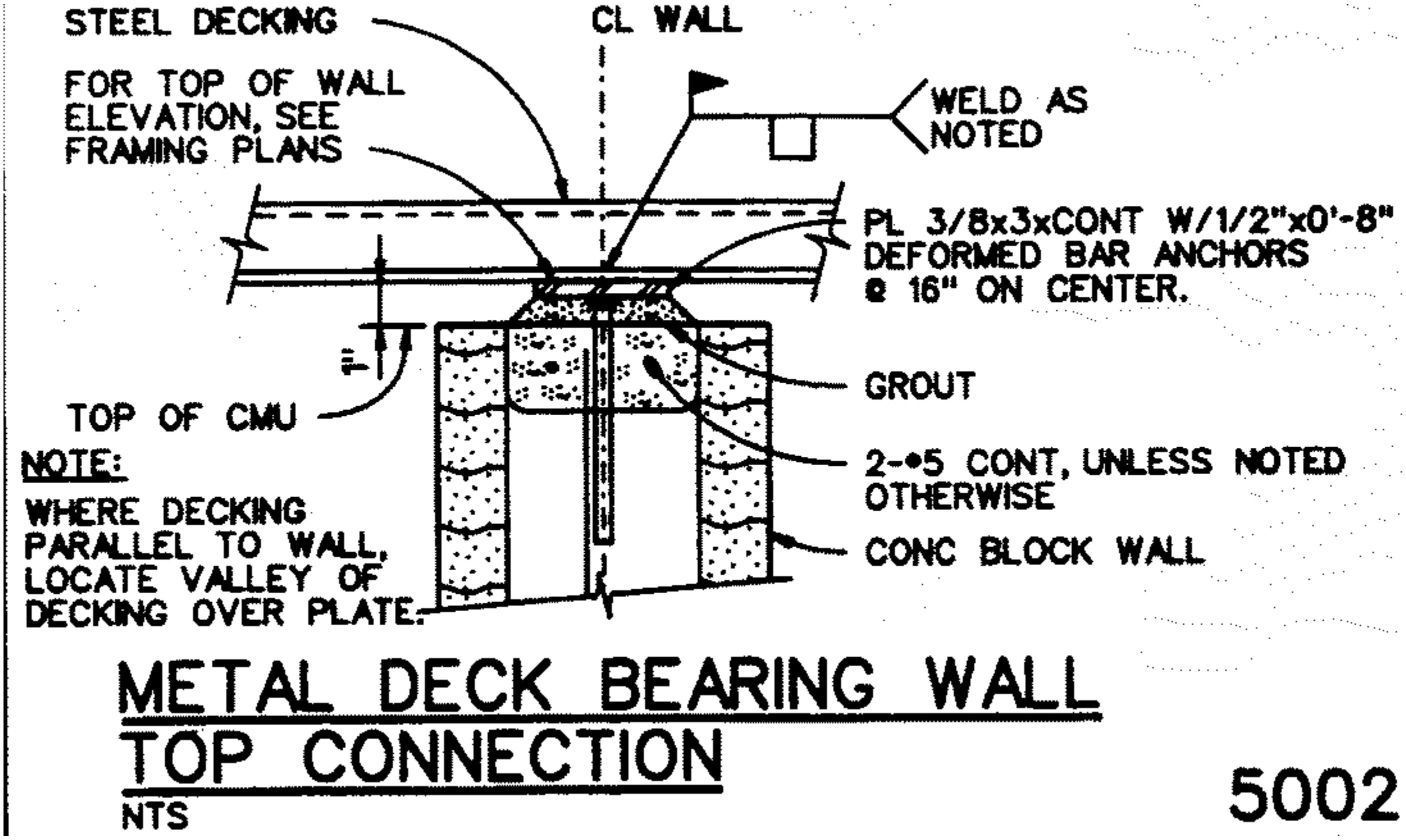
BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Process Gallery	**JOB NO.**	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)			

deck thickness =	0.0359	in	
weld spacing =	6.00	in	
load at weld =	165.5	lbs / weld	
allowable strength of screw from chart=	1091.0	lbs / weld	*ASCE 41-17 Section 9.10.1.3 allows for 2*
strength level of screw in shear =	2182.0	lbs / weld	*times allowable strength for strength level.*
Puddle weld strength DCR =	*0.08*	***OK***	



WALL ANCHORAGE CONNECTION DETAIL ALONG INTERIOR WALL ELEVATIONS

carollo
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BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Process Gallery	JOB NO.	11962A.00
DESIGN TASK		ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)					

WALL ANCHORAGE FORCE

Process Gallery: Bearing Anchorage into 8" Interior CMU Wall

4.4.3.7 Flexible Diaphragm Connection Forces. The horizontal seismic forces associated with the connection of a flexible diaphragm to either concrete or masonry walls, T_c, shall be calculated in accordance with Eq. (4-12).

$$T_c = \psi S_{XS} w_p A_p \qquad (4\text{-}12)$$

where

w_p = Unit weight of the wall;
A_p = Area of wall tributary to the connection;
ψ = 1.0 for Collapse Prevention Performance Level, 1.3 for Life Safety Performance Level, and 1.8 for Immediate Occupancy Performance Level; and
S_{XS} = Value specified in Section 4.4.2.3.

wall height to diaphragm, h_w =	14.63	ft	
unit weight of wall, w_p =	47.00	psf	(partial grout for interior walls)
Ψ =	1.55		(Interpolated between LS & IO)
S_{XS} =	0.446	g	
wall out-of-plane load =	237.7	lbs/ft	
anchor bolt spacing =	16.00	in	
wall anchorage force, T_c =	316.9	lbs	

Masonry & Steel Strength

anchor bolt size =	0.750	in
anchor bolt embed, l_b =	6.00	in
anchor bolt location from face, l_{be} =	3.81	in
anchor bolt yield stress, f_y =	36.00	ksi
masonry compressive strength, f_m =	1500	psi
projected area of anchor bolt in tension, A_{pt} =	113.10	in^2
projected area of each anchor bolt in shear, A_{pvbolt} =	22.80	in^2
cross section area of anchor bolt, A_b =	0.44	in^2

$\phi B_{vnb} = 4 * A_{pv} * (f'_m)^{0.5}$ =	3532.4	lbs	masonry breakout shear strength
$\phi B_{vnc} = 1050 * (f'_m * A_b)^{0.25}$ =	5327.4	lbs	masonry crushing shear strength
$\phi B_{vnpry} = 8 * A_{pt} * (f'_m)^{0.5}$ =	35041.9	lbs	anchor pryout shear strength
$\phi B_{vns} = 0.60 * A_b * f_y$ =	9542.6	lbs	steel yielding strength

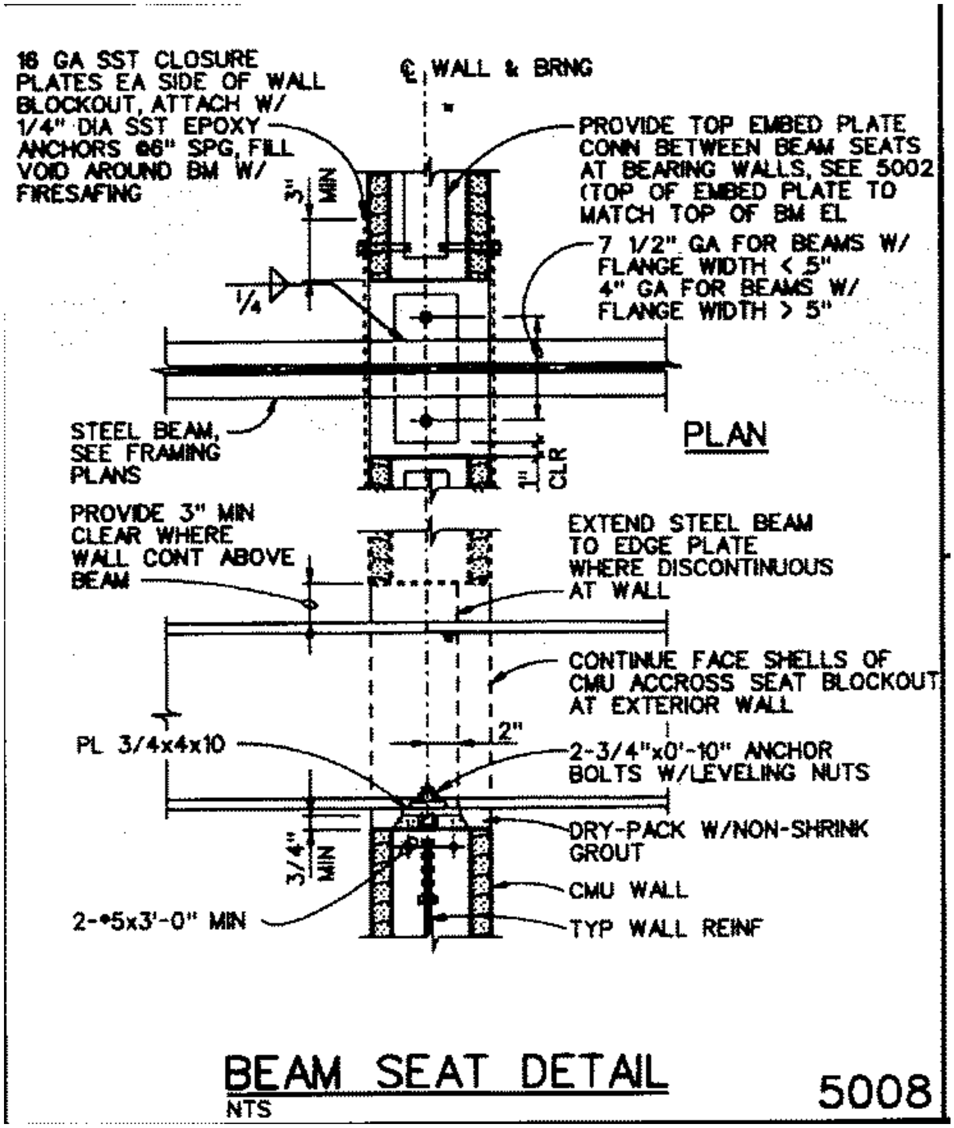
Masonry breakout strength DCR =	*0.09*	***OK***
Masonry crushing strength DCR =	*0.06*	***OK***
Anchor pryout DCR =	*0.01*	***OK***
Steel yielding DCR =	*0.03*	***OK***

<u>Puddle Weld Shear Strength</u>

Table 4: Allowable Shear Strength (lbs/connection) for Arc Spot Welds, Arc Seam Welds, Hilti Fasteners, Pneutek Fasteners and SDI Recognized Screws for Verco Deck Panel Support Connections

Deck Gage	Profile	BMT	ARC SPOT WELD	ARC SEAM WELD	HILTI X-ENDK22 or X-HSN 24	HILTI X-ENP-19	PNEUTEK SDK61	PNEUTEK SDK63	PNEUTEK K64	PNEUTEK K66	SDI RECOMMENDED SCREWS
		(in.)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)
22	B & N	0.0299	783	1231	603	650	618	691	684	736	561
20	B & N	0.0359	1091	1491	720	775	733	791	886	903	673
18	B & N	0.0478	1850	2017	947	1020	951	987	1204	1253	896
16	B & N	0.0598	2309	2564	1169	1259	1158	1125	1474	1630	1121

deck thickness =	0.0359	in	
weld spacing =	6.00	in	
load at weld =	118.8	lbs / weld	
allowable strength of screw from chart=	1091.0	lbs / weld	*ASCE 41-17 Section 9.10.1.3 allows for 2*
strength level of screw in shear =	2182.0	lbs / weld	*times allowable strength for strength level.*
Puddle weld strength DCR =	*0.05*	***OK***	



BEAM ANCHORAGE CONNECTION DETAIL ALONG EAST AND WEST WALL ELEVATIONS

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BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Process Gallery	JOB NO.	11962A.00
DESIGN TASK			ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)				

WALL ANCHORAGE FORCE

Process Gallery: Beam Anchorage into 8" CMU Wall along East and West Wall Elevations

4.4.3.7 Flexible Diaphragm Connection Forces. The horizontal seismic forces associated with the connection of a flexible diaphragm to either concrete or masonry walls, T_c, shall be calculated in accordance with Eq. (4-12).

$$T_c = \psi S_{XS} w_p A_p \qquad (4\text{-}12)$$

where

w_p = Unit weight of the wall;
A_p = Area of wall tributary to the connection;
ψ = 1.0 for Collapse Prevention Performance Level, 1.3 for Life Safety Performance Level, and 1.8 for Immediate Occupancy Performance Level; and
S_{XS} = Value specified in Section 4.4.2.3.

wall height to diaphragm, h_w =	14.63	ft	
parapet height, h_p =	0.87	ft	
unit weight of wall, w_p =	58.50	psf	(partial grout for exterior walls [CMU + veneer])
Ψ =	1.55		(Interpolated between LS & IO)
S_{XS} =	0.446	g	
wall out-of-plane load =	331.0	lbs/ft	
beam spacing =	6.67	ft	
wall anchorage force, T_c =	2207.8	lbs	

Masonry & Steel Strength

anchor bolt size =	0.750	in
anchor bolt embed, l_b =	8.00	in
anchor bolt location from face, l_{be} =	3.81	in
anchor bolt yield stress, f_y =	36.00	ksi
masonry compressive strength, f'_m =	1500	psi
projected area of single anchor bolt in tension, A_{pt} =	201.06	in^2
projected area of single anchor bolt in shear, A_{pvbolt} =	22.80	in^2
cross section area of single anchor bolt, A_b =	0.44	in^2
estimated overlap of projected area, $A_{ptoverlap}$ =	2.50	in^2
net projected area of anchor bolt in tension, A_{ptnet} =	400.87	in^2
estimated overlap of projected area, $A_{pvoverlap}$ =	1.25	in^2
net projected area of anchor bolt in shear, A_{pvnet} =	44.98	in^2

$\phi B_{vnb} = 4 * A_{pvnet} * (f'_m)^{0.5}$ =	6968.1	lbs	group masonry breakout shear strength
$\phi B_{vnc} = 1050 * (f'_m * A_b)^{0.25}$ =	10654.8	lbs	group masonry crushing shear strength
$\phi B_{vnpry} = 8 * A_{ptnet} * (f'_m)^{0.5}$ =	124206.2	lbs	group anchor pryout shear strength
$\phi B_{vns} = 0.60 * A_b * f_y$ =	19085.2	lbs	group steel yielding strength

Masonry breakout strength DCR =	*0.32*	***OK***
Masonry crushing strength DCR =	*0.21*	***OK***
Anchor pryout DCR =	*0.02*	***OK***
Steel yielding DCR =	*0.12*	***OK***

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BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Process Gallery	**JOB NO.**	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 1 Screening (CSZ Seismic Level)			

PLAN IRREGULARITIES

Beam Connection to CMU Walls (Detail 5008)

diaphragm shear strength, q_{ult} =	1466	lbs/ft	(assumed less than wall shear strength)
beam length =	30	ft	
diaphragm shear strength =	43980.0	lbs	

W16x26 Beam Tensile Strength (Assuming ϕ = 1.0 for Tier 1)

beam area, A =	7.68	in^2
steel yield stress, F_y =	50	ksi
steel tensile stress, F_u =	65	ksi
$\phi B_t = \min(F_y*A, F_u*A)$ =	384.0	kip
Masonry breakout strength DCR =	*0.11*	***OK***

Masonry & Steel Strength (Assuming ϕ = 1.0 for Tier 1)

anchor bolt size =	0.750	in	
anchor bolt embed, l_b =	8.00	in	
anchor bolt location from face, l_{be} =	3.81	in	
anchor bolt yield stress, f_y =	36.00	ksi	
masonry compressive strength, f'_m =	1500	psi	
projected area of single anchor bolt in tension, A_{pt} =	201.06	in^2	
projected area of single anchor bolt in shear, A_{pvbolt} =	22.80	in^2	
cross section area of single anchor bolt, A_b =	0.44	in^2	
estimated overlap of projected area, $A_{ptoverlap}$ =	2.50	in^2	
net projected area of anchor bolt in tension, A_{ptnet} =	400.87	in^2	
estimated overlap of projected area, $A_{pvoverlap}$ =	1.25	in^2	
net projected area of anchor bolt in shear, A_{pvnet} =	44.98	in^2	
$\phi B_{vnb} = 4*A_{pvnet}*(f'_m)^{0.5}$ =	6968.1	lbs	group masonry breakout shear strength
$\phi B_{vnc} = 1050*(f'_m*A_b)^{0.25}$ =	10654.8	lbs	group masonry crushing shear strength
$\phi B_{vnpry} = 8*A_{ptnet}*(f'_m)^{0.5}$ =	124206.2	lbs	group anchor pryout shear strength
$\phi B_{vns} = 0.60*A_b*f_y$ =	19085.2	lbs	group steel yielding strength

Masonry breakout strength DCR =	*6.31*	***NG***
Masonry crushing strength DCR =	*4.13*	***NG***
Anchor pryout DCR =	*0.35*	***OK***
Steel yielding DCR =	*2.30*	***NG***

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Workshop
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	07/07/21
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Table 17-2. Collapse Prevention Basic Configuration Checklist

Very Low Seismicity

BSE-2E Seismic Level at Limited Safety

Structural Components

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
C ☐	NC ☐	N/A ☒	U ☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps taht are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	Workshop exterior walls are wood stud with plywood sheathing.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Low Seismicity

Building System

General

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
C ☐	NC ☐	N/A ☒	U ☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement need not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	
C ☒	NC ☐	N/A ☐	U ☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Building Configuration

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1)	Building is a one-story structure.
C ☐	NC ☐	N/A ☒	U ☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	Building is a one-story structure.
C ☒	NC ☐	N/A ☐	U ☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
C ☐	NC ☐	N/A ☒	U ☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	Building is a one-story structure.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C ☐	NC ☐	N/A ☒	U ☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	Building is a one-story structure.
C ☐	NC ☐	N/A ☒	U ☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	Torsion check applies to rigid diaphragms. Structure has a flexible diaphragm.

Moderate Seismicity

Geologic Site Hazards

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)	Liquefaction has been determined to not be an issue per NGI technical memorandum.
C ☒	NC ☐	N/A ☐	U ☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)	Slope failure has been determined to not be an issue per NGI technical memorandum.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U		
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)	Surface fault rupture has been determined to not be an issue per NGI technical memorandum.

High Seismicity

Foundation Configuration

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)	Height = 15.50 ft Base = 36.00 ft Sa = 0.744 B / H = 36 / 15.5 = 2.32 0.6 * Sa = 0.6 * 0.744 = 0.45 2.32 > 0.45 (OK)
C	NC	N/A	U		
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Workshop
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	06/24/21
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Table 17-6. Collapse Prevention Structural Checklist for Building Type W2

Low and Moderate Seismicity

Lateral Seismic-Force-Resisting System

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
C ☒	NC ☐	N/A ☐	U ☐	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the following values (Commentary: Sec. A.3.2.7.1. Tier 2: Sec. 5.5.3.1.1): Structural panel sheathing 1,000 lb/ft Diagonal sheathing 700 lb/ft Straight sheathing 100 lb/ft All other conditions 100 lb/ft	West Wall Line DCR = 0.34 (OK) East Wall Line DCR = 1.02 (Slightly overstressed but considered OK) North Wall Line DCR = 0.52 (OK) South Wall Line DCR = 0.47 (OK) Interior Wall Line DCR = 0.42 (OK)
C ☐	NC ☐	N/A ☒	U ☐	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system. (Commentary: Sec. A.3.2.7.2. Tier 2: Sec. 5.5.3.6.1)	
C ☐	NC ☐	N/A ☒	U ☐	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used as shear walls on buildings more than one story high with the exception of the uppermost level of a multi-story building. (Commentary: Sec. A.3.2.7.3. Tier 2: Sec. 5.5.3.6.1)	Structure is one story.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Statement	Comments
☐	☒	☐	☐	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces. (Commentary: Sec. A.3.2.7.4. Tier 2: Sec. 5.5.3.6.1)	East elevation wall and northwest wall segments all exceed the 2:1 requirement. E1 = 14.5ft/2.5ft = 5.8 : 1 (NG) E2 = 14.5ft/4.5ft = 3.2 : 1 (NG) E3 = 14.5ft/6ft = 2.4 : 1 (NG) E4 = 14.5ft/2ft = 7.3 : 1 (NG)
☐	☐	☒	☐	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor. (Commentary: Sec. A.3.2.7.5. Tier 2: Sec.5.5.3.6.2)	
☐	☐	☒	☐	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1. (Commentary: Sec. A.3.2.7.6. Tier 2: Sec. 5.5.3.6.3)	
☐	☐	☒	☐	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels. (Commentary: Sec. A.3.2.7.7. Tier 2: Sec. 5.5.3.6.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	DESCRIPTION	COMMENTS
☐	☐	☒	☐	OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces. (Commentary: Sec. A.3.2.7.8. Tier 2: Sec. 5.5.3.6.5)	East elevation wall line has openings extending 70% of the length.

Connections

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	WOOD POSTS: There is a positive connection of wood posts to the foundation. (Commentary: Sec. A.5.3.3. Tier 2: Sec. 5.7.3.3)	
☒	☐	☐	☐	WOOD SILLS: All wood sills are bolted to the foundation. (Commentary: Sec. A.5.3.4. Tier 2: Sec. 5.7.3.3)	
☒	☐	☐	☐	GIRDER-COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

High Seismicity

Diaphragms

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
C ☒	NC ☐	N/A ☐	U ☐	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation. (Commentary: Sec. A.4.1.3. Tier 2: Sec. 5.6.1.1)	
C ☐	NC ☐	N/A ☒	U ☐	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. (Commentary: Sec. A.4.1.8. Tier 2: Sec. 5.6.1.5)	
C ☐	NC ☐	N/A ☒	U ☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	Drawings show 1/2" plywood diaphragm.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	DESCRIPTION	COMMENTS
☒	☐	☐	☐	SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. Wood commercial and industrial buildings may have rod-braced systems. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	Drawings show 1/2" plywood diaphragm.
☒	☐	☐	☐	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	The roof uses bridging but it is unclear if blocking between framing members. The spans are less than 40ft between shear walls and aspect ratio is less than 4-to-1.
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	Plans do show plywood at roof level.

Connections

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	WOOD SILL BOLTS: Sill bolts are spaced at 6 ft or less, with proper edge and end distance provided for wood and concrete. (Commentary: A.5.3.7. Tier 2: Sec. 5.7.3.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Workshop
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	06/18/2021
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

Table 17-38. Nonstructural Checklist

The Performance Level is designated LS for Life Safety or PR for Position Retention. The level of seismicity is designated as "not required" or by L, M, or H, for Low, Moderate, and High.

All Seismicity Levels For BSE-1E Tier 1, use PR (Position Retention)

Life Safety Systems

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. FIRE SUPPRESSION PIPING: Fire suppression piping is anchored and braced in accordance with NFPA-13. (Commentary: Sec. A.7.13.1. Tier 2: Sec. 13.7.4)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Fire suppression piping has flexible couplings in accordance with NFPA-13. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.4)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. EMERGENCY POWER: Equipment used to power or control life safety systems is anchored or braced. (Commentary: Sec. A.7.12.1. Tier 2: Sec. 13.7.7)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. STAIR AND SMOKE DUCTS: Stair pressurization and smoke control ducts are braced and have flexible connections at seismic joints. (Commentary: Sec. A.7.14.1. Tier 2: Sec. 13.7.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-MH; PR-MH. SPRINKLER CEILING CLEARANCE: Penetrations through panelized ceilings for fire suppression devices provide clearances in accordance with NFPA-13. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.4)	
☐	☐	☒	☐	LS-not required; PR-LMH. EMERGENCY LIGHTING: Emergency and egress lighting equipment is anchored or braced. (Commentary: Sec. A.7.3.1. Tier 2: Sec. 13.7.9)	

Hazardous Materials

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-LMH; PR-LMH. HAZARDOUS MATERIAL EQUIPMENT: Equipment mounted on vibration isolators and containing hazardous material is equipped with restraints or snubbers. (Commentary: Sec. A.7.12.2. Tier 2: 13.7.1)	
☐	☐	☒	☐	LS-LMH; PR-LMH. HAZARDOUS MATERIAL STORAGE: Breakable containers that hold hazardous material, including gas cylinders, are restrained by latched doors, shelf lips, wires, or other methods. (Commentary: Sec. A.7.15.1. Tier 2: Sec. 13.8.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Statement	Comments
☐	☐	☒	☐	LS-MH; PR-MH. HAZARDOUS MATERIAL DISTRIBUTION: Piping or ductwork conveying hazardous materials is braced or otherwise protected from damage that would allow hazardous material release. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)	
☐	☐	☒	☐	LS-MH; PR-MH. SHUT-OFF VALVES: Piping containing hazardous material, including natural gas, has shut-off valves or other devices to limit spills or leaks. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.3 and 13.7.5)	
☐	☐	☒	☐	LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Hazardous material ductwork and piping, including natural gas piping, has flexible couplings. (Commentary: Sec. A.7.15.4, Tier 2: Sec.13.7.3 and 13.7.5)	
☐	☐	☒	☐	LS-MH; PR-MH. PIPING OR DUCTS CROSSING SEISMIC JOINTS: Piping or ductwork carrying hazardous material that either crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.13.6. Tier 2: Sec.13.7.3, 13.7.5, and 13.7.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Partitions

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. UNREINFORCED MASONRY: Unreinforced masonry or hollow-clay tile partitions are braced at a spacing of at most 10 ft in Low or Moderate Seismicity, or at most 6 ft in High Seismicity. (Commentary: Sec. A.7.1.1. Tier 2: Sec. 13.6.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. HEAVY PARTITIONS SUPPORTED BY CEILINGS: The tops of masonry or hollow-clay tile partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. DRIFT: Rigid cementitious partitions are detailed to accommodate the following drift ratios: in steel moment frame, concrete moment frame, and wood frame buildings, 0.02; in other buildings, 0.005. (Commentary A.7.1.2 Tier 2: Sec. 13.6.2)	
C ☒	NC ☐	N/A ☐	U ☐	LS-not required; PR-MH. LIGHT PARTITIONS SUPPORTED BY CEILINGS: The tops of gypsum board partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-not required; PR-MH. STRUCTURAL SEPARATIONS: Partitions that cross structural separations have seismic or control joints. (Commentary: Sec. A.7.1.3. Tier 2. Sec. 13.6.2)	
☐	☐	☒	☐	LS-not required; PR-MH. TOPS: The tops of ceiling-high framed or panelized partitions have lateral bracing to the structure at a spacing equal to or less than 6 ft. (Commentary: Sec. A.7.1.4. Tier 2. Sec. 13.6.2)	

Ceilings

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-MH; PR-LMH. SUSPENDED LATH AND PLASTER: Suspended lath and plaster ceilings have attachments that resist seismic forces for every 12 ft^2 of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)	
☒	☐	☐	☐	LS-MH; PR-LMH. SUSPENDED GYPSUM BOARD: Suspended gypsum board ceilings have attachments that resist seismic forces for every 12 ft^2 of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)	Gypsum board is nailed to roof framing.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-MH. INTEGRATED CEILINGS: Integrated suspended ceilings with continuous areas greater than 144 ft^2, and ceilings of smaller areas that are not surrounded by restraining partitions, are laterally restrained at a spacing no greater than 12 ft with members attached to the structure above. Each restraint location has a minimum of four diagonal wires and compression struts, or diagonal members capable of resisting compression. (Commentary: Sec. A.7.2.2. Tier 2: Sec. 13.6.4)	
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-MH. EDGE CLEARANCE: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 have clearances from the enclosing wall or partition of at least the following: in Moderate Seismicity, 1/2 in.; in High Seismicity, 3/4 in. (Commentary: Sec. A.7.2.4. Tier 2: Sec. 13.6.4)	
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-MH. CONTINUITY ACROSS STRUCTURE JOINTS: The ceiling system does not cross any seismic joint and is not attached to multiple independent structures. (Commentary: Sec. A.7.2.5. Tier 2: Sec. 13.6.4)	
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. EDGE SUPPORT: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 are supported by closure angles or channels not less than 2 in. wide. (Commentary: Sec. A.7.2.6. Tier 2: Sec. 13.6.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-not required; PR-H. SEISMIC JOINTS: Acoustical tile or lay-in panel ceilings have seismic separation joints such that each continuous portion of the ceiling is no more than 2500 ft^2 and has a ratio of long-to-short dimension no more than 4-to-1. (Commentary: Sec. A.7.2.7. Tier 2: 13.6.4)	

Light Fixtures

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. INDEPENDENT SUPPORT: Light fixtures that weigh more per square foot than the ceiling they penetrate are supported independent of the grid ceiling suspension system by a minimum of two wires at diagonally opposite corners of each fixture. (Commentary: Sec. A.7.3.2. Tier 2: Sec. 13.6.4 and 13.7.9)	
C ☒	NC ☐	N/A ☐	U ☐	LS-not required; PR-H PENDANT SUPPORTS: Light fixtures on pendant supports are attached at a spacing equal to or less than 6 ft. Unbraced suspended fixtures are free to allow a 360-degree range of motion at an angle not less than 45 degrees from horizontal without contacting adjacent components. Alternatively, if rigid supported and/or braced, they are free to move with the structure to which they are attached without damaging adjoining components. Additionally, the connection to the structure is capable of accommodating the movement without failure. (Commentary: Sec. A.7.3.3. Tier 2: Sec. 13.7.9)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-H. LENS COVERS: Lens covers on light fixtures are attached with safety devices. (Commentary: Sec. A.7.3.4. Tier 2: Sec. 13.7.9)	Lights do not have lens covers

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Project Name City of Wilsonville
Project Number 11962A.00

Cladding and Glazing

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. CLADDING ANCHORS: Cladding components weighing more than 10 lb/ft^2 are mechanically anchored to the structure at a spacing equal to or less than the following: for Life Safety in Moderate Seismicity, 6 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 ft. (Commentary: Sec. A.7.4.1. Tier 2: Sec. 13.6.1)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH CLADDING ISOLATION: For steel or concrete moment-frame buildings, panel connections are detailed to accommodate a story drift ratio by the use of rods attached to framing with oversize holes or slotted holes of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02, and the rods have a length-to-diameter ratio of 4.0 or less. (Commentary: Sec. A.7.4.3. Tier 2: Sec. 13.6.1)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH MULTI-STORY PANELS: For multi-story panels attahed at more than one floor level panel connections are detailed to accommodate a story drift ratio by the use of rods attached to framing with oversize holes or slotted holes of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02, and the rods have a length-to-diameter ratio of 4.0 or less. (Commentary: Sec. A.7.4.4. Tier 2: Sec. 13.6.1)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH THREADED RODS: Threaded rods for panel connections detailed to accommodate drift by bending of the rod have a length-to-diameter ratio greater than 0.06 times the story height in inches for Life Safety in Moderate Seismicity and 0.12 times the story height in inches for Life Safety in High Seismicity and for Position Retention in any seismicity. (Commentary: Sec. A.7.4.9. Tier 2: Sec. 13.6.1)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U		
☐	☐	☒	☐	LS-MH; PR-MH. PANEL CONNECTIONS: Cladding panels are anchored out-of-plane with a minimum number of connections for each wall panel, as follows: for Life Safety in Moderate Seismicity, 2 connections; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 connections. (Commentary: Sec. A.7.4.5. Tier 2: Sec. 13.6.1.4)	

Project Name City of Wilsonville
Project Number 11962A.00

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-MH; PR-MH. BEARING CONNECTIONS: Where bearing connections are used, there is a minimum of two bearing connections for each cladding panel. (Commentary: Sec. A.7.4.6. Tier 2: Sec. 13.6.1.4)	
☐	☐	☒	☐	LS-MH; PR-MH. INSERTS: Where concrete cladding components use inserts, the inserts have positive anchorage or are anchored to reinforcing steel. (Commentary: Sec. A.7.4.7. Tier 2: Sec. 13.6.1.4)	
☐	☐	☒	☐	LS-MH; PR-MH. OVERHEAD GLAZING: Glazing panes of any size in curtain walls and individual interior or exterior panes over 16 ft^2 in area are laminated annealed or laminated heat-strengthened glass and are detailed to remain in the frame when cracked. (Commentary: Sec. A.7.4.8: Tier 2: Sec. 13.6.1.5)	Windows are less than 16 ft2 requirement.

Masonry Veneer

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-LMH; PR-LMH. TIES: Masonry veneer is connected to the backup with corrosion-resistant ties. There is a minimum of one tie for every 2-2/3 ft^2, and the ties have spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 36 in.; for Life Safety in High Seismicity and for Position Retention in any seismicity, 24 in. (Commentary: Sec. A.7.5.1. Tier 2: Sec. 13.6.1.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

C	NC	N/A	U	Description	
☐	☐	☒	☐	LS-LMH; PR-LMH. SHELF ANGLES: Masonry veneer is supported by shelf angles or other elements at each floor above the ground floor. (Commentary: Sec. A.7.5.2. Tier 2: Sec. 13.6.1.2)	
☐	☐	☒	☐	LS-LMH; PR-LMH. WEAKENED PLANES: Masonry veneer is anchored to the backup adjacent to weakened planes, such as at the locations of flashing. (Commentary: Sec. A.7.5.3. Tier 2: Sec. 13.6.1.2)	
☐	☐	☒	☐	LS-LMH; PR-LMH. UNREINFORCED MASONRY BACKUP: There is no unreinforced masonry backup. (Commentary: Sec. A.7.7.2. Tier 2: Section 13.6.1.1 and 13.6.1.2)	
☐	☐	☒	☐	LS-MH; PR-MH STUD TRACKS: For veneer with cold-formed steel stud backup, stud tracks are fastened to the structure at a spacing equal to or less than 24 in. on center. (Commentary: Sec. A.7.6.1. Tier 2: Sec. 13.6.1.1 and 13.6.1.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. ANCHORAGE: For veneer with concrete block or masonry backup, the backup is positively anchored to the structure at a horizontal spacing equal to or less than 4 ft along the floors and roof. (Commentary: Sec. A.7.7.1. Tier 2: Section 13.6.1.1 and 13.6.1.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-MH. WEEP HOLES: In veneer anchored to stud walls, the veneer has functioning weep holes and base flashing. (Commentary: Sec. A.7.5.6. Tier 2: Section 13.6.1.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-MH OPENINGS: For veneer with cold-formed -steel stud backup, steel studs frame window and door openings. (Commentary: Sec. A.7.6.2. Tier 2: Sec. 13.6.1.1 and 13.6.1.2)	

Parapets, Cornices, Ornamentation, and Appendages

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. URM PARAPETS OR CORNICES: Laterally unsupported unreinforced masonry parapets or cornices have height-to-thickness ratios no greater than the following: for Life Safety in Low or Moderate Seismicity, 2.5; for Life Safety in High Seismicity and for Position Retention in any seismicity, 1.5. (Commentary: Sec. A.7.8.1. Tier 2: Sec. 13.6.5)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. CANOPIES: Canopies at building exits are anchored to the structure at a spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 10 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 6 ft. (Commentary: Sec. A.7.8.2. Tier 2: Sec. 13.6.6)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-LMH. CONCRETE PARAPETS: Concrete parapets with height-to-thickness ratios greater than 2.5 have vertical reinforcement. (Commentary: Sec. A.7.8.3. Tier 2: Sec. 13.6.5)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-LMH. APPENDAGES: Cornices, parapets, signs, and other ornamentation or appendages that extend above the highest point of anchorage to the structure or cantilever from components are reinforced and anchored to the structural system at a spacing equal to or less than 6 ft. This checklist item does not apply to parapets or cornices covered by other checklist items. (Commentary: Sec. A.7.8.4. Tier 2: Sec. 13.6.6)	

Masonry Chimneys

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. URM CHIMNEYS: Unreinforced masonry chimneys extend above the roof surface no more than the following: for Life Safety in Low or Moderate Seismicity, 3 times the least dimension of the chimney; for Life Safety in High Seismicity and for Position Retention in any seismicity, 2 times the least dimension of the chimney. (Commentary: Sec. A.7.9.1. Tier 2: 13.6.7)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-LMH; PR-LMH. ANCHORAGE: Masonry chimneys are anchored at each floor level, at the topmost ceiling level, and at the roof. (Commentary: Sec. A.7.9.2. Tier 2: 13.6.7)	

Stairs

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. STAIR ENCLOSURES: Hollow-clay tile or unreinforced masonry walls around stair enclosures are restrained out-of-plane and have height-to-thickness ratios not greater than the following: for Life Safety in Low or Moderate Seismicity, 15-to-1; for Life Safety in High Seismicity and for Position Retention in any seismicity, 12-to-1. (Commentary: Sec. A.7.10.1. Tier 2: Sec. 13.6.2 and 13.6.8)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH STAIR DETAILS: The connection between the stairs and the structure does not rely on post-installed anchors in concrete or masonry, and the stair details are capable of accommodating the drift calculated using the Quick Check procedure of Section 4.4.3.1 for moment-frame structures or 0.5 in. for all other structures without including any lateral stiffness contribution from the stairs. (Commentary: Sec. A.7.10.2. Tier 2: Sec. 13.6.8)	

Contents and Furnishings

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. INDUSTRIAL STORAGE RACKS: Industrial storage racks or pallet racks more than 12 ft high meet the requirements of ANSI/MH 16.1 as modified by ASCE 7 Chapter 15. (Commentary: Sec. A.7.11.1. Tier 2: Sec. 13.8.1)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Item	Comments
☐	☒	☐	☐	LS-H; PR-MH. TALL NARROW CONTENTS: Contents more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 are anchored to the structure or to each other. (Commentary: Sec. A.7.11.2. Tier 2: Sec. 13.8.2)	Storage racks within building are on wheels and lack anchorage to structure. In addition, along the south elevation wall, not all shelving units are secured to structure.
☒	☐	☐	☐	LS-H; PR-H. FALL-PRONE CONTENTS: Equipment, stored items, or other contents weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level are braced or otherwise restrained. (Commentary: Sec. A.7.11.3. Tier 2: Sec. 13.8.2)	
☐	☐	☒	☐	LS-not required; PR-MH. ACCESS FLOORS: Access floors more than 9 in. high are braced. (Commentary: Sec. A.7.11.4. Tier 2: Sec. 13.8.3)	
☐	☐	☒	☐	LS-not required; PR-MH. EQUIPMENT ON ACCESS FLOORS: Equipment and other contents supported by access floor systems are anchored or braced to the structure independent of the access floor. (Commentary: Sec. A.7.11.5. Tier 2: Sec. 13.7.7 and 13.8.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Storage rack shelves have wheels and free to move without restraint back to structure.

Not all storage rack shelves are anchored back to structure.

C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	LS-not required; PR-H. SUSPENDED CONTENTS: Items suspended without lateral bracing are free to swing from or move with the structure from which they are suspended without damaging themselves or adjoining components. (Commentary. A.7.11.6. Tier 2: Sec. 13.8.2)	

Mechanical and Electrical Equipment

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	LS-H; PR-H. FALL-PRONE EQUIPMENT: Equipment weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level, and which is not in-line equipment, is braced. (Commentary: A.7.12.4. Tier 2: 13.7.1 and 13.7.7)	
☐	☐	☒	☐	LS-H; PR-H. IN-LINE EQUIPMENT: Equipment installed in-line with a duct or piping system, with an operating weight more than 75 lb, is supported and laterally braced independent of the duct or piping system. (Commentary: Sec. A.7.12.5. Tier 2: Sec. 13.7.1)	
☒	☐	☐	☐	LS-H; PR-MH. TALL NARROW EQUIPMENT: Equipment more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 is anchored to the floor slab or adjacent structural walls. (Commentary: Sec. A.7.12.6. Tier 2: Sec. 13.7.1 and 13.7.7)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-not required; PR-MH. MECHANICAL DOORS: Mechanically operated doors are detailed to operate at a story drift ratio of 0.01. (Commentary: Sec. A.7.12.7. Tier 2: Sec. 13.6.9)	
☒	☐	☐	☐	LS-not required; PR-H. SUSPENDED EQUIPMENT: Equipment suspended without lateral bracing is free to swing from or move with the structure from which it is suspended without damaging itself or adjoining components. (Commentary: Sec. A.7.12.8. Tier 2: Sec. 13.7.1 and 13.7.7)	
☐	☐	☒	☐	LS-not required; PR-H. VIBRATION ISOLATORS: Equipment mounted on vibration isolators is equipped with horizontal restraints or snubbers and with vertical restraints to resist overturning. (Commentary: Sec. A.7.12.9. Tier 2: Sec. 13.7.1)	
☐	☐	☒	☐	LS-not required; PR-H. HEAVY EQUIPMENT: Floor-supported or platform-supported equipment weighing more than 400 lb is anchored to the structure. (Commentary: Sec. A.7.12.10. Tier 2: 13.7.1 and 13.7.7)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
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C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-not required; PR-H. ELECTRICAL EQUIPMENT: Electrical equipment is laterally braced to the structure. (Commentary: Sec. A.7.12.11. Tier 2: 13.7.7)	
☐	☐	☒	☐	LS-not required; PR-H. CONDUIT COUPLINGS: Conduit greater than 2.5 in. trade size that is attached to panels, cabinets, or other equipment and is subject to relative seismic displacement has flexible couplings or connections. (Commentary: Sec. A.7.12.12. Tier 2: 13.7.8)	

Piping

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. FLEXIBLE COUPLINGS: Fluid and gas piping has flexible couplings. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.3 and 13.7.5)	
☐	☐	☒	☐	LS-not required; PR-H. FLUID AND GAS PIPING: Fluid and gas piping is anchored and braced to the structure to limit spills or leaks. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-H. C-CLAMPS: One-sided C-clamps that support piping larger than 2.5 in. in diameter are restrained. (Commentary: Sec. A.7.13.5. Tier 2: Sec. 13.7.3 and 13.7.5)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-H. PIPING CROSSING SEISMIC JOINTS: Piping that crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A7.13.6. Tier 2: Sec.13.7.3 and Sec. 13.7.5)	

Ducts

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-H. DUCT BRACING: Rectangular ductwork larger than 6 ft^2 in cross-sectional area and round ducts larger than 28 in. in diameter are braced. The maximum spacing of transverse bracing does not exceed 30 ft. The maximum spacing of longitudinal bracing does not exceed 60 ft. (Commentary: Sec. A.7.14.2. Tier 2: Sec. 13.7.6)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-H. DUCT SUPPORT: Ducts are not supported by piping or electrical conduit. (Commentary: Sec. A.7.14.3. Tier 2: Sec. 13.7.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-not required; PR-H. DUCTS CROSSING SEISMIC JOINTS: Ducts that cross seismic joints or isolation planes or are connected to independent structures have couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.14.5. Tier 2: Sec. 13.7.6)	

Elevators

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-H; PR-H. RETAINER GUARDS: Sheaves and drums have cable retainer guards. (Commentary: Sec. A.7.16.1. Tier 2: 13.8.6)	
☐	☐	☒	☐	LS-H; PR-H. RETAINER PLATE: A retainer plate is present at the top and bottom of both car and counterweight. (Commentary: Sec. A.7.16.2. Tier 2: 13.8.6)	
☐	☐	☒	☐	LS-not required; PR-H. ELEVATOR EQUIPMENT: Equipment, piping, and other components that are part of the elevator system are anchored. (Commentary: Sec. A.7.16.3. Tier 2: 13.8.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. SEISMIC SWITCH: Elevators capable of operating at speeds of 150 ft/min or faster are equipped with seismic switches that meet the requirements of ASME A17.1 or have trigger levels set to 20% of the acceleration of gravity at the base of the structure and 50% of the acceleration of gravity in other locations. (Commentary: Sec. A.7.16.4. Tier 2: 13.8.6)	
☐	☐	☒	☐	LS-not required; PR-H. SHAFT WALLS: Elevator shaft walls are anchored and reinforced to prevent toppling into the shaft during strong shaking. (Commentary: Sec. A.7.16.5. Tier 2: 13.8.6)	
☐	☐	☒	☐	LS-not required; PR-H. COUNTERWEIGHT RAILS: All counterweight rails and divider beams are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.6. Tier 2: 13.8.6)	
☐	☐	☒	☐	LS-not required; PR-H. BRACKETS: The brackets that tie the car rails and the counterweight rail to the structure are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.7. Tier 2: 13.8.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

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Project Number 11962A.00

C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-H. SPREADER BRACKET: Spreader brackets are not used to resist seismic forces. (Commentary: Sec. A.7.16.8. Tier 2: 13.8.6)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-H. GO-SLOW ELEVATORS: The building has a go-slow elevator system. (Commentary: Sec. A.7.16.9. Tier 2: 13.8.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

City of Wilsonville

Workshop Tier 1 Structural Calculations

OSHPD

Latitude, Longitude: 45.294444, -122.77167

Date	6/28/2021, 11:18:38 AM
Design Code Reference Document	ASCE41-17
Custom Probability	
Site Class	C - Very Dense Soil and Soft Rock

Type	Description	Value
Hazard Level		BSE-2N
S_S	spectral response (0.2 s)	0.813
S_1	spectral response (1.0 s)	0.381
S_{XS}	site-modified spectral response (0.2 s)	0.976
S_{X1}	site-modified spectral response (1.0 s)	0.571
F_a	site amplification factor (0.2 s)	1.2
F_v	site amplification factor (1.0 s)	1.5
ssuh	max direction uniform hazard (0.2 s)	0.92
crs	coefficient of risk (0.2 s)	0.884
ssrt	risk-targeted hazard (0.2 s)	0.813
ssd	deterministic hazard (0.2 s)	1.5
s1uh	max direction uniform hazard (1.0 s)	0.441
cr1	coefficient of risk (1.0 s)	0.863
s1rt	risk-targeted hazard (1.0 s)	0.381
s1d	deterministic hazard (1.0 s)	0.6

Type	Description	Value
Hazard Level		BSE-1N
S_{XS}	site-modified spectral response (0.2 s)	0.651
S_{X1}	site-modified spectral response (1.0 s)	0.381

Type	Description	Value
Hazard Level		BSE-2E
S_S	spectral response (0.2 s)	0.589
S_1	spectral response (1.0 s)	0.27
S_{XS}	site-modified spectral response (0.2 s)	0.744
S_{X1}	site-modified spectral response (1.0 s)	0.405
f_a	site amplification factor (0.2 s)	1.265
f_v	site amplification factor (1.0 s)	1.5

Type	Description	Value
Hazard Level		BSE-1E
S_S	spectral response (0.2 s)	0.223
S_1	spectral response (1.0 s)	0.082
S_{XS}	site-modified spectral response (0.2 s)	0.291
S_{X1}	site-modified spectral response (1.0 s)	0.123
F_a	site amplification factor (0.2 s)	1.3
F_v	site amplification factor (1.0 s)	1.5

Type	Description	Value
Hazard Level		TL Data
T-Sub-L	Long-period transition period in seconds	16

Engineers...Working Wonders With Water™

BY: BS	DATE Jul-21	CLIENT City of Wilsonville	SHEET
CHKD BY	DESCRIPTION Workshop Building		JOB NO. 11962A.00
DESIGN TASK	Workshop Building Seismic Weight		

Roof Loads

Roof EL 125.63

Description	Load		
1/2" plywood	1.5	psf	
Rigid insulation w/ built-up roofing	6.0		(See note 2)
2x12 wood joists @ 24"	2.0		
3 1/8"x15" glulam beam	1.0		(See note 1)
5/8" gypsum wall board interior finish	3.2		
Miscellaneous	3.0		
Dead Load for Gravity Design	**16.7**	psf	
Roof Live Load	**20.0**	psf	(Assumed)

Notes

1. Roof glulam beam self weight assumed unit beam weight, 13.2 lb/ft, divided by beam tributary width, 17.0 ft which is 13.2lb/ft/17.0ft = 0.78 lb/ft^2. Assume 1.0 psf.
2. Rigid insulation slopes from 1.5" to 6". The average insulation thickness is assumed to be 3".

Wall Loads

Wall Loads

Description	Load	
2x6 @ 16" stud wall w/ 5/8" GWB int and 1/2" gypsum sheathing ext	11.5	psf
1/2" plywood siding	1.5	
2x6 Stud Wall Load for Seismic	**13.0**	psf

Seismic Weight

Roof Weight

Roof Area	2464.0	ft^2

Roof Seismic Weight	**41.1**	**kip**

Wall Weight

Wall Height to Roof	15.50	ft
Stud Wall Length	212.00	ft
Opening Length in Stud Wall	35.00	ft
Roof Wall Seismic Weight	**17.8**	**kip**

Total Seismic Weight	**59.0**	**kip**

Notes

1. Wall seismic weight assumes half of the wall height associated with each level.

2. Per Section 4.5.2.1, the effective seismic weight includes the total dead load, 25% of the live load when area is used as storage, and 20% of the roof snow live load if greater than 30 psf (otherwise assume zero).

carollo
Engineers... Working Wonders With Water ™

BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Workshop	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 1 Screening (BSE-2E Level)		

SEISMIC BASE SHEAR

4.4.2.1 Pseudo Seismic Force. The pseudo seismic force, in a given horizontal direction of a building, shall be calculated in accordance with Eq. (4-1).

$$V = CS_aW \qquad (4\text{-}1)$$

where

V = Pseudo seismic force;
C = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response; C shall be taken from Table 4-7;
S_a = Response spectral acceleration at the fundamental period of the building in the direction under consideration. The value of S_a shall be calculated in accordance with the procedures in Section 4.4.2.3; and
W = Effective seismic weight of the building, including the total dead load and applicable portions of other gravity loads listed below:

Table 4-7. Modification Factor, *C*

Building Type[a]	Number of Stories 1	2	3	≥4
Wood and cold-formed steel shear wall (W1, W1a, W2, CFS1) Moment frame (S1, S3, C1, PC2a)	1.3	1.1	1.0	1.0
Shear wall (S4, S5, C2, C3, PC1a, PC2, RM2, URMa) Braced frame (S2) Cold-formed steel strap-brace wall (CFS2)	1.4	1.2	1.1	1.0
Unreinforced masonry (URM) Flexible diaphragms (S1a, S2a, S5a, C2a, C3a, PC1, RM1)	1.0	1.0	1.0	1.0

[a] Defined in Table 3-1.

Process Gallery

Modification Factor, C =	1.3	
S_{X1} =	0.405	(BSE-2E seismic parameter)
T =	0.149	s
S_{XS} =	0.744	(BSE-2E seismic parameter)
Spectral Acceleration, S_a =	0.744	
Seismic Weight, W =	59.0	kip
Seismic Force, V =	57.1	kip

Engineers... Working Wonders With Water ™

BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Workshop	**JOB NO.**	11962A.00
DESIGN TASK			ASCE 41-17 - Tier 1 Screening (BSE-2E Level)				

WALL SHEAR STRESS CHECK

4.4.3.3 Shear Stress in Shear Walls. The average shear stress in shear walls, v_j^{avg}, shall be calculated in accordance with Eq. (4-8).

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \quad (4\text{-}8)$$

where

V_j = Story shear at level j computed in accordance with Section 4.4.2.2;

A_w = Summation of the horizontal cross-sectional area of all shear walls in the direction of loading. Openings shall be taken into consideration where computing A_w. For masonry walls, the net area shall be used. For wood-framed walls, the length shall be used rather than the area; and

M_s = System modification factor; M_s shall be taken from Table 4-8.

Table 4-8. M_s Factors for Shear Walls

Wall Type	Level of Performance CP[a]	LS[a]	IO[a]
Reinforced concrete, precast concrete, wood, reinforced masonry, and cold-formed steel	4.5	3.0	1.5
Unreinforced masonry	1.75	1.25	1.0

[a] CP = Collapse Prevention, LS = Life Safety, IO = Immediate Occupancy.

Roof Story Base Shear, V_{roof} = 57.1 kips

System Modification Factor, M_s = 3.75 (Interpolated between LS & CP)

Roof Level

Shear Wall in N-S Direction

West Elevation Wall Line

Total length of stud walls = 44.67 ft

average shear stress, $v_{avg,NS}$ = 340.9 lb/ft *<* ***1000.0*** ***Shear Stress OK***

DCR = 0.34

East Elevation Wall Line

Total length of stud walls = 15.00 ft

average shear stress, $v_{avg,NS}$ = 1015.1 lb/ft *>* ***1000.0*** ***NG***

DCR = 1.02

Shear Wall in E-W Direction

North Elevation Wall Line

Total length of stud walls = 29.33 ft

average shear stress, $v_{avg,EW}$ = 519.1 lb/ft *<* ***1000.0*** ***Shear Stress OK***

DCR = 0.52

South Elevation Wall Line

Total length of stud walls = 32.67 ft

average shear stress, $v_{avg,EW}$ = 466.1 lb/ft *<* ***1000.0*** ***Shear Stress OK***

DCR = 0.47

Interior Wall Line

Total length of stud walls = 36.00 ft

average shear stress, $v_{avg,EW}$ = 423.0 lb/ft *<* ***1000.0*** ***Shear Stress OK***

DCR = 0.42

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Workshop
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	07/07/21
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Wo
Project Number 11962A.00

Table 17-3. Immediate Occupancy Basic Configuration Checklist

Very Low Seismicity CSZ Seismic Level at Damage Control

Structural Components

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
C ☐	NC ☐	N/A ☒	U ☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	Workshop exterior walls are wood stud with plywood sheathing.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Very Low Seismicity

Building System

General

RATING				DESCRIPTION	COMMENTS
C [x]	NC [ ]	N/A [ ]	U [ ]	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
C [ ]	NC [ ]	N/A [x]	U [ ]	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement need not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	
C [x]	NC [ ]	N/A [ ]	U [ ]	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Building Configuration

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction shall not be less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	Building is a one-story structure.
C ☐	NC ☐	N/A ☒	U ☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story shall not be less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	Building is a one-story structure.
C ☒	NC ☐	N/A ☐	U ☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
C ☐	NC ☐	N/A ☒	U ☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	Building is a one-story structure.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Statement	Comment
☐	☐	☒	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	Building is a one-story structure.
☐	☐	☒	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	Torsion check applies to rigid diaphragms. Structure has a flexible diaphragm.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville - Wo
Project Number 11962A.00

Low Seismicity

Geologic Site Hazards

C	NC	N/A	U	DESCRIPTION	COMMENTS
☒	☐	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)	Liquefaction has been determined to not be an issue per NGI technical memorandum.
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)	Slope failure has been determined to not be an issue per NGI technical memorandum.
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)	Surface fault rupture has been determined to not be an issue per NGI technical memorandum.

Moderate and High Seismicity

Foundation Configuration

C	NC	N/A	U	DESCRIPTION	COMMENTS
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)	Height = 15.50 ft Base = 36.00 ft Sa = 0.446 B/H = 36 / 15.5 = 2.32 0.6*Sa = 0.6 * 0.446 = 0.27 2.32 > 0.27 (OK)

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name ____________
Project Number ____________

C	NC	N/A	U		
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Workshop
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	07/07/21
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/03/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Table 17-7. Immediate Occupancy Structural Checklist for Building Type W2

Very Low Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	DESCRIPTION	COMMENTS
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the following values (Commentary: Sec. A.3.2.7.1. Tier 2: Sec. 5.5.3.1.1): Structural panel sheathing 1,000 lb/ft Diagonal sheathing 700 lb/ft Straight sheathing 100 lb/ft All other conditions 100 lb/ft	West Wall Line DCR = 0.34 (OK) East Wall Line DCR = 1.01 (Slightly overstressed but considered OK) North Wall Line DCR = 0.52 (OK) South Wall Line DCR = 0.47 (OK) Interior Wall Line DCR = 0.42 (OK)
☐	☐	☒	☐	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system. (Commentary: Sec. A.3.2.7.2. Tier 2: Sec. 5.5.3.6.1)	
☐	☐	☒	☐	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used as shear walls on buildings more than one story high with the exception of the uppermost level of a multi-story building. (Commentary: Sec. A.3.2.7.3. Tier 2: Sec. 5.5.3.6.1)	Structure is one story.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Item	Comments
☐	☒	☐	☐	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces. (Commentary: Sec. A.3.2.7.4. Tier 2: Sec. 5.5.3.6.1)	East elevation wall segments all exceed the 2:1 requirement. E1 = 14.5ft/2.5ft = 5.8 : 1 (NG) E2 = 14.5ft/4.5ft = 3.2 : 1 (NG) E3 = 14.5ft/6ft = 2.4 : 1 (NG) E4 = 14.5ft/2ft = 7.3 : 1 (NG)
☐	☐	☒	☐	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor. (Commentary: Sec. A.3.2.7.5. Tier 2: Sec. 5.5.3.6.2)	
☐	☐	☒	☐	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-2. (Commentary: Sec. A.3.2.7.6. Tier 2: Sec. 5.5.3.6.3)	
☐	☐	☒	☐	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels. (Commentary: Sec. A.3.2.7.7. Tier 2: Sec. 5.5.3.6.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C ☐	NC ☐	N/A ☒	U ☐	OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces. (Commentary: Sec. A.3.2.7.8. Tier 2: Sec. 5.5.3.6.5)	East elevation wall line has openings extending 70% of the length.
C ☒	NC ☐	N/A ☐	U ☐	HOLD-DOWN ANCHORS: All shear walls have hold-down anchors, constructed per acceptable construction practices, attached to the end studs. (Commentary: Sec. A.3.2.7.9. Tier 2: Sec. 5.5.3.6.6)	

Connections

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	WOOD POSTS: There is a positive connection of wood posts to the foundation. (Commentary: Sec. A.5.3.3. Tier 2: Sec. 5.7.3.3)	
C ☒	NC ☐	N/A ☐	U ☐	WOOD SILLS: All wood sills are bolted to the foundation. (Commentary: Sec. A.5.3.4. Tier 2: Sec. 5.7.3.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	GIRDER-COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	

Foundation System

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil. (Commentary: Sec.A.6.2.3.)	There are no deep foundation systems supporting structure.
☐	☐	☒	☐	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another shall not exceed one story high. (Commentary: Sec. A.6.2.4)	

Low, Moderate, and High Seismicity

Seismic-Force-Resisting System

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☒	☐	☐	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 1.5-to-1 are not used to resist seismic forces. (Commentary: Sec. A.3.2.7.4. Tier 2: Sec. 5.5.3.6.1)	East elevation wall segments all exceed the 1.5:1 requirement. E1 = 14.5ft/2.5ft = 5.8 : 1 (NG) E2 = 14.5ft/4.5ft = 3.2 : 1 (NG) E3 = 14.5ft/6ft = 2.4 : 1 (NG) E4 = 14.5ft/2ft = 7.3 : 1 (NG)

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name	City of Wilsonville - W
Project Number	11962A.00

Diaphragms

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
C ☒	NC ☐	N/A ☐	U ☐	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation. (Commentary: Sec. A.4.1.3. Tier 2: Sec. 5.6.1.1)	
C ☐	NC ☐	N/A ☒	U ☐	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities. (Commentary: Sec. A.4.1.7. Tier 2: Sec. 5.6.1.4)	No plan irregularities.
C ☐	NC ☐	N/A ☒	U ☐	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. (Commentary: Sec. A.4.1.8. Tier 2: Sec. 5.6.1.5)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Item	Comments
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	Drawings show 1/2" plywood diaphragm.
☒	☐	☐	☐	SPANS: All wood diaphragms with spans greater than 12 ft consist of wood structural panels or diagonal sheathing. Wood commercial and industrial buildings may have rod-braced systems. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	Drawings show 1/2" plywood diaphragm.
☐	☐	☐	☒	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft and aspect ratios less than or equal to 3-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	The roof uses bridging but it is unclear if blocking is used between members. The southern roof span over the shop does exceed the 30ft horizontal span requirement, but it meets aspect ratio of 3-to-1.
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

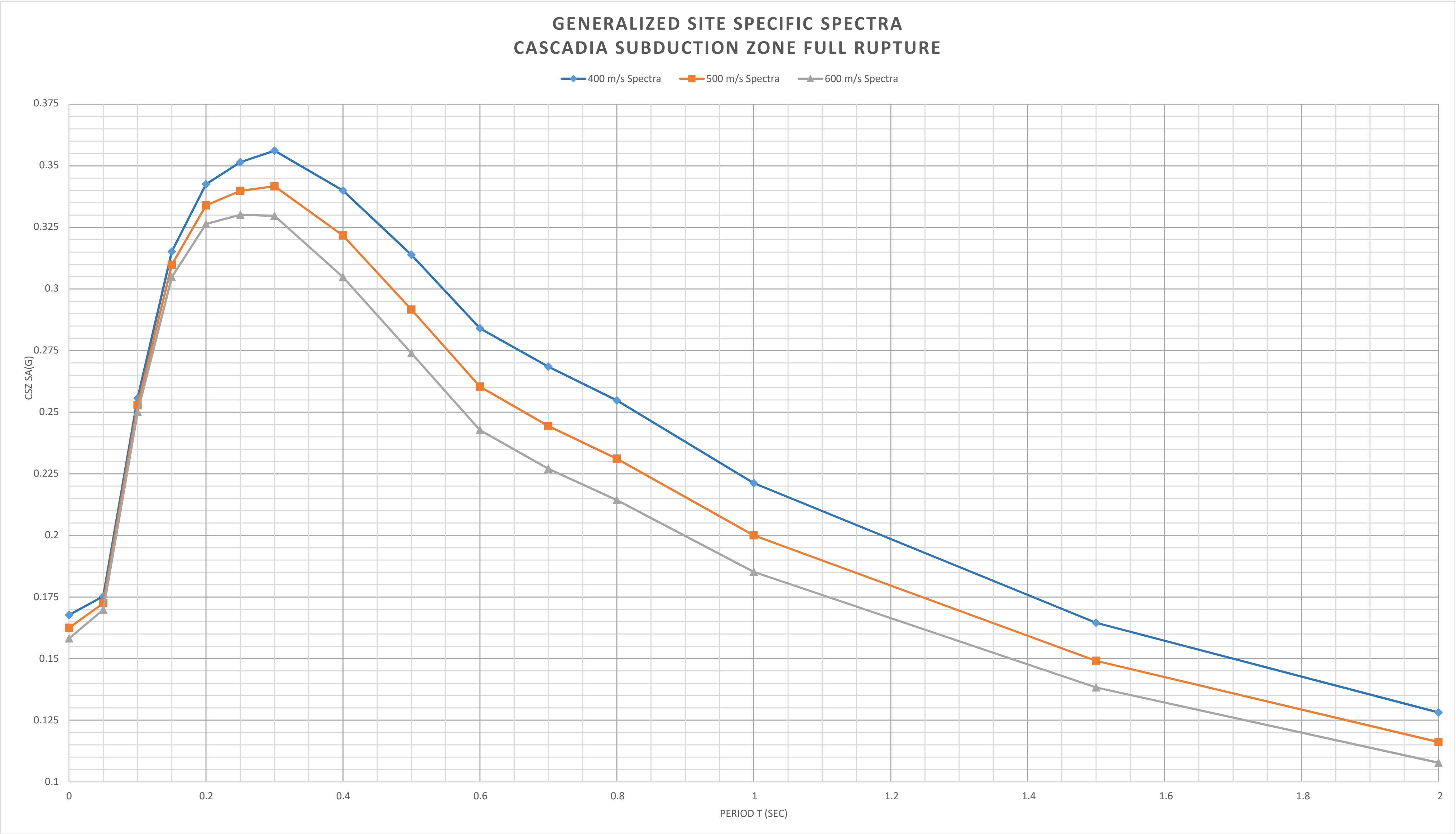
Connections

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U	WOOD SILL BOLTS: Sill bolts are spaced at 4 ft or less, with proper edge and end distance provided for wood and concrete. (Commentary: Sec. A.5.3.7. Tier 2: Sec. 5.7.3.3)	
☒	☐	☐	☐		

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

City of Wilsonville

Workshop Tier 1 Structural Calculations



Northwest Geotech, Inc.

Figure No. 3

Ss @ T=0.20 sec

Table 2: CSZ Generalized Response Spectra Ordinates					
Latitude 45.295155 degrees Longitude -122.771810 degrees					
Vs30 = 400 m/s		Vs30 = 500 m/s		Vs30 = 600 m/s	
Period T(sec)	CSZ Sa(g)	Period T (sec)	CSZ Sa (g)	Period T (sec)	CSZ Sa (g)
0	0.168	0	0.163	0	0.158
0.05	0.175	0.05	0.172	0.05	0.170
0.1	0.256	0.1	0.253	0.1	0.250
0.15	0.315	0.15	0.310	0.15	0.305
0.2	0.343	0.2	0.334	0.2	0.326
0.25	0.352	0.25	0.340	0.25	0.330
0.3	0.356	0.3	0.342	0.3	0.330
0.4	0.340	0.4	0.322	0.4	0.305
0.5	0.314	0.5	0.292	0.5	0.274
0.6	0.284	0.6	0.260	0.6	0.243
0.7	0.269	0.7	0.244	0.7	0.227
0.8	0.255	0.8	0.231	0.8	0.214
1	0.221	1	0.200	1	0.185
1.5	0.165	1.5	0.149	1.5	0.138
2	0.128	2	0.116	2	0.108
2.5	0.104	2.5	0.094	2.5	0.087
3	0.085	3	0.077	3	0.071

Engineers...Working Wonders With Water™

BY: BS	DATE Jul-21	CLIENT City of Wilsonville	SHEET
CHKD BY	DESCRIPTION Workshop Building		JOB NO. 11962A.00
DESIGN TASK	Workshop Building Seismic Weight		

Roof Loads

Roof EL 125.63

Description	Load		
1/2" plywood	1.5	psf	
Rigid insulation w/ built-up roofing	6.0		(See note 2)
2x12 wood joists @ 24"	2.0		
3 1/8"x15" glulam beam	1.0		(See note 1)
5/8" gypsum wall board interior finish	3.2		
Miscellaneous	3.0		
Dead Load for Gravity Design	**16.7**	psf	
Roof Live Load	**20.0**	psf	(Assumed)

Notes

1. Roof glulam beam self weight assumed unit beam weight, 13.2 lb/ft, divided by beam tributary width, 17.0 ft which is 13.2lb/ft/17.0ft = 0.78 lb/ft^2. Assume 1.0 psf.
2. Rigid insulation slopes from 1.5" to 6". The average insulation thickness is assumed to be 3".

Wall Loads

Wall Loads

Description	Load	
2x6 @ 16" stud wall w/ 5/8" GWB int and 1/2" gypsum sheathing ext	11.5	psf
1/2" plywood siding	1.5	
2x6 Stud Wall Load for Seismic	**13.0**	psf

Seismic Weight

Roof Weight

Roof Area 2464.0 ft^2

Roof Seismic Weight	**41.1**	**kip**

Wall Weight

Wall Height to Roof	15.50	ft
Stud Wall Length	212.00	ft
Opening Length in Stud Wall	35.00	ft
Roof Wall Seismic Weight	**17.8**	**kip**

Total Seismic Weight	**59.0**	**kip**

Notes

1. Wall seismic weight assumes half of the wall height associated with each level.

2. Per Section 4.5.2.1, the effective seismic weight includes the total dead load, 25% of the live load when area is used as storage, and 20% of the roof snow live load if greater than 30 psf (otherwise assume zero).

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BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Workshop	**JOB NO.**	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 1 Screening (BSE-2E Level)			

SEISMIC BASE SHEAR

4.4.2.1 Pseudo Seismic Force. The pseudo seismic force, in a given horizontal direction of a building, shall be calculated in accordance with Eq. (4-1).

$$V = CS_aW \tag{4-1}$$

where

V = Pseudo seismic force;
C = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response; C shall be taken from Table 4-7;
S_a = Response spectral acceleration at the fundamental period of the building in the direction under consideration. The value of S_a shall be calculated in accordance with the procedures in Section 4.4.2.3; and
W = Effective seismic weight of the building, including the total dead load and applicable portions of other gravity loads listed below:

Table 4-7. Modification Factor, C

	Number of Stories			
Building Type[a]	**1**	**2**	**3**	**≥4**
Wood and cold-formed steel shear wall (W1, W1a, W2, CFS1) Moment frame (S1, S3, C1, PC2a)	1.3	1.1	1.0	1.0
Shear wall (S4, S5, C2, C3, PC1a, PC2, RM2, URMa) Braced frame (S2) Cold-formed steel strap-brace wall (CFS2)	1.4	1.2	1.1	1.0
Unreinforced masonry (URM) Flexible diaphragms (S1a, S2a, S5a, C2a, C3a, PC1, RM1)	1.0	1.0	1.0	1.0

[a] Defined in Table 3-1.

Process Gallery

Modification Factor, C =	1.3	
S_s =	0.343	(CSZ spectral response)
S_1 =	0.221	(CSZ spectral response)
F_a =	1.3	(Site amplication factor per ASCE 7-16)
F_v =	1.5	(Site amplication factor per ASCE 7-16)
$S_{X1} = S_1 {*} F_v$ =	0.332	(CSZ seismic hazard)
T =	0.149	s
$S_{Xs} = S_s {*} F_a$ =	0.446	(CSZ seismic hazard)
Spectral Acceleration, S_a =	0.446	
Seismic Weight, W =	59.0	kip
Seismic Force, V =	34.2	kip

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BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**		Workshop		**JOB NO.**	11962A.00
DESIGN TASK		ASCE 41-17 - Tier 1 Screening (BSE-2E Level)					

WALL SHEAR STRESS CHECK

4.4.3.3 Shear Stress in Shear Walls. The average shear stress in shear walls, v_j^{avg}, shall be calculated in accordance with Eq. (4-8).

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \quad (4\text{-}8)$$

where

V_j = Story shear at level j computed in accordance with Section 4.4.2.2;

A_w = Summation of the horizontal cross-sectional area of all shear walls in the direction of loading. Openings shall be taken into consideration where computing A_w. For masonry walls, the net area shall be used. For wood-framed walls, the length shall be used rather than the area; and

M_s = System modification factor; M_s shall be taken from Table 4-8.

Table 4-8. M_s Factors for Shear Walls

	Level of Performance		
Wall Type	**CP**[a]	**LS**[a]	**IO**[a]
Reinforced concrete, precast concrete, wood, reinforced masonry, and cold-formed steel	4.5	3.0	1.5
Unreinforced masonry	1.75	1.25	1.0

[a] CP = Collapse Prevention, LS = Life Safety, IO = Immediate Occupancy.

Roof Story Base Shear, V_{roof} = 34.2 kips

System Modification Factor, M_s = 2.25 (Interpolated between LS & IO)

Roof Level

Shear Wall in N-S Direction

West Elevation Wall Line

Total length of stud walls = 44.67 ft

average shear stress, $v_{avg,NS}$ = 340.3 lb/ft *<* ***1000.0*** ***Shear Stress OK***

DCR = 0.34

East Elevation Wall Line

Total length of stud walls = 15.00 ft

average shear stress, $v_{avg,NS}$ = 1013.3 lb/ft *>* ***1000.0*** ***NG***

DCR = 1.01

Shear Wall in E-W Direction

North Elevation Wall Line

Total length of stud walls = 29.33 ft

average shear stress, $v_{avg,EW}$ = 518.2 lb/ft *<* ***1000.0*** ***Shear Stress OK***

DCR = 0.52

South Elevation Wall Line

Total length of stud walls = 32.67 ft

average shear stress, $v_{avg,EW}$ = 465.3 lb/ft *<* ***1000.0*** ***Shear Stress OK***

DCR = 0.47

Interior Wall Line

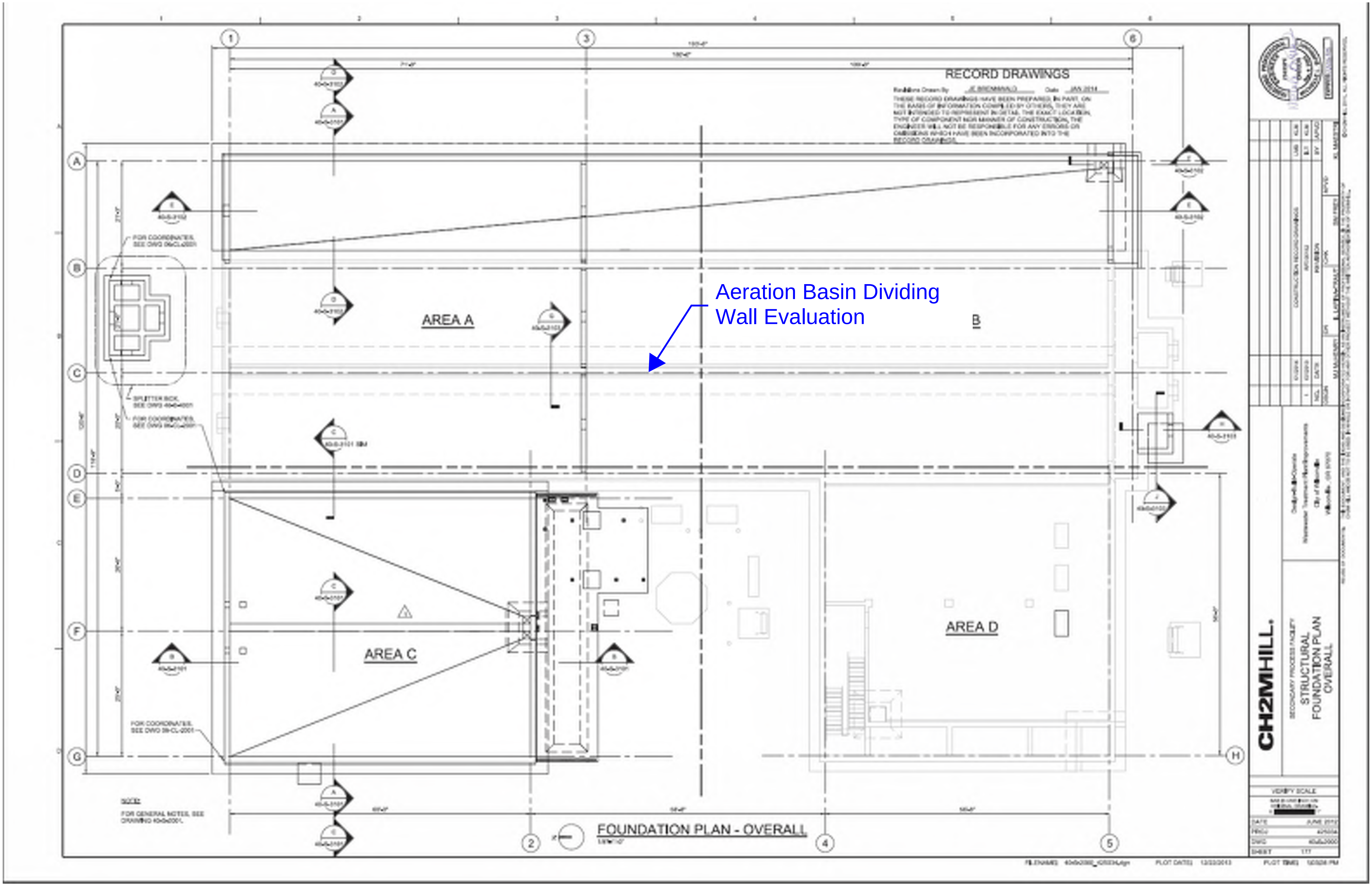
Total length of stud walls = 36.00 ft

average shear stress, $v_{avg,EW}$ = 422.2 lb/ft *<* ***1000.0*** ***Shear Stress OK***

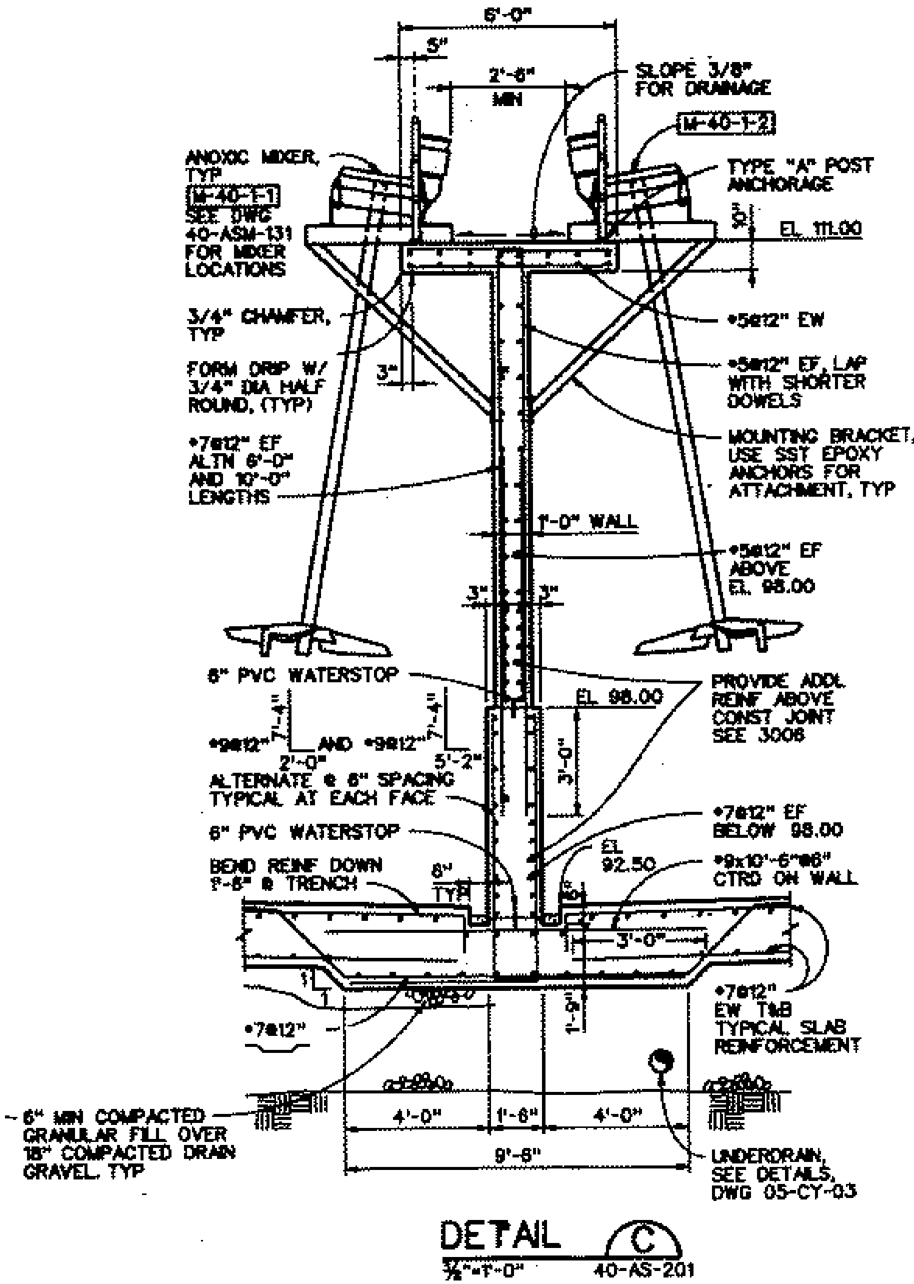
DCR = 0.42

City of Wilsonville

Aeration Basins Structural Calculations



RECORD DRAWINGS
Aeration Basin Dividing Wall Evaluation
AREA A
B
AREA C
AREA D
FOUNDATION PLAN - OVERALL
CH2MHILL.
STRUCTURAL FOUNDATION PLAN OVERALL

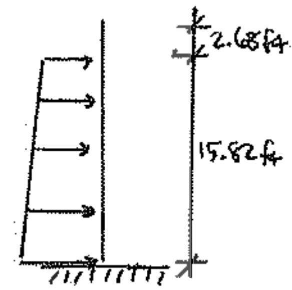


Dividing Wall Section Reinforcing

BY BS DATE 7/8/21 SUBJECT City of Wilsonville SHEET NO. ____ OF ____
CHKD. BY ____ DATE ____ Aeration Basins JOB NO. 11962A.00

Aeration Basins - Dividing Wall Check

The existing aeration basin dividing wall between Aeration Basins 1 & 2 will be checked for the seismic loads. Since there is water present on both sides, the wall will be checked for the hydrodynamic load. The wall thickness is 18" at base and extends 5'-6" with #9@6". Above this, the wall thickness is 12" with #7@12".



See attached spreadsheet for hydrodynamic loading.

Wall will be assumed to act as a cantilever. Force is at BSE-2E level.
Checking wall strength at base (#9@6" vert reinforcing)

$M_u = 50.81$ k·ft/ft　　$\phi M_n = 135.29$ k·ft/ft
$V_u = 7.78$ k/ft　　$\phi V_n = 22.77$ k/ft

Moment DCR $= \frac{50.81}{135.29} = 0.38$ (ok)
Shear DCR $= \frac{7.78}{22.77} = 0.34$ (ok)

Checking wall strength at start of 12" wall (#7@12" vert reinforcing)

$M_u = 22.08$ k·ft/ft　　$\phi M_n = 25.68$ k·ft/ft
$V_u = 4.50$ k/ft　　$\phi V_n = 13.66$ k/ft

Moment DCR $= \frac{22.08}{25.68} = 0.86$ (ok)
Shear DCR $= \frac{4.50}{13.66} = 0.33$ (ok)

Checking freeboard height in basin. For Risk Category III, $\delta = 0.7 * d_{max}$

$\delta_{transverse} = 0.7(2.15\text{ ft}) = 1.51$ ft
$\delta_{longitudinal} = 0.7(3.40\text{ ft}) = 2.38$ ft
freeboard height: 2.68ft

2.68ft > 1.51ft (ok) Freeboard is sufficient.
2.68ft > 2.38 ft (ok) Freeboard is sufficient.

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BY BS DATE 7/8/21 SUBJECT City of Wilsonville SHEET NO. ___ OF ___
CHKD. BY ___ DATE ___ Aeration Basins JOB NO. 11962A.00

Checking wall strength at base of 18" wall. Forces are at CSZ seismic level.

M_u = 38.63 k·ft/ft ϕM_n = 135.29 k·ft/ft

V_u = 5.65 k/ft ϕV_n = 22.77 k/ft

Moment DCR = $\frac{38.63\ k\cdot ft/ft}{135.29\ k\cdot ft/ft}$ = 0.29 (ok)

Shear DCR = $\frac{5.65\ k/ft}{22.77\ k/ft}$ = 0.25 (ok)

Checking wall strength at start of 12" wall.

M_u = 17.08 k·ft/ft ϕM_n = 25.68 k·ft/ft

V_u = 3.50 k/ft ϕV_n = 13.66 k/ft

Moment DCR = $\frac{17.08\ k\cdot ft/ft}{25.68\ k\cdot ft/ft}$ = 0.67 (ok)

Shear DCR = $\frac{3.50\ k/ft}{13.66\ k/ft}$ = 0.26 (ok)

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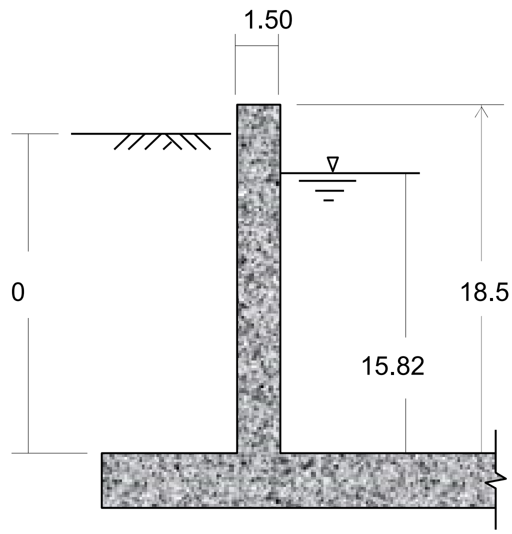
BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:		Aeration Basin Hydrodynamic Pressures (BSE-2E Seismic Level)					

Static, Seismic, and Hydrodynamic Seismic Wall Pressures using ACI 350.3-06 and an IBC 2012:

wall connection fixity =	**no roof & fixed at floor**	
tank unit width perpendicular to EQ., B =	1	ft
tank inside length in direction of seismic, L =	**19.75**	ft
tank wall thickness, t_w =	**18**	inch
wall height, H_w =	**18.5**	ft
liquid height, H_L =	**15.82**	ft
liquid specific gravity =	**1**	
liquid density, $\gamma_L = (\text{sp.gr.})^*\gamma_w$ =	0.0624	k/ft^3
acceleration due to gravity, g =	32.17	ft/sec^2
liquid mass density, $\rho_L = \gamma_L / g$ =	0.00194	$k\text{-}sec^2/ft^4$



WALL SECTION

Soil Data

The site has groundwater present.

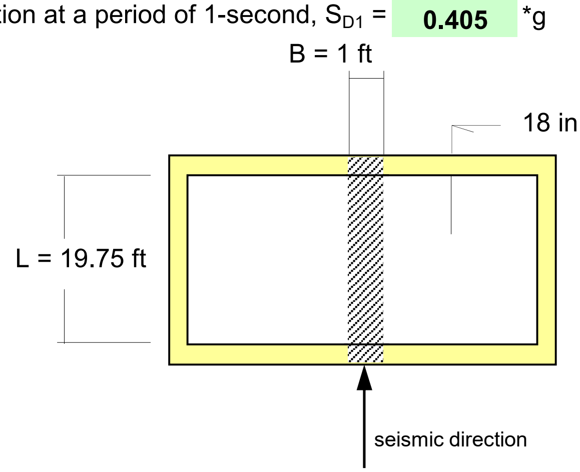
soil height above top of foundation base =	**0**	ft
groundwater ht. above foundation base =	**0**	ft
dry soil lateral pressure =	**0**	k/ft^3
saturated soil lateral pressure =	**0**	k/ft^3
dry soil unit weight =	**0**	k/ft^3
live load lateral surcharge =	**0.000**	ksf
	0	
concrete strength, f'_c =	**4**	ksi
concrete density, γ_c =	0.150	k/ft^3
concrete modulus of elasticity, E_c =	3605.0	ksi
concrete mass density, $\rho_c = \gamma_c / g$ =	0.004663	$k\text{-}sec^2/ft^4$

Seismic:

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, S_{DS} = **0.744** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, S_{D1} = **0.405** *g

Structure Risk Category =	**2**
Importance factor, I =	**1**
Response modification factor, R_{wi} =	**3**
Response modification factor, R_{wc} =	**1**



WALL PLAN

Load Cases:

case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

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DESIGN TASK:		Aeration Basin Hydrodynamic Pressures (BSE-2E Seismic Level)					

Weights:

unit 1-ft width wall mass, W_w = (18/12) * (18.5) * 0.15 = 4.16 kip

wall c.g. relative to base, h_w = 18.5 / 2 = 9.250 ft

unit width liquid mass, W_L = (19.75) * (1) * (15.82) * 32.17 = 19.50 kip

Seismic:

1). structure stiffness and dynamic property:

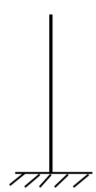
Note: per ASCE 7-10 and IBC 2012, the terms S_{ai} and S_{ac} have been appropriatetly substituted into the seismic equation of ACI 350.

Note: Wi and hi are impulsive component variables calculated on page 3.

wall mass, $m_w = H_w*(t_w / 12)*\rho_c$ = 0.12939 k-sec^2/ft^2

liquid mass, $m_i = (W_i / W_L)*(L/2)*H_L*\rho_L$ = 0.22237 k-sec^2/ft^2

centroidal distance of masses, $h = (h_w*m_w + h_i*m_i) / (m_w + m_i)$ = 7.232 ft



wall fixity condition is no roof & fixed at floor:

wall stiffness is determined using a unit mass load located at the centroidal distance h.

wall flexure stiffness, k = Ec*(tw/h)³ / 48 = 1157.99 k/ft/ft

$$\omega_i = \sqrt{\frac{k}{m_w + m_i}} =$$ (1157.99 / (0.1294 + 0.2224))ˆ½ = 57.3756 rad/sec

period of tank plus impulsive mass, $T_i = 2\pi / \omega_i$ = 2 π / 57.3756 = 0.1095 sec

(note: acceleration values to be from a maximum considered earthquake response spectra which will produce a factored load)

design factored spectral response acceleration for impulsive mass (5% damping), $S_{ai} = S_{DS}$ = 0.744 g

2). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$\lambda = \sqrt{3.16\ g\ \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)} =$$ (3.16*32.2*tanh(3.16*(0.801)))ˆ½ = 10.0189

$$\omega_c = \frac{\lambda}{\sqrt{L}} =$$ 10.0189 / (19.75)ˆ½ = 2.2544 rad/sec,

period of the convective mass, $T_c = 2\pi / \omega_c$ = 2 π / 2.2544 = 2.7870 sec

Long transition period (from map figure 22-15 ASCE 7), T_L = **16** sec

design spectral response acceleration for convective mass (0.5% damping), S_{ac} = 1.5 * Sd1 / Tc = 0.218 g

$$\text{effective mass coeff., } \varepsilon = 0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021\ ,\quad \text{but } \le\ 1.0\ =$$ 0.8063

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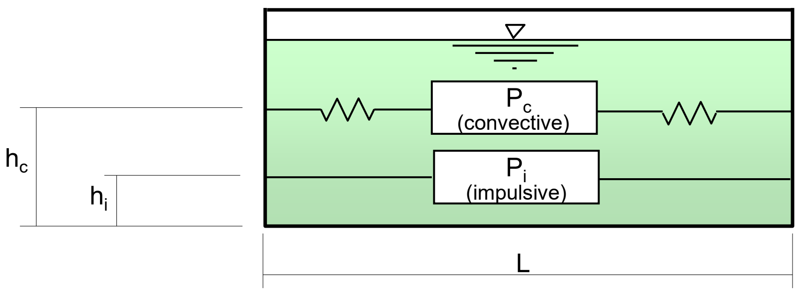
DESIGN TASK:		Aeration Basin Hydrodynamic Pressures (BSE-2E Seismic Level)					



Dynamic Model

L =	19.75	ft
B =	1	ft
H_L =	15.82	ft
W_L =	19.5	kip
L / H_L =	1.24842	
H_L / L =	0.80101	

3). lateral fluid impulsive force:

Wi = equivalent mass of the impulsive component of liquid.

$$W_i = W_L\left(\frac{\tanh\left(0.866\frac{L}{H_L}\right)}{0.866\frac{L}{H_L}}\right) =$$ 19.5*(tanh(0.866*(1.2484)) / 0.866*(1.2484)) = 14.31 kip

hi (EBP) = HL * (0.5-0.09375*(L/HL)) = 15.82*(0.5-0.09375*(1.2484)) = 6.058 ft

hi (IBP) = HL * {{(0.866*L/HL)/(2*tanh(0.866*L/HL))} -1/8 } = 8.798 ft

impulsive force, $P_i = \left(\frac{S_{ai}\ I}{R_{wi}}\right)W_i =$ (0.744 * 1 / 3)*14.31 = 3.5 kip

4). lateral fluid convective force:

Wc = equivalent mass of the convective component of liquid.

$$W_c = W_L\left(0.264\left(\frac{L}{H_L}\right)\tanh\left(3.16\left(\frac{H_L}{L}\right)\right)\right) =$$ 19.5*(0.264*(1.2484)*tanh(3.16*(0.801))) = 6.35 kip

$$h_{c\ (EBP)} = H_L\left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 1}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)}\right) = 10.491 \text{ ft}$$

$$h_{c\ (IBP)} = H_L\left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)}\right) = 11.502 \text{ ft}$$

convective force, $P_c = \left(\frac{S_{ac}\ I}{R_{wc}}\right)W_c =$ (0.218 * 1 / 1)*6.35 = 1.4 kip

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DESIGN TASK:		Aeration Basin Hydrodynamic Pressures (BSE-2E Seismic Level)					

5). <u>lateral inertia force of the accelerating wall:</u>

unit width wall mass, W_w = 4.16 kip

wall c.g. relative to base, h_w = 9.250 ft

$$\text{wall inertia force, } P_w = \left(\frac{S_{ai}\ I\ \varepsilon}{R_{wi}}\right) W_w = \qquad (0.744*1*0.8063/3)*4.16 = \quad 0.83 \text{ kip}$$

6). <u>maximum wave slosh height displacement:</u>

$$d_{(max)} = \left(\frac{L}{2}\right)\left(\frac{S_{ac}}{1.0}\ I\right) = \qquad (19.75 / 2) * (0.218 / 1.0 * 1) = \quad 2.15 \text{ ft}$$

7). <u>vertical acceleration:</u>

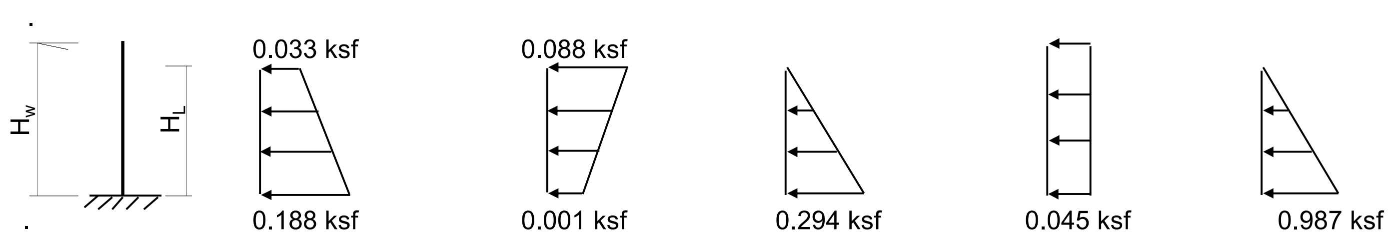
design horizontal accereration, S_{DS} = 0.744 *g

vertical spectral response acceleration (per ACI 350 para 9.4.3), $S_{av} = C_t = 0.4*S_{DS}$ = 0.2976 g

per ASCE 7-10 para. 15.7.7.2(b), use $I = R_i = b = 1.0$

$$\text{Design vertical acceleration, } \ddot{u} = \frac{S_{av}\ I\ b}{R_i} = \qquad 0.2976*1*1/1 = \quad 0.2976 \text{ g}$$

<u>8). vertical force distribution on a unit width using the linear distribution of ACI 350 sec 5.3:</u>



H_w = 18.5

H_L = 15.82

<u>impulsive:</u>

$$p_{iy} = \frac{P_i\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_i = 3.50 kip

h_i = 6.058 ft

at $y = H_L$, p_{iy} = 0.033 ksf

at base $y = 0$, p_{iy} = 0.188 ksf

<u>convective:</u>

$$p_{cy} = \frac{P_c\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_c = 1.40 kip

h_c = 10.491 ft

at $y = H_L$, p_{cy} = 0.088 ksf

at base $y = 0$, p_{cy} = 0.001 ksf

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DESIGN TASK:		Aeration Basin Hydrodynamic Pressures (BSE-2E Seismic Level)					

vertical acceleration:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

$\ddot{u}$ = 0.2976
at $y = H_L$, p_{vy} = 0.000 ksf
at base $y = 0$, p_{vy} = 0.294 ksf

wall inertia:

$$p_{wy} = \frac{S_{ai}\ \ I\ \ \varepsilon\ \gamma_c\ (t_w/12)}{R_{wi}} =$$

p_{wy} = 0.2000 * γ_c * $(t_w/12)$
at $y = H_w$, p_{wy} = 0.045 ksf
at base $y = 0$, p_{wy} = 0.045 ksf

hydrostatic:

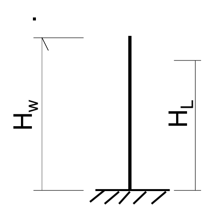
$$q_{hy} = \gamma_L\ (H_L - y) =$$

at $y = H_L$, q_{hy} = 0.000 ksf
at base $y = 0$, q_{hy} = 0.987 ksf

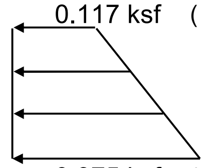
combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{(p_{iy} + p_{wy})^2 + p_{cy}^2 + p_{vy}^2} =$$

at $y = H_w$, p_y = 0.117 ksf
at base $y = 0$, p_y = 0.375 ksf



0.117 ksf (unfactored = 0.117 / 1.4 = 0.084 ksf)

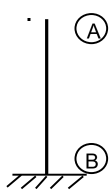


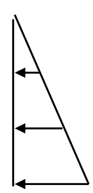
0.375 ksf (unfactored = 0.375 / 1.4 = 0.268 ksf)

resultant dynamic pressures

9). wall design pressures for hydrostatic + dynamic:

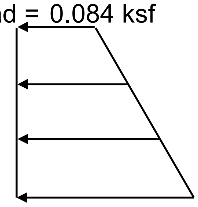
wall height, H_w = 18.5 ft
liquid height, H_L = 15.82 ft





0.987 ksf
hydrostatic

unfactored load = 0.084 ksf



unfactored load = 0.268 ksf
resultant dynamic pressures

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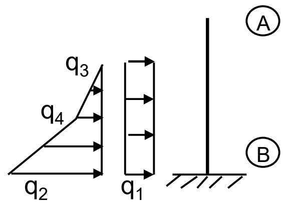
10). wall design pressures for external soil loading:

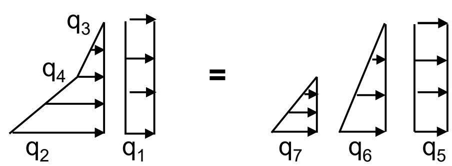
static soil:



The site has groundwater present.

wall height =	18.5	ft
soil height above top of base =	0	ft
groundwater ht. above base =	0	ft
dry soil lateral pressure =	0.000	k/ft^3
sat. soil lateral pressure =	0.000	k/ft^3
live load lateral surcharge =	0.000	ksf

equivalent static soil loadings:



LL lateral surcharge, q1 =	0.0000	ksf
unfactored soil, q2 =	0.0000	ksf
unfactored soil, q3 =	0.0000	ksf
unfactored soil, q4 =	0.000	ksf
equivalent soil loadings:		
unfactored q5 =	0.0000	ksf
unfactored q6 =	0.0000	ksf
unfactored q7 =	0.0000	ksf

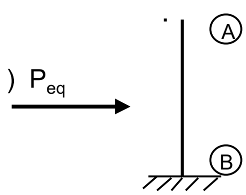
soil seismic:

resultant factored soil seismic load per foot of wall width, $P_{u\,(eq)}$ = **0** k/ft

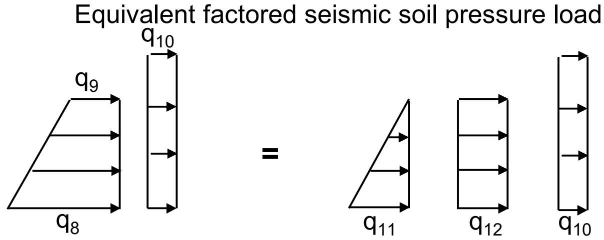
centroid location of the resultant soil seismic from the bottom of wall, h_{eq} = **0** ft

The resultant soil seismic load will be resolved into an equivalent pressure loading…

(resultant soil seismic load) P_{eq}



Equivalent factored seismic soil pressure loading & seismic wall loadings…



equivalent soil seismic, q8 =	0.0000	ksf
equivalent soil seismic, q9 =	0.0000	ksf
wall seismic (see wall page 5), q10 =	0.0450	ksf
equivalent soil seismic, q11 = q8 - q9 =	0.0000	ksf
equivalent soil seismic, q12 = q9 =	0.0000	ksf

unfactored equivalent soil seismic, q8 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q9 =0 / 1.4 =	0.0000	ksf
unfactored wall seismic , q10 = 0.045 / 1.4 =	0.0321	ksf
unfactored equivalent soil seismic, q11 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q12 =0 / 1.4 =	0.0000	ksf

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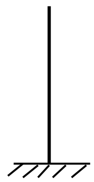
DESIGN TASK:		Aeration Basin Hydrodynamic Pressures (BSE-2E Seismic Level)					

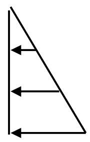
11). Summary of equivalent unfactored pressure loadings for wall load cases 1 thru 4:



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

a). load case 1: hydrostatic water

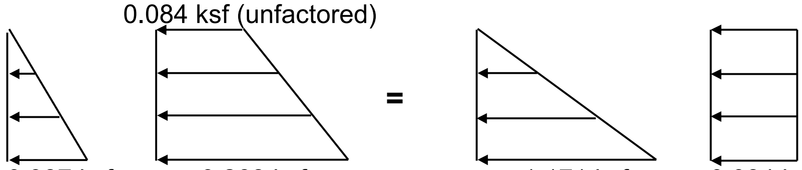
wall height = 18.5 ft
water depth = 15.82 ft



(unfactored) 0.987 ksf

b). load case 2: hydrostatic + dynamic:

0.084 ksf (unfactored)



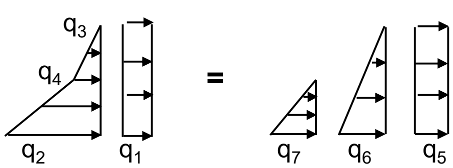
0.987 ksf (unfactored) hydrostatic	0.268 ksf (unfactored) resultant dynamic pressures	=	1.171 ksf (unfactored)	0.084 ksf (unfactored)

wall height = 18.5 ft
water depth = 15.82 ft

c). load case 3: static soil + LL surcharge:

wall height = 18.5 ft
soil height on wall = 0 ft
groundwater height = 0 ft

equivalent static soil & surcharge loadings...

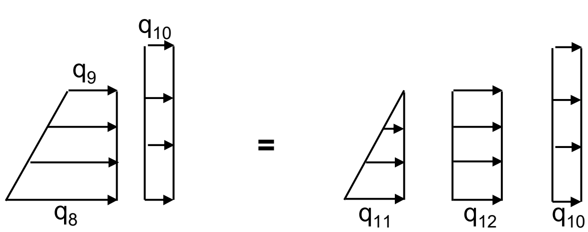


LL lateral surcharge, q1 =	0.000	ksf
unfactored soil, q2 =	0.000	ksf
unfactored soil, q3 =	0.000	ksf
unfactored soil, q4 =	0.000	ksf
equivalent soil loadings:		
unfactored q5 =	0.000	ksf
unfactored q6 =	0.000	ksf
unfactored q7 =	0.000	ksf

d). load case 4: soil seismic: (*note: add static soil pressure q6 & q7 to the seismic soil shown below)

equivalent seismic soil pressure loading & seismic wall loadings…

wall height = 18.5 ft
soil height on wall = 0 ft



unfactored equivalent soil seismic, q8 =	0.000	ksf
unfactored equivalent soil seismic, q9 =	0.000	ksf
unfactored equivalent soil seismic, q10 =	0.032	ksf
unfactored equivalent soil seismic, q11 =	0.000	ksf
unfactored equivalent soil seismic, q12 =	0.000	ksf

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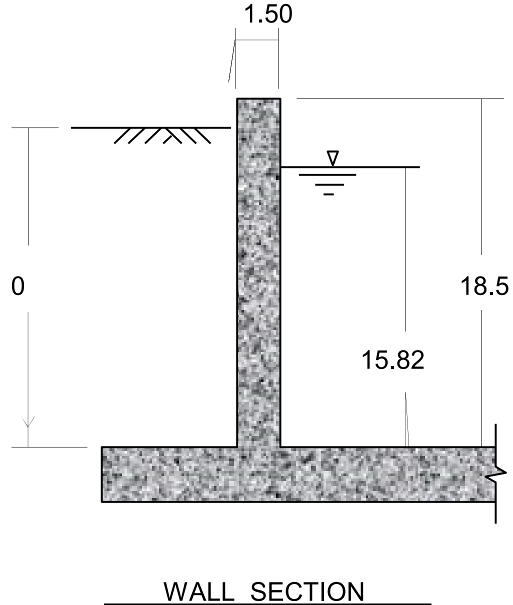
DESIGN TASK:	Aeration Basin Hydrodynamic Pressures (BSE-2E Seismic Level - Longitudinal Direction)						

Static, Seismic, and Hydrodynamic Seismic Wall Pressures using ACI 350.3-06 and an IBC 2012:

wall connection fixity =	**no roof & fixed at floor**	
tank unit width perpendicular to EQ., B =	1	ft
tank inside length in direction of seismic, L =	**175**	ft
tank wall thickness, t_w =	**18**	inch
wall height, H_w =	**18.5**	ft
liquid height, H_L =	**15.82**	ft
liquid specific gravity =	**1**	
liquid density, γ_L = (sp.gr.)*γ_w =	0.0624	k/ft^3
acceleration due to gravity, g =	32.17	ft/sec^2
liquid mass density, $\rho_L = \gamma_L / g$ =	0.00194	$k\text{-}sec^2/ft^4$



Soil Data

The site has groundwater present.

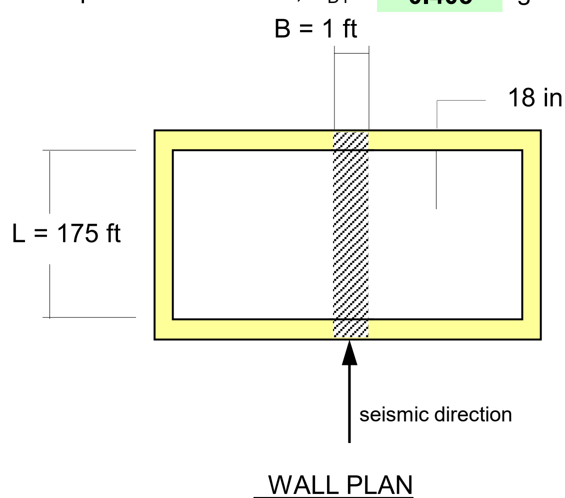
soil height above top of foundation base =	**0**	ft
groundwater ht. above foundation base =	**0**	ft
dry soil lateral pressure =	**0**	k/ft^3
saturated soil lateral pressure =	**0**	k/ft^3
dry soil unit weight =	**0**	k/ft^3
live load lateral surcharge =	**0.000**	ksf
	0	
concrete strength, f'_c =	**4**	ksi
concrete density, γ_c =	0.150	k/ft^3
concrete modulus of elasticity, E_c =	3605.0	ksi
concrete mass density, $\rho_c = \gamma_c / g$ =	0.004663	$k\text{-}sec^2/ft^4$

Seismic:

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, S_{DS} = **0.744** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, S_{D1} = **0.405** *g

Structure Risk Category =	**2**
Importance factor, I =	1
Response modification factor, R_{wi} =	**3**
Response modification factor, R_{wc} =	**1**



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

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BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:		Aeration Basin Hydrodynamic Pressures (BSE-2E Seismic Level - Longitudinal Direction)					

Weights:

unit 1-ft width wall mass, W_w = (18/12) * (18.5) * 0.15 = 4.16 kip

wall c.g. relative to base, h_w = 18.5 / 2 = 9.250 ft

unit width liquid mass, W_L = (175) * (1) * (15.82) * 32.17 = 172.75 kip

Seismic:

1). structure stiffness and dynamic property:

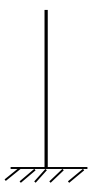
Note: per ASCE 7-10 and IBC 2012, the terms S_{ai} and S_{ac} have been appropriatetly substituted into the seismic equation of ACI 350.

Note: Wi and hi are impulsive component variables calculated on page 3.

wall mass, $m_w = H_w*(t_w / 12)*\rho_c$ = 0.12939 k-sec^2/ft^2

liquid mass, $m_i = (W_i / W_L)*(L/2)*H_L*\rho_L$ = 0.28024 k-sec^2/ft^2

centroidal distance of masses, $h = (h_w*m_w + h_i*m_i) / (m_w + m_i)$ = 6.981 ft



wall fixity condition is no roof & fixed at floor:

wall stiffness is determined using a unit mass load located at the centroidal distance h.

wall flexure stiffness, k = Ec*(tw/h)³ / 48 = 1287.44 k/ft/ft

$$\omega_i = \sqrt{\frac{k}{m_w + m_i}} =$$ (1287.44 / (0.1294 + 0.2802))^½ = 56.0621 rad/sec

period of tank plus impulsive mass, $T_i = 2\pi / \omega_i$ = 2 π / 56.0621 = 0.1121 sec

(note: acceleration values to be from a maximum considered earthquake response spectra which will produce a factored load)

design factored spectral response acceleration for impulsive mass (5% damping), $S_{ai} = S_{DS}$ = 0.744 g

2). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$\lambda = \sqrt{3.16\ g\ \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)} =$$ (3.16*32.2*tanh(3.16*(0.0904)))^½ = 5.3174

$$\omega_c = \frac{\lambda}{\sqrt{L}} =$$ 5.3174 / (175)^½ = 0.4020 rad/sec,

period of the convective mass, $T_c = 2\pi / \omega_c$ = 2 π / 0.402 = 15.6314 sec

Long transition period (from map figure 22-15 ASCE 7), T_L = **16** sec

design spectral response acceleration for convective mass (0.5% damping), S_{ac} = 1.5 * Sd1 / Tc = 0.039 g

$$\text{effective mass coeff., } \varepsilon = 0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021\ ,\ \text{ but } \le 1.0 =$$ 0.7581

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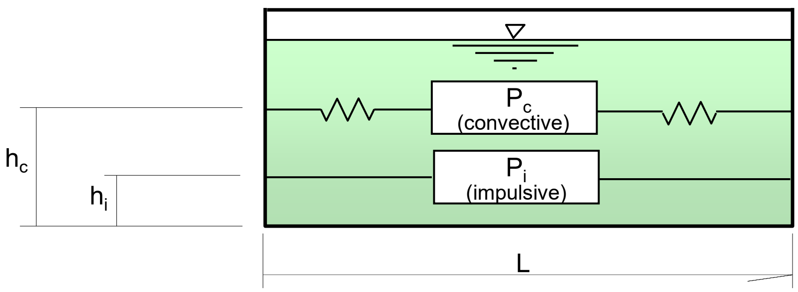
DESIGN TASK:		Aeration Basin Hydrodynamic Pressures (BSE-2E Seismic Level - Longitudinal Direction)					



Dynamic Model

L =	175	ft
B =	1	ft
H_L =	15.82	ft
W_L =	172.75	kip
L / H_L =	11.06195	
H_L / L =	0.09040	

3). lateral fluid impulsive force:

Wi = equivalent mass of the impulsive component of liquid.

$$W_i = W_L \left(\frac{\tanh\left(0.866 \frac{L}{H_L}\right)}{0.866 \frac{L}{H_L}} \right) = \quad 172.75*(\tanh(0.866*(11.0619)) / 0.866*(11.0619)) = \quad 18.03 \text{ kip}$$

hi (EBP) = HL * 0.375 = 15.82 * 0.375 = 5.933 ft

hi (IBP) = HL * {{(0.866*L/HL)/(2*tanh(0.866*L/HL))} -1/8 } = 73.798 ft

$$\text{impulsive force, } P_i = \left(\frac{S_{ai} \; I}{R_{wi}} \right) W_i = \quad (0.744 * 1 / 3)*18.03 = \quad 4.5 \text{ kip}$$

4). lateral fluid convective force:

Wc = equivalent mass of the convective component of liquid.

$$W_c = W_L \left(0.264 \left(\frac{L}{H_L} \right) \tanh\left(3.16 \left(\frac{H_L}{L} \right) \right) \right) = \quad 172.75*(0.264*(11.0619)*\tanh(3.16*(0.0904))) = \quad 140.32 \text{ kip}$$

$$h_{c\ (EBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 1}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) = \quad 7.963 \text{ ft}$$

$$h_{c\ (IBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) = \quad 201.127 \text{ ft}$$

$$\text{convective force, } P_c = \left(\frac{S_{ac} \; I}{R_{wc}} \right) W_c = \quad (0.0389 * 1 / 1)*140.32 = \quad 5.5 \text{ kip}$$

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DESIGN TASK:	Aeration Basin Hydrodynamic Pressures (BSE-2E Seismic Level - Longitudinal Direction)						

5). lateral inertia force of the accelerating wall:

unit width wall mass, W_w = 4.16 kip

wall c.g. relative to base, h_w = 9.250 ft

$$\text{wall inertia force, } P_w = \left(\frac{S_{ai}\ I\ \varepsilon}{R_{wi}}\right) W_w = \quad (0.744*1*0.7581/3)*4.16 = \quad 0.78 \text{ kip}$$

6). maximum wave slosh height displacement:

$$d_{(max)} = \left(\frac{L}{2}\right)\left(\frac{S_{ac}}{1.0}\ I\right) = \quad (175 / 2) * (0.0389 / 1.0 * 1) = \quad 3.40 \text{ ft}$$

7). vertical acceleration:

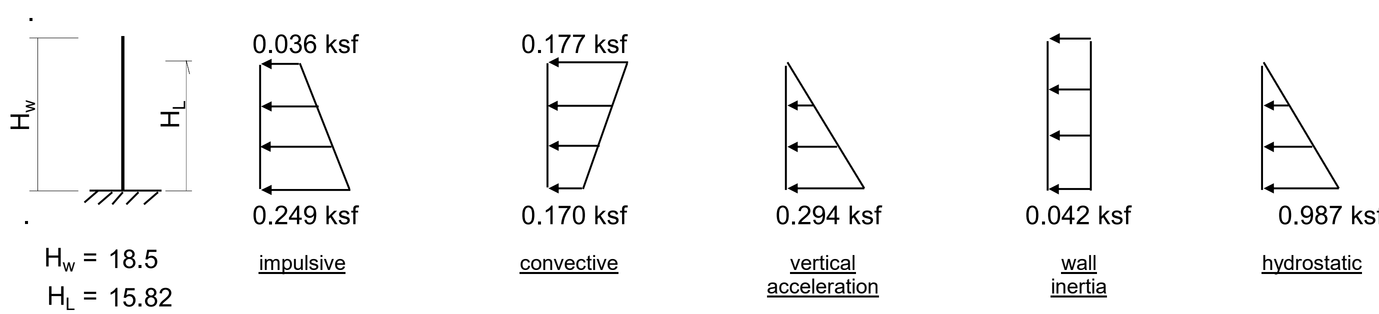
design horizontal accereration, S_{DS} = 0.744 *g

vertical spectral response acceleration (per ACI 350 para 9.4.3), $S_{av} = C_t = 0.4*S_{DS}$ = 0.2976 g

per ASCE 7-10 para. 15.7.7.2(b), use $I = R_i = b = 1.0$

$$\text{Design vertical acceleration, } \ddot{u} = \frac{S_{av}\ I\ b}{R_i} = \quad 0.2976*1*1/1 = \quad 0.2976 \text{ g}$$

8). vertical force distribution on a unit width using the linear distribution of ACI 350 sec 5.3:



H_w = 18.5

H_L = 15.82

impulsive:

$$p_{iy} = \frac{P_i\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_i = 4.50 kip

h_i = 5.933 ft

at $y = H_L$, p_{iy} = 0.036 ksf

at base $y = 0$, p_{iy} = 0.249 ksf

convective:

$$p_{cy} = \frac{P_c\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_c = 5.50 kip

h_c = 7.963 ft

at $y = H_L$, p_{cy} = 0.177 ksf

at base $y = 0$, p_{cy} = 0.170 ksf

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vertical acceleration:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

ü = 0.2976
at y = H_L, p_{vy} = 0.000 ksf
at base y = 0, p_{vy} = 0.294 ksf

wall inertia:

$$p_{wy} = \frac{S_{ai}\ \ I\ \ \varepsilon\ \gamma_c\ (t_w/12)}{R_{wi}} =$$

p_{wy} = 0.1880 * γ_c * (t_w/12)
at y = H_w, p_{wy} = 0.042 ksf
at base y = 0, p_{wy} = 0.042 ksf

hydrostatic:

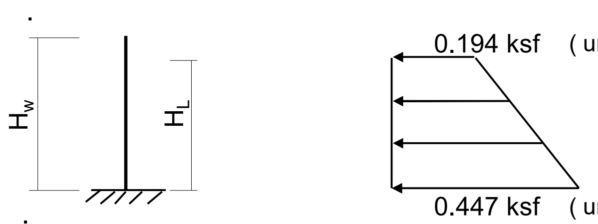
$$q_{hy} = \gamma_L\ (H_L - y) =$$

at y = H_L, q_{hy} = 0.000 ksf
at base y = 0, q_{hy} = 0.987 ksf

combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{\left(p_{iy} + p_{wy}\right)^2 + p_{cy}^2 + p_{vy}^2} =$$

at y = H_w, p_y = 0.194 ksf
at base y = 0, p_y = 0.447 ksf



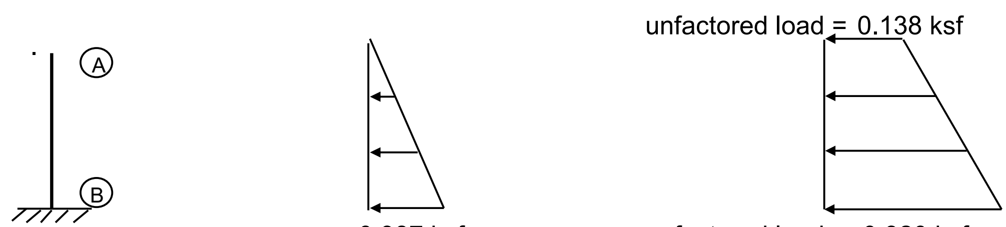
(unfactored = 0.194 / 1.4 = 0.138 ksf)

(unfactored = 0.447 / 1.4 = 0.32 ksf)

resultant dynamic pressures

9). wall design pressures for hydrostatic + dynamic:

wall height, H_w = 18.5 ft
liquid height, H_L = 15.82 ft



hydrostatic

resultant dynamic pressures

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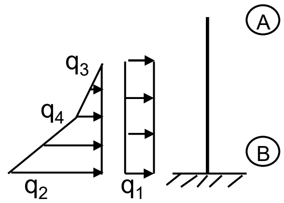
DESIGN TASK:		Aeration Basin Hydrodynamic Pressures (BSE-2E Seismic Level - Longitudinal Direction)					

10). wall design pressures for external soil loading:

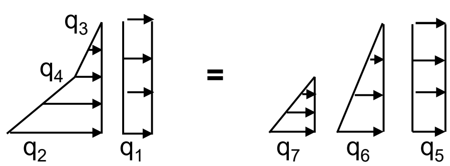
static soil:



The site has groundwater present.

wall height =	18.5	ft
soil height above top of base =	0	ft
groundwater ht. above base =	0	ft
dry soil lateral pressure =	0.000	k/ft^3
sat. soil lateral pressure =	0.000	k/ft^3
live load lateral surcharge =	0.000	ksf

equivalent static soil loadings:



LL lateral surcharge, q1 =	0.0000	ksf
unfactored soil, q2 =	0.0000	ksf
unfactored soil, q3 =	0.0000	ksf
unfactored soil, q4 =	0.000	ksf
equivalent soil loadings:		
unfactored q5 =	0.0000	ksf
unfactored q6 =	0.0000	ksf
unfactored q7 =	0.0000	ksf

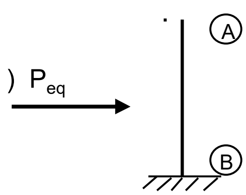
soil seismic:

resultant factored soil seismic load per foot of wall width, $P_{u\,(eq)}$ = **0** k/ft

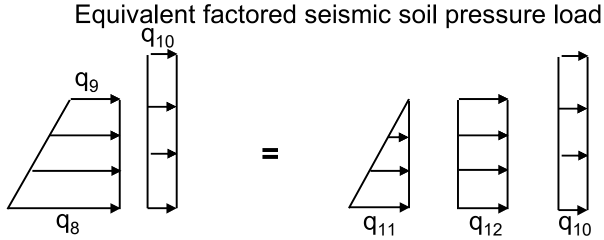
centroid location of the resultant soil seismic from the bottom of wall, h_{eq} = **0** ft

The resultant soil seismic load will be resolved into an equivalent pressure loading…

(resultant soil seismic load) P_{eq}



Equivalent factored seismic soil pressure loading & seismic wall loadings…



equivalent soil seismic, q8 =	0.0000	ksf
equivalent soil seismic, q9 =	0.0000	ksf
wall seismic (see wall page 5), q10 =	0.0423	ksf
equivalent soil seismic, q11 = q8 - q9 =	0.0000	ksf
equivalent soil seismic, q12 = q9 =	0.0000	ksf

unfactored equivalent soil seismic, q8 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q9 =0 / 1.4 =	0.0000	ksf
unfactored wall seismic , q10 = 0.0423 / 1.4 =	0.0302	ksf
unfactored equivalent soil seismic, q11 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q12 =0 / 1.4 =	0.0000	ksf

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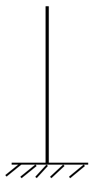
DESIGN TASK:	Aeration Basin Hydrodynamic Pressures (BSE-2E Seismic Level - Longitudinal Direction)						

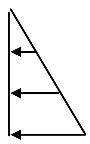
11). **Summary of equivalent unfactored pressure loadings for wall load cases 1 thru 4:**



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

a). load case 1: hydrostatic water

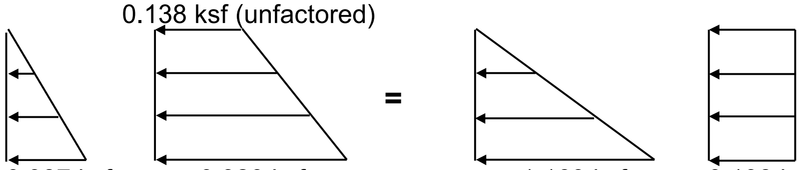
wall height = 18.5 ft
water depth = 15.82 ft



(unfactored) 0.987 ksf

b). load case 2: hydrostatic + dynamic:

0.138 ksf (unfactored)



wall height = 18.5 ft
water depth = 15.82 ft

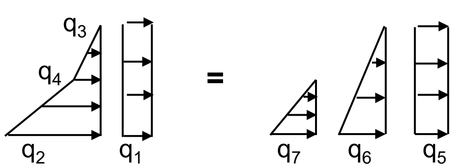
0.987 ksf (unfactored) hydrostatic	0.320 ksf (unfactored) resultant dynamic pressures	=	1.169 ksf (unfactored)	0.138 ksf (unfactored)

c). load case 3: static soil + LL surcharge:

wall height = 18.5 ft
soil height on wall = 0 ft
groundwater height = 0 ft

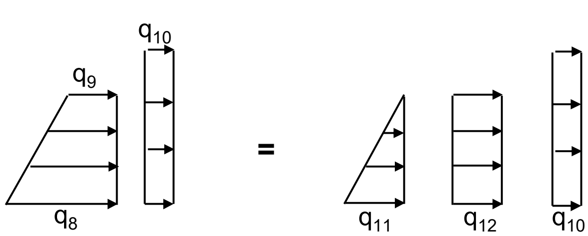
equivalent static soil & surcharge loadings...

LL lateral surcharge, q1 =	0.000	ksf
unfactored soil, q2 =	0.000	ksf
unfactored soil, q3 =	0.000	ksf
unfactored soil, q4 =	0.000	ksf
equivalent soil loadings:		
unfactored q5 =	0.000	ksf
unfactored q6 =	0.000	ksf
unfactored q7 =	0.000	ksf



d). load case 4: soil seismic: (*note: add static soil pressure q6 & q7 to the seismic soil shown below)
equivalent seismic soil pressure loading & seismic wall loadings…

wall height = 18.5 ft
soil height on wall = 0 ft



unfactored equivalent soil seismic, q8 =	0.000	ksf
unfactored equivalent soil seismic, q9 =	0.000	ksf
unfactored equivalent soil seismic, q10 =	0.030	ksf
unfactored equivalent soil seismic, q11 =	0.000	ksf
unfactored equivalent soil seismic, q12 =	0.000	ksf

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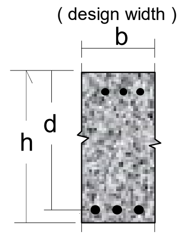
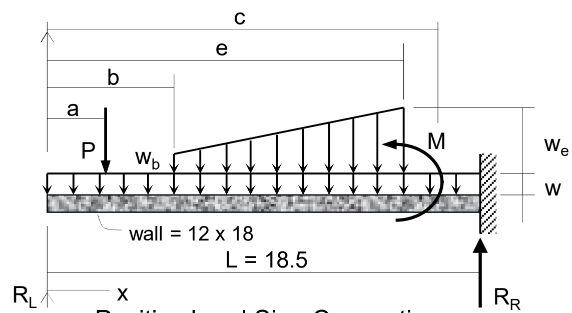
DESIGN TASK:		Dividing Wall between Aeration Basins 1&2 (Hydrodynamic Loading Only) (BSE-2E)					

Wall Data:

Design for a slab or a wall?	**Wall**	
Wall Support Fixity =	**Cantilever**	
Span, L =	**18.5**	ft
Wall width, b =	**12**	in
Thickness of Wall, h =	**18**	in
Depth to reinforcing, d =	**15**	in
concrete strength, f'_c =	**4**	ksi
reinforcing yield strength, f_y =	**60**	ksi
allow deflection, L/Δ =	**240**	

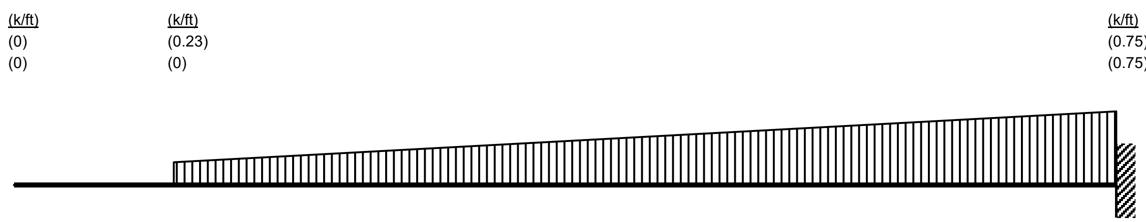


trial moment of Inertia, $I_x = 0.5\ I_g$ = 2916 in^4



Positive Load Sign Convention

Wall Loadings:



Externally Applied Service Loads to a Wall with Cantilever Support									
Uniform	Trapezoidal Loads				Point Loads		Moment Loads		Concrete Load Factors
	Begin		End						
w (k/ft)	b (ft)	w_b (k/ft)	e (ft)	w_e (k/ft)	a (ft)	P (kip)	c (ft)	M (ft-k)	
	2.68	**0.234**	**18.5**	**0.75**					**1**

optional environmental load factor for moment (ACI 350) = **1**

Results:

* Deflection is based on the trial moment of Inertia of 0.5*Ig.(actual deflection will be adjusted later on page 6.)

Calculated Reactions, Moments, and Deflections for the Wall with Cantilever Support									
Load Type	Reactions, R or R_u		Maximum Moments, M or M_u				* Maximum Short Term Deflections			
	Left End	Right End	Max Positive		Max Negative		downward		upward	
	R_L (kip)	R_R (kip)	x distance (ft)	+M (ft-k)	x distance (ft)	-M (ft-k)	x distance (ft)	Δ (in)	x distance (ft)	Δ (in)
service loads	0.00	7.78	0.000	0.00	18.500	-50.81	0.000	-0.584	18.500	0.000
factored loads	0.00	7.78	0.000	0.00	18.500	-50.81	Δ ≤ L/240 ,Okay		Δ ≤ L/240 ,Okay	
	environmental factor, M_u =			0.00		-50.81	Δmax = L/240 =	0.925 "		

Using # 9 @ 6" neg bars and # 9 @ 6" pos bars, the final adjusted long term deflection (see page 6), Δ = 0.788 "

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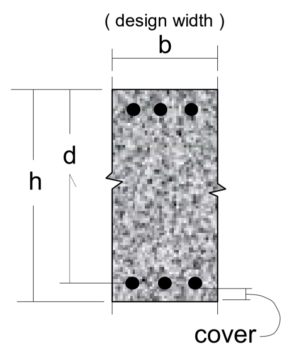
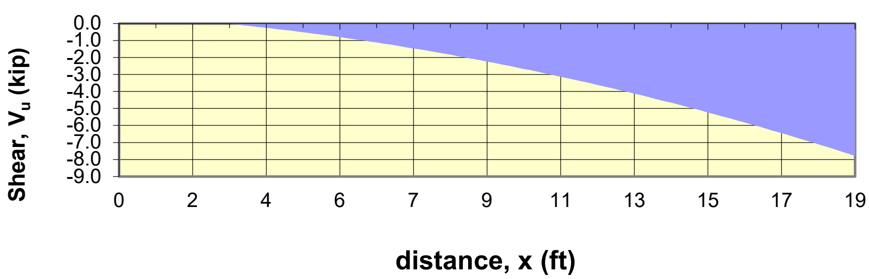
Wall Shear Capacity (Based on ACI 318, 11.2.1.1):

Maximum Shear, V_u =	7.78	kip	concrete, f'_c =	4	ksi
Wall width, b =	12	in	reinforcing, f_y =	60	ksi
Depth to reinforcing, d =	15	in	concrete modulus, $E_c = 57 * (f'_c)^{½}$ =	3605	ksi
Thickness of wall, h =	18	in	ϕ, Shear =	1.00	



Factored Shear Diagram



factored shear force, V_u = 7.780 kip

Concrete Shear Capacity, $\phi V_c = \phi * 2 * b * d * (f'_c)^{1/2}$ = 22.768 kip

øVc > Vu, OK

Minimum shrinkage-temperature requirement in the flexure direction:

wall minimum temperature / shrinkage steel ratio = **0.00500**

number of layers of reinforcement in the wall (1 or 2 ?) = **2**

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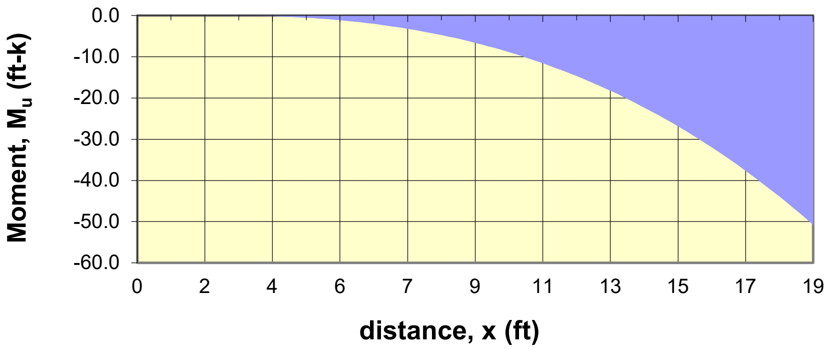
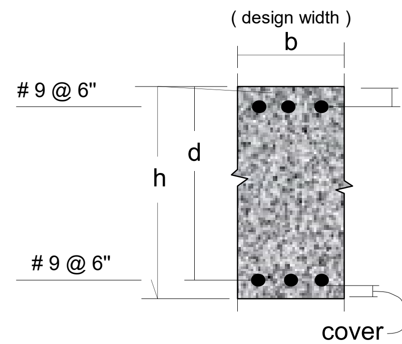
Wall Bending:

Service Moment, M(+) =	0.00	ft-k	concrete strength, f'_c =	4	ksi
Service Moment, M(-) =	50.81	ft-k	reinforcing yield strength, f_y =	60	ksi
Factored Moment, M_u(+) =	0.00	ft-k	concrete modulus, $E_c = 57 * (f'_c)^{½}$ =	3605	ksi
Factored Moment, M_u(-) =	50.81	ft-k	reinforcement modulus, E_s =	29000	ksi
Wall width, b =	12	in	$n = E_s / E_c$ =	8.044	
Depth to reinforcing, d =	15	in	β_1 =	0.85	
Thickness of wall, h =	18	in	ϕ, Bending =	0.9	

Factored Moment Diagram

(includes the 1 environmental factor)





Reinforcement

1). Negative Steel: (location at x = 18.5 ft)

Depth to negative reinforcing, d1 =	**2.5**	in
M_u(-) =	50.81	ft-k
Wall width, b =	12	in
Depth to reinforcing, d = h - d1 =	15.5	in
Area steel required, $A'_{s\,(req'd)}$ =	0.756	in²
Bar number size =	**# 9**	
Spacing of negative bars =	**6**	in
Area of steel provided, A'_s =	2.00	in²
Min area steel req'd, $A_{s\,(min)}$ =	0.62	in²
Max area allowed, $A'_{s\,(max)}$ =	3.98	in²

$R_u = M_u / (\phi*b*d^2) = 235.0$

$$\rho_{(req'd)} = \frac{0.85\,f'_c}{f_y}\left(1-\sqrt{1-\frac{2\,R_u}{0.85\,f'_c}}\right) = 0.00406$$

, $\rho = A_s / bd = 0.01075$

, $\rho_{(min)} = 0.00333$

, $\rho_{(max)} = 0.02138$

2). Positive Steel: (location at x = 0 ft)

concrete clear cover to positive steel =	**2**	in
M_u(+) =	0	ft-k
Wall width, b =	12	in
Depth to reinforcing, d =	15	in
Area steel required, $A_{s\,(req'd)}$ =	0.000	in²
Bar number size =	**# 9**	
Spacing of positive bars =	**6**	
Area of bottom steel provided, A_s =	2.00	in²
Min area steel req'd, $A_{s\,(min)}$ =	0.54	in²
Max area allowed, $A'_{s\,(max)}$ =	3.85	in²

$R_u = M_u / (\phi*b*d^2) = 0.0$

$$\rho_{(req'd)} = \frac{0.85\,f'_c}{f_y}\left(1-\sqrt{1-\frac{2\,R_u}{0.85\,f'_c}}\right) = 0.00000$$

, $\rho = A_s / bd = 0.01111$

, $\rho_{(min)} = 0.00300$

, $\rho_{(max)} = 0.02138$

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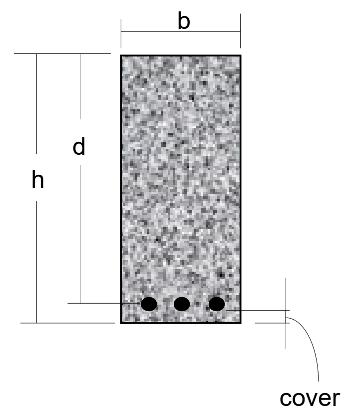
BY: BS	DATE: Aug-21	CLIENT: City of Wilsonville	SHEET:
CHKD:	DESCRIPTION: Aeration Basins		JOB NO: 11962A.00

DESIGN TASK: Dividing Wall Strength for 18" Thick Wall (Hydrodynamic Loading Only) (BSE-2E)

Concrete Shear and Moment Strength Capacity



Design for … beam, slab, wall ? **wall**

Properties and Geometry

Compression width of wall, b =	**12**	inch
Thickness of wall, h =	**18**	inch
Depth to reinforcing, d =	**15**	inch
factored design moment, M_u =	**50.81**	ft-k
factored design shear, V_u =	**7.78**	kip

f'_c (psi) =	**4000**
f_y (psi) =	**60000**
ϕ, Bending =	**1**
ϕ, Shear =	**1**
E_s (psi) =	29000000
E_c (psi) =	3604997
$n = E_s / E_c$ =	8.04
β_1 =	0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi * 2 * b * d * (f'_c)^{1/2}$ = 22.77 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **Existing 18" wall w/ #9@6"**

Area steel provided, A_s = **2** in^2 $\rho = A_s / bd$ = 0.01111

$\rho(min) < As/bd < \rho(max)$ - OK

$A_{s\ (min)}$ =	0.32	in^2	$\rho_{(min)}$ =	0.00180
$A_{s\ (max)}$ =	3.85	in^2	$\rho_{(max)}$ =	0.02138

bending strength, $\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$

$\phi * M_n$ = 1*0.01111*60*12*15² *(1-0.588*0.01111*60/4)*(ft/12) = 135.294 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

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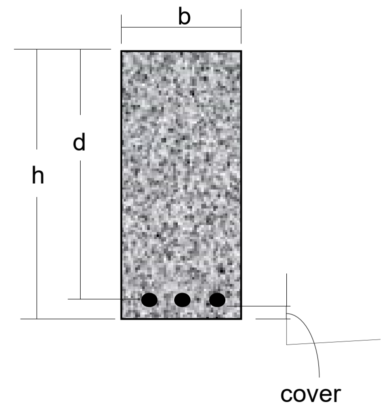
BY:	BS	**DATE:**	Aug-21	**CLIENT:** City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**		Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:	Dividing Wall Strength for 12" Thick Wall (Hydrodynamic Loading Only) (BS-2E)					

Concrete Shear and Moment Strength Capacity

Design for … beam, slab, wall ? **wall**



Properties and Geometry

Compression width of wall, b =	**12**	inch	f'_c (psi) =	**4000**
Thickness of wall, h =	**12**	inch	f_y (psi) =	**60000**
Depth to reinforcing, d =	**9**	inch	ϕ, Bending =	**1**
factored design moment, M_u =	**22.08**	ft-k	ϕ, Shear =	**1**
factored design shear, V_u =	**4.50**	kip	E_s (psi) =	29000000
			E_c (psi) =	3122019
			$n = E_s / E_c$ =	9.29
			β_1 =	0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi * 2 * b * d * (f'_c)^{1/2}$ = 13.66 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #7@12"**

Area steel provided, A_s = **0.6** in^2 $\rho = A_s / bd$ = 0.00556

$\rho(min) < As/bd < \rho(max)$ - OK

$A_{s\ (min)}$ = 0.19 in^2 $\rho_{(min)}$ = 0.00180

$A_{s\ (max)}$ = 1.73 in^2 $\rho_{(max)}$ = 0.01604

bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

$\phi * M_n$ = 1*0.00556*60*12*9² *(1-0.588*0.00556*60/3)*(ft/12) = 25.235 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

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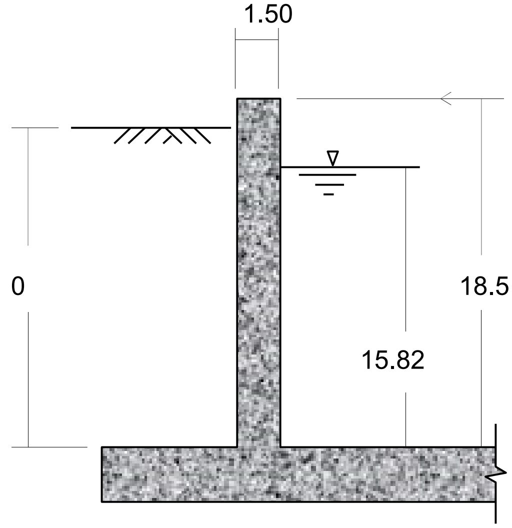
DESIGN TASK:			Aeration Basin Transverse Hydrodynamic Pressures (CSZ Seismic Level)				

Static, Seismic, and Hydrodynamic Seismic Wall Pressures using ACI 350.3-06 and an IBC 2012:

wall connection fixity =	**no roof & fixed at floor**	
tank unit width perpendicular to EQ., B =	1	ft
tank inside length in direction of seismic, L =	**19.75**	ft
tank wall thickness, t_w =	**18**	inch
wall height, H_w =	**18.5**	ft
liquid height, H_L =	**15.82**	ft
liquid specific gravity =	**1**	
liquid density, $\gamma_L = (sp.gr.)*\gamma_w$ =	0.0624	k/ft^3
acceleration due to gravity, g =	32.17	ft/sec^2
liquid mass density, $\rho_L = \gamma_L / g$ =	0.00194	$k\text{-}sec^2/ft^4$



WALL SECTION

Soil Data

The site has groundwater present.

soil height above top of foundation base =	**0**	ft
groundwater ht. above foundation base =	**0**	ft
dry soil lateral pressure =	**0**	k/ft^3
saturated soil lateral pressure =	**0**	k/ft^3
dry soil unit weight =	**0**	k/ft^3
live load lateral surcharge =	**0.000**	ksf
	0	
concrete strength, f'_c =	**4**	ksi
concrete density, γ_c =	0.150	k/ft^3
concrete modulus of elasticity, E_c =	3605.0	ksi
concrete mass density, $\rho_c = \gamma_c / g$ =	0.004663	$k\text{-}sec^2/ft^4$

Seismic:

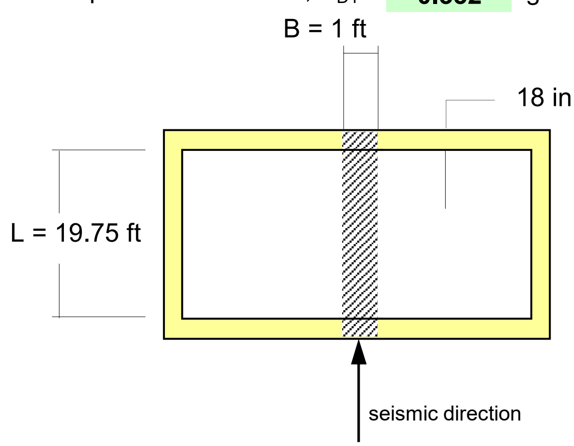
Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, S_{DS} = **0.446** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, S_{D1} = **0.332** *g

Structure Risk Category =	**3**	
Importance factor, I =	**1.25**	
Response modification factor, R_{wi} =	**3**	
Response modification factor, R_{wc} =	**1**	

Load Cases:

case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic



WALL PLAN

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BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:		Aeration Basin Transverse Hydrodynamic Pressures (CSZ Seismic Level)					

Weights:

unit 1-ft width wall mass, W_w =	(18/12) * (18.5) * 0.15 =	4.16	kip
wall c.g. relative to base, h_w =	18.5 / 2 =	9.250	ft
unit width liquid mass, W_L =	(19.75) * (1) * (15.82) * 32.17 =	19.50	kip

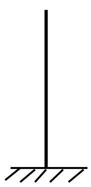
<u>Seismic:</u>

1). <u>structure stiffness and dynamic property:</u>

Note: per ASCE 7-10 and IBC 2012, the terms S_{ai} and S_{ac} have been appropriatetly substituted into the seismic equation of ACI 350.

Note: Wi and hi are impulsive component variables calculated on page 3.

wall mass, $m_w = H_w*(t_w / 12)*\rho_c$ =	0.12939	$k\text{-}sec^2/ft^2$
liquid mass, $m_i = (W_i / W_L)*(L/2)*H_L*\rho_L$ =	0.22237	$k\text{-}sec^2/ft^2$
centroidal distance of masses, $h = (h_w*m_w + h_i*m_i) / (m_w + m_i)$ =	7.232	ft



wall fixity condition is no roof & fixed at floor:

wall stiffness is determined using a unit mass load located at the centroidal distance h.

wall flexure stiffness, $k = Ec*(tw/h)^3 / 48$ = 1157.99 k/ft/ft

$$\omega_i = \sqrt{\frac{k}{m_w + m_i}} = \quad (1157.99 / (0.1294 + 0.2224))\hat{}\tfrac{1}{2} = 57.3756 \text{ rad/sec}$$

period of tank plus impulsive mass, $T_i = 2\pi / \omega_i$ = 2 π / 57.3756 = 0.1095 sec

(note: acceleration values to be from a maximum considered earthquake response spectra which will produce a factored load)

design factored spectral response acceleration for impulsive mass (5% damping), $S_{ai} = S_{DS}$ = 0.446 g

2). <u>Dynamic properties, Spectral amplification factors, and Effective mass coefficient:</u>

$$\lambda = \sqrt{3.16\ g\ \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)} = \quad (3.16*32.2*\tanh(3.16*(0.801)))\hat{}\tfrac{1}{2} = 10.0189$$

$$\omega_c = \frac{\lambda}{\sqrt{L}} = \quad 10.0189 / (19.75)\hat{}\tfrac{1}{2} = 2.2544 \text{ rad/sec,}$$

period of the convective mass, $T_c = 2\pi / \omega_c$ = 2 π / 2.2544 = 2.7870 sec

Long transition period (from map figure 22-15 ASCE 7), T_L = **16** sec

design spectral response acceleration for convective mass (0.5% damping), S_{ac} = 1.5 * Sd1 / Tc = 0.179 g

$$\text{effective mass coeff., } \varepsilon = 0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021 \text{ , but } \le 1.0 = 0.8063$$

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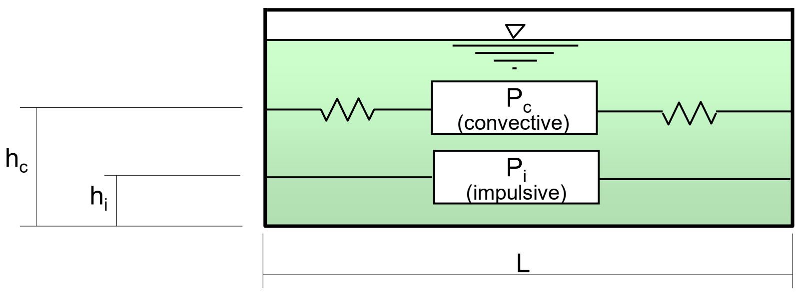
DESIGN TASK:			Aeration Basin Transverse Hydrodynamic Pressures (CSZ Seismic Level)				



Dynamic Model

L =	19.75	ft
B =	1	ft
H_L =	15.82	ft
W_L =	19.5	kip
L / H_L =	1.24842	
H_L / L =	0.80101	

3). <u>lateral fluid impulsive force:</u>

Wi = equivalent mass of the impulsive component of liquid.

$$W_i = W_L \left(\frac{\tanh\left(0.866 \frac{L}{H_L}\right)}{0.866 \frac{L}{H_L}} \right) =$$ 19.5*(tanh(0.866*(1.2484)) / 0.866*(1.2484)) = 14.31 kip

hi (EBP) = HL * (0.5-0.09375*(L/HL)) = 15.82*(0.5-0.09375*(1.2484)) = 6.058 ft

hi (IBP) = HL * {{(0.866*L/HL)/(2*tanh(0.866*L/HL))} -1/8 } = 8.798 ft

impulsive force, $P_i = \left(\frac{S_{ai} \, I}{R_{wi}} \right) W_i =$ (0.446 * 1.25 / 3)*14.31 = 2.7 kip

4). <u>lateral fluid convective force:</u>

Wc = equivalent mass of the convective component of liquid.

$$W_c = W_L \left(0.264 \left(\frac{L}{H_L} \right) \tanh\left(3.16 \left(\frac{H_L}{L} \right) \right) \right) =$$ 19.5*(0.264*(1.2484)*tanh(3.16*(0.801))) = 6.35 kip

$$h_{c\,(EBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 1}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) =$$ 10.491 ft

$$h_{c\,(IBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) =$$ 11.502 ft

convective force, $P_c = \left(\frac{S_{ac} \, I}{R_{wc}} \right) W_c =$ (0.1787 * 1.25 / 1)*6.35 = 1.4 kip

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DESIGN TASK:		Aeration Basin Transverse Hydrodynamic Pressures (CSZ Seismic Level)					

5). lateral inertia force of the accelerating wall:

unit width wall mass, W_w = 4.16 kip

wall c.g. relative to base, h_w = 9.250 ft

wall inertia force, $P_w = \left(\dfrac{S_{ai}\ I\ \varepsilon}{R_{wi}}\right) W_w =$ (0.446*1.25*0.8063/3)*4.16 = 0.62 kip

6). maximum wave slosh height displacement:

$d_{(max)} = \left(\dfrac{L}{2}\right)\left(\dfrac{S_{ac}}{1.4}\ I\right) =$ (19.75 / 2) * (0.1787 / 1.0 * 1.25) = 2.20 ft

7). vertical acceleration:

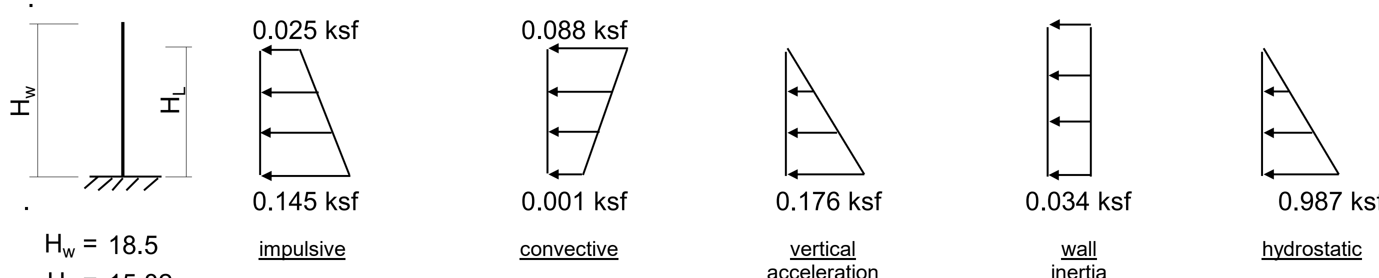
design horizontal accereration, S_{DS} = 0.446 *g

vertical spectral response acceleration (per ACI 350 para 9.4.3), $S_{av} = C_t = 0.4{*}S_{DS}$ = 0.1784 g

per ASCE 7-10 para. 15.7.7.2(b), use $I = R_i = b = 1.0$

Design vertical acceleration, $\ddot{u} = \dfrac{S_{av}\ I\ b}{R_i} =$ 0.1784*1*1/1 = 0.1784 g

8). vertical force distribution on a unit width using the linear distribution of ACI 350 sec 5.3:



H_w = 18.5

H_L = 15.82

impulsive:

$$p_{iy} = \frac{P_i\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\dfrac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_i = 2.70 kip

h_i = 6.058 ft

at $y = H_L$, p_{iy} = 0.025 ksf

at base $y = 0$, p_{iy} = 0.145 ksf

convective:

$$p_{cy} = \frac{P_c\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\dfrac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_c = 1.40 kip

h_c = 10.491 ft

at $y = H_L$, p_{cy} = 0.088 ksf

at base $y = 0$, p_{cy} = 0.001 ksf

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DESIGN TASK:		Aeration Basin Transverse Hydrodynamic Pressures (CSZ Seismic Level)					

vertical acceleration:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

$\ddot{u} =$ 0.1784
at $y = H_L$, $p_{vy} =$ 0.000 ksf
at base $y = 0$, $p_{vy} =$ 0.176 ksf

wall inertia:

$$p_{wy} = \frac{S_{ai}\ \ I\ \ \varepsilon\,\gamma_c\ (t_w/12)}{R_{wi}} =$$

$p_{wy} =$ 0.1498 * γ_c * $(t_w/12)$
at $y = H_w$, $p_{wy} =$ 0.034 ksf
at base $y = 0$, $p_{wy} =$ 0.034 ksf

hydrostatic:

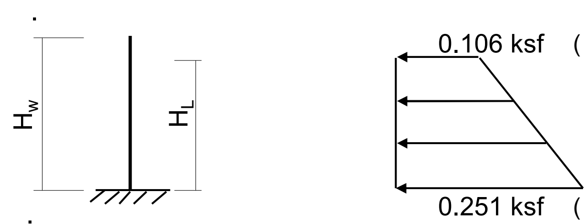
$$q_{hy} = \gamma_L\,(H_L - y) =$$

at $y = H_L$, $q_{hy} =$ 0.000 ksf
at base $y = 0$, $q_{hy} =$ 0.987 ksf

combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{\left(p_{ly} + p_{wy}\right)^2 + p_{cy}^2 + p_{vy}^2} =$$

at $y = H_w$, $p_y =$ 0.106 ksf
at base $y = 0$, $p_y =$ 0.251 ksf



0.106 ksf (unfactored = 0.106 / 1.4 = 0.075 ksf)

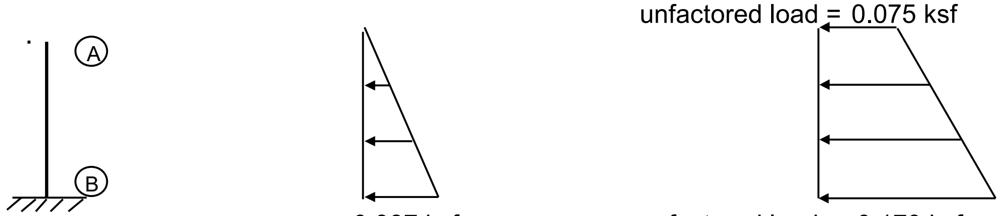
0.251 ksf (unfactored = 0.251 / 1.4 = 0.179 ksf)

resultant dynamic pressures

9). wall design pressures for hydrostatic + dynamic:

wall height, $H_w =$ 18.5 ft
liquid height, $H_L =$ 15.82 ft

unfactored load = 0.075 ksf



0.987 ksf
hydrostatic

unfactored load = 0.179 ksf
resultant dynamic pressures

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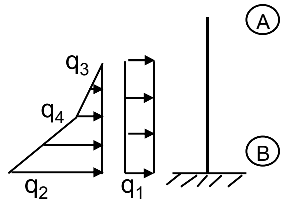
10). wall design pressures for external soil loading:

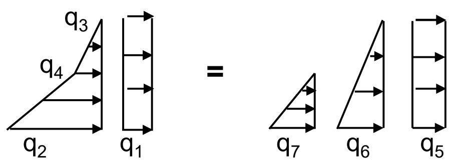
static soil:



The site has groundwater present.

wall height =	18.5	ft
soil height above top of base =	0	ft
groundwater ht. above base =	0	ft
dry soil lateral pressure =	0.000	k/ft^3
sat. soil lateral pressure =	0.000	k/ft^3
live load lateral surcharge =	0.000	ksf

equivalent static soil loadings:



LL lateral surcharge, q1 =	0.0000	ksf
unfactored soil, q2 =	0.0000	ksf
unfactored soil, q3 =	0.0000	ksf
unfactored soil, q4 =	0.000	ksf
equivalent soil loadings:		
unfactored q5 =	0.0000	ksf
unfactored q6 =	0.0000	ksf
unfactored q7 =	0.0000	ksf

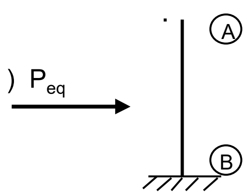
soil seismic:

resultant factored soil seismic load per foot of wall width, $P_{u\,(eq)}$ = **0** k/ft

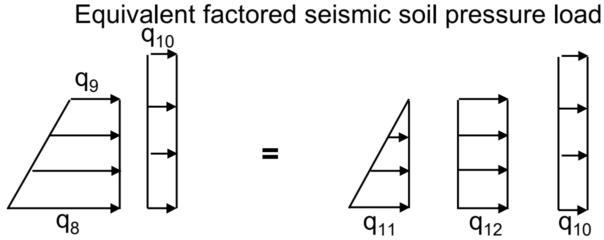
centroid location of the resultant soil seismic from the bottom of wall, h_{eq} = **0** ft

The resultant soil seismic load will be resolved into an equivalent pressure loading…

(resultant soil seismic load) P_{eq}



Equivalent factored seismic soil pressure loading & seismic wall loadings…



equivalent soil seismic, q8 =	0.0000	ksf
equivalent soil seismic, q9 =	0.0000	ksf
wall seismic (see wall page 5), q10 =	0.0337	ksf
equivalent soil seismic, q11 = q8 - q9 =	0.0000	ksf
equivalent soil seismic, q12 = q9 =	0.0000	ksf

unfactored equivalent soil seismic, q8 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q9 =0 / 1.4 =	0.0000	ksf
unfactored wall seismic , q10 = 0.0337 / 1.4 =	0.0241	ksf
unfactored equivalent soil seismic, q11 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q12 =0 / 1.4 =	0.0000	ksf

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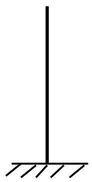
DESIGN TASK:		Aeration Basin Transverse Hydrodynamic Pressures (CSZ Seismic Level)					

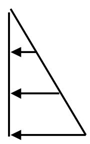
11). Summary of equivalent unfactored pressure loadings for wall load cases 1 thru 4:



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

a). load case 1: hydrostatic water

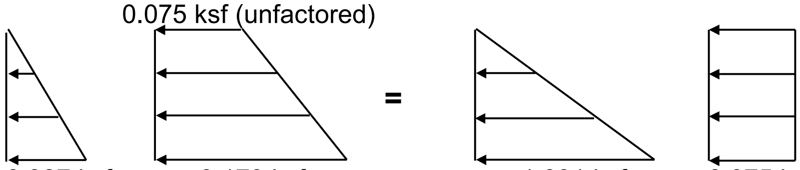
wall height = 18.5 ft
water depth = 15.82 ft



(unfactored) 0.987 ksf

b). load case 2: hydrostatic + dynamic:

0.075 ksf (unfactored)



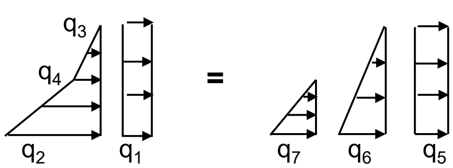
0.987 ksf (unfactored) hydrostatic	0.179 ksf (unfactored) resultant dynamic pressures	=	1.091 ksf (unfactored)	0.075 ksf (unfactored)

wall height = 18.5 ft
water depth = 15.82 ft

c). load case 3: static soil + LL surcharge:

wall height = 18.5 ft
soil height on wall = 0 ft
groundwater height = 0 ft

equivalent static soil & surcharge loadings...

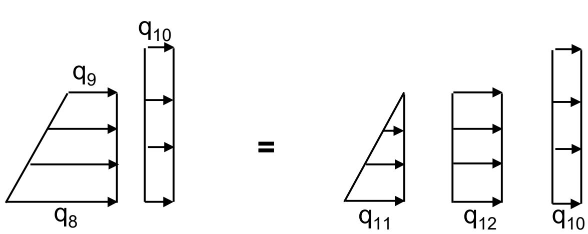


LL lateral surcharge, q1 =	0.000	ksf
unfactored soil, q2 =	0.000	ksf
unfactored soil, q3 =	0.000	ksf
unfactored soil, q4 =	0.000	ksf
equivalent soil loadings:		
unfactored q5 =	0.000	ksf
unfactored q6 =	0.000	ksf
unfactored q7 =	0.000	ksf

d). load case 4: soil seismic: (*note: add static soil pressure q6 & q7 to the seismic soil shown below)

equivalent seismic soil pressure loading & seismic wall loadings…

wall height = 18.5 ft
soil height on wall = 0 ft



unfactored equivalent soil seismic, q8 =	0.000	ksf
unfactored equivalent soil seismic, q9 =	0.000	ksf
unfactored equivalent soil seismic, q10 =	0.024	ksf
unfactored equivalent soil seismic, q11 =	0.000	ksf
unfactored equivalent soil seismic, q12 =	0.000	ksf

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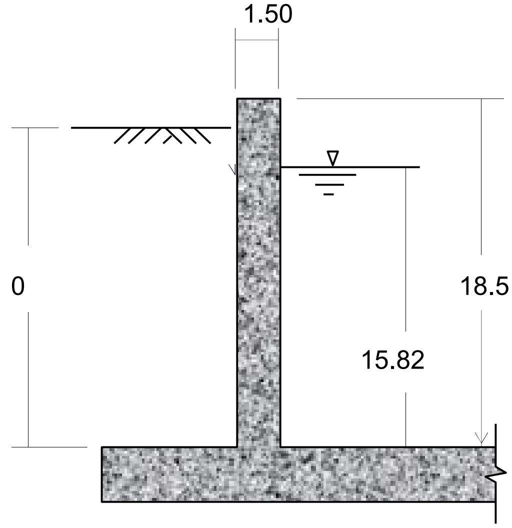
BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:		Aeration Basin Hydrodynamic Pressures (CSZ Seismic Level - Longitudinal Direction)					

Static, Seismic, and Hydrodynamic Seismic Wall Pressures using ACI 350.3-06 and an IBC 2012:

wall connection fixity =	**no roof & fixed at floor**	
tank unit width perpendicular to EQ., B =	1	ft
tank inside length in direction of seismic, L =	**175**	ft
tank wall thickness, t_w =	**18**	inch
wall height, H_w =	**18.5**	ft
liquid height, H_L =	**15.82**	ft
liquid specific gravity =	**1**	
liquid density, $\gamma_L = (\text{sp.gr.})^*\gamma_w$ =	0.0624	k/ft^3
acceleration due to gravity, g =	32.17	ft/sec^2
liquid mass density, $\rho_L = \gamma_L / g$ =	0.00194	$k\text{-}sec^2/ft^4$



WALL SECTION

Soil Data

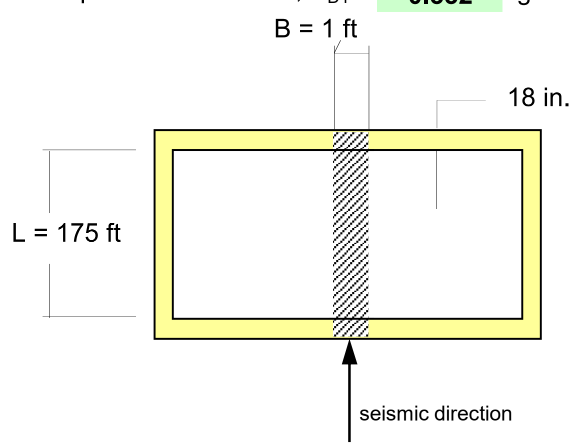
The site has groundwater present.

soil height above top of foundation base =	**0**	ft
groundwater ht. above foundation base =	**0**	ft
dry soil lateral pressure =	**0**	k/ft^3
saturated soil lateral pressure =	**0**	k/ft^3
dry soil unit weight =	**0**	k/ft^3
live load lateral surcharge =	**0.000**	ksf
	0	
concrete strength, f'_c =	**4**	ksi
concrete density, γ_c =	0.150	k/ft^3
concrete modulus of elasticity, E_c =	3605.0	ksi
concrete mass density, $\rho_c = \gamma_c / g$ =	0.004663	$k\text{-}sec^2/ft^4$

Seismic:

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, S_{DS} =	**0.446**	*g
Deisgn, 5% damped, spectral response acceleration at a period of 1-second, S_{D1} =	**0.332**	*g

Structure Risk Category =	**3**
Importance factor, I =	**1.25**
Response modification factor, R_{wi} =	**3**
Response modification factor, R_{wc} =	**1**



WALL PLAN

Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

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CHKD:		**DESCRIPTION:**			Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:		Aeration Basin Hydrodynamic Pressures (CSZ Seismic Level - Longitudinal Direction)					

Weights:

unit 1-ft width wall mass, W_w =	(18/12) * (18.5) * 0.15 =	4.16	kip
wall c.g. relative to base, h_w =	18.5 / 2 =	9.250	ft
unit width liquid mass, W_L =	(175) * (1) * (15.82) * 32.17 =	172.75	kip

Seismic:

1). structure stiffness and dynamic property:

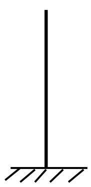
Note: per ASCE 7-10 and IBC 2012, the terms S_{ai} and S_{ac} have been appropriatetly substituted into the seismic equation of ACI 350.

Note: Wi and hi are impulsive component variables calculated on page 3.

wall mass, $m_w = H_w*(t_w / 12)*\rho_c$ = 0.12939 k-sec^2/ft^2

liquid mass, $m_i = (W_i / W_L)*(L/2)*H_L*\rho_L$ = 0.28024 k-sec^2/ft^2

centroidal distance of masses, $h = (h_w*m_w + h_i*m_i) / (m_w + m_i)$ = 6.981 ft



wall fixity condition is no roof & fixed at floor:

wall stiffness is determined using a unit mass load located at the centroidal distance h.

wall flexure stiffness, $k = Ec*(tw/h)^3 / 48$ = 1287.44 k/ft/ft

$$\omega_i = \sqrt{\frac{k}{m_w + m_i}} = \quad (1287.44 / (0.1294 + 0.2802))\hat{}½ = 56.0621 \text{ rad/sec}$$

period of tank plus impulsive mass, $T_i = 2\pi / \omega_i$ = 2 π / 56.0621 = 0.1121 sec

(note: acceleration values to be from a maximum considered earthquake response spectra which will produce a factored load)

design factored spectral response acceleration for impulsive mass (5% damping), $S_{ai} = S_{DS}$ = 0.446 g

2). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$\lambda = \sqrt{3.16\ g\ \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)} = \quad (3.16*32.2*\tanh(3.16*(0.0904)))\hat{}½ = 5.3174$$

$$\omega_c = \frac{\lambda}{\sqrt{L}} = \quad 5.3174 / (175)\hat{}½ = 0.4020 \text{ rad/sec,}$$

period of the convective mass, $T_c = 2\pi / \omega_c$ = 2 π / 0.402 = 15.6314 sec

Long transition period (from map figure 22-15 ASCE 7), T_L = **16** sec

design spectral response acceleration for convective mass (0.5% damping), S_{ac} = 1.5 * Sd1 / Tc = 0.032 g

$$\text{effective mass coeff., } \varepsilon = 0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021\ , \quad \text{but} \le 1.0 = \quad 0.7581$$

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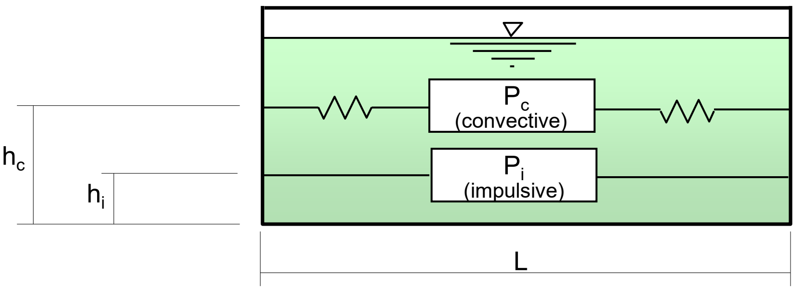
DESIGN TASK:		Aeration Basin Hydrodynamic Pressures (CSZ Seismic Level - Longitudinal Direction)					



Dynamic Model

L =	175	ft
B =	1	ft
H_L =	15.82	ft
W_L =	172.75	kip
L / H_L =	11.06195	
H_L / L =	0.09040	

3). lateral fluid impulsive force:

Wi = equivalent mass of the impulsive component of liquid.

$$W_i = W_L\left(\frac{\tanh\left(0.866\frac{L}{H_L}\right)}{0.866\frac{L}{H_L}}\right) = \quad 172.75*(\tanh(0.866*(11.0619)) / 0.866*(11.0619)) = \quad 18.03 \text{ kip}$$

hi (EBP) = HL * 0.375 = 15.82 * 0.375 = 5.933 ft

hi (IBP) = HL * {{(0.866*L/HL)/(2*tanh(0.866*L/HL))} -1/8 } = 73.798 ft

$$\text{impulsive force, } P_i = \left(\frac{S_{ai}\ I}{R_{wi}}\right)W_i = \quad (\ 0.446 * 1.25 / 3\ )*18.03 = \quad 3.4 \text{ kip}$$

4). lateral fluid convective force:

Wc = equivalent mass of the convective component of liquid.

$$W_c = W_L\left(0.264\left(\frac{L}{H_L}\right)\tanh\left(3.16\left(\frac{H_L}{L}\right)\right)\right) = \quad 172.75*(\ 0.264*(11.0619)*\tanh(3.16*(0.0904))\ ) = \quad 140.32 \text{ kip}$$

$$h_{c\ (EBP)} = H_L\left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 1}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)}\right) = \quad 7.963 \text{ ft}$$

$$h_{c\ (IBP)} = H_L\left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)}\right) = \quad 201.127 \text{ ft}$$

$$\text{convective force, } P_c = \left(\frac{S_{ac}\ I}{R_{wc}}\right)W_c = \quad (\ 0.0319 * 1.25 / 1\ )*140.32 = \quad 5.6 \text{ kip}$$

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DESIGN TASK:	Aeration Basin Hydrodynamic Pressures (CSZ Seismic Level - Longitudinal Direction)						

5). lateral inertia force of the accelerating wall:

unit width wall mass, W_w = 4.16 kip

wall c.g. relative to base, h_w = 9.250 ft

$$\text{wall inertia force, } P_w = \left(\frac{S_{ai}\ I\ \varepsilon}{R_{wi}}\right) W_w = \quad (0.446*1.25*0.7581/3)*4.16 = \quad 0.59 \text{ kip}$$

6). maximum wave slosh height displacement:

$$d_{(max)} = \left(\frac{L}{2}\right)\left(\frac{S_{ac}}{1.4}\ I\right) = \quad (175 / 2) * (0.0319 / 1.0 * 1.25) = \quad 3.49 \text{ ft}$$

7). vertical acceleration:

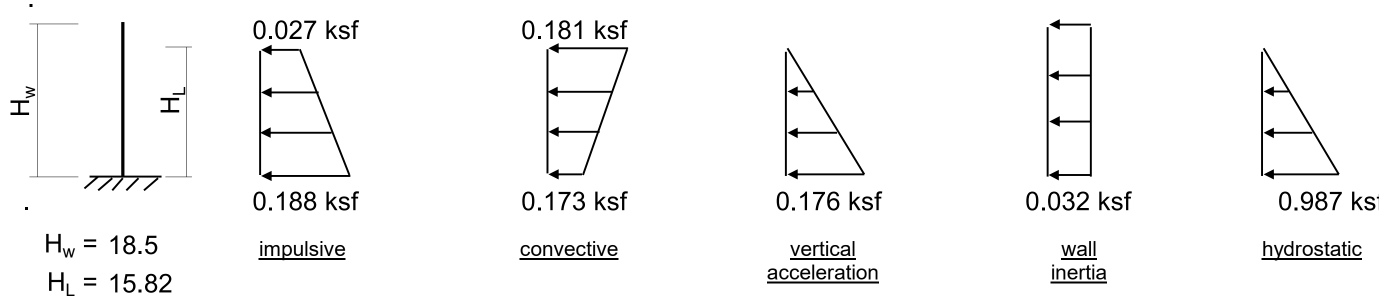
design horizontal accereration, S_{DS} = 0.446 *g

vertical spectral response acceleration (per ACI 350 para 9.4.3), $S_{av} = C_t = 0.4*S_{DS}$ = 0.1784 g

per ASCE 7-10 para. 15.7.7.2(b), use $I = R_i = b = 1.0$

$$\text{Design vertical acceleration, } \ddot{u} = \frac{S_{av}\ I\ b}{R_i} = \quad 0.1784*1*1/1 = \quad 0.1784 \text{ g}$$

8). vertical force distribution on a unit width using the linear distribution of ACI 350 sec 5.3:



H_w = 18.5

H_L = 15.82

impulsive:

$$p_{iy} = \frac{P_i\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_i = 3.40 kip

h_i = 5.933 ft

at $y = H_L$, p_{iy} = 0.027 ksf

at base $y = 0$, p_{iy} = 0.188 ksf

convective:

$$p_{cy} = \frac{P_c\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_c = 5.60 kip

h_c = 7.963 ft

at $y = H_L$, p_{cy} = 0.181 ksf

at base $y = 0$, p_{cy} = 0.173 ksf

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DESIGN TASK:	Aeration Basin Hydrodynamic Pressures (CSZ Seismic Level - Longitudinal Direction)						

vertical acceleration:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

ü = 0.1784

at y = H_L, p_{vy} = 0.000 ksf

at base y = 0, p_{vy} = 0.176 ksf

wall inertia:

$$p_{wy} = \frac{S_{ai}\;I\;\varepsilon\,\gamma_c\,(t_w/12)}{R_{wi}} =$$

p_{wy} = 0.1409 * γ_c * (t_w/12)

at y = H_w, p_{wy} = 0.032 ksf

at base y = 0, p_{wy} = 0.032 ksf

hydrostatic:

$$q_{hy} = \gamma_L\,(H_L - y) =$$

at y = H_L, q_{hy} = 0.000 ksf

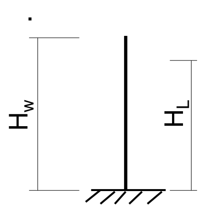
at base y = 0, q_{hy} = 0.987 ksf

combine the effects of the dynamic pressures on the wall:

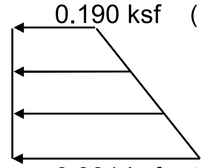
$$p_y = \sqrt{(p_{iy} + p_{wy})^2 + p_{cy}^2 + p_{vy}^2} =$$

at y = H_w, p_y = 0.190 ksf

at base y = 0, p_y = 0.331 ksf



0.190 ksf (unfactored = 0.19 / 1.4 = 0.136 ksf)



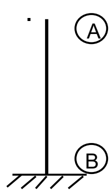
0.331 ksf (unfactored = 0.331 / 1.4 = 0.236 ksf)

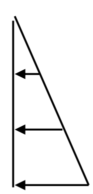
resultant dynamic pressures

9). wall design pressures for hydrostatic + dynamic:

wall height, H_w = 18.5 ft

liquid height, H_L = 15.82 ft

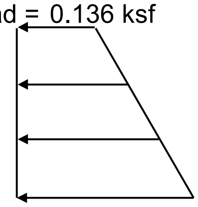




0.987 ksf

hydrostatic

unfactored load = 0.136 ksf



unfactored load = 0.236 ksf

resultant dynamic pressures

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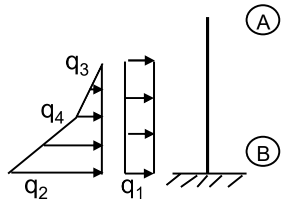
DESIGN TASK:		Aeration Basin Hydrodynamic Pressures (CSZ Seismic Level - Longitudinal Direction)					

10). wall design pressures for external soil loading:

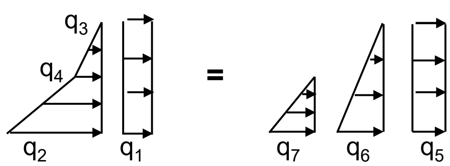
static soil:



The site has groundwater present.

wall height =	18.5	ft
soil height above top of base =	0	ft
groundwater ht. above base =	0	ft
dry soil lateral pressure =	0.000	k/ft^3
sat. soil lateral pressure =	0.000	k/ft^3
live load lateral surcharge =	0.000	ksf

equivalent static soil loadings:



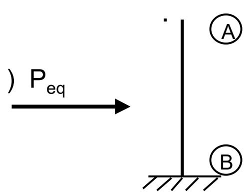
LL lateral surcharge, q1 =	0.0000	ksf
unfactored soil, q2 =	0.0000	ksf
unfactored soil, q3 =	0.0000	ksf
unfactored soil, q4 =	0.000	ksf
equivalent soil loadings:		
unfactored q5 =	0.0000	ksf
unfactored q6 =	0.0000	ksf
unfactored q7 =	0.0000	ksf

soil seismic:

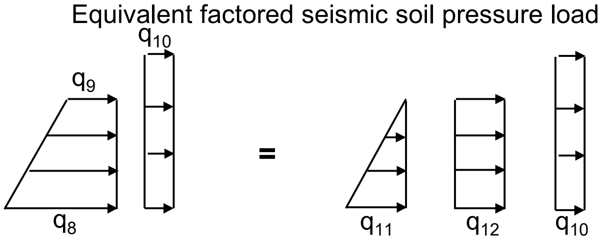
resultant factored soil seismic load per foot of wall width, $P_{u\,(eq)}$ = **0** k/ft

centroid location of the resultant soil seismic from the bottom of wall, h_{eq} = **0** ft

The resultant soil seismic load will be resolved into an equivalent pressure loading…



Equivalent factored seismic soil pressure loading & seismic wall loadings…



equivalent soil seismic, q8 =	0.0000	ksf
equivalent soil seismic, q9 =	0.0000	ksf
wall seismic (see wall page 5), q10 =	0.0317	ksf
equivalent soil seismic, q11 = q8 - q9 =	0.0000	ksf
equivalent soil seismic, q12 = q9 =	0.0000	ksf

unfactored equivalent soil seismic, q8 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q9 =0 / 1.4 =	0.0000	ksf
unfactored wall seismic , q10 = 0.0317 / 1.4 =	0.0226	ksf
unfactored equivalent soil seismic, q11 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q12 =0 / 1.4 =	0.0000	ksf

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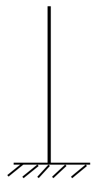
DESIGN TASK:		Aeration Basin Hydrodynamic Pressures (CSZ Seismic Level - Longitudinal Direction)					

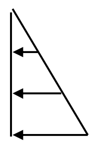
11). Summary of equivalent unfactored pressure loadings for wall load cases 1 thru 4:



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

a). load case 1: hydrostatic water

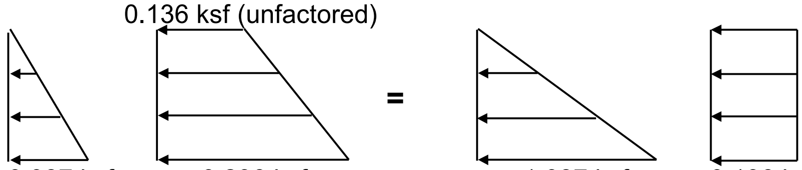
wall height = 18.5 ft
water depth = 15.82 ft



(unfactored) 0.987 ksf

b). load case 2: hydrostatic + dynamic:

0.136 ksf (unfactored)



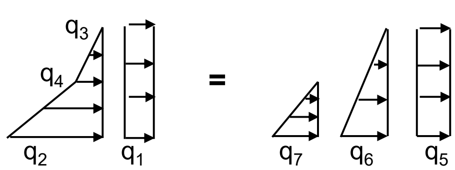
0.987 ksf (unfactored) hydrostatic	0.236 ksf (unfactored) resultant dynamic pressures	1.087 ksf (unfactored)	0.136 ksf (unfactored)

wall height = 18.5 ft
water depth = 15.82 ft

c). load case 3: static soil + LL surcharge:

wall height = 18.5 ft
soil height on wall = 0 ft
groundwater height = 0 ft

equivalent static soil & surcharge loadings...

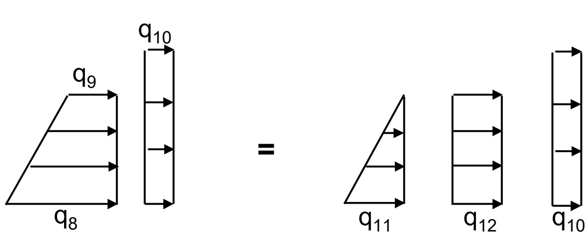


LL lateral surcharge, q1 =	0.000	ksf
unfactored soil, q2 =	0.000	ksf
unfactored soil, q3 =	0.000	ksf
unfactored soil, q4 =	0.000	ksf
equivalent soil loadings:		
unfactored q5 =	0.000	ksf
unfactored q6 =	0.000	ksf
unfactored q7 =	0.000	ksf

d). load case 4: soil seismic: (*note: add static soil pressure q6 & q7 to the seismic soil shown below)

equivalent seismic soil pressure loading & seismic wall loadings…

wall height = 18.5 ft
soil height on wall = 0 ft



unfactored equivalent soil seismic, q8 =	0.000	ksf
unfactored equivalent soil seismic, q9 =	0.000	ksf
unfactored equivalent soil seismic, q10 =	0.023	ksf
unfactored equivalent soil seismic, q11 =	0.000	ksf
unfactored equivalent soil seismic, q12 =	0.000	ksf

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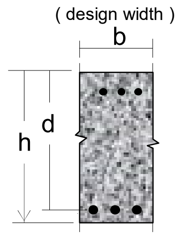
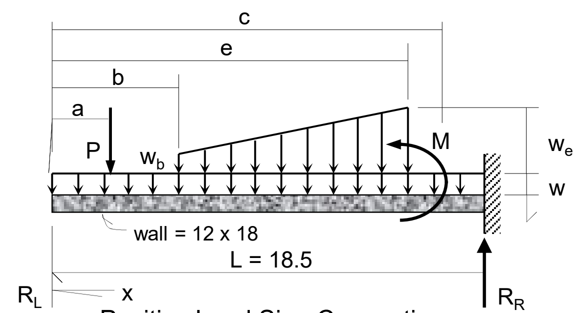
CHKD:		**DESCRIPTION:**			Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:		Dividing Wall between Aeration Basins 1&2 (Hydrodynamic Loading Only) (CSZ)					

Wall Data:

Design for a slab or a wall?	**Wall**	
Wall Support Fixity =	**Cantilever**	
Span, L =	**18.5**	ft
Wall width, b =	**12**	in
Thickness of Wall, h =	**18**	in
Depth to reinforcing, d =	**15**	in
concrete strength, f'_c =	**4**	ksi
reinforcing yield strength, f_y =	**60**	ksi
allow deflection, L/Δ =	**240**	

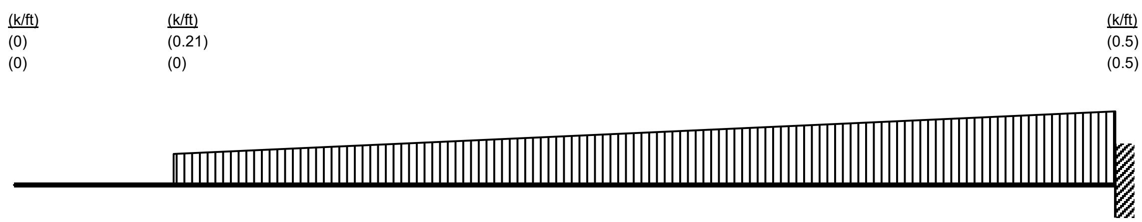


trial moment of Inertia, $I_x = 0.5\ I_g$ = 2916 in^4



Positive Load Sign Convention

Wall Loadings:



Externally Applied Service Loads to a Wall with Cantilever Support									
Uniform	Trapezoidal Loads				Point Loads		Moment Loads		Concrete Load Factors
	Begin		End						
w (k/ft)	b (ft)	w_b (k/ft)	e (ft)	w_e (k/ft)	a (ft)	P (kip)	c (ft)	M (ft-k)	
	2.68	**0.212**	**18.5**	**0.502**					**1**
optional environmental load factor for moment (ACI 350) =									**1**

Results:

* Deflection is based on the trial moment of Inertia of 0.5*Ig.(actual deflection will be adjusted later on page 6.)

Calculated Reactions, Moments, and Deflections for the Wall with Cantilever Support											
Load Type	Reactions, R or R_u		Maximum Moments, M or M_u				* Maximum Short Term Deflections				
	Left End	Right End	Max Positive		Max Negative		downward		upward		
	R_L (kip)	R_R (kip)	x distance (ft)	+M (ft-k)	x distance (ft)	-M (ft-k)	x distance (ft)	Δ (in)	x distance (ft)	Δ (in)	
service loads	0.00	5.65	0.000	0.00	18.500	-38.63	0.000	-0.455	18.500	0.000	
factored loads	0.00	5.65	0.000	0.00	18.500	-38.63	Δ ≤ L/240 ,Okay		Δ ≤ L/240 ,Okay		
	environmental factor, M_u =			0.00		-38.63	Δmax = L/240 = 0.925 "				

Using # 9 @ 6" neg bars and # 9 @ 6" pos bars, the final adjusted long term deflection (see page 6), Δ = 0.509 "

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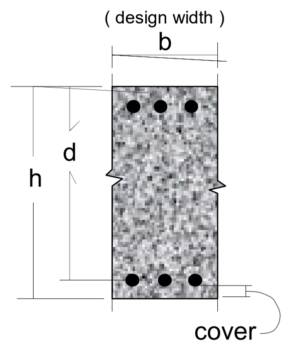
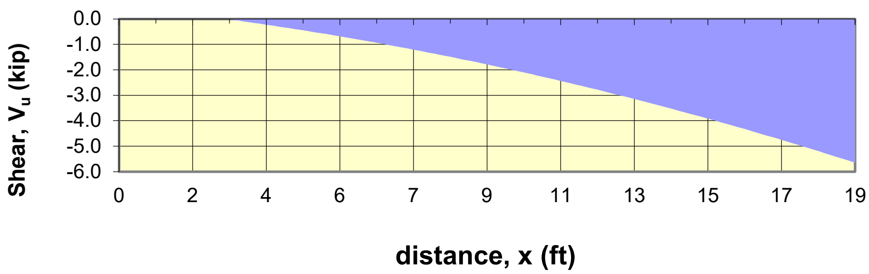
CHKD:		**DESCRIPTION:**			Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:		Dividing Wall between Aeration Basins 1&2 (Hydrodynamic Loading Only) (CSZ)					

Wall Shear Capacity (Based on ACI 318, 11.2.1.1):

Maximum Shear, V_u =	5.65	kip	concrete, f'_c =	4	ksi
Wall width, b =	12	in	reinforcing, f_y =	60	ksi
Depth to reinforcing, d =	15	in	concrete modulus, $E_c = 57 * (f'_c)^{½}$ =	3605	ksi
Thickness of wall, h =	18	in	ϕ, Shear =	1.00	



Factored Shear Diagram



factored shear force, V_u = 5.650 kip

Concrete Shear Capacity, $\phi V_c = \phi * 2*b*d*(f'_c)^{1/2}$ = 22.768 kip

øVc > Vu, OK

Minimum shrinkage-temperature requirement in the flexure direction:

wall minimum temperature / shrinkage steel ratio = **0.00500**
number of layers of reinforcement in the wall (1 or 2 ?) = **2**

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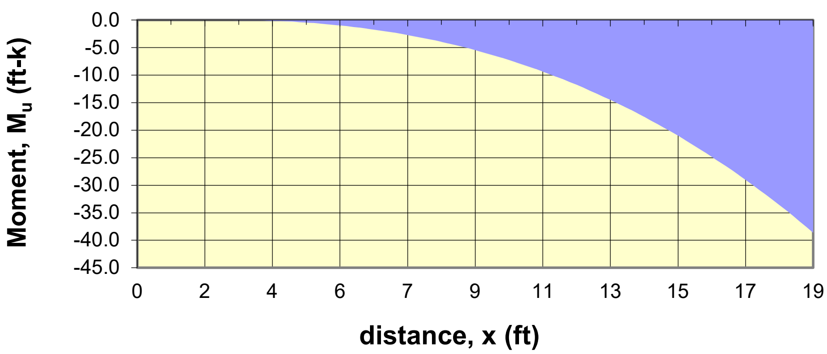
DESIGN TASK:		Dividing Wall between Aeration Basins 1&2 (Hydrodynamic Loading Only) (CSZ)					

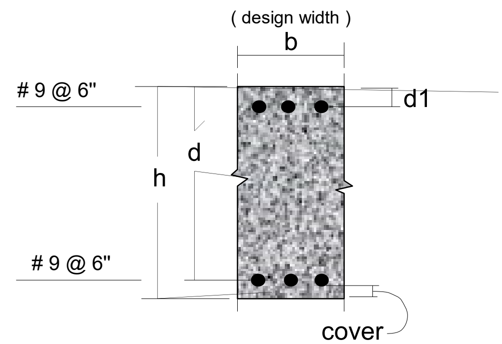
Wall Bending:

Service Moment, M(+) =	0.00	ft-k	concrete strength, f'_c =	4	ksi
Service Moment, M(-) =	38.63	ft-k	reinforcing yield strength, f_y =	60	ksi
Factored Moment, $M_u(+)$ =	0.00	ft-k	concrete modulus, $E_c = 57 * (f'_c)^{½}$ =	3605	ksi
Factored Moment, $M_u(-)$ =	38.63	ft-k	reinforcement modulus, E_s =	29000	ksi
Wall width, b =	12	in	$n = E_s / E_c$ =	8.044	
Depth to reinforcing, d =	15	in	β_1 =	0.85	
Thickness of wall, h =	18	in	ϕ, Bending =	0.9	

Factored Moment Diagram

(includes the 1 environmental factor)





Reinforcement

1). Negative Steel: (location at x = 18.5 ft)

Depth to negative reinforcing, d1 =	**2.5**	in	
$M_u(-)$ =	38.63	ft-k	$R_u = M_u / (\phi * b * d^2) = 178.7$
Wall width, b =	12	in	
Depth to reinforcing, d = h - d1 =	15.5	in	$\rho_{(req'd)} = \frac{0.85 f'_c}{f_y}\left(1-\sqrt{1-\frac{2R_u}{0.85 f'_c}}\right) = 0.00306$
Area steel required, $A'_{s\,(req'd)}$ =	0.569	in^2	
Bar number size =	**# 9**		
Spacing of negative bars =	**6**	in	
Area of steel provided, A'_s =	2.00	in^2	, $\rho = A_s / bd$ = 0.01075
Min area steel req'd, $A_{s\,(min)}$ =	0.62	in^2	, $\rho_{(min)}$ = 0.00333
Max area allowed, $A'_{s\,(max)}$ =	3.98	in^2	, $\rho_{(max)}$ = 0.02138

2). Positive Steel: (location at x = 0 ft)

concrete clear cover to positive steel =	**2**	in	
$M_u(+)$ =	0	ft-k	$R_u = M_u / (\phi * b * d^2) = 0.0$
Wall width, b =	12	in	
Depth to reinforcing, d =	15	in	$\rho_{(req'd)} = \frac{0.85 f'_c}{f_y}\left(1-\sqrt{1-\frac{2R_u}{0.85 f'_c}}\right) = 0.00000$
Area steel required, $A_{s\,(req'd)}$ =	0.000	in^2	
Bar number size =	**# 9**		
Spacing of positive bars =	**6**		
Area of bottom steel provided, A_s =	2.00	in^2	, $\rho = A_s / bd$ = 0.01111
Min area steel req'd, $A_{s\,(min)}$ =	0.54	in^2	, $\rho_{(min)}$ = 0.00300
Max area allowed, $A'_{s\,(max)}$ =	3.85	in^2	, $\rho_{(max)}$ = 0.02138

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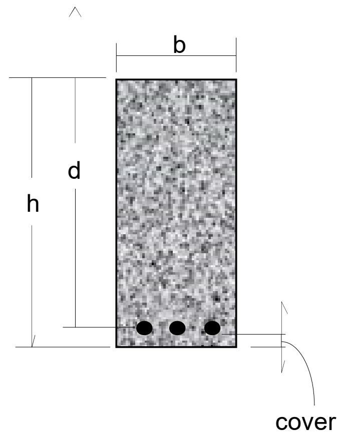
CHKD:		**DESCRIPTION:**		Aeration Basins		**JOB NO:**	11962A.00
DESIGN TASK:	Dividing Wall Strength for 18" Thick Wall (Hydrodynamic Loading Only) (CSZ)						

Concrete Shear and Moment Strength Capacity



Design for … beam, slab, wall ? **wall**

Properties and Geometry

Compression width of wall, b =	**12**	inch
Thickness of wall, h =	**18**	inch
Depth to reinforcing, d =	**15**	inch
factored design moment, M_u =	**38.63**	ft-k
factored design shear, V_u =	**5.65**	kip

f'_c (psi) =	**4000**
f_y (psi) =	**60000**
ϕ, Bending =	**1**
ϕ, Shear =	**1**
E_s (psi) =	29000000
E_c (psi) =	3604997
$n = E_s / E_c$ =	8.04
β_1 =	0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi*2*b*d*(f'_c)^{1/2}$ = 22.77 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **Existing 18" wall w/ #9@6"**

Area steel provided, A_s = **2** in^2 $\rho = A_s / bd$ = 0.01111

ρ(min) < As/bd < ρ(max) - OK

$A_{s\ (min)}$ =	0.32	in^2	$\rho_{(min)}$ =	0.00180
$A_{s\ (max)}$ =	3.85	in^2	$\rho_{(max)}$ =	0.02138

bending strength, $\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$

$\phi * M_n$ = 1*0.01111*60*12*15² *(1-0.588*0.01111*60/4)*(ft/12) = 135.294 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

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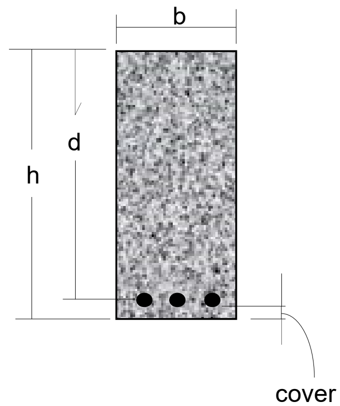
CHKD:		**DESCRIPTION:**			Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:		Dividing Wall Strength for 12" Thick Wall (Hydrodynamic Loading Only) (CSZ)					

Concrete Shear and Moment Strength Capacity



Design for … beam, slab, wall ? **wall**

Properties and Geometry

Compression width of wall, b =	**12**	inch
Thickness of wall, h =	**12**	inch
Depth to reinforcing, d =	**9**	inch
factored design moment, M_u =	**17.1**	ft-k
factored design shear, V_u =	**3.50**	kip

f'_c (psi) =	**4000**
f_y (psi) =	**60000**
ϕ, Bending =	**1**
ϕ, Shear =	**1**
E_s (psi) =	29000000
E_c (psi) =	3122019
$n = E_s / E_c$ =	9.29
β_1 =	0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi*2*b*d*(f'_c)^{1/2}$ = 13.66 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #7@12"**

Area steel provided, A_s = **0.6** in^2 $\rho = A_s / bd$ = 0.00556

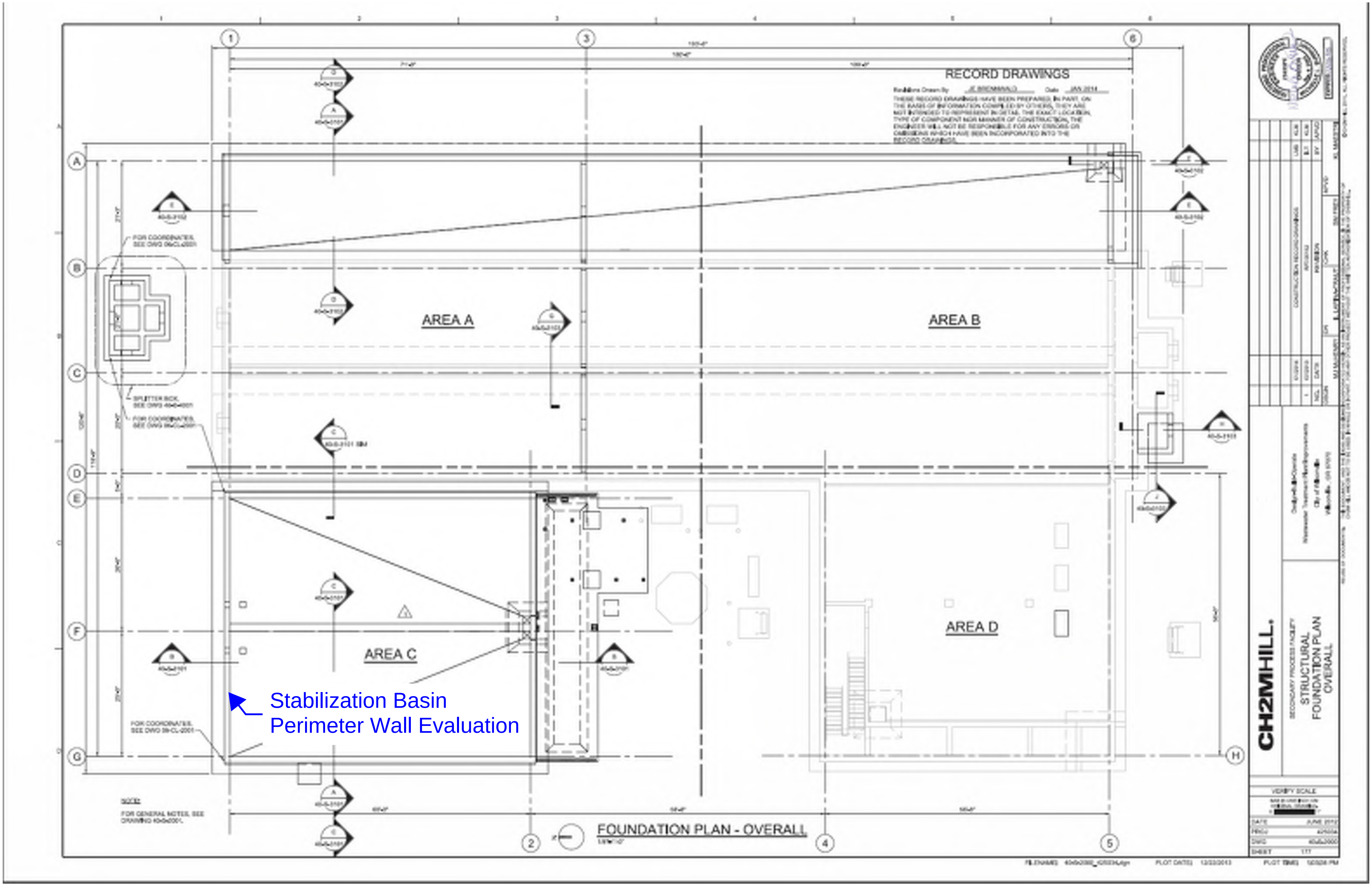
$\rho(min) < As/bd < \rho(max)$ - OK

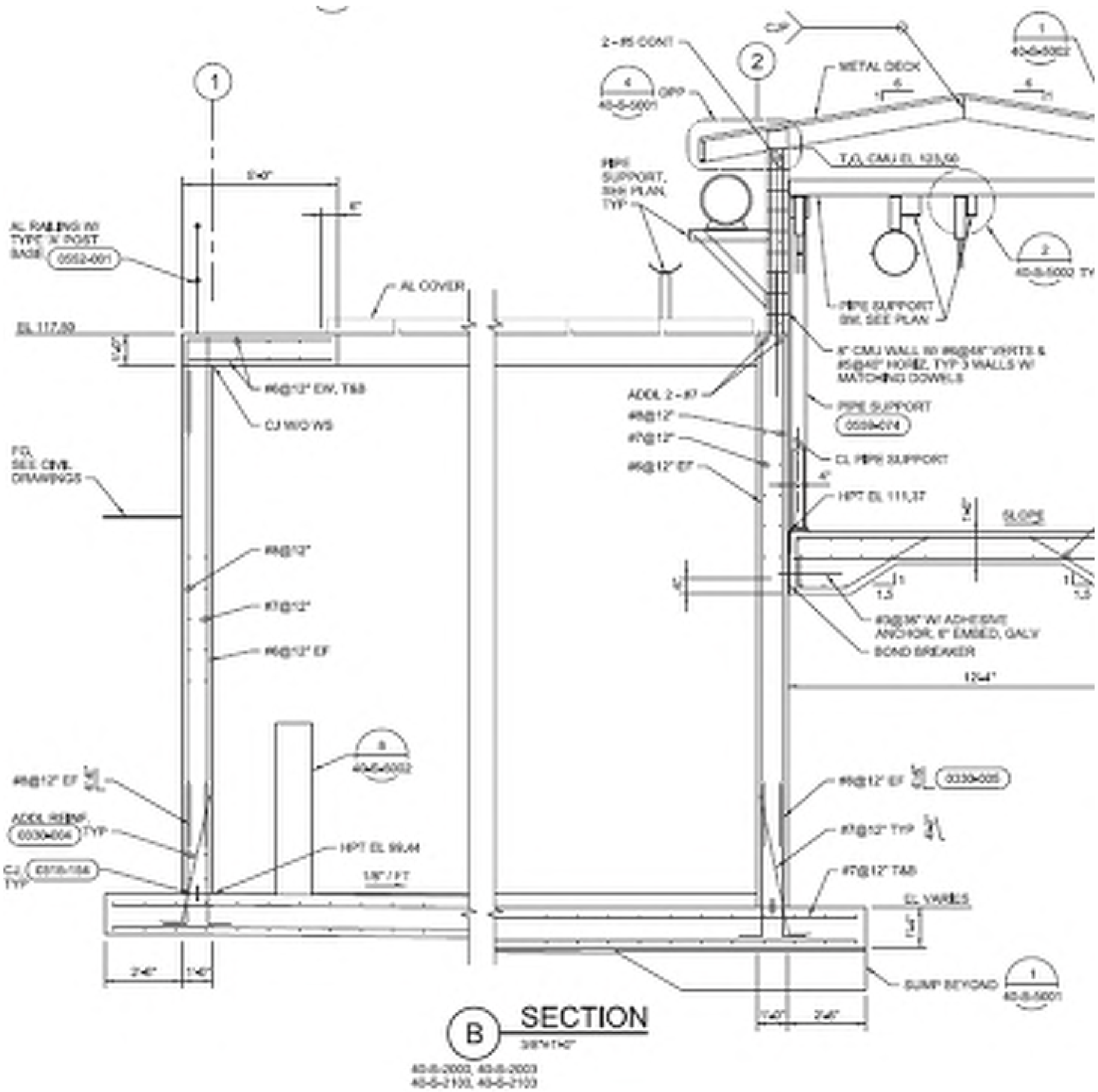
$A_{s\ (min)}$ =	0.19	in^2	$\rho_{(min)}$ =	0.00180
$A_{s\ (max)}$ =	1.73	in^2	$\rho_{(max)}$ =	0.01604

bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

$\phi*M_n$ = 1*0.00556*60*12*9² *(1-0.588*0.00556*60/3)*(ft/12) = 25.235 ft-k ≥ Mu

Moment strength ≥ design moment, Okay


RECORD DRAWINGS
AREA A
AREA B
AREA C
AREA D
Stabilization Basin
Perimeter Wall Evaluation
FOUNDATION PLAN - OVERALL
CH2MHILL
STRUCTURAL
FOUNDATION PLAN
OVERALL



Stabilization Basin Perimeter Wall Section Reinforcing

carollo
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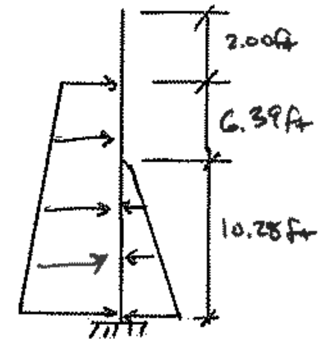
CHKD. BY ____ DATE ____ Aeration Basins JOB NO. 11962A.00

Aeration Basins Area C - Perimeter Basin Wall



The existing stabilization perimeter walls along the north and south elevations will be checked for the seismic loads. Since the basin is partially buried, there will be soil pressures present to resist the seismic effects. The perimeter walls are 12" thick with #8@12" vertical reinforcing and #7@12" horizontal reinforcing inside face and #8@12" horizontal reinforcing outside face.

For soil pressure on wall, assume a load of 40pcf triangular. See attached spreadsheet for hydrostatic & hydrodynamic loads on wall.

Checking wall strength vertically (#8@12" vert reinf). Forces are at BSE-2E.

M_{uy} = 18.35 k·ft/ft $\quad$ ϕM_n = 33.26 k·ft/ft

V_{uy} = 7.26 k/ft $\quad$ ϕV_n = 13.66 k/ft

Moment DCR = $\frac{18.35}{33.26}$ = 0.55 (ok)

Shear DCR = $\frac{7.26}{13.66}$ = 0.53 (ok)

Checking wall strength horizontally (#8@12" O.F. & #7@12" I.F. horiz reinf)

M_{ux+} = 14.45 k·ft/ft $\quad$ ϕM_n = 33.76 k·ft/ft

M_{ux-} = -7.03 k·ft/ft $\quad$ ϕM_n = 25.68 k·ft/ft

V_{ux} = 4.61 k/ft $\quad$ ϕV_n = 13.66 k/ft

+ Moment DCR = $\frac{14.45}{33.76}$ = 0.43 (ok)

- Moment DCR = $\frac{7.03}{25.68}$ = 0.27 (ok)

Shear DCR = $\frac{4.61}{13.66}$ = 0.34 (ok)

Checking freeboard height in basin. For Risk Category III, $\delta = 0.7 \times d_{max}$.

$\delta_{transverse}$ = 0.7(2.40ft) = 1.68 ft

$\delta_{longitudinal}$ = 0.7(3.17ft) = 2.22 ft

free board height = 2.00ft

2.00ft > 1.68ft (ok) Free board is sufficient.

2.00ft < 2.22ft (NG) Free board is not sufficient.

BY BS DATE 7/9/21 SUBJECT City of Wilsonville SHEET NO. ____ OF ____
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Checking wall strength vertically. Forces are at CSZ seismic level.

M_{uy} = 20.22 k·ft/ft ϕM_n = 33.26 k·ft/ft

V_{uy} = 7.92 k/ft ϕV_n = 13.66 k/ft

Moment DCR = $\frac{20.22\ k\cdot ft/ft}{33.26\ k\cdot ft/ft}$ = 0.61 (ok)

Shear DCR = $\frac{7.92\ k/ft}{13.66\ k/ft}$ = 0.58 (ok)

Checking wall strength horizontally.

M_{ux} = 16.25 k·ft/ft ϕM_n = 33.26 k·ft/ft

V_{ux} = 5.18 k/ft ϕV_n = 13.66 k/ft

Moment DCR = $\frac{16.25\ k\cdot ft/ft}{33.26\ k\cdot ft/ft}$ = 0.49 (ok)

Shear DCR = $\frac{5.18\ k/ft}{13.66\ k/ft}$ = 0.38 (ok)

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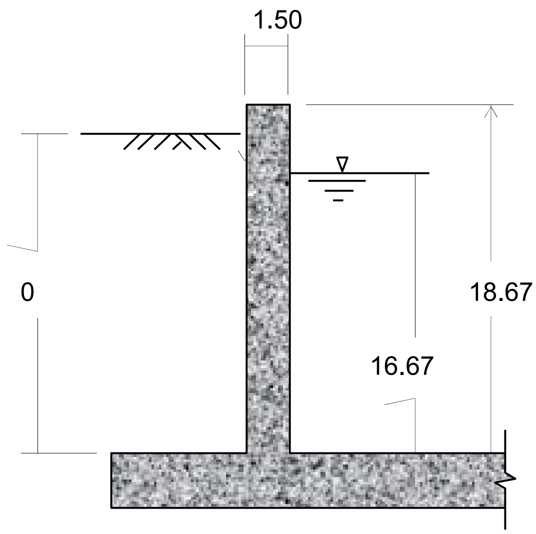
DESIGN TASK:			Stabilizatin Basins (Transverse Direction) (BSE-2E)				

Static, Seismic, and Hydrodynamic Seismic Wall Pressures using ACI 350.3-06 and an IBC 2012:

wall connection fixity =	**no roof & fixed at floor**	
tank unit width perpendicular to EQ., B =	1	ft
tank inside length in direction of seismic, L =	**25**	ft
tank wall thickness, t_w =	**18**	inch
wall height, H_w =	**18.67**	ft
liquid height, H_L =	**16.67**	ft
liquid specific gravity =	**1**	
liquid density, γ_L = (sp.gr.)*γ_w =	0.0624	k/ft^3
acceleration due to gravity, g =	32.17	ft/sec^2
liquid mass density, $\rho_L = \gamma_L / g$ =	0.00194	$k\text{-}sec^2/ft^4$



WALL SECTION

Soil Data

The site has no groundwater.

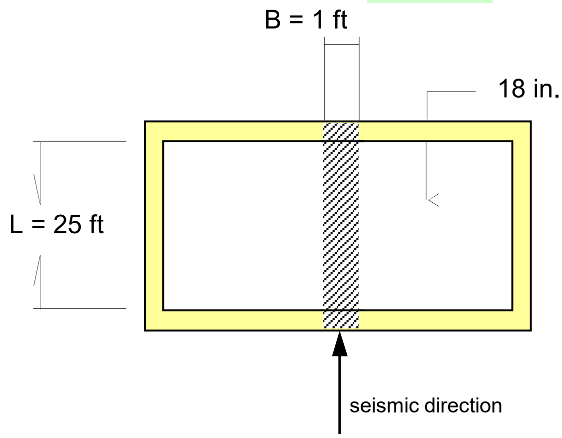
soil height above top of foundation base =	**0**	ft
groundwater ht. above foundation base =	**0**	ft
dry soil lateral pressure =	**0**	k/ft^3
saturated soil lateral pressure =	**0**	k/ft^3
dry soil unit weight =	**0**	k/ft^3
live load lateral surcharge =	**0.000**	ksf
	0	
concrete strength, f'_c =	**4**	ksi
concrete density, γ_c =	0.150	k/ft^3
concrete modulus of elasticity, E_c =	3605.0	ksi
concrete mass density, $\rho_c = \gamma_c / g$ =	0.004663	$k\text{-}sec^2/ft^4$

Seismic:

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, S_{DS} = **0.744** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, S_{D1} = **0.405** *g

Structure Risk Category =	**2**
Importance factor, I =	1
Response modification factor, R_{wi} =	**3**
Response modification factor, R_{wc} =	**1**



WALL PLAN

Load Cases:

case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

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Weights:

unit 1-ft width wall mass, W_w =	(18/12) * (18.67) * 0.15 =	4.20	kip
wall c.g. relative to base, h_w =	18.67 / 2 =	9.335	ft
unit width liquid mass, W_L =	(25) * (1) * (16.67) * 32.17 =	26.01	kip

<u>Seismic:</u>

1). <u>structure stiffness and dynamic property:</u>

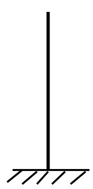
Note: per ASCE 7-10 and IBC 2012, the terms S_{ai} and S_{ac} have been appropriatetly substituted into the seismic equation of ACI 350.

Note: Wi and hi are impulsive component variables calculated on page 3.

wall mass, $m_w = H_w*(t_w / 12)*\rho_c$ = 0.13058 k-sec^2/ft^2

liquid mass, $m_i = (W_i / W_L)*(L/2)*H_L*\rho_L$ = 0.26806 k-sec^2/ft^2

centroidal distance of masses, $h = (h_w*m_w + h_i*m_i) / (m_w + m_i)$ = 7.261 ft



wall fixity condition is no roof & fixed at floor:

wall stiffness is determined using a unit mass load located at the centroidal distance h.

wall flexure stiffness, k = Ec*(tw/h)³ / 48 = 1144.17 k/ft/ft

$$\omega_i = \sqrt{\frac{k}{m_w + m_i}} =$$ (1144.17 / (0.1306 + 0.2681))ˆ½ = 53.5744 rad/sec

period of tank plus impulsive mass, $T_i = 2\pi / \omega_i$ = 2 π / 53.5744 = 0.1173 sec

(note: acceleration values to be from a maximum considered earthquake response spectra which will produce a factored load)

design factored spectral response acceleration for impulsive mass (5% damping), $S_{ai} = S_{DS}$ = 0.744 g

2). <u>Dynamic properties, Spectral amplification factors, and Effective mass coefficient:</u>

$$\lambda = \sqrt{3.16\ g\ \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)} =$$ (3.16*32.2*tanh(3.16*(0.6668)))ˆ½ = 9.9345

$$\omega_c = \frac{\lambda}{\sqrt{L}} =$$ 9.9345 / (25)ˆ½ = 1.9869 rad/sec,

period of the convective mass, $T_c = 2\pi / \omega_c$ = 2 π / 1.9869 = 3.1623 sec

Long transition period (from map figure 22-15 ASCE 7), T_L = **16** sec

design spectral response acceleration for convective mass (0.5% damping), S_{ac} = 1.5 * Sd1 / Tc = 0.192 g

$$\text{effective mass coeff., } \varepsilon = 0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021\ ,\ \text{ but } \leq\ 1.0 =$$ 0.7688

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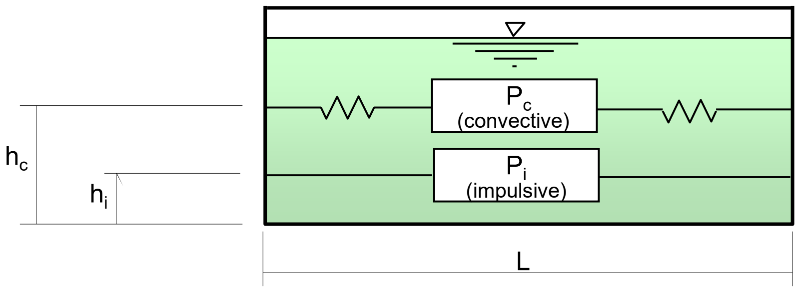
DESIGN TASK:					Stabilizatin Basins (Transverse Direction) (BSE-2E)		



L =	25	ft
B =	1	ft
H_L =	16.67	ft
W_L =	26.01	kip
L / H_L =	1.49970	
H_L / L =	0.66680	

3). lateral fluid impulsive force: Dynamic Model

Wi = equivalent mass of the impulsive component of liquid.

$$W_i = W_L \left(\frac{\tanh\left(0.866 \frac{L}{H_L}\right)}{0.866 \frac{L}{H_L}} \right) =$$ 26.01*(tanh(0.866*(1.4997)) / 0.866*(1.4997)) = 17.25 kip

hi (EBP) = HL * 0.375 = 16.67 * 0.375 = 6.251 ft

hi (IBP) = HL * {{(0.866*L/HL)/(2*tanh(0.866*L/HL))} -1/8 } = 10.483 ft

impulsive force, $P_i = \left(\frac{S_{ai}\ I}{R_{wi}} \right) W_i =$ (0.744 * 1 / 3)*17.25 = 4.3 kip

4). lateral fluid convective force:

Wc = equivalent mass of the convective component of liquid.

$$W_c = W_L \left(0.264 \left(\frac{L}{H_L} \right) \tanh\left(3.16 \left(\frac{H_L}{L} \right) \right) \right) =$$ 26.01*(0.264*(1.4997)*tanh(3.16*(0.6668))) = 10 kip

$$h_{c\ (EBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 1}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) =$$ 10.474 ft

$$h_{c\ (IBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) =$$ 12.446 ft

convective force, $P_c = \left(\frac{S_{ac}\ I}{R_{wc}} \right) W_c =$ (0.1921 * 1 / 1)*10 = 1.9 kip

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5). lateral inertia force of the accelerating wall:

unit width wall mass, W_w = 4.20 kip

wall c.g. relative to base, h_w = 9.335 ft

wall inertia force, $P_w = \left(\dfrac{S_{ai}\ I\ \varepsilon}{R_{wi}}\right) W_w =$ (0.744*1*0.7688/3)*4.2 = 0.80 kip

6). maximum wave slosh height displacement:

$d_{(max)} = \left(\dfrac{L}{2}\right)\left(\dfrac{S_{ac}}{1.4}\ I\right) =$ (25 / 2) * (0.1921 / 1.0 * 1) = 2.40 ft

7). vertical acceleration:

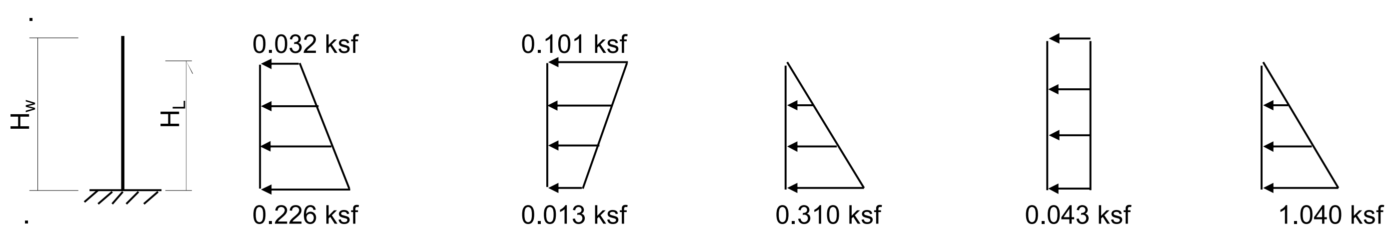
design horizontal accereration, S_{DS} = 0.744 *g

vertical spectral response acceleration (per ACI 350 para 9.4.3), $S_{av} = C_t = 0.4{*}S_{DS}$ = 0.2976 g

per ASCE 7-10 para. 15.7.7.2(b), use $I = R_i = b = 1.0$

Design vertical acceleration, $\ddot{u} = \dfrac{S_{av} I\ \ b}{R_i} =$ 0.2976*1*1/1 = 0.2976 g

8). vertical force distribution on a unit width using the linear distribution of ACI 350 sec 5.3:



H_w = 18.67
H_L = 16.67

impulsive | convective | vertical acceleration | wall inertia | hydrostatic

impulsive:

$$p_{iy} = \frac{P_i\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\dfrac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_i = 4.30 kip
h_i = 6.251 ft
at y = H_L, p_{iy} = 0.032 ksf
at base y = 0, p_{iy} = 0.226 ksf

convective:

$$p_{cy} = \frac{P_c\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\dfrac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_c = 1.90 kip
h_c = 10.474 ft
at y = H_L, p_{cy} = 0.101 ksf
at base y = 0, p_{cy} = 0.013 ksf

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vertical acceleration:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

ü = 0.2976

at y = H_L, p_{vy} = 0.000 ksf

at base y = 0, p_{vy} = 0.310 ksf

wall inertia:

$$p_{wy} = \frac{S_{ai}\ I\ \varepsilon\ \gamma_c\ (t_w/12)}{R_{wi}} =$$

p_{wy} = 0.1907 * γ_c * (t_w/12)

at y = H_w, p_{wy} = 0.043 ksf

at base y = 0, p_{wy} = 0.043 ksf

hydrostatic:

$$q_{hy} = \gamma_L\,(H_L - y) =$$

at y = H_L, q_{hy} = 0.000 ksf

at base y = 0, q_{hy} = 1.040 ksf

combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{(p_{iy} + p_{wy})^2 + p_{cy}^2 + p_{vy}^2} =$$

at y = H_w, p_y = 0.126 ksf

at base y = 0, p_y = 0.410 ksf

0.126 ksf (unfactored = 0.126 / 1.4 = 0.09 ksf)

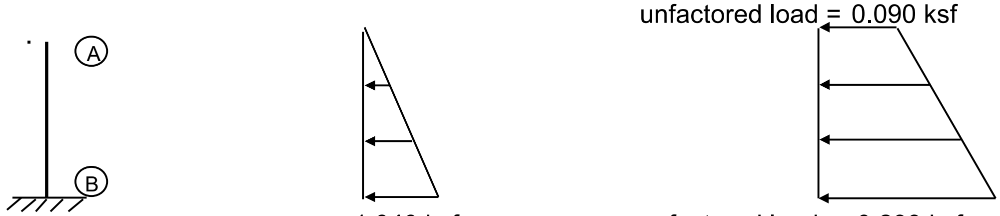
0.410 ksf (unfactored = 0.41 / 1.4 = 0.293 ksf)

resultant dynamic pressures

9). wall design pressures for hydrostatic + dynamic:

wall height, H_w = 18.67 ft

liquid height, H_L = 16.67 ft



unfactored load = 0.090 ksf

1.040 ksf

hydrostatic

unfactored load = 0.293 ksf

resultant dynamic pressures

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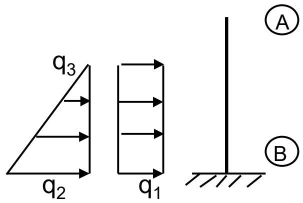
10). wall design pressures for external soil loading:

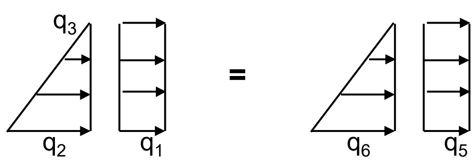
static soil:



The site has no groundwater.

wall height =	18.67	ft
soil height above top of base =	0	ft
groundwater ht. above base =	0	ft
dry soil lateral pressure =	0.000	k/ft^3
sat. soil lateral pressure =	0.000	k/ft^3
live load lateral surcharge =	0.000	ksf

equivalent static soil loadings:



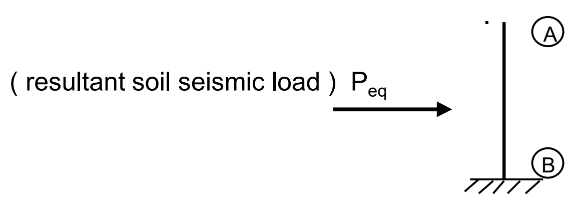
LL lateral surcharge, q1 =	0.0000	ksf
unfactored soil, q2 =	0.0000	ksf
unfactored soil, q3 =	0.0000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.0000	ksf
unfactored q6 =	0.0000	ksf

soil seismic:

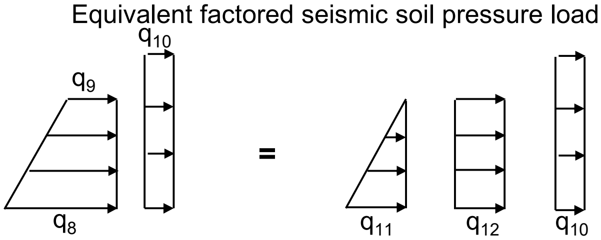
resultant factored soil seismic load per foot of wall width, $P_{u\,(eq)}$ = **0** k/ft

centroid location of the resultant soil seismic from the bottom of wall, h_{eq} = **0** ft

The resultant soil seismic load will be resolved into an equivalent pressure loading…



Equivalent factored seismic soil pressure loading & seismic wall loadings…



equivalent soil seismic, q8 =	0.0000	ksf
equivalent soil seismic, q9 =	0.0000	ksf
wall seismic (see wall page 5), q10 =	0.0429	ksf
equivalent soil seismic, q11 = q8 - q9 =	0.0000	ksf
equivalent soil seismic, q12 = q9 =	0.0000	ksf

unfactored equivalent soil seismic, q8 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q9 =0 / 1.4 =	0.0000	ksf
unfactored wall seismic , q10 = 0.0429 / 1.4 =	0.0306	ksf
unfactored equivalent soil seismic, q11 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q12 =0 / 1.4 =	0.0000	ksf

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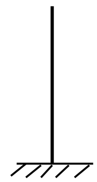
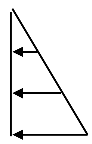
11). Summary of equivalent unfactored pressure loadings for wall load cases 1 thru 4:



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

a). load case 1: hydrostatic water

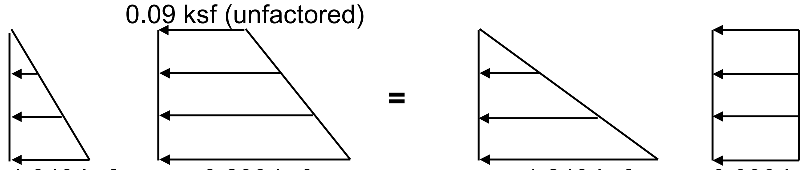
wall height = 18.67 ft
water depth = 16.67 ft



(unfactored) 1.040 ksf

b). load case 2: hydrostatic + dynamic:

0.09 ksf (unfactored)



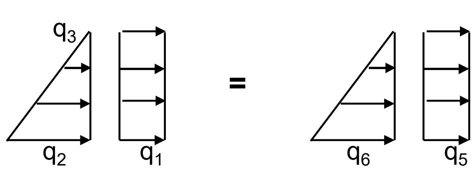
1.040 ksf (unfactored) hydrostatic | 0.293 ksf (unfactored) resultant dynamic pressures | = | 1.243 ksf (unfactored) | 0.090 ksf (unfactored)

wall height = 18.67 ft
water depth = 16.67 ft

c). load case 3: static soil + LL surcharge:

wall height = 18.67 ft
soil height on wall = 0 ft

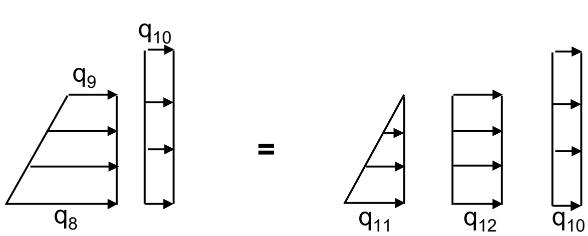
equivalent static soil & surcharge loadings...



LL lateral surcharge, q1 =	0.000	ksf
unfactored soil, q2 =	0.000	ksf
unfactored soil, q3 =	0.000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.000	ksf
unfactored q6 =	0.000	ksf

d). load case 4: soil seismic: (*note: add static soil pressure q6 & q7 to the seismic soil shown below)
equivalent seismic soil pressure loading & seismic wall loadings…

wall height = 18.67 ft
soil height on wall = 0 ft



unfactored equivalent soil seismic, q8 =	0.000	ksf
unfactored equivalent soil seismic, q9 =	0.000	ksf
unfactored equivalent soil seismic, q10 =	0.031	ksf
unfactored equivalent soil seismic, q11 =	0.000	ksf
unfactored equivalent soil seismic, q12 =	0.000	ksf

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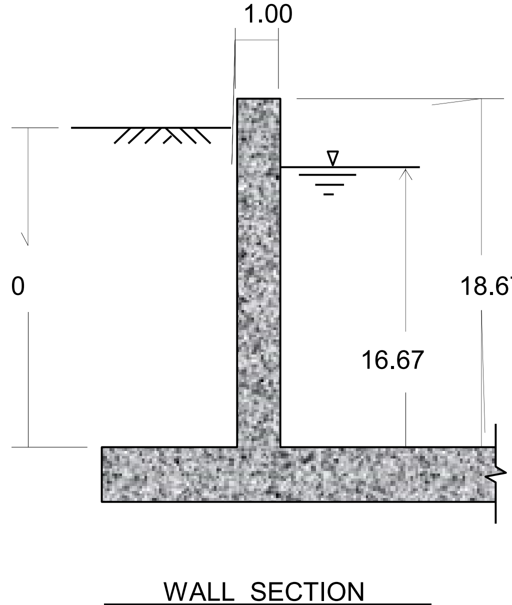
Static, Seismic, and Hydrodynamic Seismic Wall Pressures using ACI 350.3-06 and an IBC 2012:

wall connection fixity =	**no roof & fixed at floor**	
tank unit width perpendicular to EQ., B =	1	ft
tank inside length in direction of seismic, L =	**60**	ft
tank wall thickness, t_w =	**12**	inch
wall height, H_w =	**18.67**	ft
liquid height, H_L =	**16.67**	ft
liquid specific gravity =	**1**	
liquid density, γ_L = (sp.gr.)*γ_w =	0.0624	k/ft^3
acceleration due to gravity, g =	32.17	ft/sec^2
liquid mass density, $\rho_L = \gamma_L / g$ =	0.00194	$k\text{-}sec^2/ft^4$



Soil Data

The site has no groundwater.

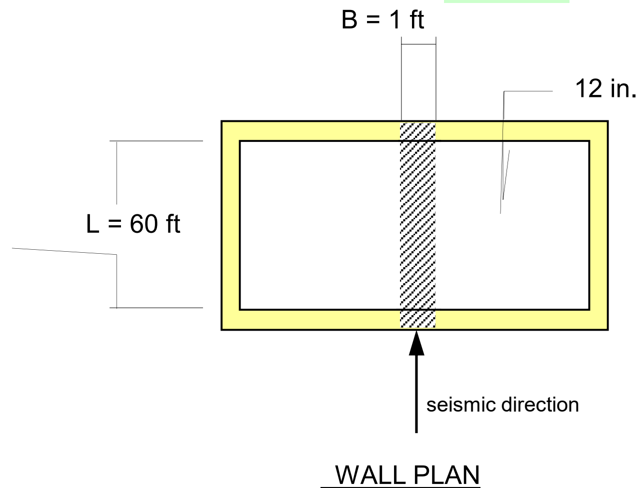
soil height above top of foundation base =	**0**	ft
groundwater ht. above foundation base =	**0**	ft
dry soil lateral pressure =	**0**	k/ft^3
saturated soil lateral pressure =	**0**	k/ft^3
dry soil unit weight =	**0**	k/ft^3
live load lateral surcharge =	**0.000**	ksf
	0	
concrete strength, f'_c =	**4**	ksi
concrete density, γ_c =	0.150	k/ft^3
concrete modulus of elasticity, E_c =	3605.0	ksi
concrete mass density, $\rho_c = \gamma_c / g$ =	0.004663	$k\text{-}sec^2/ft^4$

Seismic:

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, S_{DS} = **0.744** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, S_{D1} = **0.405** *g

Structure Risk Category =	**2**
Importance factor, I =	**1**
Response modification factor, R_{wi} =	**3**
Response modification factor, R_{wc} =	**1**



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

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Weights:

unit 1-ft width wall mass, W_w =	(12/12) * (18.67) * 0.15 =	2.80	kip
wall c.g. relative to base, h_w =	18.67 / 2 =	9.335	ft
unit width liquid mass, W_L =	(60) * (1) * (16.67) * 32.17 =	62.41	kip

Seismic:

1). structure stiffness and dynamic property:

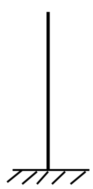
Note: per ASCE 7-10 and IBC 2012, the terms S_{ai} and S_{ac} have been appropriatetly substituted into the seismic equation of ACI 350.

Note: Wi and hi are impulsive component variables calculated on page 3.

wall mass, $m_w = H_w*(t_w / 12)*\rho_c$ = 0.08705 k-sec^2/ft^2

liquid mass, $m_i = (W_i / W_L)*(L/2)*H_L*\rho_L$ = 0.30993 k-sec^2/ft^2

centroidal distance of masses, $h = (h_w*m_w + h_i*m_i) / (m_w + m_i)$ = 6.927 ft



wall fixity condition is no roof & fixed at floor:

wall stiffness is determined using a unit mass load located at the centroidal distance h.

wall flexure stiffness, k = Ec*(tw/h)³ / 48 = 390.46 k/ft/ft

$$\omega_i = \sqrt{\frac{k}{m_w + m_i}} =$$ (390.46 / (0.0871 + 0.3099))ˆ½ = 31.3618 rad/sec

period of tank plus impulsive mass, $T_i = 2\pi / \omega_i$ = 2 π / 31.3618 = 0.2003 sec

(note: acceleration values to be from a maximum considered earthquake response spectra which will produce a factored load)

design factored spectral response acceleration for impulsive mass (5% damping), $S_{ai} = S_{DS}$ = 0.744 g

2). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$\lambda = \sqrt{3.16\ g\ \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)} =$$ (3.16*32.2*tanh(3.16*(0.2778)))ˆ½ = 8.4681

$$\omega_c = \frac{\lambda}{\sqrt{L}} =$$ 8.4681 / (60)ˆ½ = 1.0932 rad/sec,

period of the convective mass, $T_c = 2\pi / \omega_c$ = 2 π / 1.0932 = 5.7474 sec

Long transition period (from map figure 22-15 ASCE 7), T_L = **16** sec

design spectral response acceleration for convective mass (0.5% damping), S_{ac} = 1.5 * Sd1 / Tc = 0.106 g

$$\text{effective mass coeff., } \varepsilon = 0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021\ ,\ \text{but} \leq 1.0 =$$ 0.5299

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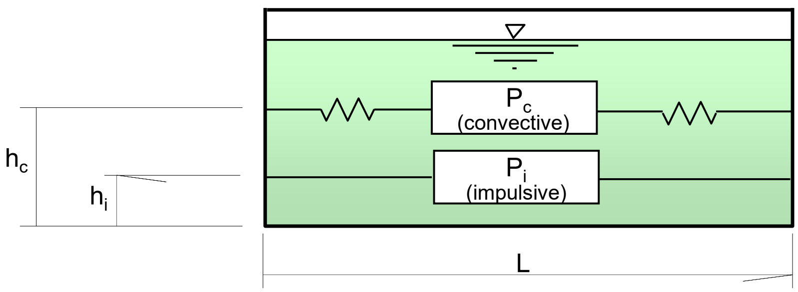

Dynamic Model

L =	60	ft
B =	1	ft
H_L =	16.67	ft
W_L =	62.41	kip
L / H_L =	3.59928	
H_L / L =	0.27783	

3). lateral fluid impulsive force:

Wi = equivalent mass of the impulsive component of liquid.

$$W_i = W_L \left(\frac{\tanh\left(0.866 \frac{L}{H_L}\right)}{0.866 \frac{L}{H_L}} \right) = \quad 62.41*(\tanh(0.866*(3.5993)) / 0.866*(3.5993)) = \quad 19.94 \text{ kip}$$

hi (EBP) = HL * 0.375 = 16.67 * 0.375 = 6.251 ft

hi (IBP) = HL * {{(0.866*L/HL)/(2*tanh(0.866*L/HL))} -1/8 } = 23.998 ft

$$\text{impulsive force, } P_i = \left(\frac{S_{ai} \ I}{R_{wi}} \right) W_i = \quad (0.744 * 1 / 3)*19.94 = \quad 4.9 \text{ kip}$$

4). lateral fluid convective force:

Wc = equivalent mass of the convective component of liquid.

$$W_c = W_L \left(0.264 \left(\frac{L}{H_L} \right) \tanh\left(3.16 \left(\frac{H_L}{L} \right) \right) \right) = \quad 62.41*(0.264*(3.5993)*\tanh(3.16*(0.2778))) = \quad 41.83 \text{ kip}$$

$$h_{c\ (EBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 1}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) = \quad 8.832 \text{ ft}$$

$$h_{c\ (IBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) = \quad 28.102 \text{ ft}$$

$$\text{convective force, } P_c = \left(\frac{S_{ac}\ I}{R_{wc}} \right) W_c = \quad (0.1057 * 1 / 1)*41.83 = \quad 4.4 \text{ kip}$$

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5). lateral inertia force of the accelerating wall:

unit width wall mass, W_w = 2.80 kip

wall c.g. relative to base, h_w = 9.335 ft

$$\text{wall inertia force, } P_w = \left(\frac{S_{ai}\ I\ \varepsilon}{R_{wi}}\right) W_w = \quad (0.744*1*0.5299/3)*2.8 = \quad 0.37 \text{ kip}$$

6). maximum wave slosh height displacement:

$$d_{(max)} = \left(\frac{L}{2}\right)\left(\frac{S_{ac}}{1.4}\ I\right) = \quad (60/2)*(0.1057/1.0*1) = \quad 3.17 \text{ ft}$$

Wave height is greater than the freeboard of 2-ft. Check effects of wave spillage.

7). vertical acceleration:

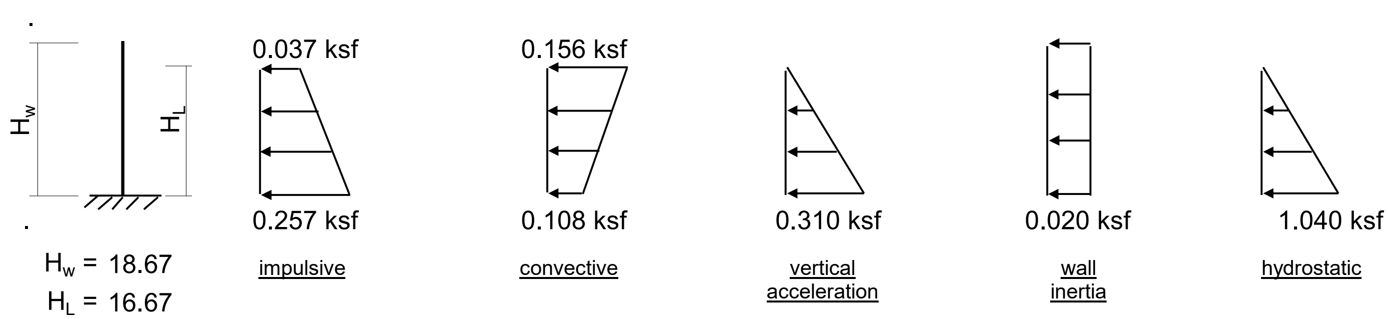
design horizontal accereration, S_{DS} = 0.744 *g

vertical spectral response acceleration (per ACI 350 para 9.4.3), $S_{av} = C_t = 0.4*S_{DS}$ = 0.2976 g

per ASCE 7-10 para. 15.7.7.2(b), use $I = R_i = b = 1.0$

$$\text{Design vertical acceleration, } \ddot{u} = \frac{S_{av}\ I\ b}{R_i} = \quad 0.2976*1*1/1 = \quad 0.2976 \text{ g}$$

8). vertical force distribution on a unit width using the linear distribution of ACI 350 sec 5.3:



impulsive:

$$p_{iy} = \frac{P_i\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_i = 4.90 kip

h_i = 6.251 ft

at $y = H_L$, p_{iy} = 0.037 ksf

at base $y = 0$, p_{iy} = 0.257 ksf

convective:

$$p_{cy} = \frac{P_c\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_c = 4.40 kip

h_c = 8.832 ft

at $y = H_L$, p_{cy} = 0.156 ksf

at base $y = 0$, p_{cy} = 0.108 ksf

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vertical acceleration:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

ü = 0.2976

at $y = H_L$, p_{vy} = 0.000 ksf

at base $y = 0$, p_{vy} = 0.310 ksf

wall inertia:

$$p_{wy} = \frac{S_{ai}\ \ I\ \ \varepsilon\,\gamma_c\ (t_w/12)}{R_{wi}} =$$

p_{wy} = 0.1314 * γ_c * (t_w/12)

at $y = H_w$, p_{wy} = 0.020 ksf

at base $y = 0$, p_{wy} = 0.020 ksf

hydrostatic:

$$q_{hy} = \gamma_L\,(H_L - y) =$$

at $y = H_L$, q_{hy} = 0.000 ksf

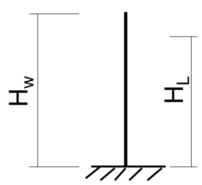
at base $y = 0$, q_{hy} = 1.040 ksf

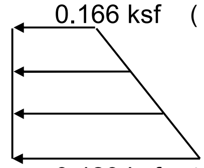
combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{\left(p_{iy} + p_{wy}\right)^2 + p_{cy}^2 + p_{vy}^2} =$$

at $y = H_w$, p_y = 0.166 ksf

at base $y = 0$, p_y = 0.429 ksf





0.166 ksf (unfactored = 0.166 / 1.4 = 0.118 ksf)

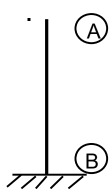
0.429 ksf (unfactored = 0.429 / 1.4 = 0.307 ksf)

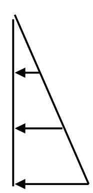
resultant dynamic pressures

9). wall design pressures for hydrostatic + dynamic:

wall height, H_w = 18.67 ft

liquid height, H_L = 16.67 ft

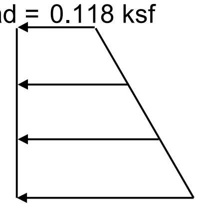




1.040 ksf

hydrostatic

unfactored load = 0.118 ksf



unfactored load = 0.307 ksf

resultant dynamic pressures

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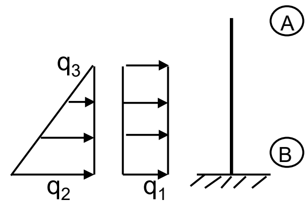
BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:				Stabilizatin Basins (Longitudinal Direction) (BSE-2E)			

10). wall design pressures for external soil loading:

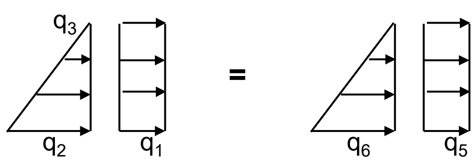
static soil:



The site has no groundwater.

wall height =	18.67	ft
soil height above top of base =	0	ft
groundwater ht. above base =	0	ft
dry soil lateral pressure =	0.000	k/ft^3
sat. soil lateral pressure =	0.000	k/ft^3
live load lateral surcharge =	0.000	ksf

equivalent static soil loadings:



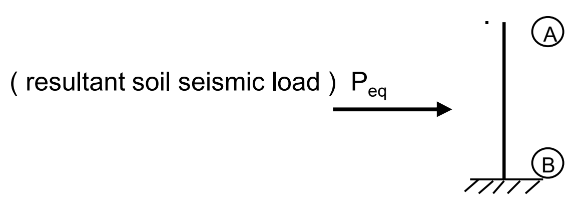
LL lateral surcharge, q1 =	0.0000	ksf
unfactored soil, q2 =	0.0000	ksf
unfactored soil, q3 =	0.0000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.0000	ksf
unfactored q6 =	0.0000	ksf

soil seismic:

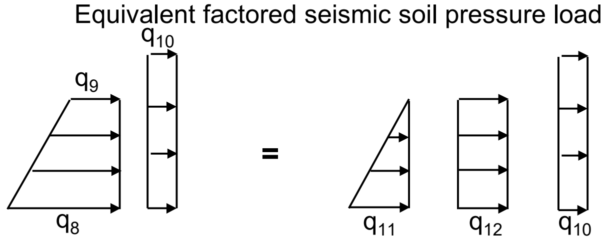
resultant factored soil seismic load per foot of wall width, $P_{u\,(eq)}$ = **0** k/ft

centroid location of the resultant soil seismic from the bottom of wall, h_{eq} = **0** ft

The resultant soil seismic load will be resolved into an equivalent pressure loading…



Equivalent factored seismic soil pressure loading & seismic wall loadings…



equivalent soil seismic, q8 =	0.0000	ksf
equivalent soil seismic, q9 =	0.0000	ksf
wall seismic (see wall page 5), q10 =	0.0197	ksf
equivalent soil seismic, q11 = q8 - q9 =	0.0000	ksf
equivalent soil seismic, q12 = q9 =	0.0000	ksf

unfactored equivalent soil seismic, q8 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q9 =0 / 1.4 =	0.0000	ksf
unfactored wall seismic , q10 = 0.0197 / 1.4 =	0.0141	ksf
unfactored equivalent soil seismic, q11 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q12 =0 / 1.4 =	0.0000	ksf

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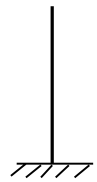
BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:				Stabilizatin Basins (Longitudinal Direction) (BSE-2E)			

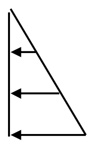
11). Summary of equivalent unfactored pressure loadings for wall load cases 1 thru 4:



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

a). load case 1: hydrostatic water

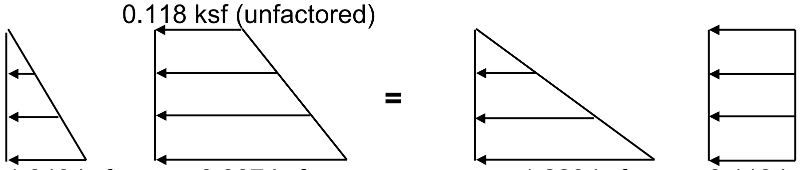
wall height = 18.67 ft
water depth = 16.67 ft



(unfactored) 1.040 ksf

b). load case 2: hydrostatic + dynamic:

0.118 ksf (unfactored)



1.040 ksf	0.307 ksf		1.229 ksf	0.118 ksf
(unfactored)	(unfactored)	=	(unfactored)	(unfactored)
hydrostatic	resultant dynamic pressures			

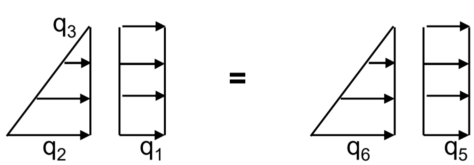
wall height = 18.67 ft
water depth = 16.67 ft

c). load case 3: static soil + LL surcharge:

wall height = 18.67 ft
soil height on wall = 0 ft

equivalent static soil & surcharge loadings...

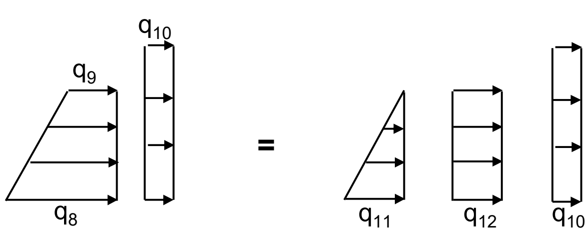
LL lateral surcharge, q1 =	0.000	ksf
unfactored soil, q2 =	0.000	ksf
unfactored soil, q3 =	0.000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.000	ksf
unfactored q6 =	0.000	ksf



d). load case 4: soil seismic: (*note: add static soil pressure q6 & q7 to the seismic soil shown below)

equivalent seismic soil pressure loading & seismic wall loadings…

wall height = 18.67 ft
soil height on wall = 0 ft



unfactored equivalent soil seismic, q8 =	0.000	ksf
unfactored equivalent soil seismic, q9 =	0.000	ksf
unfactored equivalent soil seismic, q10 =	0.014	ksf
unfactored equivalent soil seismic, q11 =	0.000	ksf
unfactored equivalent soil seismic, q12 =	0.000	ksf

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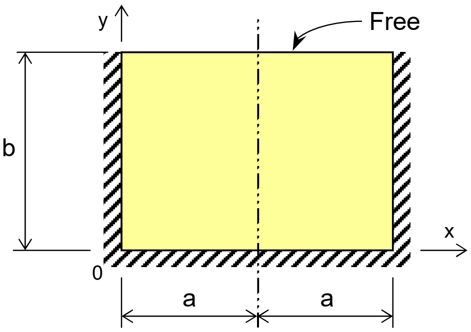
BY:	BS	**DATE:** Aug-21	**CLIENT** City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**	Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:		Stabilization Basin Perimeter Wall w/ Seismic Loads & Soil Backfill (BSE-2E)			

Rectangular Plate:

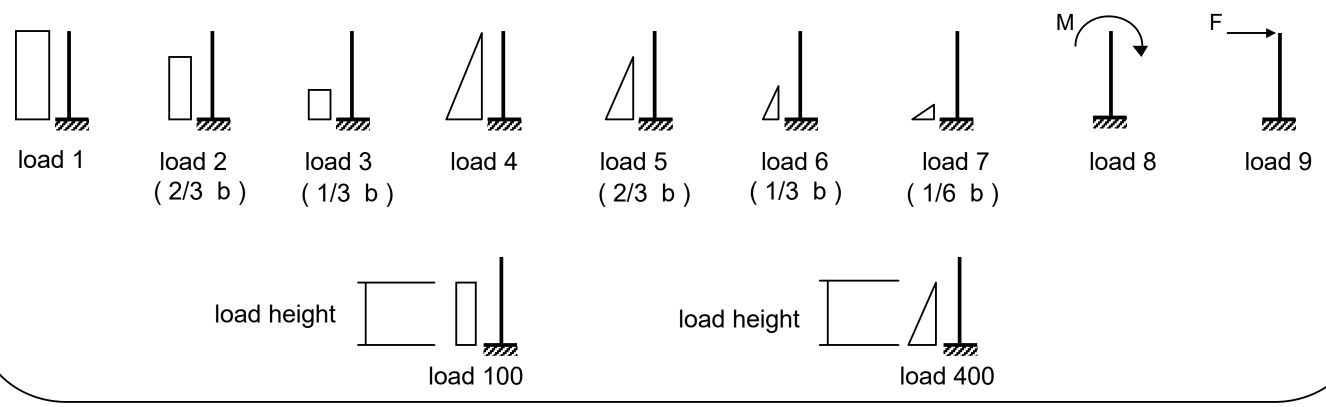
plate boundary condition case number (1, 2, 3, 4, or 5) = **1**

total plate width = 2*a = 2 * 12.5 = 25 ft
plate dimension, a = **12.5** ft
plate dimension, b = **18.67** ft

plate sides ratio, a/b = 0.6695



Available Loading Selections - (loads 1 thru 9 , 100 , or 400)



Choice of Available Loadings					
load conditions (4 max)	load type Loading Selection Number	load height, (ft) ...only for custom loads **100** or **400**	unfactored loads: q , M , or F (ksf, ft-k/ft, k/ft)	concrete load factors for moment	for shear
A	**100**	**16.670**	**0.166**	**1**	**1**
B	**400**	**16.670**	**0.263**	**1**	**1**
C	**400**	**16.670**	**1.040**	**1**	**1**
D	**400**	**10.280**	**-0.411**	**0.9**	**0.9**

Notes: 1). Load 100 = uniform load of any load height ≥ b/3; Load 400 = triangular load of any load height ≥ b/6.
2). load height must be less than or equal to "b", and uniform load height ≥ "b / 3", and triangular load height ≥ "b / 6" .
3). loads may be positive or negative.

plate thickness, h = **12** in

concrete strength, f 'c = **4** ksi
reinforcing steel strength, fy = **60** ksi
reinforcing clear cover to face of concrete = **2** in
number of curtains of reinforcing, (1 or 2) = **2**
Are bars in "x" or "y" direction closest to face of concrete ? **y**
minimum ratio of horizontal shrinkage-temperature steel = **0.00500**
minimum ratio of vertical shrinkage-temperature steel = **0.00500**

bar locations	d (in)	d' (in)
Mx bending	9"	3"
My bending	9.5"	2.5"

carollo
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BY:	BS	**DATE:** Aug-21	**CLIENT**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**		Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:		Stabilization Basin Perimieter Wall w/ Seismic Loads & Soil Backfill (BSE-2E)				

M_x - Moment Summary

		Loads: q , M , or F				Boundary Case 1				SUMMARY			
a = 12.5		0.166	0.263	1.040	-0.411					Final Moments		Reinforcing: (d = 9")	
b = 18.67		Moment Coefficient Multipliers											
a / b = 0.6695		57.862	91.674	362.512	-143.262								
		Moment Coefficients				M_x Moments, ft-k/ft				M_x	M_{ux}	$A_{s(req'd)}$	$A_{s(min)}$
x / a	y / b	A	B	C	D	A	B	C	D	ft-k/ft	ft-k/ft	in^2/ft	in^2/ft
0	1	0.1035	0.0228	0.0228	0.0037	5.99	2.09	8.28	-0.52	15.84	15.89	0.41	0.36
0	0.8	0.0996	0.0272	0.0272	0.0067	5.76	2.50	9.88	-0.96	17.18	17.27	0.44	0.36
0	0.6	0.0900	0.0303	0.0303	0.0105	5.21	2.78	10.99	-1.50	17.47	17.62	0.45	0.36
0	0.4	0.0668	0.0284	0.0284	0.0134	3.87	2.60	10.30	-1.92	14.85	15.04	0.38	0.36
0	0.2	0.0272	0.0150	0.0150	0.0092	1.57	1.37	5.43	-1.32	7.05	7.19	0.18	0.36
0	0	0.0000	0.0000	0.0000	0.0000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.36
0.2	0	0.0034	0.0022	0.0022	0.0015	0.20	0.20	0.79	-0.21	0.97	0.99	0.02	0.36
0.4	0	0.0089	0.0050	0.0050	0.0032	0.51	0.46	1.80	-0.45	2.32	2.36	0.06	0.36
0.6	0	0.0139	0.0071	0.0071	0.0043	0.80	0.65	2.58	-0.61	3.43	3.49	0.09	0.36
0.8	0	0.0171	0.0085	0.0085	0.0049	0.99	0.78	3.07	-0.70	4.14	4.21	0.10	0.36
1	0	0.0182	0.0089	0.0089	0.0051	1.05	0.82	3.24	-0.73	4.38	4.45	0.11	0.36
1	0.2	-0.0070	-0.0039	-0.0039	-0.0024	-0.41	-0.35	-1.40	0.34	-1.82	-1.86	-0.05	-0.36
1	0.4	-0.0288	-0.0115	-0.0115	-0.0051	-1.66	-1.06	-4.18	0.74	-6.17	-6.24	-0.16	-0.36
1	0.6	-0.0417	-0.0138	-0.0138	-0.0048	-2.41	-1.27	-5.02	0.68	-8.02	-8.08	-0.20	-0.36
1	0.8	-0.0473	-0.0136	-0.0136	-0.0037	-2.74	-1.25	-4.92	0.53	-8.38	-8.43	-0.21	-0.36
1	1	-0.0516	-0.0134	-0.0134	-0.0031	-2.99	-1.23	-4.85	0.45	-8.61	-8.66	-0.22	-0.36
0.8	1	-0.0459	-0.0117	-0.0117	-0.0027	-2.66	-1.07	-4.25	0.38	-7.60	-7.64	-0.19	-0.36
0.8	0.8	-0.0424	-0.0121	-0.0121	-0.0032	-2.45	-1.11	-4.38	0.46	-7.48	-7.52	-0.19	-0.36
0.8	0.6	-0.0379	-0.0127	-0.0127	-0.0044	-2.19	-1.16	-4.59	0.63	-7.32	-7.38	-0.19	-0.36
0.8	0.4	-0.0266	-0.0109	-0.0109	-0.0049	-1.54	-1.00	-3.94	0.70	-5.77	-5.84	-0.15	-0.36
0.8	0.2	-0.0066	-0.0038	-0.0038	-0.0024	-0.38	-0.35	-1.39	0.35	-1.78	-1.81	-0.04	-0.36

max negative moment, $M_{ux}(-)$ = -8.66 ft-k/ft
max negative steel req'd, $A_s(-)$ = -0.22 in^2/ft
minimum steel req'd = -0.36 in^2/ft
Use

max positive moment, $M_{ux}(+)$ = 17.62 ft-k/ft
max positive steel req'd, $A_s(+)$ = 0.45 in^2/ft
minimum steel req'd = 0.36 in^2/ft
Use

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BY:	BS	**DATE:** Aug-21	**CLIENT** City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**	Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:		Stabilization Basin Perimieter Wall w/ Seismic Loads & Soil Backfill (BSE-2E)			

M_y - Moment Summary

		Loads: q , M , or F				Boundary Case 1				SUMMARY			
a = 12.5		0.166	0.263	1.040	-0.411					Final Moments		Reinforcing: (d = 9.5")	
b = 18.67		Moment Coefficient Multipliers											
a / b = 0.6695		57.862	91.674	362.512	-143.262								
		Moment Coefficients				M_y Moments, ft-k/ft				M_y	M_{uy}	$A_{s(req'd)}$	$A_{s(min)}$
x / a	y / b	A	B	C	D	A	B	C	D	ft-k/ft	ft-k/ft	in^2/ft	in^2/ft
0	1	0.0000	0.0000	0.0000	0.0000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.36
0	0.8	0.0199	0.0055	0.0055	0.0014	1.15	0.50	1.99	-0.20	3.45	3.47	0.08	0.36
0	0.6	0.0180	0.0061	0.0061	0.0021	1.04	0.56	2.21	-0.30	3.50	3.53	0.08	0.36
0	0.4	0.0134	0.0057	0.0057	0.0027	0.77	0.52	2.06	-0.38	2.97	3.01	0.07	0.36
0	0.2	0.0055	0.0030	0.0030	0.0018	0.32	0.28	1.09	-0.26	1.42	1.45	0.03	0.36
0	0	0.0000	0.0000	0.0000	0.0000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.36
0.2	0	0.0170	0.0109	0.0109	0.0077	0.98	1.00	3.97	-1.11	4.85	4.96	0.12	0.36
0.4	0	0.0446	0.0248	0.0248	0.0158	2.58	2.27	8.99	-2.26	11.59	11.81	0.28	0.38
0.6	0	0.0692	0.0357	0.0357	0.0213	4.01	3.28	12.96	-3.06	17.19	17.49	0.42	0.38
0.8	0	0.0854	0.0424	0.0424	0.0244	4.94	3.88	15.36	-3.50	20.68	21.03	0.51	0.38
1	0	0.0910	0.0445	0.0445	0.0254	5.26	4.08	16.15	-3.64	21.86	22.22	0.54	0.38
1	0.2	0.0101	-0.0015	-0.0015	-0.0041	0.58	-0.14	-0.56	0.58	0.47	0.41	0.01	0.36
1	0.4	-0.0211	-0.0132	-0.0132	-0.0079	-1.22	-1.21	-4.79	1.13	-6.09	-6.20	-0.15	-0.36
1	0.6	-0.0245	-0.0099	-0.0099	-0.0036	-1.42	-0.90	-3.58	0.51	-5.39	-5.44	-0.13	-0.36
1	0.8	-0.0125	-0.0037	-0.0037	-0.0004	-0.72	-0.34	-1.34	0.05	-2.35	-2.36	-0.06	-0.36
1	1	0.0000	0.0000	0.0000	0.0000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.36
1	0.4	-0.0211	-0.0132	-0.0132	-0.0079	-1.22	-1.21	-4.79	1.13	-6.09	-6.20	-0.15	-0.36
0.8	0.4	-0.0199	-0.0126	-0.0126	-0.0075	-1.15	-1.15	-4.55	1.07	-5.78	-5.89	-0.14	-0.36
0.6	0.4	-0.0161	-0.0104	-0.0104	-0.0063	-0.93	-0.96	-3.78	0.90	-4.77	-4.86	-0.11	-0.36
0.4	0.4	-0.0094	-0.0067	-0.0067	-0.0041	-0.54	-0.61	-2.42	0.59	-2.98	-3.04	-0.07	-0.36
0.2	0.4	0.0007	-0.0010	-0.0010	-0.0010	0.04	-0.09	-0.37	0.14	-0.29	-0.30	-0.01	-0.36

max negative moment, $M_{uy}(-)$ = -6.20 ft-k/ft
max negative steel req'd, $A_s(-)$ = -0.15 in^2/ft
minimum steel req'd = -0.36 in^2/ft
Use

max positive moment, $M_{uy}(+)$ = 22.22 ft-k/ft
max positive steel req'd, $A_s(+)$ = 0.54 in^2/ft
minimum steel req'd = 0.38 in^2/ft
Use

Engineers...Working Wonders With Water ™

BY:	BS	DATE:	Aug-21	CLIENT	City of Wilsonville	SHEET:	
CHKD:		DESCRIPTION:			Aeration Basins	JOB NO:	11962A.00
DESIGN TASK:					Stabilization Basin Perimieter Wall w/ Seismic Loads & Soil Backfill (BSE-2E)		

Shear Summary

		Loads: q , M , or F				Boundary Case 1				SUMMARY		
a = 12.5		0.166	0.263	1.040	-0.411							
b = 18.67		Shear Coefficient Multipliers								Final Shears		
a / b = 0.6695		3.099	4.910	19.417	-7.673							
		Shear Coefficients				Shears, k/ft				V	V_u	ϕV_c
x / a	y / b	A	B	C	D	A	B	C	D	k/ft	k/ft	k/ft
0	1	0.4370	0.0425	0.0425	-0.0160	1.35	0.21	0.83	0.12	2.51	2.50	10.81
0	0.8	0.5116	0.1304	0.1304	0.0208	1.59	0.64	2.53	-0.16	4.60	4.61	10.81
0	0.6	0.5283	0.1797	0.1797	0.0567	1.64	0.88	3.49	-0.44	5.57	5.62	10.81
0	0.4	0.4585	0.2287	0.2287	0.1202	1.42	1.12	4.44	-0.92	6.06	6.15	10.81
0	0.2	0.1599	0.1437	0.1437	0.1168	0.50	0.71	2.79	-0.90	3.09	3.18	10.81
0	0.00	-0.0543	-0.0108	-0.0108	0.0075	-0.17	-0.05	-0.21	-0.06	-0.49	-0.48	10.81
0.2	0	0.0863	0.1156	0.1156	0.1155	0.27	0.57	2.24	-0.89	2.19	2.28	10.81
0.4	0	0.3128	0.2396	0.2396	0.1922	0.97	1.18	4.65	-1.48	5.32	5.47	10.81
0.6	0	0.4704	0.3103	0.3103	0.2286	1.46	1.52	6.02	-1.75	7.25	7.43	10.81
0.8	0	0.5594	0.3460	0.3460	0.2448	1.73	1.70	6.72	-1.88	8.27	8.46	10.81
1	0	0.5880	0.3568	0.3568	0.2494	1.82	1.75	6.93	-1.91	8.59	8.78	10.81

Concrete strength reduction factor for shear, ϕ = 0.75

d = 9.5 in

maximum shear, V_u = 8.78 k/ft **OK**

$\phi V_c = \phi*2*(f'c)^{1/2}*b*d$ = (0.75*2*(4000)^½ *12*9.5)/1000 = 10.81 k/ft

Reference:

"Moments and Reactions for Rectangular Plates"
Engineering Monograph No. 27
By: W. T. Moody, United States Bureau of Reclamation

Notes:

Quadratic interpolation is used for intermediate values within the Moody tables and between Moody figures.
The positive sign convention for moments M_x and M_y is tension on the loaded face of the plate.
The M_x moment is in the direction of the x-axis and the M_y moment is in the direction of the y-axis by plate sign convention.

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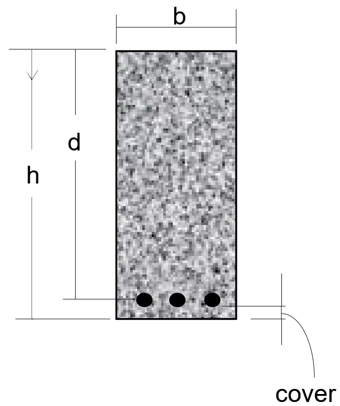
BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:		Stabilization Basins Perimeter Wall Strength (Vertical Reinforcing) (BSE-2E)					

Concrete Shear and Moment Strength Capacity

Design for … beam, slab, wall ? **wall**



Properties and Geometry

Compression width of wall, b =	**12**	inch	f'_c (psi) =	**4000**
Thickness of wall, h =	**12**	inch	f_y (psi) =	**60000**
Depth to reinforcing, d =	**9**	inch	ϕ, Bending =	**1**
factored design moment, M_u =	**22.22**	ft-k	ϕ, Shear =	**1**
factored design shear, V_u =	**8.78**	kip	E_s (psi) =	29000000
			E_c (psi) =	3604997
			$n = E_s / E_c$ =	8.04
			β_1 =	0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi*2*b*d*(f'_c)^{1/2}$ = 13.66 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #8@12" vert reinf**

Area steel provided, A_s = **0.79** in^2 $\rho = A_s / bd$ = 0.00731

ρ(min) < As/bd < ρ(max) - OK

$A_{s\ (min)}$ =	0.19	in^2	$\rho_{(min)}$ =	0.00180
$A_{s\ (max)}$ =	2.31	in^2	$\rho_{(max)}$ =	0.02138

bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

$\phi*M_n$ = 1*0.00731*60*12*9² *(1-0.588*0.00731*60/4)*(ft/12) = 33.256 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

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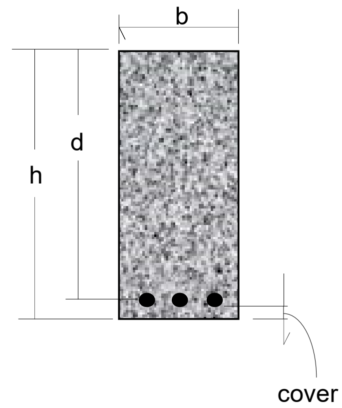
DESIGN TASK:	Stabilization Basins Perimeter Wall Strength (Horizontal Reinforcing) (BSE-2E)				

Concrete Shear and Moment Strength Capacity



Design for … beam, slab, wall ? **wall**

Properties and Geometry

Compression width of wall, b = **12** inch

Thickness of wall, h = **12** inch

Depth to reinforcing, d = **9** inch

factored design moment, M_u = **17.62** ft-k

factored design shear, V_u = **5.62** kip

f'_c (psi) = **4000**

f_y (psi) = **60000**

ϕ, Bending = **1**

ϕ, Shear = **1**

E_s (psi) = 29000000

E_c (psi) = 3604997

$n = E_s / E_c$ = 8.04

β_1 = 0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi * 2 * b * d * (f'_c)^{1/2}$ = 13.66 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #8@12" horiz reinf outside face**

Area steel provided, A_s = **0.79** in² $\rho = A_s / bd$ = 0.00731

ρ(min) < As/bd < ρ(max) - OK

$A_{s\,(min)}$ = 0.19 in² $\rho_{(min)}$ = 0.00180

$A_{s\,(max)}$ = 2.31 in² $\rho_{(max)}$ = 0.02138

bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

$\phi * M_n$ = 1*0.00731*60*12*9² *(1-0.588*0.00731*60/4)*(ft/12) = 33.256 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

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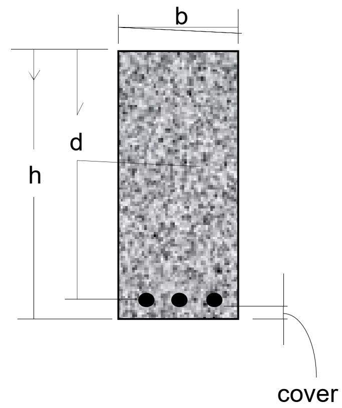
DESIGN TASK:		Stabilization Basins Perimeter Wall Strength (Horizontal Reinforcing) (BSE-2E)					

Concrete Shear and Moment Strength Capacity

Design for … beam, slab, wall ? **wall**



Properties and Geometry

Compression width of wall, b = **12** inch

Thickness of wall, h = **12** inch

Depth to reinforcing, d = **9** inch

factored design moment, M_u = **8.66** ft-k

factored design shear, V_u = **5.62** kip

f'_c (psi) = **4000**

f_y (psi) = **60000**

ϕ, Bending = **1**

ϕ, Shear = **1**

E_s (psi) = 29000000

E_c (psi) = 3604997

$n = E_s / E_c$ = 8.04

β_1 = 0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi*2*b*d*(f'_c)^{1/2}$ = 13.66 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #7@12" horiz reinf inside face**

Area steel provided, A_s = **0.6** in^2 $\rho = A_s / bd$ = 0.00556

$\rho(min) < As/bd < \rho(max)$ - OK

$A_{s\ (min)}$ = 0.19 in^2 $\rho_{(min)}$ = 0.00180

$A_{s\ (max)}$ = 2.31 in^2 $\rho_{(max)}$ = 0.02138

bending strength, $$\phi M_n = \phi\ \rho\ f_y\ b\ d^2 \left(1 - \frac{0.588\ \rho\ f_y}{f'_c}\right)$$

$\phi*M_n$ = 1*0.00556*60*12*9² *(1-0.588*0.00556*60/4)*(ft/12) = 25.676 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

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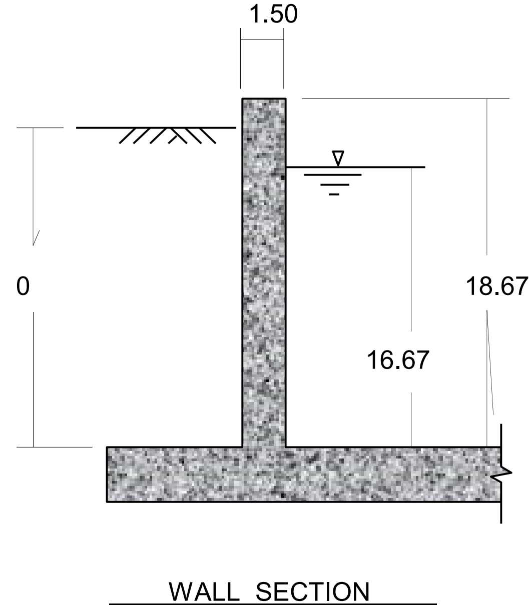
DESIGN TASK:					Stabilizatin Basins (Transverse Direction) (CSZ)		

Static, Seismic, and Hydrodynamic Seismic Wall Pressures using ACI 350.3-06 and an IBC 2012:

wall connection fixity =	**no roof & fixed at floor**	
tank unit width perpendicular to EQ., B =	1	ft
tank inside length in direction of seismic, L =	**25**	ft
tank wall thickness, t_w =	**18**	inch
wall height, H_w =	**18.67**	ft
liquid height, H_L =	**16.67**	ft
liquid specific gravity =	**1**	
liquid density, $\gamma_L = (sp.gr.)*\gamma_w$ =	0.0624	k/ft^3
acceleration due to gravity, g =	32.17	ft/sec^2
liquid mass density, $\rho_L = \gamma_L / g$ =	0.00194	$k\text{-}sec^2/ft^4$



Soil Data

The site has no groundwater.

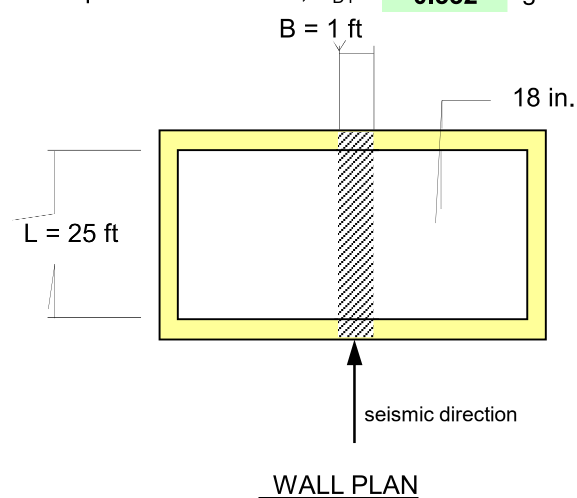
soil height above top of foundation base =	**0**	ft
groundwater ht. above foundation base =	**0**	ft
dry soil lateral pressure =	**0**	k/ft^3
saturated soil lateral pressure =	**0**	k/ft^3
dry soil unit weight =	**0**	k/ft^3
live load lateral surcharge =	**0.000**	ksf
	0	
concrete strength, f'_c =	**4**	ksi
concrete density, γ_c =	0.150	k/ft^3
concrete modulus of elasticity, E_c =	3605.0	ksi
concrete mass density, $\rho_c = \gamma_c / g$ =	0.004663	$k\text{-}sec^2/ft^4$

Seismic:

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, S_{DS} = **0.446** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, S_{D1} = **0.332** *g

Structure Risk Category =	**3**
Importance factor, I =	**1.25**
Response modification factor, R_{wi} =	**3**
Response modification factor, R_{wc} =	**1**



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

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DESIGN TASK:			Stabilizatin Basins (Transverse Direction) (CSZ)				

Weights:

unit 1-ft width wall mass, W_w = (18/12) * (18.67) * 0.15 = 4.20 kip
wall c.g. relative to base, h_w = 18.67 / 2 = 9.335 ft

unit width liquid mass, W_L = (25) * (1) * (16.67) * 32.17 = 26.01 kip

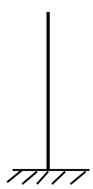
Seismic:

1). structure stiffness and dynamic property:

Note: per ASCE 7-10 and IBC 2012, the terms S_{ai} and S_{ac} have been appropriatetly substituted into the seismic equation of ACI 350.

Note: Wi and hi are impulsive component variables calculated on page 3.

wall mass, $m_w = H_w*(t_w / 12)*\rho_c$ = 0.13058 k-sec^2/ft^2
liquid mass, $m_i = (W_i / W_L)*(L/2)*H_L*\rho_L$ = 0.26806 k-sec^2/ft^2
centroidal distance of masses, $h = (h_w*m_w + h_i*m_i) / (m_w + m_i)$ = 7.261 ft



wall fixity condition is no roof & fixed at floor:
wall stiffness is determined using a unit mass load located at the centroidal distance h.
wall flexure stiffness, k = Ec*(tw/h)³ / 48 = 1144.17 k/ft/ft

$$\omega_i = \sqrt{\frac{k}{m_w + m_i}} =$$ (1144.17 / (0.1306 + 0.2681))ˆ½ = 53.5744 rad/sec

period of tank plus impulsive mass, $T_i = 2\pi / \omega_i$ = 2 π / 53.5744 = 0.1173 sec

(note: acceleration values to be from a maximum considered earthquake response spectra which will produce a factored load)

design factored spectral response acceleration for impulsive mass (5% damping), $S_{ai} = S_{DS}$ = 0.446 g

2). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$\lambda = \sqrt{3.16\ g\ \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)} =$$ (3.16*32.2*tanh(3.16*(0.6668)))ˆ½ = 9.9345

$$\omega_c = \frac{\lambda}{\sqrt{L}} =$$ 9.9345 / (25)ˆ½ = 1.9869 rad/sec,

period of the convective mass, $T_c = 2\pi / \omega_c$ = 2 π / 1.9869 = 3.1623 sec

Long transition period (from map figure 22-15 ASCE 7), T_L = **16** sec

design spectral response acceleration for convective mass (0.5% damping), S_{ac} = 1.5 * Sd1 / Tc = 0.158 g

$$\text{effective mass coeff., } \varepsilon = 0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021\ ,\ \text{ but } \leq\ 1.0 =$$ 0.7688

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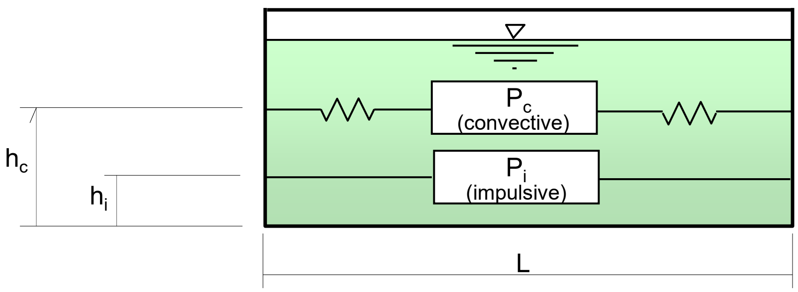
DESIGN TASK:				Stabilizatin Basins (Transverse Direction) (CSZ)			



Dynamic Model

L =	25	ft
B =	1	ft
H_L =	16.67	ft
W_L =	26.01	kip
L / H_L =	1.49970	
H_L / L =	0.66680	

3). lateral fluid impulsive force:

Wi = equivalent mass of the impulsive component of liquid.

$$W_i = W_L\left(\frac{\tanh\left(0.866\frac{L}{H_L}\right)}{0.866\frac{L}{H_L}}\right) =$$ 26.01*(tanh(0.866*(1.4997)) / 0.866*(1.4997)) = 17.25 kip

hi (EBP) = HL * 0.375 = 16.67 * 0.375 = 6.251 ft

hi (IBP) = HL * {{(0.866*L/HL)/(2*tanh(0.866*L/HL))} -1/8 } = 10.483 ft

impulsive force, $P_i = \left(\frac{S_{ai}\ I}{R_{wi}}\right)W_i =$ (0.446 * 1.25 / 3)*17.25 = 3.2 kip

4). lateral fluid convective force:

Wc = equivalent mass of the convective component of liquid.

$$W_c = W_L\left(0.264\left(\frac{L}{H_L}\right)\tanh\left(3.16\left(\frac{H_L}{L}\right)\right)\right) =$$ 26.01*(0.264*(1.4997)*tanh(3.16*(0.6668))) = 10 kip

$$h_{c\ (EBP)} = H_L\left(1-\frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right)-1}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)}\right) = 10.474 \text{ ft}$$

$$h_{c\ (IBP)} = H_L\left(1-\frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right)-2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)}\right) = 12.446 \text{ ft}$$

convective force, $P_c = \left(\frac{S_{ac}\ I}{R_{wc}}\right)W_c =$ (0.1575 * 1.25 / 1)*10 = 2.0 kip

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5). <u>lateral inertia force of the accelerating wall:</u>

unit width wall mass, W_w = 4.20 kip

wall c.g. relative to base, h_w = 9.335 ft

wall inertia force, $P_w = \left(\dfrac{S_{ai}\ I\ \varepsilon}{R_{wi}}\right) W_w =$ (0.446*1.25*0.7688/3)*4.2 = 0.60 kip

6). <u>maximum wave slosh height displacement:</u>

$d_{(max)} = \left(\dfrac{L}{2}\right)\left(\dfrac{S_{ac}}{1.4}\ I\right) =$ (25 / 2) * (0.1575 / 1.0 * 1.25) = 2.46 ft

7). <u>vertical acceleration:</u>

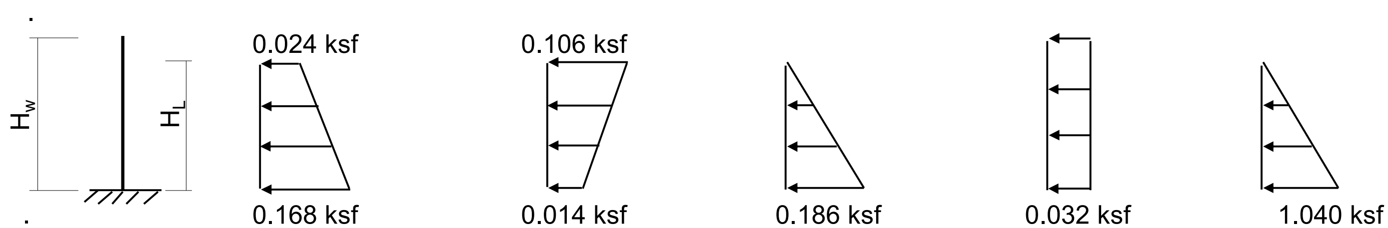
design horizontal accereration, S_{DS} = 0.446 *g

vertical spectral response acceleration (per ACI 350 para 9.4.3), $S_{av} = C_t = 0.4{*}S_{DS}$ = 0.1784 g

per ASCE 7-10 para. 15.7.7.2(b), use $I = R_i = b = 1.0$

Design vertical acceleration, $\ddot{u} = \dfrac{S_{av}\ I\ b}{R_i} =$ 0.1784*1*1/1 = 0.1784 g

8). <u>vertical force distribution on a unit width using the linear distribution of ACI 350 sec 5.3:</u>



H_w = 18.67

H_L = 16.67

<u>impulsive:</u>

$$p_{iy} = \frac{P_i\left[4H_L - 6h_i - \left(6H_L - 12h_i\right)\left(\dfrac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_i =	3.20	kip
h_i =	6.251	ft
at y = H_L, p_{iy} =	0.024	ksf
at base y = 0, p_{iy} =	0.168	ksf

<u>convective:</u>

$$p_{cy} = \frac{P_c\left[4H_L - 6h_c - \left(6H_L - 12h_c\right)\left(\dfrac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_c =	2.00	kip
h_c =	10.474	ft
at y = H_L, p_{cy} =	0.106	ksf
at base y = 0, p_{cy} =	0.014	ksf

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vertical acceleration:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

ü = 0.1784

at $y = H_L$, p_{vy} = 0.000 ksf

at base $y = 0$, p_{vy} = 0.186 ksf

wall inertia:

$$p_{wy} = \frac{S_{ai}\ I\ \varepsilon\ \gamma_c\ (t_w/12)}{R_{wi}} =$$

p_{wy} = 0.1429 * γ_c * $(t_w/12)$

at $y = H_w$, p_{wy} = 0.032 ksf

at base $y = 0$, p_{wy} = 0.032 ksf

hydrostatic:

$$q_{hy} = \gamma_L\,(H_L - y) =$$

at $y = H_L$, q_{hy} = 0.000 ksf

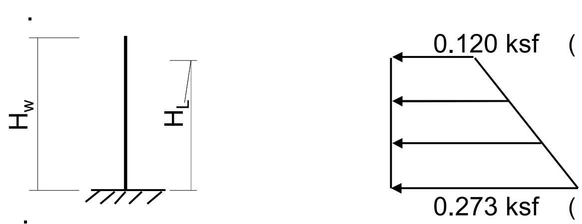
at base $y = 0$, q_{hy} = 1.040 ksf

combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{(p_{ly} + p_{wy})^2 + p_{cy}^2 + p_{vy}^2} =$$

at $y = H_w$, p_y = 0.120 ksf

at base $y = 0$, p_y = 0.273 ksf

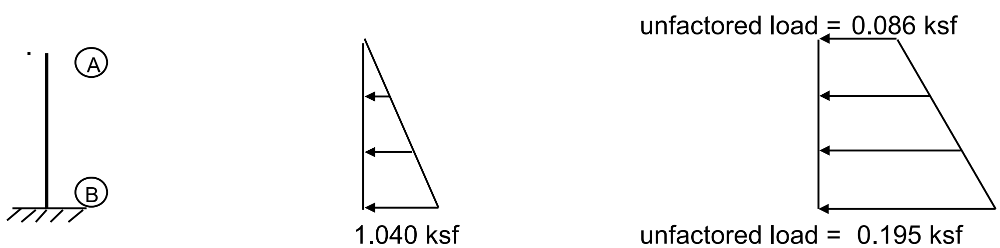


resultant dynamic pressures

9). wall design pressures for hydrostatic + dynamic:

wall height, H_w = 18.67 ft

liquid height, H_L = 16.67 ft



hydrostatic

resultant dynamic pressures

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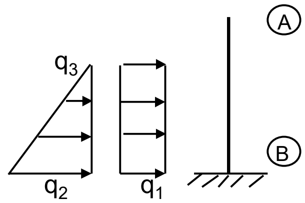
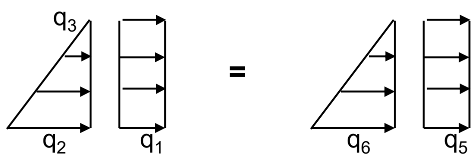
10). wall design pressures for external soil loading:

static soil:



The site has no groundwater.		
wall height =	18.67	ft
soil height above top of base =	0	ft
groundwater ht. above base =	0	ft
dry soil lateral pressure =	0.000	k/ft^3
sat. soil lateral pressure =	0.000	k/ft^3
live load lateral surcharge =	0.000	ksf

equivalent static soil loadings:



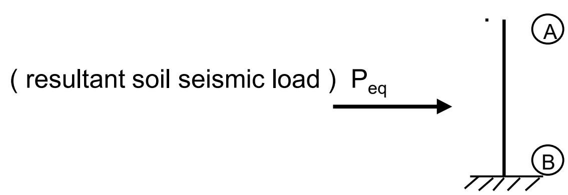
LL lateral surcharge, q1 =	0.0000	ksf
unfactored soil, q2 =	0.0000	ksf
unfactored soil, q3 =	0.0000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.0000	ksf
unfactored q6 =	0.0000	ksf

soil seismic:

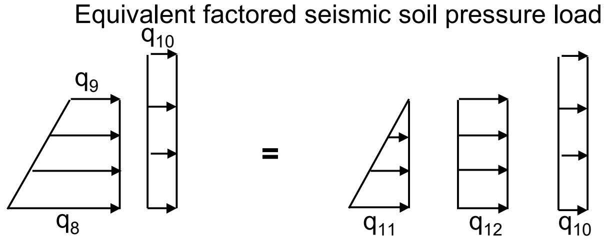
resultant factored soil seismic load per foot of wall width, $P_{u\ (eq)}$ = **0** k/ft

centroid location of the resultant soil seismic from the bottom of wall, h_{eq} = **0** ft

The resultant soil seismic load will be resolved into an equivalent pressure loading…



Equivalent factored seismic soil pressure loading & seismic wall loadings…



equivalent soil seismic, q8 =	0.0000	ksf
equivalent soil seismic, q9 =	0.0000	ksf
wall seismic (see wall page 5), q10 =	0.0321	ksf
equivalent soil seismic, q11 = q8 - q9 =	0.0000	ksf
equivalent soil seismic, q12 = q9 =	0.0000	ksf

unfactored equivalent soil seismic, q8 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q9 =0 / 1.4 =	0.0000	ksf
unfactored wall seismic , q10 = 0.0321 / 1.4 =	0.0230	ksf
unfactored equivalent soil seismic, q11 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q12 =0 / 1.4 =	0.0000	ksf

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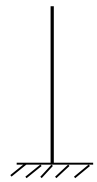
CHKD:		**DESCRIPTION:**			Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:				Stabilizatin Basins (Transverse Direction) (CSZ)			

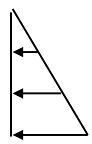
11). Summary of equivalent unfactored pressure loadings for wall load cases 1 thru 4:



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

a). load case 1: hydrostatic water

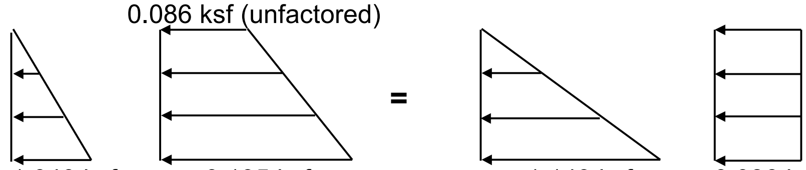
wall height = 18.67 ft
water depth = 16.67 ft



(unfactored) 1.040 ksf

b). load case 2: hydrostatic + dynamic:

0.086 ksf (unfactored)



1.040 ksf (unfactored) hydrostatic	0.195 ksf (unfactored) resultant dynamic pressures	=	1.149 ksf (unfactored)	0.086 ksf (unfactored)

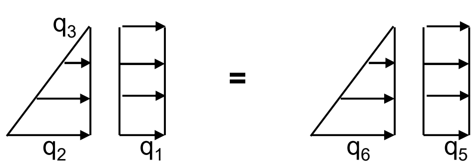
wall height = 18.67 ft
water depth = 16.67 ft

c). load case 3: static soil + LL surcharge:

wall height = 18.67 ft
soil height on wall = 0 ft

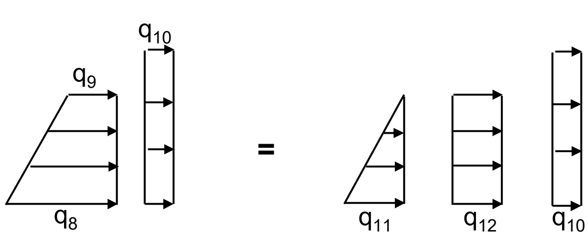
equivalent static soil & surcharge loadings...

LL lateral surcharge, q1 =	0.000	ksf
unfactored soil, q2 =	0.000	ksf
unfactored soil, q3 =	0.000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.000	ksf
unfactored q6 =	0.000	ksf



d). load case 4: soil seismic: (*note: add static soil pressure q6 & q7 to the seismic soil shown below)
equivalent seismic soil pressure loading & seismic wall loadings…

wall height = 18.67 ft
soil height on wall = 0 ft



unfactored equivalent soil seismic, q8 =	0.000	ksf
unfactored equivalent soil seismic, q9 =	0.000	ksf
unfactored equivalent soil seismic, q10 =	0.023	ksf
unfactored equivalent soil seismic, q11 =	0.000	ksf
unfactored equivalent soil seismic, q12 =	0.000	ksf

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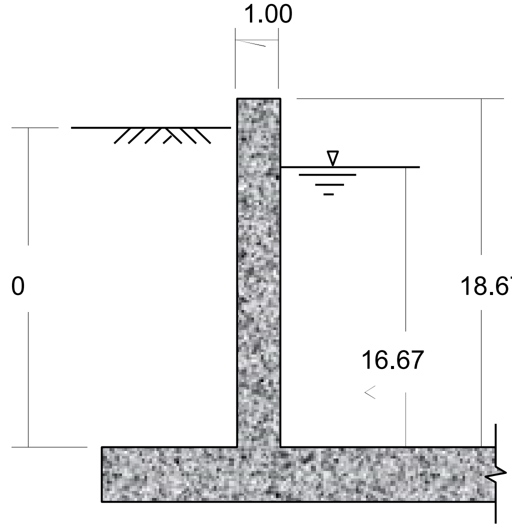
DESIGN TASK:					Stabilizatin Basins (Longitudinal Direction) (CSZ)		

Static, Seismic, and Hydrodynamic Seismic Wall Pressures using ACI 350.3-06 and an IBC 2012:

wall connection fixity =	**no roof & fixed at floor**	
tank unit width perpendicular to EQ., B =	1	ft
tank inside length in direction of seismic, L =	**60**	ft
tank wall thickness, t_w =	**12**	inch
wall height, H_w =	**18.67**	ft
liquid height, H_L =	**16.67**	ft
liquid specific gravity =	**1**	
liquid density, γ_L = (sp.gr.)*γ_w =	0.0624	k/ft^3
acceleration due to gravity, g =	32.17	ft/sec^2
liquid mass density, $\rho_L = \gamma_L / g$ =	0.00194	$k\text{-}sec^2/ft^4$



WALL SECTION

Soil Data

The site has no groundwater.

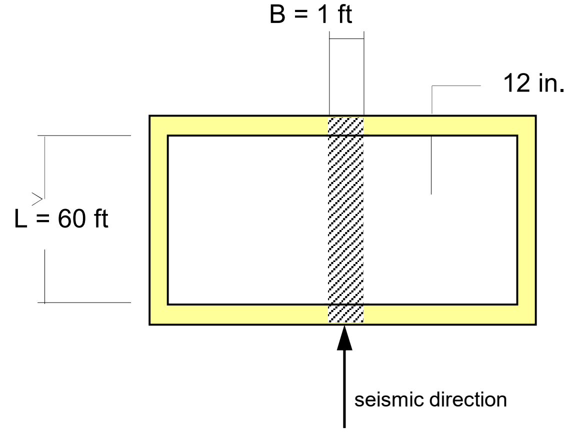
soil height above top of foundation base =	**0**	ft
groundwater ht. above foundation base =	**0**	ft
dry soil lateral pressure =	**0**	k/ft^3
saturated soil lateral pressure =	**0**	k/ft^3
dry soil unit weight =	**0**	k/ft^3
live load lateral surcharge =	**0.000**	ksf
	0	
concrete strength, f'_c =	**4**	ksi
concrete density, γ_c =	0.150	k/ft^3
concrete modulus of elasticity, E_c =	3605.0	ksi
concrete mass density, $\rho_c = \gamma_c / g$ =	0.004663	$k\text{-}sec^2/ft^4$

Seismic:

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, S_{DS} = **0.446** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, S_{D1} = **0.332** *g

Structure Risk Category =	**3**
Importance factor, I =	**1.25**
Response modification factor, R_{wi} =	**3**
Response modification factor, R_{wc} =	**1**



WALL PLAN

Load Cases:
- case 1 = water
- case 2 = water + water seismic + wall seismic
- case 3 = soil + lateral surcharge
- case 4 = soil + soil seismic + wall seismic

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DESIGN TASK:			Stabilizatin Basins (Longitudinal Direction) (CSZ)				

Weights:

unit 1-ft width wall mass, W_w =	(12/12) * (18.67) * 0.15 =	2.80	kip
wall c.g. relative to base, h_w =	18.67 / 2 =	9.335	ft
unit width liquid mass, W_L =	(60) * (1) * (16.67) * 32.17 =	62.41	kip

Seismic:

1). structure stiffness and dynamic property:

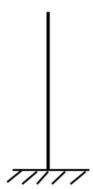
Note: per ASCE 7-10 and IBC 2012, the terms S_{ai} and S_{ac} have been appropriatetly substituted into the seismic equation of ACI 350.

Note: Wi and hi are impulsive component variables calculated on page 3.

wall mass, $m_w = H_w*(t_w / 12)*\rho_c$ = 0.08705 k-sec^2/ft^2

liquid mass, $m_i = (W_i / W_L)*(L/2)*H_L*\rho_L$ = 0.30993 k-sec^2/ft^2

centroidal distance of masses, $h = (h_w*m_w + h_i*m_i) / (m_w + m_i)$ = 6.927 ft



wall fixity condition is no roof & fixed at floor:

wall stiffness is determined using a unit mass load located at the centroidal distance h.

wall flexure stiffness, k = Ec*(tw/h)³ / 48 = 390.46 k/ft/ft

$$\omega_i = \sqrt{\frac{k}{m_w + m_i}} =$$ (390.46 / (0.0871 + 0.3099))ˆ½ = 31.3618 rad/sec

period of tank plus impulsive mass, $T_i = 2\pi / \omega_i$ = 2 π / 31.3618 = 0.2003 sec

(note: acceleration values to be from a maximum considered earthquake response spectra which will produce a factored load)

design factored spectral response acceleration for impulsive mass (5% damping), $S_{ai} = S_{DS}$ = 0.446 g

2). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$\lambda = \sqrt{3.16\ g\ \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)} =$$ (3.16*32.2*tanh(3.16*(0.2778)))ˆ½ = 8.4681

$$\omega_c = \frac{\lambda}{\sqrt{L}} =$$ 8.4681 / (60)ˆ½ = 1.0932 rad/sec,

period of the convective mass, $T_c = 2\pi / \omega_c$ = 2 π / 1.0932 = 5.7474 sec

Long transition period (from map figure 22-15 ASCE 7), T_L = **16** sec

design spectral response acceleration for convective mass (0.5% damping), S_{ac} = 1.5 * Sd1 / Tc = 0.087 g

$$\text{effective mass coeff., } \varepsilon = 0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021\ ,\quad \text{but} \le\ 1.0\ =$$ 0.5299

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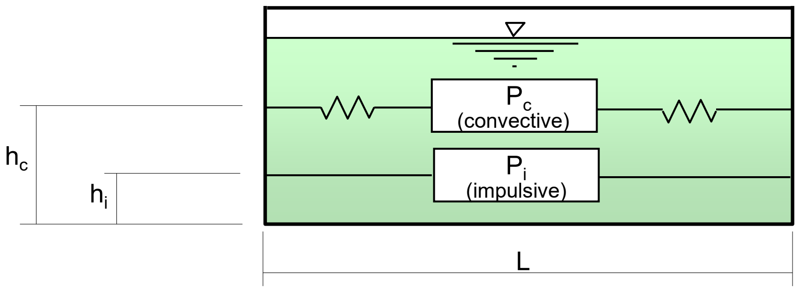
DESIGN TASK:				Stabilizatin Basins (Longitudinal Direction) (CSZ)			



L =	60	ft
B =	1	ft
H_L =	16.67	ft
W_L =	62.41	kip
L / H_L =	3.59928	
H_L / L =	0.27783	

Dynamic Model

3). lateral fluid impulsive force:

Wi = equivalent mass of the impulsive component of liquid.

$$W_i = W_L \left(\frac{\tanh\left(0.866 \frac{L}{H_L}\right)}{0.866 \frac{L}{H_L}} \right) =$$ 62.41*(tanh(0.866*(3.5993)) / 0.866*(3.5993)) = 19.94 kip

hi (EBP) = HL * 0.375 = 16.67 * 0.375 = 6.251 ft

hi (IBP) = HL * {{(0.866*L/HL)/(2*tanh(0.866*L/HL))} -1/8 } = 23.998 ft

impulsive force, $P_i = \left(\frac{S_{ai} \; I}{R_{wi}} \right) W_i =$ (0.446 * 1.25 / 3)*19.94 = 3.7 kip

4). lateral fluid convective force:

Wc = equivalent mass of the convective component of liquid.

$$W_c = W_L \left(0.264 \left(\frac{L}{H_L} \right) \tanh\left(3.16 \left(\frac{H_L}{L} \right) \right) \right) =$$ 62.41*(0.264*(3.5993)*tanh(3.16*(0.2778))) = 41.83 kip

$$h_{c\ (EBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 1}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) = \quad 8.832 \text{ ft}$$

$$h_{c\ (IBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) = \quad 28.102 \text{ ft}$$

convective force, $P_c = \left(\frac{S_{ac} \, I}{R_{wc}} \right) W_c =$ (0.0866 * 1.25 / 1)*41.83 = 4.5 kip

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5). lateral inertia force of the accelerating wall:

unit width wall mass, W_w = 2.80 kip

wall c.g. relative to base, h_w = 9.335 ft

$$\text{wall inertia force, } P_w = \left(\frac{S_{ai}\ I\ \varepsilon}{R_{wi}}\right) W_w = \quad (\ 0.446*1.25*0.5299/3\ )*2.8 = \quad 0.28 \ \text{kip}$$

6). maximum wave slosh height displacement:

$$d_{(max)} = \left(\frac{L}{2}\right)\left(\frac{S_{ac}}{1.4}\ I\right) = \quad (\ 60 / 2)\ *\ (\ 0.0866 / 1.4 * 1.25\ ) = \quad 3.25 \ \text{ft}$$

Wave height is greater than the freeboard of 2-ft. Check effects of wave spillage.

7). vertical acceleration:

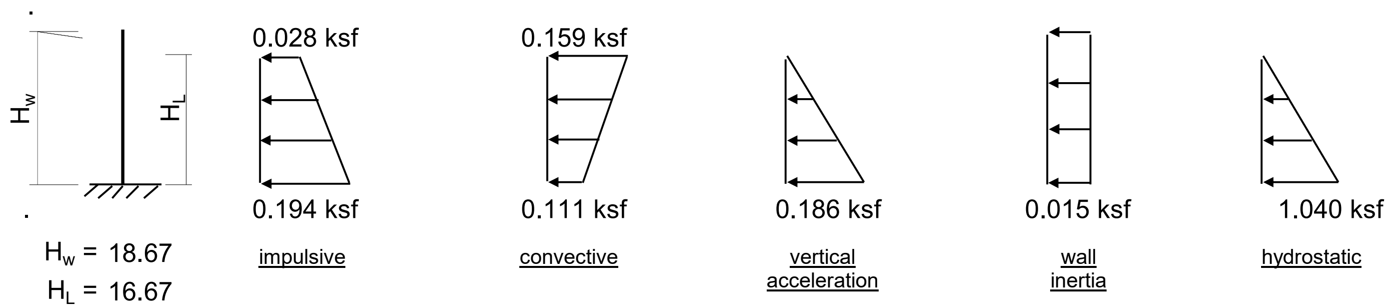
design horizontal accereration, S_{DS} = 0.446 *g

vertical spectral response acceleration (per ACI 350 para 9.4.3), $S_{av} = C_t = 0.4*S_{DS}$ = 0.1784 g

per ASCE 7-10 para. 15.7.7.2(b), use $I = R_i = b = 1.0$

$$\text{Design vertical acceleration, } \ddot{u} = \frac{S_{av}\ I\ b}{R_i} = \quad 0.1784*1*1/1 = \quad 0.1784 \ \text{g}$$

8). vertical force distribution on a unit width using the linear distribution of ACI 350 sec 5.3:



H_w = 18.67

H_L = 16.67

impulsive:

$$p_{iy} = \frac{P_i\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_i =	3.70	kip
h_i =	6.251	ft
at $y = H_L$, p_{iy} =	0.028	ksf
at base $y = 0$, p_{iy} =	0.194	ksf

convective:

$$p_{cy} = \frac{P_c\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_c =	4.50	kip
h_c =	8.832	ft
at $y = H_L$, p_{cy} =	0.159	ksf
at base $y = 0$, p_{cy} =	0.111	ksf

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vertical acceleration:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

$\ddot{u}$ = 0.1784
at y = H_L, p_{vy} = 0.000 ksf
at base y = 0, p_{vy} = 0.186 ksf

wall inertia:

$$p_{wy} = \frac{S_{ai}\;\; I\;\; \varepsilon\,\gamma_c\,(t_w/12)}{R_{wi}} =$$

p_{wy} = 0.0985 * γ_c * (t_w/12)
at y = H_w, p_{wy} = 0.015 ksf
at base y = 0, p_{wy} = 0.015 ksf

hydrostatic:

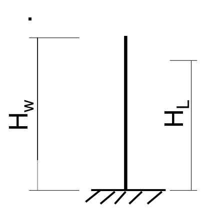
$$q_{hy} = \gamma_L\,(H_L - y) =$$

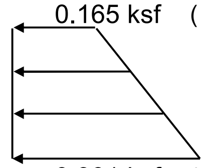
at y = H_L, q_{hy} = 0.000 ksf
at base y = 0, q_{hy} = 1.040 ksf

combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{(p_{iy} + p_{wy})^2 + p_{cy}^2 + p_{vy}^2} =$$

at y = H_w, p_y = 0.165 ksf
at base y = 0, p_y = 0.301 ksf





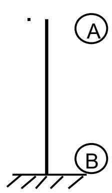
0.165 ksf (unfactored = 0.165 / 1.4 = 0.118 ksf)

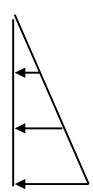
0.301 ksf (unfactored = 0.301 / 1.4 = 0.215 ksf)

resultant dynamic pressures

9). wall design pressures for hydrostatic + dynamic:

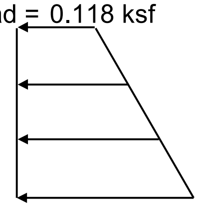
wall height, H_w = 18.67 ft
liquid height, H_L = 16.67 ft





1.040 ksf
hydrostatic

unfactored load = 0.118 ksf



unfactored load = 0.215 ksf
resultant dynamic pressures

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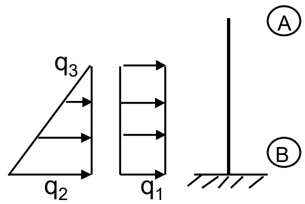
DESIGN TASK:				Stabilizatin Basins (Longitudinal Direction) (CSZ)			

10). wall design pressures for external soil loading:

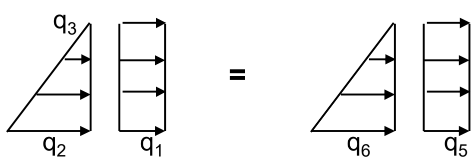
static soil:



The site has no groundwater.

wall height =	18.67	ft
soil height above top of base =	0	ft
groundwater ht. above base =	0	ft
dry soil lateral pressure =	0.000	k/ft^3
sat. soil lateral pressure =	0.000	k/ft^3
live load lateral surcharge =	0.000	ksf

equivalent static soil loadings:



LL lateral surcharge, q1 =	0.0000	ksf
unfactored soil, q2 =	0.0000	ksf
unfactored soil, q3 =	0.0000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.0000	ksf
unfactored q6 =	0.0000	ksf

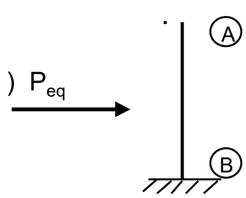
soil seismic:

resultant factored soil seismic load per foot of wall width, $P_{u\,(eq)}$ = **0** k/ft

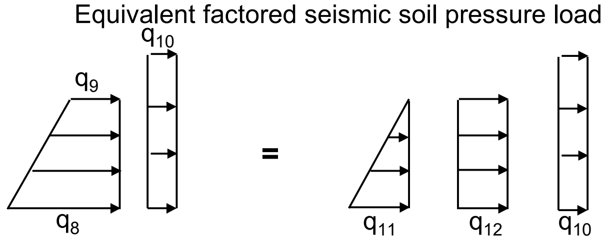
centroid location of the resultant soil seismic from the bottom of wall, h_{eq} = **0** ft

The resultant soil seismic load will be resolved into an equivalent pressure loading…

(resultant soil seismic load) P_{eq}



Equivalent factored seismic soil pressure loading & seismic wall loadings…



equivalent soil seismic, q8 =	0.0000	ksf
equivalent soil seismic, q9 =	0.0000	ksf
wall seismic (see wall page 5), q10 =	0.0148	ksf
equivalent soil seismic, q11 = q8 - q9 =	0.0000	ksf
equivalent soil seismic, q12 = q9 =	0.0000	ksf

unfactored equivalent soil seismic, q8 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q9 =0 / 1.4 =	0.0000	ksf
unfactored wall seismic , q10 = 0.0148 / 1.4 =	0.0106	ksf
unfactored equivalent soil seismic, q11 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q12 =0 / 1.4 =	0.0000	ksf

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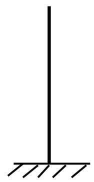
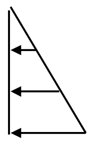
11). Summary of equivalent unfactored pressure loadings for wall load cases 1 thru 4:



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

a). load case 1: hydrostatic water

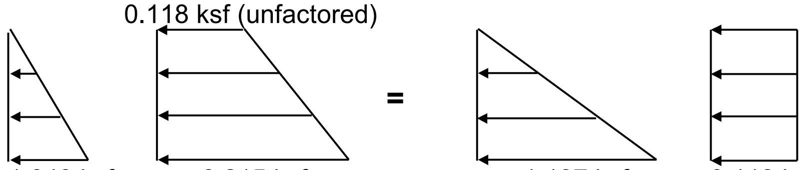
wall height = 18.67 ft
water depth = 16.67 ft



(unfactored) 1.040 ksf

b). load case 2: hydrostatic + dynamic:

0.118 ksf (unfactored)



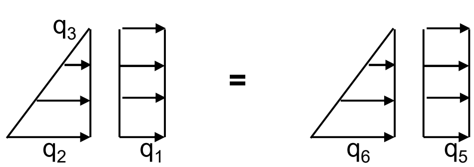
wall height = 18.67 ft
water depth = 16.67 ft

1.040 ksf	0.215 ksf	1.137 ksf	0.118 ksf
(unfactored)	(unfactored)	(unfactored)	(unfactored)
hydrostatic	resultant dynamic pressures		

c). load case 3: static soil + LL surcharge:

wall height = 18.67 ft
soil height on wall = 0 ft

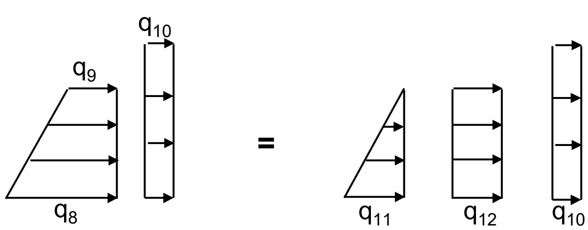
equivalent static soil & surcharge loadings...



LL lateral surcharge, q1 =	0.000	ksf
unfactored soil, q2 =	0.000	ksf
unfactored soil, q3 =	0.000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.000	ksf
unfactored q6 =	0.000	ksf

d). load case 4: soil seismic: (*note: add static soil pressure q6 & q7 to the seismic soil shown below)
equivalent seismic soil pressure loading & seismic wall loadings…

wall height = 18.67 ft
soil height on wall = 0 ft



unfactored equivalent soil seismic, q8 =	0.000	ksf
unfactored equivalent soil seismic, q9 =	0.000	ksf
unfactored equivalent soil seismic, q10 =	0.011	ksf
unfactored equivalent soil seismic, q11 =	0.000	ksf
unfactored equivalent soil seismic, q12 =	0.000	ksf

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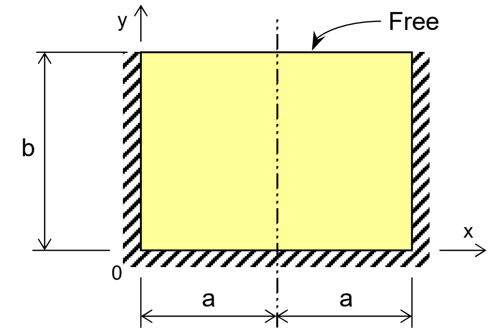
CHKD:	**DESCRIPTION:**		Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:			Stabilization Basin Perimieter Wall w/ Seismic Loads & Soil Backfill (CSZ)		

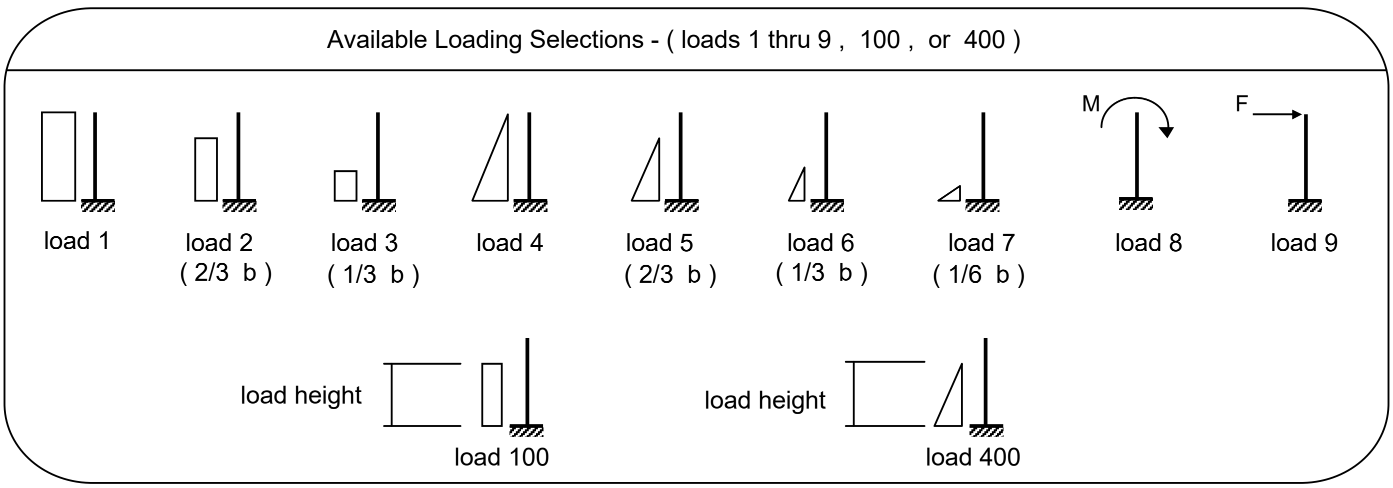
Rectangular Plate:

plate boundary condition case number (1, 2, 3, 4, or 5) = **1**

total plate width = 2*a = 2 * 12.5 = 25 ft
plate dimension, a = **12.5** ft
plate dimension, b = **18.67** ft

plate sides ratio, a/b = 0.6695





Choice of Available Loadings					
load conditions (4 max)	load type Loading Selection Number	load height, (ft) ...only for custom loads 100 or 400	unfactored loads: q , M , or F (ksf, ft-k/ft, k/ft)	concrete load factors for moment	concrete load factors for shear
A	**100**	**16.670**	**0.165**	**1**	**1**
B	**400**	**16.670**	**0.136**	**1**	**1**
C	**400**	**16.670**	**1.040**	**1**	**1**
D	**400**	**10.280**	**-0.411**	**0.9**	**0.9**

Notes: 1). Load 100 = uniform load of any load height ≥ b/3; Load 400 = triangular load of any load height ≥ b/6.
2). load height must be less than or equal to "b", and uniform load height ≥ "b / 3", and triangular load height ≥ "b / 6" .
3). loads may be positive or negative.

plate thickness, h = **12** in

concrete strength, f 'c = **4** ksi
reinforcing steel strength, fy = **60** ksi
reinforcing clear cover to face of concrete = **2** in
number of curtains of reinforcing, (1 or 2) = **2**
Are bars in "x" or "y" direction closest to face of concrete ? **y**
minimum ratio of horizontal shrinkage-temperature steel = **0.00500**
minimum ratio of vertical shrinkage-temperature steel = **0.00500**

bar locations	d (in)	d' (in)
Mx bending	9"	3"
My bending	9.5"	2.5"

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BY:	BS	DATE:	Aug-21	CLIENT	City of Wilsonville	SHEET:	
CHKD:		DESCRIPTION:			Aeration Basins	JOB NO:	11962A.00
DESIGN TASK:					Stabilization Basin Perimieter Wall w/ Seismic Loads & Soil Backfill (CSZ)		

M_x - Moment Summary

		Loads: q , M , or F				Boundary Case 1				SUMMARY			
a =	12.5	0.165	0.136	1.040	-0.411					Final Moments		Reinforcing: (d = 9")	
b =	18.67	Moment Coefficient Multipliers											
a / b =	0.6695	57.514	47.405	362.512	-143.262								
		Moment Coefficients				M_x Moments, ft-k/ft				M_x	M_{ux}	$A_{s(req'd)}$	$A_{s(min)}$
x / a	y / b	A	B	C	D	A	B	C	D	ft-k/ft	ft-k/ft	in^2/ft	in^2/ft
0	1	0.1035	0.0228	0.0228	0.0037	5.95	1.08	8.28	-0.52	14.79	14.84	0.38	0.36
0	0.8	0.0996	0.0272	0.0272	0.0067	5.73	1.29	9.88	-0.96	15.94	16.03	0.41	0.36
0	0.6	0.0900	0.0303	0.0303	0.0105	5.17	1.44	10.99	-1.50	16.10	16.25	0.42	0.36
0	0.4	0.0668	0.0284	0.0284	0.0134	3.84	1.35	10.30	-1.92	13.57	13.76	0.35	0.36
0	0.2	0.0272	0.0150	0.0150	0.0092	1.56	0.71	5.43	-1.32	6.38	6.51	0.16	0.36
0	0	0.0000	0.0000	0.0000	0.0000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.36
0.2	0	0.0034	0.0022	0.0022	0.0015	0.19	0.10	0.79	-0.21	0.88	0.90	0.02	0.36
0.4	0	0.0089	0.0050	0.0050	0.0032	0.51	0.24	1.80	-0.45	2.09	2.14	0.05	0.36
0.6	0	0.0139	0.0071	0.0071	0.0043	0.80	0.34	2.58	-0.61	3.11	3.17	0.08	0.36
0.8	0	0.0171	0.0085	0.0085	0.0049	0.98	0.40	3.07	-0.70	3.76	3.83	0.10	0.36
1	0	0.0182	0.0089	0.0089	0.0051	1.05	0.42	3.24	-0.73	3.98	4.05	0.10	0.36
1	0.2	-0.0070	-0.0039	-0.0039	-0.0024	-0.40	-0.18	-1.40	0.34	-1.65	-1.68	-0.04	-0.36
1	0.4	-0.0288	-0.0115	-0.0115	-0.0051	-1.65	-0.55	-4.18	0.74	-5.65	-5.72	-0.14	-0.36
1	0.6	-0.0417	-0.0138	-0.0138	-0.0048	-2.40	-0.66	-5.02	0.68	-7.39	-7.46	-0.19	-0.36
1	0.8	-0.0473	-0.0136	-0.0136	-0.0037	-2.72	-0.64	-4.92	0.53	-7.76	-7.81	-0.20	-0.36
1	1	-0.0516	-0.0134	-0.0134	-0.0031	-2.97	-0.63	-4.85	0.45	-8.00	-8.05	-0.20	-0.36
0.8	1	-0.0459	-0.0117	-0.0117	-0.0027	-2.64	-0.56	-4.25	0.38	-7.07	-7.10	-0.18	-0.36
0.8	0.8	-0.0424	-0.0121	-0.0121	-0.0032	-2.44	-0.57	-4.38	0.46	-6.93	-6.97	-0.17	-0.36
0.8	0.6	-0.0379	-0.0127	-0.0127	-0.0044	-2.18	-0.60	-4.59	0.63	-6.75	-6.81	-0.17	-0.36
0.8	0.4	-0.0266	-0.0109	-0.0109	-0.0049	-1.53	-0.51	-3.94	0.70	-5.28	-5.35	-0.13	-0.36
0.8	0.2	-0.0066	-0.0038	-0.0038	-0.0024	-0.38	-0.18	-1.39	0.35	-1.61	-1.64	-0.04	-0.36

max negative moment, $M_{ux}(-)$ = -8.05 ft-k/ft

max negative steel req'd, $A_s(-)$ = -0.20 in^2/ft

minimum steel req'd = -0.36 in^2/ft

Use

max positive moment, $M_{ux}(+)$ = 16.25 ft-k/ft

max positive steel req'd, $A_s(+)$ = 0.42 in^2/ft

minimum steel req'd = 0.36 in^2/ft

Use

Engineers...Working Wonders With Water ™

BY:	BS	DATE:	Aug-21	CLIENT	City of Wilsonville	SHEET:	
CHKD:		DESCRIPTION:			Aeration Basins	JOB NO:	11962A.00
DESIGN TASK:					Stabilization Basin Perimieter Wall w/ Seismic Loads & Soil Backfill (CSZ)		

M_y - Moment Summary

		Loads: q, M, or F				Boundary Case 1				SUMMARY			
a =	12.5	0.165	0.136	1.040	-0.411					Final Moments		Reinforcing: (d = 9.5")	
b =	18.67	Moment Coefficient Multipliers											
a / b =	0.6695	57.514	47.405	362.512	-143.262								
		Moment Coefficients				M_y Moments, ft-k/ft				M_y	M_{uy}	$A_{s(req'd)}$	$A_{s(min)}$
x / a	y / b	A	B	C	D	A	B	C	D	ft-k/ft	ft-k/ft	in^2/ft	in^2/ft
0	1	0.0000	0.0000	0.0000	0.0000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.36
0	0.8	0.0199	0.0055	0.0055	0.0014	1.14	0.26	1.99	-0.20	3.20	3.22	0.08	0.36
0	0.6	0.0180	0.0061	0.0061	0.0021	1.03	0.29	2.21	-0.30	3.22	3.25	0.08	0.36
0	0.4	0.0134	0.0057	0.0057	0.0027	0.77	0.27	2.06	-0.38	2.72	2.76	0.06	0.36
0	0.2	0.0055	0.0030	0.0030	0.0018	0.31	0.14	1.09	-0.26	1.29	1.31	0.03	0.36
0	0	0.0000	0.0000	0.0000	0.0000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.36
0.2	0	0.0170	0.0109	0.0109	0.0077	0.98	0.52	3.97	-1.11	4.36	4.47	0.11	0.36
0.4	0	0.0446	0.0248	0.0248	0.0158	2.56	1.18	8.99	-2.26	10.47	10.70	0.26	0.36
0.6	0	0.0692	0.0357	0.0357	0.0213	3.98	1.69	12.96	-3.06	15.58	15.88	0.38	0.38
0.8	0	0.0854	0.0424	0.0424	0.0244	4.91	2.01	15.36	-3.50	18.78	19.13	0.46	0.38
1	0	0.0910	0.0445	0.0445	0.0254	5.23	2.11	16.15	-3.64	19.85	20.22	0.49	0.38
1	0.2	0.0101	-0.0015	-0.0015	-0.0041	0.58	-0.07	-0.56	0.58	0.53	0.47	0.01	0.36
1	0.4	-0.0211	-0.0132	-0.0132	-0.0079	-1.21	-0.63	-4.79	1.13	-5.50	-5.61	-0.13	-0.36
1	0.6	-0.0245	-0.0099	-0.0099	-0.0036	-1.41	-0.47	-3.58	0.51	-4.94	-4.99	-0.12	-0.36
1	0.8	-0.0125	-0.0037	-0.0037	-0.0004	-0.72	-0.18	-1.34	0.05	-2.18	-2.19	-0.05	-0.36
1	1	0.0000	0.0000	0.0000	0.0000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.36
1	0.4	-0.0211	-0.0132	-0.0132	-0.0079	-1.21	-0.63	-4.79	1.13	-5.50	-5.61	-0.13	-0.36
0.8	0.4	-0.0199	-0.0126	-0.0126	-0.0075	-1.14	-0.60	-4.55	1.07	-5.22	-5.33	-0.13	-0.36
0.6	0.4	-0.0161	-0.0104	-0.0104	-0.0063	-0.92	-0.49	-3.78	0.90	-4.30	-4.39	-0.10	-0.36
0.4	0.4	-0.0094	-0.0067	-0.0067	-0.0041	-0.54	-0.32	-2.42	0.59	-2.68	-2.74	-0.06	-0.36
0.2	0.4	0.0007	-0.0010	-0.0010	-0.0010	0.04	-0.05	-0.37	0.14	-0.24	-0.26	-0.01	-0.36

max negative moment, $M_{uy}(-)$ = -5.61 ft-k/ft

max negative steel req'd, $A_s(-)$ = -0.13 in^2/ft

minimum steel req'd = -0.36 in^2/ft

Use

max positive moment, $M_{uy}(+)$ = 20.22 ft-k/ft

max positive steel req'd, $A_s(+)$ = 0.49 in^2/ft

minimum steel req'd = 0.38 in^2/ft

Use

Engineers...Working Wonders With Water ™

BY:	BS	DATE:	Aug-21	CLIENT	City of Wilsonville	SHEET:	
CHKD:		DESCRIPTION:			Aeration Basins	JOB NO:	11962A.00
DESIGN TASK:					Stabilization Basin Perimieter Wall w/ Seismic Loads & Soil Backfill (CSZ)		

Shear Summary

		Loads: q , M , or F				Boundary Case 1				SUMMARY		
a = 12.5		0.165	0.136	1.040	-0.411					Final Shears		
b = 18.67		Shear Coefficient Multipliers										
a / b = 0.6695		3.081	2.539	19.417	-7.673							
		Shear Coefficients				Shears, k/ft				V	V_u	ϕV_c
x / a	y / b	A	B	C	D	A	B	C	D	k/ft	k/ft	k/ft
0	1	0.4370	0.0425	0.0425	-0.0160	1.35	0.11	0.83	0.12	2.40	2.39	10.81
0	0.8	0.5116	0.1304	0.1304	0.0208	1.58	0.33	2.53	-0.16	4.28	4.29	10.81
0	0.6	0.5283	0.1797	0.1797	0.0567	1.63	0.46	3.49	-0.44	5.14	5.18	10.81
0	0.4	0.4585	0.2287	0.2287	0.1202	1.41	0.58	4.44	-0.92	5.51	5.60	10.81
0	0.2	0.1599	0.1437	0.1437	0.1168	0.49	0.36	2.79	-0.90	2.75	2.84	10.81
0	0.00	-0.0543	-0.0108	-0.0108	0.0075	-0.17	-0.03	-0.21	-0.06	-0.46	-0.46	10.81
0.2	0	0.0863	0.1156	0.1156	0.1155	0.27	0.29	2.24	-0.89	1.92	2.01	10.81
0.4	0	0.3128	0.2396	0.2396	0.1922	0.96	0.61	4.65	-1.48	4.75	4.90	10.81
0.6	0	0.4704	0.3103	0.3103	0.2286	1.45	0.79	6.02	-1.75	6.51	6.68	10.81
0.8	0	0.5594	0.3460	0.3460	0.2448	1.72	0.88	6.72	-1.88	7.44	7.63	10.81
1	0	0.5880	0.3568	0.3568	0.2494	1.81	0.91	6.93	-1.91	7.73	7.92	10.81

Concrete strength reduction factor for shear, ϕ = 0.75

d = 9.5 in

maximum shear, V_u = 7.92 k/ft **OK**

$\phi V_c = \phi * 2 * (f'c)^{1/2} * b * d$ = (0.75*2*(4000)^½ *12*9.5)/1000 = 10.81 k/ft

Reference:

"Moments and Reactions for Rectangular Plates"
Engineering Monograph No. 27
By: W. T. Moody, United States Bureau of Reclamation

Notes:

Quadratic interpolation is used for intermediate values within the Moody tables and between Moody figures.
The positive sign convention for moments M_x and M_y is tension on the loaded face of the plate.
The M_x moment is in the direction of the x-axis and the M_y moment is in the direction of the y-axis by plate sign convention.

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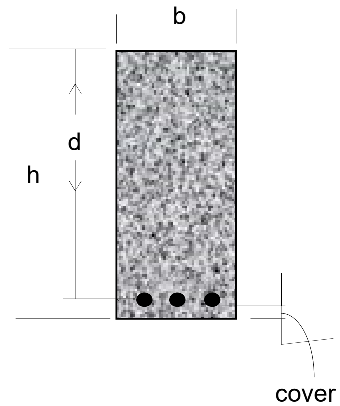
BY:	BS	DATE:	Aug-21	CLIENT:	City of Wilsonville	SHEET:	
CHKD:		DESCRIPTION:			Aeration Basins	JOB NO:	11962A.00
DESIGN TASK:		Stabilization Basins Perimeter Wall Strength (Vertical Reinforcing) (CSZ)					

Concrete Shear and Moment Strength Capacity



Design for … beam, slab, wall ? **wall**

Properties and Geometry

Compression width of wall, b =	**12**	inch
Thickness of wall, h =	**12**	inch
Depth to reinforcing, d =	**9**	inch
factored design moment, M_u =	**20.22**	ft-k
factored design shear, V_u =	**7.92**	kip

f'_c (psi) =	**4000**
f_y (psi) =	**60000**
ϕ, Bending =	**1**
ϕ, Shear =	**1**
E_s (psi) =	29000000
E_c (psi) =	3604997
$n = E_s / E_c$ =	8.04
β_1 =	0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi*2*b*d*(f'_c)^{1/2}$ = 13.66 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #8@12" vert reinf**

Area steel provided, A_s = **0.79** in^2 $\rho = A_s / bd$ = 0.00731

$\rho(min) < As/bd < \rho(max)$ - OK

$A_{s\,(min)}$ = 0.19 in^2 $\rho_{(min)}$ = 0.00180

$A_{s\,(max)}$ = 2.31 in^2 $\rho_{(max)}$ = 0.02138

bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

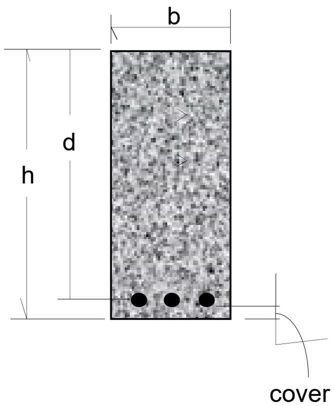
$\phi * M_n$ = 1*0.00731*60*12*9² *(1-0.588*0.00731*60/4)*(ft/12) = 33.256 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

Engineers...Working Wonders With Water™

BY:	BS	**DATE:**	Aug-21	**CLIENT:** City of Wilsonville	**SHEET:**
CHKD:		**DESCRIPTION:**		Aeration Basins	**JOB NO:** 11962A.00
DESIGN TASK:		Stabilization Basins Perimeter Wall Strength (Horizontal Reinforcing) (CSZ)			

Concrete Shear and Moment Strength Capacity



Design for ... beam, slab, wall ? **wall**

Properties and Geometry

Compression width of wall, b = **12** inch

Thickness of wall, h = **12** inch

Depth to reinforcing, d = **9** inch

factored design moment, M_u = **16.25** ft-k

factored design shear, V_u = **5.18** kip

f'_c (psi) = **4000**

f_y (psi) = **60000**

ϕ, Bending = **1**

ϕ, Shear = **1**

E_s (psi) = 29000000

E_c (psi) = 3604997

$n = E_s / E_c$ = 8.04

β_1 = 0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi * 2 * b * d * (f'_c)^{1/2}$ = 13.66 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #8@12" horiz reinf outside face**

Area steel provided, A_s = **0.79** in^2 $\rho = A_s / bd$ = 0.00731

$\rho(min) < As/bd < \rho(max)$ - OK

$A_{s\ (min)}$ = 0.19 in^2 $\rho_{(min)}$ = 0.00180

$A_{s\ (max)}$ = 2.31 in^2 $\rho_{(max)}$ = 0.02138

bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

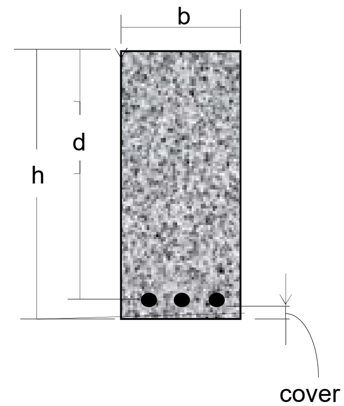
$\phi * M_n$ = 1*0.00731*60*12*9² *(1-0.588*0.00731*60/4)*(ft/12) = 33.256 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

Engineers...Working Wonders With Water™

BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Aeration Basins	**JOB NO:**	11962A.00
DESIGN TASK:		Stabilization Basins Perimeter Wall Strength (Horizontal Reinforcing) (CSZ)					

Concrete Shear and Moment Strength Capacity



Design for … beam, slab, wall ? **wall**

Properties and Geometry

Compression width of wall, b = **12** inch
Thickness of wall, h = **12** inch
Depth to reinforcing, d = **9** inch
factored design moment, M_u = **8.05** ft-k
factored design shear, V_u = **5.18** kip

f'_c (psi) = **4000**
f_y (psi) = **60000**
ϕ, Bending = **1**
ϕ, Shear = **1**
E_s (psi) = 29000000
E_c (psi) = 3604997
$n = E_s / E_c$ = 8.04
β_1 = 0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi * 2 * b * d * (f'_c)^{1/2}$ = 13.66 kip ≥ Vu
stirrup spacing, s = **0** in
stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #7@12" horiz reinf inside face**
Area steel provided, A_s = **0.6** in^2 $\rho = A_s / bd$ = 0.00556
ρ(min) < As/bd < ρ(max) - OK

$A_{s\ (min)}$ = 0.19 in^2 $\rho_{(min)}$ = 0.00180
$A_{s\ (max)}$ = 2.31 in^2 $\rho_{(max)}$ = 0.02138

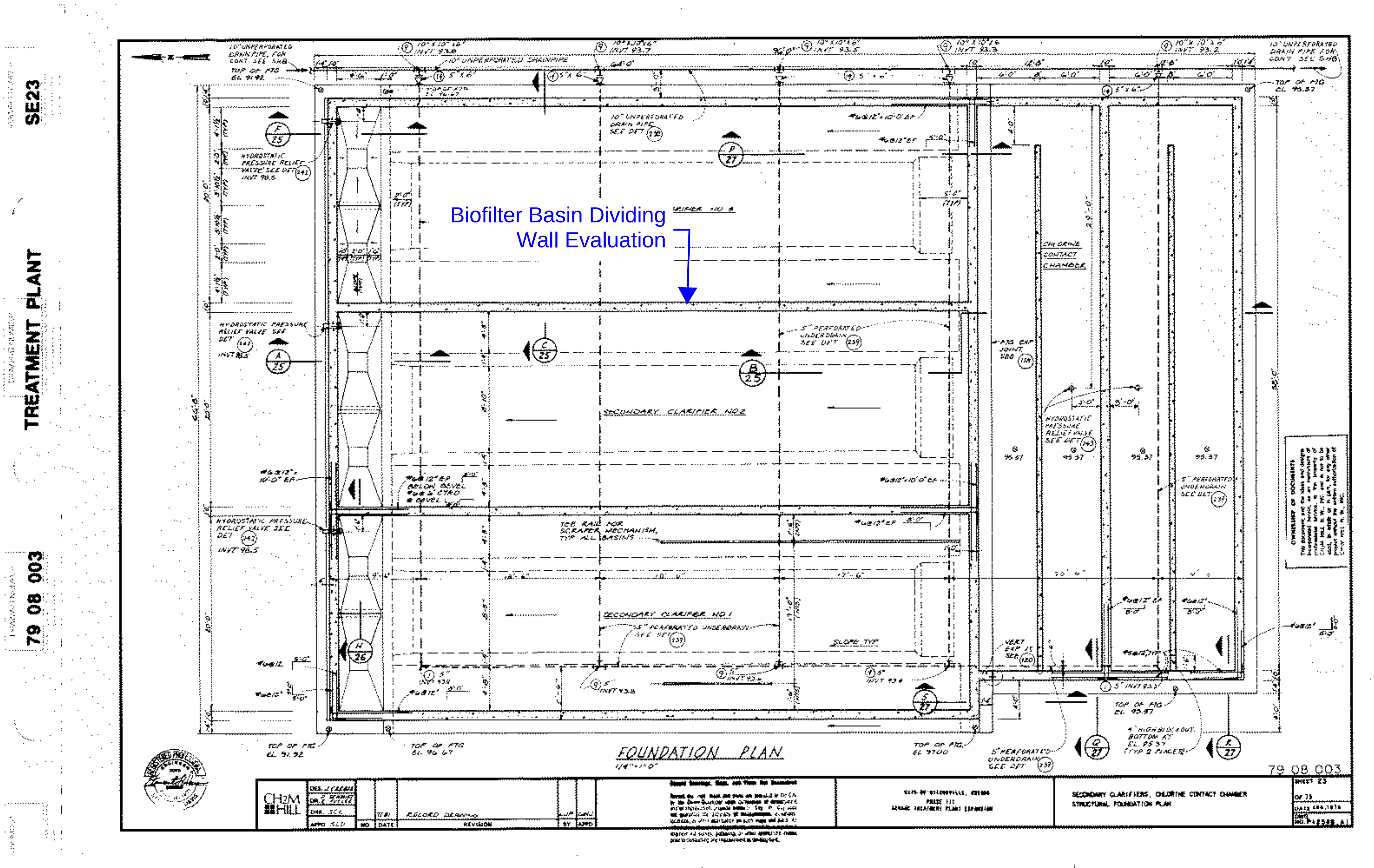
bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

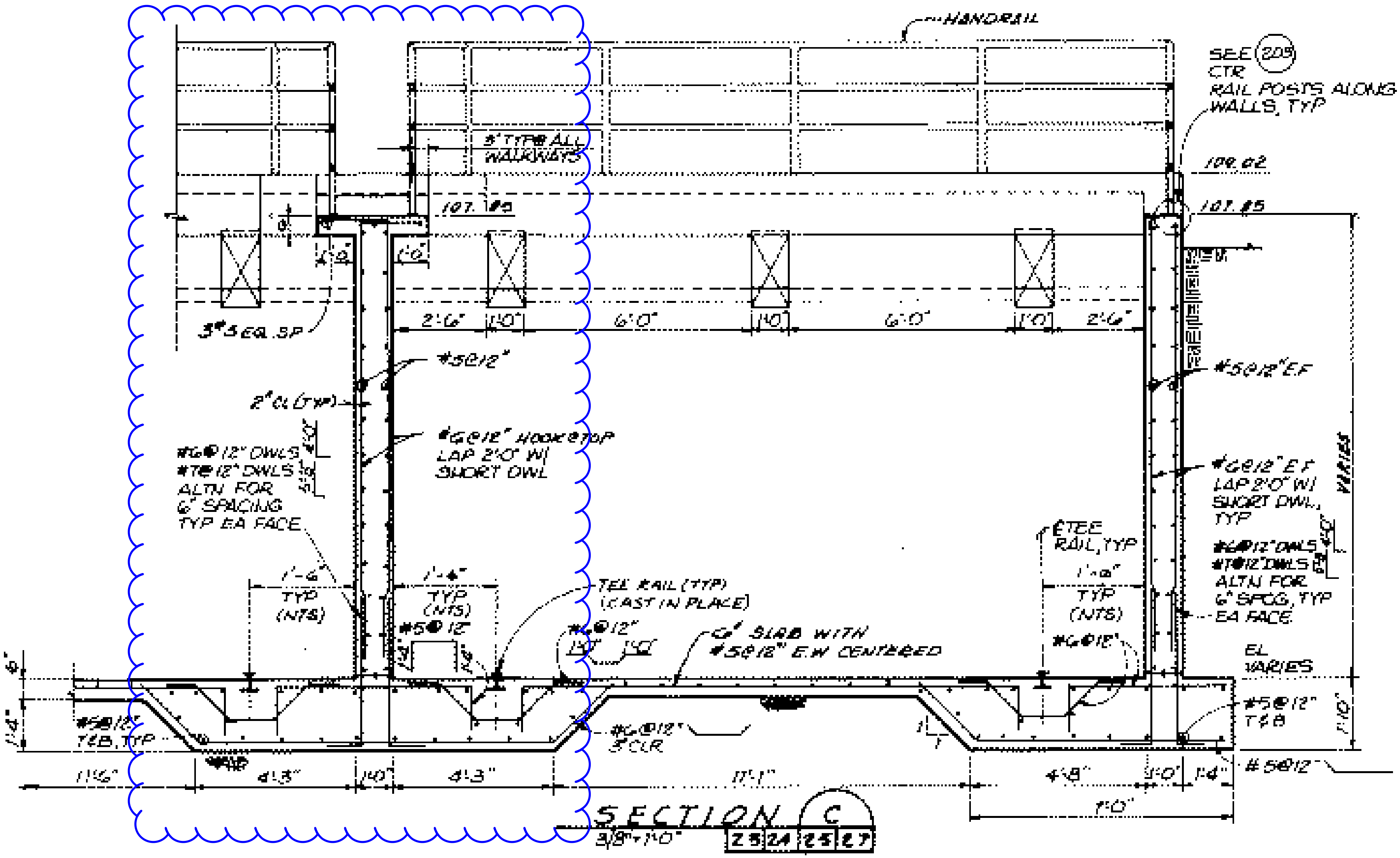
$\phi * M_n$ = 1*0.00556*60*12*9² *(1-0.588*0.00556*60/4)*(ft/12) = 25.676 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

City of Wilsonville

Sludge Storage Basins and Biofilter Structural Calculations


Biofilter Basin Dividing
Wall Evaluation
SE23
TREATMENT PLANT
79 08 003
SECONDARY CLARIFIER NO 2
SECONDARY CLARIFIER NO 1
CHLORINE CONTACT CHAMBER
FOUNDATION PLAN
1/4"=1'-0"
79 08 003
CH2M HILL
RECORD DRAWING
SECONDARY CLARIFIERS, CHLORINE CONTACT CHAMBER
STRUCTURAL FOUNDATION PLAN



Dividing Wall Section Reinforcing

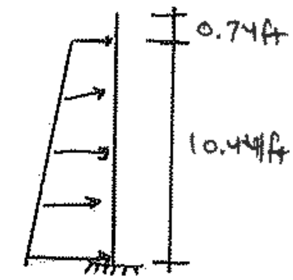
Engineers...Working Wonders With Water™

BY BS DATE 7/8/21 SUBJECT City of Wilsonville SHEET NO. ____ OF ____

CHKD. BY ____ DATE ____ Sludge Storage & Biofilters JOB NO. 11962A.00

Sludge Storage & Biofilters - Dividing Wall Check

The existing interior wall between the sludge storage and biofilters will be evaluated to resist the hydrostatic & hydrodynamic loads from basins. The forces will be evaluated per ACI 350 and using the CSZ seismic coefficient.



For the dividing wall between the sludge storage & biofilter basins, the assumption made is water is present on one side only.

The dividing wall is 12" thick with #6@12" & #7@12" alternating dowels at base of wall, and #6@12" wall reinforcing.

The dowels extend up into wall 2'-2" & 3'-5" respectively.

See attached spreadsheet for hydrostatic & hydrodynamic loads.

Checking wall strength for out-of-plane flexure and shear demands. Using ASCE 41-17 for the wall capacities, the ϕ factor shall be set $\phi = 1.0$. Forces are at BSE-2E seismic level.

Wall Check (#6@12" & #7@12" alternate [6" effective])

$M_{uy} = 13.62$ k-ft / ft $\qquad \phi M_n = 41.50$ k-ft/ft $\qquad$ Moment DCR $= \frac{13.62}{41.50} = 0.33$

$V_{uy} = 5.29$ k/ft $\qquad \phi V_n = 11.83$ k/ft $\qquad$ Shear DCR $= \frac{5.29}{11.83} = 0.45$

(ok)

Wall Check (#6@12")

$M_{ux} = 10.38$ k-ft/ft $\qquad \phi M_n = 18.85$ k-ft/ft

$V_{ux} = 3.42$ k/ft $\qquad \phi V_n = 11.83$ k/ft

Moment DCR $= \frac{10.38}{18.85} = 0.55$ (ok)

Shear DCR $= \frac{3.42}{11.83} = 0.29$ (ok)

Checking freeboard height in basin. For Risk Category III, $\delta = 0.7 \times d_{max}$.

$\delta_{transverse} = 0.7(2.10ft) = 1.47ft$

$\delta_{longitudinal} = 0.7(2.66ft) = 1.87ft$

freeboard height = 0.74 ft

0.74 ft < 1.47 ft (NG) Not enough freeboard.

0.74 ft < 1.87 ft (NG) Not enough freeboard.

carollo
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BY	BS	DATE	7/8/21	SUBJECT	City of Wilsonville	SHEET NO.	OF
CHKD. BY		DATE			Sludge Storage & Biofilters	JOB NO.	11962A.00

Checking wall strength for out-of-plane flexure and shear demands.
Forces are at CSZ seismic level.

Vertical Wall Strength

$M_{uy} = 12.49$ k·ft/ft $\qquad \phi M_n = 41.50$ k·ft/ft

$V_{uy} = 4.84$ k/ft $\qquad \phi V_n = 11.83$ k/ft

Moment DCR $= \frac{12.49 \text{ k·ft/ft}}{41.50 \text{ k·ft/ft}} = 0.30$ (ok)

Shear DCR $= \frac{4.84 \text{ k/ft}}{11.83 \text{ k/ft}} = 0.41$ (ok)

Horizontal Wall Strength

$M_{ux} = 9.56$ k·ft/ft $\qquad \phi M_n = 18.85$ k·ft/ft

$V_{ux} = 3.16$ k/ft $\qquad \phi V_n = 11.83$ k/ft

Moment DCR $= \frac{9.56 \text{ k·ft/ft}}{18.85 \text{ k·ft/ft}} = 0.51$ (ok)

Shear DCR $= \frac{3.16 \text{ k/ft}}{11.83 \text{ k/ft}} = 0.27$ (ok)

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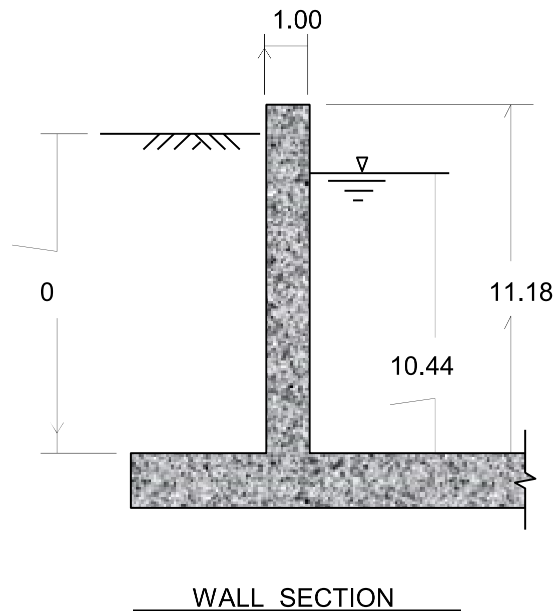
CHKD:		**DESCRIPTION:**			Sludge Storage Basins & Biofilter	**JOB NO:**	11962A.00
DESIGN TASK:	Interior Dividing Wall between Sludge Storage & Biofilter Basins (Transverse Direction) (BSE-2E)						

Static, Seismic, and Hydrodynamic Seismic Wall Pressures using ACI 350.3-06 and an IBC 2012:

wall connection fixity =	**no roof & fixed at floor**	
tank unit width perpendicular to EQ., B =	1	ft
tank inside length in direction of seismic, L =	**20**	ft
tank wall thickness, t_w =	**12**	inch
wall height, H_w =	**11.18**	ft
liquid height, H_L =	**10.44**	ft
liquid specific gravity =	**1**	
liquid density, γ_L = (sp.gr.)*γ_w =	0.0624	k/ft^3
acceleration due to gravity, g =	32.17	ft/sec^2
liquid mass density, $\rho_L = \gamma_L / g$ =	0.00194	$k\text{-}sec^2/ft^4$



Soil Data

The site has no groundwater.

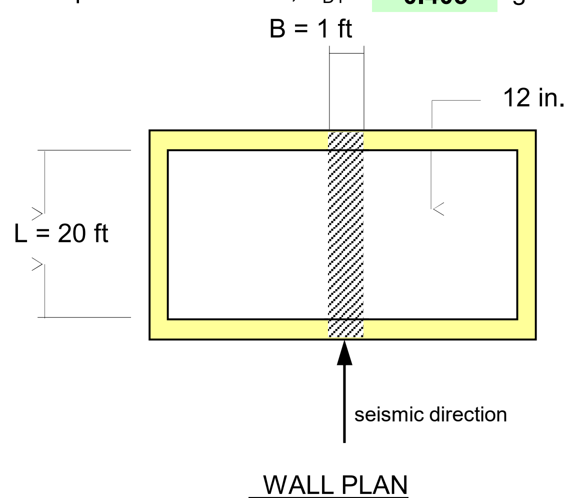
soil height above top of foundation base =	**0**	ft
groundwater ht. above foundation base =	**0**	ft
dry soil lateral pressure =	**0**	k/ft^3
saturated soil lateral pressure =	**0**	k/ft^3
dry soil unit weight =	**0**	k/ft^3
live load lateral surcharge =	**0.000**	ksf
	0	
concrete strength, f'_c =	**3**	ksi
concrete density, γ_c =	0.150	k/ft^3
concrete modulus of elasticity, E_c =	3122.0	ksi
concrete mass density, $\rho_c = \gamma_c / g$ =	0.004663	$k\text{-}sec^2/ft^4$

Seismic:

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, S_{DS} = **0.744** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, S_{D1} = **0.405** *g

Structure Risk Category =	**2**	
Importance factor, I =	1	
Response modification factor, R_{wi} =	**3**	
Response modification factor, R_{wc} =	**1**	



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

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DESIGN TASK:		Interior Dividing Wall between Sludge Storage & Biofilter Basins (Transverse Direction) (BSE-2E)					

Weights:

unit 1-ft width wall mass, W_w = (12/12) * (11.18) * 0.15 = 1.68 kip

wall c.g. relative to base, h_w = 11.18 / 2 = 5.590 ft

unit width liquid mass, W_L = (20) * (1) * (10.44) * 32.17 = 13.03 kip

Seismic:

1). structure stiffness and dynamic property:

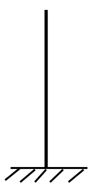
Note: per ASCE 7-10 and IBC 2012, the terms S_{ai} and S_{ac} have been appropriatetly substituted into the seismic equation of ACI 350.

Note: Wi and hi are impulsive component variables calculated on page 3.

wall mass, $m_w = H_w*(t_w / 12)*\rho_c$ = 0.05213 k-sec^2/ft^2

liquid mass, $m_i = (W_i / W_L)*(L/2)*H_L*\rho_L$ = 0.11345 k-sec^2/ft^2

centroidal distance of masses, $h = (h_w*m_w + h_i*m_i) / (m_w + m_i)$ = 4.442 ft



wall fixity condition is no roof & fixed at floor:

wall stiffness is determined using a unit mass load located at the centroidal distance h.

wall flexure stiffness, k = Ec*(tw/h)³ / 48 = 1282.34 k/ft/ft

$$\omega_i = \sqrt{\frac{k}{m_w + m_i}} = \quad (1282.34 / (0.0521 + 0.1135))\hat{}½ = 88.0026 \text{ rad/sec}$$

period of tank plus impulsive mass, $T_i = 2\pi / \omega_i$ = 2 π / 88.0026 = 0.0714 sec

(note: acceleration values to be from a maximum considered earthquake response spectra which will produce a factored load)

design factored spectral response acceleration for impulsive mass (5% damping), $S_{ai} = S_{DS}$ = 0.744 g

2). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$\lambda = \sqrt{3.16\ g\ \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)} = \quad (3.16*32.2*\tanh(3.16*(0.522)))\hat{}½ = 9.7169$$

$$\omega_c = \frac{\lambda}{\sqrt{L}} = \quad 9.7169 / (20)\hat{}½ = 2.1728 \text{ rad/sec,}$$

period of the convective mass, $T_c = 2\pi / \omega_c$ = 2 π / 2.1728 = 2.8918 sec

Long transition period (from map figure 22-15 ASCE 7), T_L = **16** sec

design spectral response acceleration for convective mass (0.5% damping), S_{ac} = 1.5 * Sd1 / Tc = 0.210 g

$$\text{effective mass coeff., } \varepsilon = 0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021\ , \text{ but } \le 1.0 = 0.7109$$

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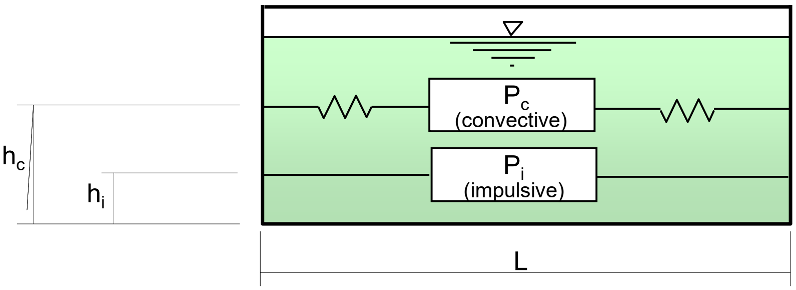

L =	20	ft
B =	1	ft
H_L =	10.44	ft
W_L =	13.03	kip
L / H_L =	1.91571	
H_L / L =	0.52200	

3). lateral fluid impulsive force: Dynamic Model

Wi = equivalent mass of the impulsive component of liquid.

$$W_i = W_L\left(\frac{\tanh\left(0.866\frac{L}{H_L}\right)}{0.866\frac{L}{H_L}}\right) =$$ 13.03*(tanh(0.866*(1.9157)) / 0.866*(1.9157)) = 7.3 kip

hi (EBP) = HL * 0.375 = 10.44 * 0.375 = 3.915 ft

hi (IBP) = HL * {{(0.866*L/HL)/(2*tanh(0.866*L/HL))} -1/8 } = 8.006 ft

impulsive force, $P_i = \left(\frac{S_{ai}\ I}{R_{wi}}\right)W_i =$ (0.744 * 1 / 3)*7.3 = 1.8 kip

4). lateral fluid convective force:

Wc = equivalent mass of the convective component of liquid.

$$W_c = W_L\left(0.264\left(\frac{L}{H_L}\right)\tanh\left(3.16\left(\frac{H_L}{L}\right)\right)\right) =$$ 13.03*(0.264*(1.9157)*tanh(3.16*(0.522))) = 6.12 kip

$$h_{c\ (EBP)} = H_L\left(1-\frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right)-1}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)}\right) =$$ 6.151 ft

$$h_{c\ (IBP)} = H_L\left(1-\frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right)-2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)}\right) =$$ 8.702 ft

convective force, $P_c = \left(\frac{S_{ac}\ I}{R_{wc}}\right)W_c =$ (0.2101 * 1 / 1)*6.12 = 1.3 kip

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5). <u>lateral inertia force of the accelerating wall:</u>

unit width wall mass, W_w = 1.68 kip

wall c.g. relative to base, h_w = 5.590 ft

wall inertia force, $P_w = \left(\dfrac{S_{ai}\ I\ \varepsilon}{R_{wi}}\right) W_w =$ (0.744*1*0.7109/3)*1.68 = 0.30 kip

6). <u>maximum wave slosh height displacement:</u>

$d_{(max)} = \left(\dfrac{L}{2}\right)\left(\dfrac{S_{ac}}{1.4}\ I\right) =$ (20 / 2) * (0.2101 / 1.0 * 1) = 2.10 ft

Wave height is greater than the freeboard of 0.74-ft. Check effects of wave spillage.

7). <u>vertical acceleration:</u>

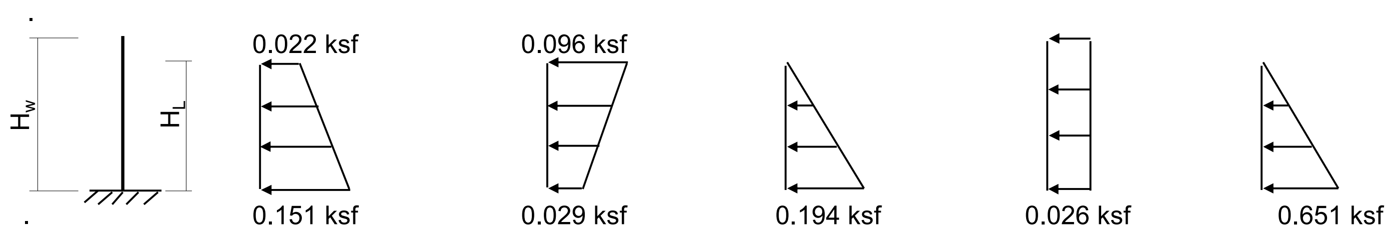
design horizontal accereration, S_{DS} = 0.744 *g

vertical spectral response acceleration (per ACI 350 para 9.4.3), $S_{av} = C_t = 0.4{*}S_{DS}$ = 0.2976 g

per ASCE 7-10 para. 15.7.7.2(b), use $I = R_i = b = 1.0$

Design vertical acceleration, $\ddot{u} = \dfrac{S_{av}\ I\ b}{R_i} =$ 0.2976*1*1/1 = 0.2976 g

8). <u>vertical force distribution on a unit width using the linear distribution of ACI 350 sec 5.3:</u>



H_w = 11.18

H_L = 10.44

<u>impulsive:</u>

$$p_{iy} = \frac{P_i\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\dfrac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_i = 1.80 kip

h_i = 3.915 ft

at $y = H_L$, p_{iy} = 0.022 ksf

at base $y = 0$, p_{iy} = 0.151 ksf

<u>convective:</u>

$$p_{cy} = \frac{P_c\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\dfrac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_c = 1.30 kip

h_c = 6.151 ft

at $y = H_L$, p_{cy} = 0.096 ksf

at base $y = 0$, p_{cy} = 0.029 ksf

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vertical acceleration:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

ü = 0.2976

at y = H_L, p_{vy} = 0.000 ksf

at base y = 0, p_{vy} = 0.194 ksf

wall inertia:

$$p_{wy} = \frac{S_{ai}\;\; I\;\; \varepsilon\, \gamma_c\;(t_w/12)}{R_{wi}} =$$

p_{wy} = 0.1763 * γ_c * (t_w/12)

at y = H_w, p_{wy} = 0.026 ksf

at base y = 0, p_{wy} = 0.026 ksf

hydrostatic:

$$q_{hy} = \gamma_L\,(H_L - y) =$$

at y = H_L, q_{hy} = 0.000 ksf

at base y = 0, q_{hy} = 0.651 ksf

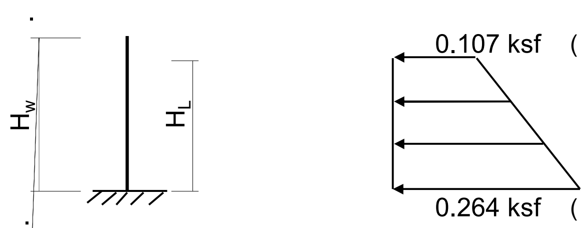
combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{(p_{iy} + p_{wy})^2 + p_{cy}^2 + p_{vy}^2} =$$

at y = H_w, p_y = 0.107 ksf

at base y = 0, p_y = 0.264 ksf

0.107 ksf (unfactored = 0.107 / 1.4 = 0.076 ksf)



0.264 ksf (unfactored = 0.264 / 1.4 = 0.189 ksf)

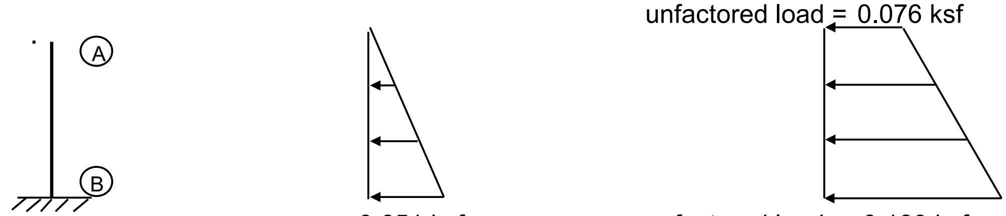
resultant dynamic pressures

9). wall design pressures for hydrostatic + dynamic:

wall height, H_w = 11.18 ft

liquid height, H_L = 10.44 ft

unfactored load = 0.076 ksf



0.651 ksf

hydrostatic

unfactored load = 0.189 ksf

resultant dynamic pressures

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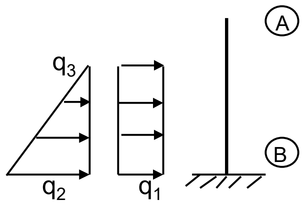
10). wall design pressures for external soil loading:

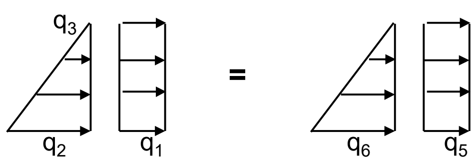
static soil:



The site has no groundwater.

wall height =	11.18	ft
soil height above top of base =	0	ft
groundwater ht. above base =	0	ft
dry soil lateral pressure =	0.000	k/ft^3
sat. soil lateral pressure =	0.000	k/ft^3
live load lateral surcharge =	0.000	ksf

equivalent static soil loadings:



LL lateral surcharge, q1 =	0.0000	ksf
unfactored soil, q2 =	0.0000	ksf
unfactored soil, q3 =	0.0000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.0000	ksf
unfactored q6 =	0.0000	ksf

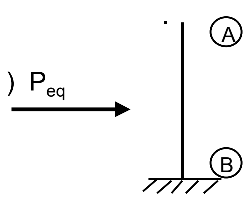
soil seismic:

resultant factored soil seismic load per foot of wall width, $P_{u\,(eq)}$ = **0** k/ft

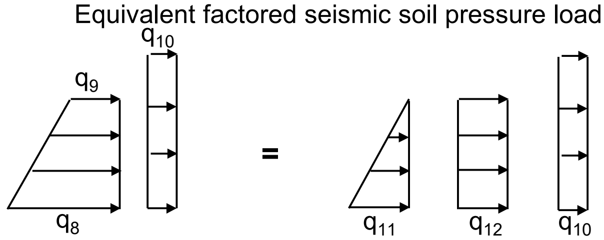
centroid location of the resultant soil seismic from the bottom of wall, h_{eq} = **0** ft

The resultant soil seismic load will be resolved into an equivalent pressure loading…

(resultant soil seismic load) P_{eq}



Equivalent factored seismic soil pressure loading & seismic wall loadings…



equivalent soil seismic, q8 =	0.0000	ksf
equivalent soil seismic, q9 =	0.0000	ksf
wall seismic (see wall page 5), q10 =	0.0264	ksf
equivalent soil seismic, q11 = q8 - q9 =	0.0000	ksf
equivalent soil seismic, q12 = q9 =	0.0000	ksf

unfactored equivalent soil seismic, q8 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q9 =0 / 1.4 =	0.0000	ksf
unfactored wall seismic , q10 = 0.0264 / 1.4 =	0.0189	ksf
unfactored equivalent soil seismic, q11 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q12 =0 / 1.4 =	0.0000	ksf

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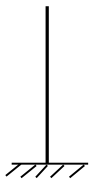
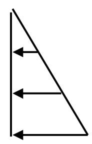
11). Summary of equivalent unfactored pressure loadings for wall load cases 1 thru 4:



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

a). load case 1: hydrostatic water

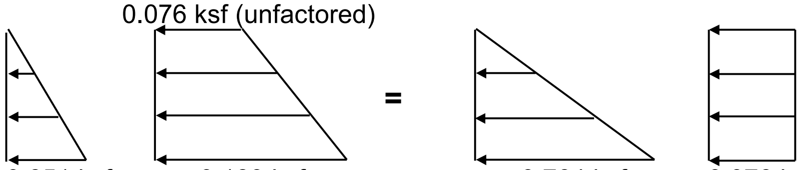
wall height = 11.18 ft
water depth = 10.44 ft



(unfactored) 0.651 ksf

b). load case 2: hydrostatic + dynamic:

0.076 ksf (unfactored)



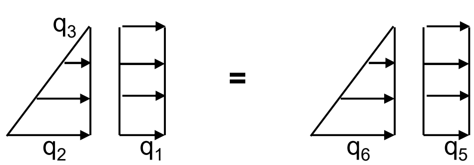
0.651 ksf	0.189 ksf	0.764 ksf	0.076 ksf
(unfactored)	(unfactored)	(unfactored)	(unfactored)
hydrostatic	resultant dynamic pressures		

wall height = 11.18 ft
water depth = 10.44 ft

c). load case 3: static soil + LL surcharge:

wall height = 11.18 ft
soil height on wall = 0 ft

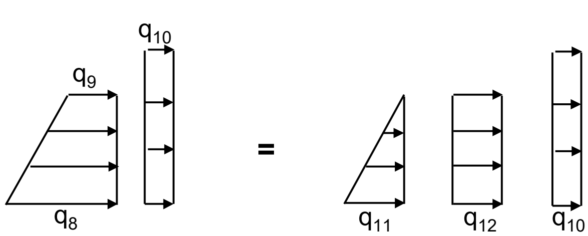
equivalent static soil & surcharge loadings...



LL lateral surcharge, q1 =	0.000	ksf
unfactored soil, q2 =	0.000	ksf
unfactored soil, q3 =	0.000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.000	ksf
unfactored q6 =	0.000	ksf

d). load case 4: soil seismic: (*note: add static soil pressure q6 & q7 to the seismic soil shown below)
equivalent seismic soil pressure loading & seismic wall loadings…

wall height = 11.18 ft
soil height on wall = 0 ft



unfactored equivalent soil seismic, q8 =	0.000	ksf
unfactored equivalent soil seismic, q9 =	0.000	ksf
unfactored equivalent soil seismic, q10 =	0.019	ksf
unfactored equivalent soil seismic, q11 =	0.000	ksf
unfactored equivalent soil seismic, q12 =	0.000	ksf

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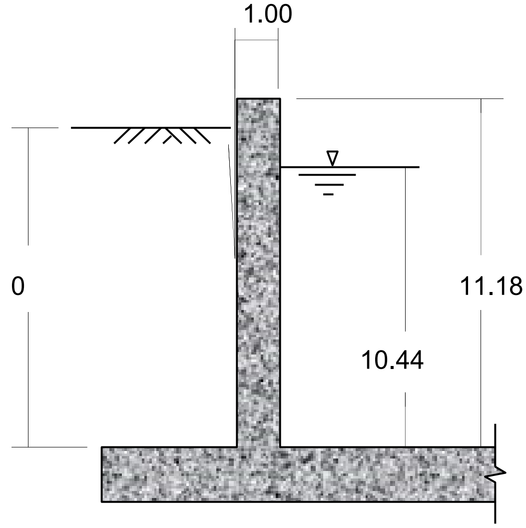
Static, Seismic, and Hydrodynamic Seismic Wall Pressures using ACI 350.3-06 and an IBC 2012:

wall connection fixity =	**no roof & fixed at floor**	
tank unit width perpendicular to EQ., B =	1	ft
tank inside length in direction of seismic, L =	**58.5**	ft
tank wall thickness, t_w =	**12**	inch
wall height, H_w =	**11.18**	ft
liquid height, H_L =	**10.44**	ft
liquid specific gravity =	**1**	
liquid density, $\gamma_L = (\text{sp.gr.})^*\gamma_w$ =	0.0624	k/ft^3
acceleration due to gravity, g =	32.17	ft/sec^2
liquid mass density, $\rho_L = \gamma_L / g$ =	0.00194	$k\text{-}sec^2/ft^4$



WALL SECTION

Soil Data

The site has no groundwater.

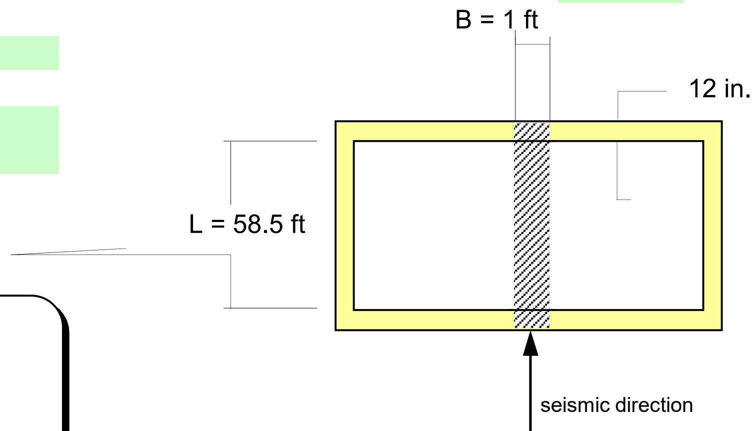
soil height above top of foundation base =	**0**	ft
groundwater ht. above foundation base =	**0**	ft
dry soil lateral pressure =	**0**	k/ft^3
saturated soil lateral pressure =	**0**	k/ft^3
dry soil unit weight =	**0**	k/ft^3
live load lateral surcharge =	**0.000**	ksf
	0	
concrete strength, f'_c =	**3**	ksi
concrete density, γ_c =	0.150	k/ft^3
concrete modulus of elasticity, E_c =	3122.0	ksi
concrete mass density, $\rho_c = \gamma_c / g$ =	0.004663	$k\text{-}sec^2/ft^4$

Seismic:

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, S_{DS} = **0.744** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, S_{D1} = **0.405** *g

Structure Risk Category =	**2**
Importance factor, I =	**1**
Response modification factor, R_{wi} =	**3**
Response modification factor, R_{wc} =	**1**



WALL PLAN

Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

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DESIGN TASK:		Interior Dividing Wall between Sludge Storage & Biofilter Basins (Longitudinal Direction) (BSE-2E)					

Weights:

unit 1-ft width wall mass, W_w = (12/12) * (11.18) * 0.15 = 1.68 kip

wall c.g. relative to base, h_w = 11.18 / 2 = 5.590 ft

unit width liquid mass, W_L = (58.5) * (1) * (10.44) * 32.17 = 38.11 kip

Seismic:

1). structure stiffness and dynamic property:

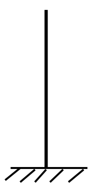
Note: per ASCE 7-10 and IBC 2012, the terms S_{ai} and S_{ac} have been appropriatetly substituted into the seismic equation of ACI 350.

Note: Wi and hi are impulsive component variables calculated on page 3.

wall mass, $m_w = H_w*(t_w / 12)*\rho_c$ = 0.05213 $k\text{-}sec^2/ft^2$

liquid mass, $m_i = (W_i / W_L)*(L/2)*H_L*\rho_L$ = 0.12201 $k\text{-}sec^2/ft^2$

centroidal distance of masses, $h = (h_w*m_w + h_i*m_i) / (m_w + m_i)$ = 4.416 ft



wall fixity condition is no roof & fixed at floor:

wall stiffness is determined using a unit mass load located at the centroidal distance h.

wall flexure stiffness, k = Ec*(tw/h)³ / 48 = 1305.12 k/ft/ft

$$\omega_i = \sqrt{\frac{k}{m_w + m_i}} =$$ (1305.12 / (0.0521 + 0.122))ˆ½ = 86.5722 rad/sec

period of tank plus impulsive mass, $T_i = 2\pi / \omega_i$ = 2 π / 86.5722 = 0.0726 sec

(note: acceleration values to be from a maximum considered earthquake response spectra which will produce a factored load)

design factored spectral response acceleration for impulsive mass (5% damping), $S_{ai} = S_{DS}$ = 0.744 g

2). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$\lambda = \sqrt{3.16\ g\ \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)} =$$ (3.16*32.2*tanh(3.16*(0.1785)))ˆ½ = 7.2067

$$\omega_c = \frac{\lambda}{\sqrt{L}} =$$ 7.2067 / (58.5)ˆ½ = 0.9422 rad/sec,

period of the convective mass, $T_c = 2\pi / \omega_c$ = 2 π / 0.9422 = 6.6684 sec

Long transition period (from map figure 22-15 ASCE 7), T_L = **16** sec

design spectral response acceleration for convective mass (0.5% damping), S_{ac} = 1.5 * Sd1 / Tc = 0.091 g

$$\text{effective mass coeff., } \varepsilon = 0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021\ ,\ \ \text{but} \le 1.0 =$$ 0.4260

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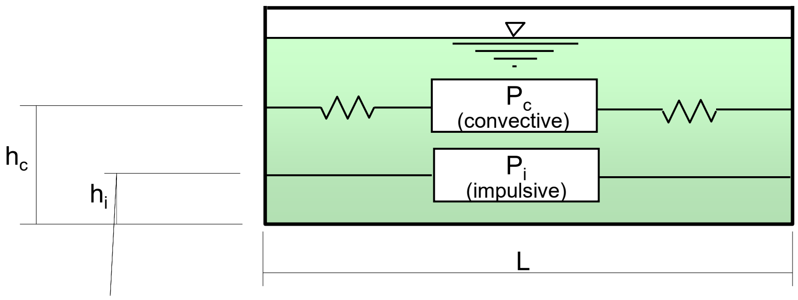
DESIGN TASK:		Interior Dividing Wall between Sludge Storage & Biofilter Basins (Longitudinal Direction) (BSE-2E)					



Dynamic Model

L =	58.5	ft
B =	1	ft
H_L =	10.44	ft
W_L =	38.11	kip
L / H_L =	5.60345	
H_L / L =	0.17846	

3). lateral fluid impulsive force:

Wi = equivalent mass of the impulsive component of liquid.

$$W_i = W_L \left(\frac{\tanh\left(0.866 \frac{L}{H_L}\right)}{0.866 \frac{L}{H_L}} \right) =$$ 38.11*(tanh(0.866*(5.6034)) / 0.866*(5.6034)) = 7.85 kip

hi (EBP) = HL * 0.375 = 10.44 * 0.375 = 3.915 ft

hi (IBP) = HL * {{(0.866*L/HL)/(2*tanh(0.866*L/HL))} -1/8 } = 24.029 ft

impulsive force, $P_i = \left(\frac{S_{ai} \ I}{R_{wi}}\right) W_i =$ (0.744 * 1 / 3)*7.85 = 1.9 kip

4). lateral fluid convective force:

Wc = equivalent mass of the convective component of liquid.

$$W_c = W_L \left(0.264 \left(\frac{L}{H_L}\right) \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)\right) =$$ 38.11*(0.264*(5.6034)*tanh(3.16*(0.1785))) = 28.8 kip

$$h_{c\ (EBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 1}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) =$$ 5.354 ft

$$h_{c\ (IBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) =$$ 36.815 ft

convective force, $P_c = \left(\frac{S_{ac} \ I}{R_{wc}}\right) W_c =$ (0.0911 * 1 / 1)*28.8 = 2.6 kip

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5). lateral inertia force of the accelerating wall:

unit width wall mass, W_w = 1.68 kip

wall c.g. relative to base, h_w = 5.590 ft

$$\text{wall inertia force, } P_w = \left(\frac{S_{ai}\ I\ \varepsilon}{R_{wi}}\right) W_w = \quad (\,0.744{*}1{*}0.426/3\,){*}1.68 = \quad 0.18 \text{ kip}$$

6). maximum wave slosh height displacement:

$$d_{(max)} = \left(\frac{L}{2}\right)\left(\frac{S_{ac}}{1.4}\ I\right) = \quad (\,58.5 / 2) * (\,0.0911 / 1.0 * 1\,) = \quad 2.66 \text{ ft}$$

Wave height is greater than the freeboard of 0.74-ft. Check effects of wave spillage.

7). vertical acceleration:

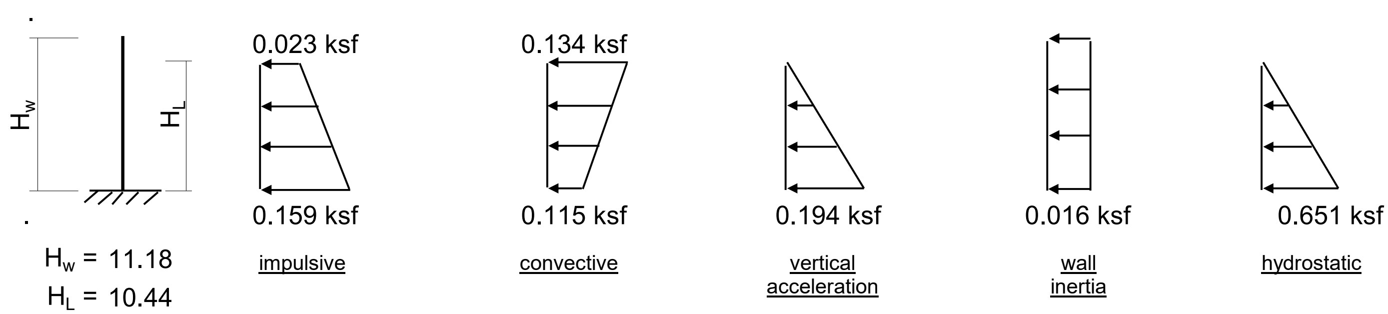
design horizontal accereration, S_{DS} = 0.744 *g

vertical spectral response acceleration (per ACI 350 para 9.4.3), $S_{av} = C_t = 0.4{*}S_{DS}$ = 0.2976 g

per ASCE 7-10 para. 15.7.7.2(b), use $I = R_i = b = 1.0$

$$\text{Design vertical acceleration, } \ddot{u} = \frac{S_{av}\ I\ b}{R_i} = \quad 0.2976{*}1{*}1/1 = \quad 0.2976 \text{ g}$$

8). vertical force distribution on a unit width using the linear distribution of ACI 350 sec 5.3:



H_w = 11.18

H_L = 10.44

impulsive:

$$p_{iy} = \frac{P_i\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_i = 1.90 kip

h_i = 3.915 ft

at $y = H_L$, p_{iy} = 0.023 ksf

at base $y = 0$, p_{iy} = 0.159 ksf

convective:

$$p_{cy} = \frac{P_c\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_c = 2.60 kip

h_c = 5.354 ft

at $y = H_L$, p_{cy} = 0.134 ksf

at base $y = 0$, p_{cy} = 0.115 ksf

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vertical acceleration:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

ü = 0.2976
at y = H_L, p_{vy} = 0.000 ksf
at base y = 0, p_{vy} = 0.194 ksf

wall inertia:

$$p_{wy} = \frac{S_{ai}\;I\;\varepsilon\,\gamma_c\,(t_w/12)}{R_{wi}} =$$

p_{wy} = 0.1056 * γ_c * $(t_w/12)$
at y = H_w, p_{wy} = 0.016 ksf
at base y = 0, p_{wy} = 0.016 ksf

hydrostatic:

$$q_{hy} = \gamma_L\,(H_L - y) =$$

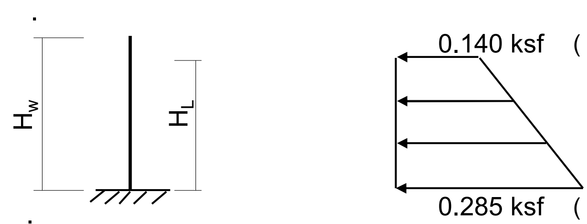
at y = H_L, q_{hy} = 0.000 ksf
at base y = 0, q_{hy} = 0.651 ksf

combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{\left(p_{iy} + p_{wy}\right)^2 + p_{cy}^2 + p_{vy}^2} =$$

at y = H_w, p_y = 0.140 ksf
at base y = 0, p_y = 0.285 ksf

0.140 ksf (unfactored = 0.14 / 1.4 = 0.1 ksf)



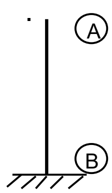
0.285 ksf (unfactored = 0.285 / 1.4 = 0.204 ksf)

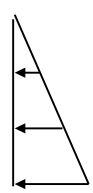
resultant dynamic pressures

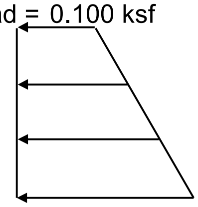
9). wall design pressures for hydrostatic + dynamic:

wall height, H_w = 11.18 ft
liquid height, H_L = 10.44 ft

unfactored load = 0.100 ksf







0.651 ksf
hydrostatic

unfactored load = 0.204 ksf
resultant dynamic pressures

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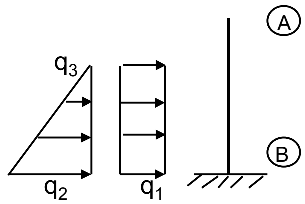
DESIGN TASK:	Interior Dividing Wall between Sludge Storage & Biofilter Basins (Longitudinal Direction) (BSE-2E)						

10). wall design pressures for external soil loading:

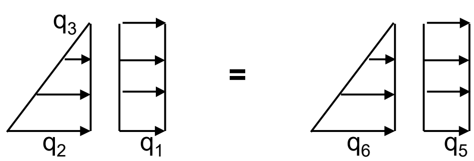
static soil:



The site has no groundwater.

wall height =	11.18	ft
soil height above top of base =	0	ft
groundwater ht. above base =	0	ft
dry soil lateral pressure =	0.000	k/ft^3
sat. soil lateral pressure =	0.000	k/ft^3
live load lateral surcharge =	0.000	ksf

equivalent static soil loadings:



LL lateral surcharge, q1 =	0.0000	ksf
unfactored soil, q2 =	0.0000	ksf
unfactored soil, q3 =	0.0000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.0000	ksf
unfactored q6 =	0.0000	ksf

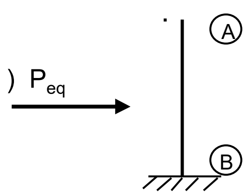
soil seismic:

resultant factored soil seismic load per foot of wall width, $P_{u\,(eq)}$ = **0** k/ft

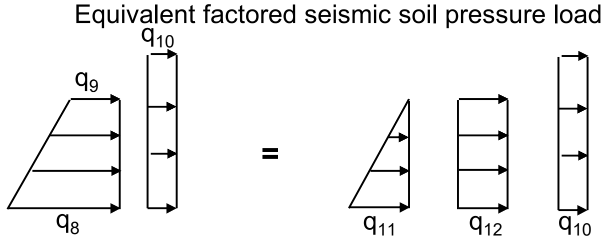
centroid location of the resultant soil seismic from the bottom of wall, h_{eq} = **0** ft

The resultant soil seismic load will be resolved into an equivalent pressure loading…

(resultant soil seismic load) P_{eq}



Equivalent factored seismic soil pressure loading & seismic wall loadings…



equivalent soil seismic, q8 =	0.0000	ksf
equivalent soil seismic, q9 =	0.0000	ksf
wall seismic (see wall page 5), q10 =	0.0158	ksf
equivalent soil seismic, q11 = q8 - q9 =	0.0000	ksf
equivalent soil seismic, q12 = q9 =	0.0000	ksf

unfactored equivalent soil seismic, q8 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q9 =0 / 1.4 =	0.0000	ksf
unfactored wall seismic , q10 = 0.0158 / 1.4 =	0.0113	ksf
unfactored equivalent soil seismic, q11 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q12 =0 / 1.4 =	0.0000	ksf

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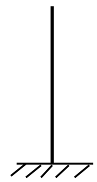
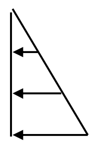
11). Summary of equivalent unfactored pressure loadings for wall load cases 1 thru 4:



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

a). load case 1: hydrostatic water

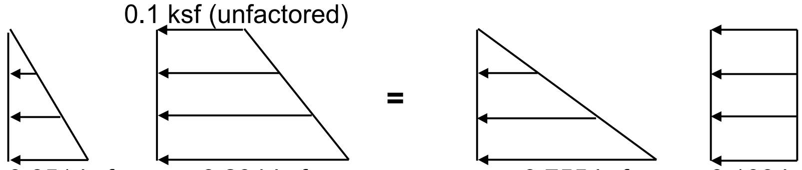
wall height = 11.18 ft
water depth = 10.44 ft



(unfactored) 0.651 ksf

b). load case 2: hydrostatic + dynamic:

0.1 ksf (unfactored)



wall height = 11.18 ft
water depth = 10.44 ft

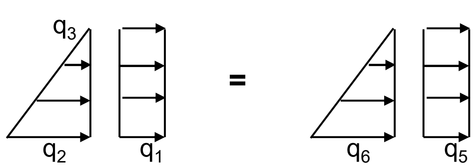
0.651 ksf (unfactored) hydrostatic	0.204 ksf (unfactored) resultant dynamic pressures	0.755 ksf (unfactored)	0.100 ksf (unfactored)

c). load case 3: static soil + LL surcharge:

wall height = 11.18 ft
soil height on wall = 0 ft

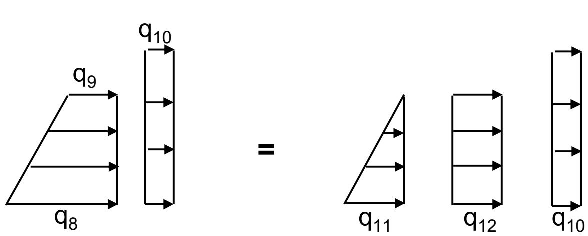
equivalent static soil & surcharge loadings...

LL lateral surcharge, q1 =	0.000	ksf
unfactored soil, q2 =	0.000	ksf
unfactored soil, q3 =	0.000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.000	ksf
unfactored q6 =	0.000	ksf



d). load case 4: soil seismic: (*note: add static soil pressure q6 & q7 to the seismic soil shown below)
equivalent seismic soil pressure loading & seismic wall loadings…

wall height = 11.18 ft
soil height on wall = 0 ft



unfactored equivalent soil seismic, q8 =	0.000	ksf
unfactored equivalent soil seismic, q9 =	0.000	ksf
unfactored equivalent soil seismic, q10 =	0.011	ksf
unfactored equivalent soil seismic, q11 =	0.000	ksf
unfactored equivalent soil seismic, q12 =	0.000	ksf

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DESIGN TASK:		Biofilter Dividing Wall Evaluationf ro Hydrostatic + Hydrodynamic Loads (BSE-2E)					

Rectangular Plate:

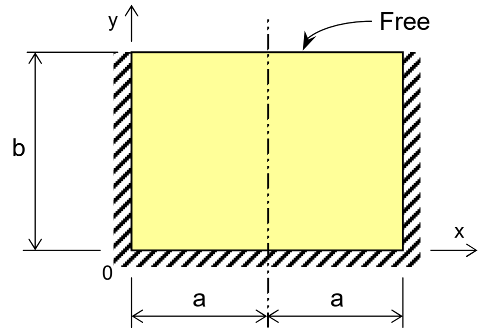
plate boundary condition case number (1, 2, 3, 4, or 5) = **1**

total plate width = 2*a = 2 * 14.63 = 29.26 ft

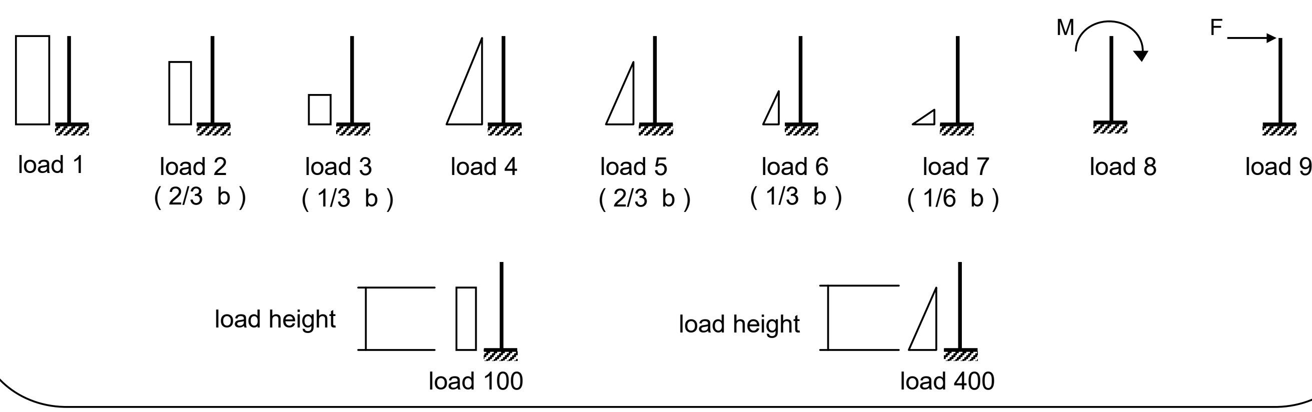
plate dimension, a = **14.63** ft

plate dimension, b = **11.18** ft

plate sides ratio, a/b = 1.3086



Available Loading Selections - (loads 1 thru 9 , 100 , or 400)



Choice of Available Loadings					
load conditions (4 max)	load type Loading Selection Number	load height, (ft) ...only for custom loads **100** or **400**	unfactored loads: q , M , or F (ksf, ft-k/ft, k/ft)	concrete load factors for moment	for shear
A	**100**	**10.440**	**0.107**	**1**	**1**
B	**400**	**10.440**	**0.157**	**1**	**1**
C	**400**	**10.440**	**0.651**	**1**	**1**
D					

Notes: 1). Load 100 = uniform load of any load height ≥ b/3; Load 400 = triangular load of any load height ≥ b/6.

2). load height must be less than or equal to "b", and uniform load height ≥ "b / 3", and triangular load height ≥ "b / 6" .

3). loads may be positive or negative.

plate thickness, h = **12** in

concrete strength, f 'c = **3** ksi

reinforcing steel strength, fy = **60** ksi

reinforcing clear cover to face of concrete = **2** in

number of curtains of reinforcing, (1 or 2) = **2**

Are bars in "x" or "y" direction closest to face of concrete ? **y**

minimum ratio of horizontal shrinkage-temperature steel = **0.00500**

minimum ratio of vertical shrinkage-temperature steel = **0.00500**

bar locations	d (in)	d' (in)
Mx bending	9"	3"
My bending	9.5"	2.5"

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M_x - Moment Summary

a = 14.63
b = 11.18
a / b = 1.3086

Loads: q , M , or F			
0.107	0.157	0.651	
Moment Coefficient Multipliers			
13.374	19.624	81.370	

Boundary Case 1

		Moment Coefficients				M_x Moments, ft-k/ft				SUMMARY: Final Moments		Reinforcing: (d = 9")	
x / a	y / b	A	B	C	D	A	B	C	D	M_x ft-k/ft	M_{ux} ft-k/ft	$A_{s(req'd)}$ in^2/ft	$A_{s(min)}$ in^2/ft
0	1	0.2643	0.0678	0.0678		3.53	1.33	5.51		10.38	10.38	0.26	0.36
0	0.8	0.2149	0.0599	0.0599		2.87	1.17	4.87		8.92	8.92	0.23	0.36
0	0.6	0.1536	0.0483	0.0483		2.05	0.95	3.93		6.94	6.94	0.17	0.36
0	0.4	0.0895	0.0331	0.0331		1.20	0.65	2.69		4.54	4.54	0.11	0.36
0	0.2	0.0288	0.0130	0.0130		0.39	0.25	1.05		1.69	1.69	0.04	0.36
0	0	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
0.2	0	0.0118	0.0061	0.0061		0.16	0.12	0.50		0.78	0.78	0.02	0.36
0.4	0	0.0288	0.0123	0.0123		0.38	0.24	1.00		1.62	1.62	0.04	0.36
0.6	0	0.0423	0.0166	0.0166		0.57	0.33	1.35		2.25	2.25	0.06	0.36
0.8	0	0.0506	0.0191	0.0191		0.68	0.38	1.56		2.61	2.61	0.06	0.36
1	0	0.0533	0.0199	0.0199		0.71	0.39	1.62		2.72	2.72	0.07	0.36
1	0.2	0.0163	0.0038	0.0038		0.22	0.08	0.31		0.60	0.60	0.01	0.36
1	0.4	-0.0189	-0.0077	-0.0077		-0.25	-0.15	-0.63		-1.03	-1.03	-0.03	-0.36
1	0.6	-0.0478	-0.0152	-0.0152		-0.64	-0.30	-1.23		-2.17	-2.17	-0.05	-0.36
1	0.8	-0.0691	-0.0199	-0.0199		-0.92	-0.39	-1.62		-2.93	-2.93	-0.07	-0.36
1	1	-0.0847	-0.0233	-0.0233		-1.13	-0.46	-1.90		-3.49	-3.49	-0.09	-0.36
0.8	1	-0.0802	-0.0220	-0.0220		-1.07	-0.43	-1.79		-3.30	-3.30	-0.08	-0.36
0.8	0.8	-0.0656	-0.0189	-0.0189		-0.88	-0.37	-1.54		-2.79	-2.79	-0.07	-0.36
0.8	0.6	-0.0460	-0.0147	-0.0147		-0.61	-0.29	-1.20		-2.10	-2.10	-0.05	-0.36
0.8	0.4	-0.0187	-0.0078	-0.0078		-0.25	-0.15	-0.63		-1.04	-1.04	-0.03	-0.36
0.8	0.2	0.0148	0.0033	0.0033		0.20	0.07	0.27		0.53	0.53	0.01	0.36

max negative moment, $M_{ux}(-)$ = -3.49 ft-k/ft
max negative steel req'd, $A_s(-)$ = -0.09 in^2/ft
minimum steel req'd = -0.36 in^2/ft
Use

max positive moment, $M_{ux}(+)$ = 10.38 ft-k/ft
max positive steel req'd, $A_s(+)$ = 0.26 in^2/ft
minimum steel req'd = 0.36 in^2/ft
Use

Engineers...Working Wonders With Water™

BY:	BS	DATE:	Aug-21	CLIENT	City of Wilsonville	SHEET:	
CHKD:		DESCRIPTION:			Sludge Storage and Biofilters Basins	JOB NO:	11962A.00
DESIGN TASK:		Biofilter Dividing Wall Evaluationf ro Hydrostatic + Hydrodynamic Loads (BSE-2E)					

M_y - Moment Summary

		Loads: q , M , or F				Boundary Case 1				SUMMARY			
a =	14.63	0.107	0.157	0.651						Final Moments		Reinforcing: (d = 9.5")	
b =	11.18	Moment Coefficient Multipliers											
a / b =	1.3086	13.374	19.624	81.370									
		Moment Coefficients				M_y Moments, ft-k/ft				M_y	M_{uy}	$A_{s(req'd)}$	$A_{s(min)}$
x / a	y / b	A	B	C	D	A	B	C	D	ft-k/ft	ft-k/ft	in^2/ft	in^2/ft
0	1	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
0	0.8	0.0430	0.0119	0.0119		0.57	0.23	0.97		1.78	1.78	0.04	0.36
0	0.6	0.0307	0.0097	0.0097		0.41	0.19	0.79		1.39	1.39	0.03	0.36
0	0.4	0.0179	0.0066	0.0066		0.24	0.13	0.54		0.91	0.91	0.02	0.36
0	0.2	0.0058	0.0026	0.0026		0.08	0.05	0.21		0.34	0.34	0.01	0.36
0	0	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
0.2	0	0.0590	0.0306	0.0306		0.79	0.60	2.49		3.88	3.88	0.09	0.36
0.4	0	0.1438	0.0615	0.0615		1.92	1.21	5.00		8.13	8.13	0.19	0.36
0.6	0	0.2117	0.0831	0.0831		2.83	1.63	6.76		11.23	11.23	0.27	0.36
0.8	0	0.2528	0.0955	0.0955		3.38	1.87	7.77		13.03	13.03	0.31	0.38
1	0	0.2664	0.0996	0.0996		3.56	1.95	8.10		13.62	13.62	0.33	0.38
1	0.2	0.1166	0.0290	0.0290		1.56	0.57	2.36		4.49	4.49	0.11	0.36
1	0.4	0.0254	-0.0040	-0.0040		0.34	-0.08	-0.33		-0.07	-0.07	0.00	-0.36
1	0.6	-0.0171	-0.0125	-0.0125		-0.23	-0.24	-1.02		-1.49	-1.49	-0.03	-0.36
1	0.8	-0.0210	-0.0084	-0.0084		-0.28	-0.16	-0.68		-1.13	-1.13	-0.03	-0.36
1	1	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
1	0.4	0.0254	-0.0040	-0.0040		0.34	-0.08	-0.33		-0.07	-0.07	0.00	-0.36
0.8	0.4	0.0228	-0.0046	-0.0046		0.30	-0.09	-0.38		-0.16	-0.16	0.00	-0.36
0.6	0.4	0.0161	-0.0059	-0.0059		0.22	-0.12	-0.48		-0.38	-0.38	-0.01	-0.36
0.4	0.4	0.0089	-0.0063	-0.0063		0.12	-0.12	-0.52		-0.52	-0.52	-0.01	-0.36
0.2	0.4	0.0070	-0.0032	-0.0032		0.09	-0.06	-0.26		-0.23	-0.23	-0.01	-0.36

max negative moment, $M_{uy}(-)$ = -1.49 ft-k/ft

max negative steel req'd, $A_s(-)$ = -0.03 in^2/ft

minimum steel req'd = -0.36 in^2/ft

Use

max positive moment, $M_{uy}(+)$ = 13.62 ft-k/ft

max positive steel req'd, $A_s(+)$ = 0.33 in^2/ft

minimum steel req'd = 0.38 in^2/ft

Use

Engineers...Working Wonders With Water™

BY:	BS	DATE:	Aug-21	CLIENT	City of Wilsonville	SHEET:	
CHKD:		DESCRIPTION:			Sludge Storage and Biofilters Basins	JOB NO:	11962A.00
DESIGN TASK:		Biofilter Dividing Wall Evaluationf ro Hydrostatic + Hydrodynamic Loads (BSE-2E)					

Shear Summary

		Loads: q , M , or F				Boundary Case 1				SUMMARY		
a = 14.63		0.107	0.157	0.651								
b = 11.18		Shear Coefficient Multipliers								Final Shears		
a / b = 1.3086		1.196	1.755	7.278								
		Shear Coefficients				Shears, k/ft				V	V_u	ϕV_c
x / a	y / b	A	B	C	D	A	B	C	D	k/ft	k/ft	k/ft
0	1	1.1848	0.2216	0.2216		1.42	0.39	1.61		3.42	3.42	9.37
0	0.8	0.8863	0.2371	0.2371		1.06	0.42	1.73		3.20	3.20	9.37
0	0.6	0.5861	0.2104	0.2104		0.70	0.37	1.53		2.60	2.60	9.37
0	0.4	0.3774	0.2142	0.2142		0.45	0.38	1.56		2.39	2.39	9.37
0	0.2	0.0094	0.1033	0.1033		0.01	0.18	0.75		0.94	0.94	9.37
0	0.00	-0.1013	-0.0175	-0.0175		-0.12	-0.03	-0.13		-0.28	-0.28	9.37
0.2	0	0.2143	0.2201	0.2201		0.26	0.39	1.60		2.24	2.24	9.37
0.4	0	0.5674	0.3580	0.3580		0.68	0.63	2.61		3.91	3.91	9.37
0.6	0	0.7792	0.4251	0.4251		0.93	0.75	3.09		4.77	4.77	9.37
0.8	0	0.8844	0.4555	0.4555		1.06	0.80	3.32		5.17	5.17	9.37
1	0	0.9158	0.4643	0.4643		1.10	0.81	3.38		5.29	5.29	9.37

Concrete strength reduction factor for shear, ϕ = 1.00

d = 9.0 in

maximum shear, V_u = 5.29 k/ft **OK**

$\phi V_c = \phi * 2 * (f'c)^{1/2} * b * d$ = (1.00*2*(3000)ˆ½ *12*9.0)/1000 = 11.83 k/ft

Reference:

"Moments and Reactions for Rectangular Plates"
Engineering Monograph No. 27
By: W. T. Moody, United States Bureau of Reclamation

Notes:

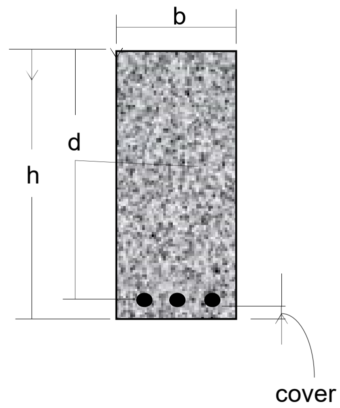
Quadratic interpolation is used for intermediate values within the Moody tables and between Moody figures.
The positive sign convention for moments M_x and M_y is tension on the loaded face of the plate.
The M_x moment is in the direction of the x-axis and the M_y moment is in the direction of the y-axis by plate sign convention.

Engineers...Working Wonders With Water™

BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**
CHKD:		**DESCRIPTION:**		Sludge Storage and Biofilter Basins		**JOB NO:** 11962A.00

DESIGN TASK: 3iofilter Dividing Wall Strength Check for Hydrostatic + Hydrodynamic Loads (Vertical Reinforcing) (BSE-2E

Concrete Shear and Moment Strength Capacity



Design for … beam, slab, wall ? **wall**

Properties and Geometry

Compression width of wall, b =	**12**	inch
Thickness of wall, h =	**12**	inch
Depth to reinforcing, d =	**9**	inch
factored design moment, M_u =	**13.62**	ft-k
factored design shear, V_u =	**5.29**	kip

f'_c (psi) =	**3000**
f_y (psi) =	**60000**
ϕ, Bending =	**1**
ϕ, Shear =	**1**
E_s (psi) =	29000000
E_c (psi) =	3122019
$n = E_s / E_c$ =	9.29
β_1 =	0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi * 2 * b * d * (f'_c)^{1/2}$ = 11.83 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #6@12" & #7@12" alternating (effective 6" spacing)**

Area steel provided, A_s = **1.04** in^2 $\rho = A_s / bd$ = 0.00963

$\rho(min)$ < As/bd < $\rho(max)$ - OK

$A_{s\ (min)}$ = 0.19 in^2 $\rho_{(min)}$ = 0.00180

$A_{s\ (max)}$ = 1.73 in^2 $\rho_{(max)}$ = 0.01604

bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

$\phi * M_n$ = 1*0.00963*60*12*9² *(1-0.588*0.00963*60/3)*(ft/12) = 41.498 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

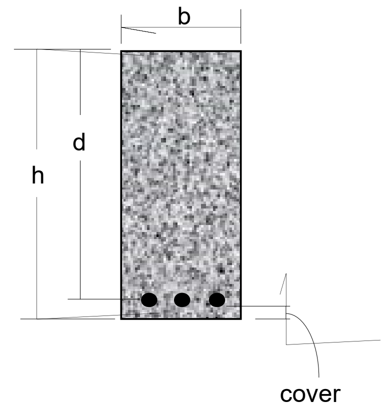
File: concrete_strength2013

Engineers...Working Wonders With Water ™

BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Sludge Storage and Biofilter Basins	**JOB NO:**	11962A.00
DESIGN TASK:	ofilter Dividing Wall Strength Check for Hydrostatic + Hydrodynamic Loads (Horizontal Reinforcing) (BSE-2I						

Concrete Shear and Moment Strength Capacity

Design for … beam, slab, wall ? **wall**



Properties and Geometry

Compression width of wall, b =	**12**	inch
Thickness of wall, h =	**12**	inch
Depth to reinforcing, d =	**9**	inch
factored design moment, M_u =	**10.38**	ft-k
factored design shear, V_u =	**3.42**	kip

f'_c (psi) =	**3000**
f_y (psi) =	**60000**
ϕ, Bending =	**1**
ϕ, Shear =	**1**
E_s (psi) =	29000000
E_c (psi) =	3122019
$n = E_s / E_c$ =	9.29
β_1 =	0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi * 2 * b * d * (f'_c)^{1/2}$ = 11.83 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #6@12"**

Area steel provided, A_s = **0.44** in^2 $\rho = A_s / bd$ = 0.00407

$\rho(min) < As/bd < \rho(max)$ - OK

$A_{s\ (min)}$ =	0.19	in^2	$\rho_{(min)}$ =	0.00180
$A_{s\ (max)}$ =	1.73	in^2	$\rho_{(max)}$ =	0.01604

bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

$\phi * M_n$ = 1*0.00407*60*12*9² *(1-0.588*0.00407*60/3)*(ft/12) = 18.851 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

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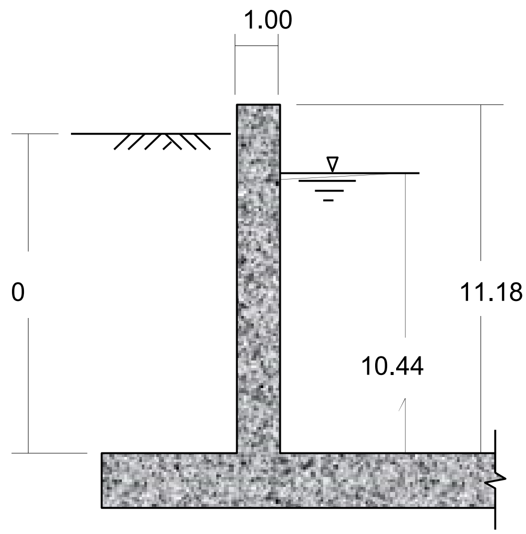
BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Sludge Storage Basins & Biofilter	**JOB NO:**	11962A.00
DESIGN TASK:		Interior Dividing Wall between Sludge Storage & Biofilter Basins (Transverse Direction) (CSZ)					

Static, Seismic, and Hydrodynamic Seismic Wall Pressures using ACI 350.3-06 and an IBC 2012:

wall connection fixity =	**no roof & fixed at floor**	
tank unit width perpendicular to EQ., B =	1	ft
tank inside length in direction of seismic, L =	**20**	ft
tank wall thickness, t_w =	**12**	inch
wall height, H_w =	**11.18**	ft
liquid height, H_L =	**10.44**	ft
liquid specific gravity =	**1**	
liquid density, $\gamma_L = (\text{sp.gr.})^*\gamma_w$ =	0.0624	k/ft^3
acceleration due to gravity, g =	32.17	ft/sec^2
liquid mass density, $\rho_L = \gamma_L / g$ =	0.00194	$k\text{-}sec^2/ft^4$



WALL SECTION

Soil Data

The site has no groundwater.

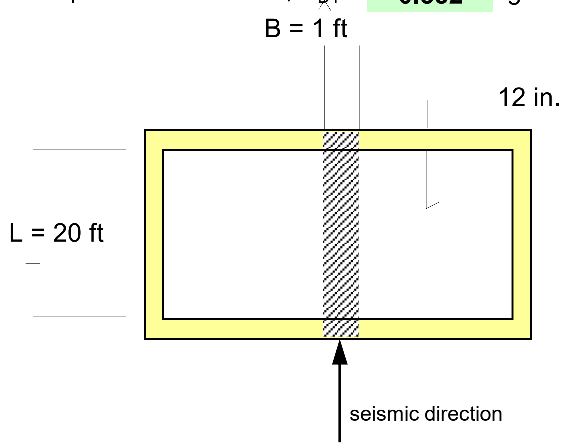
soil height above top of foundation base =	**0**	ft
groundwater ht. above foundation base =	**0**	ft
dry soil lateral pressure =	**0**	k/ft^3
saturated soil lateral pressure =	**0**	k/ft^3
dry soil unit weight =	**0**	k/ft^3
live load lateral surcharge =	**0.000**	ksf
	0	
concrete strength, f'_c =	**3**	ksi
concrete density, γ_c =	0.150	k/ft^3
concrete modulus of elasticity, E_c =	3122.0	ksi
concrete mass density, $\rho_c = \gamma_c / g$ =	0.004663	$k\text{-}sec^2/ft^4$

Seismic:

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, S_{DS} = **0.446** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, S_{D1} = **0.332** *g

Structure Risk Category =	**3**	
Importance factor, I =	**1.25**	
Response modification factor, R_{wi} =	**3**	
Response modification factor, R_{wc} =	**1**	



WALL PLAN

Load Cases:

case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

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BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**		Sludge Storage Basins & Biofilter		**JOB NO:**	11962A.00
DESIGN TASK:		Interior Dividing Wall between Sludge Storage & Biofilter Basins (Transverse Direction) (CSZ)					

Weights:

unit 1-ft width wall mass, W_w =	(12/12) * (11.18) * 0.15 =	1.68	kip
wall c.g. relative to base, h_w =	11.18 / 2 =	5.590	ft
unit width liquid mass, W_L =	(20) * (1) * (10.44) * 32.17 =	13.03	kip

Seismic:

1). structure stiffness and dynamic property:

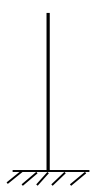
Note: per ASCE 7-10 and IBC 2012, the terms S_{ai} and S_{ac} have been appropriatetly substituted into the seismic equation of ACI 350.

Note: Wi and hi are impulsive component variables calculated on page 3.

wall mass, $m_w = H_w*(t_w / 12)*\rho_c$ = 0.05213 $k\text{-}sec^2/ft^2$

liquid mass, $m_i = (W_i / W_L)*(L/2)*H_L*\rho_L$ = 0.11345 $k\text{-}sec^2/ft^2$

centroidal distance of masses, $h = (h_w*m_w + h_i*m_i) / (m_w + m_i)$ = 4.442 ft



wall fixity condition is no roof & fixed at floor:

wall stiffness is determined using a unit mass load located at the centroidal distance h.

wall flexure stiffness, k = Ec*(tw/h)³ / 48 = 1282.34 k/ft/ft

$$\omega_i = \sqrt{\frac{k}{m_w + m_i}} = \quad (1282.34 / (0.0521 + 0.1135))\hat{}½ = 88.0026 \text{ rad/sec}$$

period of tank plus impulsive mass, $T_i = 2\pi / \omega_i$ = 2 π / 88.0026 = 0.0714 sec

(note: acceleration values to be from a maximum considered earthquake response spectra which will produce a factored load)

design factored spectral response acceleration for impulsive mass (5% damping), $S_{ai} = S_{DS}$ = 0.446 g

2). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$\lambda = \sqrt{3.16\ g\ \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)} = \quad (3.16*32.2*tanh(3.16*(0.522)))\hat{}½ = 9.7169$$

$$\omega_c = \frac{\lambda}{\sqrt{L}} = \quad 9.7169 / (20)\hat{}½ = 2.1728 \text{ rad/sec,}$$

period of the convective mass, $T_c = 2\pi / \omega_c$ = 2 π / 2.1728 = 2.8918 sec

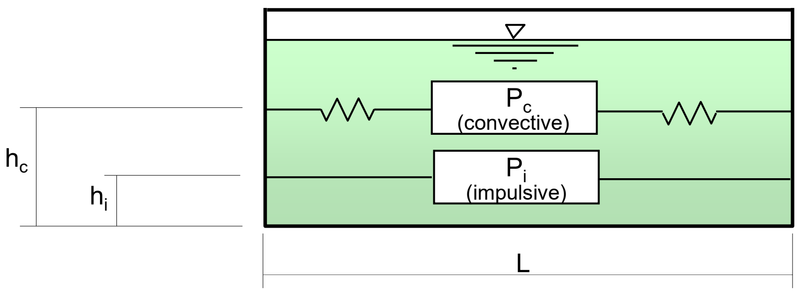
Long transition period (from map figure 22-15 ASCE 7), T_L = **16** sec

design spectral response acceleration for convective mass (0.5% damping), S_{ac} = 1.5 * Sd1 / Tc = 0.172 g

$$\text{effective mass coeff., } \varepsilon = 0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021 \text{ , but } \le 1.0 = 0.7109$$

Engineers...Working Wonders With Water™

BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Sludge Storage Basins & Biofilter	**JOB NO:**	11962A.00
DESIGN TASK:		Interior Dividing Wall between Sludge Storage & Biofilter Basins (Transverse Direction) (CSZ)					



L =	20	ft
B =	1	ft
H_L =	10.44	ft
W_L =	13.03	kip
L / H_L =	1.91571	
H_L / L =	0.52200	

3). lateral fluid impulsive force: Dynamic Model

Wi = equivalent mass of the impulsive component of liquid.

$$W_i = W_L \left(\frac{\tanh\left(0.866 \frac{L}{H_L}\right)}{0.866 \frac{L}{H_L}} \right) =$$

13.03*(tanh(0.866*(1.9157)) / 0.866*(1.9157)) = 7.3 kip

hi (EBP) = HL * 0.375 = 10.44 * 0.375 = 3.915 ft

hi (IBP) = HL * {{(0.866*L/HL)/(2*tanh(0.866*L/HL))} -1/8 } = 8.006 ft

impulsive force, $P_i = \left(\frac{S_{ai} \; I}{R_{wi}}\right) W_i =$ (0.446 * 1.25 / 3)*7.3 = 1.4 kip

4). lateral fluid convective force:

Wc = equivalent mass of the convective component of liquid.

$$W_c = W_L \left(0.264 \left(\frac{L}{H_L}\right) \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)\right) =$$

13.03*(0.264*(1.9157)*tanh(3.16*(0.522))) = 6.12 kip

$$h_{c\ (EBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 1}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) = 6.151 \text{ ft}$$

$$h_{c\ (IBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) = 8.702 \text{ ft}$$

convective force, $P_c = \left(\frac{S_{ac} \; I}{R_{wc}}\right) W_c =$ (0.1722 * 1.25 / 1)*6.12 = 1.3 kip

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CHKD:		**DESCRIPTION:**		Sludge Storage Basins & Biofilter		**JOB NO:**	11962A.00
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5). lateral inertia force of the accelerating wall:

unit width wall mass, W_w = 1.68 kip

wall c.g. relative to base, h_w = 5.590 ft

$$\text{wall inertia force, } P_w = \left(\frac{S_{ai}\ I\ \varepsilon}{R_{wi}}\right) W_w = \quad (\ 0.446{*}1.25{*}0.7109/3\ ){*}1.68 = \quad 0.22 \text{ kip}$$

6). maximum wave slosh height displacement:

$$d_{(max)} = \left(\frac{L}{2}\right)\left(\frac{S_{ac}}{1.4}\ I\right) = \quad (\ 20\ /\ 2)\ {*}\ (\ 0.1722\ /\ 1.0\ {*}\ 1.25\ ) = \quad 2.15\text{ft}$$

Wave height is greater than the freeboard of 0.74-ft. Check effects of wave spillage.

7). vertical acceleration:

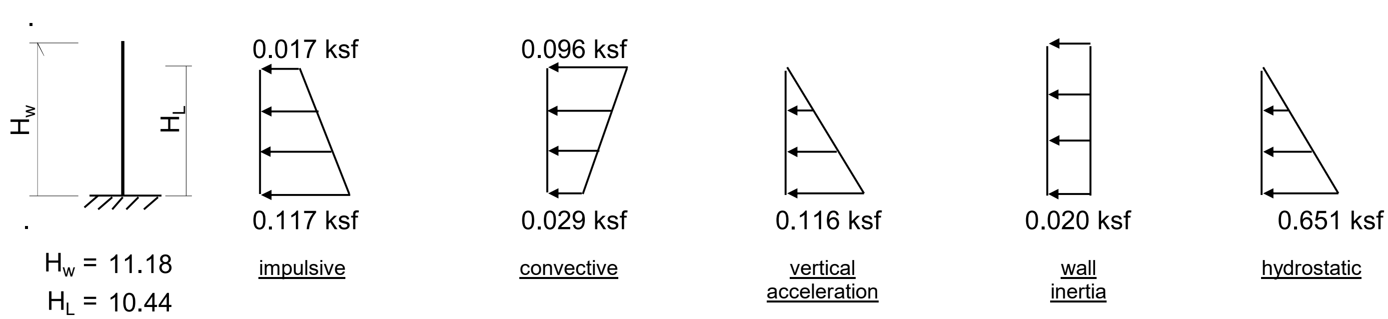
design horizontal accereration, S_{DS} = 0.446 *g

vertical spectral response acceleration (per ACI 350 para 9.4.3), $S_{av} = C_t = 0.4{*}S_{DS}$ = 0.1784 g

per ASCE 7-10 para. 15.7.7.2(b), use $I = R_i = b = 1.0$

$$\text{Design vertical acceleration, } \ddot{u} = \frac{S_{av}\ I\ b}{R_i} = \quad 0.1784{*}1{*}1/1 = \quad 0.1784 \text{ g}$$

8). vertical force distribution on a unit width using the linear distribution of ACI 350 sec 5.3:



H_w = 11.18

H_L = 10.44

impulsive:

$$p_{iy} = \frac{P_i\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_i = 1.40 kip

h_i = 3.915 ft

at $y = H_L$, p_{iy} = 0.017 ksf

at base $y = 0$, p_{iy} = 0.117 ksf

convective:

$$p_{cy} = \frac{P_c\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_c = 1.30 kip

h_c = 6.151 ft

at $y = H_L$, p_{cy} = 0.096 ksf

at base $y = 0$, p_{cy} = 0.029 ksf

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vertical acceleration:

$$p_{vy} = \ddot{u}\, \gamma_L\, (H_L - y) =$$

$\ddot{u} =$ 0.1784
at $y = H_L$, p_{vy} = 0.000 ksf
at base $y = 0$, p_{vy} = 0.116 ksf

wall inertia:

$$p_{wy} = \frac{S_{ai}\ \ I\ \ \varepsilon\ \gamma_c\ (t_w/12)}{R_{wi}} =$$

p_{wy} = 0.1321 * γ_c * $(t_w/12)$
at $y = H_w$, p_{wy} = 0.020 ksf
at base $y = 0$, p_{wy} = 0.020 ksf

hydrostatic:

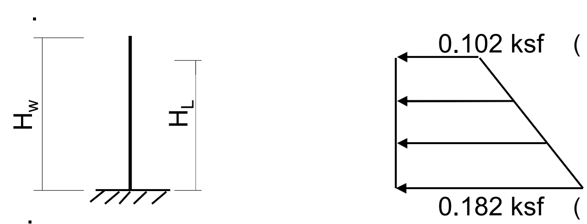
$$q_{hy} = \gamma_L\ (H_L - y) =$$

at $y = H_L$, q_{hy} = 0.000 ksf
at base $y = 0$, q_{hy} = 0.651 ksf

combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{\left(p_{iy} + p_{wy}\right)^2 + p_{cy}^2 + p_{vy}^2} =$$

at $y = H_w$, p_y = 0.102 ksf
at base $y = 0$, p_y = 0.182 ksf



0.102 ksf (unfactored = 0.102 / 1.4 = 0.073 ksf)

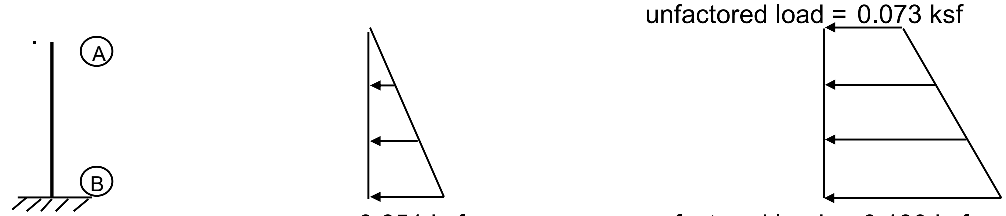
0.182 ksf (unfactored = 0.182 / 1.4 = 0.13 ksf)

resultant dynamic pressures

9). wall design pressures for hydrostatic + dynamic:

wall height, H_w = 11.18 ft
liquid height, H_L = 10.44 ft

unfactored load = 0.073 ksf



0.651 ksf
hydrostatic

unfactored load = 0.130 ksf
resultant dynamic pressures

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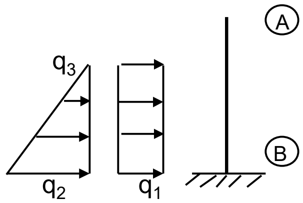
10). wall design pressures for external soil loading:

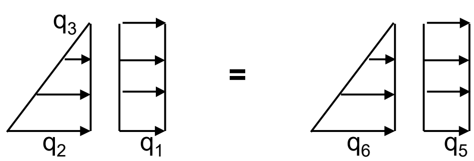
static soil:



The site has no groundwater.

wall height =	11.18	ft
soil height above top of base =	0	ft
groundwater ht. above base =	0	ft
dry soil lateral pressure =	0.000	k/ft^3
sat. soil lateral pressure =	0.000	k/ft^3
live load lateral surcharge =	0.000	ksf

equivalent static soil loadings:



LL lateral surcharge, q1 =	0.0000	ksf
unfactored soil, q2 =	0.0000	ksf
unfactored soil, q3 =	0.0000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.0000	ksf
unfactored q6 =	0.0000	ksf

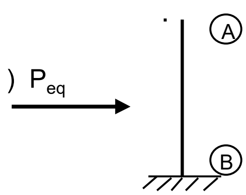
soil seismic:

resultant factored soil seismic load per foot of wall width, $P_{u\,(eq)}$ = **0** k/ft

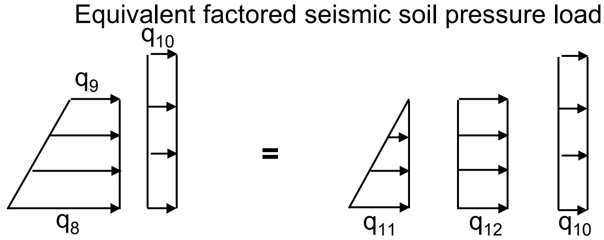
centroid location of the resultant soil seismic from the bottom of wall, h_{eq} = **0** ft

The resultant soil seismic load will be resolved into an equivalent pressure loading…

(resultant soil seismic load) P_{eq}



Equivalent factored seismic soil pressure loading & seismic wall loadings…



equivalent soil seismic, q8 =	0.0000	ksf
equivalent soil seismic, q9 =	0.0000	ksf
wall seismic (see wall page 5), q10 =	0.0198	ksf
equivalent soil seismic, q11 = q8 - q9 =	0.0000	ksf
equivalent soil seismic, q12 = q9 =	0.0000	ksf

unfactored equivalent soil seismic, q8 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q9 =0 / 1.4 =	0.0000	ksf
unfactored wall seismic , q10 = 0.0198 / 1.4 =	0.0142	ksf
unfactored equivalent soil seismic, q11 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q12 =0 / 1.4 =	0.0000	ksf

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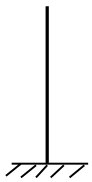
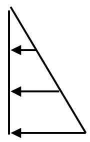
11). Summary of equivalent unfactored pressure loadings for wall load cases 1 thru 4:



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

a). load case 1: hydrostatic water

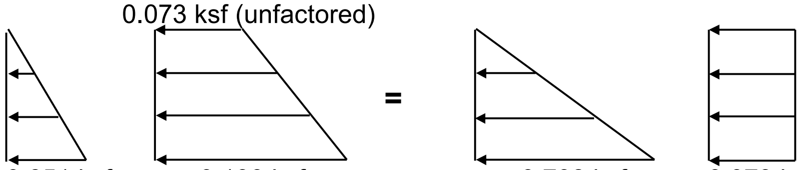
wall height = 11.18 ft
water depth = 10.44 ft



(unfactored) 0.651 ksf

b). load case 2: hydrostatic + dynamic:

0.073 ksf (unfactored)



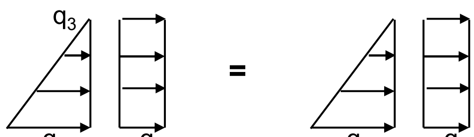
0.651 ksf	0.130 ksf	0.708 ksf	0.073 ksf
(unfactored)	(unfactored)	(unfactored)	(unfactored)
hydrostatic	resultant dynamic pressures		

wall height = 11.18 ft
water depth = 10.44 ft

c). load case 3: static soil + LL surcharge:

wall height = 11.18 ft
soil height on wall = 0 ft

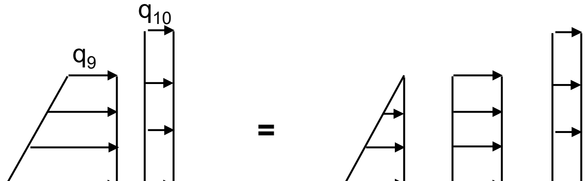
equivalent static soil & surcharge loadings...



LL lateral surcharge, q1 =	0.000	ksf
unfactored soil, q2 =	0.000	ksf
unfactored soil, q3 =	0.000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.000	ksf
unfactored q6 =	0.000	ksf

d). load case 4: soil seismic: (*note: add static soil pressure q6 & q7 to the seismic soil shown below)
equivalent seismic soil pressure loading & seismic wall loadings…

wall height = 11.18 ft
soil height on wall = 0 ft



unfactored equivalent soil seismic, q8 =	0.000	ksf
unfactored equivalent soil seismic, q9 =	0.000	ksf
unfactored equivalent soil seismic, q10 =	0.014	ksf
unfactored equivalent soil seismic, q11 =	0.000	ksf
unfactored equivalent soil seismic, q12 =	0.000	ksf

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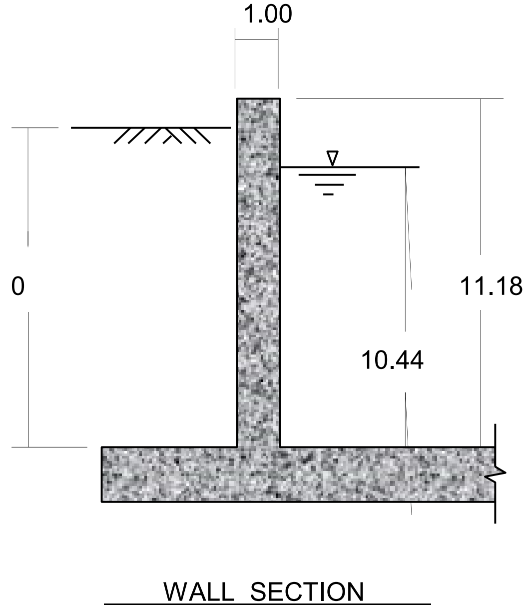
Static, Seismic, and Hydrodynamic Seismic Wall Pressures using ACI 350.3-06 and an IBC 2012:

wall connection fixity =	**no roof & fixed at floor**	
tank unit width perpendicular to EQ., B =	1	ft
tank inside length in direction of seismic, L =	**58.5**	ft
tank wall thickness, t_w =	**12**	inch
wall height, H_w =	**11.18**	ft
liquid height, H_L =	**10.44**	ft
liquid specific gravity =	**1**	
liquid density, $\gamma_L = (\text{sp.gr.})^*\gamma_w$ =	0.0624	k/ft^3
acceleration due to gravity, g =	32.17	ft/sec^2
liquid mass density, $\rho_L = \gamma_L / g$ =	0.00194	$k\text{-}sec^2/ft^4$



Soil Data

The site has no groundwater.

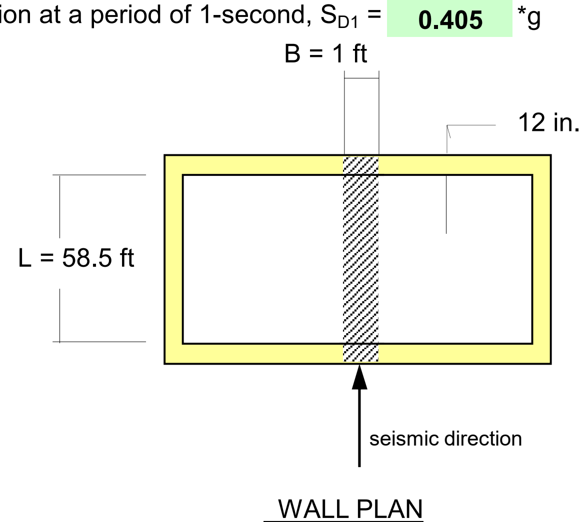
soil height above top of foundation base =	**0**	ft
groundwater ht. above foundation base =	**0**	ft
dry soil lateral pressure =	**0**	k/ft^3
saturated soil lateral pressure =	**0**	k/ft^3
dry soil unit weight =	**0**	k/ft^3
live load lateral surcharge =	**0.000**	ksf
	0	
concrete strength, f'_c =	**3**	ksi
concrete density, γ_c =	0.150	k/ft^3
concrete modulus of elasticity, E_c =	3122.0	ksi
concrete mass density, $\rho_c = \gamma_c / g$ =	0.004663	$k\text{-}sec^2/ft^4$

Seismic:

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, S_{DS} = **0.446** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, S_{D1} = **0.405** *g

Structure Risk Category =	**3**
Importance factor, I =	**1.25**
Response modification factor, R_{wi} =	**3**
Response modification factor, R_{wc} =	**1**



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

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Weights:

unit 1-ft width wall mass, W_w = (12/12) * (11.18) * 0.15 = 1.68 kip

wall c.g. relative to base, h_w = 11.18 / 2 = 5.590 ft

unit width liquid mass, W_L = (58.5) * (1) * (10.44) * 32.17 = 38.11 kip

<u>Seismic:</u>

1). <u>structure stiffness and dynamic property:</u>

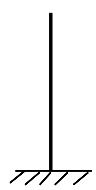
Note: per ASCE 7-10 and IBC 2012, the terms S_{ai} and S_{ac} have been appropriatetly substituted into the seismic equation of ACI 350.

Note: Wi and hi are impulsive component variables calculated on page 3.

wall mass, $m_w = H_w*(t_w / 12)*\rho_c$ = 0.05213 $k\text{-}sec^2/ft^2$

liquid mass, $m_i = (W_i / W_L)*(L/2)*H_L*\rho_L$ = 0.12201 $k\text{-}sec^2/ft^2$

centroidal distance of masses, $h = (h_w*m_w + h_i*m_i) / (m_w + m_i)$ = 4.416 ft



wall fixity condition is no roof & fixed at floor:

wall stiffness is determined using a unit mass load located at the centroidal distance h.

wall flexure stiffness, k = Ec*(tw/h)³ / 48 = 1305.12 k/ft/ft

$$\omega_i = \sqrt{\frac{k}{m_w + m_i}} =$$ (1305.12 / (0.0521 + 0.122))ˆ½ = 86.5722 rad/sec

period of tank plus impulsive mass, $T_i = 2\pi / \omega_i$ = 2 π / 86.5722 = 0.0726 sec

(note: acceleration values to be from a maximum considered earthquake response spectra which will produce a factored load)

design factored spectral response acceleration for impulsive mass (5% damping), $S_{ai} = S_{DS}$ = 0.446 g

2). <u>Dynamic properties, Spectral amplification factors, and Effective mass coefficient:</u>

$$\lambda = \sqrt{3.16\ g\ \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)} =$$ (3.16*32.2*tanh(3.16*(0.1785)))ˆ½ = 7.2067

$$\omega_c = \frac{\lambda}{\sqrt{L}} =$$ 7.2067 / (58.5)ˆ½ = 0.9422 rad/sec,

period of the convective mass, $T_c = 2\pi / \omega_c$ = 2 π / 0.9422 = 6.6684 sec

Long transition period (from map figure 22-15 ASCE 7), T_L = **16** sec

design spectral response acceleration for convective mass (0.5% damping), S_{ac} = 1.5 * Sd1 / Tc = 0.091 g

$$\text{effective mass coeff., } \varepsilon = 0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021\ ,\quad \text{but } \le\ 1.0\ =$$ 0.4260

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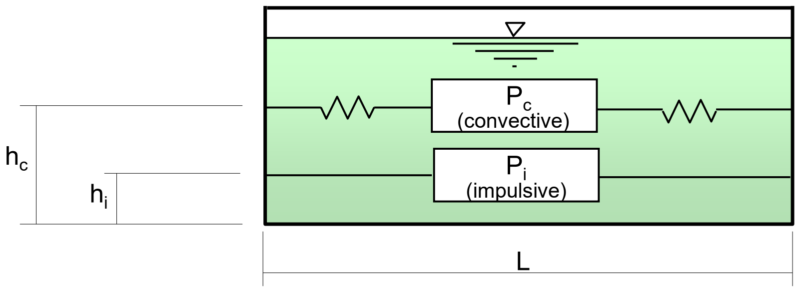

Dynamic Model

L =	58.5	ft
B =	1	ft
H_L =	10.44	ft
W_L =	38.11	kip
L / H_L =	5.60345	
H_L / L =	0.17846	

3). lateral fluid impulsive force:

Wi = equivalent mass of the impulsive component of liquid.

$$W_i = W_L \left(\frac{\tanh\left(0.866 \frac{L}{H_L}\right)}{0.866 \frac{L}{H_L}} \right) =$$ 38.11*(tanh(0.866*(5.6034)) / 0.866*(5.6034)) = 7.85 kip

hi (EBP) = HL * 0.375 = 10.44 * 0.375 = 3.915 ft

hi (IBP) = HL * {{(0.866*L/HL)/(2*tanh(0.866*L/HL))} -1/8 } = 24.029 ft

impulsive force, $P_i = \left(\frac{S_{ai}\ I}{R_{wi}} \right) W_i =$ (0.446 * 1.25 / 3)*7.85 = 1.5 kip

4). lateral fluid convective force:

Wc = equivalent mass of the convective component of liquid.

$$W_c = W_L \left(0.264 \left(\frac{L}{H_L} \right) \tanh\left(3.16 \left(\frac{H_L}{L} \right) \right) \right) =$$ 38.11*(0.264*(5.6034)*tanh(3.16*(0.1785))) = 28.8 kip

$$h_{c\ (EBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 1}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) = \quad 5.354 \text{ ft}$$

$$h_{c\ (IBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) = \quad 36.815 \text{ ft}$$

convective force, $P_c = \left(\frac{S_{ac}\ I}{R_{wc}} \right) W_c =$ (0.0911 * 1.25 / 1)*28.8 = 3.3 kip

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5). lateral inertia force of the accelerating wall:

unit width wall mass, W_w = 1.68 kip

wall c.g. relative to base, h_w = 5.590 ft

$$\text{wall inertia force, } P_w = \left(\frac{S_{ai}\ I\ \varepsilon}{R_{wi}}\right) W_w = \qquad (\ 0.446*1.25*0.426/3\ )*1.68 = \quad 0.13 \text{ kip}$$

6). maximum wave slosh height displacement:

$$d_{(max)} = \left(\frac{L}{2}\right)\left(\frac{S_{ac}}{1.4}\ I\right) = \quad (\ 58.5 / 2)\ *\ (\ 0.0911 / 1.0\ *\ 1.25\ ) = \quad 3.33 \text{ ft}$$

Wave height is greater than the freeboard of 0.74-ft. Check effects of wave spillage.

7). vertical acceleration:

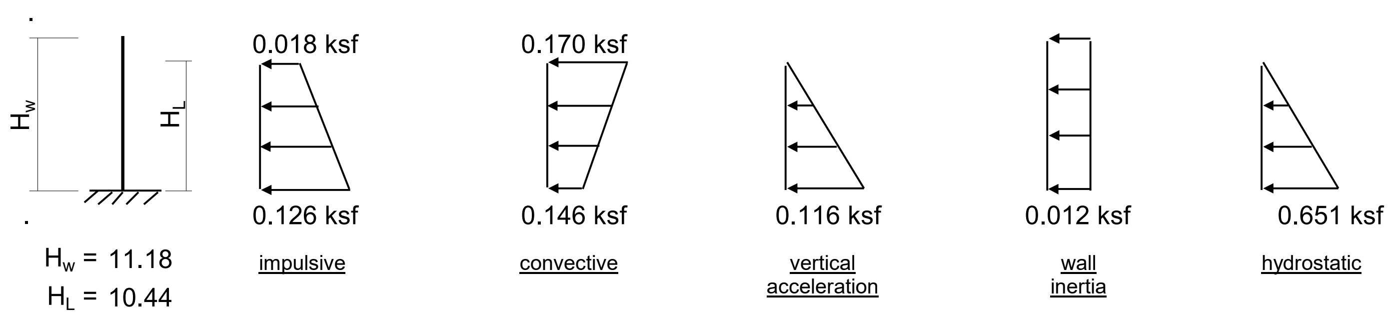
design horizontal accereration, S_{DS} = 0.446 *g

vertical spectral response acceleration (per ACI 350 para 9.4.3), $S_{av} = C_t = 0.4*S_{DS}$ = 0.1784 g

per ASCE 7-10 para. 15.7.7.2(b), use $I = R_i = b = 1.0$

$$\text{Design vertical acceleration, } \ddot{u} = \frac{S_{av}\ I\ b}{R_i} = \qquad 0.1784*1*1/1 = \quad 0.1784 \text{ g}$$

8). vertical force distribution on a unit width using the linear distribution of ACI 350 sec 5.3:



H_w = 11.18

H_L = 10.44

impulsive:

$$p_{iy} = \frac{P_i\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_i = 1.50 kip

h_i = 3.915 ft

at $y = H_L$, p_{iy} = 0.018 ksf

at base $y = 0$, p_{iy} = 0.126 ksf

convective:

$$p_{cy} = \frac{P_c\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_c = 3.30 kip

h_c = 5.354 ft

at $y = H_L$, p_{cy} = 0.170 ksf

at base $y = 0$, p_{cy} = 0.146 ksf

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vertical acceleration:

$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$

ü = 0.1784

at y = H_L, p_{vy} = 0.000 ksf

at base y = 0, p_{vy} = 0.116 ksf

wall inertia:

$$p_{wy} = \frac{S_{ai}\;I\;\varepsilon\;\gamma_c\;(t_w/12)}{R_{wi}} =$$

p_{wy} = 0.0792 * γ_c * (t_w/12)

at y = H_w, p_{wy} = 0.012 ksf

at base y = 0, p_{wy} = 0.012 ksf

hydrostatic:

$q_{hy} = \gamma_L\,(H_L - y) =$

at y = H_L, q_{hy} = 0.000 ksf

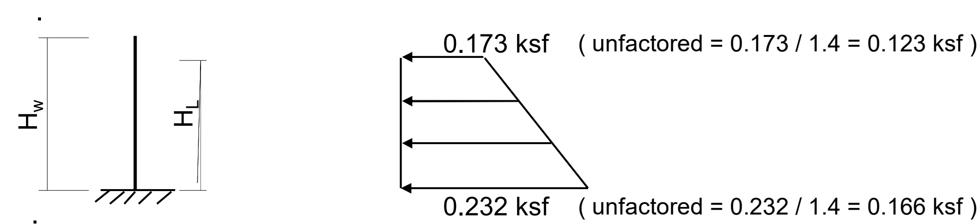
at base y = 0, q_{hy} = 0.651 ksf

combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{(p_{iy} + p_{wy})^2 + p_{cy}^2 + p_{vy}^2} =$$

at y = H_w, p_y = 0.173 ksf

at base y = 0, p_y = 0.232 ksf

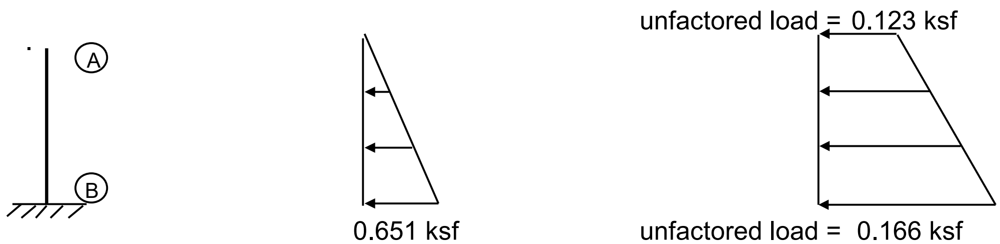


resultant dynamic pressures

9). wall design pressures for hydrostatic + dynamic:

wall height, H_w = 11.18 ft

liquid height, H_L = 10.44 ft



hydrostatic

resultant dynamic pressures

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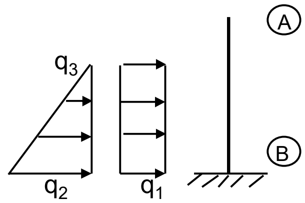
BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Sludge Storage Basins & Biofilter	**JOB NO:**	11962A.00
DESIGN TASK:	Interior Dividing Wall between Sludge Storage & Biofilter Basins (Longitudinal Direction) (CSZ)						

10). wall design pressures for external soil loading:

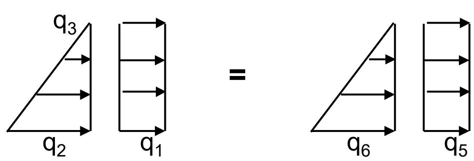
static soil:



The site has no groundwater.

wall height =	11.18	ft
soil height above top of base =	0	ft
groundwater ht. above base =	0	ft
dry soil lateral pressure =	0.000	k/ft^3
sat. soil lateral pressure =	0.000	k/ft^3
live load lateral surcharge =	0.000	ksf

equivalent static soil loadings:



LL lateral surcharge, q1 =	0.0000	ksf
unfactored soil, q2 =	0.0000	ksf
unfactored soil, q3 =	0.0000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.0000	ksf
unfactored q6 =	0.0000	ksf

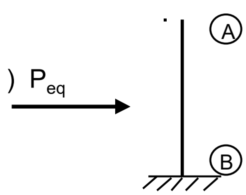
soil seismic:

resultant factored soil seismic load per foot of wall width, $P_{u\,(eq)}$ = **0** k/ft

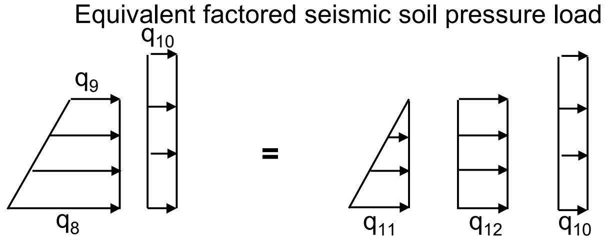
centroid location of the resultant soil seismic from the bottom of wall, h_{eq} = **0** ft

The resultant soil seismic load will be resolved into an equivalent pressure loading…

(resultant soil seismic load) P_{eq}



Equivalent factored seismic soil pressure loading & seismic wall loadings…



equivalent soil seismic, q8 =	0.0000	ksf
equivalent soil seismic, q9 =	0.0000	ksf
wall seismic (see wall page 5), q10 =	0.0119	ksf
equivalent soil seismic, q11 = q8 - q9 =	0.0000	ksf
equivalent soil seismic, q12 = q9 =	0.0000	ksf

unfactored equivalent soil seismic, q8 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q9 =0 / 1.4 =	0.0000	ksf
unfactored wall seismic , q10 = 0.0119 / 1.4 =	0.0085	ksf
unfactored equivalent soil seismic, q11 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q12 =0 / 1.4 =	0.0000	ksf

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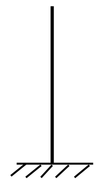
DESIGN TASK:	Interior Dividing Wall between Sludge Storage & Biofilter Basins (Longitudinal Direction) (CSZ)						

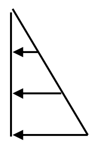
11). **Summary of equivalent unfactored pressure loadings for wall load cases 1 thru 4:**



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

a). load case 1: hydrostatic water

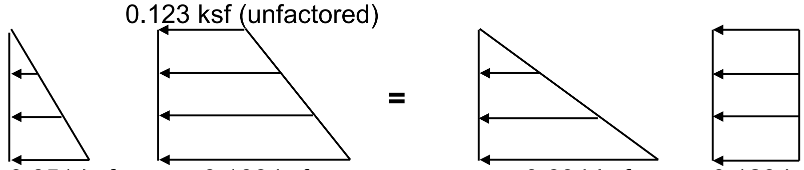
wall height = 11.18 ft
water depth = 10.44 ft



(unfactored) 0.651 ksf

b). load case 2: hydrostatic + dynamic:

0.123 ksf (unfactored)



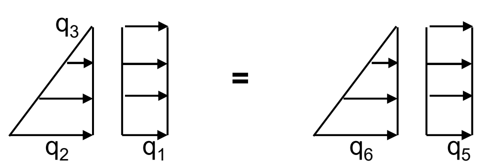
0.651 ksf (unfactored) hydrostatic — 0.166 ksf (unfactored) resultant dynamic pressures — = — 0.694 ksf (unfactored) — 0.123 ksf (unfactored)

wall height = 11.18 ft
water depth = 10.44 ft

c). load case 3: static soil + LL surcharge:

wall height = 11.18 ft
soil height on wall = 0 ft

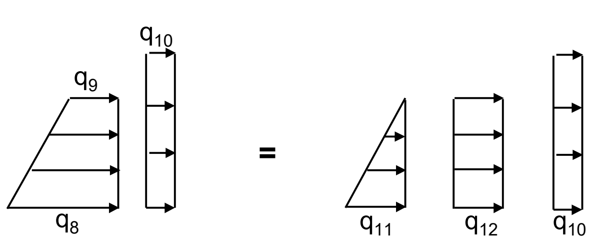
equivalent static soil & surcharge loadings...



LL lateral surcharge, q1 =	0.000	ksf
unfactored soil, q2 =	0.000	ksf
unfactored soil, q3 =	0.000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.000	ksf
unfactored q6 =	0.000	ksf

d). load case 4: soil seismic: (*note: add static soil pressure q6 & q7 to the seismic soil shown below)
equivalent seismic soil pressure loading & seismic wall loadings…

wall height = 11.18 ft
soil height on wall = 0 ft



unfactored equivalent soil seismic, q8 =	0.000	ksf
unfactored equivalent soil seismic, q9 =	0.000	ksf
unfactored equivalent soil seismic, q10 =	0.008	ksf
unfactored equivalent soil seismic, q11 =	0.000	ksf
unfactored equivalent soil seismic, q12 =	0.000	ksf

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BY:	BS	**DATE:**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Sludge Storage and Biofilters Basins	**JOB NO:**	11962A.00
DESIGN TASK:		Biofilter Dividing Wall Evaluation for Hydrostatic + Hydrodynamic Loads (CSZ)					

Rectangular Plate:

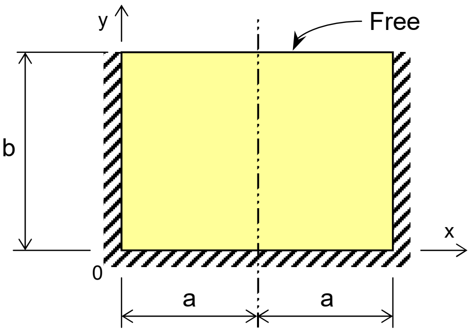
plate boundary condition case number (1, 2, 3, 4, or 5) = **1**

total plate width = 2*a = 2 * 14.63 = 29.26 ft

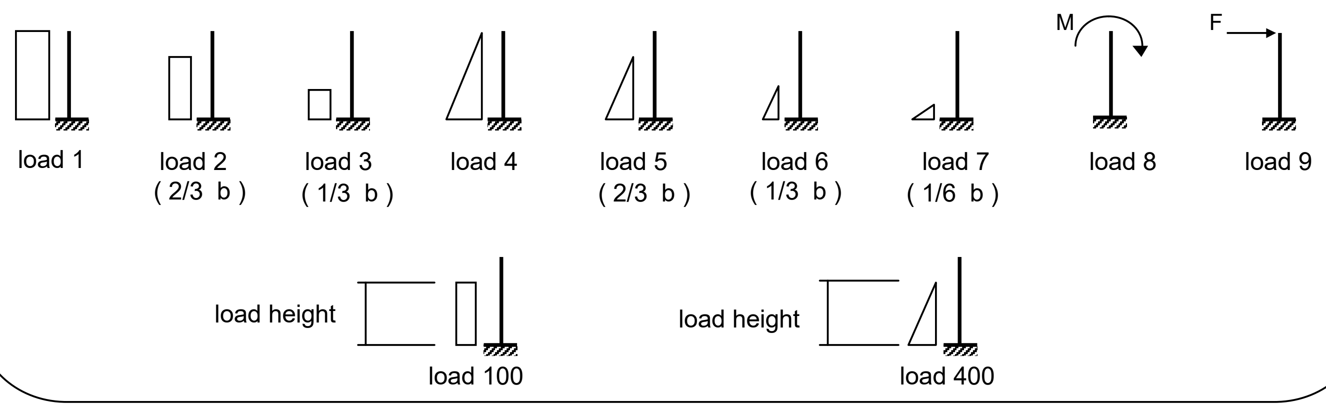
plate dimension, a = **14.63** ft

plate dimension, b = **11.18** ft

plate sides ratio, a/b = 1.3086



Available Loading Selections - (loads 1 thru 9 , 100 , or 400)



Choice of Available Loadings					
load conditions (4 max)	load type Loading Selection Number	load height, (ft) ...only for custom loads **100** or **400**	unfactored loads: q , M , or F (ksf, ft-k/ft, k/ft)	concrete load factors for moment	for shear
A	**100**	**10.440**	**0.102**	**1**	**1**
B	**400**	**10.440**	**0.080**	**1**	**1**
C	**400**	**10.440**	**0.651**	**1**	**1**
D					

Notes: 1). Load 100 = uniform load of any load height ≥ b/3; Load 400 = triangular load of any load height ≥ b/6.

2). load height must be less than or equal to "b", and uniform load height ≥ "b / 3", and triangular load height ≥ "b / 6" .

3). loads may be positive or negative.

plate thickness, h = **12** in

concrete strength, f 'c = **3** ksi

reinforcing steel strength, fy = **60** ksi

reinforcing clear cover to face of concrete = **2** in

number of curtains of reinforcing, (1 or 2) = **2**

Are bars in "x" or "y" direction closest to face of concrete ? **y**

minimum ratio of horizontal shrinkage-temperature steel = **0.00500**

minimum ratio of vertical shrinkage-temperature steel = **0.00500**

bar locations	d (in)	d' (in)
Mx bending	9"	3"
My bending	9.5"	2.5"

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CHKD:		DESCRIPTION:			Sludge Storage and Biofilters Basins	JOB NO:	11962A.00
DESIGN TASK:					Biofilter Dividing Wall Evaluation for Hydrostatic + Hydrodynamic Loads (CSZ)		

M_x - Moment Summary

a = 14.63		Loads: q , M , or F				Boundary Case 1				SUMMARY			
b = 11.18		0.102	0.080	0.651						Final Moments		Reinforcing: (d = 9")	
a / b = 1.3086		Moment Coefficient Multipliers											
		12.749	9.999	81.370									
		Moment Coefficients				M_x Moments, ft-k/ft				M_x	M_{ux}	$A_{s(req'd)}$	$A_{s(min)}$
x / a	y / b	A	B	C	D	A	B	C	D	ft-k/ft	ft-k/ft	in^2/ft	in^2/ft
0	1	0.2643	0.0678	0.0678		3.37	0.68	5.51		9.56	9.56	0.24	0.36
0	0.8	0.2149	0.0599	0.0599		2.74	0.60	4.87		8.21	8.21	0.21	0.36
0	0.6	0.1536	0.0483	0.0483		1.96	0.48	3.93		6.37	6.37	0.16	0.36
0	0.4	0.0895	0.0331	0.0331		1.14	0.33	2.69		4.16	4.16	0.10	0.36
0	0.2	0.0288	0.0130	0.0130		0.37	0.13	1.05		1.55	1.55	0.04	0.36
0	0	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
0.2	0	0.0118	0.0061	0.0061		0.15	0.06	0.50		0.71	0.71	0.02	0.36
0.4	0	0.0288	0.0123	0.0123		0.37	0.12	1.00		1.49	1.49	0.04	0.36
0.6	0	0.0423	0.0166	0.0166		0.54	0.17	1.35		2.06	2.06	0.05	0.36
0.8	0	0.0506	0.0191	0.0191		0.64	0.19	1.56		2.39	2.39	0.06	0.36
1	0	0.0533	0.0199	0.0199		0.68	0.20	1.62		2.50	2.50	0.06	0.36
1	0.2	0.0163	0.0038	0.0038		0.21	0.04	0.31		0.56	0.56	0.01	0.36
1	0.4	-0.0189	-0.0077	-0.0077		-0.24	-0.08	-0.63		-0.94	-0.94	-0.02	-0.36
1	0.6	-0.0478	-0.0152	-0.0152		-0.61	-0.15	-1.23		-1.99	-1.99	-0.05	-0.36
1	0.8	-0.0691	-0.0199	-0.0199		-0.88	-0.20	-1.62		-2.70	-2.70	-0.07	-0.36
1	1	-0.0847	-0.0233	-0.0233		-1.08	-0.23	-1.90		-3.21	-3.21	-0.08	-0.36
0.8	1	-0.0802	-0.0220	-0.0220		-1.02	-0.22	-1.79		-3.03	-3.03	-0.08	-0.36
0.8	0.8	-0.0656	-0.0189	-0.0189		-0.84	-0.19	-1.54		-2.56	-2.56	-0.06	-0.36
0.8	0.6	-0.0460	-0.0147	-0.0147		-0.59	-0.15	-1.20		-1.93	-1.93	-0.05	-0.36
0.8	0.4	-0.0187	-0.0078	-0.0078		-0.24	-0.08	-0.63		-0.95	-0.95	-0.02	-0.36
0.8	0.2	0.0148	0.0033	0.0033		0.19	0.03	0.27		0.49	0.49	0.01	0.36

max negative moment, $M_{ux}(-)$ = -3.21 ft-k/ft

max negative steel req'd, $A_s(-)$ = -0.08 in^2/ft

minimum steel req'd = -0.36 in^2/ft

Use

max positive moment, $M_{ux}(+)$ = 9.56 ft-k/ft

max positive steel req'd, $A_s(+)$ = 0.24 in^2/ft

minimum steel req'd = 0.36 in^2/ft

Use

Engineers...Working Wonders With Water™

BY:	BS	DATE:	Aug-21	CLIENT	City of Wilsonville	SHEET:	
CHKD:		DESCRIPTION:			Sludge Storage and Biofilters Basins	JOB NO:	11962A.00
DESIGN TASK:		Biofilter Dividing Wall Evaluation for Hydrostatic + Hydrodynamic Loads (CSZ)					

M_y - Moment Summary

a = 14.63; b = 11.18; a / b = 1.3086

Loads: q, M, or F: 0.102 | 0.080 | 0.651

Moment Coefficient Multipliers: 12.749 | 9.999 | 81.370

Boundary Case 1

SUMMARY — Reinforcing: (d = 9.5")

x / a	y / b	Moment Coefficients A	B	C	D	M_y Moments, ft-k/ft A	B	C	D	Final Moments M_y ft-k/ft	M_{uy} ft-k/ft	$A_{s(req'd)}$ in^2/ft	$A_{s(min)}$ in^2/ft
0	1	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
0	0.8	0.0430	0.0119	0.0119		0.55	0.12	0.97		1.64	1.64	0.04	0.36
0	0.6	0.0307	0.0097	0.0097		0.39	0.10	0.79		1.28	1.28	0.03	0.36
0	0.4	0.0179	0.0066	0.0066		0.23	0.07	0.54		0.83	0.83	0.02	0.36
0	0.2	0.0058	0.0026	0.0026		0.07	0.03	0.21		0.31	0.31	0.01	0.36
0	0	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
0.2	0	0.0590	0.0306	0.0306		0.75	0.31	2.49		3.55	3.55	0.08	0.36
0.4	0	0.1438	0.0615	0.0615		1.83	0.61	5.00		7.45	7.45	0.18	0.36
0.6	0	0.2117	0.0831	0.0831		2.70	0.83	6.76		10.29	10.29	0.25	0.36
0.8	0	0.2528	0.0955	0.0955		3.22	0.96	7.77		11.95	11.95	0.29	0.38
1	0	0.2664	0.0996	0.0996		3.40	1.00	8.10		12.49	12.49	0.30	0.38
1	0.2	0.1166	0.0290	0.0290		1.49	0.29	2.36		4.14	4.14	0.10	0.36
1	0.4	0.0254	-0.0040	-0.0040		0.32	-0.04	-0.33		-0.05	-0.05	0.00	-0.36
1	0.6	-0.0171	-0.0125	-0.0125		-0.22	-0.12	-1.02		-1.36	-1.36	-0.03	-0.36
1	0.8	-0.0210	-0.0084	-0.0084		-0.27	-0.08	-0.68		-1.03	-1.03	-0.02	-0.36
1	1	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
1	0.4	0.0254	-0.0040	-0.0040		0.32	-0.04	-0.33		-0.05	-0.05	0.00	-0.36
0.8	0.4	0.0228	-0.0046	-0.0046		0.29	-0.05	-0.38		-0.13	-0.13	0.00	-0.36
0.6	0.4	0.0161	-0.0059	-0.0059		0.21	-0.06	-0.48		-0.33	-0.33	-0.01	-0.36
0.4	0.4	0.0089	-0.0063	-0.0063		0.11	-0.06	-0.52		-0.47	-0.47	-0.01	-0.36
0.2	0.4	0.0070	-0.0032	-0.0032		0.09	-0.03	-0.26		-0.20	-0.20	0.00	-0.36

max negative moment, $M_{uy}(-)$ = -1.36 ft-k/ft
max negative steel req'd, $A_s(-)$ = -0.03 in^2/ft
minimum steel req'd = -0.36 in^2/ft
Use

max positive moment, $M_{uy}(+)$ = 12.49 ft-k/ft
max positive steel req'd, $A_s(+)$ = 0.30 in^2/ft
minimum steel req'd = 0.38 in^2/ft
Use

Engineers...Working Wonders With Water ™

BY:	BS	DATE:	Aug-21	CLIENT	City of Wilsonville	SHEET:	
CHKD:		DESCRIPTION:			Sludge Storage and Biofilters Basins	JOB NO:	11962A.00
DESIGN TASK:					Biofilter Dividing Wall Evaluation for Hydrostatic + Hydrodynamic Loads (CSZ)		

Shear Summary

a = 14.63
b = 11.18
a / b = 1.3086

	Loads: q, M, or F			
Loads	0.102	0.080	0.651	
Shear Coefficient Multipliers	1.140	0.894	7.278	

Boundary Case 1

		Shear Coefficients				Shears, k/ft				SUMMARY: Final Shears		
x / a	y / b	A	B	C	D	A	B	C	D	V k/ft	V_u k/ft	ϕV_c k/ft
0	1	1.1848	0.2216	0.2216		1.35	0.20	1.61		3.16	3.16	9.37
0	0.8	0.8863	0.2371	0.2371		1.01	0.21	1.73		2.95	2.95	9.37
0	0.6	0.5861	0.2104	0.2104		0.67	0.19	1.53		2.39	2.39	9.37
0	0.4	0.3774	0.2142	0.2142		0.43	0.19	1.56		2.18	2.18	9.37
0	0.2	0.0094	0.1033	0.1033		0.01	0.09	0.75		0.85	0.85	9.37
0	0.00	-0.1013	-0.0175	-0.0175		-0.12	-0.02	-0.13		-0.26	-0.26	9.37
0.2	0	0.2143	0.2201	0.2201		0.24	0.20	1.60		2.04	2.04	9.37
0.4	0	0.5674	0.3580	0.3580		0.65	0.32	2.61		3.57	3.57	9.37
0.6	0	0.7792	0.4251	0.4251		0.89	0.38	3.09		4.36	4.36	9.37
0.8	0	0.8844	0.4555	0.4555		1.01	0.41	3.32		4.73	4.73	9.37
1	0	0.9158	0.4643	0.4643		1.04	0.42	3.38		4.84	4.84	9.37

Concrete strength reduction factor for shear, ϕ = 1.00

= 9.0 in

maximum shear, V_u = 4.84 k/ft **OK**

$\phi V_c = \phi * 2 * (f'c)^{1/2} * b * d$ = (1.00*2*(3000)^½ *12*9.0)/1000 = 11.83 k/ft

Reference:

"Moments and Reactions for Rectangular Plates"
Engineering Monograph No. 27
By: W. T. Moody, United States Bureau of Reclamation

Notes:

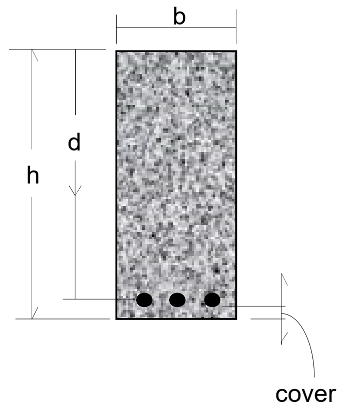
Quadratic interpolation is used for intermediate values within the Moody tables and between Moody figures.
The positive sign convention for moments M_x and M_y is tension on the loaded face of the plate.
The M_x moment is in the direction of the x-axis and the M_y moment is in the direction of the y-axis by plate sign convention.

Engineers...Working Wonders With Water™

BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**		Sludge Storage and Biofilter Basins		**JOB NO:**	11962A.00
DESIGN TASK:	Biofilter Dividing Wall Strength Check for Hydrostatic + Hydrodynamic Loads (Vertical Reinforcing) (CSZ)						

Concrete Shear and Moment Strength Capacity

Design for … beam, slab, wall ? **wall**



Properties and Geometry

Compression width of wall, b = **12** inch
Thickness of wall, h = **12** inch
Depth to reinforcing, d = **9** inch
factored design moment, M_u = **12.49** ft-k
factored design shear, V_u = **4.84** kip

f'_c (psi) = **3000**
f_y (psi) = **60000**
ϕ, Bending = **1**
ϕ, Shear = **1**
E_s (psi) = 29000000
E_c (psi) = 3122019
$n = E_s / E_c$ = 9.29
β_1 = 0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi * 2 * b * d * (f'_c)^{1/2}$ = 11.83 kip ≥ Vu
stirrup spacing, s = **0** in
stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #6@12" & #7@12" alternating (effective 6" spacing)**
Area steel provided, A_s = **1.04** in^2 $\rho = A_s / bd$ = 0.00963
$\rho(min) < As/bd < \rho(max)$ - OK

$A_{s\,(min)}$ = 0.19 in^2 $\rho_{(min)}$ = 0.00180
$A_{s\,(max)}$ = 1.73 in^2 $\rho_{(max)}$ = 0.01604

bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

$\phi * M_n$ = 1*0.00963*60*12*9² *(1-0.588*0.00963*60/3)*(ft/12) = 41.498 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

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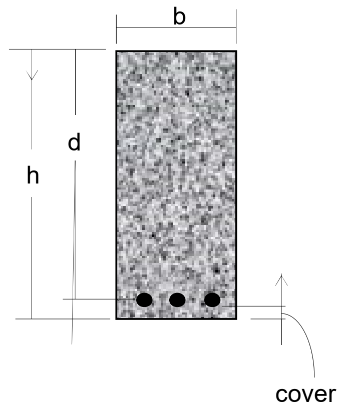
BY: BS	**DATE:** Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:	**DESCRIPTION:**		Sludge Storage and Biofilter Basins	**JOB NO:**	11962A.00

DESIGN TASK: Biofilter Dividing Wall Strength Check for Hydrostatic + Hydrodynamic Loads (Horizontal Reinforcing) (CSZ)

Concrete Shear and Moment Strength Capacity

Design for ... beam, slab, wall ? **wall**



Properties and Geometry

Compression width of wall, b = **12** inch

Thickness of wall, h = **12** inch

Depth to reinforcing, d = **9** inch

factored design moment, M_u = **9.56** ft-k

factored design shear, V_u = **3.16** kip

f'_c (psi) = **3000**

f_y (psi) = **60000**

ϕ, Bending = **1**

ϕ, Shear = **1**

E_s (psi) = 29000000

E_c (psi) = 3122019

$n = E_s / E_c$ = 9.29

β_1 = 0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi * 2 * b * d * (f'_c)^{1/2}$ = 11.83 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #6@12"**

Area steel provided, A_s = **0.44** in^2

$\rho = A_s / bd$ = 0.00407

ρ(min) < As/bd < ρ(max) - OK

$A_{s\,(min)}$ = 0.19 in^2 $\rho_{(min)}$ = 0.00180

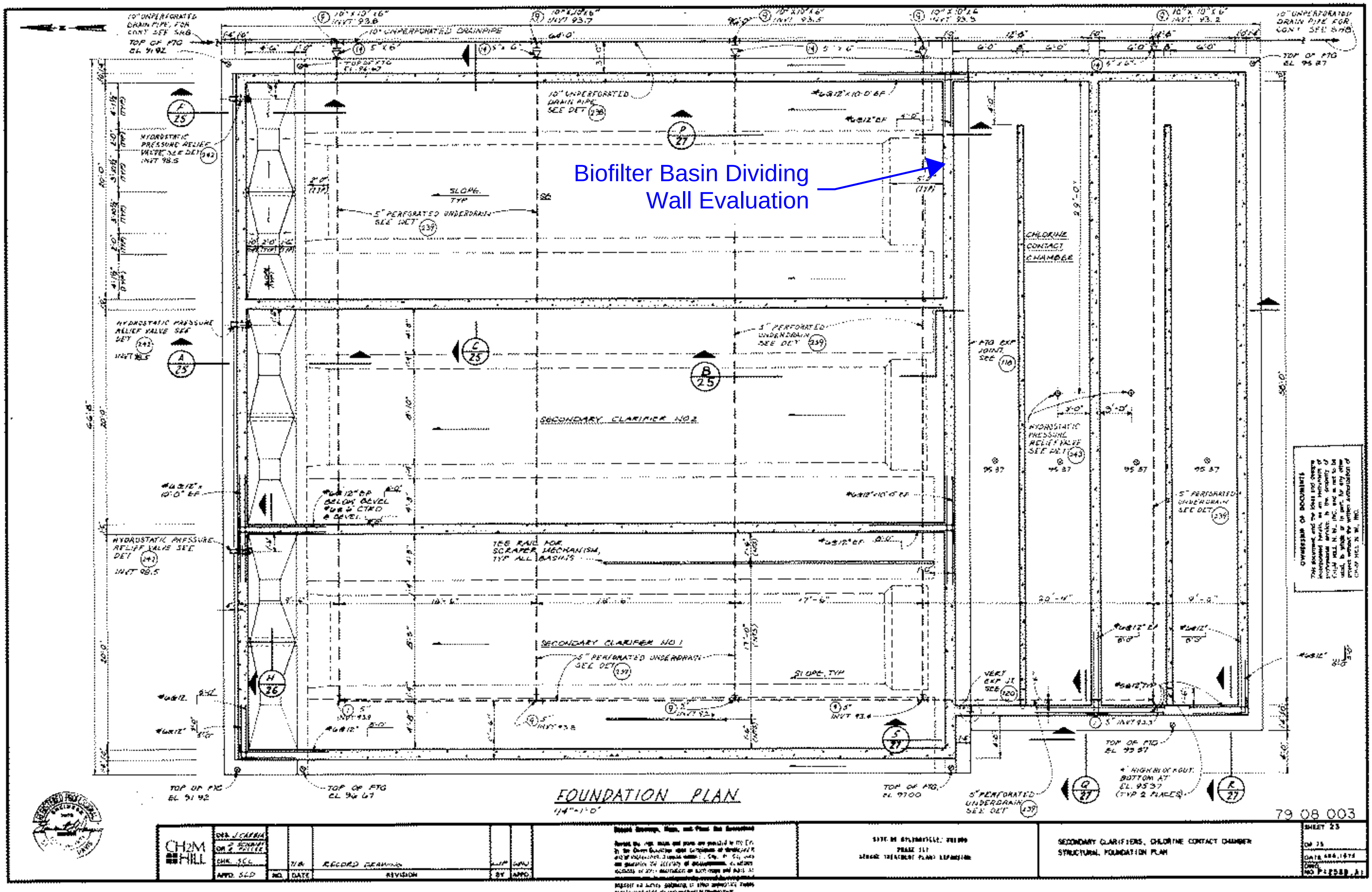
$A_{s\,(max)}$ = 1.73 in^2 $\rho_{(max)}$ = 0.01604

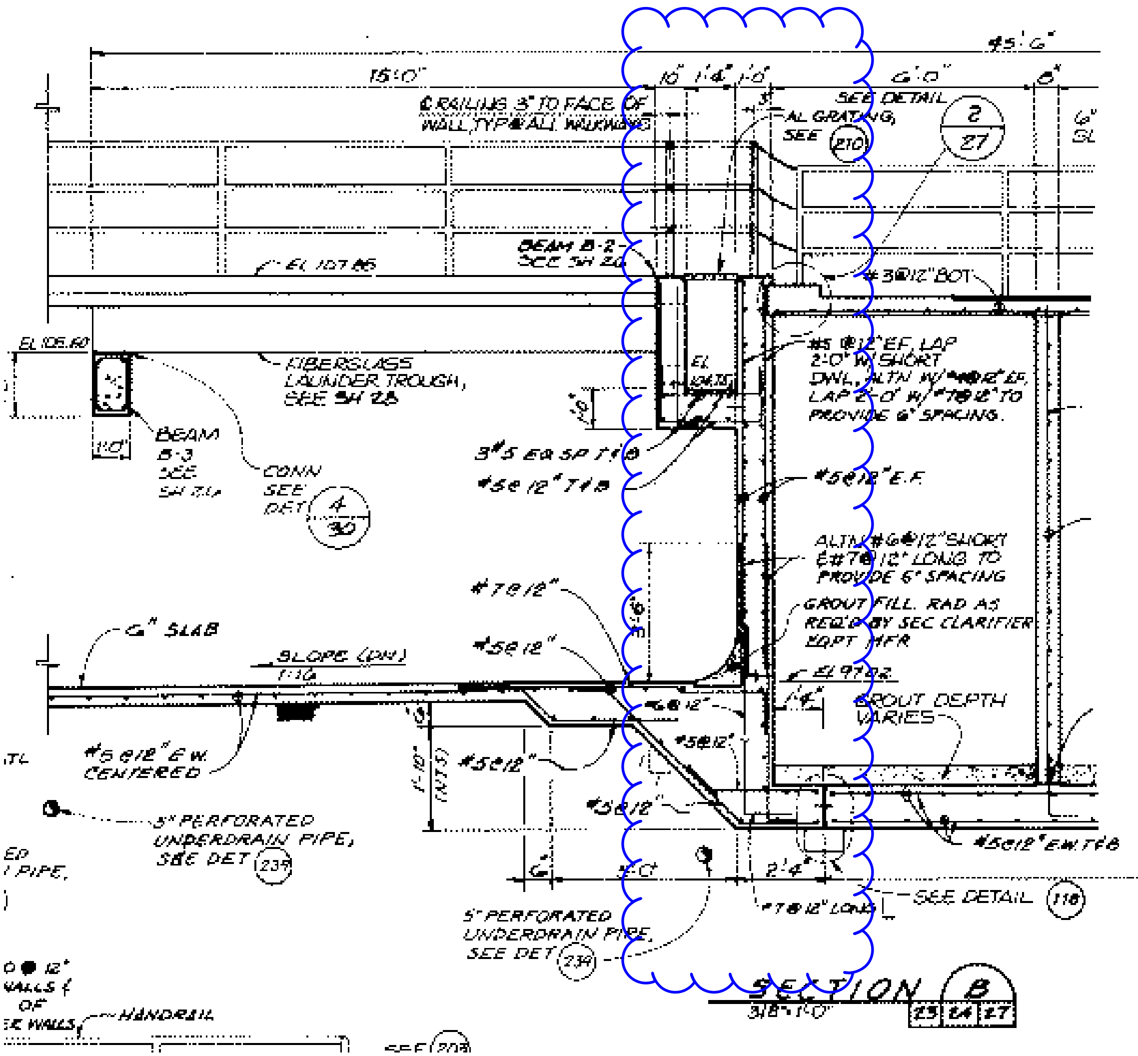
bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

$\phi * M_n$ = 1*0.00407*60*12*9² *(1-0.588*0.00407*60/3)*(ft/12) = 18.851 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

SE23 TREATMENT PLANT 79 08 003





Dividing Wall between Biofilter and WAS Basins

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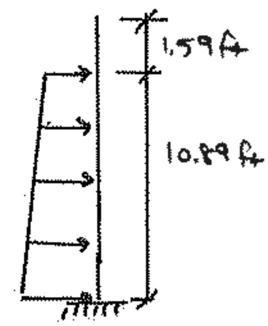
BY BS DATE 7/9/21 SUBJECT City of Wilsonville SHEET NO. ____ OF ____

CHKD. BY ____ DATE ____ Sludge Storage & Biofilters JOB NO. 11962A.00

Sludge Storage & Biofilters - Dividing Perimeter Wall

The dividing wall between the Biofilter basin and the WAS Storage Tank will be checked for the seismic loads. Since the Biofilter basin is considered empty, the water within Storage basin will be both the hydrostatic and hydrodynamic loads. The dividing wall is 12" thick with alternating #6@12" & #7@12" for effective 6" spacing for vertical reinforcing. The horizontal wall reinforcing is #5@12".



See attached spreadsheet for hydrostatic & hydrodynamic loads.

BSE-2E Seismic Level:

Checking wall strength vertically (alternating #6@12" & #7@12")

M_{uy} = 8.74 k.ft/ft ϕM_n = 41.50 k.ft/ft

V_{uy} = 4.73 k/ft ϕV_n = 11.83 k/ft

Moment DCR = $\frac{8.74}{41.50}$ = 0.21 (ok)

Shear DCR = $\frac{4.73}{11.83}$ = 0.40 (ok)

Checking wall strength horizontally (#5@12")

M_{ux} = 5.97 k.ft/ft ϕM_n = 13.48 k.ft/ft

V_{ux} = 2.14 k/ft ϕV_n = 11.83 k/ft

Moment DCR = $\frac{5.97}{13.48}$ = 0.44 (ok)

Shear DCR = $\frac{2.14}{11.83}$ = 0.18 (ok)

CSZ Seismic Level:

Checking wall strength vertically

M_{uy} = 7.91 k.ft/ft ϕM_n = 41.50 k.ft/ft

V_{uy} = 4.28 k/ft ϕV_n = 11.83 k/ft

Moment DCR = $\frac{7.91 \text{ k.ft/ft}}{41.50 \text{ k.ft/ft}}$ = 0.19 (ok)

Shear DCR = $\frac{4.28 \text{ k/ft}}{11.83 \text{ k/ft}}$ = 0.36 (ok)

Checking wall strength horizontally

M_{ux} = 5.40 k.ft/ft ϕM_n = 13.48 k.ft/ft

V_{ux} = 1.94 k/ft ϕV_n = 11.83 k/ft

Moment DCR = $\frac{5.40 \text{ k.ft/ft}}{13.48 \text{ k.ft/ft}}$ = 0.40 (ok)

Shear DCR = $\frac{1.94 \text{ k/ft}}{11.83 \text{ k/ft}}$ = 0.16 (ok)

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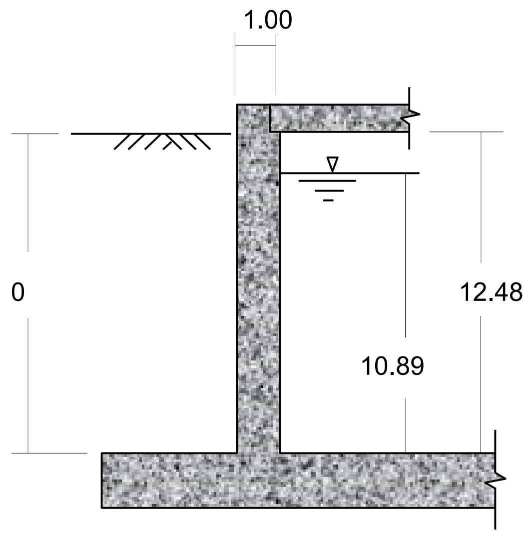
BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Sludge Storage and Biofilters	**JOB NO:**	11962A.00
DESIGN TASK:					WAS Storage Basin Transverse Loading (BSE-2E)		

Static, Seismic, and Hydrodynamic Seismic Wall Pressures using ACI 350.3-06 and an IBC 2012:

wall connection fixity =	**pinned at roof & fixed at floor**	
tank unit width perpendicular to EQ., B =	1	ft
tank inside length in direction of seismic, L =	**12.67**	ft
tank wall thickness, t_w =	**12**	inch
wall height to underside of roof, H_w =	**12.48**	ft
liquid height, H_L =	**10.89**	ft
liquid specific gravity =	**1**	
liquid density, $\gamma_L = (sp.gr.)*\gamma_w$ =	0.0624	k/ft^3
acceleration due to gravity, g =	32.17	ft/sec^2
liquid mass density, $\rho_L = \gamma_L / g$ =	0.00194	$k\text{-}sec^2/ft^4$



WALL SECTION

(wall fixity = pinned at roof & fixed at floor)

Soil Data

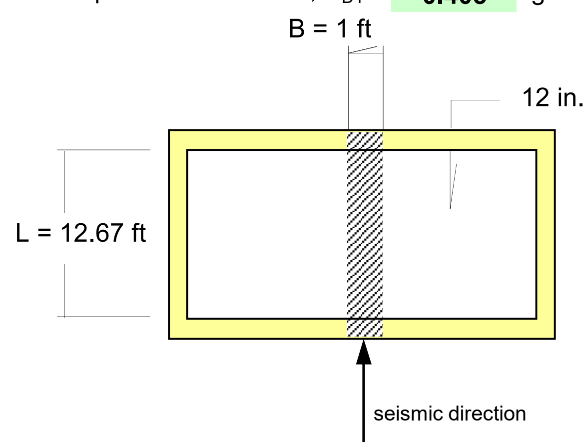
The site has no groundwater.

soil height above top of foundation base =	**0**	ft
groundwater ht. above foundation base =	**0**	ft
dry soil lateral pressure =	**0**	k/ft^3
saturated soil lateral pressure =	**0**	k/ft^3
dry soil unit weight =	**0**	k/ft^3
live load lateral surcharge =	**0.000**	ksf
	0	
concrete strength, f'_c =	**3**	ksi
concrete density, γ_c =	0.150	k/ft^3
concrete modulus of elasticity, E_c =	3122.0	ksi
concrete mass density, $\rho_c = \gamma_c / g$ =	0.004663	$k\text{-}sec^2/ft^4$

Seismic:

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, S_{DS} =	**0.744**	*g
Deisgn, 5% damped, spectral response acceleration at a period of 1-second, S_{D1} =	**0.405**	*g

Structure Risk Category =	**2**
Importance factor, I =	**1**
Response modification factor, R_{wi} =	**3**
Response modification factor, R_{wc} =	**1**



WALL PLAN

Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

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BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Sludge Storage and Biofilters	**JOB NO:**	11962A.00
DESIGN TASK:					WAS Storage Basin Transverse Loading (BSE-2E)		

Weights:

unit 1-ft width wall mass, W_w = (12/12) * (12.48) * 0.15 = 1.87 kip

wall c.g. relative to base, h_w = 12.48 / 2 = 6.240 ft

unit width liquid mass, W_L = (12.67) * (1) * (10.89) * 32.17 = 8.61 kip

Seismic:

1). structure stiffness and dynamic property:

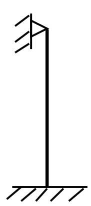
Note: per ASCE 7-10 and IBC 2012, the terms S_{ai} and S_{ac} have been appropriatetly substituted into the seismic equation of ACI 350.

Note: Wi and hi are impulsive component variables calculated on page 3.

wall mass, $m_w = H_w*(t_w / 12)*\rho_c$ = 0.05819 k-sec^2/ft^2

liquid mass, $m_i = (W_i / W_L)*(L/2)*H_L*\rho_L$ = 0.10164 k-sec^2/ft^2

centroidal distance of masses, $h = (h_w*m_w + h_i*m_i) / (m_w + m_i)$ = 4.979 ft



wall fixity condition is pinned at roof & fixed at floor:

wall stiffness is determined using a unit mass load located at the centroidal distance h.

wall flexure stiffness, k = Ec*(tw*Hw/h)³ / (12*(4*Hw-h)*(Hw-h)²) = 2799.85 k/ft/ft

$$\omega_i = \sqrt{\frac{k}{m_w + m_i}} =$$ (2799.85 / (0.0582 + 0.1016))ˆ½ = 132.3522 rad/sec

period of tank plus impulsive mass, $T_i = 2\pi / \omega_i$ = 2 π / 132.3522 = 0.0475 sec

(note: acceleration values to be from a maximum considered earthquake response spectra which will produce a factored load)

design factored spectral response acceleration for impulsive mass (5% damping), $S_{ai} = S_{DS}$ = 0.744 g

2). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$\lambda = \sqrt{3.16\ g\ \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)} =$$ (3.16*32.2*tanh(3.16*(0.8595)))ˆ½ = 10.0385

$$\omega_c = \frac{\lambda}{\sqrt{L}} =$$ 10.0385 / (12.67)ˆ½ = 2.8202 rad/sec,

period of the convective mass, $T_c = 2\pi / \omega_c$ = 2 π / 2.8202 = 2.2279 sec

Long transition period (from map figure 22-15 ASCE 7), T_L = **16** sec

design spectral response acceleration for convective mass (0.5% damping), S_{ac} = 1.5 * Sd1 / Tc = 0.273 g

$$\text{effective mass coeff., } \varepsilon = 0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021\ ,\ \text{ but } \le\ 1.0\ =$$ 0.8195

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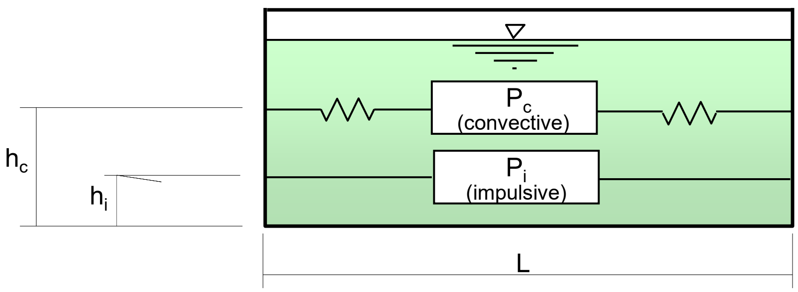
DESIGN TASK:					WAS Storage Basin Transverse Loading (BSE-2E)		



L =	12.67	ft
B =	1	ft
H_L =	10.89	ft
W_L =	8.61	kip
L / H_L =	1.16345	
H_L / L =	0.85951	

3). lateral fluid impulsive force: Dynamic Model

Wi = equivalent mass of the impulsive component of liquid.

$$W_i = W_L \left(\frac{\tanh\left(0.866 \frac{L}{H_L}\right)}{0.866 \frac{L}{H_L}} \right) =$$ 8.61*(tanh(0.866*(1.1635)) / 0.866*(1.1635)) = 6.54 kip

hi (EBP) = HL * (0.5-0.09375*(L/HL)) = 10.89*(0.5-0.09375*(1.1635)) = 4.257 ft

hi (IBP) = HL * {{(0.866*L/HL)/(2*tanh(0.866*L/HL))} -1/8 } = 5.813 ft

impulsive force, $P_i = \left(\frac{S_{ai} \; I}{R_{wi}} \right) W_i =$ (0.744 * 1 / 3)*6.54 = 1.6 kip

4). lateral fluid convective force:

Wc = equivalent mass of the convective component of liquid.

$$W_c = W_L \left(0.264 \left(\frac{L}{H_L} \right) \tanh\left(3.16 \left(\frac{H_L}{L} \right) \right) \right) =$$ 8.61*(0.264*(1.1635)*tanh(3.16*(0.8595))) = 2.62 kip

$$h_{c\ (EBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 1}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) =$$ 7.378 ft

$$h_{c\ (IBP)} = H_L \left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)} \right) =$$ 7.916 ft

convective force, $P_c = \left(\frac{S_{ac} \; I}{R_{wc}} \right) W_c =$ (0.2727 * 1 / 1)*2.62 = 0.7 kip

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DESIGN TASK:					WAS Storage Basin Transverse Loading (BSE-2E)		

5). <u>lateral inertia force of the accelerating wall:</u>

unit width wall mass, W_w = 1.87 kip

wall c.g. relative to base, h_w = 6.240 ft

wall inertia force, $P_w = \left(\frac{S_{ai}\ I\ \varepsilon}{R_{wi}}\right) W_w =$ (0.744*1*0.8195/3)*1.87 = 0.38 kip

6). <u>maximum wave slosh height displacement:</u>

$d_{(max)} = \left(\frac{L}{2}\right)\left(\frac{S_{ac}}{1.4}\ I\right) =$ (12.67 / 2) * (0.2727 / 1.0 * 1) = 1.72 ft

7). <u>vertical acceleration:</u>

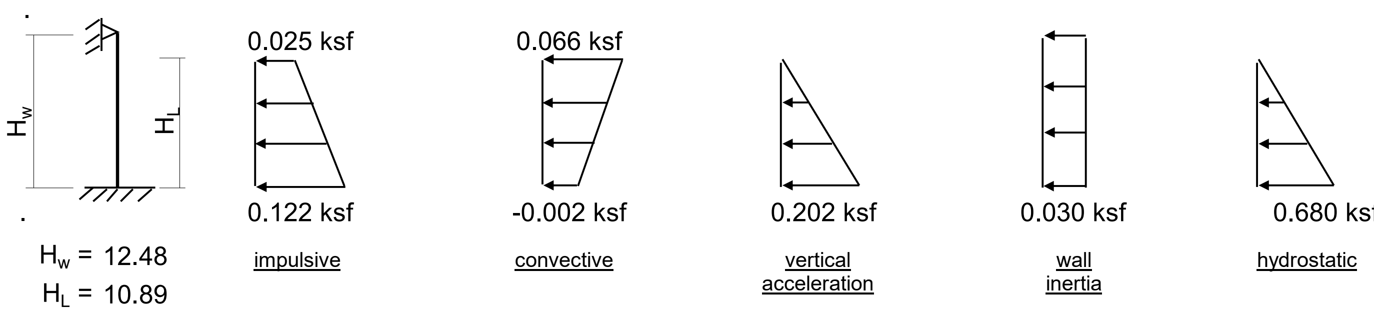
design horizontal accereration, S_{DS} = 0.744 *g

vertical spectral response acceleration (per ACI 350 para 9.4.3), $S_{av} = C_t = 0.4{*}S_{DS}$ = 0.2976 g

per ASCE 7-10 para. 15.7.7.2(b), use $I = R_i = b = 1.0$

Design vertical acceleration, $\ddot{u} = \frac{S_{av}\ I\ b}{R_i} =$ 0.2976*1*1/1 = 0.2976 g

8). <u>vertical force distribution on a unit width using the linear distribution of ACI 350 sec 5.3:</u>



<u>impulsive:</u>

$$p_{iy} = \frac{P_i\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_i = 1.60 kip

h_i = 4.257 ft

at $y = H_L$, p_{iy} = 0.025 ksf

at base $y = 0$, p_{iy} = 0.122 ksf

<u>convective:</u>

$$p_{cy} = \frac{P_c\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_c = 0.70 kip

h_c = 7.378 ft

at $y = H_L$, p_{cy} = 0.066 ksf

at base $y = 0$, p_{cy} = -0.002 ksf

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DESIGN TASK:					WAS Storage Basin Transverse Loading (BSE-2E)		

vertical acceleration:

$$p_{vy} = \ddot{u}\, \gamma_L\, (H_L - y) =$$

ü = 0.2976
at y = H_L, p_{vy} = 0.000 ksf
at base y = 0, p_{vy} = 0.202 ksf

wall inertia:

$$p_{wy} = \frac{S_{ai}\ \ I\ \ \varepsilon\ \gamma_c\ (t_w/12)}{R_{wi}} =$$

p_{wy} = 0.2032 * γ_c * (t_w/12)
at y = H_w, p_{wy} = 0.030 ksf
at base y = 0, p_{wy} = 0.030 ksf

hydrostatic:

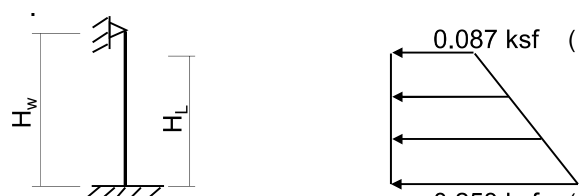
$$q_{hy} = \gamma_L\ (H_L - y) =$$

at y = H_L, q_{hy} = 0.000 ksf
at base y = 0, q_{hy} = 0.680 ksf

combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{(p_{iy} + p_{wy})^2 + p_{cy}^2 + p_{vy}^2} =$$

at y = H_w, p_y = 0.087 ksf
at base y = 0, p_y = 0.253 ksf



(unfactored = 0.087 / 1.4 = 0.062 ksf)

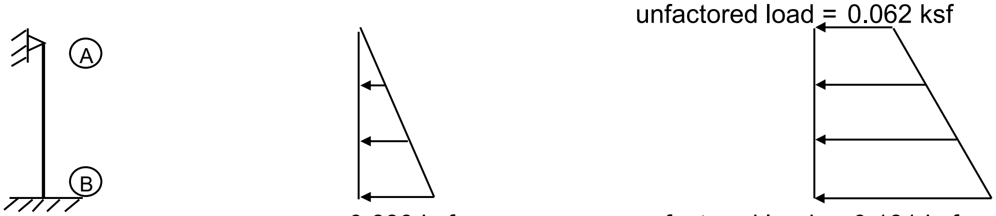
(unfactored = 0.253 / 1.4 = 0.181 ksf)

resultant dynamic pressures

9). wall design pressures for hydrostatic + dynamic:

wall height, H_w = 12.48 ft
liquid height, H_L = 10.89 ft

unfactored load = 0.062 ksf



0.680 ksf
hydrostatic

unfactored load = 0.181 ksf
resultant dynamic pressures

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BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
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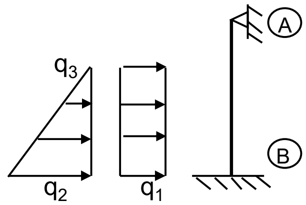
DESIGN TASK:					WAS Storage Basin Transverse Loading (BSE-2E)		

10). wall design pressures for external soil loading:

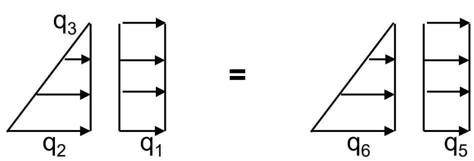
static soil:



The site has no groundwater.

wall height =	12.48	ft
soil height above top of base =	0	ft
groundwater ht. above base =	0	ft
dry soil lateral pressure =	0.000	k/ft^3
sat. soil lateral pressure =	0.000	k/ft^3
live load lateral surcharge =	0.000	ksf

equivalent static soil loadings:



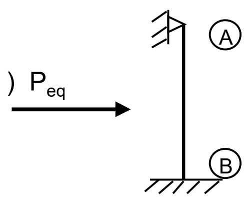
LL lateral surcharge, q1 =	0.0000	ksf
unfactored soil, q2 =	0.0000	ksf
unfactored soil, q3 =	0.0000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.0000	ksf
unfactored q6 =	0.0000	ksf

soil seismic:

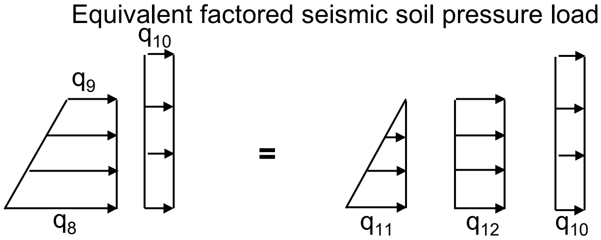
resultant factored soil seismic load per foot of wall width, $P_{u\,(eq)}$ = **0** k/ft

centroid location of the resultant soil seismic from the bottom of wall, h_{eq} = **0** ft

The resultant soil seismic load will be resolved into an equivalent pressure loading…



Equivalent factored seismic soil pressure loading & seismic wall loadings…



equivalent soil seismic, q8 =	0.0000	ksf
equivalent soil seismic, q9 =	0.0000	ksf
wall seismic (see wall page 5), q10 =	0.0305	ksf
equivalent soil seismic, q11 = q8 - q9 =	0.0000	ksf
equivalent soil seismic, q12 = q9 =	0.0000	ksf

unfactored equivalent soil seismic, q8 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q9 =0 / 1.4 =	0.0000	ksf
unfactored wall seismic , q10 = 0.0305 / 1.4 =	0.0218	ksf
unfactored equivalent soil seismic, q11 =0 / 1.4 =	0.0000	ksf
unfactored equivalent soil seismic, q12 =0 / 1.4 =	0.0000	ksf

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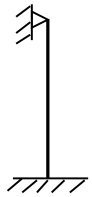
CHKD:		**DESCRIPTION:**			Sludge Storage and Biofilters	**JOB NO:**	11962A.00
DESIGN TASK:					WAS Storage Basin Transverse Loading (BSE-2E)		

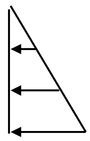
11). Summary of equivalent unfactored pressure loadings for wall load cases 1 thru 4:



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

a). load case 1: hydrostatic water

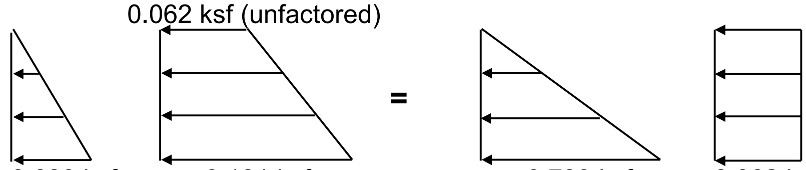
wall height = 12.48 ft
water depth = 10.89 ft



(unfactored) 0.680 ksf

b). load case 2: hydrostatic + dynamic:

0.062 ksf (unfactored)



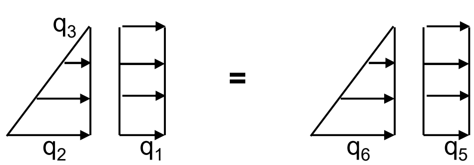
0.680 ksf	0.181 ksf		0.799 ksf	0.062 ksf
(unfactored)	(unfactored)		(unfactored)	(unfactored)
hydrostatic	resultant dynamic pressures			

wall height = 12.48 ft
water depth = 10.89 ft

c). load case 3: static soil + LL surcharge:

wall height = 12.48 ft
soil height on wall = 0 ft

equivalent static soil & surcharge loadings...

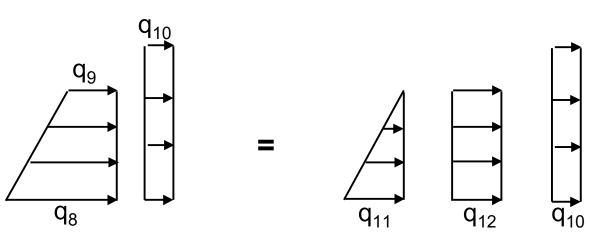


LL lateral surcharge, q1 =	0.000	ksf
unfactored soil, q2 =	0.000	ksf
unfactored soil, q3 =	0.000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.000	ksf
unfactored q6 =	0.000	ksf

d). load case 4: soil seismic: (*note: add static soil pressure q6 & q7 to the seismic soil shown below)

equivalent seismic soil pressure loading & seismic wall loadings…

wall height = 12.48 ft
soil height on wall = 0 ft



unfactored equivalent soil seismic, q8 =	0.000	ksf
unfactored equivalent soil seismic, q9 =	0.000	ksf
unfactored equivalent soil seismic, q10 =	0.022	ksf
unfactored equivalent soil seismic, q11 =	0.000	ksf
unfactored equivalent soil seismic, q12 =	0.000	ksf

Engineers...Working Wonders With Water™

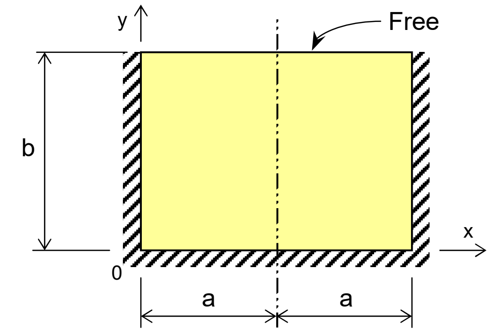
BY:	BS	**DATE:**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Sludge Storage and Biofilters Basin	**JOB NO:**	11962A.00
DESIGN TASK:					Biofilter Basin Dividing Wall along WAS Storage Basin (BSE-2E)		

Rectangular Plate:

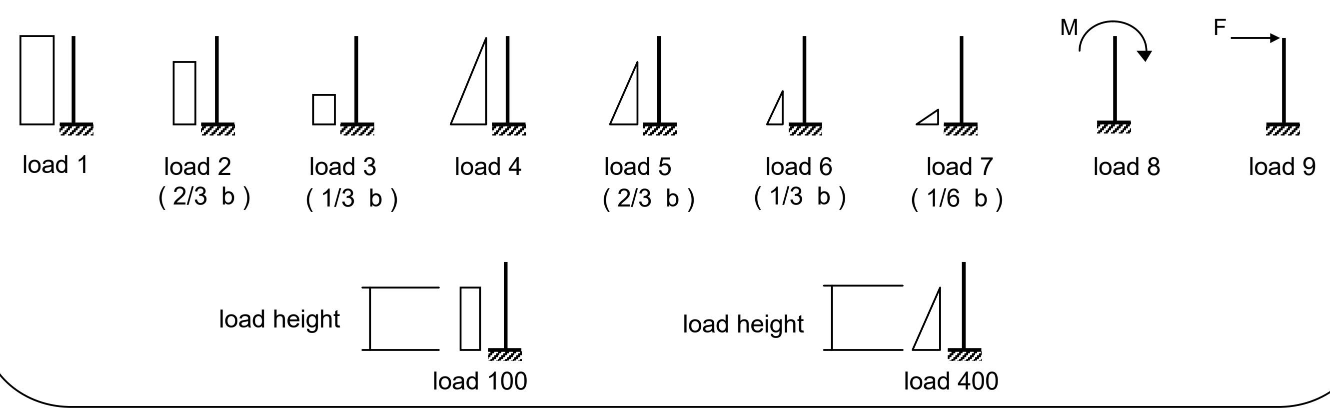
plate boundary condition case number (1, 2, 3, 4, or 5) = **1**

total plate width = 2*a = 2 * 10 = 20 ft
plate dimension, a = **10** ft
plate dimension, b = **12.48** ft

plate sides ratio, a/b = 0.8013



Available Loading Selections - (loads 1 thru 9 , 100 , or 400)



Choice of Available Loadings					
load conditions (4 max)	load type Loading Selection Number	load height, (ft) ...only for custom loads **100** or **400**	unfactored loads: q , M , or F (ksf, ft-k/ft, k/ft)	concrete load factors for moment	for shear
A	**100**	**10.890**	**0.087**	**1**	**1**
B	**400**	**10.890**	**0.166**	**1**	**1**
C	**400**	**10.890**	**0.680**	**1**	**1**
D					

Notes: 1). Load 100 = uniform load of any load height ≥ b/3; Load 400 = triangular load of any load height ≥ b/6.
2). load height must be less than or equal to "b", and uniform load height ≥ "b / 3", and triangular load height ≥ "b / 6" .
3). loads may be positive or negative.

plate thickness, h = **12** in

concrete strength, f 'c = **3** ksi
reinforcing steel strength, fy = **60** ksi
reinforcing clear cover to face of concrete = **2** in
number of curtains of reinforcing, (1 or 2) = **2**
Are bars in "x" or "y" direction closest to face of concrete ? **y**
minimum ratio of horizontal shrinkage-temperature steel = **0.00500**
minimum ratio of vertical shrinkage-temperature steel = **0.00500**

bar locations	d (in)	d' (in)
Mx bending	9"	3"
My bending	9.5"	2.5"

Engineers...Working Wonders With Water ™

BY:	BS	DATE:	Aug-21	CLIENT	City of Wilsonville	SHEET:	
CHKD:		DESCRIPTION:			Sludge Storage and Biofilters Basin	JOB NO:	11962A.00
DESIGN TASK:					Biofilter Basin Dividing Wall along WAS Storage Basin (BSE-2E)		

M_x - Moment Summary

a = 10		Loads: q , M , or F				Boundary Case 1				SUMMARY			
b = 12.48		0.087	0.166	0.680						Final Moments		Reinforcing: (d = 9")	
a / b = 0.8013		Moment Coefficient Multipliers											
		13.550	25.855	105.910									
		Moment Coefficients				M_x Moments, ft-k/ft				M_x	M_{ux}	$A_{s(req'd)}$	$A_{s(min)}$
x / a	y / b	A	B	C	D	A	B	C	D	ft-k/ft	ft-k/ft	in^2/ft	in^2/ft
0	1	0.1290	0.0303	0.0303		1.75	0.78	3.21		5.75	5.75	0.14	0.36
0	0.8	0.1198	0.0330	0.0330		1.62	0.85	3.49		5.97	5.97	0.15	0.36
0	0.6	0.1031	0.0336	0.0336		1.40	0.87	3.56		5.83	5.83	0.15	0.36
0	0.4	0.0727	0.0293	0.0293		0.98	0.76	3.10		4.84	4.84	0.12	0.36
0	0.2	0.0278	0.0143	0.0143		0.38	0.37	1.52		2.27	2.27	0.06	0.36
0	0	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
0.2	0	0.0047	0.0029	0.0029		0.06	0.07	0.30		0.44	0.44	0.01	0.36
0.4	0	0.0121	0.0062	0.0062		0.16	0.16	0.66		0.99	0.99	0.02	0.36
0.6	0	0.0186	0.0088	0.0088		0.25	0.23	0.93		1.41	1.41	0.03	0.36
0.8	0	0.0227	0.0103	0.0103		0.31	0.27	1.09		1.66	1.66	0.04	0.36
1	0	0.0241	0.0108	0.0108		0.33	0.28	1.14		1.75	1.75	0.04	0.36
1	0.2	-0.0038	-0.0025	-0.0025		-0.05	-0.06	-0.26		-0.38	-0.38	-0.01	-0.36
1	0.4	-0.0288	-0.0110	-0.0110		-0.39	-0.28	-1.16		-1.84	-1.84	-0.05	-0.36
1	0.6	-0.0453	-0.0145	-0.0145		-0.61	-0.38	-1.54		-2.53	-2.53	-0.06	-0.36
1	0.8	-0.0538	-0.0155	-0.0155		-0.73	-0.40	-1.64		-2.77	-2.77	-0.07	-0.36
1	1	-0.0602	-0.0161	-0.0161		-0.82	-0.42	-1.70		-2.94	-2.94	-0.07	-0.36
0.8	1	-0.0541	-0.0143	-0.0143		-0.73	-0.37	-1.51		-2.61	-2.61	-0.07	-0.36
0.8	0.8	-0.0486	-0.0139	-0.0139		-0.66	-0.36	-1.47		-2.49	-2.49	-0.06	-0.36
0.8	0.6	-0.0415	-0.0134	-0.0134		-0.56	-0.35	-1.42		-2.33	-2.33	-0.06	-0.36
0.8	0.4	-0.0270	-0.0105	-0.0105		-0.37	-0.27	-1.11		-1.75	-1.75	-0.04	-0.36
0.8	0.2	-0.0038	-0.0026	-0.0026		-0.05	-0.07	-0.28		-0.39	-0.39	-0.01	-0.36

max negative moment, $M_{ux}(-)$ = -2.94 ft-k/ft
max negative steel req'd, $A_s(-)$ = -0.07 in^2/ft
minimum steel req'd = -0.36 in^2/ft
Use

max positive moment, $M_{ux}(+)$ = 5.97 ft-k/ft
max positive steel req'd, $A_s(+)$ = 0.15 in^2/ft
minimum steel req'd = 0.36 in^2/ft
Use

Engineers...Working Wonders With Water™

BY:	BS	**DATE:**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Sludge Storage and Biofilters Basin	**JOB NO:**	11962A.00
DESIGN TASK:					Biofilter Basin Dividing Wall along WAS Storage Basin (BSE-2E)		

M_y - Moment Summary

a = 10
b = 12.48
a / b = 0.8013

Loads: q , M , or F: 0.087 | 0.166 | 0.680

Moment Coefficient Multipliers: 13.550 | 25.855 | 105.910

Boundary Case 1

		Moment Coefficients				M_y Moments, ft-k/ft				SUMMARY: Final Moments		Reinforcing: (d = 9.5")	
x / a	y / b	A	B	C	D	A	B	C	D	M_y ft-k/ft	M_{uy} ft-k/ft	$A_{s(req'd)}$ in^2/ft	$A_{s(min)}$ in^2/ft
0	1	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
0	0.8	0.0239	0.0066	0.0066		0.32	0.17	0.70		1.20	1.20	0.03	0.36
0	0.6	0.0206	0.0068	0.0068		0.28	0.17	0.71		1.17	1.17	0.03	0.36
0	0.4	0.0146	0.0059	0.0059		0.20	0.15	0.62		0.97	0.97	0.02	0.36
0	0.2	0.0056	0.0029	0.0029		0.08	0.07	0.31		0.46	0.46	0.01	0.36
0	0	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
0.2	0	0.0237	0.0143	0.0143		0.32	0.37	1.52		2.21	2.21	0.05	0.36
0.4	0	0.0607	0.0311	0.0311		0.82	0.81	3.30		4.93	4.93	0.12	0.36
0.6	0	0.0928	0.0439	0.0439		1.26	1.13	4.65		7.04	7.04	0.17	0.36
0.8	0	0.1135	0.0514	0.0514		1.54	1.33	5.45		8.31	8.31	0.20	0.36
1	0	0.1205	0.0539	0.0539		1.63	1.39	5.71		8.74	8.74	0.21	0.36
1	0.2	0.0232	0.0022	0.0022		0.31	0.06	0.23		0.61	0.61	0.01	0.36
1	0.4	-0.0204	-0.0137	-0.0137		-0.28	-0.35	-1.45		-2.08	-2.08	-0.05	-0.36
1	0.6	-0.0289	-0.0116	-0.0116		-0.39	-0.30	-1.23		-1.92	-1.92	-0.05	-0.36
1	0.8	-0.0158	-0.0049	-0.0049		-0.21	-0.13	-0.52		-0.87	-0.87	-0.02	-0.36
1	1	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
1	0.4	-0.0204	-0.0137	-0.0137		-0.28	-0.35	-1.45		-2.08	-2.08	-0.05	-0.36
0.8	0.4	-0.0196	-0.0132	-0.0132		-0.26	-0.34	-1.39		-2.00	-2.00	-0.05	-0.36
0.6	0.4	-0.0165	-0.0113	-0.0113		-0.22	-0.29	-1.20		-1.71	-1.71	-0.04	-0.36
0.4	0.4	-0.0104	-0.0078	-0.0078		-0.14	-0.20	-0.82		-1.16	-1.16	-0.03	-0.36
0.2	0.4	0.0000	-0.0019	-0.0019		0.00	-0.05	-0.20		-0.25	-0.25	-0.01	-0.36

max negative moment, $M_{uy}(-)$ = -2.08 ft-k/ft
max negative steel req'd, $A_s(-)$ = -0.05 in^2/ft
minimum steel req'd = -0.36 in^2/ft
Use

max positive moment, $M_{uy}(+)$ = 8.74 ft-k/ft
max positive steel req'd, $A_s(+)$ = 0.21 in^2/ft
minimum steel req'd = 0.36 in^2/ft
Use

Engineers...Working Wonders With Water ™

BY:	BS	DATE:	Aug-21	CLIENT	City of Wilsonville	SHEET:	
CHKD:		DESCRIPTION:			Sludge Storage and Biofilters Basin	JOB NO:	11962A.00
DESIGN TASK:					Biofilter Basin Dividing Wall along WAS Storage Basin (BSE-2E)		

Shear Summary

a = 10
b = 12.48
a / b = 0.8013

	Loads: q , M , or F			
Values	0.087	0.166	0.680	
Shear Coefficient Multipliers	1.086	2.072	8.486	

Boundary Case 1

SUMMARY — Final Shears

x / a	y / b	Shear Coefficients A	B	C	D	Shears, k/ft A	B	C	D	V k/ft	V_u k/ft	ϕV_c k/ft
0	1	0.5081	0.0647	0.0647		0.55	0.13	0.55		1.23	1.23	9.37
0	0.8	0.5607	0.1453	0.1453		0.61	0.30	1.23		2.14	2.14	9.37
0	0.6	0.5511	0.1819	0.1819		0.60	0.38	1.54		2.52	2.52	9.37
0	0.4	0.4592	0.2241	0.2241		0.50	0.46	1.90		2.86	2.86	9.37
0	0.2	0.1324	0.1333	0.1333		0.14	0.28	1.13		1.55	1.55	9.37
0	0.00	-0.0675	-0.0137	-0.0137		-0.07	-0.03	-0.12		-0.22	-0.22	9.37
0.2	0	0.1171	0.1421	0.1421		0.13	0.29	1.21		1.63	1.63	9.37
0.4	0	0.3714	0.2679	0.2679		0.40	0.56	2.27		3.23	3.23	9.37
0.6	0	0.5420	0.3360	0.3360		0.59	0.70	2.85		4.14	4.14	9.37
0.8	0	0.6359	0.3692	0.3692		0.69	0.76	3.13		4.59	4.59	9.37
1	0	0.6655	0.3792	0.3792		0.72	0.79	3.22		4.73	4.73	9.37

Concrete strength reduction factor for shear, ϕ = 1.00

d = 9.0 in

maximum shear, V_u = 4.73 k/ft **OK**

$\phi V_c = \phi*2*(f\ 'c)^{1/2}*b*d$ = (1.00*2*(3000)^½ *12*9.0)/1000 = 11.83 k/ft

Reference:

"Moments and Reactions for Rectangular Plates"
Engineering Monograph No. 27
By: W. T. Moody, United States Bureau of Reclamation

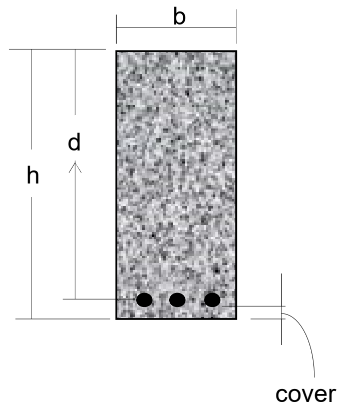
Notes:

Quadratic interpolation is used for intermediate values within the Moody tables and between Moody figures.
The positive sign convention for moments M_x and M_y is tension on the loaded face of the plate.
The M_x moment is in the direction of the x-axis and the M_y moment is in the direction of the y-axis by plate sign convention.

Engineers...Working Wonders With Water ™

BY: BS	**DATE:** Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:	**DESCRIPTION:**		Sludge Storage and Biofilter Basin	**JOB NO:**	11962A.00
DESIGN TASK:	Biofilter Dividing Wall along WAS Storage Basin (Vertical Reinforcing) (BSE-2E)				

Concrete Shear and Moment Strength Capacity



Design for … beam, slab, wall ? **wall**

Properties and Geometry

Compression width of wall, b =	**12**	inch	f'_c (psi) =	**3000**
Thickness of wall, h =	**12**	inch	f_y (psi) =	**60000**
Depth to reinforcing, d =	**9**	inch	ϕ, Bending =	**1**
factored design moment, M_u =	**8.74**	ft-k	ϕ, Shear =	**1**
factored design shear, V_u =	**4.73**	kip	E_s (psi) =	29000000
			E_c (psi) =	3122019
			$n = E_s / E_c$ =	9.29
			β_1 =	0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi * 2 * b * d * (f'_c)^{1/2}$ = 11.83 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #6@12" & #7@12" alternating for 6" effective spacing**

Area steel provided, A_s = **1.04** in^2 $\rho = A_s / bd$ = 0.00963

$\rho(min) < As/bd < \rho(max)$ - OK

$A_{s\,(min)}$ =	0.19	in^2	$\rho_{(min)}$ =	0.00180
$A_{s\,(max)}$ =	1.73	in^2	$\rho_{(max)}$ =	0.01604

bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

$\phi * M_n$ = 1*0.00963*60*12*9² *(1-0.588*0.00963*60/3)*(ft/12) = 41.498 ft-k ≥ Mu

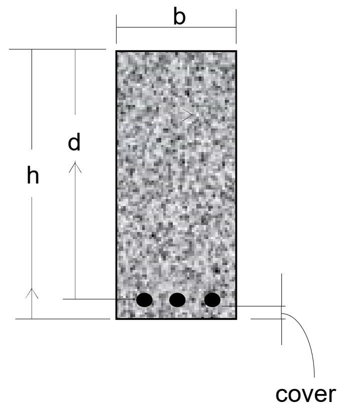
Moment strength ≥ design moment, Okay

Engineers...Working Wonders With Water ™

BY:	BS	**DATE:**	Aug-21	**CLIENT:** City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**	Sludge Storage and Biofilter Basin		**JOB NO:**	11962A.00
DESIGN TASK:	Biofilter Dividing Wall along WAS Storage Basin (Horizontal Reinforcing) (BSE-2E)					

Concrete Shear and Moment Strength Capacity

Design for … beam, slab, wall ? **wall**



Properties and Geometry

Compression width of wall, b = **12** inch
Thickness of wall, h = **12** inch
Depth to reinforcing, d = **9** inch
factored design moment, M_u = **5.97** ft-k
factored design shear, V_u = **2.14** kip

f'_c (psi) = **3000**
f_y (psi) = **60000**
ϕ, Bending = **1**
ϕ, Shear = **1**
E_s (psi) = 29000000
E_c (psi) = 3122019
$n = E_s / E_c$ = 9.29
β_1 = 0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi * 2 * b * d * (f'_c)^{1/2}$ = 11.83 kip ≥ Vu
stirrup spacing, s = **0** in
stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #5@12"**
Area steel provided, A_s = **0.31** in^2 $\rho = A_s / bd$ = 0.00287
$\rho(min) < As/bd < \rho(max)$ - OK

$A_{s\,(min)}$ = 0.19 in^2 $\rho_{(min)}$ = 0.00180
$A_{s\,(max)}$ = 1.73 in^2 $\rho_{(max)}$ = 0.01604

bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

$\phi * M_n$ = 1*0.00287*60*12*9² *(1-0.588*0.00287*60/3)*(ft/12) = 13.479 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

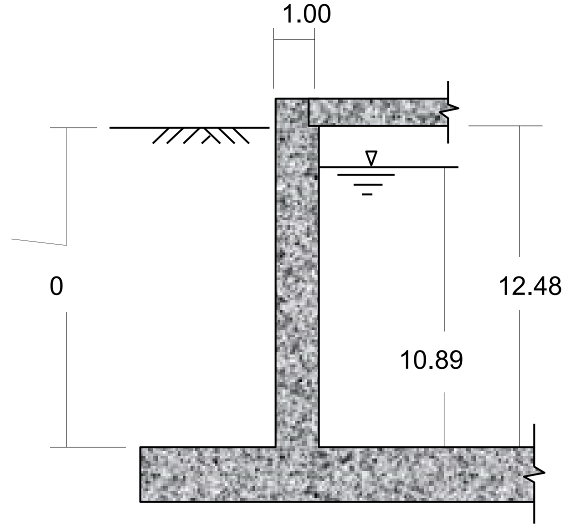
Engineers...Working Wonders With Water™

BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Sludge Storage and Biofilters	**JOB NO:**	11962A.00
DESIGN TASK:					WAS Storage Basin Transverse Loading (CSZ)		

Static, Seismic, and Hydrodynamic Seismic Wall Pressures using ACI 350.3-06 and an IBC 2012:

wall connection fixity = **pinned at roof & fixed at floor**

tank unit width perpendicular to EQ., B =	1	ft
tank inside length in direction of seismic, L =	**12.67**	ft
tank wall thickness, t_w =	**12**	inch
wall height to underside of roof, H_w =	**12.48**	ft
liquid height, H_L =	**10.89**	ft
liquid specific gravity =	**1**	
liquid density, $\gamma_L = (\text{sp.gr.}) * \gamma_w$ =	0.0624	k/ft^3
acceleration due to gravity, g =	32.17	ft/sec^2
liquid mass density, $\rho_L = \gamma_L / g$ =	0.00194	$k\text{-}sec^2/ft^4$



WALL SECTION

(wall fixity = pinned at roof & fixed at floor)

Soil Data

The site has no groundwater.

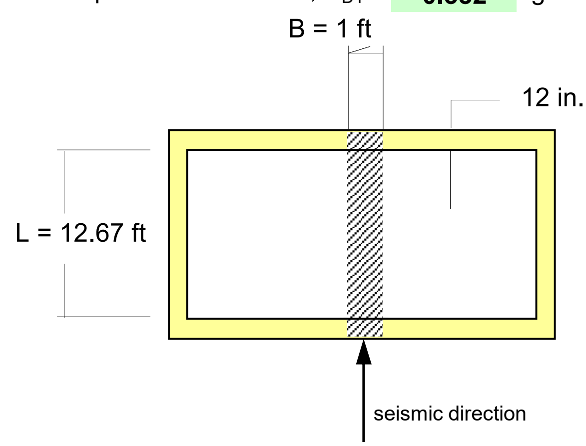
soil height above top of foundation base =	**0**	ft
groundwater ht. above foundation base =	**0**	ft
dry soil lateral pressure =	**0**	k/ft^3
saturated soil lateral pressure =	**0**	k/ft^3
dry soil unit weight =	**0**	k/ft^3
live load lateral surcharge =	**0.000**	ksf
	0	
concrete strength, f'_c =	**3**	ksi
concrete density, γ_c =	0.150	k/ft^3
concrete modulus of elasticity, E_c =	3122.0	ksi
concrete mass density, $\rho_c = \gamma_c / g$ =	0.004663	$k\text{-}sec^2/ft^4$

Seismic:

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, S_{DS} = **0.446** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, S_{D1} = **0.332** *g

Structure Risk Category =	**3**
Importance factor, I =	**1.25**
Response modification factor, R_{wi} =	**3**
Response modification factor, R_{wc} =	**1**



WALL PLAN

Load Cases:

case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

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BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Sludge Storage and Biofilters	**JOB NO:**	11962A.00
DESIGN TASK:					WAS Storage Basin Transverse Loading (CSZ)		

Weights:

unit 1-ft width wall mass, W_w =	(12/12) * (12.48) * 0.15 =	1.87	kip
wall c.g. relative to base, h_w =	12.48 / 2 =	6.240	ft
unit width liquid mass, W_L =	(12.67) * (1) * (10.89) * 32.17 =	8.61	kip

Seismic:

1). structure stiffness and dynamic property:

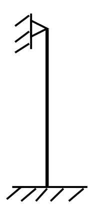
Note: per ASCE 7-10 and IBC 2012, the terms S_{ai} and S_{ac} have been appropriatetly substituted into the seismic equation of ACI 350.

Note: Wi and hi are impulsive component variables calculated on page 3.

wall mass, $m_w = H_w*(t_w / 12)*\rho_c$ = 0.05819 k-sec^2/ft^2

liquid mass, $m_i = (W_i / W_L)*(L/2)*H_L*\rho_L$ = 0.10164 k-sec^2/ft^2

centroidal distance of masses, $h = (h_w*m_w + h_i*m_i) / (m_w + m_i)$ = 4.979 ft



wall fixity condition is pinned at roof & fixed at floor:

wall stiffness is determined using a unit mass load located at the centroidal distance h.

wall flexure stiffness, k = Ec*(tw*Hw/h)³ / (12*(4*Hw-h)*(Hw-h)²) = 2799.85 k/ft/ft

$$\omega_i = \sqrt{\frac{k}{m_w + m_i}} = \quad (2799.85 / (0.0582 + 0.1016))\hat{}½ = 132.3522 \text{ rad/sec}$$

period of tank plus impulsive mass, $T_i = 2\pi / \omega_i$ = 2 π / 132.3522 = 0.0475 sec

(note: acceleration values to be from a maximum considered earthquake response spectra which will produce a factored load)

design factored spectral response acceleration for impulsive mass (5% damping), $S_{ai} = S_{DS}$ = 0.446 g

2). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$\lambda = \sqrt{3.16\ g\ \tanh\left(3.16\left(\frac{H_L}{L}\right)\right)} = \quad (3.16*32.2*tanh(3.16*(0.8595)))\hat{}½ = 10.0385$$

$$\omega_c = \frac{\lambda}{\sqrt{L}} = \quad 10.0385 / (12.67)\hat{}½ = 2.8202 \text{ rad/sec,}$$

period of the convective mass, $T_c = 2\pi / \omega_c$ = 2 π / 2.8202 = 2.2279 sec

Long transition period (from map figure 22-15 ASCE 7), T_L = **16** sec

design spectral response acceleration for convective mass (0.5% damping), S_{ac} = 1.5 * Sd1 / Tc = 0.224 g

$$\text{effective mass coeff., } \varepsilon = 0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021\ ,\ \text{ but } \le 1.0 = 0.8195$$

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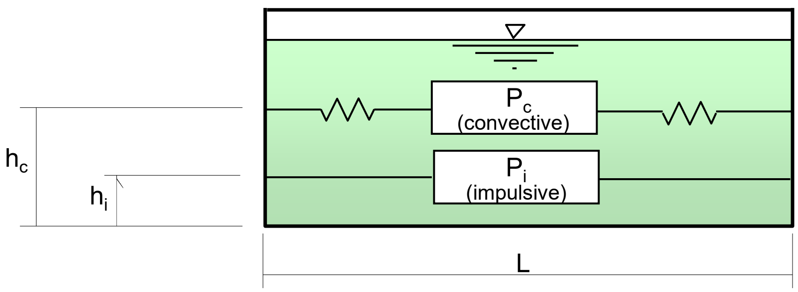
DESIGN TASK:					WAS Storage Basin Transverse Loading (CSZ)		



L =	12.67	ft
B =	1	ft
H_L =	10.89	ft
W_L =	8.61	kip
L / H_L =	1.16345	
H_L / L =	0.85951	

3). lateral fluid impulsive force: Dynamic Model

Wi = equivalent mass of the impulsive component of liquid.

$$W_i = W_L\left(\frac{\tanh\left(0.866\frac{L}{H_L}\right)}{0.866\frac{L}{H_L}}\right) =$$ 8.61*(tanh(0.866*(1.1635)) / 0.866*(1.1635)) = 6.54 kip

hi (EBP) = HL * (0.5-0.09375*(L/HL)) = 10.89*(0.5-0.09375*(1.1635)) = 4.257 ft

hi (IBP) = HL * {{(0.866*L/HL)/(2*tanh(0.866*L/HL))} -1/8 } = 5.813 ft

impulsive force, $P_i = \left(\frac{S_{ai}\ I}{R_{wi}}\right)W_i =$ (0.446 * 1.25 / 3)*6.54 = 1.2 kip

4). lateral fluid convective force:

Wc = equivalent mass of the convective component of liquid.

$$W_c = W_L\left(0.264\left(\frac{L}{H_L}\right)\tanh\left(3.16\left(\frac{H_L}{L}\right)\right)\right) =$$ 8.61*(0.264*(1.1635)*tanh(3.16*(0.8595))) = 2.62 kip

$$h_{c\ (EBP)} = H_L\left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 1}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)}\right) = 7.378 \text{ ft}$$

$$h_{c\ (IBP)} = H_L\left(1 - \frac{\cosh\left(3.16\left(\frac{H_L}{L}\right)\right) - 2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left(3.16\left(\frac{H_L}{L}\right)\right)}\right) = 7.916 \text{ ft}$$

convective force, $P_c = \left(\frac{S_{ac}\ I}{R_{wc}}\right)W_c =$ (0.2235 * 1.25 / 1)*2.62 = 0.7 kip

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5). lateral inertia force of the accelerating wall:

unit width wall mass, W_w = 1.87 kip

wall c.g. relative to base, h_w = 6.240 ft

$$\text{wall inertia force, } P_w = \left(\frac{S_{ai}\ I\ \varepsilon}{R_{wi}}\right) W_w = \quad (\ 0.446*1.25*0.8195/3\ )*1.87 = \quad 0.28 \text{ kip}$$

6). maximum wave slosh height displacement:

$$d_{(max)} = \left(\frac{L}{2}\right)\left(\frac{S_{ac}}{1.4}\ I\right) = (\ 12.67\ /\ 2)\ *\ (\ 0.2235\ /\ 1.0\ *\ 1.25\ ) = \quad 1.77 \text{ ft}$$

7). vertical acceleration:

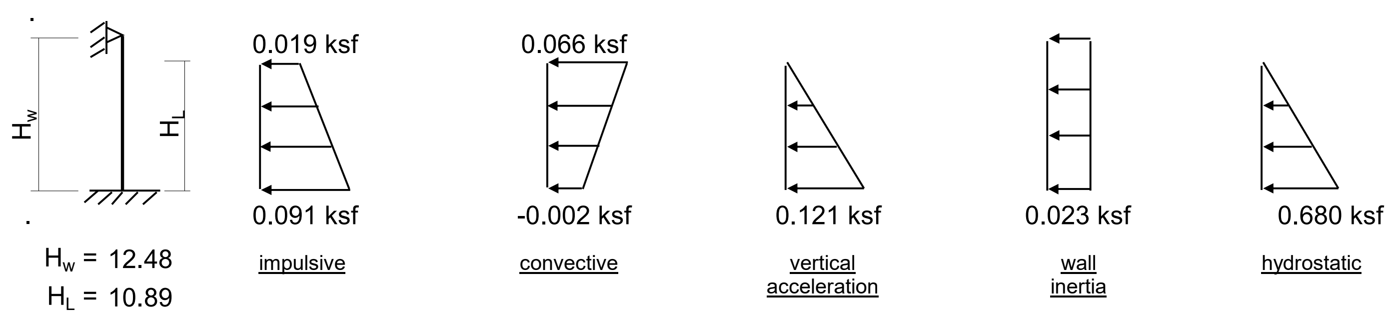
design horizontal accereration, S_{DS} = 0.446 *g

vertical spectral response acceleration (per ACI 350 para 9.4.3), $S_{av} = C_t = 0.4*S_{DS}$ = 0.1784 g

per ASCE 7-10 para. 15.7.7.2(b), use $I = R_i = b = 1.0$

$$\text{Design vertical acceleration, } \ddot{u} = \frac{S_{av}\ I\ b}{R_i} = \quad 0.1784*1*1/1 = \quad 0.1784 \text{ g}$$

8). vertical force distribution on a unit width using the linear distribution of ACI 350 sec 5.3:



impulsive:

$$p_{iy} = \frac{P_i\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_i = 1.20 kip

h_i = 4.257 ft

at $y = H_L$, p_{iy} = 0.019 ksf

at base $y = 0$, p_{iy} = 0.091 ksf

convective:

$$p_{cy} = \frac{P_c\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{2\ B\ H_L^2} =$$

P_c = 0.70 kip

h_c = 7.378 ft

at $y = H_L$, p_{cy} = 0.066 ksf

at base $y = 0$, p_{cy} = -0.002 ksf

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vertical acceleration:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

$\ddot{u}$ = 0.1784
at y = H_L, p_{vy} = 0.000 ksf
at base y = 0, p_{vy} = 0.121 ksf

wall inertia:

$$p_{wy} = \frac{S_{ai}\;\; I\;\; \varepsilon\;\gamma_c\;(t_w/12)}{R_{wi}} =$$

p_{wy} = 0.1523 * γ_c * (t_w/12)
at y = H_w, p_{wy} = 0.023 ksf
at base y = 0, p_{wy} = 0.023 ksf

hydrostatic:

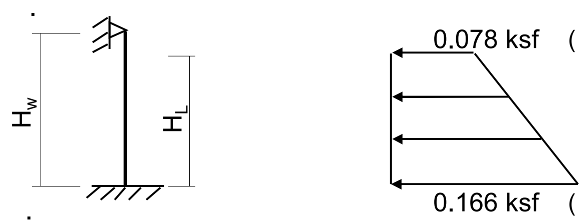
$$q_{hy} = \gamma_L\,(H_L - y) =$$

at y = H_L, q_{hy} = 0.000 ksf
at base y = 0, q_{hy} = 0.680 ksf

combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{(p_{iy} + p_{wy})^2 + p_{cy}^2 + p_{vy}^2} =$$

at y = H_w, p_y = 0.078 ksf
at base y = 0, p_y = 0.166 ksf



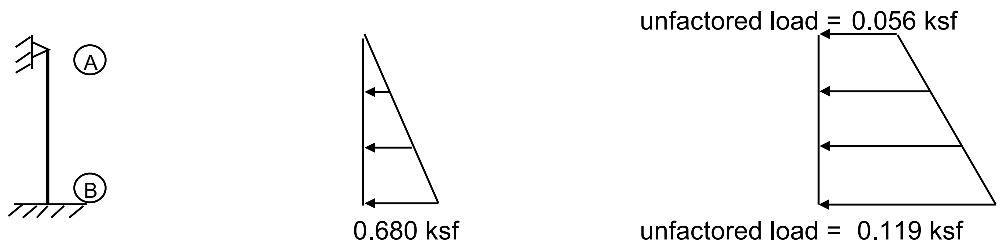
(unfactored = 0.078 / 1.4 = 0.056 ksf)

(unfactored = 0.166 / 1.4 = 0.119 ksf)

resultant dynamic pressures

9). wall design pressures for hydrostatic + dynamic:

wall height, H_w = 12.48 ft
liquid height, H_L = 10.89 ft



hydrostatic

resultant dynamic pressures

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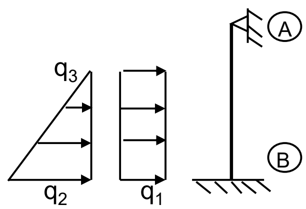
10). wall design pressures for external soil loading:

static soil:

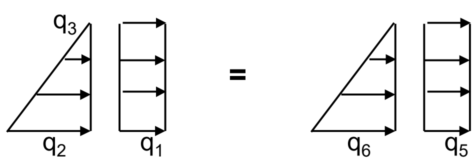


The site has no groundwater.

wall height = 12.48 ft
soil height above top of base = 0 ft
groundwater ht. above base = 0 ft
dry soil lateral pressure = 0.000 k/ft^3
sat. soil lateral pressure = 0.000 k/ft^3
live load lateral surcharge = 0.000 ksf

equivalent static soil loadings:

LL lateral surcharge, q1 = 0.0000 ksf
unfactored soil, q2 = 0.0000 ksf
unfactored soil, q3 = 0.0000 ksf
0.000



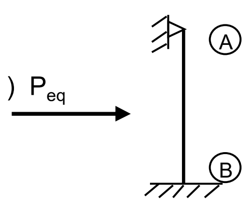
equivalent soil loadings:
unfactored q5 = 0.0000 ksf
unfactored q6 = 0.0000 ksf

soil seismic:

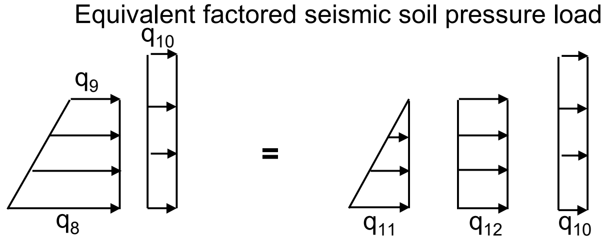
resultant factored soil seismic load per foot of wall width, $P_{u\,(eq)}$ = **0** k/ft

centroid location of the resultant soil seismic from the bottom of wall, h_{eq} = **0** ft

The resultant soil seismic load will be resolved into an equivalent pressure loading…



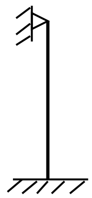
Equivalent factored seismic soil pressure loading & seismic wall loadings…



equivalent soil seismic, q8 = 0.0000 ksf
equivalent soil seismic, q9 = 0.0000 ksf
wall seismic (see wall page 5), q10 = 0.0228 ksf
equivalent soil seismic, q11 = q8 - q9 = 0.0000 ksf
equivalent soil seismic, q12 = q9 = 0.0000 ksf

unfactored equivalent soil seismic, q8 =0 / 1.4 = 0.0000 ksf
unfactored equivalent soil seismic, q9 =0 / 1.4 = 0.0000 ksf
unfactored wall seismic , q10 = 0.0228 / 1.4 = 0.0163 ksf
unfactored equivalent soil seismic, q11 =0 / 1.4 = 0.0000 ksf
unfactored equivalent soil seismic, q12 =0 / 1.4 = 0.0000 ksf

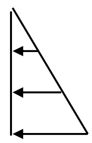
11). Summary of equivalent unfactored pressure loadings for wall load cases 1 thru 4:



Load Cases:
case 1 = water
case 2 = water + water seismic + wall seismic
case 3 = soil + lateral surcharge
case 4 = soil + soil seismic + wall seismic

a). load case 1: hydrostatic water

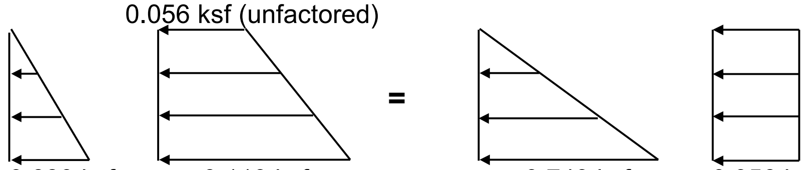
wall height = 12.48 ft
water depth = 10.89 ft



(unfactored) 0.680 ksf

b). load case 2: hydrostatic + dynamic:

0.056 ksf (unfactored)



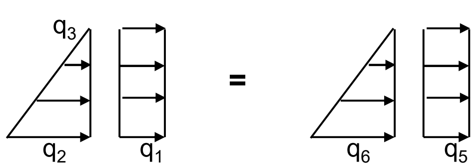
0.680 ksf (unfactored) hydrostatic	0.119 ksf (unfactored) resultant dynamic pressures	=	0.743 ksf (unfactored)	0.056 ksf (unfactored)

wall height = 12.48 ft
water depth = 10.89 ft

c). load case 3: static soil + LL surcharge:

wall height = 12.48 ft
soil height on wall = 0 ft

equivalent static soil & surcharge loadings...

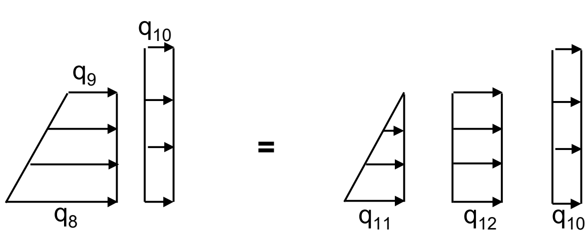


LL lateral surcharge, q1 =	0.000	ksf
unfactored soil, q2 =	0.000	ksf
unfactored soil, q3 =	0.000	ksf
	0.000	
equivalent soil loadings:		
unfactored q5 =	0.000	ksf
unfactored q6 =	0.000	ksf

d). load case 4: soil seismic: (*note: add static soil pressure q6 & q7 to the seismic soil shown below)

equivalent seismic soil pressure loading & seismic wall loadings…

wall height = 12.48 ft
soil height on wall = 0 ft



unfactored equivalent soil seismic, q8 =	0.000	ksf
unfactored equivalent soil seismic, q9 =	0.000	ksf
unfactored equivalent soil seismic, q10 =	0.016	ksf
unfactored equivalent soil seismic, q11 =	0.000	ksf
unfactored equivalent soil seismic, q12 =	0.000	ksf

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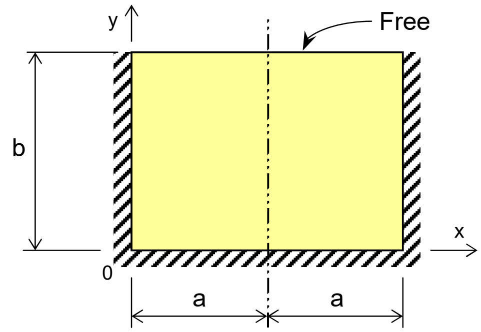
CHKD:		DESCRIPTION:			Sludge Storage and Biofilters Basin	JOB NO:	11962A.00
DESIGN TASK:					Biofilter Basin Dividing Wall along WAS Storage Basin (CSZ)		

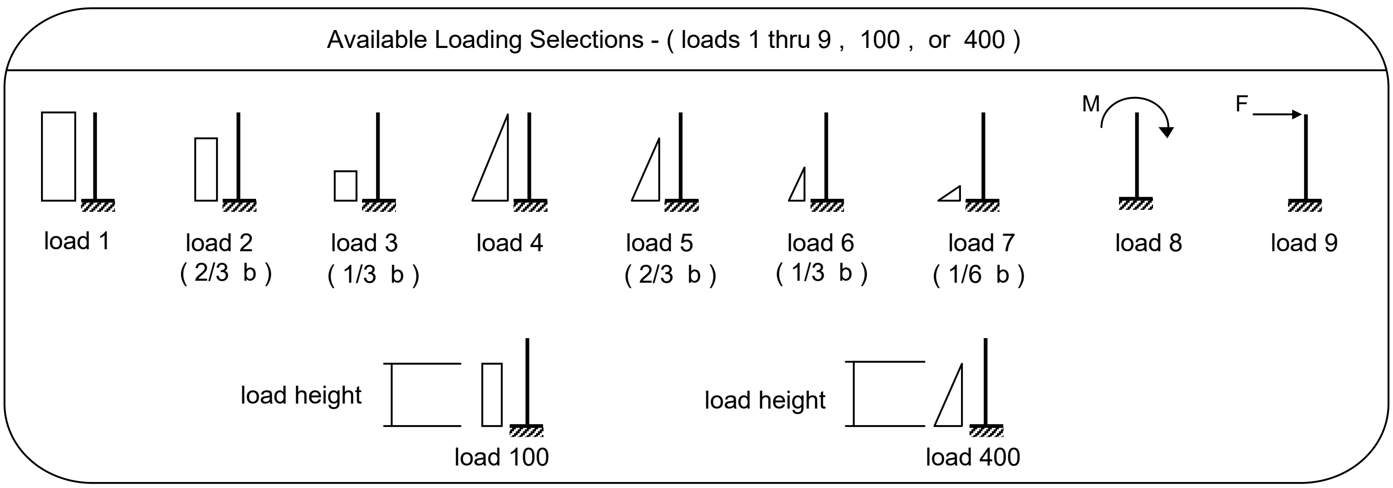
Rectangular Plate:

plate boundary condition case number (1, 2, 3, 4, or 5) = **1**

total plate width = 2*a = 2 * 10 = 20 ft
plate dimension, a = **10** ft
plate dimension, b = **12.48** ft

plate sides ratio, a/b = 0.8013





Choice of Available Loadings					
load conditions (4 max)	load type Loading Selection Number	load height, (ft) ...only for custom loads **100** or **400**	unfactored loads: q , M , or F (ksf, ft-k/ft, k/ft)	concrete load factors for moment	for shear
A	**100**	**10.890**	**0.078**	**1**	**1**
B	**400**	**10.890**	**0.088**	**1**	**1**
C	**400**	**10.890**	**0.680**	**1**	**1**
D					

Notes: 1). Load 100 = uniform load of any load height ≥ b/3; Load 400 = triangular load of any load height ≥ b/6.
2). load height must be less than or equal to "b", and uniform load height ≥ "b / 3", and triangular load height ≥ "b / 6" .
3). loads may be positive or negative.

plate thickness, h = **12** in

concrete strength, f 'c = **3** ksi
reinforcing steel strength, fy = **60** ksi
reinforcing clear cover to face of concrete = **2** in
number of curtains of reinforcing, (1 or 2) = **2**
Are bars in "x" or "y" direction closest to face of concrete ? **y**
minimum ratio of horizontal shrinkage-temperature steel = **0.00500**
minimum ratio of vertical shrinkage-temperature steel = **0.00500**

bar locations	d (in)	d' (in)
Mx bending	9"	3"
My bending	9.5"	2.5"

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M_x - Moment Summary

a = 10, b = 12.48, a / b = 0.8013		Loads: q, M, or F				Boundary Case 1				SUMMARY			
		0.078	0.088	0.680						Final Moments		Reinforcing: (d = 9")	
		Moment Coefficient Multipliers											
		12.149	13.706	105.910									
		Moment Coefficients				M_x Moments, ft-k/ft				M_x	M_{ux}	$A_{s(req'd)}$	$A_{s(min)}$
x / a	y / b	A	B	C	D	A	B	C	D	ft-k/ft	ft-k/ft	in^2/ft	in^2/ft
0	1	0.1290	0.0303	0.0303		1.57	0.42	3.21		5.20	5.20	0.13	0.36
0	0.8	0.1198	0.0330	0.0330		1.46	0.45	3.49		5.40	5.40	0.14	0.36
0	0.6	0.1031	0.0336	0.0336		1.25	0.46	3.56		5.28	5.28	0.13	0.36
0	0.4	0.0727	0.0293	0.0293		0.88	0.40	3.10		4.38	4.38	0.11	0.36
0	0.2	0.0278	0.0143	0.0143		0.34	0.20	1.52		2.05	2.05	0.05	0.36
0	0	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
0.2	0	0.0047	0.0029	0.0029		0.06	0.04	0.30		0.40	0.40	0.01	0.36
0.4	0	0.0121	0.0062	0.0062		0.15	0.09	0.66		0.89	0.89	0.02	0.36
0.6	0	0.0186	0.0088	0.0088		0.23	0.12	0.93		1.28	1.28	0.03	0.36
0.8	0	0.0227	0.0103	0.0103		0.28	0.14	1.09		1.51	1.51	0.04	0.36
1	0	0.0241	0.0108	0.0108		0.29	0.15	1.14		1.59	1.59	0.04	0.36
1	0.2	-0.0038	-0.0025	-0.0025		-0.05	-0.03	-0.26		-0.34	-0.34	-0.01	-0.36
1	0.4	-0.0288	-0.0110	-0.0110		-0.35	-0.15	-1.16		-1.67	-1.67	-0.04	-0.36
1	0.6	-0.0453	-0.0145	-0.0145		-0.55	-0.20	-1.54		-2.29	-2.29	-0.06	-0.36
1	0.8	-0.0538	-0.0155	-0.0155		-0.65	-0.21	-1.64		-2.50	-2.50	-0.06	-0.36
1	1	-0.0602	-0.0161	-0.0161		-0.73	-0.22	-1.70		-2.66	-2.66	-0.07	-0.36
0.8	1	-0.0541	-0.0143	-0.0143		-0.66	-0.20	-1.51		-2.37	-2.37	-0.06	-0.36
0.8	0.8	-0.0486	-0.0139	-0.0139		-0.59	-0.19	-1.47		-2.25	-2.25	-0.06	-0.36
0.8	0.6	-0.0415	-0.0134	-0.0134		-0.50	-0.18	-1.42		-2.11	-2.11	-0.05	-0.36
0.8	0.4	-0.0270	-0.0105	-0.0105		-0.33	-0.14	-1.11		-1.58	-1.58	-0.04	-0.36
0.8	0.2	-0.0038	-0.0026	-0.0026		-0.05	-0.04	-0.28		-0.36	-0.36	-0.01	-0.36

max negative moment, $M_{ux}(-)$ = -2.66 ft-k/ft
max negative steel req'd, $A_s(-)$ = -0.07 in^2/ft
minimum steel req'd = -0.36 in^2/ft
Use

max positive moment, $M_{ux}(+)$ = 5.40 ft-k/ft
max positive steel req'd, $A_s(+)$ = 0.14 in^2/ft
minimum steel req'd = 0.36 in^2/ft
Use

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BY:	BS	DATE:	Aug-21	CLIENT	City of Wilsonville	SHEET:	
CHKD:		DESCRIPTION:			Sludge Storage and Biofilters Basin	JOB NO:	11962A.00
DESIGN TASK:					Biofilter Basin Dividing Wall along WAS Storage Basin (CSZ)		

M_y - Moment Summary

		Loads: q , M , or F				Boundary Case 1				SUMMARY			
a =	10	0.078	0.088	0.680						Final Moments		Reinforcing: (d = 9.5")	
b =	12.48	Moment Coefficient Multipliers											
a / b =	0.8013	12.149	13.706	105.910									
		Moment Coefficients				M_y Moments, ft-k/ft				M_y	M_{uy}	$A_{s(req'd)}$	$A_{s(min)}$
x / a	y / b	A	B	C	D	A	B	C	D	ft-k/ft	ft-k/ft	in^2/ft	in^2/ft
0	1	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
0	0.8	0.0239	0.0066	0.0066		0.29	0.09	0.70		1.08	1.08	0.03	0.36
0	0.6	0.0206	0.0068	0.0068		0.25	0.09	0.71		1.06	1.06	0.02	0.36
0	0.4	0.0146	0.0059	0.0059		0.18	0.08	0.62		0.88	0.88	0.02	0.36
0	0.2	0.0056	0.0029	0.0029		0.07	0.04	0.31		0.41	0.41	0.01	0.36
0	0	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
0.2	0	0.0237	0.0143	0.0143		0.29	0.20	1.52		2.00	2.00	0.05	0.36
0.4	0	0.0607	0.0311	0.0311		0.74	0.43	3.30		4.46	4.46	0.11	0.36
0.6	0	0.0928	0.0439	0.0439		1.13	0.60	4.65		6.38	6.38	0.15	0.36
0.8	0	0.1135	0.0514	0.0514		1.38	0.70	5.45		7.53	7.53	0.18	0.36
1	0	0.1205	0.0539	0.0539		1.46	0.74	5.71		7.91	7.91	0.19	0.36
1	0.2	0.0232	0.0022	0.0022		0.28	0.03	0.23		0.55	0.55	0.01	0.36
1	0.4	-0.0204	-0.0137	-0.0137		-0.25	-0.19	-1.45		-1.89	-1.89	-0.04	-0.36
1	0.6	-0.0289	-0.0116	-0.0116		-0.35	-0.16	-1.23		-1.74	-1.74	-0.04	-0.36
1	0.8	-0.0158	-0.0049	-0.0049		-0.19	-0.07	-0.52		-0.78	-0.78	-0.02	-0.36
1	1	0.0000	0.0000	0.0000		0.00	0.00	0.00		0.00	0.00	0.00	0.36
1	0.4	-0.0204	-0.0137	-0.0137		-0.25	-0.19	-1.45		-1.89	-1.89	-0.04	-0.36
0.8	0.4	-0.0196	-0.0132	-0.0132		-0.24	-0.18	-1.39		-1.81	-1.81	-0.04	-0.36
0.6	0.4	-0.0165	-0.0113	-0.0113		-0.20	-0.16	-1.20		-1.55	-1.55	-0.04	-0.36
0.4	0.4	-0.0104	-0.0078	-0.0078		-0.13	-0.11	-0.82		-1.06	-1.06	-0.02	-0.36
0.2	0.4	0.0000	-0.0019	-0.0019		0.00	-0.03	-0.20		-0.23	-0.23	-0.01	-0.36

max negative moment, $M_{uy}(-)$ = -1.89 ft-k/ft
max negative steel req'd, $A_s(-)$ = -0.04 in^2/ft
minimum steel req'd = -0.36 in^2/ft
Use

max positive moment, $M_{uy}(+)$ = 7.91 ft-k/ft
max positive steel req'd, $A_s(+)$ = 0.19 in^2/ft
minimum steel req'd = 0.36 in^2/ft
Use

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BY:	BS	DATE:	Aug-21	CLIENT	City of Wilsonville	SHEET:	
CHKD:		DESCRIPTION:			Sludge Storage and Biofilters Basin	JOB NO:	11962A.00
DESIGN TASK:					Biofilter Basin Dividing Wall along WAS Storage Basin (CSZ)		

Shear Summary

		Loads: q , M , or F				Boundary Case 1				SUMMARY		
a =	10	0.078	0.088	0.680								
b =	12.48	Shear Coefficient Multipliers								Final Shears		
a / b =	0.8013	0.973	1.098	8.486								
		Shear Coefficients				Shears, k/ft				V	V_u	ϕV_c
x / a	y / b	A	B	C	D	A	B	C	D	k/ft	k/ft	k/ft
0	1	0.5081	0.0647	0.0647		0.49	0.07	0.55		1.11	1.11	9.37
0	0.8	0.5607	0.1453	0.1453		0.55	0.16	1.23		1.94	1.94	9.37
0	0.6	0.5511	0.1819	0.1819		0.54	0.20	1.54		2.28	2.28	9.37
0	0.4	0.4592	0.2241	0.2241		0.45	0.25	1.90		2.60	2.60	9.37
0	0.2	0.1324	0.1333	0.1333		0.13	0.15	1.13		1.41	1.41	9.37
0	0.00	-0.0675	-0.0137	-0.0137		-0.07	-0.02	-0.12		-0.20	-0.20	9.37
0.2	0	0.1171	0.1421	0.1421		0.11	0.16	1.21		1.48	1.48	9.37
0.4	0	0.3714	0.2679	0.2679		0.36	0.29	2.27		2.93	2.93	9.37
0.6	0	0.5420	0.3360	0.3360		0.53	0.37	2.85		3.75	3.75	9.37
0.8	0	0.6359	0.3692	0.3692		0.62	0.41	3.13		4.16	4.16	9.37
1	0	0.6655	0.3792	0.3792		0.65	0.42	3.22		4.28	4.28	9.37

Concrete strength reduction factor for shear, ϕ = 1.00

d = 9.0 in

maximum shear, V_u = 4.28 k/ft **OK**

$\phi V_c = \phi*2*(f'c)^{1/2}*b*d$ = (1.00*2*(3000)ˆ½ *12*9.0)/1000 = 11.83 k/ft

Reference:

"Moments and Reactions for Rectangular Plates"
Engineering Monograph No. 27
By: W. T. Moody, United States Bureau of Reclamation

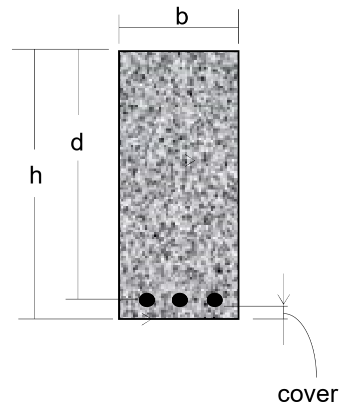
Notes:

Quadratic interpolation is used for intermediate values within the Moody tables and between Moody figures.
The positive sign convention for moments M_x and M_y is tension on the loaded face of the plate.
The M_x moment is in the direction of the x-axis and the M_y moment is in the direction of the y-axis by plate sign convention.

Carollo Engineers...Working Wonders With Water ™

BY:	BS	**DATE:** Aug-21	**CLIENT:** City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**	Sludge Storage and Biofilter Basin	**JOB NO:**	11962A.00
DESIGN TASK:		Biofilter Dividing Wall along WAS Storage Basin (Vertical Reinforcing) (CSZ)			

Concrete Shear and Moment Strength Capacity



Design for ... beam, slab, wall ? **wall**

Properties and Geometry

Compression width of wall, b = **12** inch

Thickness of wall, h = **12** inch

Depth to reinforcing, d = **9** inch

factored design moment, M_u = **7.91** ft-k

factored design shear, V_u = **4.28** kip

f'_c (psi) = **3000**

f_y (psi) = **60000**

ϕ, Bending = **1**

ϕ, Shear = **1**

E_s (psi) = 29000000

E_c (psi) = 3122019

$n = E_s / E_c$ = 9.29

β_1 = 0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi * 2 * b * d * (f'_c)^{1/2}$ = 11.83 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #6@12" & #7@12" alternating for 6" effective spacing**

Area steel provided, A_s = **1.04** in^2 $\rho = A_s / bd$ = 0.00963

$\rho(min)$ < As/bd < $\rho(max)$ - OK

$A_{s\ (min)}$ = 0.19 in^2 $\rho_{(min)}$ = 0.00180

$A_{s\ (max)}$ = 1.73 in^2 $\rho_{(max)}$ = 0.01604

bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

$\phi * M_n$ = 1*0.00963*60*12*9² *(1-0.588*0.00963*60/3)*(ft/12) = 41.498 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

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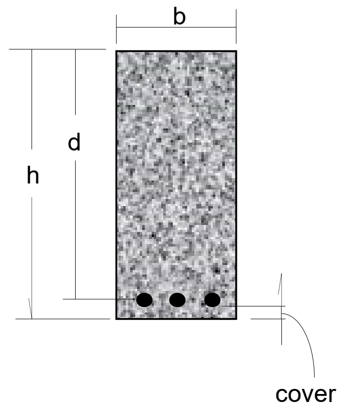
BY:	BS	**DATE:**	Aug-21	**CLIENT:**	City of Wilsonville	**SHEET:**	
CHKD:		**DESCRIPTION:**			Sludge Storage and Biofilter Basin	**JOB NO:**	11962A.00
DESIGN TASK:		Biofilter Dividing Wall along WAS Storage Basin (Horizontal Reinforcing) (CSZ)					

Concrete Shear and Moment Strength Capacity



Design for … beam, slab, wall ? **wall**

Properties and Geometry

Compression width of wall, b =	**12**	inch	f'_c (psi) =	**3000**
Thickness of wall, h =	**12**	inch	f_y (psi) =	**60000**
Depth to reinforcing, d =	**9**	inch	ϕ, Bending =	**1**
factored design moment, M_u =	**5.4**	ft-k	ϕ, Shear =	**1**
factored design shear, V_u =	**1.94**	kip	E_s (psi) =	29000000
			E_c (psi) =	3122019
			$n = E_s / E_c$ =	9.29
			β_1 =	0.85

Nominal Shear Strength (Based on ACI 318-11.3.1.1)

concrete shear strength, $\phi V_c = \phi * 2 * b * d * (f'_c)^{1/2}$ = 11.83 kip ≥ Vu

stirrup spacing, s = **0** in

stirrup U-bar size = **0**

Shear strength ≥ design shear, Okay

Bending Strength (ACI 318 - 10.2.1 thru 10.2.7)

comment : **existing 12" wall w/ #5@12"**

Area steel provided, A_s = **0.31** in^2 $\rho = A_s / bd$ = 0.00287

$\rho(min) < As/bd < \rho(max)$ - OK

$A_{s\ (min)}$ =	0.19	in^2	$\rho_{(min)}$ =	0.00180
$A_{s\ (max)}$ =	1.73	in^2	$\rho_{(max)}$ =	0.01604

bending strength, $$\phi M_n = \phi\, \rho\, f_y\, b\, d^2 \left(1 - \frac{0.588\, \rho\, f_y}{f'_c}\right)$$

$\phi * M_n$ = 1*0.00287*60*12*9² *(1-0.588*0.00287*60/3)*(ft/12) = 13.479 ft-k ≥ Mu

Moment strength ≥ design moment, Okay

ASCE 41-17 Tier 1 Checklists

FIRM:	Carollo Engineers
PROJECT NAME:	City of Wilsonville - Overall Plant Facilities
SEISMICITY LEVEL:	High
PROJECT NUMBER:	11962A.00
COMPLETED BY:	B. Stuetzel
DATE COMPLETED:	08/06/21
REVIEWED BY:	James A. Doering, SE
REVIEW DATE:	08/10/21

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

Table 17-38. Nonstructural Checklist

The Performance Level is designated LS for Life Safety or PR for Position Retention. The level of seismicity is designated as "not required" or by L, M, or H, for Low, Moderate, and High.

All Seismicity Levels For BSE-1E Tier 1, use PR (Position Retention)

Life Safety Systems

RATING				DESCRIPTION	COMMENTS
C ☒	NC ☐	N/A ☐	U ☐	LS-LMH; PR-LMH. FIRE SUPPRESSION PIPING: Fire suppression piping is anchored and braced in accordance with NFPA-13. (Commentary: Sec. A.7.13.1. Tier 2: Sec. 13.7.4)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Fire suppression piping has flexible couplings in accordance with NFPA-13. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.4)	
C ☒	NC ☐	N/A ☐	U ☐	LS-LMH; PR-LMH. EMERGENCY POWER: Equipment used to power or control life safety systems is anchored or braced. (Commentary: Sec. A.7.12.1. Tier 2: Sec. 13.7.7)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. STAIR AND SMOKE DUCTS: Stair pressurization and smoke control ducts are braced and have flexible connections at seismic joints. (Commentary: Sec. A.7.14.1. Tier 2: Sec. 13.7.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-MH; PR-MH. SPRINKLER CEILING CLEARANCE: Penetrations through panelized ceilings for fire suppression devices provide clearances in accordance with NFPA-13. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.4)	
☒	☐	☐	☐	LS-not required; PR-LMH. EMERGENCY LIGHTING: Emergency and egress lighting equipment is anchored or braced. (Commentary: Sec. A.7.3.1. Tier 2: Sec. 13.7.9)	

Hazardous Materials

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-LMH; PR-LMH. HAZARDOUS MATERIAL EQUIPMENT: Equipment mounted on vibration isolators and containing hazardous material is equipped with restraints or snubbers. (Commentary: Sec. A.7.12.2. Tier 2: 13.7.1)	
☐	☐	☒	☐	LS-LMH; PR-LMH. HAZARDOUS MATERIAL STORAGE: Breakable containers that hold hazardous material, including gas cylinders, are restrained by latched doors, shelf lips, wires, or other methods. (Commentary: Sec. A.7.15.1. Tier 2: Sec. 13.8.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

C	NC	N/A	U	Item	Comments
X				LS-MH; PR-MH. HAZARDOUS MATERIAL DISTRIBUTION: Piping or ductwork conveying hazardous materials is braced or otherwise protected from damage that would allow hazardous material release. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)	
X				LS-MH; PR-MH. SHUT-OFF VALVES: Piping containing hazardous material, including natural gas, has shut-off valves or other devices to limit spills or leaks. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.3 and 13.7.5)	
			X	LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Hazardous material ductwork and piping, including natural gas piping, has flexible couplings. (Commentary: Sec. A.7.15.4, Tier 2: Sec.13.7.3 and 13.7.5)	Within the structures, there doesn't appear to be any flexible couplings. The natural gas piping does carry between the buildings underground, so it is possible for flexible couplings to be buried.
		X		LS-MH; PR-MH. PIPING OR DUCTS CROSSING SEISMIC JOINTS: Piping or ductwork carrying hazardous material that either crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.13.6. Tier 2: Sec.13.7.3, 13.7.5, and 13.7.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

Partitions

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. UNREINFORCED MASONRY: Unreinforced masonry or hollow-clay tile partitions are braced at a spacing of at most 10 ft in Low or Moderate Seismicity, or at most 6 ft in High Seismicity. (Commentary: Sec. A.7.1.1. Tier 2: Sec. 13.6.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. HEAVY PARTITIONS SUPPORTED BY CEILINGS: The tops of masonry or hollow-clay tile partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. DRIFT: Rigid cementitious partitions are detailed to accommodate the following drift ratios: in steel moment frame, concrete moment frame, and wood frame buildings, 0.02; in other buildings, 0.005. (Commentary A.7.1.2 Tier 2: Sec. 13.6.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-MH. LIGHT PARTITIONS SUPPORTED BY CEILINGS: The tops of gypsum board partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-not required; PR-MH. STRUCTURAL SEPARATIONS: Partitions that cross structural separations have seismic or control joints. (Commentary: Sec. A.7.1.3. Tier 2. Sec. 13.6.2)	
☐	☐	☒	☐	LS-not required; PR-MH. TOPS: The tops of ceiling-high framed or panelized partitions have lateral bracing to the structure at a spacing equal to or less than 6 ft. (Commentary: Sec. A.7.1.4. Tier 2. Sec. 13.6.2)	

Ceilings

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-LMH. SUSPENDED LATH AND PLASTER: Suspended lath and plaster ceilings have attachments that resist seismic forces for every 12 ft^2 of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-LMH. SUSPENDED GYPSUM BOARD: Suspended gypsum board ceilings have attachments that resist seismic forces for every 12 ft^2 of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-MH. INTEGRATED CEILINGS: Integrated suspended ceilings with continuous areas greater than 144 ft^2, and ceilings of smaller areas that are not surrounded by restraining partitions, are laterally restrained at a spacing no greater than 12 ft with members attached to the structure above. Each restraint location has a minimum of four diagonal wires and compression struts, or diagonal members capable of resisting compression. (Commentary: Sec. A.7.2.2. Tier 2: Sec. 13.6.4)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-MH. EDGE CLEARANCE: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 have clearances from the enclosing wall or partition of at least the following: in Moderate Seismicity, 1/2 in.; in High Seismicity, 3/4 in. (Commentary: Sec. A.7.2.4. Tier 2: Sec. 13.6.4)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-MH. CONTINUITY ACROSS STRUCTURE JOINTS: The ceiling system does not cross any seismic joint and is not attached to multiple independent structures. (Commentary: Sec. A.7.2.5. Tier 2: Sec. 13.6.4)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-H. EDGE SUPPORT: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 are supported by closure angles or channels not less than 2 in. wide. (Commentary: Sec. A.7.2.6. Tier 2: Sec. 13.6.4)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-not required; PR-H. SEISMIC JOINTS: Acoustical tile or lay-in panel ceilings have seismic separation joints such that each continuous portion of the ceiling is no more than 2500 ft^2 and has a ratio of long-to-short dimension no more than 4-to-1. (Commentary: Sec. A.7.2.7. Tier 2: 13.6.4)	

Light Fixtures

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-MH; PR-MH. INDEPENDENT SUPPORT: Light fixtures that weigh more per square foot than the ceiling they penetrate are supported independent of the grid ceiling suspension system by a minimum of two wires at diagonally opposite corners of each fixture. (Commentary: Sec. A.7.3.2. Tier 2: Sec. 13.6.4 and 13.7.9)	
☒	☐	☐	☐	LS-not required; PR-H PENDANT SUPPORTS: Light fixtures on pendant supports are attached at a spacing equal to or less than 6 ft. Unbraced suspended fixtures are free to allow a 360-degree range of motion at an angle not less than 45 degrees from horizontal without contacting adjacent components. Alternatively, if rigid supported and/or braced, they are free to move with the structure to which they are attached without damaging adjoining components. Additionally, the connection to the structure is capable of accommodating the movement without failure. (Commentary: Sec. A.7.3.3. Tier 2: Sec. 13.7.9)	
☒	☐	☐	☐	LS-not required; PR-H. LENS COVERS: Lens covers on light fixtures are attached with safety devices. (Commentary: Sec. A.7.3.4. Tier 2: Sec. 13.7.9)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

Cladding and Glazing

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. CLADDING ANCHORS: Cladding components weighing more than 10 lb/ft^2 are mechanically anchored to the structure at a spacing equal to or less than the following: for Life Safety in Moderate Seismicity, 6 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 ft. (Commentary: Sec. A.7.4.1. Tier 2: Sec. 13.6.1)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH CLADDING ISOLATION: For steel or concrete moment-frame buildings, panel connections are detailed to accommodate a story drift ratio by the use of rods attached to framing with oversize holes or slotted holes of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02, and the rods have a length-to-diameter ratio of 4.0 or less. (Commentary: Sec. A.7.4.3. Tier 2: Sec. 13.6.1)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH MULTI-STORY PANELS: For multi-story panels attahed at more than one floor level panel connections are detailed to accommodate a story drift ratio by the use of rods attached to framing with oversize holes or slotted holes of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02, and the rods have a length-to-diameter ratio of 4.0 or less. (Commentary: Sec. A.7.4.4. Tier 2: Sec. 13.6.1)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH THREADED RODS: Threaded rods for panel connections detailed to accommodate drift by bending of the rod have a length-to-diameter ratio greater than 0.06 times the story height in inches for Life Safety in Moderate Seismicity and 0.12 times the story height in inches for Life Safety in High Seismicity and for Position Retention in any seismicity. (Commentary: Sec. A.7.4.9. Tier 2: Sec. 13.6.1)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name City of Wilsonville
Project Number 11962A.00

C	NC	N/A	U		
☐	☐	☒	☐	LS-MH; PR-MH. PANEL CONNECTIONS: Cladding panels are anchored out-of-plane with a minimum number of connections for each wall panel, as follows: for Life Safety in Moderate Seismicity, 2 connections; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 connections. (Commentary: Sec. A.7.4.5. Tier 2: Sec. 13.6.1.4)	

Rating				Description	Comments
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. BEARING CONNECTIONS: Where bearing connections are used, there is a minimum of two bearing connections for each cladding panel. (Commentary: Sec. A.7.4.6. Tier 2: Sec. 13.6.1.4)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. INSERTS: Where concrete cladding components use inserts, the inserts have positive anchorage or are anchored to reinforcing steel. (Commentary: Sec. A.7.4.7. Tier 2: Sec. 13.6.1.4)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. OVERHEAD GLAZING: Glazing panes of any size in curtain walls and individual interior or exterior panes over 16 ft^2 in area are laminated annealed or laminated heat-strengthened glass and are detailed to remain in the frame when cracked. (Commentary: Sec. A.7.4.8: Tier 2: Sec. 13.6.1.5)	Windows are less than 16 ft2 requirement.

Masonry Veneer

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. TIES: Masonry veneer is connected to the backup with corrosion-resistant ties. There is a minimum of one tie for every 2-2/3 ft^2, and the ties have spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 36 in.; for Life Safety in High Seismicity and for Position Retention in any seismicity, 24 in. (Commentary: Sec. A.7.5.1. Tier 2: Sec. 13.6.1.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

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C	NC	N/A	U	Statement	Comments
☐	☐	☒	☐	LS-LMH; PR-LMH. SHELF ANGLES: Masonry veneer is supported by shelf angles or other elements at each floor above the ground floor. (Commentary: Sec. A.7.5.2. Tier 2: Sec. 13.6.1.2)	
☐	☐	☒	☐	LS-LMH; PR-LMH. WEAKENED PLANES: Masonry veneer is anchored to the backup adjacent to weakened planes, such as at the locations of flashing. (Commentary: Sec. A.7.5.3. Tier 2: Sec. 13.6.1.2)	
☐	☐	☒	☐	LS-LMH; PR-LMH. UNREINFORCED MASONRY BACKUP: There is no unreinforced masonry backup. (Commentary: Sec. A.7.7.2. Tier 2: Section 13.6.1.1 and 13.6.1.2)	
☐	☐	☒	☐	LS-MH; PR-MH STUD TRACKS: For veneer with cold-formed steel stud backup, stud tracks are fastened to the structure at a spacing equal to or less than 24 in. on center. (Commentary: Sec. A.7.6.1. Tier 2: Sec. 13.6.1.1 and 13.6.1.2)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-MH. ANCHORAGE: For veneer with concrete block or masonry backup, the backup is positively anchored to the structure at a horizontal spacing equal to or less than 4 ft along the floors and roof. (Commentary: Sec. A.7.7.1. Tier 2: Section 13.6.1.1 and 13.6.1.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-MH. WEEP HOLES: In veneer anchored to stud walls, the veneer has functioning weep holes and base flashing. (Commentary: Sec. A.7.5.6. Tier 2: Section 13.6.1.2)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-MH OPENINGS: For veneer with cold-formed -steel stud backup, steel studs frame window and door openings. (Commentary: Sec. A.7.6.2. Tier 2: Sec. 13.6.1.1 and 13.6.1.2)	

Parapets, Cornices, Ornamentation, and Appendages

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. URM PARAPETS OR CORNICES: Laterally unsupported unreinforced masonry parapets or cornices have height-to-thickness ratios no greater than the following: for Life Safety in Low or Moderate Seismicity, 2.5; for Life Safety in High Seismicity and for Position Retention in any seismicity, 1.5. (Commentary: Sec. A.7.8.1. Tier 2: Sec. 13.6.5)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Rating				Description	Comments
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. CANOPIES: Canopies at building exits are anchored to the structure at a spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 10 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 6 ft. (Commentary: Sec. A.7.8.2. Tier 2: Sec. 13.6.6)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-LMH. CONCRETE PARAPETS: Concrete parapets with height-to-thickness ratios greater than 2.5 have vertical reinforcement. (Commentary: Sec. A.7.8.3. Tier 2: Sec. 13.6.5)	
C ☐	NC ☐	N/A ☒	U ☐	LS-MH; PR-LMH. APPENDAGES: Cornices, parapets, signs, and other ornamentation or appendages that extend above the highest point of anchorage to the structure or cantilever from components are reinforced and anchored to the structural system at a spacing equal to or less than 6 ft. This checklist item does not apply to parapets or cornices covered by other checklist items. (Commentary: Sec. A.7.8.4. Tier 2: Sec. 13.6.6)	

Masonry Chimneys

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-LMH; PR-LMH. URM CHIMNEYS: Unreinforced masonry chimneys extend above the roof surface no more than the following: for Life Safety in Low or Moderate Seismicity, 3 times the least dimension of the chimney; for Life Safety in High Seismicity and for Position Retention in any seismicity, 2 times the least dimension of the chimney. (Commentary: Sec. A.7.9.1. Tier 2: 13.6.7)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

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C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-LMH; PR-LMH. ANCHORAGE: Masonry chimneys are anchored at each floor level, at the topmost ceiling level, and at the roof. (Commentary: Sec. A.7.9.2. Tier 2: 13.6.7)	

Stairs

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-LMH; PR-LMH. STAIR ENCLOSURES: Hollow-clay tile or unreinforced masonry walls around stair enclosures are restrained out-of-plane and have height-to-thickness ratios not greater than the following: for Life Safety in Low or Moderate Seismicity, 15-to-1; for Life Safety in High Seismicity and for Position Retention in any seismicity, 12-to-1. (Commentary: Sec. A.7.10.1. Tier 2: Sec. 13.6.2 and 13.6.8)	
☐	☐	☒	☐	LS-LMH; PR-LMH STAIR DETAILS: The connection between the stairs and the structure does not rely on post-installed anchors in concrete or masonry, and the stair details are capable of accommodating the drift calculated using the Quick Check procedure of Section 4.4.3.1 for moment-frame structures or 0.5 in. for all other structures without including any lateral stiffness contribution from the stairs. (Commentary: Sec. A.7.10.2. Tier 2: Sec. 13.6.8)	

Contents and Furnishings

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☐	☐	☒	☐	LS-MH; PR-MH. INDUSTRIAL STORAGE RACKS: Industrial storage racks or pallet racks more than 12 ft high meet the requirements of ANSI/MH 16.1 as modified by ASCE 7 Chapter 15. (Commentary: Sec. A.7.11.1. Tier 2: Sec. 13.8.1)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

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C	NC	N/A	U	Statement	Comments
☐	☒	☐	☐	LS-H; PR-MH. TALL NARROW CONTENTS: Contents more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 are anchored to the structure or to each other. (Commentary: Sec. A.7.11.2. Tier 2: Sec. 13.8.2)	Storage racks within the Headworks building appear to be unanchored to structure.
☒	☐	☐	☐	LS-H; PR-H. FALL-PRONE CONTENTS: Equipment, stored items, or other contents weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level are braced or otherwise restrained. (Commentary: Sec. A.7.11.3. Tier 2: Sec. 13.8.2)	
☐	☐	☒	☐	LS-not required; PR-MH. ACCESS FLOORS: Access floors more than 9 in. high are braced. (Commentary: Sec. A.7.11.4. Tier 2: Sec. 13.8.3)	
☐	☐	☒	☐	LS-not required; PR-MH. EQUIPMENT ON ACCESS FLOORS: Equipment and other contents supported by access floor systems are anchored or braced to the structure independent of the access floor. (Commentary: Sec. A.7.11.5. Tier 2: Sec. 13.7.7 and 13.8.3)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Storage racks within the Headworks building appear to be unanchored to structure.

C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	LS-not required; PR-H. SUSPENDED CONTENTS: Items suspended without lateral bracing are free to swing from or move with the structure from which they are suspended without damaging themselves or adjoining components. (Commentary. A.7.11.6. Tier 2: Sec. 13.8.2)	

Mechanical and Electrical Equipment

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	LS-H; PR-H. FALL-PRONE EQUIPMENT: Equipment weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level, and which is not in-line equipment, is braced. (Commentary: A.7.12.4. Tier 2: 13.7.1 and 13.7.7)	
☐	☒	☐	☐	LS-H; PR-H. IN-LINE EQUIPMENT: Equipment installed in-line with a duct or piping system, with an operating weight more than 75 lb, is supported and laterally braced independent of the duct or piping system. (Commentary: Sec. A.7.12.5. Tier 2: Sec. 13.7.1)	Recirculation pump at the Disk Filters is anchored to plate, but there doesn't seem to be any plate resistance to overturning. See next page for photo showing condition.
☒	☐	☐	☐	LS-H; PR-MH. TALL NARROW EQUIPMENT: Equipment more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 is anchored to the floor slab or adjacent structural walls. (Commentary: Sec. A.7.12.6. Tier 2: Sec. 13.7.1 and 13.7.7)	

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Recirculation pump is anchored to plate below, but there doesn't appear to be any support for equipment overturning.

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C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-MH. MECHANICAL DOORS: Mechanically operated doors are detailed to operate at a story drift ratio of 0.01. (Commentary: Sec. A.7.12.7. Tier 2: Sec. 13.6.9)	
C ☒	NC ☐	N/A ☐	U ☐	LS-not required; PR-H. SUSPENDED EQUIPMENT: Equipment suspended without lateral bracing is free to swing from or move with the structure from which it is suspended without damaging itself or adjoining components. (Commentary: Sec. A.7.12.8. Tier 2: Sec. 13.7.1 and 13.7.7)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-H. VIBRATION ISOLATORS: Equipment mounted on vibration isolators is equipped with horizontal restraints or snubbers and with vertical restraints to resist overturning. (Commentary: Sec. A.7.12.9. Tier 2: Sec. 13.7.1)	
C ☐	NC ☒	N/A ☐	U ☐	LS-not required; PR-H. HEAVY EQUIPMENT: Floor-supported or platform-supported equipment weighing more than 400 lb is anchored to the structure. (Commentary: Sec. A.7.12.10. Tier 2: 13.7.1 and 13.7.7)	(2) ACCU units located to the west of aeration basins lack to structural pad below.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

ACCU units lack anchorage to structural pad near Aeration Basins.

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C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	LS-not required; PR-H. ELECTRICAL EQUIPMENT: Electrical equipment is laterally braced to the structure. (Commentary: Sec. A.7.12.11. Tier 2: 13.7.7)	
☐	☐	☒	☐	LS-not required; PR-H. CONDUIT COUPLINGS: Conduit greater than 2.5 in. trade size that is attached to panels, cabinets, or other equipment and is subject to relative seismic displacement has flexible couplings or connections. (Commentary: Sec. A.7.12.12. Tier 2: 13.7.8)	

Piping

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	LS-not required; PR-H. FLEXIBLE COUPLINGS: Fluid and gas piping has flexible couplings. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.3 and 13.7.5)	
☒	☐	☐	☐	LS-not required; PR-H. FLUID AND GAS PIPING: Fluid and gas piping is anchored and braced to the structure to limit spills or leaks. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	LS-not required; PR-H. C-CLAMPS: One-sided C-clamps that support piping larger than 2.5 in. in diameter are restrained. (Commentary: Sec. A.7.13.5. Tier 2: Sec. 13.7.3 and 13.7.5)	
☐	☐	☒	☐	LS-not required; PR-H. PIPING CROSSING SEISMIC JOINTS: Piping that crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A7.13.6. Tier 2: Sec.13.7.3 and Sec. 13.7.5)	

Ducts

RATING				DESCRIPTION	COMMENTS
C	NC	N/A	U		
☒	☐	☐	☐	LS-not required; PR-H. DUCT BRACING: Rectangular ductwork larger than 6 ft^2 in cross-sectional area and round ducts larger than 28 in. in diameter are braced. The maximum spacing of transverse bracing does not exceed 30 ft. The maximum spacing of longitudinal bracing does not exceed 60 ft. (Commentary: Sec. A.7.14.2. Tier 2: Sec. 13.7.6)	
☒	☐	☐	☐	LS-not required; PR-H. DUCT SUPPORT: Ducts are not supported by piping or electrical conduit. (Commentary: Sec. A.7.14.3. Tier 2: Sec. 13.7.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-H. DUCTS CROSSING SEISMIC JOINTS: Ducts that cross seismic joints or isolation planes or are connected to independent structures have couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.14.5. Tier 2: Sec. 13.7.6)	

Elevators

RATING				DESCRIPTION	COMMENTS
C ☐	NC ☐	N/A ☒	U ☐	LS-H; PR-H. RETAINER GUARDS: Sheaves and drums have cable retainer guards. (Commentary: Sec. A.7.16.1. Tier 2: 13.8.6)	
C ☐	NC ☐	N/A ☒	U ☐	LS-H; PR-H. RETAINER PLATE: A retainer plate is present at the top and bottom of both car and counterweight. (Commentary: Sec. A.7.16.2. Tier 2: 13.8.6)	
C ☐	NC ☐	N/A ☒	U ☐	LS-not required; PR-H. ELEVATOR EQUIPMENT: Equipment, piping, and other components that are part of the elevator system are anchored. (Commentary: Sec. A.7.16.3. Tier 2: 13.8.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U	Description	Comment
☐	☐	☒	☐	LS-not required; PR-H. SEISMIC SWITCH: Elevators capable of operating at speeds of 150 ft/min or faster are equipped with seismic switches that meet the requirements of ASME A17.1 or have trigger levels set to 20% of the acceleration of gravity at the base of the structure and 50% of the acceleration of gravity in other locations. (Commentary: Sec. A.7.16.4. Tier 2: 13.8.6)	
☐	☐	☒	☐	LS-not required; PR-H. SHAFT WALLS: Elevator shaft walls are anchored and reinforced to prevent toppling into the shaft during strong shaking. (Commentary: Sec. A.7.16.5. Tier 2: 13.8.6)	
☐	☐	☒	☐	LS-not required; PR-H. COUNTERWEIGHT RAILS: All counterweight rails and divider beams are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.6. Tier 2: 13.8.6)	
☐	☐	☒	☐	LS-not required; PR-H. BRACKETS: The brackets that tie the car rails and the counterweight rail to the structure are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.7. Tier 2: 13.8.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

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C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. SPREADER BRACKET: Spreader brackets are not used to resist seismic forces. (Commentary: Sec. A.7.16.8. Tier 2: 13.8.6)	
C	NC	N/A	U		
☐	☐	☒	☐	LS-not required; PR-H. GO-SLOW ELEVATORS: The building has a go-slow elevator system. (Commentary: Sec. A.7.16.9. Tier 2: 13.8.6)	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

City of Wilsonville

Tier 2 Structural Calculations

OPERATIONS BUILDING - TIER 2 CALCULATIONS

SEISMIC BASE SHEAR FOR OPERATIONS BUILDING

7.4.1.3.1 Pseudo Seismic Force for LSP. The pseudo lateral force in a given horizontal direction of a building shall be determined using Eq. (7-21). This force shall be used to evaluate or retrofit the vertical elements of the seismic-force-resisting system.

$$V = C_1 C_2 C_m S_a W \qquad (7\text{-}21)$$

Table 7-3. Alternate Values for Modification Factors C_1C_2

Fundamental Period	$m_{max} < 2$	$2 \le m_{max} < 6$	$m_{max} \ge 6$
$T \le 0.3$	1.1	1.4	1.8
$0.3 < T \le 1.0$	1.0	1.1	1.2
$T > 1.0$	1.0	1.0	1.1

Table 7-4. Values for Effective Mass Factor C_m

No. of Stories	Concrete Moment Frame	Concrete Shear Wall	Concrete Pier-Spandrel	Steel Moment Frame	Steel Concentrically Braced Frame	Steel Eccentrically Braced Frame	Other
1–2	1.0	1.0	1.0	1.0	1.0	1.0	1.0
3 or more	0.9	0.8	0.8	0.9	0.9	0.9	1.0

Note: C_m shall be taken as 1.0 if the fundamental period, T, in the direction of response under consideration is greater than 1.0 s.

spectral response acceleration, S_{xs} =	0.744	g	(BSE-2E seismic hazard)
spectral response acceleration, S_{x1} =	0.405	g	(BSE-2E seismic hazard)
building period, T =	0.114	s	
response spectrum acceleration, S_a =	0.744	g	
effective seismic weight, W =	190.9	kip	
C_1C_2 =	1.4		(Table 11-6 for masonry walls, m=2.5)
effective mass factor, C_m =	1.0		
seismic lateral force, V =	198.8	kip	

FILE LOCATION (CVO): CVO1\PRJ1\PRJ\WILSONDAT\ wl98s401.dlv

GENERAL STRUCTURAL NOTES

GENERAL

1. ALL MATERIALS AND WORKMANSHIP SHALL CONFORM TO THE UNIFORM BUILDING CODE, 1994 EDITION (U.B.C.) AS AMENDED BY THE STATE OF OREGON.
2. LOADS: ROOF SNOW LOAD = 25 PSF PLUS DRIFTING
   UBC WIND PRESSURE = 80 MPH WIND SPEED EXPOSURE C, I=1.0
   UBC SEISMIC ZONE = 3, I=1.0, S=1.5
3. REFER TO INDIVIDUAL STRUCTURE DRAWINGS FOR ADDITIONAL LOADS, NOTES, AND REQUIREMENTS.
4. NET ALLOWABLE SOIL BEARING PRESSURE = 4000 PSF
5. DRAINED EQUIVALENT FLUID PRESSURE = 85 PCF/FT AT REST
   = 35 PCF/FT ACTIVE
6. UNDRAINED EQUIVALENT FLUID PRESSURE = 100 PCF/FT AT REST
   = 85 PCF/FT ACTIVE
7. DATUM: SEE SITE DRAWINGS.
8. NO STRUCTURAL MEMBERS SHALL BE CUT FOR PIPES, DUCTS, ETC. UNLESS SPECIFICALLY DETAILED OR APPROVED IN WRITING BY THE ENGINEER.
9. PROVISIONS FOR FUTURE EXPANSION: NO INTERNAL EXPANSION DETAILED.
10. STANDARD DETAILS AS SHOWN ON THE DRAWINGS ARE INTENDED TO BE TYPICAL AND SHALL APPLY TO ALL SIMILAR SITUATIONS OCCURRING ON THE PROJECT, WHETHER OR NOT THEY ARE KEYED IN EACH LOCATION. CONSULT THE ENGINEER FOR REVIEW PRIOR TO CONSTRUCTION.
11. VISITS TO THE JOB SITE BY THE ENGINEER TO OBSERVE THE CONSTRUCTION DO NOT IN ANY WAY MEAN THAT THEY ARE GUARANTORS OF THE CONSTRUCTOR'S WORK, NOR RESPONSIBLE FOR COMPREHENSIVE OR SPECIAL INSPECTIONS, COORDINATION, SUPERVISION, NOR SAFETY AT THE JOB SITE.
12. SPECIAL INSPECTION (OWNER FURNISHED) IS REQUIRED IN ACCORDANCE WITH UBC SECTION 306 ON THE FOLLOWING PORTIONS OF THE WORK:
    CONCRETE PLACEMENT
    REINFORCING STEEL PLACEMENT
    STRUCTURAL WELDING
    ANCHORS, EMBEDS AND BOLTS INSTALLED IN CONCRETE
    HIGH STRENGTH BOLTS
    GRADING, EXCAVATION, AND FILLING
    MASONRY CONSTRUCTION WHEN INDICATED
13. ALL SPECIFIED CONCRETE AND GROUT TESTING DURING CONSTRUCTION WILL BE OWNER FURNISHED. ALL SPECIFIED LABORATORY TEST MIXES ARE THE RESPONSIBILITY OF THE CONTRACTOR.
14. VERIFY ALL OPENING DIMENSIONS IN WALLS, SLABS, AND DECKS WITH THE ARCHITECTURAL, MECHANICAL, AND ELECTRICAL DRAWINGS.
15. FOR ABBREVIATIONS NOT LISTED, SEE "ABBREVIATIONS FOR USE ON DRAWINGS AND TEXT", PUBLISHED BY THE AMERICAN NATIONAL STANDARDS INSTITUTE INC. (ANSI).

FOUNDATIONS:

1. PROVIDE AND INSTALL MINIMUM 6 INCHES COMPACTED GRANULAR FILL AS SPECIFIED UNDER ALL SLABS AND FOOTINGS TO UNDISTURBED EARTH.
2. NO BACKFILL SHALL BE PLACED BEHIND WALLS UNTIL THE CONCRETE HAS ATTAINED 100% OF ITS SPECIFIED COMPRESSIVE STRENGTH.
3. WALLS TIED TO ELEVATED FLOOR OR ROOF SLABS SHALL BE BRACED AND REMOVAL OF BRACING FOLLOWED BY BACKFILLING SHALL NOT BE ALLOWED UNTIL THE SLAB IS COMPLETE AND HAS ATTAINED 80% OF ITS SPECIFIED COMPRESSIVE STRENGTH.
4. EXCAVATIONS SHALL BE SHORED AS REQUIRED TO PREVENT SUBSIDENCE OR DAMAGE TO ADJACENT EXISTING STRUCTURES, STREETS, UTILITIES, ETC.
5. ALL SOIL BEARING SURFACES SHALL BE INSPECTED BY THE SOILS ENGINEER PRIOR TO PLACEMENT OF REINFORCING STEEL.
5. THE AERATION BASINS AND ATTACHED PROCESS GALLERY BUILDING AND THE TWO SECONDARY CLARIFIERS SHALL HAVE AN UNDERDRAIN SYSTEM. REFER TO DRAWINGS 05-CY-01 AND 05-CY-02.

CONCRETE:

1. ALL CAST-IN-PLACE CONCRETE SHALL HAVE A MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 4000 PSI.
2. REINFORCING STEEL SHALL CONFORM TO ASTM A615, GRADE 60. REINFORCING TO BE WELDED SHALL CONFORM TO ASTM A706, GRADE 60. FABRICATION AND PLACEMENT OF REINFORCING STEEL SHALL BE IN ACCORDANCE WITH CRSI MSP-1 "MANUAL OF STANDARD PRACTICE" AND ACI 301 "SPECIFICATIONS FOR STRUCTURAL CONCRETE FOR BUILDING".
3. CONSTRUCTION JOINTS INDICATED ARE SUGGESTED LOCATIONS. CONTRACTOR MAY REVISE LOCATION OF JOINTS, SUBJECT TO SPECIFIED REQUIREMENTS, AND SHALL SUBMIT ALL JOINT LOCATIONS FOR REVIEW BY THE ENGINEER. ADDITIONAL CONSTRUCTION JOINT LOCATIONS, AS REQUIRED FOR CONSTRUCTION, SHALL BE SUBMITTED FOR REVIEW.
4. CONTINUOUS GALVANIZED STEEL WATERSTOP AS SPECIFIED SHALL BE INSTALLED IN ALL CONSTRUCTION JOINTS IN WALLS OF WATER HOLDING BASINS AND CHANNELS, EXCEPT WHERE INDICATED OTHERWISE. AT CONTRACTOR'S OPTION, PLASTIC WATERSTOPS MAY BE USED IN PLACE OF GALVANIZED STEEL WATERSTOPS.
5. ROUGHEN AND CLEAN ALL CONSTRUCTION JOINTS IN WALLS AND SLABS AS SPECIFIED PRIOR TO PLACING ADJACENT CONCRETE. SANDBLASTING OR OTHER PREPARATION OF HORIZONTAL AND VERTICAL JOINTS IS REQUIRED AS SPECIFIED.
6. THE CONTRACTOR SHALL COORDINATE PLACEMENT OF ALL OPENINGS, CURBS, DOWELS, SLEEVES, CONDUITS, BOLTS AND INSERTS PRIOR TO PLACEMENT OF CONCRETE.
7. NO ALUMINUM CONDUIT OR PRODUCTS CONTAINING ALUMINUM OR ANY OTHER MATERIAL INJURIOUS TO THE CONCRETE SHALL BE EMBEDDED IN THE CONCRETE.

MASONRY

1. MORTAR SHALL CONFORM TO ASTM C270, TYPE S, HYDRATED. MASONRY CEMENT SHALL NOT BE USED.
2. GROUT SHALL CONFORM TO ASTM C476 COURSE GROUT AND SHALL HAVE A MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 2000 PSI.
3. ALL CONCRETE MASONRY UNITS SHALL BE GRADE N, TYPE I AND HAVE A UNIT COMPRESSIVE STRESS OF 1900 PSI, AND SHALL CONFORM TO ASTM C90.
4. THE DESIGN f'm OF THE FINISHED ASSEMBLY SHALL BE 1500 PSI.
5. ALL CELLS IN BUILDING WALLS SHALL BE PARTIALLY GROUTED EXCEPT WHERE INDICATED OTHERWISE.
6. LAP ALL BARS 48 BAR DIAMETERS MINIMUM. STAGGER ALL ADJACENT LAP SPLICES SEPARATED BY 3 INCHES OR LESS. 24 INCHES.
7. PROVIDE FULL HEIGHT VERTICAL BARS AT EDGES OF ALL OPENINGS AND FULL HEIGHT VERTICAL BARS IN 3 CELLS AT ALL CORNERS. PROVIDE MATCHING DOWELS FOR ALL VERTICAL BARS. PROVIDE REINFORCED LINTELS ABOVE AND REINFORCED BOND BEAMS BELOW ALL OPENINGS. PROVIDE HORIZONTAL CORNER BARS WITH MINIMUM 2'-6" LEGS AT ALL CORNERS. SEE DETAILS 4001, 4003, AND 4004.
8. MASONRY UNIT AND GROUT TESTING SHALL BE IN CONFORMANCE WITH 1991 UBC 2405(C)3, "UNIT STRENGTH METHOD". TESTING WILL BE OWNER FURNISHED.
9. THE MINIMUM REINFORCING FOR ALL CONCRETE BLOCK WALLS SHALL BE AS FOLLOWS:

WALL THICKNESS	VERTICAL REINFORCING	HORIZONTAL REINFORCING	LOCATION
8"	#6@32"	#5@48"	CENTERED

PROVIDE LARGER SIZES AND MORE REINFORCING IN ALL WALLS WHERE REQUIRED BY THE DETAILS ON THE DRAWINGS OR BY THE SPECIFICATIONS.

STRUCTURAL STEEL:

1. ALL STRUCTURAL STEEL SHALL CONFORM TO ASTM A-36 UNLESS SHOWN OTHERWISE. SQUARE OR RECTANGULAR STEEL TUBING SHALL CONFORM TO ASTM A-500, GRADE B.
2. ALL STRUCTURAL STEEL SHALL BE FABRICATED AND ERECTED IN CONFORMANCE WITH THE AISC MANUAL OF STEEL CONSTRUCTION, CURRENT EDITION.
3. ALL BOLTS SHALL BE HIGH STRENGTH BOLTS CONFORMING TO ASTM A325-N OR A325-SC UNLESS OTHERWISE SHOWN. BOLTS INDICATED AS MACHINE BOLTS (MB) OR ANCHOR BOLTS (AB) SHALL CONFORM TO ASTM A307 FOR CARBON STEEL, A193 FOR STAINLESS STEEL, AND A153 FOR GALVANIZED STEEL EXCEPT WHERE SPECIFICALLY INDICATED OTHERWISE. ALL JOINT CONTACT SURFACES SHALL BE CLEAN AND FREE FROM OIL, DIRT AND PAINT.
4. ALL WELDS SHALL BE DONE BY AWS CERTIFIED WELDERS AND SHALL CONFORM TO AWS D 1.1, LATEST EDITION. ALL BUTT WELDS ARE FULL PENETRATION UNLESS INDICATED OTHERWISE. WELD FILLER METAL SHALL BE AWS A5.1 OR A5.5 E70XX ELECTRODES.
5. ALL WELDS FOUND DEFECTIVE SHALL BE REPAIRED AND/OR REPLACED AND RETESTED FOR ADEQUACY AT THE CONTRACTOR'S EXPENSE.
6. AT ALL FIELD WELDS, AT EMBED PLATES, AND ANGLES, LOW HEAT AND INTERMITTENT WELDS SHALL BE UTILIZED TO AVOID SPALLING OR CRACKING THE EXISTING CONCRETE.
7. ALL STRUCTURAL STEEL TO BE EMBEDDED IN CONCRETE SHALL BE CLEAN AND FREE OF PAINT, OIL OR DIRT.
8. NO HOLES OTHER THAN THOSE SPECIFICALLY DETAILED SHALL BE ALLOWED THROUGH STRUCTURAL STEEL MEMBERS. NO CUTTING OR BURNING OF STRUCTURAL STEEL WILL BE PERMITTED WITHOUT THE APPROVAL OF THE ENGINEER.

METAL DECKING:

1. SEE ROOF AND ELEVATED FLOOR PLANS FOR DECK SIZE AND WELDING REQUIREMENTS.
2. WELDING SHALL BE IN ACCORDANCE WITH AWS D1.3 "STRUCTURAL WELDING CODE - SHEET STEEL". WELDING FILLER METAL SHALL BE AWS A5.1 OR A5.5 E70XX ELECTRODES. WELDERS SHALL BE AWS CERTIFIED.
3. DECKING SHALL HAVE MINIMUM 2" BEARING ON ALL SUPPORTS.

STEEL JOISTS

1. SEE ROOF PLANS FOR DESIGN LOAD REQUIREMENTS, MINIMUM SIZE, SPACING, AND BRIDGING REQUIREMENTS.
2. MANUFACTURER SHALL BE A MEMBER OF THE STEEL JOIST INSTITUTE (SJI).

REINFORCING NOTES:

1. THE MINIMUM REINFORCING FOR ALL CONCRETE WALLS AND SLABS SHALL BE AS FOLLOWS:

WALL THICKNESS	REINF EACH WAY	LOCATION
6"	#4@12"	CENTERED
8"	#5@12"	CENTERED
10"	#4@12"	EACH FACE
12"	#5@12"	EACH FACE

PROVIDE LARGER SIZES AND MORE REINFORCING IN ALL SECTIONS OF CONCRETE WHERE REQUIRED BY THE DETAILS ON THE DRAWINGS OR BY THE SPECIFICATIONS.

2. CLEARANCE FOR REINFORCEMENT BARS, UNLESS SHOWN OTHERWISE, SHALL BE:
   - WHEN PLACED ON GROUND: —— 3"
   - ALL OTHER CONCRETE SURFACES:
     #5 BAR OR SMALLER —— 1½"
     #6 BAR OR LARGER —— 2"
   - INTERIOR BUILDING SLAB SURFACES (NOT OVER WETWELLS) —— ¾" CLR
3. REFER TO WALL CORNER AND WALL INTERSECTION REINFORCING DETAIL. IN GENERAL, THE WALL CORNER REINFORCING SIZES AND SPACINGS SHALL BE AS CALLED OUT ON THE PLANS AND REFERENCED TO THESE DETAILS AND THE TYPICAL HORIZONTAL WALL REINFORCING SHALL LAP WITH THE HORIZONTAL REINFORCING.
4. ALL BENDS, UNLESS OTHERWISE SHOWN, SHALL BE A 90 DEGREE STANDARD HOOK AS DEFINED IN LATEST EDITION OF ACI 318.
5. ALL WALL CORNER AND WALL INTERSECTION REINFORCEMENT BARS SHALL BE CONTINUOUS AROUND CORNERS AND THROUGH COLUMNS OR PILASTERS. REINFORCEMENT SHALL BE EXTENDED INTO CONNECTING WALLS AND LAPPED ON THE OPPOSITE FACE OF THE CONNECTING WALLS, AS INDICATED ELSEWHERE. ALTERNATE HORIZONTAL BAR LAPS ON OPPOSITE FACES OF WALLS.
6. VERTICAL WALL BARS SHALL BE LAPPED WITH DOWELS FROM BASE SLABS AND EXTENDED INTO THE TOP FACE OF ROOF SLABS AND LAPPED WITH TOP SLAB REINFORCEMENT. PROVIDE A MINIMUM OF TWO FULL HEIGHT VERTICAL BARS WITH MATCHING DOWELS AT WALL ENDS, CORNERS AND INTERSECTIONS WITH SIZE TO MATCH TYPICAL VERTICAL REINFORCING STEEL SHOWN OR REQUIRED BY NOTES ABOVE.
7. UNLESS INDICATED OTHERWISE, CONTRACTOR MAY SPLICE CONTINUOUS SLAB OR LONGITUDINAL BEAM BARS AT LOCATIONS OF HIS CHOOSING, EXCEPT THAT TOP BAR SPLICES SHALL BE LOCATED AT MIDSPAN AND BOTTOM BAR SPLICES SHALL BE LOCATED AT SUPPORTS. ALL REINFORCEMENT BENDS AND LAPS, UNLESS OTHERWISE NOTED, SHALL SATISFY THE FOLLOWING MINIMUM REQUIREMENT:

DETAIL OF REINFORCEMENT - LAP LENGTHS

BAR SIZE		#6 OR SMALLER	#7	#8	#9	#10	#11
CONC DESIGN STRENGTH		▲ 4000 PSI					
GR 40	TOP BAR*	32 DIA, MIN 2'-0"					
	OTHER BAR	22 DIA, MIN 1'-6"					
GR 60	TOP BAR*	48 DIA, MIN 2'-6"	3'-8"	4'-9"	6'-0" ▲	7'-8" ▲	9'-6" ▲
	OTHER BAR	38 DIA, MIN 2'-0"	2'-9"	3'-6"	4'-6" ▲	5'-10" ▲	7'-2" ▲

* TOP BARS SHALL BE DEFINED AS ANY HORIZONTAL BARS PLACED SUCH THAT MORE THAN 12" OF FRESH CONCRETE IS CAST IN THE MEMBER BELOW THE BAR, IN ANY SINGLE POUR. HORIZONTAL WALL BARS ARE CONSIDERED TOP BARS.

8. ▲ INCREASE LAP LENGTHS SHOWN ABOVE BY 25% WHERE BARS ARE SPACED CLOSER THAN 8" O.C. OR LESS THAN 3" CLEAR FROM FACE OF MEMBER TO EDGE BAR MEASURED IN DIRECTION OF SPACING.

RECORD DRAWINGS

Revisions Drawn By: R. COWAN Date: 5/98

THESE RECORD DRAWINGS HAVE BEEN PREPARED, IN PART, ON THE BASIS OF INFORMATION COMPILED BY OTHERS. THEY ARE NOT INTENDED TO REPRESENT IN DETAIL THE EXACT LOCATION, TYPE OF COMPONENT NOR MANNER OF CONSTRUCTION. THE ENGINEER WILL NOT BE RESPONSIBLE FOR ANY ERRORS OR OMISSIONS WHICH HAVE BEEN INCORPORATED INTO THE RECORD DRAWINGS.

The Contract Documents Drawings are the original documents dated DEC 1995 (date on drawing), as subsequently amended, which define the scope, extent, and character of the work. The original Contract Document drawing was sealed and signed by (name of sealer) DOYLE L. LEEK, State of Oregon PE No. 11937 (sealer's PE No.).

CH2M HILL

DSGN	D LEEK
DR	E PUIG
CHK	SK RITKIN
APVD	W BRACKEN

NO.	DATE	REVISION	BY	APVD
	5/98	RECORD DRAWINGS	RKC	JBL

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 ━━━━ 1"
IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

CITY OF WILSONVILLE
WASTEWATER TREATMENT PLANT
WILSONVILLE, OREGON

DESIGN DETAILS
STRUCTURAL DETAILS

3000

SHEET 92
DWG NO. 98-S-401
DATE DEC 1995
PROJ NO. 117645.A4

PLOT DATE: 14-MAY-1998 09:40:35 wl98s401.dlv

DRAWING NUMBER

93 10 014

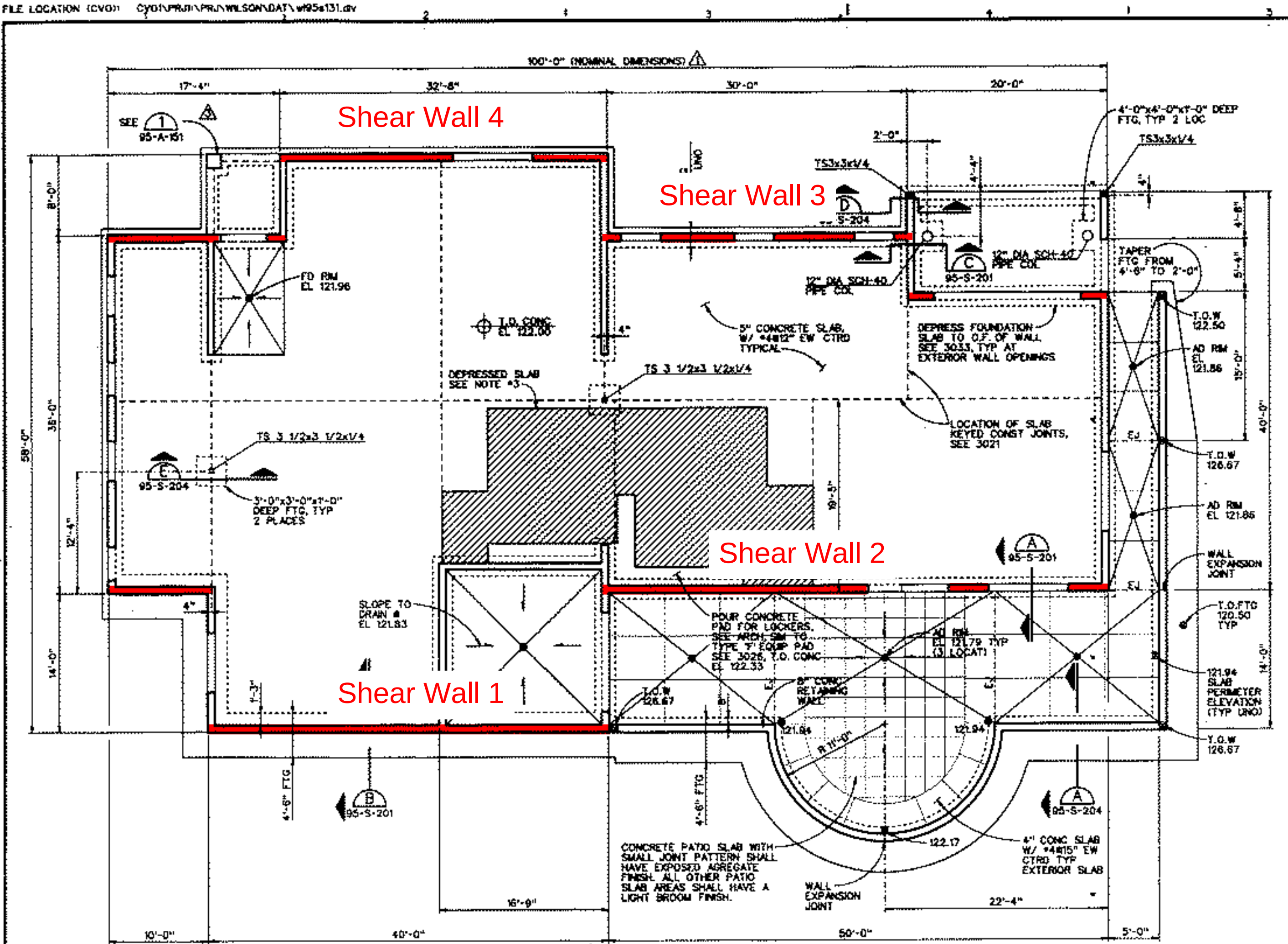
DRAWING NUMBER

WWTP UPGRADE

DRAWING NUMBER

SE 23

FILE LOCATION (CVO): CVO\PRJ\PRJ\WILSON\DAT\wl95s131.dgn



OPERATIONS BUILDING STRUCTURAL NOTES

GENERAL

1. SEE STRUCTURAL NOTES, DRAWING 95-S-401
2. SEE ARCHITECTURAL FLOOR PLAN, DRAWING 95-A-131 FOR DIMENSIONS NOT SHOWN.
3. DENOTES SLAB DEPRESSED 2" FOR TILE. SEE ARCHITECTURAL DRAWINGS.

CODES AND LOADING

1. ALL MATERIALS AND WORKMANSHIP SHALL CONFORM TO THE UNIFORM BUILDING CODE, 1994 EDITION (U.B.C.) WITH THE 1993 OREGON AMENDMENTS.
2. LOADS:
   - ROOF SNOW LOAD = 25 PSF PLUS DRIFTING
   - UBC WIND PRESSURE = 80 MPH WIND SPEED EXPOSURE C, I=1.0
   - UBC SEISMIC ZONE = 3, I=1.0, S=1.5
   - FLOOR LIVE LOAD = 200 PSF, UNIFORM LOAD OR 2000 LB, CONCENTRATED LOAD

METAL DECKING

1. GALVANIZED STEEL ROOF DECKING SHALL BE 1 1/2", 20 GAUGE WIDE RIB TYPE AND SHALL HAVE A MINIMUM POSITIVE SECTION MODULUS OF 0.232 IN^3 AND A MINIMUM MOMENT OF INERTIA OF 0.210 IN^4, UNLESS NOTED OTHERWISE.
2. STEEL ROOF DECKING SHALL BE FASTENED TO THE STEEL SUPPORTING MEMBERS BY WELDING ALL SUPPORTS PERPENDICULAR TO THE DECKING SPAN WITH 6-5/8" DIAMETER PUDDLE WELDS PER 3'-0" PANEL WIDTH. FASTEN TO ALL SUPPORTS PARALLEL TO THE DECK SPAN WITH 5/8" DIAMETER PUDDLE WELDS AT 1'-6" ON CENTER UNLESS NOTED. SEAMS SHALL BE FASTENED WITH #10 TEK SELF TAPPING SCREWS AT 1'-0" SPACING.
3. WELD DECKING TO ALL OPENINGS AND MISC. FRAMING WITH 5/8 INCH DIA PUDDLE WELDS AT 6 INCHES MAX. ON CENTER, BUT NOT LESS THAN 3 WELDS PER OPENING SIDE OR PER MEMBER.

OPEN WEB STEEL JOISTS

1. JOISTS SHALL BE DESIGNED, FABRICATED AND ERECTED IN ACCORDANCE WITH THE STANDARD SPECIFICATIONS OF THE AISC AND THE STEEL JOIST INSTITUTE.
2. JOISTS SHALL BE CAMBERED FOR DEAD LOAD. PROVIDE STANDARD SJI CAMBER. JOIST CAMBER SHALL BE SHOWN ON SHOP DRAWINGS AND SUBMITTED FOR REVIEW.
3. ALL OPEN WEB JOISTS SHALL HAVE DOUBLE ANGLE CHORDS.
4. JOIST BRIDGING IN ADDITION TO THAT SHOWN ON THE DRAWINGS SHALL BE AS PER MANUFACTURER'S RECOMENDATIONS.
5. JOIST DEPTH AND TYPE SHALL BE AS SHOWN. JOIST CHORD SIZE SHALL BE THE MINIMUM SHOWN. THE JOIST MANUFACTURER SHALL VERIFY THAT JOISTS ARE DESIGNED TO SUPPORT A UNIFORM ROOF DEAD LOAD EQUAL TO 20 PSF PLUS THE LIVE, SNOW AND CONCENTRATED LOADS SHOWN ON THE PLANS. PROVIDE ADDITIONAL DIAGONAL WEB MEMBERS AT CONCENTRATED LOAD LOCATIONS AS REQUIRED BY THE JOIST MANUFACTURER.

FOUNDATION PLAN

3/16"=1'-0"

RECORD DRAWINGS

THESE RECORD DRAWINGS HAVE BEEN PREPARED, IN PART, ON THE BASIS OF INFORMATION COMPILED BY OTHERS. THEY ARE NOT INTENDED TO REPRESENT IN DETAIL THE EXACT LOCATION, TYPE OF COMPONENT NOR MANNER OF CONSTRUCTION. THE ENGINEER WILL NOT BE RESPONSIBLE FOR ANY ERRORS OR OMISSIONS WHICH HAVE BEEN INCORPORATED INTO THE RECORD DRAWINGS.

NO.	DATE	REVISION	BY	APVD
	5/98	RECORD DRAWINGS	RKC	JBL
	7/96	CL #53 REVISIONS TO COLUMN & FOOTING	RKC	JBL
	11/96	CL #54 SHEET METAL DECKING SEAM CONNECTION	RKC	JBL
	7/96	CL #15 ADDRESSES NOMINAL BUILDING DIMENSIONS	RKC	JBL

DSGN D. LEEK · DR E. PUIG · CHK S. M. PITOCK · APVD W. BRACKEN

CH2M HILL

REUSE OF DOCUMENTS

THIS DOCUMENT, AND THE IDEAS AND DESIGNS INCORPORATED HEREIN, AS AN INSTRUMENT OF PROFESSIONAL SERVICE, IS THE PROPERTY OF CH2M HILL AND IS NOT TO BE USED, IN WHOLE OR IN PART, FOR ANY OTHER PROJECT WITHOUT THE WRITTEN AUTHORIZATION OF CH2M HILL.

VERIFY SCALE: BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

CITY OF WILSONVILLE
WASTEWATER TREATMENT PLANT
WILSONVILLE, OREGON

OPERATIONS BUILDING
STRUCTURAL FOUNDATION PLAN

SHEET 79 · DWG 95-S-121 · DATE DEC 1995 · PROJ NO. 117645.A4

PLOT DATE= 12-MAY-1998 16:34:13 wl95s131.dgn

Engineers...Working Wonders With Water ™

BY:	BS	DATE	Sep-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Operations Building	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (BSE-2E)		

CMU IN-PLANE SHEAR WALL CHECK IN BUILDING N-S DIRECTION

11.3.4.2 Strength of Reinforced Masonry Walls and Wall Piers In-Plane Actions. The strength of existing, retrofitted, and new RM wall or wall pier components in flexure, shear, and axial compression shall be determined in accordance with this section. Design actions (axial, flexure, and shear) on components shall be determined in accordance with Chapter 7 of this standard, considering gravity loads and the maximum forces that can be transmitted based on a limit-state analysis.

11.3.4.2.1 Flexural Strength of Walls and Wall Piers. Expected flexural strength of an RM wall or wall pier shall be determined based on strength design procedures specified in TMS 402.

11.3.4.2.2 Shear Strength of Walls and Wall Piers. The expected and lower-bound shear strength of RM wall or wall pier components shall be determined based on strength design procedures specified in TMS 402.

11.3.4.3 Acceptance Criteria for In-Plane Actions of Reinforced Masonry Walls. The shear required to develop the expected flexural strength of RM walls and wall piers shall be compared with the lower-bound shear strength. For RM wall components governed by flexure, flexural actions shall be considered deformation controlled. For RM components governed by shear, shear actions shall be considered deformation controlled. Axial compression on RM wall or wall pier components shall be considered a force-controlled action.

7.5.2 Linear Procedures

7.5.2.1 Forces and Deformations. Component forces and deformations shall be calculated in accordance with linear analysis procedures of Sections 7.4.1 or 7.4.2.

7.5.2.1.1 Deformation-Controlled Actions for LSP or LDP. Deformation-controlled actions, Q_{UD}, shall be calculated in accordance with Eq. (7-34):

$$Q_{UD} = Q_G + Q_E \tag{7-34}$$

where

Q_{UD} = Deformation-controlled action caused by gravity loads and earthquake forces.

Q_G = Action caused by gravity loads as defined in Section 7.2.2; and

Q_E = Action caused by the response to the selected Seismic Hazard Level calculated using either Section 7.4.1 or Section 7.4.2;

Shear wall 1

Roof seismic load, V =	198.8	kip
diaphragm span, L =	60.00	ft
roof tributary width for seismic, T_w =	8	ft
tributary seismic load on shear wall, Q_E =	26.5	kip
wall height, h =	10.17	ft
tributary seismic moment on shear wall, Mu =	269.6	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	40	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	1692.0	in²
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\,d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	233.0	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	261.6	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	5544.0	lbs
vertical compressive stress, $f_{ae} = P/(d*t)$ =	1.5	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.001	

h/L = 0.25
steel reinforcing ratio, ρ_g = 0.003
ρ_g*f_{ye}/f_{me} = 0.08

m-factor = 7.0 (interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ = 0.90
masonry shear wall strength, κmϕQCE = 1648.3 kip

demand capacity ratio, DCR = 0.02 ***OK***

Shear wall 2

Roof seismic load, V = 198.8 kip
diaphragm span, L = 60.00 ft
roof tributary width for seismic, T_w = 25 ft
tributary seismic load on shear wall, Q_E = 82.8 kip

wall height, h = 10.17 ft
tributary seismic moment on shear wall, Mu = 842.4 kip*ft
masonry strength, f'_m = 1500 psi
shear wall length, d = 44 ft
vertitcal shear wall grout spacing = 32 in
horizontal shear wall grout spacing = 48 in
shear wall thickness, t = 7.625 in
A_n = 1853.0 in^2
Φ = 1.0 (assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m = 258.0 kip
horizontal masonry shear wall strength, ϕQCE_s = 28.7 kip
combined masonry shear wall strength, ϕQCE = 286.7 kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P = 49500.0 lbs
vertical compressive stress, f_{ae} = P/(d*t) = 12.3 psi
factor for expected strength, F_{exp} = 1.3 (ASCE 41-17 Table 11-1)
expected compressive strength, f_{me} = F_{exp}*f'_m = 1950.0 psi
f_{ae}/f_{me} = 0.006
h/L = 0.23
steel reinforcing ratio, ρ_g = 0.003
ρ_g*f_{ye}/f_{me} = 0.08

m-factor = 7.0 (interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ = 0.90
masonry shear wall strength, κmϕQCE = 1806.3 kip

demand capacity ratio, DCR = 0.05 ***OK***

Shear wall 3

Roof seismic load, V = 198.8 kip
diaphragm span, L = 60.00 ft
roof tributary width for seismic, T_w = 22 ft
tributary seismic load on shear wall, Q_E = 72.9 kip

wall height, h = 10.17 ft
tributary seismic moment on shear wall, Mu = 741.3 kip*ft
masonry strength, f'_m = 1500 psi
shear wall length, d = 40.67 ft
vertitcal shear wall grout spacing = 32 in

horizontal shear wall grout spacing = 48 in
shear wall thickness, t = 7.625 in
A_n = 1712.1 in^2
Φ = 1.0 (assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m = 236.2 kip
horizontal masonry shear wall strength, ϕQCE_s = 28.7 kip
combined masonry shear wall strength, ϕQCE = 264.9 kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P = 43560.0 lbs
vertical compressive stress, f_{ae} = P/(d*t) = 11.7 psi
factor for expected strength, F_{exp} = 1.3 (ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ = 1950.0 psi
f_{ae}/f_{me} = 0.006 psi
h/L = 0.25
steel reinforcing ratio, ρ_g = 0.003
ρ_g*f_{ye}/f_{me} = 0.08

m-factor = 7.0 (interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ = 0.90
masonry shear wall strength, κmϕQCE = 1668.8 kip

demand capacity ratio, DCR = 0.04 ***OK***

<u>*Shear wall 4*</u>

Roof seismic load, V = 198.8 kip
diaphragm span, L = 60.00 ft
roof tributary width for seismic, T_w = 5 ft
tributary seismic load on shear wall, Q_E = 16.6 kip

wall height, h = 10.17 ft
tributary seismic moment on shear wall, Mu = 168.5 kip*ft
masonry strength, f'_m = 1500 psi
shear wall length, d = 24.67 ft
vertitcal shear wall grout spacing = 32 in
horizontal shear wall grout spacing = 48 in
shear wall thickness, t = 7.625 in
A_n = 1068.1 in^2
Φ = 1.0 (assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m = 135.6 kip
horizontal masonry shear wall strength, ϕQCE_s = 28.7 kip
combined masonry shear wall strength, ϕQCE = 164.3 kip

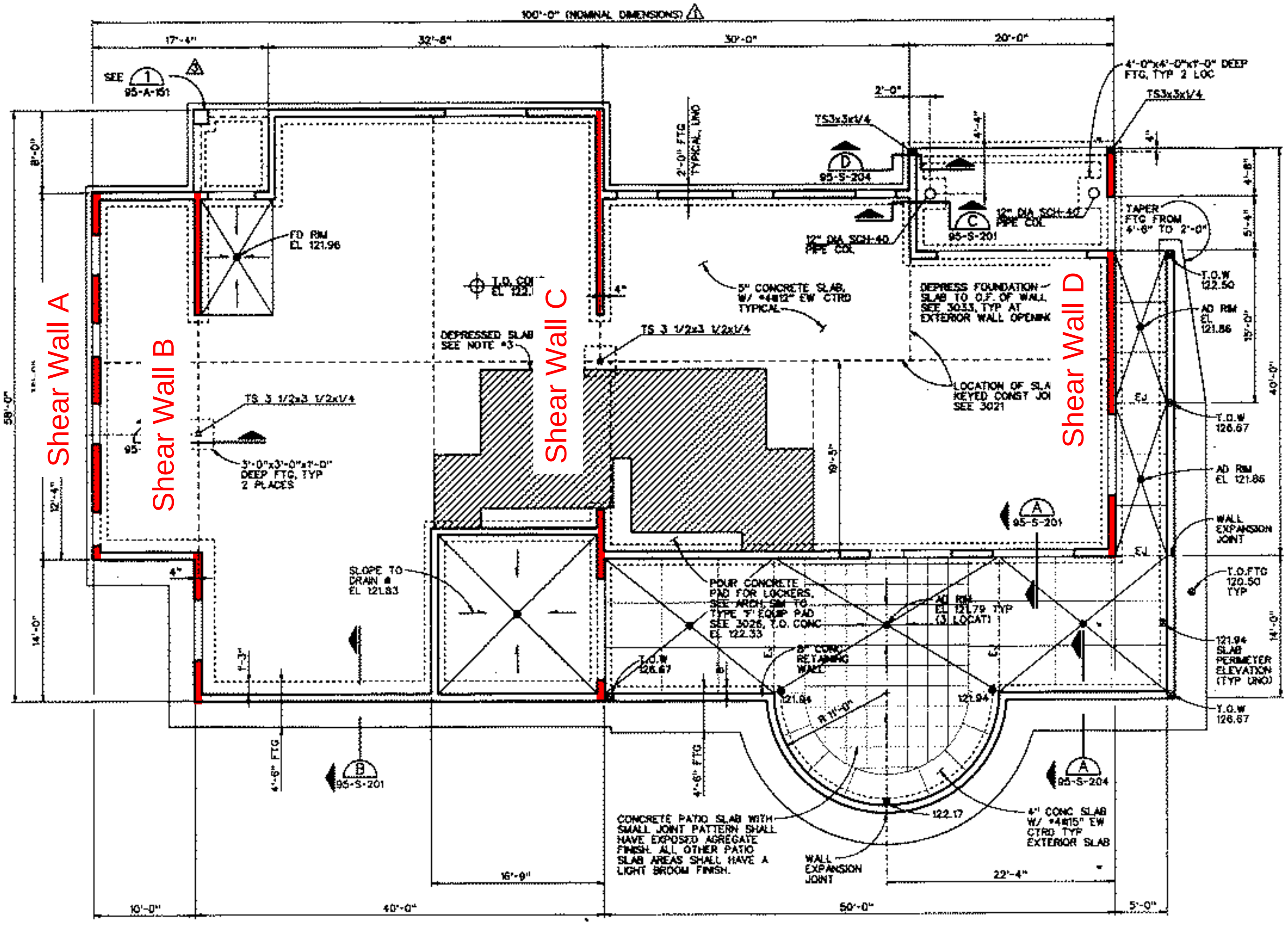
Determining m-factor for wall governed by flexure

roof axial load on wall, P = 2587.5 lbs
vertical compressive stress, f_{ae} = P/(d*t) = 1.1 psi
factor for expected strength, F_{exp} = 1.3 (ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ = 1950.0 psi
f_{ae}/f_{me} = 0.001 psi
h/L = 0.41
steel reinforcing ratio, ρ_g = 0.003
ρ_g*f_{ye}/f_{me} = 0.08

m-factor =	7.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	1035.1	kip

demand capacity ratio, DCR = *0.02* ***OK***

FILE LOCATION (CVO): CVO1\PRJ\\PRJ\WILSON\DAT\ wl95s131.dgn



FOUNDATION PLAN
3/16"=1'-0"

OPERATIONS BUILDING STRUCTURAL NOTES

GENERAL

1. SEE STRUCTURAL NOTES, DRAWING 95-S-401
2. SEE ARCHITECTURAL FLOOR PLAN, DRAWING 95-A-131 FOR DIMENSIONS NOT SHOWN.
3. DENOTES SLAB DEPRESSED 2" FOR TILE. SEE ARCHITECTURAL DRAWINGS.

CODES AND LOADING

1. ALL MATERIALS AND WORKMANSHIP SHALL CONFORM TO THE UNIFORM BUILDING CODE, 1994 EDITION (U.B.C.) WITH THE 1993 OREGON AMENDMENTS.
2. LOADS:
   - ROOF SNOW LOAD = 25 PSF PLUS DRIFTING
   - UBC WIND PRESSURE = 80 MPH WIND SPEED EXPOSURE C, I=1.0
   - UBC SEISMIC ZONE = 3, I=1.0, S=1.5
   - FLOOR LIVE LOAD = 200 PSF, UNIFORM LOAD OR 2000 LB, CONCENTRATED LOAD

METAL DECKING

1. GALVANIZED STEEL ROOF DECKING SHALL BE 1 1/2", 20 GAUGE WIDE RIB TYPE AND SHALL HAVE A MINIMUM POSITIVE SECTION MODULUS OF 0.232 IN^3 AND A MINIMUM MOMENT OF INERTIA OF 0.210 IN^4, UNLESS NOTED OTHERWISE.
2. STEEL ROOF DECKING SHALL BE FASTENED TO THE STEEL SUPPORTING MEMBERS BY WELDING ALL SUPPORTS PERPENDICULAR TO THE DECKING SPAN WITH 6-5/8" DIAMETER PUDDLE WELDS PER 3'-0" PANEL WIDTH. FASTEN TO ALL SUPPORTS PARALLEL TO THE DECK SPAN WITH 5/8" DIAMETER PUDDLE WELDS AT 1'-6" ON CENTER UNLESS NOTED. SEAMS SHALL BE FASTENED WITH #10 TEK SELF TAPPING SCREWS AT 1'-0" SPACING.
3. WELD DECKING TO ALL OPENINGS AND MISC. FRAMING WITH 5/8 INCH DIA PUDDLE WELDS AT 6 INCHES MAX. ON CENTER, BUT NOT LESS THAN 3 WELDS PER OPENING SIDE OR PER MEMBER.

OPEN WEB STEEL JOISTS

1. JOISTS SHALL BE DESIGNED, FABRICATED AND ERECTED IN ACCORDANCE WITH THE STANDARD SPECIFICATIONS OF THE AISC AND THE STEEL JOIST INSTITUTE.
2. JOISTS SHALL BE CAMBERED FOR DEAD LOAD. PROVIDE STANDARD SJI CAMBER. JOIST CAMBER SHALL BE SHOWN ON SHOP DRAWINGS AND SUBMITTED FOR REVIEW.
3. ALL OPEN WEB JOISTS SHALL HAVE DOUBLE ANGLE CHORDS.
4. JOIST BRIDGING IN ADDITION TO THAT SHOWN ON THE DRAWINGS SHALL BE AS PER MANUFACTURER'S RECOMENDATIONS.
5. JOIST DEPTH AND TYPE SHALL BE AS SHOWN. JOIST CHORD SIZE SHALL BE THE MINIMUM SHOWN. THE JOIST MANUFACTURER SHALL VERIFY THAT JOISTS ARE DESIGNED TO SUPPORT A UNIFORM ROOF DEAD LOAD EQUAL TO 20 PSF PLUS THE LIVE, SNOW AND CONCENTRATED LOADS SHOWN ON THE PLANS. PROVIDE ADDITIONAL DIAGONAL WEB MEMBERS AT CONCENTRATED LOAD LOCATIONS AS REQUIRED BY THE JOIST MANUFACTURER.

RECORD DRAWINGS

Revisions Drawn By ___ Date 5/98

THESE RECORD DRAWINGS HAVE BEEN PREPARED, IN PART, ON THE BASIS OF INFORMATION COMPILED BY OTHERS. THEY ARE NOT INTENDED TO REPRESENT IN DETAIL THE EXACT LOCATION, TYPE OF COMPONENT NOR MANNER OF CONSTRUCTION. THE ENGINEER WILL NOT BE RESPONSIBLE FOR ANY ERRORS OR OMISSIONS WHICH HAVE BEEN INCORPORATED INTO THE RECORD DRAWINGS.

DSGN	D LEEK		
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CHK	S M PITION		
APVD	W BRACKEN		

NO.	DATE	REVISION	BY	APVD
	5/98	RECORD DRAWINGS	RKC	JBL
	7/96	CL #53 REVISIONS TO COLUMN & FOOTING	RKC	JBL
	11/96	CL #54 SHEET METAL DECKING SEAM CONNECTION	RKC	JBL
	7/96	CL #15 ADDRESSES NOMINAL BUILDING DIMENSIONS	RKC	JBL

REUSE OF DOCUMENTS

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CITY OF WILSONVILLE
WASTEWATER TREATMENT PLANT
WILSONVILLE, OREGON

OPERATIONS BUILDING
STRUCTURAL FOUNDATION PLAN

SHEET	79
DWG NO.	95-S-121
DATE	DEC 1995
PROJ NO.	117645.A4

PLOT DATE= 12-MAY-1998 16:34:13 wl95s131.dgn

Engineers...Working Wonders With Water™

BY:	BS	DATE	Sep-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Operations Building	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (BSE-2E)		

CMU IN-PLANE SHEAR WALL CHECK IN BUILDING E-W DIRECTION

11.3.4.2 Strength of Reinforced Masonry Walls and Wall Piers In-Plane Actions. The strength of existing, retrofitted, and new RM wall or wall pier components in flexure, shear, and axial compression shall be determined in accordance with this section. Design actions (axial, flexure, and shear) on components shall be determined in accordance with Chapter 7 of this standard, considering gravity loads and the maximum forces that can be transmitted based on a limit-state analysis.

11.3.4.2.1 Flexural Strength of Walls and Wall Piers. Expected flexural strength of an RM wall or wall pier shall be determined based on strength design procedures specified in TMS 402.

11.3.4.2.2 Shear Strength of Walls and Wall Piers. The expected and lower-bound shear strength of RM wall or wall pier components shall be determined based on strength design procedures specified in TMS 402.

11.3.4.3 Acceptance Criteria for In-Plane Actions of Reinforced Masonry Walls. The shear required to develop the expected flexural strength of RM walls and wall piers shall be compared with the lower-bound shear strength. For RM wall components governed by flexure, flexural actions shall be considered deformation controlled. For RM components governed by shear, shear actions shall be considered deformation controlled. Axial compression on RM wall or wall pier components shall be considered a force-controlled action.

7.5.2 Linear Procedures

7.5.2.1 Forces and Deformations. Component forces and deformations shall be calculated in accordance with linear analysis procedures of Sections 7.4.1 or 7.4.2.

7.5.2.1.1 Deformation-Controlled Actions for LSP or LDP. Deformation-controlled actions, Q_{UD}, shall be calculated in accordance with Eq. (7-34):

$$Q_{UD} = Q_G + Q_E \quad (7\text{-}34)$$

where

Q_{UD} = Deformation-controlled action caused by gravity loads and earthquake forces.

Q_G = Action caused by gravity loads as defined in Section 7.2.2; and

Q_E = Action caused by the response to the selected Seismic Hazard Level calculated using either Section 7.4.1 or Section 7.4.2;

Shear wall A

Roof seismic load, V =	198.8	kip
diaphragm span, L =	102.00	ft
roof tributary width for seismic, T_w =	6	ft
tributary seismic load on shear wall, Q_E =	11.7	kip
wall height, h =	10.17	ft
tributary seismic moment on shear wall, Mu =	118.9	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	20	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	887.0	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\,d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	106.8	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	135.5	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	3564.0	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	0.3	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp} * f'_m$ =	1950.0	psi

f_{ae}/f_{me} =	0.000	
h/L =	0.51	
steel reinforcing ratio, ρ_g =	0.003	
ρ_g*f_{ye}/f_{me} =	0.08	
m-factor =	7.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	853.8	kip
demand capacity ratio, DCR =	*0.01*	***OK***

Shear wall B

Roof seismic load, V =	198.8	kip
diaphragm span, L =	102.00	ft
roof tributary width for seismic, T_w =	25	ft
tributary seismic load on shear wall, Q_E =	48.7	kip
wall height, h =	10.17	ft
tributary seismic moment on shear wall, Mu =	495.5	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	20.67	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	907.1	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	110.3	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	139.0	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	28710.0	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	2.4	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.001	
h/L =	0.49	
steel reinforcing ratio, ρ_g =	0.003	
ρ_g*f_{ye}/f_{me} =	0.08	
m-factor =	7.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	875.4	kip
demand capacity ratio, DCR =	*0.06*	***OK***

Shear wall C

Roof seismic load, V =	198.8	kip
diaphragm span, L =	102.00	ft
roof tributary width for seismic, T_w =	45	ft
tributary seismic load on shear wall, Q_E =	87.7	kip
wall height, h =	10.17	ft
tributary seismic moment on shear wall, Mu =	892.0	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	28.67	ft

vertitcal shear wall grout spacing = 32 in
horizontal shear wall grout spacing = 48 in
shear wall thickness, t = 7.625 in
A_n = 1229.1 in^2
Φ = 1.0 (assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m = 160.9 kip
horizontal masonry shear wall strength, ϕQCE_s = 28.7 kip
combined masonry shear wall strength, ϕQCE = 189.5 kip

Determining m-factor for wall governed by flexure
roof axial load on wall, P = 40788.0 lbs
vertical compressive stress, f_{ae} = P/(d*t) = 2.5 psi
factor for expected strength, F_{exp} = 1.3 (ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp} * f'_m$ = 1950.0 psi
f_{ae}/f_{me} = 0.001
h/L = 0.35
steel reinforcing ratio, ρ_g = 0.003
$\rho_g * f_{ye}/f_{me}$ = 0.08

m-factor = 7.0 (interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ = 0.90
masonry shear wall strength, κmϕQCE = 1194.1 kip

demand capacity ratio, DCR = 0.07 ***OK***

<u>*Shear wall D*</u>
Roof seismic load, V = 198.8 kip
diaphragm span, L = 102.00 ft
roof tributary width for seismic, T_w = 26 ft
tributary seismic load on shear wall, Q_E = 50.7 kip

wall height, h = 10.17 ft
tributary seismic moment on shear wall, Mu = 515.4 kip*ft
masonry strength, f'_m = 1500 psi
shear wall length, d = 26.67 ft
vertitcal shear wall grout spacing = 32 in
horizontal shear wall grout spacing = 48 in
shear wall thickness, t = 7.625 in
A_n = 1128.1 in^2
Φ = 1.0 (assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m = 145.6 kip
horizontal masonry shear wall strength, ϕQCE_s = 28.7 kip
combined masonry shear wall strength, ϕQCE = 174.3 kip

Determining m-factor for wall governed by flexure
roof axial load on wall, P = 17820.0 lbs
vertical compressive stress, f_{ae} = P/(d*t) = 1.2 psi
factor for expected strength, F_{exp} = 1.3 (ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp} * f'_m$ = 1950.0 psi
f_{ae}/f_{me} = 0.001
h/L = 0.38
steel reinforcing ratio, ρ_g = 0.003

ρ_g*f_{ye}/f_{me} =	0.08	
m-factor =	7.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	1098.0	kip
demand capacity ratio, DCR =	*0.05*	***OK***

Carollo Engineers. Working Wonders With Water

BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Operations Building	**JOB NO.**	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (CSZ)		

SEISMIC BASE SHEAR FOR OPERATIONS BUILDING

7.4.1.3.1 Pseudo Seismic Force for LSP. The pseudo lateral force in a given horizontal direction of a building shall be determined using Eq. (7-21). This force shall be used to evaluate or retrofit the vertical elements of the seismic-force-resisting system.

$$V = C_1 C_2 C_m S_a W \tag{7-21}$$

Table 7-3. Alternate Values for Modification Factors C_1C_2

Fundamental Period	$m_{max} < 2$	$2 \le m_{max} < 6$	$m_{max} \ge 6$
$T \le 0.3$	1.1	1.4	1.8
$0.3 < T \le 1.0$	1.0	1.1	1.2
$T > 1.0$	1.0	1.0	1.1

Table 7-4. Values for Effective Mass Factor C_m

No. of Stories	Concrete Moment Frame	Concrete Shear Wall	Concrete Pier-Spandrel	Steel Moment Frame	Steel Concentrically Braced Frame	Steel Eccentrically Braced Frame	Other
1–2	1.0	1.0	1.0	1.0	1.0	1.0	1.0
3 or more	0.9	0.8	0.8	0.9	0.9	0.9	1.0

Note: C_m shall be taken as 1.0 if the fundamental period, T, in the direction of response under consideration is greater than 1.0 s.

spectral response acceleration, S_{xs} =	0.446	g	(CSZ seismic hazard)
spectral response acceleration, S_{x1} =	0.332	g	(CSZ seismic hazard)
building period, T =	0.114	s	
response spectrum acceleration, S_a =	0.446	g	
effective seismic weight, W =	190.9	kip	
C_1C_2 =	1.4		(Table 11-6 for masonry walls, m=2.0)
effective mass factor, C_m =	1.0		
seismic lateral force, V =	119.2	kip	

DRAWING NUMBER

93 10 014

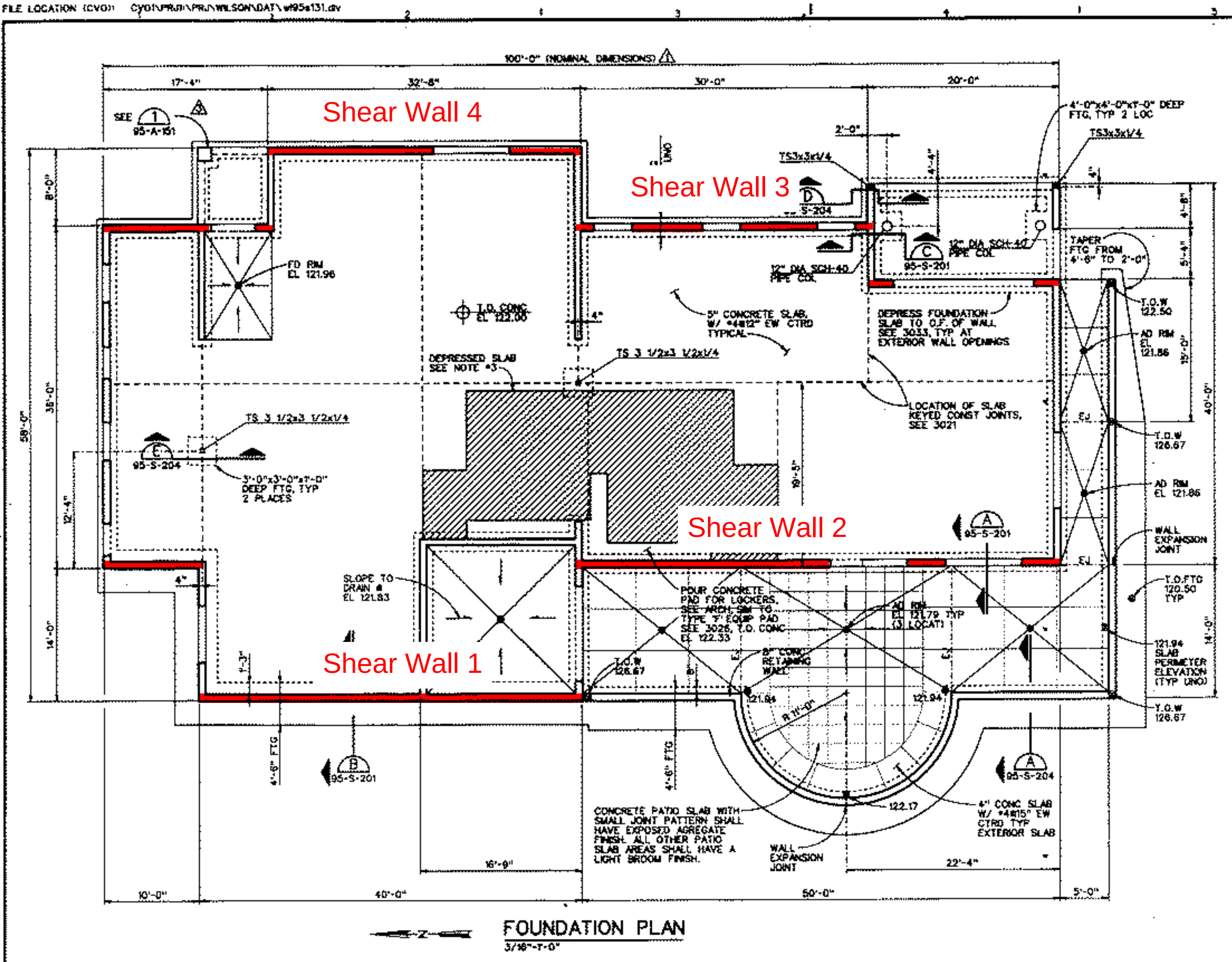
DRAWING NUMBER

WWTP UPGRADE

DRAWING NUMBER

SE 23

FILE LOCATION (CVO): CVO1\PRJ\\PRJ\WILSON\DAT\ wl95s131.dgn



FOUNDATION PLAN

3/16"=1'-0"

OPERATIONS BUILDING STRUCTURAL NOTES

GENERAL

1. SEE STRUCTURAL NOTES, DRAWING 95-S-401
2. SEE ARCHITECTURAL FLOOR PLAN, DRAWING 95-A-131 FOR DIMENSIONS NOT SHOWN.
3. DENOTES SLAB DEPRESSED 2" FOR TILE. SEE ARCHITECTURAL DRAWINGS.

CODES AND LOADING

1. ALL MATERIALS AND WORKMANSHIP SHALL CONFORM TO THE UNIFORM BUILDING CODE, 1994 EDITION (U.B.C.) WITH THE 1993 OREGON AMENDMENTS.
2. LOADS: ROOF SNOW LOAD = 25 PSF PLUS DRIFTING
   UBC WIND PRESSURE = 80 MPH WIND SPEED EXPOSURE C, I=1.0
   UBC SEISMIC ZONE = 3, I=1.0, S=1.5
   FLOOR LIVE LOAD = 200 PSF, UNIFORM LOAD OR 2000 LB, CONCENTRATED LOAD

METAL DECKING

1. GALVANIZED STEEL ROOF DECKING SHALL BE 1 1/2", 20 GAUGE WIDE RIB TYPE AND SHALL HAVE A MINIMUM POSITIVE SECTION MODULUS OF 0.232 IN^3 AND A MINIMUM MOMENT OF INERTIA OF 0.210 IN^4, UNLESS NOTED OTHERWISE.
2. STEEL ROOF DECKING SHALL BE FASTENED TO THE STEEL SUPPORTING MEMBERS BY WELDING ALL SUPPORTS PERPENDICULAR TO THE DECKING SPAN WITH 5-5/8" DIAMETER PUDDLE WELDS PER 3'-0" PANEL WIDTH. FASTEN TO ALL SUPPORTS PARALLEL TO THE DECK SPAN WITH 5/8" DIAMETER PUDDLE WELDS AT 1'-6" ON CENTER UNLESS NOTED. SEAMS SHALL BE FASTENED WITH #10 TEK SELF TAPPING SCREWS AT 1'-0" SPACING.
3. WELD DECKING TO ALL OPENINGS AND MISC. FRAMING WITH 5/8 INCH DIA PUDDLE WELDS AT 6 INCHES MAX. ON CENTER, BUT NOT LESS THAN 3 WELDS PER OPENING SIDE OR PER MEMBER.

OPEN WEB STEEL JOISTS

1. JOISTS SHALL BE DESIGNED, FABRICATED AND ERECTED IN ACCORDANCE WITH THE STANDARD SPECIFICATIONS OF THE AISC AND THE STEEL JOIST INSTITUTE.
2. JOISTS SHALL BE CAMBERED FOR DEAD LOAD. PROVIDE STANDARD SJI CAMBER. JOIST CAMBER SHALL BE SHOWN ON SHOP DRAWINGS AND SUBMITTED FOR REVIEW.
3. ALL OPEN WEB JOISTS SHALL HAVE DOUBLE ANGLE CHORDS.
4. JOIST BRIDGING IN ADDITION TO THAT SHOWN ON THE DRAWINGS SHALL BE AS PER MANUFACTURER'S RECOMENDATIONS.
5. JOIST DEPTH AND TYPE SHALL BE AS SHOWN. JOIST CHORD SIZE SHALL BE THE MINIMUM SHOWN. THE JOIST MANUFACTURER SHALL VERIFY THAT JOISTS ARE DESIGNED TO SUPPORT A UNIFORM ROOF DEAD LOAD EQUAL TO 20 PSF PLUS THE LIVE, SNOW AND CONCENTRATED LOADS SHOWN ON THE PLANS. PROVIDE ADDITIONAL DIAGONAL WEB MEMBERS AT CONCENTRATED LOAD LOCATIONS AS REQUIRED BY THE JOIST MANUFACTURER.

RECORD DRAWINGS

REUSE OF DOCUMENTS

CITY OF WILSONVILLE WASTEWATER TREATMENT PLANT

WILSONVILLE, OREGON

OPERATIONS BUILDING

STRUCTURAL FOUNDATION PLAN

SHEET	79
DWG NO.	95-S-121
DATE	DEC 1995
PROJ NO.	117645.A4

PLOT DATE: 12-MAY-1998 15:34:13 wl95s131.dgn

Engineers...Working Wonders With Water ™

BY:	BS	DATE	Sep-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Operations Building	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (CSZ)		

CMU IN-PLANE SHEAR WALL CHECK IN BUILDING N-S DIRECTION

11.3.4.2 Strength of Reinforced Masonry Walls and Wall Piers In-Plane Actions. The strength of existing, retrofitted, and new RM wall or wall pier components in flexure, shear, and axial compression shall be determined in accordance with this section. Design actions (axial, flexure, and shear) on components shall be determined in accordance with Chapter 7 of this standard, considering gravity loads and the maximum forces that can be transmitted based on a limit-state analysis.

11.3.4.2.1 Flexural Strength of Walls and Wall Piers. Expected flexural strength of an RM wall or wall pier shall be determined based on strength design procedures specified in TMS 402.

11.3.4.2.2 Shear Strength of Walls and Wall Piers. The expected and lower-bound shear strength of RM wall or wall pier components shall be determined based on strength design procedures specified in TMS 402.

11.3.4.3 Acceptance Criteria for In-Plane Actions of Reinforced Masonry Walls. The shear required to develop the expected flexural strength of RM walls and wall piers shall be compared with the lower-bound shear strength. For RM wall components governed by flexure, flexural actions shall be considered deformation controlled. For RM components governed by shear, shear actions shall be considered deformation controlled. Axial compression on RM wall or wall pier components shall be considered a force-controlled action.

7.5.2 Linear Procedures

7.5.2.1 Forces and Deformations. Component forces and deformations shall be calculated in accordance with linear analysis procedures of Sections 7.4.1 or 7.4.2.

7.5.2.1.1 Deformation-Controlled Actions for LSP or LDP. Deformation-controlled actions, Q_{UD}, shall be calculated in accordance with Eq. (7-34):

$$Q_{UD} = Q_G + Q_E \qquad (7\text{-}34)$$

where

Q_{UD} = Deformation-controlled action caused by gravity loads and earthquake forces.

Q_G = Action caused by gravity loads as defined in Section 7.2.2; and

Q_E = Action caused by the response to the selected Seismic Hazard Level calculated using either Section 7.4.1 or Section 7.4.2;

Shear wall 1

Roof seismic load, V =	119.2	kip
diaphragm span, L =	60.00	ft
roof tributary width for seismic, T_w =	8	ft
tributary seismic load on shear wall, Q_E =	15.9	kip
wall height, h =	10.17	ft
tributary seismic moment on shear wall, Mu =	161.6	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	40	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	1692.0	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	233.0	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	261.6	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	5544.0	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	1.5	psi

factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.001	
h/L =	0.25	
steel reinforcing ratio, ρ_g =	0.003	
ρ_g*f_{ye}/f_{me} =	0.08	
m-factor =	5.0	(interpolated between LS & IO. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	1177.4	kip
demand capacity ratio, DCR =	*0.01*	***OK***

Shear wall 2

Roof seismic load, V =	119.2	kip
diaphragm span, L =	60.00	ft
roof tributary width for seismic, T_w =	25	ft
tributary seismic load on shear wall, Q_E =	49.7	kip
wall height, h =	10.17	ft
tributary seismic moment on shear wall, Mu =	505.1	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	44	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	1853.0	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	258.0	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	286.7	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	49500.0	lbs
vertical compressive stress, $f_{ae} = P/(d*t)$ =	12.3	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.006	
h/L =	0.23	
steel reinforcing ratio, ρ_g =	0.003	
ρ_g*f_{ye}/f_{me} =	0.08	
m-factor =	5.0	(interpolated between LS & IO. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	1290.2	kip
demand capacity ratio, DCR =	*0.04*	***OK***

Shear wall 3

Roof seismic load, V =	119.2	kip
diaphragm span, L =	60.00	ft
roof tributary width for seismic, T_w =	22	ft
tributary seismic load on shear wall, Q_E =	43.7	kip

wall height, h =	10.17	ft
tributary seismic moment on shear wall, Mu =	444.5	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	40.67	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	1712.1	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	236.2	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	264.9	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	43560.0	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	11.7	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.006	psi
h/L =	0.25	
steel reinforcing ratio, ρ_g =	0.003	
ρ_g*f_{ye}/f_{me} =	0.08	
m-factor =	5.0	(interpolated between LS & IO. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmφQCE =	1192.0	kip

demand capacity ratio, DCR = *0.04* ***OK***

<u>Shear wall 4</u>

Roof seismic load, V =	119.2	kip
diaphragm span, L =	60.00	ft
roof tributary width for seismic, T_w =	5	ft
tributary seismic load on shear wall, Q_E =	9.9	kip

wall height, h =	10.17	ft
tributary seismic moment on shear wall, Mu =	101.0	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	24.67	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	1068.1	in^2
Φ =	1.0	(assumed per Tier 2)

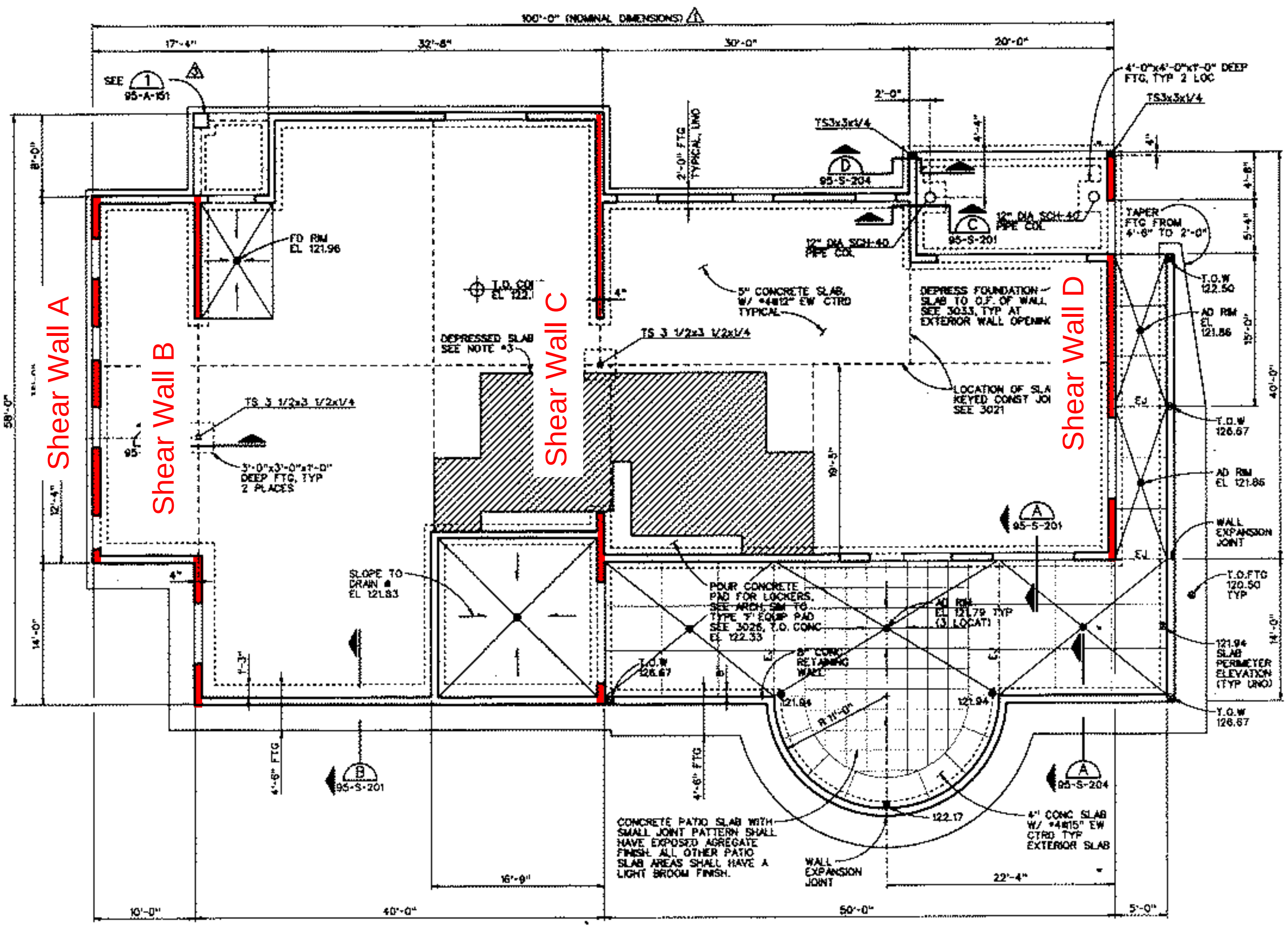
$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	135.6	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	164.3	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	2587.5	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	1.1	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, f_{me} = F_{exp}*f'_m =	1950.0	psi
f_{ae}/f_{me} =	0.001	psi
h/L =	0.41	
steel reinforcing ratio, ρ_g =	0.003	
ρ_g*f_{ye}/f_{me} =	0.08	
m-factor =	5.0	(interpolated between LS & IO. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	739.4	kip
demand capacity ratio, DCR =	*0.01*	***OK***

FILE LOCATION (CVO): CVO1\PRJ\\PRJ\WILSON\DAT\wl95s131.dv



FOUNDATION PLAN
3/16"=1'-0"

OPERATIONS BUILDING STRUCTURAL NOTES

GENERAL

1. SEE STRUCTURAL NOTES, DRAWING 95-S-401
2. SEE ARCHITECTURAL FLOOR PLAN, DRAWING 95-A-131 FOR DIMENSIONS NOT SHOWN.
3. DENOTES SLAB DEPRESSED 2" FOR TILE. SEE ARCHITECTURAL DRAWINGS.

CODES AND LOADING

1. ALL MATERIALS AND WORKMANSHIP SHALL CONFORM TO THE UNIFORM BUILDING CODE, 1994 EDITION (U.B.C.) WITH THE 1993 OREGON AMENDMENTS.
2. LOADS: ROOF SNOW LOAD = 25 PSF PLUS DRIFTING
   UBC WIND PRESSURE = 80 MPH WIND SPEED EXPOSURE C, I=1.0
   UBC SEISMIC ZONE = 3, I=1.0, S=1.5
   FLOOR LIVE LOAD = 200 PSF, UNIFORM LOAD OR 2000 LB, CONCENTRATED LOAD

METAL DECKING

1. GALVANIZED STEEL ROOF DECKING SHALL BE 1 1/2", 20 GAUGE WIDE RIB TYPE AND SHALL HAVE A MINIMUM POSITIVE SECTION MODULUS OF 0.232 IN^3 AND A MINIMUM MOMENT OF INERTIA OF 0.210 IN^4, UNLESS NOTED OTHERWISE.
2. STEEL ROOF DECKING SHALL BE FASTENED TO THE STEEL SUPPORTING MEMBERS BY WELDING ALL SUPPORTS PERPENDICULAR TO THE DECKING SPAN WITH 6-5/8" DIAMETER PUDDLE WELDS PER 3'-0" PANEL WIDTH. FASTEN TO ALL SUPPORTS PARALLEL TO THE DECK SPAN WITH 5/8" DIAMETER PUDDLE WELDS AT 1'-6" ON CENTER UNLESS NOTED. SEAMS SHALL BE FASTENED WITH #10 TEK SELF TAPPING SCREWS AT 1'-0" SPACING.
3. WELD DECKING TO ALL OPENINGS AND MISC. FRAMING WITH 5/8 INCH DIA PUDDLE WELDS AT 6 INCHES MAX. ON CENTER, BUT NOT LESS THAN 3 WELDS PER OPENING SIDE OR PER MEMBER.

OPEN WEB STEEL JOISTS

1. JOISTS SHALL BE DESIGNED, FABRICATED AND ERECTED IN ACCORDANCE WITH THE STANDARD SPECIFICATIONS OF THE AISC AND THE STEEL JOIST INSTITUTE.
2. JOISTS SHALL BE CAMBERED FOR DEAD LOAD. PROVIDE STANDARD SJI CAMBER. JOIST CAMBER SHALL BE SHOWN ON SHOP DRAWINGS AND SUBMITTED FOR REVIEW.
3. ALL OPEN WEB JOISTS SHALL HAVE DOUBLE ANGLE CHORDS.
4. JOIST BRIDGING IN ADDITION TO THAT SHOWN ON THE DRAWINGS SHALL BE AS PER MANUFACTURER'S RECOMENDATIONS.
5. JOIST DEPTH AND TYPE SHALL BE AS SHOWN. JOIST CHORD SIZE SHALL BE THE MINIMUM SHOWN. THE JOIST MANUFACTURER SHALL VERIFY THAT JOISTS ARE DESIGNED TO SUPPORT A UNIFORM ROOF DEAD LOAD EQUAL TO 20 PSF PLUS THE LIVE, SNOW AND CONCENTRATED LOADS SHOWN ON THE PLANS. PROVIDE ADDITIONAL DIAGONAL WEB MEMBERS AT CONCENTRATED LOAD LOCATIONS AS REQUIRED BY THE JOIST MANUFACTURER.

RECORD DRAWINGS

Revisions Drawn By: S. COBAN Date: 3/98

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DSGN: D. LEEK	5/98	RECORD DRAWINGS	
DR: E. PUIG	7/96	CL #53 REVISIONS TO COLUMN & FOOTING	RKC / JBL
CHK: S. M. PITION	11/96	CL #54 SHEET METAL DECKING SEAM CONNECTION	RKC / JBL
APVD: M. BRACKEN	7/96	CL #55 ADDRESSES NOMINAL BUILDING DIMENSIONS	RKC / JBL
	NO. DATE	REVISION	BY / APVD

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CITY OF WILSONVILLE
WASTEWATER TREATMENT PLANT
WILSONVILLE, OREGON

OPERATIONS BUILDING
STRUCTURAL FOUNDATION PLAN

SHEET	79
DWG NO.	95-S-121
DATE	DEC 1995
PROJ NO.	117645.A4

PLOT DATE=12-MAY-1998 16:34:13 wl95s131.dv

Engineers...Working Wonders With Water™

BY:	BS	DATE	Sep-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Operations Building	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (CSZ)		

CMU IN-PLANE SHEAR WALL CHECK IN BUILDING E-W DIRECTION

11.3.4.2 Strength of Reinforced Masonry Walls and Wall Piers In-Plane Actions. The strength of existing, retrofitted, and new RM wall or wall pier components in flexure, shear, and axial compression shall be determined in accordance with this section. Design actions (axial, flexure, and shear) on components shall be determined in accordance with Chapter 7 of this standard, considering gravity loads and the maximum forces that can be transmitted based on a limit-state analysis.

11.3.4.2.1 Flexural Strength of Walls and Wall Piers. Expected flexural strength of an RM wall or wall pier shall be determined based on strength design procedures specified in TMS 402.

11.3.4.2.2 Shear Strength of Walls and Wall Piers. The expected and lower-bound shear strength of RM wall or wall pier components shall be determined based on strength design procedures specified in TMS 402.

11.3.4.3 Acceptance Criteria for In-Plane Actions of Reinforced Masonry Walls. The shear required to develop the expected flexural strength of RM walls and wall piers shall be compared with the lower-bound shear strength. For RM wall components governed by flexure, flexural actions shall be considered deformation controlled. For RM components governed by shear, shear actions shall be considered deformation controlled. Axial compression on RM wall or wall pier components shall be considered a force-controlled action.

7.5.2 Linear Procedures

7.5.2.1 Forces and Deformations. Component forces and deformations shall be calculated in accordance with linear analysis procedures of Sections 7.4.1 or 7.4.2.

7.5.2.1.1 Deformation-Controlled Actions for LSP or LDP. Deformation-controlled actions, Q_{UD}, shall be calculated in accordance with Eq. (7-34):

$$Q_{UD} = Q_G + Q_E \qquad (7\text{-}34)$$

where

Q_{UD} = Deformation-controlled action caused by gravity loads and earthquake forces.

Q_G = Action caused by gravity loads as defined in Section 7.2.2; and

Q_E = Action caused by the response to the selected Seismic Hazard Level calculated using either Section 7.4.1 or Section 7.4.2;

Shear wall A

Roof seismic load, V =	119.2	kip
diaphragm span, L =	102.00	ft
roof tributary width for seismic, T_w =	6	ft
tributary seismic load on shear wall, Q_E =	7.0	kip
wall height, h =	10.17	ft
tributary seismic moment on shear wall, Mu =	71.3	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	20	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	887.0	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\,d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	106.8	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	135.5	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	3564.0	lbs

vertical compressive stress, $f_{ae} = P/(d*t)$ =	0.3	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.000	
h/L =	0.51	
steel reinforcing ratio, ρ_g =	0.003	
ρ_g*f_{ye}/f_{me} =	0.08	
m-factor =	5.0	(interpolated between LS & IO. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	609.8	kip

demand capacity ratio, DCR = *0.01* ***OK***

Shear wall B

Roof seismic load, V =	119.2	kip
diaphragm span, L =	102.00	ft
roof tributary width for seismic, T_w =	25	ft
tributary seismic load on shear wall, Q_E =	29.2	kip
wall height, h =	10.17	ft
tributary seismic moment on shear wall, Mu =	297.1	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	20.67	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	907.1	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\,d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	110.3	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	139.0	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	28710.0	lbs
vertical compressive stress, $f_{ae} = P/(d*t)$ =	2.4	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.001	
h/L =	0.49	
steel reinforcing ratio, ρ_g =	0.003	
ρ_g*f_{ye}/f_{me} =	0.08	
m-factor =	5.0	(interpolated between LS & IO. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	625.3	kip

demand capacity ratio, DCR = *0.05* ***OK***

Shear wall C

Roof seismic load, V =	119.2	kip
diaphragm span, L =	102.00	ft
roof tributary width for seismic, T_w =	45	ft

tributary seismic load on shear wall, Q_E =	52.6	kip
wall height, h =	10.17	ft
tributary seismic moment on shear wall, Mu =	534.8	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	28.67	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	1229.1	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right] A_n \sqrt{f'_m} + 0.25 P_u\right]$$

masonry shear wall strength, ϕQCE_m =	160.9	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	189.5	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	40788.0	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	2.5	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp} * f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.001	
h/L =	0.35	
steel reinforcing ratio, ρ_g =	0.003	
$\rho_g * f_{ye}/f_{me}$ =	0.08	
m-factor =	5.0	(interpolated between LS & IO. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmφQCE =	852.9	kip

demand capacity ratio, DCR = *0.06* ***OK***

Shear wall D

Roof seismic load, V =	119.2	kip
diaphragm span, L =	102.00	ft
roof tributary width for seismic, T_w =	26	ft
tributary seismic load on shear wall, Q_E =	30.4	kip
wall height, h =	10.17	ft
tributary seismic moment on shear wall, Mu =	309.0	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	26.67	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	1128.1	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right] A_n \sqrt{f'_m} + 0.25 P_u\right]$$

masonry shear wall strength, ϕQCE_m =	145.6	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	174.3	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	17820.0	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	1.2	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.001	
h/L =	0.38	
steel reinforcing ratio, ρ_g =	0.003	
ρ_g*f_{ye}/f_{me} =	0.08	
m-factor =	5.0	(interpolated between LS & IO. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	784.3	kip
demand capacity ratio, DCR =	*0.04*	***OK***

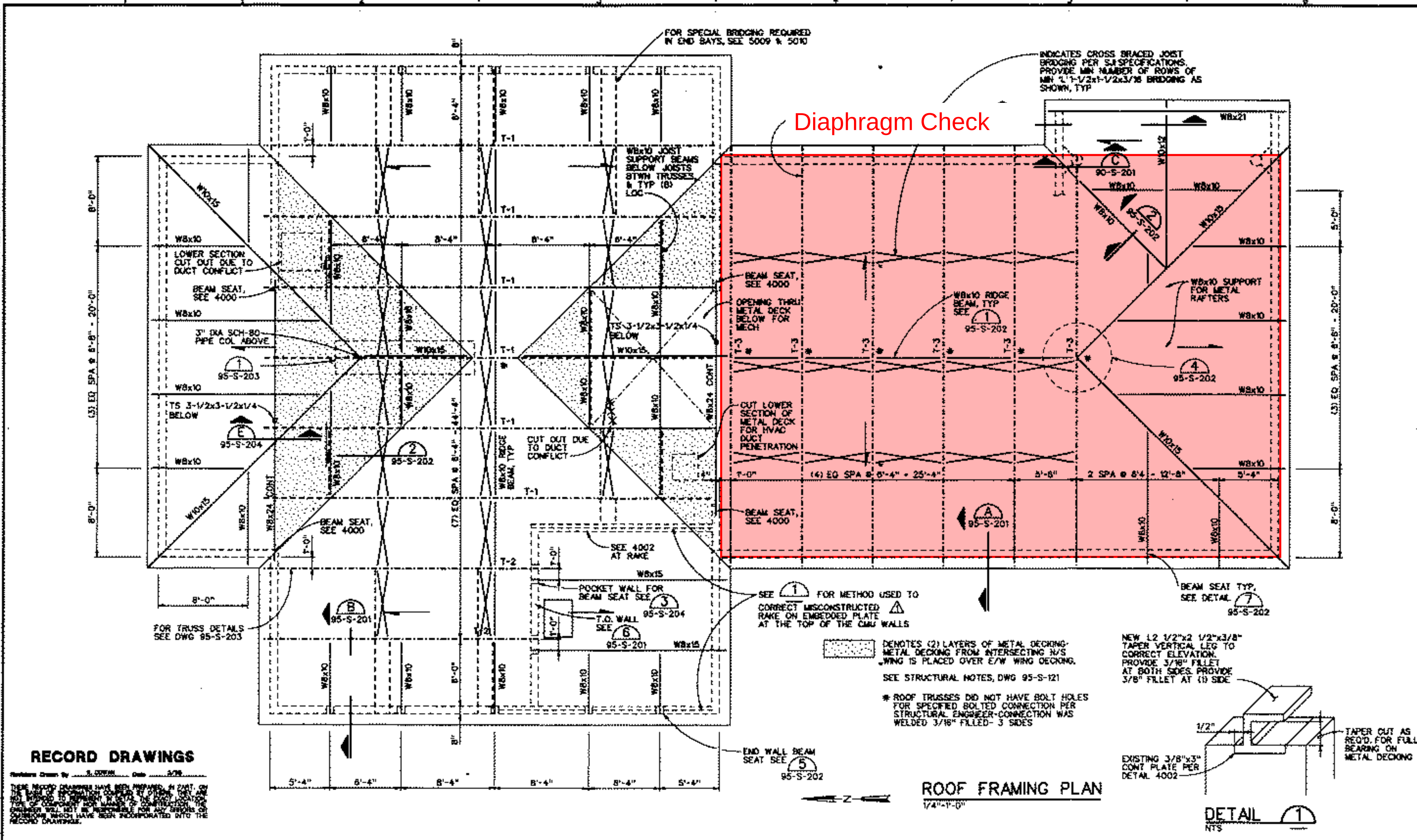
DRAWING NUMBER
93 10 014

WWTP UPGRADE

DRAWING NUMBER
SE 23

459

FILE LOCATION (CVO): CVO1\PRJ1\PRJ\WILSON\DAT\ w195s151.dw



RECORD DRAWINGS

Revisions Drawn By: S. CONWAY Date: 3/98

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The Contract Document Drawings are the printed documents dated DEC 1995 (date on drawing), as subsequently amended, which define the scope, extent, and character of the work. The original Contract Document drawing was sealed and signed by Dennis Doyle, P.E. State of Oregon PE No. 10867

CH2M HILL

DSGN	D. LEEK
DR	CG BELLAMY
CHK	S.M. PITMAN
APVD	M. BRACKEN

NO.	DATE	REVISION	BY	APVD
	5/98	RECORD DRAWINGS	RKC	JBL
	1/97	CL #73 RAKE CORRECTION AT THE OPS BUILDING	RKC	JBL

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CITY OF WILSONVILLE
WASTEWATER TREATMENT PLANT
WILSONVILLE, OREGON

OPERATIONS BUILDING
STRUCTURAL
ROOF FRAMING PLAN

SHEET	80
DWG	95-S-151
DATE	DEC 1995
PROJ NO.	117645.A4

PLOT DATE- 12-MAY-1998 13:38:32 w195s151.dw

Engineers...Working Wonders With Water ™

BY:	BS	**DATE**	Sep-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Operations Building	**JOB NO.**	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (CSZ)		

DIAPHRAGM METAL DECK CHECK

9.10.1.3 Strength of Bare Metal Deck Diaphragms. The strength of bare metal deck diaphragms shall be determined in accordance with Section 9.3.2 and the requirements of this section.

Expected strength, Q_{CE}, for bare metal deck diaphragms shall be taken as 2 times allowable values specified in approved codes and standards, unless a larger value is justified by test data. Alternatively, lower-bound strength shall be taken as nominal strength published in approved codes or standards, except that the strength reduction factor, ϕ, shall be taken as equal to 1.0.

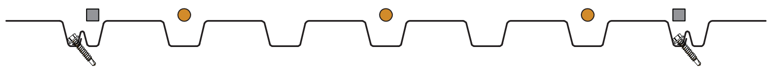
Lower-bound strengths, Q_{CL}, of welded connectors shall be as specified in AWS D1.3, or other approved standard.

Roof seismic load, V =	119.2	kip	
diaphragm span, L =	102.00	ft	
roof unit diaphragm load, v =	1.17	kip/ft	
Roof span between shear walls, L_1 =	50.00	ft	
Roof depth, d =	36.00	ft	
diaphragm shear, v_1 =	0.812	kip/ft	
diaphragm strength, Q_{allow} =	530	lbs/ft	
expected diaphragm strength, Q_{CE} =	1060	lbs/ft	(expected strength shall be 2x the allowable per ASCE 41-17 Section 9.10.1.3)
m-factor =	1.625	(interpolated between LS & IO. ASCE 41-17 Table 9-6)	
knowledge factor, κ =	0.90		
diaphragm strength, $\kappa m \phi Q_{CE}$ =	1.550	kip/ft	

demand capacity ratio, DCR = *0.52* ***OK***

Type HSB®-36-SS

- **36/5 Weld Pattern at Supports**
- **Sidelaps Connected with #10 Screws**



Allowable Diaphragm Shear Strength, q (plf) and Flexibility Factors, F ((in./lb)x10^{6})

DECK GAGE	SIDELAP ATTACHMENT		SPAN (ft-in.) 4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
22	#10 @ 24"	q	431	378	310	289	249	242	218		
		F	-2.3+190R	0.2+152R	2.9+126R	3.9+108R	5.6+94R	6.1+83R	7.4+75R		
	#10 @ 18"	q	480	417	343	317	298	264	257		
		F	-3.3+190R	-0.7+152R	1.8+126R	3+108R	3.8+95R	5.2+84R	5.7+75R		
	#10 @ 12"	q	527	456	408	373	347	329	316		
		F	-4+190R	-1.3+152R	0.5+127R	1.8+109R	2.8+95R	3.5+84R	4.1+76R		
	#10 @ 8"	q	607	565	506	485	445	438	414		
		F	-4.8+191R	-2.5+153R	-0.6+127R	0.4+109R	1.5+95R	2.1+85R	2.8+76R		
	#10 @ 6"	q	682	627	589	561	539	522	509		
		F	-5.4+191R	-2.9+153R	-1.3+127R	-0.1+109R	0.8+95R	1.5+85R	2+76R		
	#10 @ 4"	q	817	769	736	712	693	678	666		
		F	-6+191R	-3.6+153R	-2+127R	-0.9+109R	0+96R	0.7+85R	1.2+76R		
20	#10 @ 24"	q	601	526	433	403	349	335	301	297	272
		F	0.9+120R	2.5+95R	4.5+79R	5.1+68R	6.5+59R	6.7+52R	7.7+47R	7.8+43R	8.6+39R
	#10 @ 18"	q	662	577	476	440	413	363	352	344	315
		F	0+120R	1.7+96R	3.5+79R	4.3+68R	4.8+60R	5.9+53R	6.2+47R	6.4+43R	7.1+39R
	#10 @ 12"	q	716	629	561	513	477	449	430	414	401
		F	-0.6+120R	1.1+96R	2.3+80R	3.2+68R	3.8+60R	4.3+53R	4.8+48R	5.1+43R	5.4+40R
	#10 @ 8"	q	820	760	683	658	606	592	558	554	530
		F	-1.5+121R	0+96R	1.3+80R	2+69R	2.7+60R	3+54R	3.5+48R	3.7+44R	4.1+40R
	#10 @ 6"	q	916	841	788	750	720	697	678	662	649
		F	-2+121R	-0.4+97R	0.7+80R	1.4+69R	2+60R	2.5+54R	2.8+48R	3.1+44R	3.4+40R
	#10 @ 4"	q	1089	1024	979	945	920	899	883	869	857
		F	-2.5+121R	-1+97R	0+81R	0.8+69R	1.3+60R	1.7+54R	2.1+48R	2.4+44R	2.6+40R
18	#10 @ 24"	q	1002	885	731	677	588	562	502	491	450
		F	3.2+58R	4+46R	5.4+38R	5.6+33R	6.6+28R	6.6+25R	7.4+22R	7.4+20R	8+18R
	#10 @ 18"	q	1085	956	797	734	687	606	581	563	516
		F	2.4+58R	3.3+46R	4.5+38R	4.9+33R	5.2+29R	6+25R	6.1+23R	6.2+21R	6.7+19R
	#10 @ 12"	q	1166	1024	925	847	786	738	700	670	647
		F	1.9+58R	2.8+47R	3.5+39R	4+33R	4.3+29R	4.6+26R	4.9+23R	5.1+21R	5.2+19R
	#10 @ 8"	q	1321	1219	1094	1049	973	951	898	886	845
		F	1.1+59R	1.9+47R	2.6+39R	2.9+34R	3.3+29R	3.5+26R	3.8+23R	3.9+21R	4.1+19R
	#10 @ 6"	q	1465	1340	1253	1189	1139	1100	1068	1042	1020
		F	0.7+59R	1.5+47R	2.1+39R	2.5+34R	2.8+29R	3+26R	3.2+24R	3.3+21R	3.4+20R
	#10 @ 4"	q	1721	1615	1540	1484	1441	1407	1379	1356	1337
		F	0.2+59R	1+47R	1.5+39R	1.9+34R	2.1+30R	2.4+26R	2.5+24R	2.7+21R	2.8+20R
16	#10 @ 24"	q	1277	1139	946	884	768	739	661	647	590
		F	3.8+33R	4.3+26R	5.3+21R	5.4+18R	6.2+16R	6.2+14R	6.9+12R	6.8+11R	7.3+10R
	#10 @ 18"	q	1393	1235	1038	963	906	801	771	748	683
		F	3.1+33R	3.7+26R	4.6+22R	4.8+18R	5+16R	5.6+14R	5.7+13R	5.7+12R	6.2+10R
	#10 @ 12"	q	1505	1330	1208	1118	1044	985	937	899	867
		F	2.6+33R	3.2+26R	3.6+22R	4+19R	4.2+16R	4.4+15R	4.6+13R	4.7+12R	4.8+11R
	#10 @ 8"	q	1717	1597	1440	1389	1292	1268	1200	1188	1138
		F	2+33R	2.4+27R	2.9+22R	3+19R	3.3+17R	3.4+15R	3.6+13R	3.6+12R	3.8+11R
	#10 @ 6"	q	1914	1763	1658	1580	1520	1472	1433	1402	1375
		F	1.6+34R	2.1+27R	2.4+22R	2.6+19R	2.8+17R	2.9+15R	3.1+13R	3.2+12R	3.2+11R
	#10 @ 4"	q	2258	2132	2043	1977	1926	1886	1853	1825	1802
		F	1.1+34R	1.6+27R	1.9+22R	2.1+19R	2.3+17R	2.4+15R	2.5+13R	2.6+12R	2.6+11R

See footnotes on page 28.

Deck Span = 6'-8"
q = 530 psf (interpolated)

PROCESS GALLERY - TIER 2 CALCULATIONS

Carollo
Engineers... Working Wonders With Water ™

BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Process Gallery	**JOB NO.**	11962A.00
DESIGN TASK				ASCE 41-17 - Tier 2 (BSE-2E)			

SEISMIC BASE SHEAR FOR PROCESS GALLERY

7.4.1.3.1 Pseudo Seismic Force for LSP. The pseudo lateral force in a given horizontal direction of a building shall be determined using Eq. (7-21). This force shall be used to evaluate or retrofit the vertical elements of the seismic-force-resisting system.

$$V = C_1 C_2 C_m S_a W \tag{7-21}$$

Table 7-3. Alternate Values for Modification Factors C_1C_2

Fundamental Period	$m_{max} < 2$	$2 \le m_{max} < 6$	$m_{max} \ge 6$
$T \le 0.3$	1.1	1.4	1.8
$0.3 < T \le 1.0$	1.0	1.1	1.2
$T > 1.0$	1.0	1.0	1.1

Table 7-4. Values for Effective Mass Factor C_m

No. of Stories	Concrete Moment Frame	Concrete Shear Wall	Concrete Pier-Spandrel	Steel Moment Frame	Steel Concentrically Braced Frame	Steel Eccentrically Braced Frame	Other
1–2	1.0	1.0	1.0	1.0	1.0	1.0	1.0
3 or more	0.9	0.8	0.8	0.9	0.9	0.9	1.0

Note: C_m shall be taken as 1.0 if the fundamental period, T, in the direction of response under consideration is greater than 1.0 s.

spectral response acceleration, S_{xs} =	0.744	g	(BSE-2E seismic hazard)
spectral response acceleration, S_{x1} =	0.405	g	(BSE-2E seismic hazard)
building period, T =	0.114	s	
response spectrum acceleration, S_a =	0.744	g	
effective seismic weight, W =	1267.3	kip	
C_1C_2 =	1.4		(Table 11-6 for masonry walls, m=2.5)
effective mass factor, C_m =	1.0		
seismic lateral force, V =	1320.0	kip	

Story	Weight, w_x (kip)	Floor Height, h_x (ft)	k factor	$w_xh_x^k$ (kip*ft^2)	C_{vx}	Force on Level, F_x (kip)	Story Force, V_j (kip)
Roof	189.8	32.63	1.0	6193.2	0.242	319.5	319.5
1st	1077.5	18.00	1.0	19395.0	0.758	1000.5	1320.0

$\Sigma w_xh_x^k$ = 25588.2

FILE LOCATION (CVO): CVO\PRJ\PRJ\WILSON\DAT\wl98s401.dlv

GENERAL STRUCTURAL NOTES

GENERAL

1. ALL MATERIALS AND WORKMANSHIP SHALL CONFORM TO THE UNIFORM BUILDING CODE, 1994 EDITION (U.B.C.) AS AMENDED BY THE STATE OF OREGON.
2. LOADS: ROOF SNOW LOAD = 25 PSF PLUS DRIFTING
   UBC WIND PRESSURE = 80 MPH WIND SPEED EXPOSURE C, I=1.0
   UBC SEISMIC ZONE = 3, I=1.0, S=1.5
3. REFER TO INDIVIDUAL STRUCTURE DRAWINGS FOR ADDITIONAL LOADS, NOTES, AND REQUIREMENTS.
4. NET ALLOWABLE SOIL BEARING PRESSURE = 4000 PSF
5. DRAINED EQUIVALENT FLUID PRESSURE = 85 PCF/FT AT REST; = 35 PCF/FT ACTIVE
6. UNDRAINED EQUIVALENT FLUID PRESSURE = 100 PCF/FT AT REST; = 85 PCF/FT ACTIVE
7. DATUM: SEE SITE DRAWINGS.
8. NO STRUCTURAL MEMBERS SHALL BE CUT FOR PIPES, DUCTS, ETC. UNLESS SPECIFICALLY DETAILED OR APPROVED IN WRITING BY THE ENGINEER.
9. PROVISIONS FOR FUTURE EXPANSION: NO INTERNAL EXPANSION DETAILED.
10. STANDARD DETAILS AS SHOWN ON THE DRAWINGS ARE INTENDED TO BE TYPICAL AND SHALL APPLY TO ALL SIMILAR SITUATIONS OCCURRING ON THE PROJECT, WHETHER OR NOT THEY ARE KEYED IN EACH LOCATION. CONSULT THE ENGINEER FOR REVIEW PRIOR TO CONSTRUCTION.
11. VISITS TO THE JOB SITE BY THE ENGINEER TO OBSERVE THE CONSTRUCTION DO NOT IN ANY WAY MEAN THAT THEY ARE GUARANTORS OF THE CONSTRUCTOR'S WORK, NOR RESPONSIBLE FOR COMPREHENSIVE OR SPECIAL INSPECTIONS, COORDINATION, SUPERVISION, NOR SAFETY AT THE JOB SITE.
12. SPECIAL INSPECTION (OWNER FURNISHED) IS REQUIRED IN ACCORDANCE WITH UBC SECTION 306 ON THE FOLLOWING PORTIONS OF THE WORK:
    - CONCRETE PLACEMENT
    - REINFORCING STEEL PLACEMENT
    - STRUCTURAL WELDING
    - ANCHORS, EMBEDS AND BOLTS INSTALLED IN CONCRETE
    - HIGH STRENGTH BOLTS
    - GRADING, EXCAVATION, AND FILLING
    - MASONRY CONSTRUCTION WHEN INDICATED
13. ALL SPECIFIED CONCRETE AND GROUT TESTING DURING CONSTRUCTION WILL BE OWNER FURNISHED. ALL SPECIFIED LABORATORY TEST MIXES ARE THE RESPONSIBILITY OF THE CONTRACTOR.
14. VERIFY ALL OPENING DIMENSIONS IN WALLS, SLABS, AND DECKS WITH THE ARCHITECTURAL, MECHANICAL, AND ELECTRICAL DRAWINGS.
15. FOR ABBREVIATIONS NOT LISTED, SEE "ABBREVIATIONS FOR USE ON DRAWINGS AND TEXT", PUBLISHED BY THE AMERICAN NATIONAL STANDARDS INSTITUTE INC. (ANSI).

FOUNDATIONS:

1. PROVIDE AND INSTALL MINIMUM 6 INCHES COMPACTED GRANULAR FILL AS SPECIFIED UNDER ALL SLABS AND FOOTINGS TO UNDISTURBED EARTH.
2. NO BACKFILL SHALL BE PLACED BEHIND WALLS UNTIL THE CONCRETE HAS ATTAINED 100% OF ITS SPECIFIED COMPRESSIVE STRENGTH.
3. WALLS TIED TO ELEVATED FLOOR OR ROOF SLABS SHALL BE BRACED AND REMOVAL OF BRACING FOLLOWED BY BACKFILLING SHALL NOT BE ALLOWED UNTIL THE SLAB IS COMPLETE AND HAS ATTAINED 80% OF ITS SPECIFIED COMPRESSIVE STRENGTH.
4. EXCAVATIONS SHALL BE SHORED AS REQUIRED TO PREVENT SUBSIDENCE OR DAMAGE TO ADJACENT EXISTING STRUCTURES, STREETS, UTILITIES, ETC.
5. ALL SOIL BEARING SURFACES SHALL BE INSPECTED BY THE SOILS ENGINEER PRIOR TO PLACEMENT OF REINFORCING STEEL.
5. THE AERATION BASINS AND ATTACHED PROCESS GALLERY BUILDING AND THE TWO SECONDARY CLARIFIERS SHALL HAVE AN UNDERDRAIN SYSTEM. REFER TO DRAWINGS 05-CY-01 AND 05-CY-02.

CONCRETE:

1. ALL CAST-IN-PLACE CONCRETE SHALL HAVE A MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 4000 PSI.
2. REINFORCING STEEL SHALL CONFORM TO ASTM A615, GRADE 60. REINFORCING TO BE WELDED SHALL CONFORM TO ASTM A706, GRADE 60. FABRICATION AND PLACEMENT OF REINFORCING STEEL SHALL BE IN ACCORDANCE WITH CRSI MSP-1 "MANUAL OF STANDARD PRACTICE" AND ACI 301 "SPECIFICATIONS FOR STRUCTURAL CONCRETE FOR BUILDING".
3. CONSTRUCTION JOINTS INDICATED ARE SUGGESTED LOCATIONS. CONTRACTOR MAY REVISE LOCATION OF JOINTS, SUBJECT TO SPECIFIED REQUIREMENTS, AND SHALL SUBMIT ALL JOINT LOCATIONS FOR REVIEW BY THE ENGINEER. ADDITIONAL CONSTRUCTION JOINT LOCATIONS, AS REQUIRED FOR CONSTRUCTION, SHALL BE SUBMITTED FOR REVIEW.
4. CONTINUOUS GALVANIZED STEEL WATERSTOP AS SPECIFIED SHALL BE INSTALLED IN ALL CONSTRUCTION JOINTS IN WALLS OF WATER HOLDING BASINS AND CHANNELS, EXCEPT WHERE INDICATED OTHERWISE. AT CONTRACTOR'S OPTION, PLASTIC WATERSTOPS MAY BE USED IN PLACE OF GALVANIZED STEEL WATERSTOPS.
5. ROUGHEN AND CLEAN ALL CONSTRUCTION JOINTS IN WALLS AND SLABS AS SPECIFIED PRIOR TO PLACING ADJACENT CONCRETE. SANDBLASTING OR OTHER PREPARATION OF HORIZONTAL AND VERTICAL JOINTS IS REQUIRED AS SPECIFIED.
6. THE CONTRACTOR SHALL COORDINATE PLACEMENT OF ALL OPENINGS, CURBS, DOWELS, SLEEVES, CONDUITS, BOLTS AND INSERTS PRIOR TO PLACEMENT OF CONCRETE.
7. NO ALUMINUM CONDUIT OR PRODUCTS CONTAINING ALUMINUM OR ANY OTHER MATERIAL INJURIOUS TO THE CONCRETE SHALL BE EMBEDDED IN THE CONCRETE.

MASONRY

1. MORTAR SHALL CONFORM TO ASTM C270, TYPE S, HYDRATED. MASONRY CEMENT SHALL NOT BE USED.
2. GROUT SHALL CONFORM TO ASTM C476 COURSE GROUT AND SHALL HAVE A MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 2000 PSI.
3. ALL CONCRETE MASONRY UNITS SHALL BE GRADE N, TYPE I AND HAVE A UNIT COMPRESSIVE MASONRY STRESS OF 1900 PSI, AND SHALL CONFORM TO ASTM C90.
4. THE DESIGN f'm OF THE FINISHED ASSEMBLY SHALL BE 1500 PSI.
5. ALL CELLS IN BUILDING WALLS SHALL BE PARTIALLY GROUTED EXCEPT WHERE INDICATED OTHERWISE.
6. LAP ALL BARS 48 BAR DIAMETERS MINIMUM. STAGGER ALL ADJACENT LAP SPLICES SEPARATED BY 3 INCHES OR LESS, 24 INCHES.
7. PROVIDE FULL HEIGHT VERTICAL BARS AT EDGES OF ALL OPENINGS AND FULL HEIGHT VERTICAL BARS IN 3 CELLS AT ALL CORNERS. PROVIDE MATCHING DOWELS FOR ALL VERTICAL BARS. PROVIDE REINFORCED LINTELS ABOVE AND REINFORCED BOND BEAMS BELOW ALL OPENINGS. PROVIDE HORIZONTAL CORNER BARS WITH MINIMUM 2'-6" LEGS AT ALL CORNERS. SEE DETAILS 4001, 4003, AND 4004.
8. MASONRY UNIT AND GROUT TESTING SHALL BE IN CONFORMANCE WITH 1991 UBC 2406(C)3, "UNIT STRENGTH METHOD". TESTING WILL BE OWNER FURNISHED.
9. THE MINIMUM REINFORCING FOR ALL CONCRETE BLOCK WALLS SHALL BE AS FOLLOWS:

WALL THICKNESS	VERTICAL REINFORCING	HORIZONTAL REINFORCING	LOCATION
8"	#6@32"	#5@48"	CENTERED

PROVIDE LARGER SIZES AND MORE REINFORCING IN ALL WALLS WHERE REQUIRED BY THE DETAILS ON THE DRAWINGS OR BY THE SPECIFICATIONS.

STRUCTURAL STEEL:

1. ALL STRUCTURAL STEEL SHALL CONFORM TO ASTM A-36 UNLESS SHOWN OTHERWISE. SQUARE OR RECTANGULAR STEEL TUBING SHALL CONFORM TO ASTM A-500, GRADE B.
2. ALL STRUCTURAL STEEL SHALL BE FABRICATED AND ERECTED IN CONFORMANCE WITH THE AISC MANUAL OF STEEL CONSTRUCTION, CURRENT EDITION.
3. ALL BOLTS SHALL BE HIGH STRENGTH BOLTS CONFORMING TO ASTM A325-N OR A325-SC UNLESS OTHERWISE SHOWN. BOLTS INDICATED AS MACHINE BOLTS (MB) OR ANCHOR BOLTS (AB) SHALL CONFORM TO ASTM A307 FOR CARBON STEEL, A193 FOR STAINLESS STEEL, AND A153 FOR GALVANIZED STEEL EXCEPT WHERE SPECIFICALLY INDICATED OTHERWISE. ALL JOINT CONTACT SURFACES SHALL BE CLEAN AND FREE FROM OIL, DIRT AND PAINT.
4. ALL WELDS SHALL BE DONE BY AWS CERTIFIED WELDERS AND SHALL CONFORM TO AWS D 1.1, LATEST EDITION. ALL BUTT WELDS ARE FULL PENETRATION UNLESS INDICATED OTHERWISE. WELD FILLER METAL SHALL BE AWS A5.1 OR A5.5 E70XX ELECTRODES.
5. ALL WELDS FOUND DEFECTIVE SHALL BE REPAIRED AND/OR REPLACED AND RETESTED FOR ADEQUACY AT THE CONTRACTOR'S EXPENSE.
6. AT ALL FIELD WELDS, AT EMBED PLATES, AND ANGLES, LOW HEAT AND INTERMITTENT WELDS SHALL BE UTILIZED TO AVOID SPALLING OR CRACKING THE EXISTING CONCRETE.
7. ALL STRUCTURAL STEEL TO BE EMBEDDED IN CONCRETE SHALL BE CLEAN AND FREE OF PAINT, OIL OR DIRT.
8. NO HOLES OTHER THAN THOSE SPECIFICALLY DETAILED SHALL BE ALLOWED THROUGH STRUCTURAL STEEL MEMBERS. NO CUTTING OR BURNING OF STRUCTURAL STEEL WILL BE PERMITTED WITHOUT THE APPROVAL OF THE ENGINEER.

METAL DECKING:

1. SEE ROOF AND ELEVATED FLOOR PLANS FOR DECK SIZE AND WELDING REQUIREMENTS.
2. WELDING SHALL BE IN ACCORDANCE WITH AWS D1.3 "STRUCTURAL WELDING CODE - SHEET STEEL". WELDING FILLER METAL SHALL BE AWS A5.1 OR A5.5 E70XX ELECTRODES. WELDERS SHALL BE AWS CERTIFIED.
3. DECKING SHALL HAVE MINIMUM 2" BEARING ON ALL SUPPORTS.

STEEL JOISTS

1. SEE ROOF PLANS FOR DESIGN LOAD REQUIREMENTS, MINIMUM SIZE, SPACING, AND BRIDGING REQUIREMENTS.
2. MANUFACTURER SHALL BE A MEMBER OF THE STEEL JOIST INSTITUTE (SJI).

REINFORCING NOTES:

1. THE MINIMUM REINFORCING FOR ALL CONCRETE WALLS AND SLABS SHALL BE AS FOLLOWS:

WALL THICKNESS	REINF EACH WAY	LOCATION
6"	#4@12"	CENTERED
8"	#5@12"	CENTERED
10"	#4@12"	EACH FACE
12"	#5@12"	EACH FACE

PROVIDE LARGER SIZES AND MORE REINFORCING IN ALL SECTIONS OF CONCRETE WHERE REQUIRED BY THE DETAILS ON THE DRAWINGS OR BY THE SPECIFICATIONS.

2. CLEARANCE FOR REINFORCEMENT BARS, UNLESS SHOWN OTHERWISE, SHALL BE:
   - WHEN PLACED ON GROUND: ——— 3"
   - ALL OTHER CONCRETE SURFACES:
     - #5 BAR OR SMALLER ——— 1½"
     - #6 BAR OR LARGER ——— 2"
   - INTERIOR BUILDING SLAB SURFACES (NOT OVER WETWELLS) ——— ¾" CLR
3. REFER TO WALL CORNER AND WALL INTERSECTION REINFORCING DETAIL. IN GENERAL, THE WALL CORNER REINFORCING SIZES AND SPACINGS SHALL BE AS CALLED OUT ON THE PLANS AND REFERENCED TO THESE DETAILS AND THE TYPICAL HORIZONTAL WALL REINFORCING SHALL LAP WITH THE HORIZONTAL REINFORCING.
4. ALL BENDS, UNLESS OTHERWISE SHOWN, SHALL BE A 90 DEGREE STANDARD HOOK AS DEFINED IN LATEST EDITION OF ACI 318.
5. ALL WALL CORNER AND WALL INTERSECTION REINFORCEMENT BARS SHALL BE CONTINUOUS AROUND CORNERS AND THROUGH COLUMNS OR PILASTERS. REINFORCEMENT SHALL BE EXTENDED INTO CONNECTING WALLS AND LAPPED ON THE OPPOSITE FACE OF THE CONNECTING WALLS, AS INDICATED ELSEWHERE. ALTERNATE HORIZONTAL BAR LAPS ON OPPOSITE FACES OF WALLS.
6. VERTICAL WALL BARS SHALL BE LAPPED WITH DOWELS FROM BASE SLABS AND EXTENDED INTO THE TOP FACE OF ROOF SLABS AND LAPPED WITH TOP SLAB REINFORCEMENT. PROVIDE A MINIMUM OF TWO FULL HEIGHT VERTICAL BARS WITH MATCHING DOWELS AT WALL ENDS, CORNERS AND INTERSECTIONS WITH SIZE TO MATCH TYPICAL VERTICAL REINFORCING STEEL SHOWN OR REQUIRED BY NOTES ABOVE.
7. UNLESS INDICATED OTHERWISE, CONTRACTOR MAY SPLICE CONTINUOUS SLAB OR LONGITUDINAL BEAM BARS AT LOCATIONS OF HIS CHOOSING, EXCEPT THAT TOP BAR SPLICES SHALL BE LOCATED AT MIDSPAN AND BOTTOM BAR SPLICES SHALL BE LOCATED AT SUPPORTS. ALL REINFORCEMENT BENDS AND LAPS, UNLESS OTHERWISE NOTED, SHALL SATISFY THE FOLLOWING MINIMUM REQUIREMENT:

DETAIL OF REINFORCEMENT - LAP LENGTHS

BAR SIZE		#6 OR SMALLER	#7	#8	#9	#10	#11
CONC DESIGN STRENGTH		▲ 4000 PSI					
GR 40	TOP BAR*	32 DIA, MIN 2'-0"					
	OTHER BAR	22 DIA, MIN 1'-6"					
GR 60	TOP BAR*	48 DIA, MIN 2'-6"	3'-8"	4'-9"	6'-0" ▲	7'-8" ▲	9'-6" ▲
	OTHER BAR	36 DIA, MIN 2'-0"	2'-8"	3'-6"	4'-8" ▲	5'-10" ▲	7'-2" ▲

*TOP BARS SHALL BE DEFINED AS ANY HORIZONTAL BARS PLACED SUCH THAT MORE THAN 12" OF FRESH CONCRETE IS CAST IN THE MEMBER BELOW THE BAR, IN ANY SINGLE POUR. HORIZONTAL WALL BARS ARE CONSIDERED TOP BARS.

8. ▲ INCREASE LAP LENGTHS SHOWN ABOVE BY 25% WHERE BARS ARE SPACED CLOSER THAN 6" O.C. OR LESS THAN 3" CLEAR FROM FACE OF MEMBER TO EDGE BAR MEASURED IN DIRECTION OF SPACING.

RECORD DRAWINGS

Revisions Drawn By: R. COSTAIN Date: 5/98

THESE RECORD DRAWINGS HAVE BEEN PREPARED, IN PART, ON THE BASIS OF INFORMATION COMPILED BY OTHERS. THEY ARE NOT INTENDED TO REPRESENT IN DETAIL THE EXACT LOCATION, TYPE OF COMPONENT NOR MANNER OF CONSTRUCTION. THE ENGINEER WILL NOT BE RESPONSIBLE FOR ANY ERRORS OR OMISSIONS WHICH HAVE BEEN INCORPORATED INTO THE RECORD DRAWINGS.

3000

The Contract Document Drawings are the printed documents dated DEC 1995 (date on drawing), as subsequently amended, which define the scope, extent, and character of the work. The original Contract Document drawing was sealed and signed by (name of sealer) COYLE L. ... State of Oregon PE No. ... (sealer's PE No.).

CH2M HILL

DSGN	D. LEEK
DR	E. PUIG
CHK	SM PITKIN
APVD	W. BRACKEN

NO.	DATE	REVISION	BY	APVD
	5/98	RECORD DRAWINGS	RKC	JBL

REUSE OF DOCUMENTS

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VERIFY SCALE

BAR IS ONE INCH ON ORIGINAL DRAWING.

0 ——— 1"

IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

CITY OF WILSONVILLE
WASTEWATER TREATMENT PLANT
WILSONVILLE, OREGON

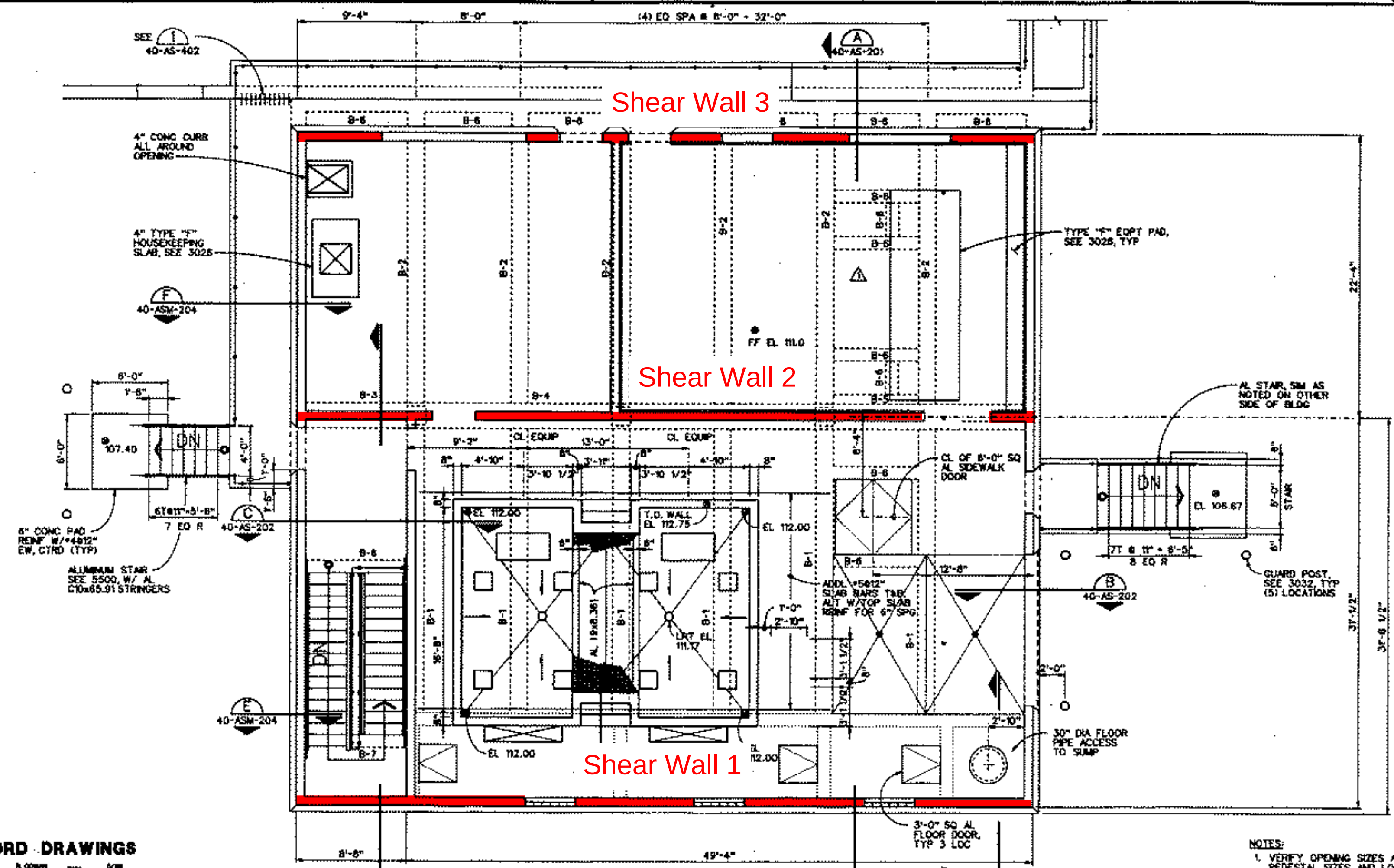
DESIGN DETAILS
STRUCTURAL DETAILS

SHEET	92
DWG NO.	98-S-401
DATE	DEC 1995
PROJ NO.	117645.A4

SF-1

PLOT DATE: 14-MAY-1998 09:40:35 wl98s401.dlv

FILE LOCATION (CVO): CVO1\PROJ\PRJ\WILSON\DAT\ wl40a135.dv



UPPER FLOOR FRAMING PLAN
1/4"=1'-0"

NOTES:
1. VERIFY OPENING SIZES AND EQUIPMENT PEDESTAL SIZES AND LOCATIONS WITH EQUIPMENT MANUFACTURER'S SHOP DRAWINGS.
2. DESIGN FLOOR LOAD = 200 psf
3. SEE STRUCTURAL NOTES 40-AS-151

RECORD DRAWINGS

5/98	RECORD DRAWINGS		
1/97	CL #32A CONDUIT CONCRETE ENCASEMENT		
NO. DATE	REVISION	BY	APVD

REUSE OF DOCUMENTS

CITY OF WILSONVILLE WASTEWATER TREATMENT PLANT WILSONVILLE, OREGON

AERATION BASINS & PROCESS GALLERY
ARCHITECTURAL/STRUCTURAL UPPER FLOOR FRAMING PLAN

SHEET 39
DWG 40-AS-135
DATE DEC 1995
PROJ NO. 117645.A4

PLOT DATE=06-MAY-1998 10:39:18 wl40a135.dv

Engineers...Working Wonders With Water™

BY:	BS	DATE	Sep-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Process Gallery	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (BSE-2E)		

CMU IN-PLANE SHEAR WALL CHECK IN BUILDING N-S DIRECTION

11.3.4.2 Strength of Reinforced Masonry Walls and Wall Piers In-Plane Actions. The strength of existing, retrofitted, and new RM wall or wall pier components in flexure, shear, and axial compression shall be determined in accordance with this section. Design actions (axial, flexure, and shear) on components shall be determined in accordance with Chapter 7 of this standard, considering gravity loads and the maximum forces that can be transmitted based on a limit-state analysis.

11.3.4.2.1 Flexural Strength of Walls and Wall Piers. Expected flexural strength of an RM wall or wall pier shall be determined based on strength design procedures specified in TMS 402.

11.3.4.2.2 Shear Strength of Walls and Wall Piers. The expected and lower-bound shear strength of RM wall or wall pier components shall be determined based on strength design procedures specified in TMS 402.

11.3.4.3 Acceptance Criteria for In-Plane Actions of Reinforced Masonry Walls. The shear required to develop the expected flexural strength of RM walls and wall piers shall be compared with the lower-bound shear strength. For RM wall components governed by flexure, flexural actions shall be considered deformation controlled. For RM components governed by shear, shear actions shall be considered deformation controlled. Axial compression on RM wall or wall pier components shall be considered a force-controlled action.

7.5.2 Linear Procedures

7.5.2.1 Forces and Deformations. Component forces and deformations shall be calculated in accordance with linear analysis procedures of Sections 7.4.1 or 7.4.2.

7.5.2.1.1 Deformation-Controlled Actions for LSP or LDP. Deformation-controlled actions, Q_{UD}, shall be calculated in accordance with Eq. (7-34):

$$Q_{UD} = Q_G + Q_E \qquad (7\text{-}34)$$

where

Q_{UD} = Deformation-controlled action caused by gravity loads and earthquake forces.

Q_G = Action caused by gravity loads as defined in Section 7.2.2; and

Q_E = Action caused by the response to the selected Seismic Hazard Level calculated using either Section 7.4.1 or Section 7.4.2;

Shear wall 1

Roof seismic load, V =	319.5	kip
diaphragm span, L =	52.00	ft
roof tributary width for seismic, T_w =	15	ft
tributary seismic load on shear wall, Q_E =	92.2	kip
wall height, h =	14.63	ft
tributary seismic moment on shear wall, Mu =	1348.4	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	46	ft
vertitcal shear wall grout spacing =	24	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	2077.0	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\,d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	277.0	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	305.7	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	13608.3	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	0.5	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp} \cdot f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.000	

h/L =	0.32	
steel reinforcing ratio, ρ_g =	0.004	
ρ_g*f_{ye}/f_{me} =	0.11	
m-factor =	7.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	1925.7	kip

demand capacity ratio, DCR = 0.05 **OK**

Shear wall 2

Roof seismic load, V =	319.5	kip
diaphragm span, L =	52.00	ft
roof tributary width for seismic, T_w =	26	ft
tributary seismic load on shear wall, Q_E =	159.8	kip
wall height, h =	14.63	ft
tributary seismic moment on shear wall, Mu =	2337.1	kip*ft
masonry strength, f'$_m$ =	1500	psi
shear wall length, d =	48	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	2014.0	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	270.4	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	299.1	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	23369.7	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	0.8	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, f_{me} = F_{exp}*f'$_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.000	
h/L =	0.30	
steel reinforcing ratio, ρ_g =	0.003	
ρ_g*f_{ye}/f_{me} =	0.09	
m-factor =	7.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	1884.2	kip

demand capacity ratio, DCR = 0.08 **OK**

Shear wall 3

Roof seismic load, V =	319.5	kip
diaphragm span, L =	52.00	ft
roof tributary width for seismic, T_w =	10.67	ft
tributary seismic load on shear wall, Q_E =	65.6	kip
wall height, h =	14.63	ft
tributary seismic moment on shear wall, Mu =	959.1	kip*ft
masonry strength, f'$_m$ =	1500	psi
shear wall length, d =	28	ft
vertitcal shear wall grout spacing =	24	in

horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	1291.0	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	154.3	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	183.0	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	9761.4	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	0.6	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.000	
h/L =	0.52	
steel reinforcing ratio, ρ_g =	0.004	
ρ_g*f_{ye}/f_{me} =	0.11	

m-factor =	2.5	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmφQCE =	411.7	kip

demand capacity ratio, DCR = 0.16 ***OK***

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DRAWING NUMBER

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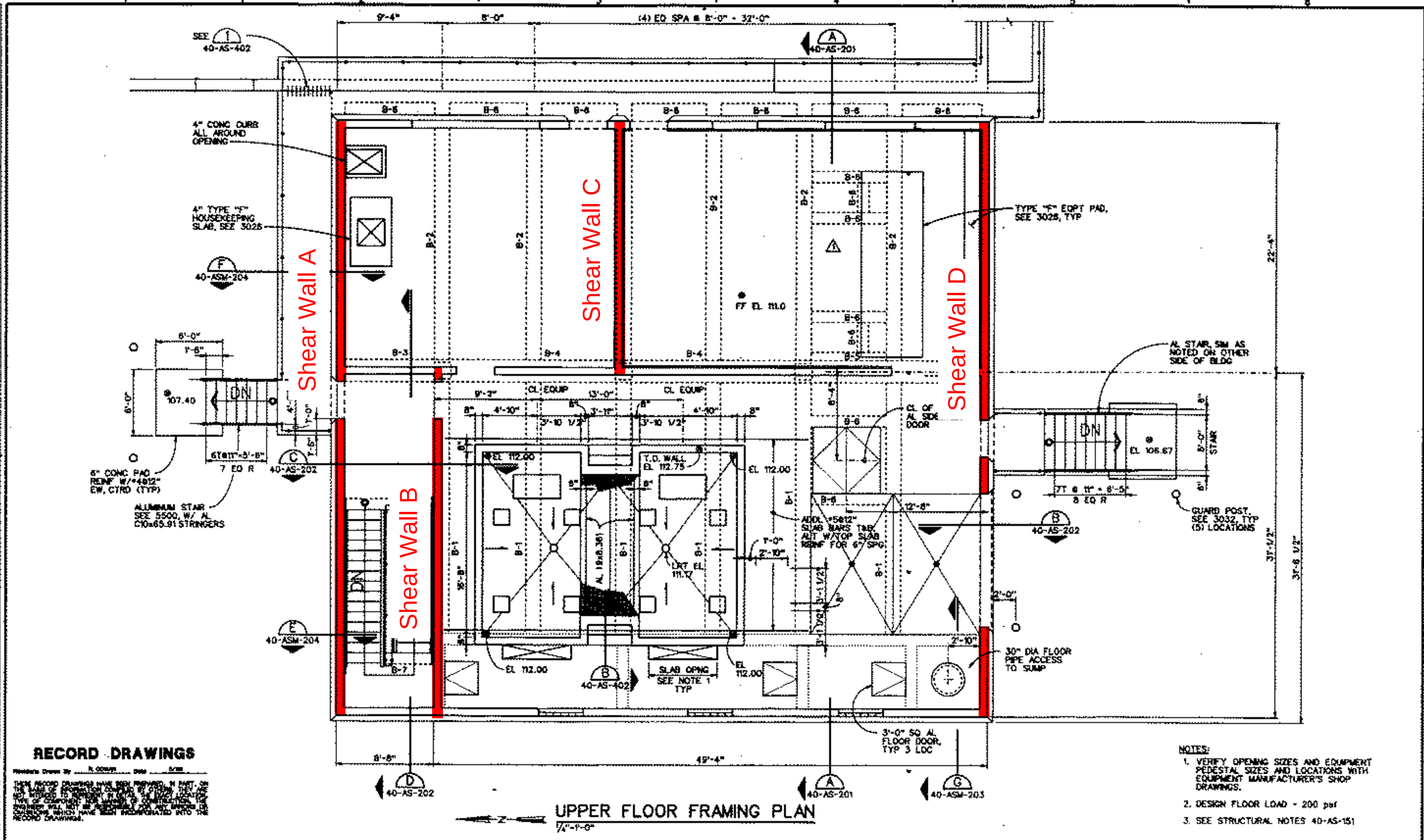

UPPER FLOOR FRAMING PLAN

1/4"=1'-0"

NOTES:

1. VERIFY OPENING SIZES AND EQUIPMENT PEDESTAL SIZES AND LOCATIONS WITH EQUIPMENT MANUFACTURER'S SHOP DRAWINGS.
2. DESIGN FLOOR LOAD = 200 psf
3. SEE STRUCTURAL NOTES 40-AS-151

RECORD DRAWINGS

5/98	RECORD DRAWINGS		
1/97	CL #32A CONDUIT CONCRETE ENCASEMENT	RKC	JBL
DATE	REVISION	BY	APVD

REUSE OF DOCUMENTS

CITY OF WILSONVILLE WASTEWATER TREATMENT PLANT

WILSONVILLE, OREGON

AERATION BASINS & PROCESS GALLERY

ARCHITECTURAL/STRUCTURAL UPPER FLOOR FRAMING PLAN

SHEET	39
DWG	40-AS-135
DATE	DEC 1995
PROJ	117645.A4

PLOT DATE= 06-MAY-1998 10:39:18 wl40s135.dv

Engineers...Working Wonders With Water™

BY:	BS	DATE	Sep-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Process Gallery	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (BSE-2E)		

CMU IN-PLANE SHEAR WALL CHECK IN BUILDING E-W DIRECTION

11.3.4.2 Strength of Reinforced Masonry Walls and Wall Piers In-Plane Actions. The strength of existing, retrofitted, and new RM wall or wall pier components in flexure, shear, and axial compression shall be determined in accordance with this section. Design actions (axial, flexure, and shear) on components shall be determined in accordance with Chapter 7 of this standard, considering gravity loads and the maximum forces that can be transmitted based on a limit-state analysis.

11.3.4.2.1 Flexural Strength of Walls and Wall Piers. Expected flexural strength of an RM wall or wall pier shall be determined based on strength design procedures specified in TMS 402.

11.3.4.2.2 Shear Strength of Walls and Wall Piers. The expected and lower-bound shear strength of RM wall or wall pier components shall be determined based on strength design procedures specified in TMS 402.

11.3.4.3 Acceptance Criteria for In-Plane Actions of Reinforced Masonry Walls. The shear required to develop the expected flexural strength of RM walls and wall piers shall be compared with the lower-bound shear strength. For RM wall components governed by flexure, flexural actions shall be considered deformation controlled. For RM components governed by shear, shear actions shall be considered deformation controlled. Axial compression on RM wall or wall pier components shall be considered a force-controlled action.

7.5.2 Linear Procedures

7.5.2.1 Forces and Deformations. Component forces and deformations shall be calculated in accordance with linear analysis procedures of Sections 7.4.1 or 7.4.2.

7.5.2.1.1 Deformation-Controlled Actions for LSP or LDP. Deformation-controlled actions, Q_{UD}, shall be calculated in accordance with Eq. (7-34):

$$Q_{UD} = Q_G + Q_E \tag{7-34}$$

where

Q_{UD} = Deformation-controlled action caused by gravity loads and earthquake forces.

Q_G = Action caused by gravity loads as defined in Section 7.2.2; and

Q_E = Action caused by the response to the selected Seismic Hazard Level calculated using either Section 7.4.1 or Section 7.4.2;

Shear wall A

Roof seismic load, V =	319.5	kip
diaphragm span, L =	56.67	ft
roof tributary width for seismic, T_w =	4	ft
tributary seismic load on shear wall, Q_E =	22.6	kip
wall height, h =	14.63	ft
tributary seismic moment on shear wall, Mu =	329.9	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	50	ft
vertitcal shear wall grout spacing =	24	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	2238.0	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\, d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	302.3	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	331.0	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	3537.1	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	0.1	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi

f_{ae}/f_{me} =	0.000	
h/L =	0.29	
steel reinforcing ratio, ρ_g =	0.004	
ρ_g*f_{ye}/f_{me} =	0.11	
m-factor =	7.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	2085.3	kip

demand capacity ratio, DCR = 0.01 **OK**

Shear wall B

Roof seismic load, V =	319.5	kip
diaphragm span, L =	56.67	ft
roof tributary width for seismic, T_w =	12	ft
tributary seismic load on shear wall, Q_E =	67.7	kip
wall height, h =	14.63	ft
tributary seismic moment on shear wall, Mu =	989.8	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	26.67	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	1128.1	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	132.8	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	161.5	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	10064.7	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	0.7	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.000	
h/L =	0.55	
steel reinforcing ratio, ρ_g =	0.004	
ρ_g*f_{ye}/f_{me} =	0.11	
m-factor =	7.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	1017.4	kip

demand capacity ratio, DCR = 0.07 **OK**

Shear wall C

Roof seismic load, V =	319.5	kip
diaphragm span, L =	56.67	ft
roof tributary width for seismic, T_w =	24.33	ft
tributary seismic load on shear wall, Q_E =	137.2	kip
wall height, h =	14.63	ft
tributary seismic moment on shear wall, Mu =	2006.8	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	21.33	ft

vertitcal shear wall grout spacing = 32 in
horizontal shear wall grout spacing = 48 in
shear wall thickness, t = 7.625 in
A_n = 926.9 in^2
Φ = 1.0 (assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m = 100.5 kip
horizontal masonry shear wall strength, ϕQCE_s = 28.7 kip
combined masonry shear wall strength, ϕQCE = 129.2 kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P = 19994.8 lbs
vertical compressive stress, f_{ae} = P/(d*t) = 1.6 psi
factor for expected strength, F_{exp} = 1.3 (ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp} * f'_m$ = 1950.0 psi
f_{ae}/f_{me} = 0.001
h/L = 0.69
steel reinforcing ratio, ρ_g = 0.004
$\rho_g * f_{ye}/f_{me}$ = 0.11

m-factor = 7.0 (interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ = 0.90
masonry shear wall strength, κmϕQCE = 813.8 kip

demand capacity ratio, DCR = 0.17 ***OK***

<u>*Shear wall D*</u>

Roof seismic load, V = 319.5 kip
diaphragm span, L = 56.67 ft
roof tributary width for seismic, T_w = 16.33 ft
tributary seismic load on shear wall, Q_E = 92.1 kip

wall height, h = 14.63 ft
tributary seismic moment on shear wall, Mu = 1346.9 kip*ft
masonry strength, f'_m = 1500 psi
shear wall length, d = 38 ft
vertitcal shear wall grout spacing = 24 in
horizontal shear wall grout spacing = 48 in
shear wall thickness, t = 7.625 in
A_n = 1714.0 in^2
Φ = 1.0 (assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m = 220.8 kip
horizontal masonry shear wall strength, ϕQCE_s = 28.7 kip
combined masonry shear wall strength, ϕQCE = 249.5 kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P = 13463.2 lbs
vertical compressive stress, f_{ae} = P/(d*t) = 0.6 psi
factor for expected strength, F_{exp} = 1.3 (ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp} * f'_m$ = 1950.0 psi
f_{ae}/f_{me} = 0.000
h/L = 0.39
steel reinforcing ratio, ρ_g = 0.004

ρ_g*f_{ye}/f_{me} = 0.11

m-factor = 7.0 (interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ = 0.90
masonry shear wall strength, κmφQCE = 1571.7 kip

demand capacity ratio, DCR = 0.06 ***OK***

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PLAN HOLD CORPORATION • IRVINE, CALIFORNIA
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PLAN HOLD CORPORATION • IRVINE, CALIFORNIA

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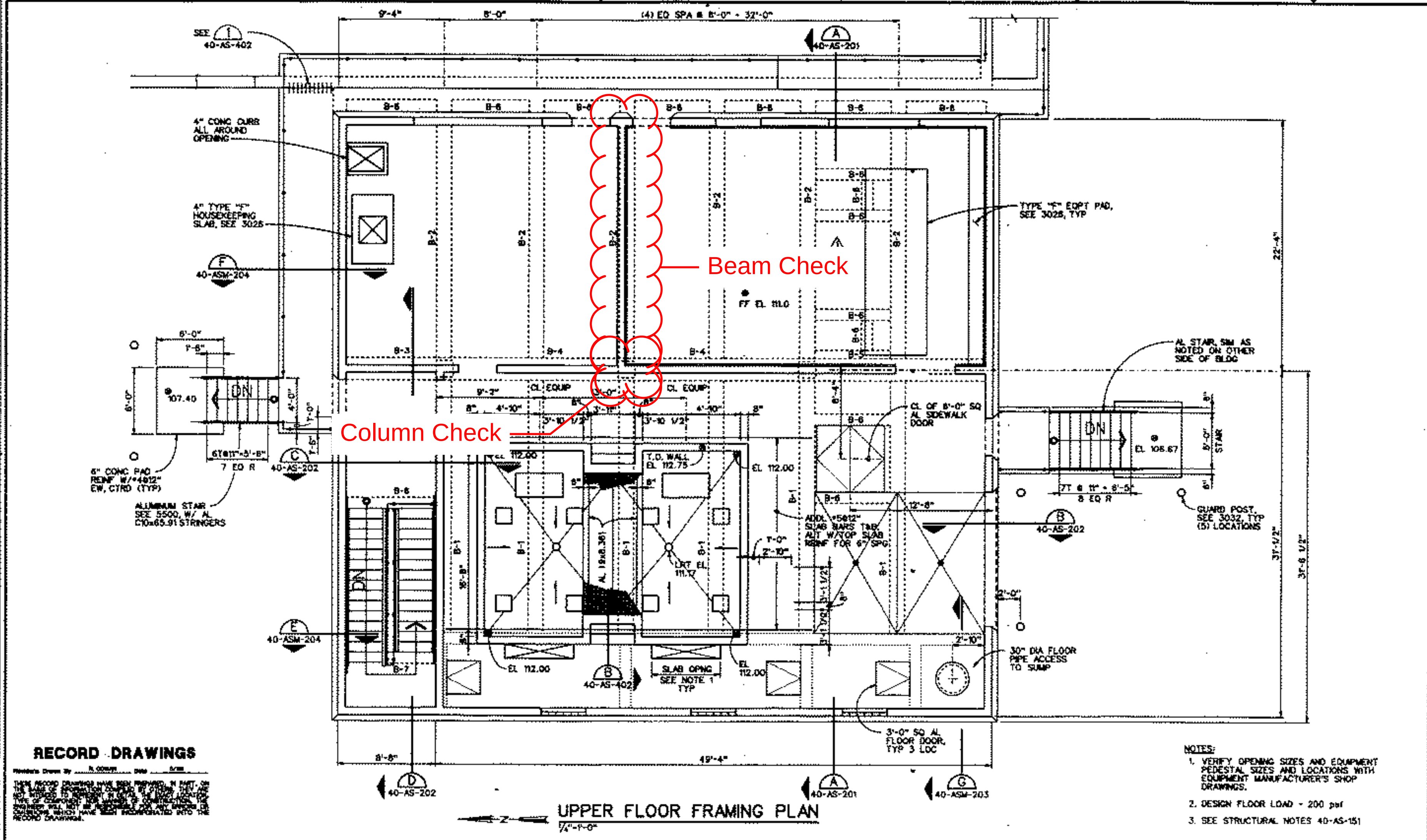

RECORD DRAWINGS

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CITY OF WILSONVILLE
WASTEWATER TREATMENT PLANT
WILSONVILLE, OREGON

AERATION BASINS & PROCESS GALLERY
ARCHITECTURAL/STRUCTURAL
UPPER FLOOR FRAMING PLAN

SHEET 39
DWG 40-AS-135
DATE DEC 1995
PROJ NO. 117645.A4

5/98 RECORD DRAWINGS
1/97 CL #32A CONDUIT CONCRETE ENCASEMENT

PLOT DATE=06-MAY-1998 10:39:18 wi40a135.dv

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PLAN HOLD CORPORATION • IRVINE, CALIFORNIA

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PLAN HOLD CORPORATION • IRVINE, CALIFORNIA

FILE LOCATION (CVO): CVO\PROJ\PRJ\WILSON\DAT\ w40s401.dv

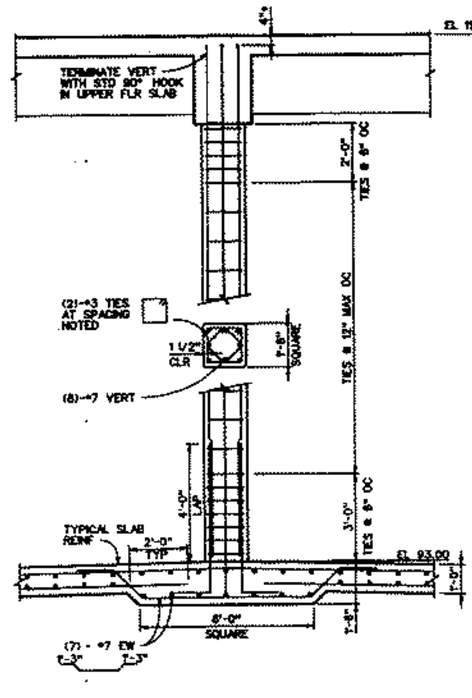
BEAM SCHEDULE

BEAM NO. (SEE PLANS)	SIZE W	SIZE **D	TOP REINF AT LEFT SUPPORT T	TOP REINF AT LEFT SUPPORT TC	BOTTOM REINF B	BOTTOM REINF BC	* TOP REINF AT RIGHT SUPPORT T	* TOP REINF AT RIGHT SUPPORT TC	S BARS	SHEAR REINFORCEMENT NO. SIZE	SHEAR REINFORCEMENT SPACING FROM LEFT SUPPORT	REMARKS
B-1	18"	32"	-	2-#7	-	2-#8	2-#7	2-#7	2-#5	2-#4	[illegible]	
B-2	18"	32"	2-#7	2-#7	-	2-#8	-	2-#7	2-#5	2-#4	1@3", 10@6", 5@12", 8@6", 1@3"	
B-3	24"	36"	-	2-#8	-	2-#9	2-#8	2-#9	2-#6	2-#4	2@3", 5@12", 2@3"	
B-4	24"	36"	2-#8	2-#9	-	3-#9	2-#8	2-#8	2-#6	2-#4	1@3", 24@6", 1@3"	
B-5	24"	36"	2-#8	2-#9	-	4-#8	2-#9	-	2-#6	2-#4	10@3", 5@6", 8@12", 2@3"	
B-6	12"	24"	-	2-#5	-	2-#5	-	2-#5	1-#5	2-#4	1@3", 5@12", 1@3"	
B-7	12"	18"	-	2-#5	-	2-#5	-	2-#5	1-#5	2-#4	1@3", 5@12", 1@3"	

NOTES:

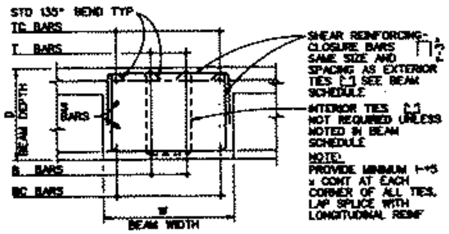
*1 TOP REINFORCEMENT MAY BE CALLED-OUT TWICE IN SCHEDULE (I.E. "TOP REINF. AT RIGHT SUPPORT" OF BEAM THAT IS CONTINUOUS OVER THE RIGHT SUPPORT IS CALLED-OUT AS "TOP REINF. AT LEFT SUPPORT" OF ADJACENT BEAM.).

LEFT SUPPORT IS DESIGNATED AS THE SUPPORT CLOSEST THE LEFT SIDE OR BOTTOM OF SHEET ON WHICH FRAMING PLAN IS DRAWN, UNLESS NOTED OTHERWISE ON PLAN.



COLUMN/FTG DETAIL 1 / 40-AS-121

1/2"=1'-0"



TYPICAL BEAM SECTION

NTS

RECORD DRAWINGS

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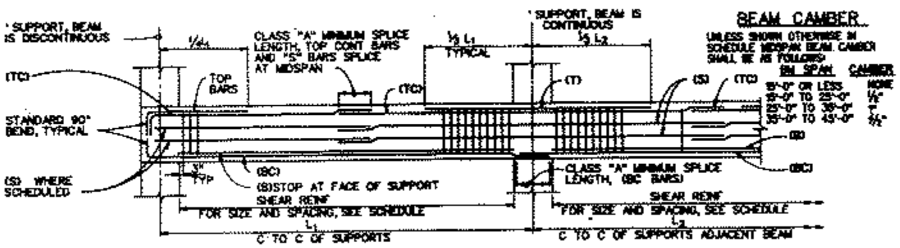

BEAM CAMBER

UNLESS SHOWN OTHERWISE IN SCHEDULE MIDSPAN BEAM CAMBER SHALL BE AS FOLLOWS:

BM SPAN	CAMBER
15'-0" OR LESS	NONE
15'-0" TO 25'-0"	1/4"
25'-0" TO 35'-0"	1/2"
35'-0" TO 45'-0"	3/4"

TYPICAL BEAM REINFORCING

NTS

The Contract Document Drawings are the printed documents dated DEC 1995 ... The original Contract Document drawing was sealed and signed by ... State of Oregon PE No. ...

CH2M HILL

DSGN	D LEEK
DR	CS BELLAMY
CHK	SM PITTON
APVD	M BRACKEN

NO.	DATE	REVISION	BY	APVD
	3/98	RECORD DRAWINGS		

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VERIFY SCALE

CITY OF WILSONVILLE
WASTEWATER TREATMENT PLANT
WILSONVILLE, OREGON

AERATION BASINS & PROCESS GALLERY

ARCHITECTURAL/STRUCTURAL DETAILS & BEAM SCHEDULE

SHEET 46

DWG 40-AS-401

DATE DEC 1995

PROJ NO. 117645.A4

PLOT DATE: 06-MAY-1998 13:18:13 w40s401.dv

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Engineers... Working Wonders With Water ®

BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Process Gallery	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (BSE-2E)		

BEAM AND COLUMN CHECK SUPPORTING CMU WALL ABOVE (VERTICAL IRREGULARITY TIER 1 FINDING)

5.4.2.3 Vertical Irregularities. An analysis shall be performed in accordance with Section 5.2.4, and the demand-capacity ratio (DCR) shall be determined in accordance with Section 7.3.1.1 for all elements of the seismic-force-resisting system in the noncompliant stories. The adequacy of the elements and connections below the vertical discontinuities shall be evaluated in accordance with Section 5.2.5 as force-controlled elements. The adequacy of struts and diaphragms to transfer loads to adjacent seismic-force-resisting elements as force-controlled elements shall be evaluated.

7.5.2.1.2 Force-Controlled Actions for LSP or LDP. Force-controlled actions, Q_{UF}, shall be calculated using one of the following methods:

1. Q_{UF} shall be taken as the maximum action that can be developed in a component based on a limit-state analysis considering the expected strength of the components delivering force to the component under consideration, or the maximum action developed in the component as limit-ed by the nonlinear response of the building.
2. Alternatively, Q_{UF} shall be calculated in accordance with Eq. (7-35).

$$Q_{UF} = Q_G \pm \frac{\chi Q_E}{C_1 C_2 J} \qquad (7\text{-}35)$$

Roof seismic load, V =	319.5	kip	
diaphragm span, L =	56.67	ft	
roof tributary width for seismic, T_w =	24.33	ft	
wall height, h =	14.63	ft	
tributary seismic load on shear wall, V_E =	137.2	kip	
seismic overturning on shear wall, M_E =	2006.8	kip*ft	
Wall length, L_w =	21.33	ft	
Factor for adjusting action, χ =	1.15	(interpolated between LS & CP)	
C_1C_2 =	1.4		
Force delivery reduction factor, J =	2		

Beam B2 (16"x32") Check

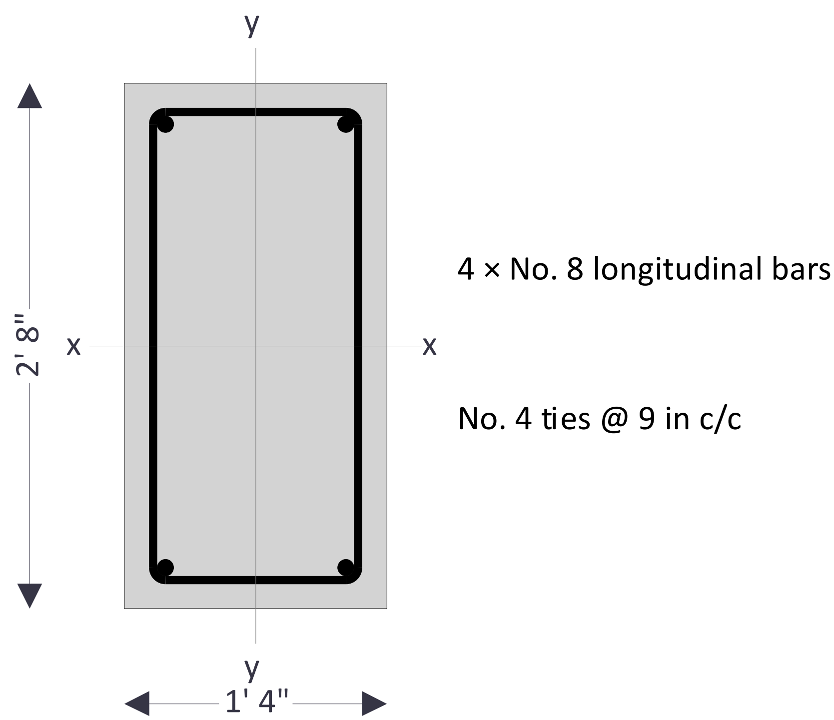
Roof unit weight, w_{Droof} =	15.3	psf	
Wall unit weight, w_{Dwall} =	47.0	lb/ft	
Floor unit weight, w_{Dfloor} =	183	psf	
Floor unit live load, w_{Lfloor} =	200	psf	
Tributary width to beam, T_{wbeam} =	8	ft	
supported gravity loads on beam, Q_D =	1633.4	lb/ft	
supported live loads on beam, Q_L =	400	lb/ft	(assume only 25% of LL)
supported combined loads on beam, Q_G =	2236.7	lb/ft	
Axial load on beam, Q_{UF} =	56.3	kip	
Bending moment demand on beam, Q_{UF} =	127.2	kip*ft	
Shear demand on beam, Q_{UF} =	23.9	kip	
Beam axial strength, Q_{CL} =	1534.8	kip	(From TEDDS calculation)
Beam bending strength, Q_{CL} =	296.6	kip*ft	(From TEDDS calculation)
Beam shear strength, Q_{CL} =	58.7	kip	(From TEDDS calculation)
knowledge factor, κ =	0.90		
Beam axial strength, κ*Q_{CL} =	1381.32	kip	
Beam bending strength, κ*Q_{CL} =	266.94	kip*ft	
Beam shear strength, κ*Q_{CL} =	52.83	kip	
Axial DCR =	*0.04*	***OK***	
Moment DCR =	*0.48*	***OK***	
Shear DCR =	*0.45*	***OK***	

Column (18"x18") Check

Roof unit weight, w_{Droof} =	15.3	psf	
Wall unit weight, w_{Dwall} =	47.0	lb/ft	
Floor unit weight, w_{Dfloor} =	183	psf	
Floor unit live load, w_{Lfloor} =	200	psf	
Tributary area to column, $T_{wcolumn}$ =	213.36	ft^2	
supported gravity loads on column, Q_D =	43.2	kip	
supported live loads on column, Q_L =	10.7	kip	(assume only 25% of LL)
supported combined loads on column, Q_G =	59.2	kip	
supported overturning loads on column, Q_E =	38.6	kip	
Axial compression load on column, Q_{UFcomp} =	97.9	kip	
Axial tension load on column, Q_{UFten} =	0.2	kip	
Bending moment demand on column, Q_{UF} =	48.9	kip*ft	
Shear demand on column, Q_{UF} =	6.9	kip	
Column axial strength, Q_{CL} =	1099.1	kip	(From TEDDS calculation)
Column bending strength, Q_{CL} =	230.5	kip*ft	(From TEDDS calculation)
Column shear strength, Q_{CL} =	33.8	kip	(From TEDDS calculation)
knowledge factor, κ =	0.90		
Column axial strength, κ*Q_{CL} =	989.19	kip	
Column bending strength, κ*Q_{CL} =	207.45	kip*ft	
Column shear strength, κ*Q_{CL} =	30.42	kip	
Axial DCR =	*0.10*	***OK***	
Moment DCR =	*0.24*	***OK***	
Shear DCR =	*0.23*	***OK***	

RC RECTANGULAR COLUMN DESIGN (ACI318-14)

Tedds calculation version 2.2.02



Applied loads

Ultimate axial force acting on column	P_{u_act} = **56.3** kips
Ultimate smaller end moment about x axis	M_{1x_act} = **0.001** kips_ft
Ultimate larger end moment about x axis	M_{2x_act} = **127.2** kips_ft
Column curvature about x axis	single curvature
Ratio of DL moment to total moment	β_d = **0.600**

Geometry of column

Depth of column (larger dimension of column)	h = **32.0** in
Width of column (smaller dimension of column)	b = **16.0** in
Clear cover to reinforcement (both sides)	c_c = **1.5** in
Unsupported height of column about x axis	l_{ux} = **21.3** ft
Effective height factor about x axis	k_x = **1.00**
Column state about the x axis	Braced
Unsupported height of column about y axis	l_{uy} = **21.3** ft
Effective height factor about y axis	k_y = **1.00**
Column state about the y axis	Braced

Check on overall column dimensions

Column dimensions are OK - h < 4b

Reinforcement of column

Numbers of bars of longitudinal steel	N = **4**
Longitudinal steel bar diameter number	D_{bar_num} = **8**

Diameter of longitudinal bar D_{long} = **1.000** in

Stirrup bar diameter number D_{stir_num} = **4**

Diameter of stirrup bar D_{stir} = **0.500** in

Specified yield strength of reinforcement f_y = **60000** psi

Specified compressive strength of concrete f'_c = **4000** psi

Modulus of elasticity of bar reinforcement $E_s = \mathbf{29 \times 10^6}$ psi

Modulus of elasticity of concrete $E_c = 57000 \times f'_c{}^{1/2} \times (1psi)^{1/2}$ = **3604997** psi

Yield strain $\varepsilon_y = f_y / E_s$ = **0.00207**

Ultimate design strain ε_c = **0.003** in/in

Check for minimum area of steel - 10.6.1.1

Gross area of column $A_g = h \times b$ = **512.000** in^2

Area of steel $A_{st} = N \times (\pi \times D_{long}^2) / 4$ = **3.142** in^2

Minimum area of steel required $A_{st\_min} = 0.01 \times A_g$ = **5.120** in^2

$A_{st} < A_{st\_min}$, FAIL- Minimum steel check

Check for maximum area of steel - 10.6.1.1

Permissible maximum area of steel $A_{st\_max} = 0.08 \times A_g$ = **40.960** in^2

$A_{st} < A_{st\_max}$, PASS - Maximum steel check

Slenderness check about x axis

Radius of gyration $r_x = 0.3 \times h$ = **9.6** in

Actual slenderness ratio $s_{rx\_act} = k_x \times l_{ux} / r_x$ = **26.66**

Permissible slenderness ratio $s_{rx\_perm} = \min(34 - 12 * (M_{1x\_act} / M_{2x\_act}),40)$ = **34**

Slenderness effects may be neglected about the X axis

Slenderness check about y axis

Radius of gyration $r_y = 0.3 \times b$ = **4.8** in

Actual slenderness ratio $s_{ry\_act} = k_y \times l_{uy} / r_y$ = **53.33**

Permissible slenderness ratio $s_{ry\_perm} = \min(34 - 12 * (M_{1y\_act} / M_{2y\_act}),40)$ = **34**

Column is slender about the Y axis

Magnified moments about y axis

Moment of inertia of section $I_{gy} = (h \times b^3) / 12$ = **10922.667** in^4

Euler's buckling load $P_{cy} = (\pi^2 / (k_y \times l_{uy})^2) \times (0.4 \times E_c \times I_{gy} / (1+\beta_d))$ = **1482.96** kips

Correction factor for actual to equiv. mmt.diagram C_{my} = 1.0

Moment magnifier $\delta_{nsy} = \max(C_{my} / (1 - (P_{u\_act} / (0.75 \times P_{cy}))),1.0)$ = **1.053**

Minimum factored moment about y axis $M_{2y\_min} = P_{u\_act} \times (0.6 \text{ in} + 0.03 \times b)$ = **5.07** kip_ft

Minimum magnified moment about y axis $M_{cy\_min} = \delta_{nsy} \times M_{2y\_min}$ = **5.34** kip_ft

Axial load capacity of axially loaded column

Strength reduction factor ϕ = 1.00

Area of steel on compression face $A'_s = A_{st} / 2$ = **1.571** in^2

Area of steel on tension face $A_s = A_{st} / 2$ = **1.571** in^2

Net axial load capacity of column $P_n = 0.8 \times (0.85 \times f'_c \times (A_g - A_{st}) + f_y \times A_{st})$ = **1534.891** kips

Ultimate axial load capacity of column $P_u = \phi \times P_n$ = **1534.891** kips

PASS : Column is safe in axial loading

Net moments for biaxial column

Assuming strength reduction factor $\phi = 0.65$

Net moment about major (X) axis $M_{nx} = M_{ux\_act} / \phi =$ **0** kips_ft

Net moment about minor (Y) axis $M_{ny} = M_{cy\_min} / \phi =$ **8.21** kips_ft

Uniaxially loaded column about major axis

Details of column cross-section

c/d_t ratio $r_{xb} =$ **0.102**

Effective cover to reinforcement $d' = c_c + D_{stir} + (D_{long}/2) =$ **2.500** in

Spacing between bars $s = ((h - (2\times d')))/((N/2)-1) =$ **27.000** in

Depth of tension steel $d_t = h - d' =$ **29.500** in

Depth of NA from extreme compression face $c_x = r_{xb} \times d_t =$ **3.008** in

Factor of depth of compressive stress block $\beta_1 =$ **0.850**

Depth of equivalent rectangular stress block $a_x = \min((\beta_1 \times c_x), h) =$ **2.557** in

Yield strain in steel $\varepsilon_{sx} = f_y / E_s =$ **0.002**

Strength reduction factor $\phi_x =$ **0.900**

Details of concrete block

Force carried by concrete

Forces carried by concrete $P_{xcon} = 0.85 \times f'_c \times b \times a_x =$ **139.079** kips

Moment carried by concrete

Moment carried by concrete $M_{xcon} = P_{xcon} \times ((h/2) - (a_x/2)) =$ **170.624** kip_ft

Details of steel layer 1

Depth of layer $x_{x1} =$ **2.500** in

Strain of layer $\varepsilon_{x1} = \varepsilon_c * (1 - x_{x1} / c_x) =$ **0.00051**

Stress in layer $\sigma_{x1} = \min(f_y, E_s * \varepsilon_{x1}) - 0.85 * f'_c =$ **11287.26** psi

Force carried by layer $P_{x1} = N_x * A_{bar} * \sigma_{x1} =$ **17.730** kips

Moment carried by steel layer $M_{x1} = P_{x1} * ((h / 2) - x_{x1}) =$ **19.946** kip_ft

Details of steel layer 2

Depth of layer $x_{x2} =$ **29.500** in

Strain of layer $\varepsilon_{x2} = \varepsilon_c * (1 - x_{x2} / c_x) =$ **-0.02642**

Stress in layer $\sigma_{x2} = \max(-1 * f_y, E_s * \varepsilon_{x2}) =$ **-60000.00** psi

Force carried by layer $P_{x2} = N_x * A_{bar} * \sigma_{x2} =$ **-94.248** kips

Moment carried by steel layer $M_{x2} = P_{x2} * ((h / 2) - x_{x2}) =$ **106.029** kip_ft

Force carried by steel

Sum of forces by steel $P_{xs} =$ **-76.5** kips

Total force carried by column

Nominal axial load strength $P_{nx} =$ **62.561** kips

Strength reduction factor $\phi_x =$ **1.000**

Ultimate axial load carrying capacity of column $P_{ux} = \phi_x \times P_{nx} =$ **56.305** kips

Total moment carried by column

Total moment carried by column $M_{ox} =$ **296.599** kip_ft

Ultimate moment strength capacity of column $M_{ux} = \phi_x \times M_{ox} =$ **296.599** kip_ft

Equivalent required uniaxial moment about x axis

Equivalent required uniaxial nominal moment $M_{nxe} = M_{nx} + M_{ny} \times h / b \times ((1 - \beta) / \beta) =$ **16.422** kip_ft

Equivalent required uniaxial ultimate moment $M_{uxe} = M_{nxe} \times \phi_x =$ **14.780** kip_ft

Check load capacity about the x axis

Factored axial load $P_{u\_act} =$ **56.3** kips

Ultimate axial capacity $P_{ux} =$ **56.3** kips

PASS - Ultimate axial capacity exceeds factored axial load

Equivalent required uniaxial factored moment $M_{uxe} =$ **14.8** kip_ft

Ultimate moment capacity about the x axis $M_{ux} =$ **266.9** kip_ft

PASS - Ultimate moment capacity exceeds factored moment about x axis

Uniaxially loaded column about minor axis

Details of column cross-section

c/d_t ratio $r_{yb} =$ **0.151**

Effective cover to reinforcement $d' = c_c + D_{stir} + (D_{long}/2) =$ **2.500** in

Spacing between bars $s = ((b – (2\times d')))/ ((N/2)-1) =$ **11.000** in

Depth of tension steel $b_t = b - d' =$ **13.500** in

Depth of NA from extreme compression face $c_y = r_{yb} \times b_t =$ **2.034** in

Factor of depth of compressive stress block $\beta_1 =$ **0.850**

Depth of equivalent rectangular stress block $a_y = \min((\beta_1 \times c_y), b) =$ **1.729** in

Yield strain in steel $\varepsilon_{sy} = f_y / E_s =$ **0.002**

Strength reduction factor $\phi_y =$ **0.900**

Details of concrete block

Force carried by concrete

Forces carried by concrete $P_{ycon} = 0.85 \times f'_c \times h \times a_y =$ **188.109** kips

Moment carried by concrete

Moment carried by concrete $M_{ycon} = P_{ycon} \times ((b/2) – (a_y/2)) =$ **111.855** kip_ft

Details of steel layer 1

Depth of layer $x_{y1} =$ **2.500** in

Strain of layer $\varepsilon_{y1} = \varepsilon_c * (1 - x_{y1} / c_y) =$ **-0.00069**

Stress in layer $\sigma_{y1} = \max(-1 * f_y, E_s * \varepsilon_{y1}) =$ **-19929.49** psi

Force carried by layer $P_{y1} = N_y * A_{bar} * \sigma_{y1} =$ **-31.305** kips

Moment carried by steel layer $M_{y1} = P_{y1} * ((b / 2) - x_{y1}) =$ **-14.348** kip_ft

Details of steel layer 2

Depth of layer $x_{y2} =$ **13.500** in

Strain of layer $\varepsilon_{y2} = \varepsilon_c * (1 - x_{y2} / c_y) =$ **-0.01691**

Stress in layer $\sigma_{y2} = \max(-1 * f_y, E_s * \varepsilon_{y2}) =$ **-60000.00** psi

Force carried by layer $P_{y2} = N_y * A_{bar} * \sigma_{y2} =$ **-94.248** kips

Moment carried by steel layer $M_{y2} = P_{y2} * ((b / 2) - x_{y2}) =$ **43.197** kip_ft

Force carried by steel

Sum of forces by steel P_{ys} = **-125.6** kips

Total force carried by column

Nominal axial load strength P_{ny} = **62.556** kips

Strength reduction factor ϕ_y = **1.000**

Ultimate axial load carrying capacity of column $P_{uy} = \phi_y \times P_{ny}$ = **62.556** kips

Moment carried by biaxial column minor axis

Nominal moment strength M_{oy} = **140.703** kip_ft

Contour beta factor

Contour beta factor β = **0.500**

M_{nx}_upon_M_{ox1} = M_{nx} / M_{ox} = **0.000**

M_{ny}_upon_M_{oy} = **1.000**

Net moment along minor axis resisted by column M_{ny1} = M_{oy} ×(M_{ny}_upon_M_{oy}) = **140.703** kip_ft

Ultimate moment along minor axis $M_{uy} = M_{ny1} \times \phi_y$ = **126.633** kip_ft

Check load capacity about the y axis

Factored axial load P_{u_act} = **56.3** kips

Ultimate axial capacity P_{uy} = **56.3** kips

PASS - Ultimate axial capacity exceeds factored axial load

Factored moment about the y axis $M_{uy\_max} = \phi * M_{ny}$ = **5.3** kip_ft

Ultimate moment capacity about the y axis M_{uy} = **126.6** kip_ft

PASS - Ultimate moment capacity exceeds factored moment about y axis

Design of column ties - 25.7.2

Spacing of lateral ties s_{v_ties} = **9.000** in

16 times longitudinal bar diameter $s_{v1} = 16 \times D_{long}$ = **16.000** in

48 times tie bar diameter $s_{v2} = 48 \times D_{stir}$ = **24.000** in

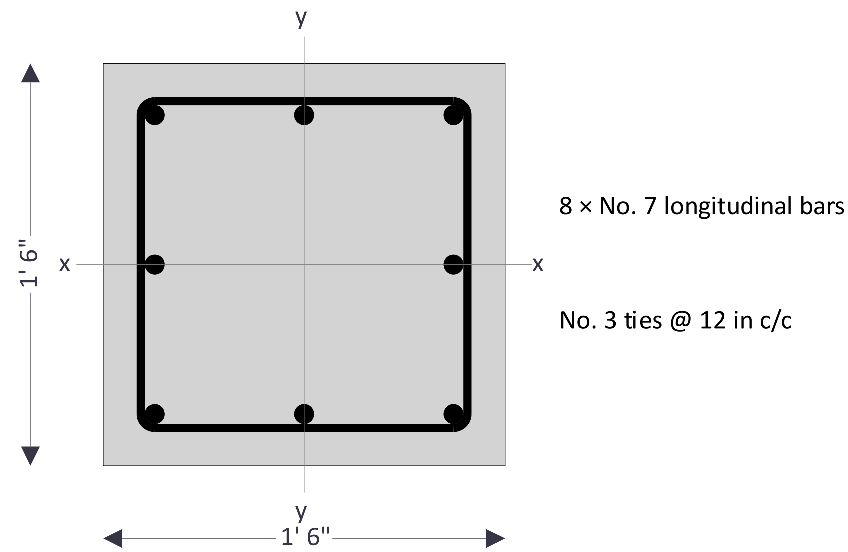
Least column dimension s_{v3} = min (h,b) = **16.000** in

Required tie spacing s = min(s_{v1},s_{v2},s_{v3}) = **16.000** in

s_{v_ties} < s PASS

RC RECTANGULAR COLUMN DESIGN (ACI318-14)

Tedds calculation version 2.2.02



Applied loads

Ultimate axial force acting on column	P_{u_act} = **97.9** kips
Ultimate smaller end moment about x axis	M_{1x_act} = **0.001** kips_ft
Ultimate larger end moment about x axis	M_{2x_act} = **48.9** kips_ft
Column curvature about x axis	single curvature
Ratio of DL moment to total moment	β_d = **0.600**

Geometry of column

Depth of column (larger dimension of column)	h = **18.0** in
Width of column (smaller dimension of column)	b = **18.0** in
Clear cover to reinforcement (both sides)	c_c = **1.5** in
Unsupported height of column about x axis	l_{ux} = **18.0** ft
Effective height factor about x axis	k_x = **1.00**
Column state about the x axis	Braced
Unsupported height of column about y axis	l_{uy} = **18.0** ft
Effective height factor about y axis	k_y = **1.00**
Column state about the y axis	Braced

Check on overall column dimensions

Column dimensions are OK - h < 4b

Reinforcement of column

Numbers of bars of longitudinal steel	N = **8**
Longitudinal steel bar diameter number	D_{bar_num} = **7**
Diameter of longitudinal bar	D_{long} = **0.875** in
Stirrup bar diameter number	D_{stir_num} = **3**
Diameter of stirrup bar	D_{stir} = **0.375** in
Specified yield strength of reinforcement	f_y = **60000** psi

Specified compressive strength of concrete f'_c = **4000** psi

Modulus of elasticity of bar reinforcement E_s = **29 × 10⁶** psi

Modulus of elasticity of concrete $E_c = 57000 \times f'_c{}^{1/2} \times (1psi)^{1/2}$ = **3604997** psi

Yield strain $\varepsilon_y = f_y / E_s$ = **0.00207**

Ultimate design strain ε_c = **0.003** in/in

Check for minimum area of steel - 10.6.1.1

Gross area of column $A_g = h \times b$ = **324.000** in²

Area of steel $A_{st} = N \times (\pi \times D_{long}^2) / 4$ = **4.811** in²

Minimum area of steel required $A_{st\_min} = 0.01 \times A_g$ = **3.240** in²

$A_{st} > A_{st\_min}$, PASS- Minimum steel check

Check for maximum area of steel - 10.6.1.1

Permissible maximum area of steel $A_{st\_max} = 0.08 \times A_g$ = **25.920** in²

$A_{st} < A_{st\_max}$, PASS - Maximum steel check

Slenderness check about x axis

Radius of gyration $r_x = 0.3 \times h$ = **5.4** in

Actual slenderness ratio $s_{rx\_act} = k_x \times l_{ux} / r_x$ = **40**

Permissible slenderness ratio $s_{rx\_perm} = \min(34 - 12 * (M_{1x\_act} / M_{2x\_act}),40)$ = **34**

Column is slender about the X axis

Magnified moments about x axis

Moment of inertia of section $I_{gx} = (b \times h^3) / 12$ = **8748** in⁴

Euler's buckling load $P_{cx} = (\pi^2 / (k_x \times l_{ux})^2) \times (0.4 \times E_c \times I_{gx} / (1+\beta_d))$ = **1667.81** kips

Correction factor for actual to equiv. mmt.diagram $C_{mx} = 0.6 + (0.4 * M_{1x\_act} / M_{2x\_act})$ = **0.600**

Moment magnifier $\delta_{nsx} = \max(C_{mx} / (1 - (P_{u\_act} / (0.75 \times P_{cx}))),1.0)$ = **1**

Magnified moment about x axis $M_{cx} = \delta_{nsx} \times M_{2x\_act}$ = **48.9** kip_ft

Minimum factored moment about x axis $M_{2x\_min} = P_{u\_act} \times (0.6 \text{ in} + 0.03 \times h)$ = **9.3** kip_ft

Minimum magnified moment about x axis $M_{cx\_min} = \delta_{nsx} \times M_{2x\_min}$ = **9.3** kip_ft

Slenderness check about y axis

Radius of gyration $r_y = 0.3 \times b$ = **5.4** in

Actual slenderness ratio $s_{ry\_act} = k_y \times l_{uy} / r_y$ = **40**

Permissible slenderness ratio $s_{ry\_perm} = \min(34 - 12 * (M_{1y\_act} / M_{2y\_act}),40)$ = **34**

Column is slender about the Y axis

Magnified moments about y axis

Moment of inertia of section $I_{gy} = (h \times b^3) / 12$ = **8748** in⁴

Euler's buckling load $P_{cy} = (\pi^2 / (k_y \times l_{uy})^2) \times (0.4 \times E_c \times I_{gy} / (1+\beta_d))$ = **1667.81** kips

Correction factor for actual to equiv. mmt.diagram C_{my} = 1.0

Moment magnifier $\delta_{nsy} = \max(C_{my} / (1 - (P_{u\_act} / (0.75 \times P_{cy}))),1.0)$ = **1.085**

Minimum factored moment about y axis $M_{2y\_min} = P_{u\_act} \times (0.6 \text{ in} + 0.03 \times b)$ = **9.3** kip_ft

Minimum magnified moment about y axis $M_{cy\_min} = \delta_{nsy} \times M_{2y\_min}$ = **10.09** kip_ft

Axial load capacity of axially loaded column

Strength reduction factor ϕ = 1.00

Area of steel on compression face $A'_s = A_{st} / 2 =$ **2.405** in^2

Area of steel on tension face $A_s = A_{st} / 2 =$ **2.405** in^2

Net axial load capacity of column $P_n = 0.8 \times (0.85 \times f'_c \times (A_g - A_{st}) + f_y \times A_{st}) =$ **1099.102** kips

Ultimate axial load capacity of column $P_u = \phi \times P_n =$ **1099.102** kips

PASS : Column is safe in axial loading

Net moments for biaxial column

Assuming strength reduction factor $\phi = 0.65$

Net moment about major (X) axis $M_{nx} = M_{cx} / \phi =$ **75.23** kips_ft

Net moment about minor (Y) axis $M_{ny} = M_{cy\_min} / \phi =$ **15.52** kips_ft

Uniaxially loaded column about major axis

Details of column cross-section

c/d_t ratio $r_{xb} =$ **0.273**

Effective cover to reinforcement $d' = c_c + D_{stir} + (D_{long}/2) =$ **2.312** in

Spacing between bars $s = ((h - (2 \times d')))/ ((N/2)-1) =$ **4.458** in

Depth of tension steel $d_t = h - d' =$ **15.688** in

Depth of NA from extreme compression face $c_x = r_{xb} \times d_t =$ **4.287** in

Factor of depth of compressive stress block $\beta_1 =$ **0.850**

Depth of equivalent rectangular stress block $a_x = \min((\beta_1 \times c_x), h) =$ **3.644** in

Yield strain in steel $\varepsilon_{sx} = f_y / E_s =$ **0.002**

Strength reduction factor $\phi_x =$ **1.000**

Details of concrete block

Force carried by concrete

Forces carried by concrete $P_{xcon} = 0.85 \times f'_c \times b \times a_x =$ **223.022** kips

Moment carried by concrete

Moment carried by concrete $M_{xcon} = P_{xcon} \times ((h/2) - (a_x/2)) =$ **133.403** kip_ft

Details of steel layer 1

Depth of layer $x_{x1} =$ **2.312** in

Strain of layer $\varepsilon_{x1} = \varepsilon_c * (1 - x_{x1} / c_x) =$ **0.00138**

Stress in layer $\sigma_{x1} = \min(f_y, E_s * \varepsilon_{x1}) - 0.85 * f'_c =$ **36672.92** psi

Force carried by layer $P_{x1} = N_x * A_{bar} * \sigma_{x1} =$ **66.157** kips

Moment carried by steel layer $M_{x1} = P_{x1} * ((h / 2) - x_{x1}) =$ **36.868** kip_ft

Details of steel layer 2

Depth of layer $x_{x2} =$ **9.000** in

Strain of layer $\varepsilon_{x2} = \varepsilon_c * (1 - x_{x2} / c_x) =$ **-0.00330**

Stress in layer $\sigma_{x2} = \max(-1 * f_y, E_s * \varepsilon_{x2}) =$ **-60000.00** psi

Force carried by layer $P_{x2} = 2 * A_{bar} * \sigma_{x2} =$ **-72.158** kips

Moment carried by steel layer $M_{x2} = P_{x2} * ((h / 2) - x_{x2}) =$ **0.000** kip_ft

Details of steel layer 3

Depth of layer $x_{x3} =$ **15.688** in

Strain of layer $\varepsilon_{x3} = \varepsilon_c * (1 - x_{x3} / c_x) =$ **-0.00798**

Stress in layer $\sigma_{x3} = \max(-1 * f_y, E_s * \varepsilon_{x3}) =$ **-60000.00** psi

Force carried by layer $P_{x3} = N_x * A_{bar} * \sigma_{x3} =$ **-108.238** kips

Moment carried by steel layer $M_{x3} = P_{x3} * ((h / 2) - x_{x3}) =$ **60.320** kip_ft

Force carried by steel

Sum of forces by steel $P_{xs} =$ **-114.2** kips

Total force carried by column

Nominal axial load strength $P_{nx} =$ **108.782** kips

Strength reduction factor $\phi_x =$ **1.000**

Ultimate axial load carrying capacity of column $P_{ux} = \phi_x \times P_{nx} =$ **97.904** kips

Total moment carried by column

Total moment carried by column $M_{ox} =$ **230.591** kip_ft

Ultimate moment strength capacity of column $M_{ux} = \phi_x \times M_{ox} =$ **230.591** kip_ft

Equivalent required uniaxial moment about x axis

Equivalent required uniaxial nominal moment $M_{nxe} = M_{nx} + M_{ny} \times h / b \times ((1 - \beta) / \beta) =$ **90.754** kip_ft

Equivalent required uniaxial ultimate moment $M_{uxe} = M_{nxe} \times \phi_x =$ **90.754** kip_ft

Check load capacity about the x axis

Factored axial load $P_{u\_act} =$ **97.9** kips

Ultimate axial capacity $P_{ux} =$ **97.9** kips

PASS - Ultimate axial capacity exceeds factored axial load

Equivalent required uniaxial factored moment $M_{uxe} =$ **81.7** kip_ft

Ultimate moment capacity about the x axis $M_{ux} =$ **207.5** kip_ft

PASS - Ultimate moment capacity exceeds factored moment about x axis

Uniaxially loaded column about minor axis

Details of column cross-section

c/d_t ratio $r_{yb} =$ **0.273**

Effective cover to reinforcement $d' = c_c + D_{stir} + (D_{long}/2) =$ **2.312** in

Spacing between bars $s = ((b - (2\times d')))/ ((N/2)-1) =$ **4.458** in

Depth of tension steel $b_t = b - d' =$ **15.688** in

Depth of NA from extreme compression face $c_y = r_{yb} \times b_t =$ **4.287** in

Factor of depth of compressive stress block $\beta_1 =$ **0.850**

Depth of equivalent rectangular stress block $a_y = \min((\beta_1 \times c_y), b) =$ **3.644** in

Yield strain in steel $\varepsilon_{sy} = f_y / E_s =$ **0.002**

Strength reduction factor $\phi_y =$ **0.900**

Details of concrete block

Force carried by concrete

Forces carried by concrete $P_{ycon} = 0.85 \times f'_c \times h \times a_y =$ **223.022** kips

Moment carried by concrete

Moment carried by concrete $M_{ycon} = P_{ycon} \times ((b/2) - (a_y/2)) =$ **133.403** kip_ft

Details of steel layer 1

Depth of layer $x_{y1} =$ **2.312** in

Strain of layer $\varepsilon_{y1} = \varepsilon_c * (1 - x_{y1} / c_y) =$ **0.00138**

Stress in layer $\sigma_{y1} = \min(f_y, E_s * \varepsilon_{y1}) - 0.85 * f'_c$ = **36672.92** psi

Force carried by layer $P_{y1} = N_y * A_{bar} * \sigma_{y1}$ = **66.157** kips

Moment carried by steel layer $M_{y1} = P_{y1} * ((b / 2) - x_{y1})$ = **36.868** kip_ft

Details of steel layer 2

Depth of layer x_{y2} = **9.000** in

Strain of layer $\varepsilon_{y2} = \varepsilon_c * (1 - x_{y2} / c_y)$ = **-0.00330**

Stress in layer $\sigma_{y2} = \max(-1 * f_y, E_s * \varepsilon_{y2})$ = **-60000.00** psi

Force carried by layer $P_{y2} = 2 * A_{bar} * \sigma_{y2}$ = **-72.158** kips

Moment carried by steel layer $M_{y2} = P_{y2} * ((b / 2) - x_{y2})$ = **0.000** kip_ft

Details of steel layer 3

Depth of layer x_{y3} = **15.688** in

Strain of layer $\varepsilon_{y3} = \varepsilon_c * (1 - x_{y3} / c_y)$ = **-0.00798**

Stress in layer $\sigma_{y3} = \max(-1 * f_y, E_s * \varepsilon_{y3})$ = **-60000.00** psi

Force carried by layer $P_{y3} = N_y * A_{bar} * \sigma_{y3}$ = **-108.238** kips

Moment carried by steel layer $M_{y3} = P_{y3} * ((b / 2) - x_{y3})$ = **60.320** kip_ft

Force carried by steel

Sum of forces by steel P_{ys} = **-114.2** kips

Total force carried by column

Nominal axial load strength P_{ny} = **108.782** kips

Strength reduction factor ϕ_y = **1.000**

Ultimate axial load carrying capacity of column $P_{uy} = \phi_y \times P_{ny}$ = **97.904** kips

Moment carried by biaxial column minor axis

Nominal moment strength M_{oy} = **230.591** kip_ft

Contour beta factor

Contour beta factor β = **0.500**

M_{nx}_upon_$M_{ox1} = M_{nx} / M_{ox}$ = **0.326**

M_{ny}_upon_M_{oy} = **0.674**

Net moment along minor axis resisted by column $M_{ny1} = M_{oy} \times (M_{ny}$_upon_$M_{oy})$ = **155.419** kip_ft

Ultimate moment along minor axis $M_{uy} = M_{ny1} \times \phi_y$ = **155.419** kip_ft

Check load capacity about the y axis

Factored axial load P_{u_act} = **97.9** kips

Ultimate axial capacity P_{uy} = **97.9** kips

PASS - Ultimate axial capacity exceeds factored axial load

Factored moment about the y axis $M_{uy\_max} = \phi * M_{ny}$ = **10.1** kip_ft

Ultimate moment capacity about the y axis M_{uy} = **139.9** kip_ft

PASS - Ultimate moment capacity exceeds factored moment about y axis

Design of column ties - 25.7.2

Spacing of lateral ties s_{v_ties} = **12.000** in

16 times longitudinal bar diameter $s_{v1} = 16 \times D_{long}$ = **14.000** in

48 times tie bar diameter $s_{v2} = 48 \times D_{stir}$ = **18.000** in

Least column dimension $s_{v3} = \min(h,b)$ = **18.000** in

Required tie spacing $s = \min(s_{v1}, s_{v2}, s_{v3})$ = **14.000** in

$s_{v\_ties} < s$ PASS

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BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Process Gallery	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (CSZ)		

SEISMIC BASE SHEAR FOR PROCESS GALLERY

7.4.1.3.1 Pseudo Seismic Force for LSP. The pseudo lateral force in a given horizontal direction of a building shall be determined using Eq. (7-21). This force shall be used to evaluate or retrofit the vertical elements of the seismic-force-resisting system.

$$V = C_1 C_2 C_m S_a W \qquad (7\text{-}21)$$

Table 7-3. Alternate Values for Modification Factors C_1C_2

Fundamental Period	$m_{max} < 2$	$2 \le m_{max} < 6$	$m_{max} \ge 6$
$T \le 0.3$	1.1	1.4	1.8
$0.3 < T \le 1.0$	1.0	1.1	1.2
$T > 1.0$	1.0	1.0	1.1

Table 7-4. Values for Effective Mass Factor C_m

No. of Stories	Concrete Moment Frame	Concrete Shear Wall	Concrete Pier-Spandrel	Steel Moment Frame	Steel Concentrically Braced Frame	Steel Eccentrically Braced Frame	Other
1–2	1.0	1.0	1.0	1.0	1.0	1.0	1.0
3 or more	0.9	0.8	0.8	0.9	0.9	0.9	1.0

Note: C_m shall be taken as 1.0 if the fundamental period, T, in the direction of response under consideration is greater than 1.0 s.

spectral response acceleration, S_{xs} =	0.446	g	(CSZ seismic hazard)
spectral response acceleration, S_{x1} =	0.332	g	(CSZ seismic hazard)
building period, T =	0.114	s	
response spectrum acceleration, S_a =	0.446	g	
effective seismic weight, W =	1267.3	kip	
C_1C_2 =	1.4		(Table 11-6 for masonry walls, m=2.0)
effective mass factor, C_m =	1.0		
seismic lateral force, V =	791.3	kip	

Story	Weight, w_x (kip)	Floor Height, h_x (ft)	k factor	$w_xh_x^k$ (kip*ft^2)	C_{vx}	Force on Level, F_x (kip)	Story Force, V_j (kip)
Roof	189.8	32.63	1.0	6193.2	0.242	191.5	191.5
1st	1077.5	18.00	1.0	19395.0	0.758	599.8	791.3

$\Sigma w_xh_x^k$ = 25588.2

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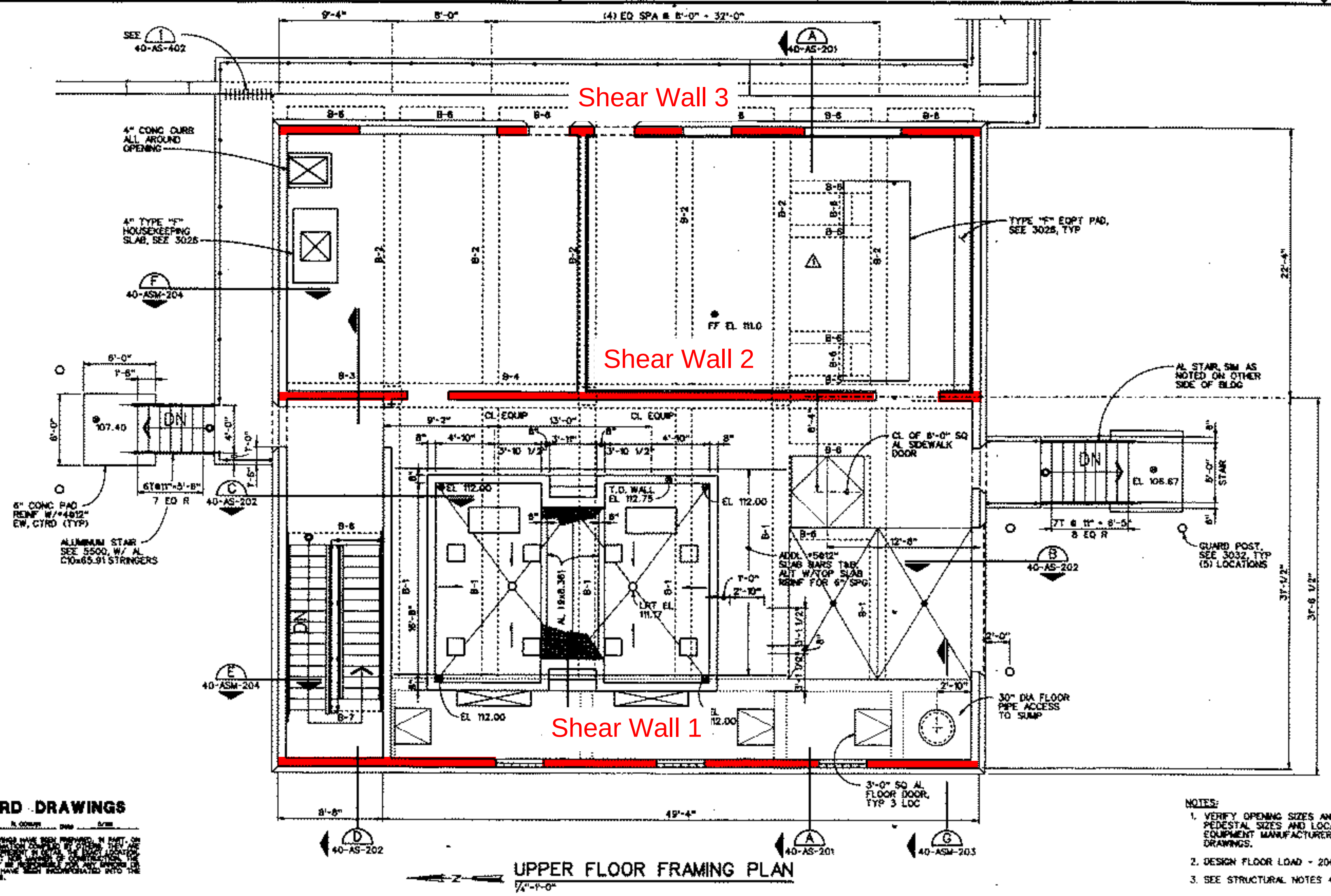

UPPER FLOOR FRAMING PLAN
1/4"=1'-0"

NOTES:
1. VERIFY OPENING SIZES AND EQUIPMENT PEDESTAL SIZES AND LOCATIONS WITH EQUIPMENT MANUFACTURER'S SHOP DRAWINGS.
2. DESIGN FLOOR LOAD = 200 psf
3. SEE STRUCTURAL NOTES 40-AS-151

RECORD DRAWINGS

CITY OF WILSONVILLE WASTEWATER TREATMENT PLANT WILSONVILLE, OREGON

AERATION BASINS & PROCESS GALLERY
ARCHITECTURAL/STRUCTURAL UPPER FLOOR FRAMING PLAN

SHEET 39 | DWG 40-AS-135 | DATE DEC 1995 | PROJ NO. 117645.A4

PLOT DATE=06-MAY-1998 10:39:18 wl40s135.dv

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BY:	BS	**DATE**	Sep-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Process Gallery	**JOB NO.**	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (CSZ)		

CMU IN-PLANE SHEAR WALL CHECK IN BUILDING N-S DIRECTION

11.3.4.2 Strength of Reinforced Masonry Walls and Wall Piers In-Plane Actions. The strength of existing, retrofitted, and new RM wall or wall pier components in flexure, shear, and axial compression shall be determined in accordance with this section. Design actions (axial, flexure, and shear) on components shall be determined in accordance with Chapter 7 of this standard, considering gravity loads and the maximum forces that can be transmitted based on a limit-state analysis.

11.3.4.2.1 Flexural Strength of Walls and Wall Piers. Expected flexural strength of an RM wall or wall pier shall be determined based on strength design procedures specified in TMS 402.

11.3.4.2.2 Shear Strength of Walls and Wall Piers. The expected and lower-bound shear strength of RM wall or wall pier components shall be determined based on strength design procedures specified in TMS 402.

11.3.4.3 Acceptance Criteria for In-Plane Actions of Reinforced Masonry Walls. The shear required to develop the expected flexural strength of RM walls and wall piers shall be compared with the lower-bound shear strength. For RM wall components governed by flexure, flexural actions shall be considered deformation controlled. For RM components governed by shear, shear actions shall be considered deformation controlled. Axial compression on RM wall or wall pier components shall be considered a force-controlled action.

7.5.2 Linear Procedures

7.5.2.1 Forces and Deformations. Component forces and deformations shall be calculated in accordance with linear analysis procedures of Sections 7.4.1 or 7.4.2.

7.5.2.1.1 Deformation-Controlled Actions for LSP or LDP. Deformation-controlled actions, Q_{UD}, shall be calculated in accordance with Eq. (7-34):

$$Q_{UD} = Q_G + Q_E \tag{7-34}$$

where

Q_{UD} = Deformation-controlled action caused by gravity loads and earthquake forces.

Q_G = Action caused by gravity loads as defined in Section 7.2.2; and

Q_E = Action caused by the response to the selected Seismic Hazard Level calculated using either Section 7.4.1 or Section 7.4.2;

<u>*Shear wall 1*</u>

Roof seismic load, V =	191.5	kip
diaphragm span, L =	52.00	ft
roof tributary width for seismic, T_w =	15	ft
tributary seismic load on shear wall, Q_E =	55.2	kip
wall height, h =	14.63	ft
tributary seismic moment on shear wall, Mu =	808.2	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	46	ft
vertitcal shear wall grout spacing =	24	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	2077.0	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\,d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	277.0	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	305.7	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	13608.3	lbs
vertical compressive stress, $f_{ae} = P/(d*t)$ =	0.5	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.000	

h/L =	0.32	
steel reinforcing ratio, ρ_g =	0.004	
ρ_g*f_{ye}/f_{me} =	0.11	
m-factor =	5.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	1375.5	kip
demand capacity ratio, DCR =	*0.04*	***OK***

Shear wall 2

Roof seismic load, V =	191.5	kip
diaphragm span, L =	52.00	ft
roof tributary width for seismic, T_w =	26	ft
tributary seismic load on shear wall, Q_E =	95.8	kip
wall height, h =	14.63	ft
tributary seismic moment on shear wall, Mu =	1400.8	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	48	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	2014.0	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\, d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	270.4	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	299.1	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	23369.7	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	0.8	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, f_{me} = F_{exp}*f'_m =	1950.0	psi
f_{ae}/f_{me} =	0.000	
h/L =	0.30	
steel reinforcing ratio, ρ_g =	0.003	
ρ_g*f_{ye}/f_{me} =	0.09	
m-factor =	5.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	1345.8	kip
demand capacity ratio, DCR =	*0.07*	***OK***

Shear wall 3

Roof seismic load, V =	191.5	kip
diaphragm span, L =	52.00	ft
roof tributary width for seismic, T_w =	10.67	ft
tributary seismic load on shear wall, Q_E =	39.3	kip
wall height, h =	14.63	ft
tributary seismic moment on shear wall, Mu =	574.9	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	28	ft
vertitcal shear wall grout spacing =	24	in

horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	1291.0	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	154.3	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	183.0	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	9761.4	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	0.6	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp} * f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.000	
h/L =	0.52	
steel reinforcing ratio, ρ_g =	0.004	
$\rho_g * f_{ye}/f_{me}$ =	0.11	

m-factor =	5.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	823.3	kip

demand capacity ratio, DCR = *0.05* ***OK***

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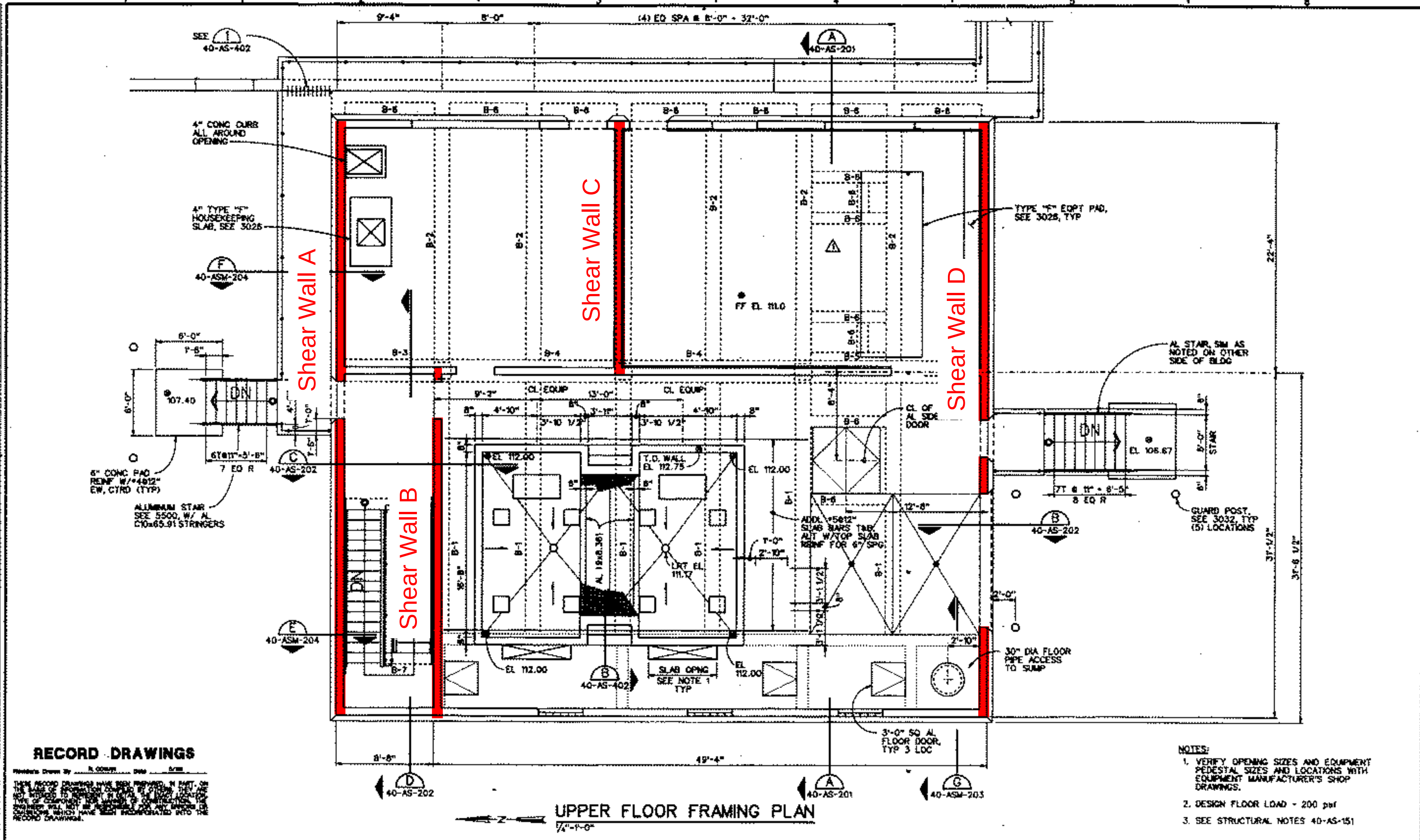

RECORD DRAWINGS

THESE RECORD DRAWINGS HAVE BEEN PREPARED, IN PART, ON THE BASIS OF INFORMATION COMPILED BY OTHERS. THEY ARE NOT INTENDED TO REPRESENT IN DETAIL THE EXACT LOCATION, TYPE OF COMPONENT NOR MANNER OF CONSTRUCTION. THE ENGINEER WILL NOT BE RESPONSIBLE FOR ANY ERRORS OR OMISSIONS WHICH HAVE BEEN INCORPORATED INTO THE RECORD DRAWINGS.

NOTES:

1. VERIFY OPENING SIZES AND EQUIPMENT PEDESTAL SIZES AND LOCATIONS WITH EQUIPMENT MANUFACTURER'S SHOP DRAWINGS.
2. DESIGN FLOOR LOAD = 200 psf
3. SEE STRUCTURAL NOTES 40-AS-151

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RECORD DRAWINGS

CL #32A CONDUIT CONCRETE ENCASEMENT

CITY OF WILSONVILLE WASTEWATER TREATMENT PLANT WILSONVILLE, OREGON

AERATION BASINS & PROCESS GALLERY

ARCHITECTURAL/STRUCTURAL UPPER FLOOR FRAMING PLAN

SHEET 39

DWG 40-AS-135

DATE DEC 1995

PROJ 117645.A4

PLOT DATE=06-MAY-1998 10:39:18 wl40s135.dv

Engineers...Working Wonders With Water™

BY:	BS	DATE	Sep-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Process Gallery	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (CSZ)		

CMU IN-PLANE SHEAR WALL CHECK IN BUILDING E-W DIRECTION

11.3.4.2 Strength of Reinforced Masonry Walls and Wall Piers In-Plane Actions. The strength of existing, retrofitted, and new RM wall or wall pier components in flexure, shear, and axial compression shall be determined in accordance with this section. Design actions (axial, flexure, and shear) on components shall be determined in accordance with Chapter 7 of this standard, considering gravity loads and the maximum forces that can be transmitted based on a limit-state analysis.

11.3.4.2.1 Flexural Strength of Walls and Wall Piers. Expected flexural strength of an RM wall or wall pier shall be determined based on strength design procedures specified in TMS 402.

11.3.4.2.2 Shear Strength of Walls and Wall Piers. The expected and lower-bound shear strength of RM wall or wall pier components shall be determined based on strength design procedures specified in TMS 402.

11.3.4.3 Acceptance Criteria for In-Plane Actions of Reinforced Masonry Walls. The shear required to develop the expected flexural strength of RM walls and wall piers shall be compared with the lower-bound shear strength. For RM wall components governed by flexure, flexural actions shall be considered deformation controlled. For RM components governed by shear, shear actions shall be considered deformation controlled. Axial compression on RM wall or wall pier components shall be considered a force-controlled action.

7.5.2 Linear Procedures

7.5.2.1 Forces and Deformations. Component forces and deformations shall be calculated in accordance with linear analysis procedures of Sections 7.4.1 or 7.4.2.

7.5.2.1.1 Deformation-Controlled Actions for LSP or LDP. Deformation-controlled actions, Q_{UD}, shall be calculated in accordance with Eq. (7-34):

$$Q_{UD} = Q_G + Q_E \qquad (7\text{-}34)$$

where

Q_{UD} = Deformation-controlled action caused by gravity loads and earthquake forces.

Q_G = Action caused by gravity loads as defined in Section 7.2.2; and

Q_E = Action caused by the response to the selected Seismic Hazard Level calculated using either Section 7.4.1 or Section 7.4.2;

Shear wall A

Roof seismic load, V =	191.5	kip
diaphragm span, L =	56.67	ft
roof tributary width for seismic, T_w =	4	ft
tributary seismic load on shear wall, Q_E =	13.5	kip
wall height, h =	14.63	ft
tributary seismic moment on shear wall, Mu =	197.8	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	50	ft
vertitcal shear wall grout spacing =	24	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	2238.0	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\,d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	302.3	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	331.0	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	3537.1	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	0.1	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi

f_{ae}/f_{me} =	0.000	
h/L =	0.29	
steel reinforcing ratio, ρ_g =	0.004	
ρ_g*f_{ye}/f_{me} =	0.11	
m-factor =	5.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	1489.5	kip

demand capacity ratio, DCR = 0.01 **OK**

Shear wall B

Roof seismic load, V =	191.5	kip
diaphragm span, L =	56.67	ft
roof tributary width for seismic, T_w =	12	ft
tributary seismic load on shear wall, Q_E =	40.6	kip
wall height, h =	14.63	ft
tributary seismic moment on shear wall, Mu =	593.3	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	26.67	ft
vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	1128.1	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\,d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	132.8	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	161.5	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	10064.7	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	0.7	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.000	
h/L =	0.55	
steel reinforcing ratio, ρ_g =	0.004	
ρ_g*f_{ye}/f_{me} =	0.11	
m-factor =	5.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	726.7	kip

demand capacity ratio, DCR = 0.06 **OK**

Shear wall C

Roof seismic load, V =	191.5	kip
diaphragm span, L =	56.67	ft
roof tributary width for seismic, T_w =	24.33	ft
tributary seismic load on shear wall, Q_E =	82.2	kip
wall height, h =	14.63	ft
tributary seismic moment on shear wall, Mu =	1202.8	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	21.33	ft

vertitcal shear wall grout spacing =	32	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	926.9	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	100.5	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	129.2	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	19994.8	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	1.6	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.001	
h/L =	0.69	
steel reinforcing ratio, ρ_g =	0.004	
ρ_g*f_{ye}/f_{me} =	0.11	

m-factor =	5.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmφQCE =	581.3	kip

demand capacity ratio, DCR = *0.14* ***OK***

<u>*Shear wall D*</u>

Roof seismic load, V =	191.5	kip
diaphragm span, L =	56.67	ft
roof tributary width for seismic, T_w =	16.33	ft
tributary seismic load on shear wall, Q_E =	55.2	kip

wall height, h =	14.63	ft
tributary seismic moment on shear wall, Mu =	807.3	kip*ft
masonry strength, f'_m =	1500	psi
shear wall length, d =	38	ft
vertitcal shear wall grout spacing =	24	in
horizontal shear wall grout spacing =	48	in
shear wall thickness, t =	7.625	in
A_n =	1714.0	in^2
Φ =	1.0	(assumed per Tier 2)

$$\phi V_m = \phi\left[\left[4.0 - 1.75\left(\frac{M}{V\ d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

masonry shear wall strength, ϕQCE_m =	220.8	kip
horizontal masonry shear wall strength, ϕQCE_s =	28.7	kip
combined masonry shear wall strength, ϕQCE =	249.5	kip

Determining m-factor for wall governed by flexure

roof axial load on wall, P =	13463.2	lbs
vertical compressive stress, f_{ae} = P/(d*t) =	0.6	psi
factor for expected strength, F_{exp} =	1.3	(ASCE 41-17 Table 11-1)
expected compressive strength, $f_{me} = F_{exp}*f'_m$ =	1950.0	psi
f_{ae}/f_{me} =	0.000	
h/L =	0.39	
steel reinforcing ratio, ρ_g =	0.004	

ρ_g*f_{ye}/f_{me} =	0.11	
m-factor =	5.0	(interpolated between LS & CP. ASCE 41-17 Table 11-6)
knowledge factor, κ =	0.90	
masonry shear wall strength, κmϕQCE =	1122.7	kip
demand capacity ratio, DCR =	*0.05*	***OK***

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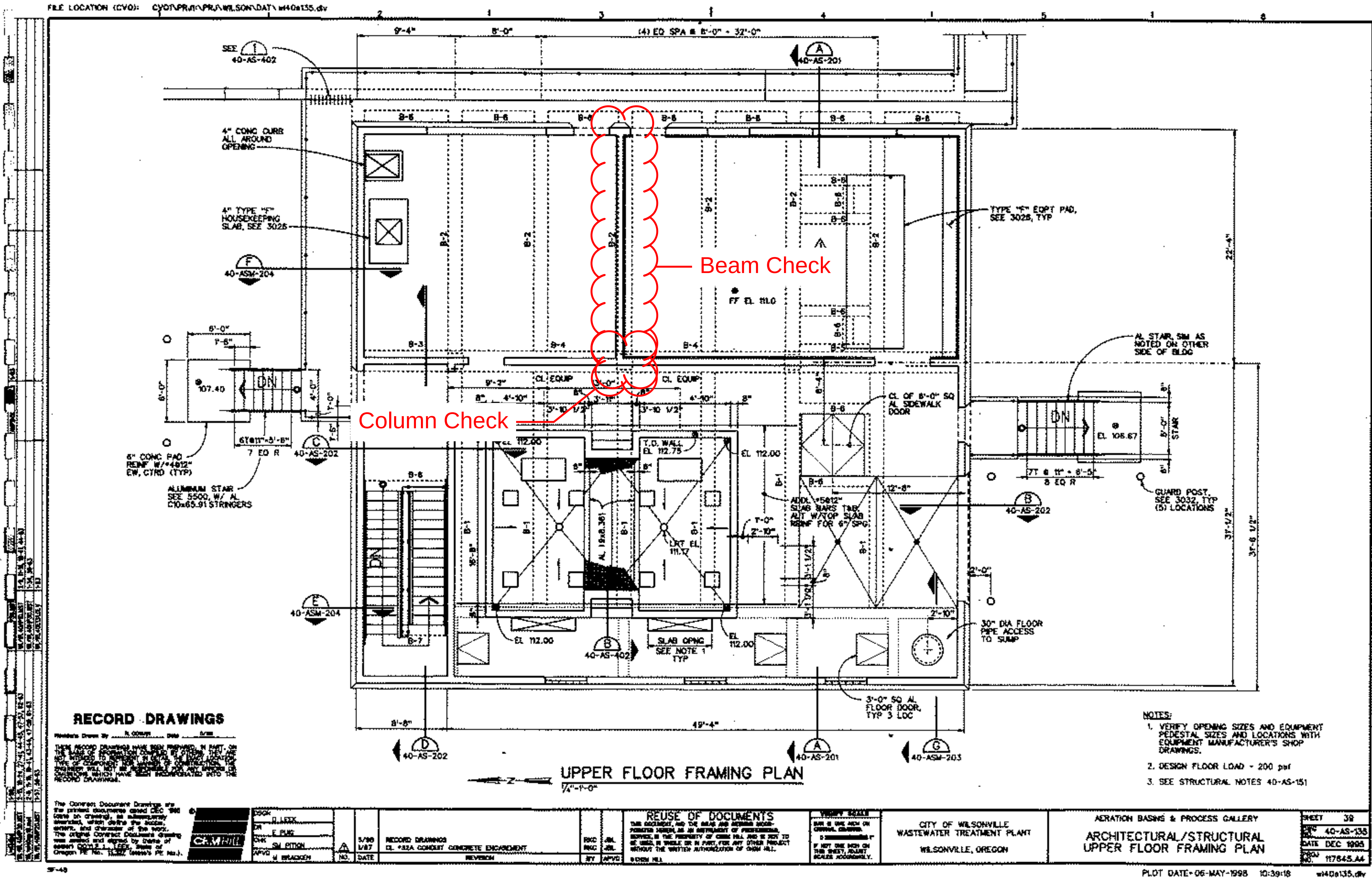

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REORDER BY NUMBER D75AR

DRAWING NUMBER
WWTP UPGRADE
PLAN HOLD CORPORATION • IRVINE, CALIFORNIA
REORDER BY NUMBER D75AR

DRAWING NUMBER
SE 23
500
PLAN HOLD CORPORATION • IRVINE, CALIFORNIA
REORDER BY NUMBER D75AR

FILE LOCATION (CVO): CVO\PROJ\PRJ\WILSON\DAT\ wl40s401.dv

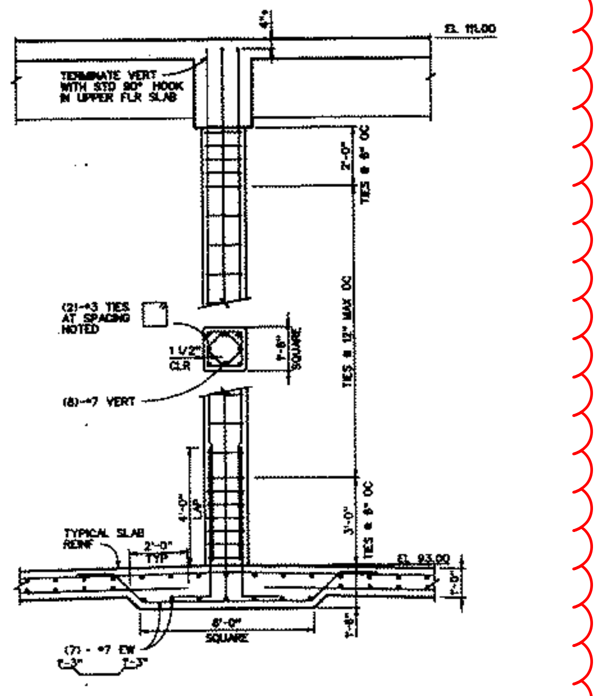
BEAM SCHEDULE

BEAM NO. (SEE PLANS)	SIZE W	SIZE *D	TOP REINF AT LEFT SUPPORT T	TOP REINF AT LEFT SUPPORT TC	BOTTOM REINF B	BOTTOM REINF BC	*TOP REINF AT RIGHT SUPPORT T	*TOP REINF AT RIGHT SUPPORT TC	S BARS	SHEAR REINFORCEMENT NO. SIZE	SHEAR REINFORCEMENT SPACING FROM LEFT SUPPORT	REMARKS
B-1	18"	32"	-	2-#7	-	2-#8	2-#7	2-#7	2-#5	2-#4	[illegible]	
B-2	18"	32"	2-#7	2-#7	-	2-#8	-	2-#7	2-#5	2-#4	1@3", 10@6", 5@12", 8@6", 1@3"	
B-3	24"	36"	-	2-#6	-	2-#9	2-#8	2-#9	2-#5	2-#4	2@3", 5@12", 2@3"	
B-4	24"	36"	2-#8	2-#9	-	3-#9	2-#8	2-#8	2-#5	2-#4	1@3", 24@6", 1@3"	
B-5	24"	36"	2-#8	2-#9	-	4-#8	2-#8	-	2-#5	2-#4	10@3", 5@6", 6@12", 2@3"	
B-6	12"	24"	-	2-#5	-	2-#5	-	2-#5	1-#5	2-#4	1@3", 5@12", 1@3"	
B-7	12"	18"	-	2-#5	-	2-#5	-	2-#5	1-#5	2-#4	1@3", 5@12", 1@3"	

NOTES:

*1 TOP REINFORCEMENT MAY BE CALLED-OUT TWICE IN SCHEDULE (I.E. "TOP REINF. AT RIGHT SUPPORT" OF BEAM THAT IS CONTINUOUS OVER THE RIGHT SUPPORT IS CALLED-OUT AS "TOP REINF. AT LEFT SUPPORT" OF ADJACENT BEAM.).

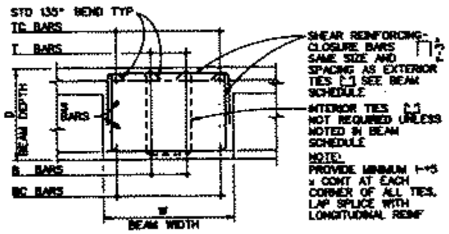
LEFT SUPPORT IS DESIGNATED AS THE SUPPORT CLOSEST THE LEFT SIDE OR BOTTOM OF SHEET ON WHICH FRAMING PLAN IS DRAWN, UNLESS NOTED OTHERWISE ON PLAN.



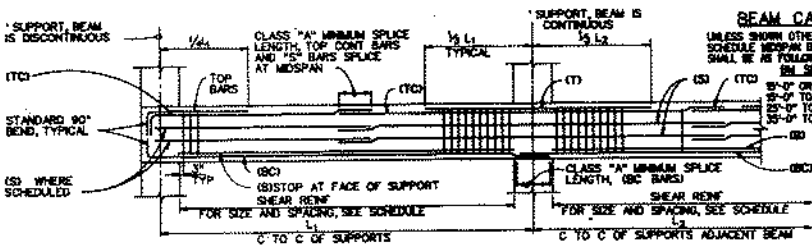
COLUMN/FTG DETAIL (1 / 40-AS-121)
3/4"=1'-0"

RECORD DRAWINGS

THESE RECORD DRAWINGS HAVE BEEN PREPARED, IN PART, ON THE BASIS OF INFORMATION COMPILED BY OTHERS. ... THE RECORD DRAWINGS.



TYPICAL BEAM SECTION
NTS



TYPICAL BEAM REINFORCING
NTS

BEAM CAMBER
UNLESS SHOWN OTHERWISE IN SCHEDULE MIDSPAN BEAM CAMBER SHALL BE AS FOLLOWS:

BM SPAN	CAMBER
15'-0" OR LESS	NONE
15'-0" TO 25'-0"	1/4"
25'-0" TO 35'-0"	1/2"
35'-0" TO 45'-0"	3/4"

REUSE OF DOCUMENTS

CITY OF WILSONVILLE WASTEWATER TREATMENT PLANT
WILSONVILLE, OREGON

AERATION BASINS & PROCESS GALLERY
ARCHITECTURAL/STRUCTURAL DETAILS & BEAM SCHEDULE

SHEET 46
DWG 40-AS-401
DATE DEC 1995
PROJ 117645.A4

RECORD DRAWINGS

PLOT DATE: 06-MAY-1998 13:18:13 wl40s401.dv

BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Process Gallery	**JOB NO.**	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (CSZ)		

BEAM AND COLUMN CHECK SUPPORTING CMU WALL ABOVE (VERTICAL IRREGULARITY TIER 1 FINDING)

5.4.2.3 Vertical Irregularities. An analysis shall be performed in accordance with Section 5.2.4, and the demand-capacity ratio (DCR) shall be determined in accordance with Section 7.3.1.1 for all elements of the seismic-force-resisting system in the noncompliant stories. The adequacy of the elements and connections below the vertical discontinuities shall be evaluated in accordance with Section 5.2.5 as force-controlled elements. The adequacy of struts and diaphragms to transfer loads to adjacent seismic-force-resisting elements as force-controlled elements shall be evaluated.

7.5.2.1.2 Force-Controlled Actions for LSP or LDP. Force-controlled actions, Q_{UF}, shall be calculated using one of the following methods:

1. Q_{UF} shall be taken as the maximum action that can be developed in a component based on a limit-state analysis considering the expected strength of the components delivering force to the component under consideration, or the maximum action developed in the component as limited by the nonlinear response of the building.
2. Alternatively, Q_{UF} shall be calculated in accordance with Eq. (7-35).

$$Q_{UF} = Q_G \pm \frac{\chi Q_E}{C_1 C_2 J} \qquad (7\text{-}35)$$

Roof seismic load, V =	191.5	kip
diaphragm span, L =	56.67	ft
roof tributary width for seismic, T_w =	24.33	ft
wall height, h =	14.63	ft
tributary seismic load on shear wall, V_E =	82.2	kip
seismic overturning on shear wall, M_E =	1202.8	kip*ft
Wall length, L_w =	21.33	ft
Factor for adjusting action, χ =	1.3	(interpolated between LS & IO)
C_1C_2 =	1.4	
Force delivery reduction factor, J =	2	

Beam B2 (16"x32") Check

Roof unit weight, w_{Droof} =	15.3	psf	
Wall unit weight, w_{Dwall} =	47.0	lb/ft	
Floor unit weight, w_{Dfloor} =	183	psf	
Floor unit live load, w_{Lfloor} =	200	psf	
Tributary width to beam, T_{wbeam} =	8	ft	
supported gravity loads on beam, Q_D =	1633.4	lb/ft	
supported live loads on beam, Q_L =	400	lb/ft	(assume only 25% of LL)
supported combined loads on beam, Q_G =	2236.7	lb/ft	
Axial load on beam, Q_{UF} =	38.2	kip	
Bending moment demand on beam, Q_{UF} =	127.2	kip*ft	
Shear demand on beam, Q_{UF} =	23.9	kip	
Beam axial strength, Q_{CL} =	1534.8	kip	(From TEDDS calculation)
Beam bending strength, Q_{CL} =	296.6	kip*ft	(From TEDDS calculation)
Beam shear strength, Q_{CL} =	58.7	kip	(From TEDDS calculation)
knowledge factor, κ =	0.90		
Beam axial strength, $\kappa*Q_{CL}$ =	1381.32	kip	
Beam bending strength, $\kappa*Q_{CL}$ =	266.94	kip*ft	
Beam shear strength, $\kappa*Q_{CL}$ =	52.83	kip	
Axial DCR =	*0.03*	***OK***	
Moment DCR =	*0.48*	***OK***	
Shear DCR =	*0.45*	***OK***	

Column (18"x18") Check

Roof unit weight, w_{Droof} =	15.3	psf	
Wall unit weight, w_{Dwall} =	47.0	lb/ft	
Floor unit weight, w_{Dfloor} =	183	psf	
Floor unit live load, w_{Lfloor} =	200	psf	
Tributary area to column, $T_{wcolumn}$ =	213.36	ft^2	
supported gravity loads on column, Q_D =	43.2	kip	
supported live loads on column, Q_L =	10.7	kip	(assume only 25% of LL)
supported combined loads on column, Q_G =	59.2	kip	
supported overturning loads on column, Q_E =	26.2	kip	
Axial compression load on column, Q_{UFcomp} =	85.4	kip	
Axial tension load on column, Q_{UFten} =	12.7	kip	
Bending moment demand on column, Q_{UF} =	42.7	kip*ft	
Shear demand on column, Q_{UF} =	4.1	kip	
Column axial strength, Q_{CL} =	1099.1	kip	(From TEDDS calculation)
Column bending strength, Q_{CL} =	230.5	kip*ft	(From TEDDS calculation)
Column shear strength, Q_{CL} =	33.8	kip	(From TEDDS calculation)
knowledge factor, κ =	0.90		
Column axial strength, κ*Q_{CL} =	989.19	kip	
Column bending strength, κ*Q_{CL} =	207.45	kip*ft	
Column shear strength, κ*Q_{CL} =	30.42	kip	
Axial DCR =	*0.09*	***OK***	
Moment DCR =	*0.21*	***OK***	
Shear DCR =	*0.14*	***OK***	

503

DRAWING NUMBER
93 10 014
PLAN HOLD CORPORATION • IRVINE, CALIFORNIA
REORDER BY NUMBER 075AF

DRAWING NUMBER
WWTP UPGRADE
PLAN HOLD CORPORATION • IRVINE, CALIFORNIA
REORDER BY NUMBER 075AF

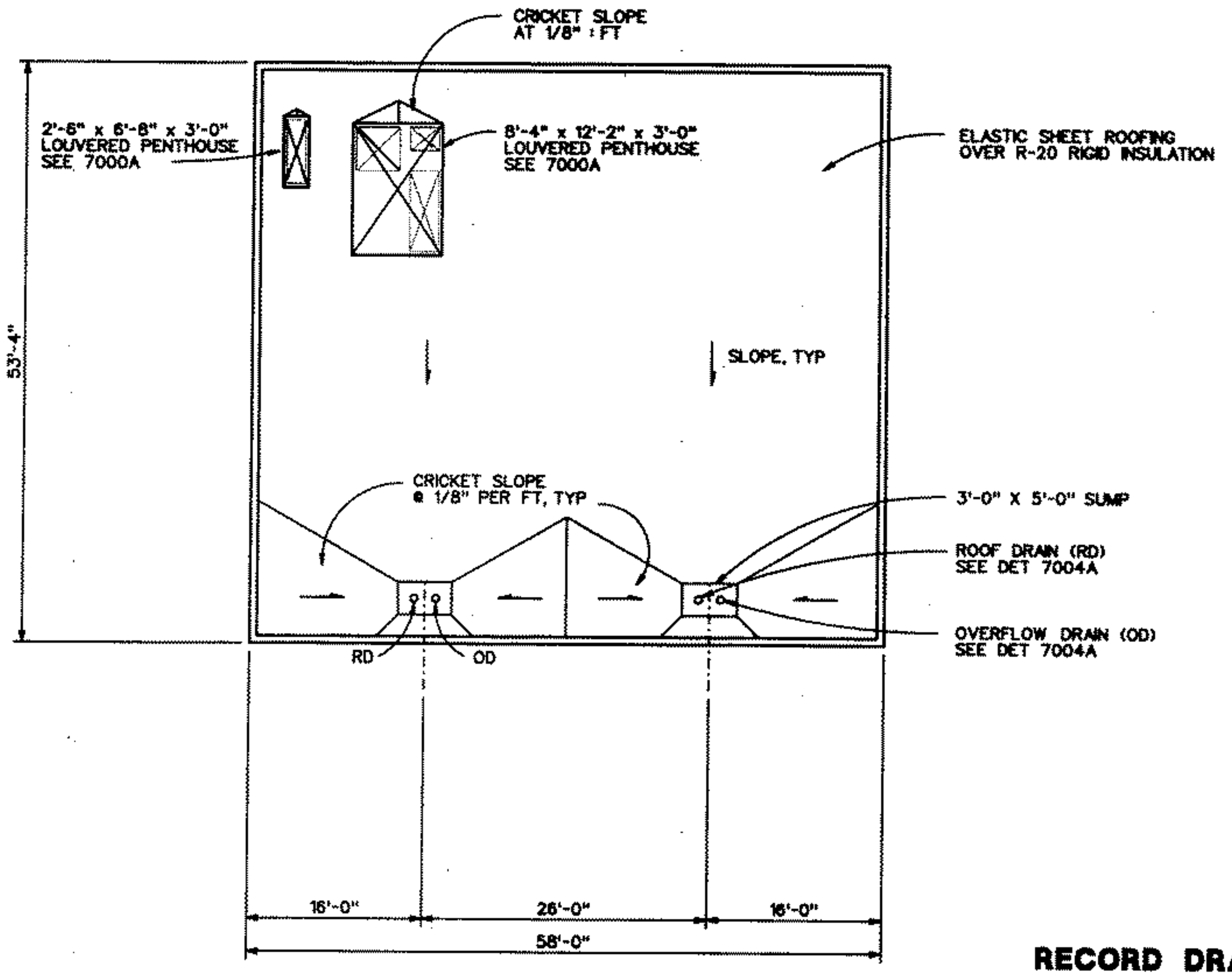
DRAWING NUMBER
SE 23
PLAN HOLD CORPORATION • IRVINE, CALIFORNIA
REORDER BY NUMBER 075AR

FILE LOCATION (CVO): CY01\PRJ1:\PRJ\WILSON\DAT\ wl40s151.dlv

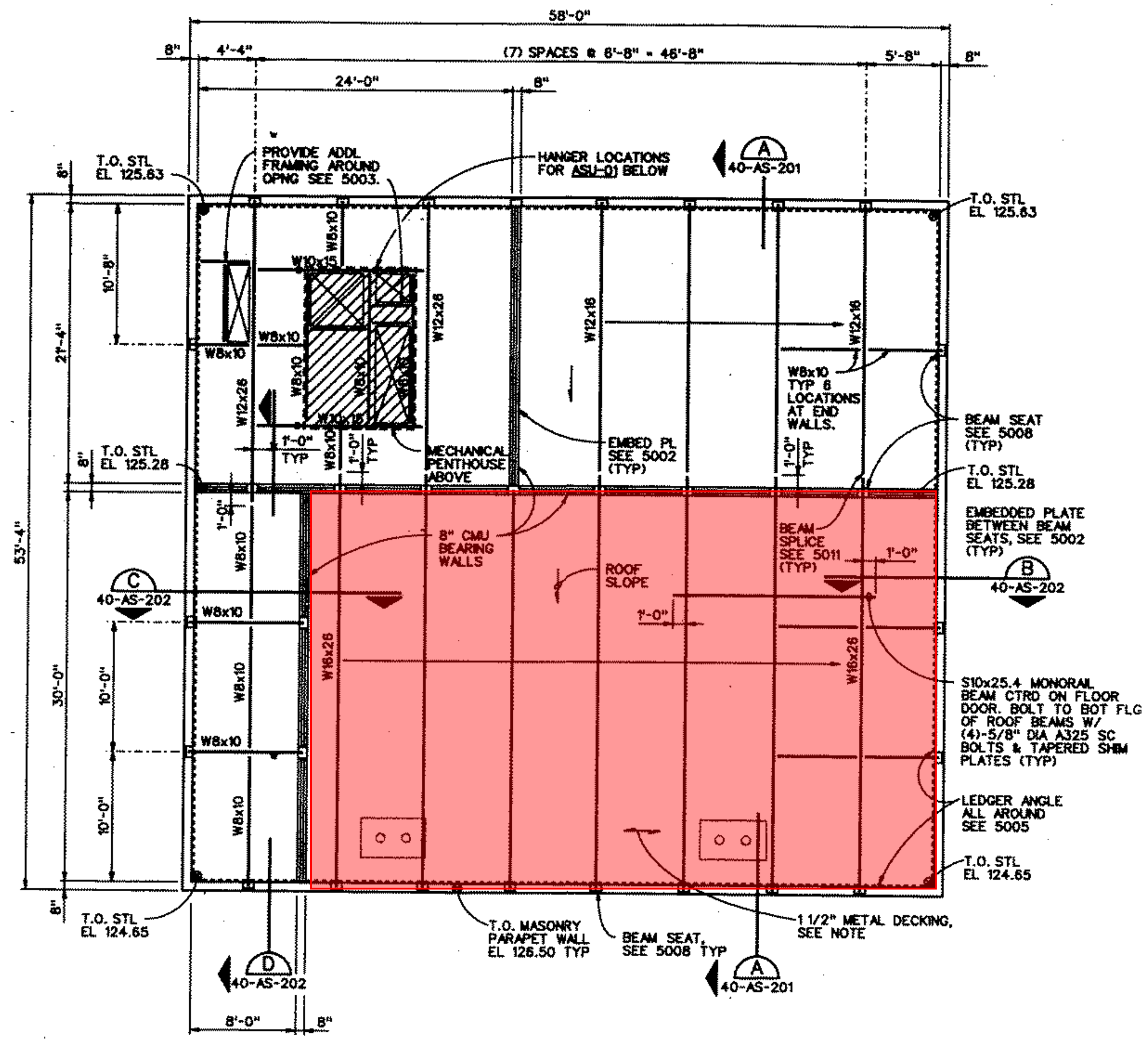
NOTES:

1. SNOW LOAD 25 psf + DRIFT
2. ROOF DECKING SHALL BE 1 1/2"-20 GA GALV STEEL TYPE B (WIDE RIB) ROOF DECKING WITH MINIMUM SECTION PROPERTIES:
   I = 0.21 IN^4 PER FT WIDTH
   S = 0.23 IN^3 PER FT WIDTH
3. FASTEN DECKING W/ 5/8" DIA PUDDLE WELDS @ 0'-6" CTRS AT EACH SUPPORT (7 MIN PER 36" PANEL WIDTH) AND @ 1'-0" CTRS ALONG ALL SUPPORTS PARALLEL TO DECK SPAN. FASTEN SIDE SEAMS W/ WELDS @ 1'-0" CTRS.
4. MONORAIL CAPACITY 1 TON.
5. LOCATE W10x12 ENDWALL SUPPORT BEAMS UNDER DECKING PANEL BOTTOM FLUTE. WELD DECK TO BEAMS W/ 5/8" DIA PUDDLE WELDS @ 1'-0" CTRS.
6. WELD DECK TO ALL OPENING FRAMING W/ 5/8" DIA PUDDLE WELDS @ 6" OC MAXIMUM BUT NOT LESS THAN 3 WELDS PER OPENING SIDE OR PER MEMBER.

⚠1 USED #10 TEK SELFTAPPING SCREWS IN LIEU OF STANDING SIDE SEAM.



ROOF PLAN
1/8"=1'-0"



ROOF FRAMING PLAN
3/16"=1'-0"

Diaphragm Span Check

RECORD DRAWINGS
Revisions Drawn By R. COWAN Date 5/98
THESE RECORD DRAWINGS HAVE BEEN PREPARED, IN PART, ON THE BASIS OF INFORMATION COMPILED BY OTHERS. THEY ARE NOT INTENDED TO REPRESENT IN DETAIL THE EXACT LOCATION, TYPE OF COMPONENT NOR MANNER OF CONSTRUCTION. THE ENGINEER WILL NOT BE RESPONSIBLE FOR ANY ERRORS OR OMISSIONS WHICH HAVE BEEN INCORPORATED INTO THE RECORD DRAWINGS.

The Contract Document Drawings are the printed documents dated DEC 1995 (date on drawing), as subsequently amended, which define the scope, extent, and character of the work. The original Contract Document drawing was sealed and signed by (name of sealer) DOYLE L. LEEK, State of Oregon PE No. 11327 (state's PE No.).

CH2M HILL

DSGN	D. LEEK
DR	E. PUIG
CHK	SM. PITKIN
APVD	M. BRACKEN

NO.	DATE	REVISION	BY	APVD
	5/98	RECORD DRAWINGS	RKC	JBL
⚠1	11/96	CL #54 SHEET METAL DECKING SEAM CONNECTION	RKC	JBL

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"
IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

CITY OF WILSONVILLE
WASTEWATER TREATMENT PLANT
WILSONVILLE, OREGON

AERATION BASINS & PROCESS GALLERY
ARCHITECTURAL/STRUCTURAL
ROOF FRAMING & ROOF PLAN

SHEET	40
DWG NO.	40-AS-151
DATE	DEC 1995
PROJ NO.	117645.A4

SF-96

PLOT DATE= 03-JUN-1998 14:40:44 wl40s151.dlv

Engineers...Working Wonders With Water ™

BY:	BS	DATE	Sep-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Process Gallery	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (CSZ)		

DIAPHRAGM METAL DECK CHECK

9.10.1.3 Strength of Bare Metal Deck Diaphragms. The strength of bare metal deck diaphragms shall be determined in accordance with Section 9.3.2 and the requirements of this section.

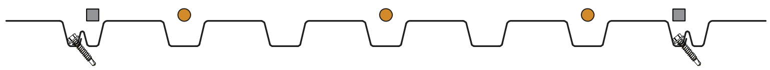
Expected strength, Q_{CE}, for bare metal deck diaphragms shall be taken as 2 times allowable values specified in approved codes and standards, unless a larger value is justified by test data. Alternatively, lower-bound strength shall be taken as nominal strength published in approved codes or standards, except that the strength reduction factor, ϕ, shall be taken as equal to 1.0.

Lower-bound strengths, Q_{CL}, of welded connectors shall be as specified in AWS D1.3, or other approved standard.

Roof seismic load, V =	191.5	kip	
diaphragm span, L =	58.00	ft	
roof unit diaphragm load, v =	3.30	kip/ft	
Roof span between shear walls, L_1 =	48.00	ft	
Roof depth, d =	53.33	ft	
diaphragm shear, v_1 =	1.486	kip/ft	
diaphragm strength, Q_{allow} =	530	lbs/ft	
expected diaphragm strength, Q_{CE} =	1060	lbs/ft	(expected strength shall be 2x the allowable per ASCE 41-17 Section 9.10.1.3)
m-factor =	1.625	(interpolated between LS & IO. ASCE 41-17 Table 9-6)	
knowledge factor, κ =	0.90		
diaphragm strength, $\kappa m \phi Q_{CE}$ =	1.550	kip/ft	
demand capacity ratio, DCR =	*0.96*	***OK***	

Type HSB®-36-SS

- **36/5 Weld Pattern at Supports**
- **Sidelaps Connected with #10 Screws**



Allowable Diaphragm Shear Strength, q (plf) and Flexibility Factors, F ((in./lb)x10⁶)

DECK GAGE	SIDELAP ATTACHMENT		SPAN (ft-in.) 4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
22	#10 @ 24"	q	431	378	310	289	249	242	218		
		F	-2.3+190R	0.2+152R	2.9+126R	3.9+108R	5.6+94R	6.1+83R	7.4+75R		
	#10 @ 18"	q	480	417	343	317	298	264	257		
		F	-3.3+190R	-0.7+152R	1.8+126R	3+108R	3.8+95R	5.2+84R	5.7+75R		
	#10 @ 12"	q	527	456	408	373	347	329	316		
		F	-4+190R	-1.3+152R	0.5+127R	1.8+109R	2.8+95R	3.5+84R	4.1+76R		
	#10 @ 8"	q	607	565	506	485	445	438	414		
		F	-4.8+191R	-2.5+153R	-0.6+127R	0.4+109R	1.5+95R	2.1+85R	2.8+76R		
	#10 @ 6"	q	682	627	589	561	539	522	509		
		F	-5.4+191R	-2.9+153R	-1.3+127R	-0.1+109R	0.8+95R	1.5+85R	2+76R		
	#10 @ 4"	q	817	769	736	712	693	678	666		
		F	-6+191R	-3.6+153R	-2+127R	-0.9+109R	0+96R	0.7+85R	1.2+76R		
20	#10 @ 24"	q	601	526	433	403	349	335	301	297	272
		F	0.9+120R	2.5+95R	4.5+79R	5.1+68R	6.5+59R	6.7+52R	7.7+47R	7.8+43R	8.6+39R
	#10 @ 18"	q	662	577	476	440	413	363	352	344	315
		F	0+120R	1.7+96R	3.5+79R	4.3+68R	4.8+60R	5.9+53R	6.2+47R	6.4+43R	7.1+39R
	#10 @ 12"	q	716	629	561	513	477	449	430	414	401
		F	-0.6+120R	1.1+96R	2.3+80R	3.2+68R	3.8+60R	4.3+53R	4.8+48R	5.1+43R	5.4+40R
	#10 @ 8"	q	820	760	683	658	606	592	558	554	530
		F	-1.5+121R	0+96R	1.3+80R	2+69R	2.7+60R	3+54R	3.5+48R	3.7+44R	4.1+40R
	#10 @ 6"	q	916	841	788	750	720	697	678	662	649
		F	-2+121R	-0.4+97R	0.7+80R	1.4+69R	2+60R	2.5+54R	2.8+48R	3.1+44R	3.4+40R
	#10 @ 4"	q	1089	1024	979	945	920	899	883	869	857
		F	-2.5+121R	-1+97R	0+81R	0.8+69R	1.3+60R	1.7+54R	2.1+48R	2.4+44R	2.6+40R
18	#10 @ 24"	q	1002	885	731	677	588	562	502	491	450
		F	3.2+58R	4+46R	5.4+38R	5.6+33R	6.6+28R	6.6+25R	7.4+22R	7.4+20R	8+18R
	#10 @ 18"	q	1085	956	797	734	687	606	581	563	516
		F	2.4+58R	3.3+46R	4.5+38R	4.9+33R	5.2+29R	6+25R	6.1+23R	6.2+21R	6.7+19R
	#10 @ 12"	q	1166	1024	925	847	786	738	700	670	647
		F	1.9+58R	2.8+47R	3.5+39R	4+33R	4.3+29R	4.6+26R	4.9+23R	5.1+21R	5.2+19R
	#10 @ 8"	q	1321	1219	1094	1049	973	951	898	886	845
		F	1.1+59R	1.9+47R	2.6+39R	2.9+34R	3.3+29R	3.5+26R	3.8+23R	3.9+21R	4.1+19R
	#10 @ 6"	q	1465	1340	1253	1189	1139	1100	1068	1042	1020
		F	0.7+59R	1.5+47R	2.1+39R	2.5+34R	2.8+29R	3+26R	3.2+24R	3.3+21R	3.4+20R
	#10 @ 4"	q	1721	1615	1540	1484	1441	1407	1379	1356	1337
		F	0.2+59R	1+47R	1.5+39R	1.9+34R	2.1+30R	2.4+26R	2.5+24R	2.7+21R	2.8+20R
16	#10 @ 24"	q	1277	1139	946	884	768	739	661	647	590
		F	3.8+33R	4.3+26R	5.3+21R	5.4+18R	6.2+16R	6.2+14R	6.9+12R	6.8+11R	7.3+10R
	#10 @ 18"	q	1393	1235	1038	963	906	801	771	748	683
		F	3.1+33R	3.7+26R	4.6+22R	4.8+18R	5+16R	5.6+14R	5.7+13R	5.7+12R	6.2+10R
	#10 @ 12"	q	1505	1330	1208	1118	1044	985	937	899	867
		F	2.6+33R	3.2+26R	3.6+22R	4+19R	4.2+16R	4.4+15R	4.6+13R	4.7+12R	4.8+11R
	#10 @ 8"	q	1717	1597	1440	1389	1292	1268	1200	1188	1138
		F	2+33R	2.4+27R	2.9+22R	3+19R	3.3+17R	3.4+15R	3.6+13R	3.6+12R	3.8+11R
	#10 @ 6"	q	1914	1763	1658	1580	1520	1472	1433	1402	1375
		F	1.6+34R	2.1+27R	2.4+22R	2.6+19R	2.8+17R	2.9+15R	3.1+13R	3.2+12R	3.2+11R
	#10 @ 4"	q	2258	2132	2043	1977	1926	1886	1853	1825	1802
		F	1.1+34R	1.6+27R	1.9+22R	2.1+19R	2.3+17R	2.4+15R	2.5+13R	2.6+12R	2.6+11R

See footnotes on page 28.

Deck Span = 6'-8"
q = 530 psf (interpolated)

WORKSHOP - TIER 2 CALCULATIONS

carollo
Engineers... Working Wonders With Water

BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Workshop	**JOB NO.**	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (BSE-2E)		

SEISMIC BASE SHEAR FOR WORKSHOP

7.4.1.3.1 Pseudo Seismic Force for LSP. The pseudo lateral force in a given horizontal direction of a building shall be determined using Eq. (7-21). This force shall be used to evaluate or retrofit the vertical elements of the seismic-force-resisting system.

$$V = C_1 C_2 C_m S_a W \qquad (7\text{-}21)$$

Table 7-3. Alternate Values for Modification Factors C_1C_2

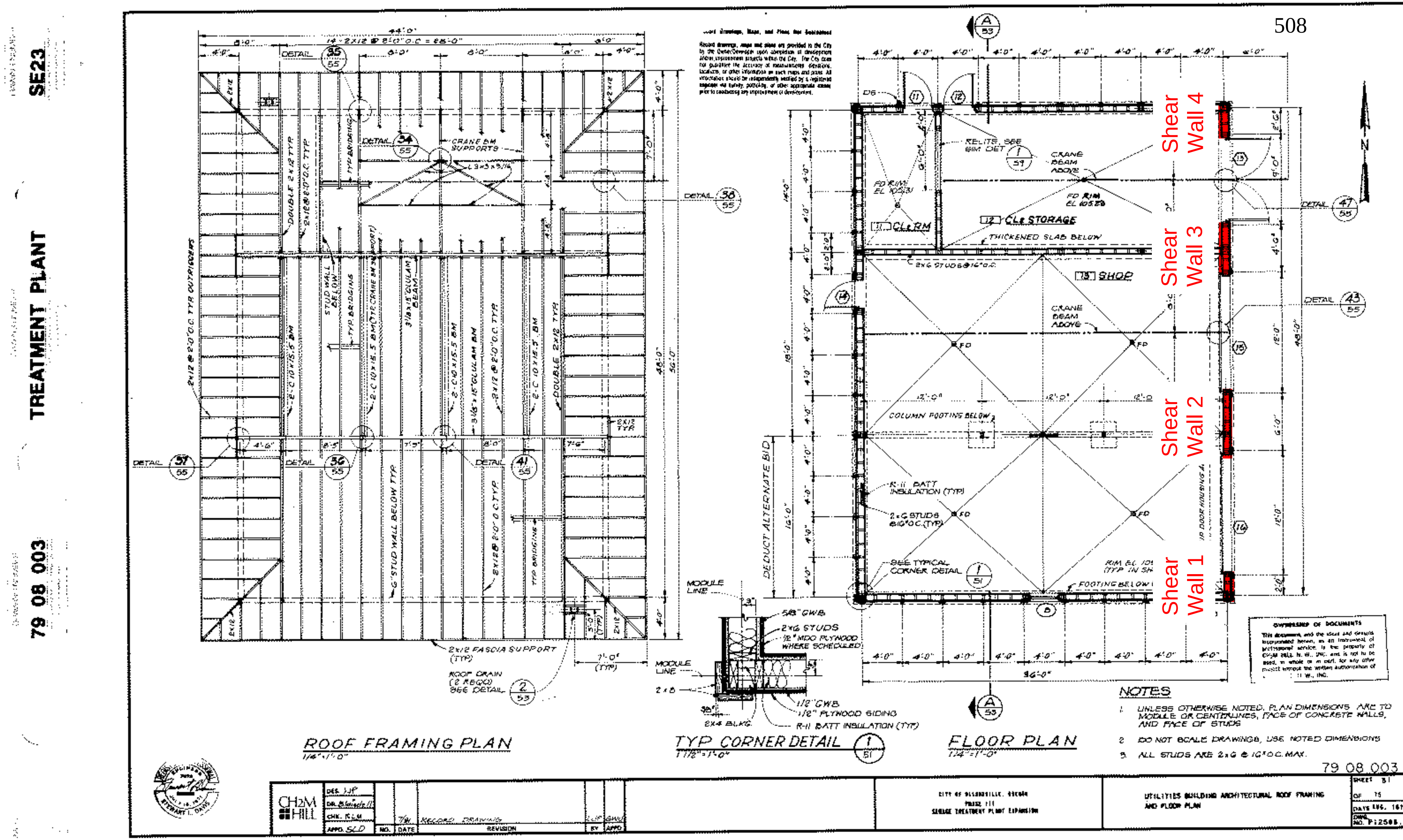
Fundamental Period	$m_{max} < 2$	$2 \le m_{max} < 6$	$m_{max} \ge 6$
$T \le 0.3$	1.1	1.4	1.8
$0.3 < T \le 1.0$	1.0	1.1	1.2
$T > 1.0$	1.0	1.0	1.1

Table 7-4. Values for Effective Mass Factor C_m

No. of Stories	Concrete Moment Frame	Concrete Shear Wall	Concrete Pier-Spandrel	Steel Moment Frame	Steel Concentrically Braced Frame	Steel Eccentrically Braced Frame	Other
1–2	1.0	1.0	1.0	1.0	1.0	1.0	1.0
3 or more	0.9	0.8	0.8	0.9	0.9	0.9	1.0

Note: C_m shall be taken as 1.0 if the fundamental period, T, in the direction of response under consideration is greater than 1.0 s.

spectral response acceleration, S_{xs} =	0.744	g	(BSE-2E seismic hazard)
spectral response acceleration, S_{x1} =	0.405	g	(BSE-2E seismic hazard)
building period, T =	0.149	s	
response spectrum acceleration, S_a =	0.744	g	
effective seismic weight, W =	59.0	kip	
C_1C_2 =	1.4		(Table 12-3 for wood structural panels, m=4.15)
effective mass factor, C_m =	1.0		
seismic lateral force, V =	61.5	kip	



SE23
TREATMENT PLANT
79 08 003
Shear Wall 4
Shear Wall 3
Shear Wall 2
Shear Wall 1
ROOF FRAMING PLAN
1/4"=1'-0"
TYP CORNER DETAIL
1 1/2"=1'-0"
FLOOR PLAN
1/4"=1'-0"
CL STORAGE
CL RM
SHOP
CRANE BEAM ABOVE
THICKENED SLAB BELOW
COLUMN FOOTING BELOW
R-11 BATT INSULATION (TYP)
2x6 STUDS @16"O.C.(TYP)
SEE TYPICAL CORNER DETAIL
DEDUCT ALTERNATE BID
5/8" GWB
2x6 STUDS
1/2" MDO PLYWOOD WHERE SCHEDULED
1/2" GWB
1/2" PLYWOOD SIDING
R-11 BATT INSULATION (TYP)
2x4 BLKG.
MODULE LINE
2x12 FASCIA SUPPORT (TYP)
ROOF DRAIN (2 REQD) SEE DETAIL
2x12 @ 2'-0" O.C. TYP OUTRIGGERS
DOUBLE 2x12 TYP
CRANE BM SUPPORTS
44'-0"
56'-0"
48'-0"
NOTES
1. UNLESS OTHERWISE NOTED, PLAN DIMENSIONS ARE TO MODULE OR CENTERLINES, FACE OF CONCRETE WALLS, AND FACE OF STUDS
2. DO NOT SCALE DRAWINGS, USE NOTED DIMENSIONS
3. ALL STUDS ARE 2x6 @ 16"O.C. MAX.
79 08 003
UTILITIES BUILDING ARCHITECTURAL ROOF FRAMING AND FLOOR PLAN
SHEET 31
OF 75
RECORD DRAWING

Engineers...Working Wonders With Water ™

BY:	BS	DATE	Sep-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Workshop	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (BSE-2E)		

NARROW SHEAR WALL CHECK ALONG EAST ELEVATION

5.5.3.6.1 Stucco, Gypsum Wallboard, Plaster, or Narrow Shear Walls. The overturning and shear demands for noncompliant walls shall be calculated in accordance with Section 5.2.4, and the adequacy shall be evaluated in accordance with Section 5.2.5.

12.4.3.6.2 Strength of Wood Structural Panel Sheathing or Siding Shear Walls. The expected strength of wood structural panel shear walls shall be taken as mean maximum strengths obtained experimentally. Expected strengths of wood structural panel shear walls shall be permitted to be based on 1.5 times yield strengths. Yield strengths shall be determined using LRFD procedures contained in AWC SDPWS, except that the resistance factor, ϕ, shall be taken as 1.0 and expected material properties shall be determined in accordance with Section 12.2.2.

Approved allowable stress values for fasteners shall be permitted to be converted in accordance with Section 12.2.2.5.1 where the strength of a shear wall is computed using principles of mechanics.

For existing wood structural panel shear walls framed with 2-in. nominal framing at adjoining panel edges where 3-in. nominal framing is required per AWC SDPWS, the expected strength shall not be taken as greater than 0.90 times the expected strength associated with use of 3-in. nominal framing at adjoining panel edges.

7.5.2 Linear Procedures

7.5.2.1 Forces and Deformations. Component forces and deformations shall be calculated in accordance with linear analysis procedures of Sections 7.4.1 or 7.4.2.

7.5.2.1.1 Deformation-Controlled Actions for LSP or LDP. Deformation-controlled actions, Q_{UD}, shall be calculated in accordance with Eq. (7-34):

$$Q_{UD} = Q_G + Q_E \qquad (7\text{-}34)$$

where

Q_{UD} = Deformation-controlled action caused by gravity loads and earthquake forces.

Q_G = Action caused by gravity loads as defined in Section 7.2.2; and

Q_E = Action caused by the response to the selected Seismic Hazard Level calculated using either Section 7.4.1 or Section 7.4.2;

12.4.3.6.3 Acceptance Criteria for Wood Structural Panel Sheathing or Siding Shear Walls. For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described in Eq. (12-1), and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

12.3.3.1 Wood Construction. Unless otherwise specified in this standard, connections between wood components of a seismic-force-resisting system shall be considered in accordance with this section. Demands on connectors, including nails, screws, lags, bolts, split rings, and shear plates used to link wood components to other wood or metal components shall be considered deformation-controlled actions. Demands on bodies of connections and bodies of connection hardware shall be considered force-controlled actions.

7.5.2.1.2 Force-Controlled Actions for LSP or LDP. Force-controlled actions, Q_{UF}, shall be calculated using one of the following methods:

1. Q_{UF} shall be taken as the maximum action that can be developed in a component based on a limit-state analysis considering the expected strength of the components delivering force to the component under consideration, or the maximum action developed in the component as limited by the nonlinear response of the building.
2. Alternatively, Q_{UF} shall be calculated in accordance with Eq. (7-35).

$$Q_{UF} = Q_G \pm \frac{\chi Q_E}{C_1 C_2 J} \qquad (7\text{-}35)$$

Based on the findings from Tier 1, the east elevation wall is considered to have narrow shear walls. These shear walls will be checked to resist the seismic load on structure. From ASCE 41-17 Section 12.4.3.6.2, the shear walls will be considered deformation-controlled actions. The anchor connections for these shear walls will be considered force-controlled.

Roof seismic load, V =	61.5	kip	
diaphragm span, L =	44.00	ft	
roof tributary width for seismic, T_w =	22	ft	
wall length, L_{wall} =	48	ft	
effective shear wall length, L_{sweff} =	10.5	ft	(wall lengths considered to act as shear walls)
unit roof seismic load on shear wall, v_E =	0.64	kip/ft	
unit effective base seismic load on shear wall, v_{Eeff} =	2.93	kip/ft	

Wall Double Top Plate Check for Tension & Compression

Diaphragm bending moment, $M = (V/2)*L_{wall}/4$ =	369.0	kip*ft	
Tension/ Compression force on top plate, $T_C = M/L$ =	8.4	kip	
top plate net area, A_{net} =	8.25	in^2	(3-2x6 plates but only one plate effective at joint)
tension/compression stress, $f_{t\text{-}c}$ =	1016.5	psi	

Double Top Plate Check for Tension

design tension value, F_t =	575.0	psi	(assumed Douglas Fir-Larch No. 2)
wet service factor, C_M =	1.0		

temperature factor, C_t =	1.0		
size factor, C_F =	1.0		
incising factor, C_i =	1.0		
format conversion factor for tension, K_F =	2.7		
adjusted tension design stress, F'_t =	1552.5	psi	
knowledge factor, κ =	0.90		
DCR = $f_t/(κ*F'_t)$ =	*0.73*	**OK**	

Double Top Plate Check for Compression

design compression value perpendicular to grain, F_c =	625.0	psi	(assumed Douglas Fir-Larch No. 2)
wet service factor, C_M =	1.0		
temperature factor, C_t =	1.0		
incising factor, C_i =	1.0		
bearing area factor, C_b =	1.0		
format conversion factor for tension, K_F =	2.4		
adjusted compression design stress, F'_c =	1500.0	psi	
knowledge factor, κ =	0.90		
DCR = $f_c/(κ*F'_c)$ =	*0.75*	**OK**	

There is no detail provided to show how the top plates in wall are spliced together. This connection cannot be checked and as such considered deficient. Mitigation is required to provide chord connection.

Shear wall 1 - Shear Wall Strength Check

wall height, h =	15.5	ft
shear wall length, L =	2	ft
shear wall ratio, h/L =	7.75	***> 3.5 (NG)***

As noted in ASCE 41-17 Table 12-3 footnote b, since the aspect ratio exceeds the maximum ratio this wall cannot be considered to act as part of the lateral resisting system.

Shear wall 2 - Shear Wall Strength Check

wall height, h =	15.5	ft	
shear wall length, L =	6	ft	
shear wall ratio, h/L =	2.58	***< 3.5***	
tributary effective seismic shear on shear wall, V_{ueff} =	17.6	kip	
tributary effective seismic moment on shear wall, M_{ueff} =	272.4	kip*ft	
shear wall strength, V_n =	400	lbs/ft	(per AWC SDPWS-2008 Table 4.3B)
expected yield strength, Q_{CE} =	600	lbs/ft	(increased by 1.5 per ASCE 41-17 12.4.3.6.2)
m-factor =	4.15	(interpolated between LS & CP. ASCE 41-17 Table 12-3)	
knowledge factor, κ =	0.90		
wood shear wall strength, κmϕQCE =	13.4	kip	
demand capacity ratio, DCR =	*1.31*	**NG**	

Shear wall 2 Base Plate Anchorage (1/2" expansion anchor @ 4'-0" spacing)

Factor for adjusting action, χ =	1.15	(interpolated between LS & CP)	
C_1C_2 =	1.4		
Force delivery reduction factor, J =	2		
anchor spacing =	4	ft	
Seismic shear force on sill bolt, V_{sill} =	4.81	kip	(Connection considered force-controlled)
Anchor steel shear strength =	5.49	kip	(From Hilti Profis Calculation)
Anchor pryout strength =	10.91	kip	(From Hilti Profis Calculation)
Concrete edge failure strength =	17.84	kip	(From Hilti Profis Calculation)
knowledge factor, κ =	0.90		
steel strength DCR =	*0.97*	**OK**	
pullout strength DCR =	*0.49*	**OK**	
concrete breakout strength DCR =	*0.30*	**OK**	

2x6 Sill Plate check for Shear

Seismic shear force on sill bolt, V_{sill} =	4.81	kip

eference lateral design value for bolt in single shear, Z =	650	lbs	(1/2"Ø bolt in assumed Douglas Fir-Larch)
wet service factor, C_M =	1		
temperature factor, C_t =	1		
group action factor, C_g =	1		
geometry factor, C_Δ =	1		
end grain factor, C_{eg} =	1		
diaphragm factor, C_{di} =	1		
toe-nail factor, C_{tn} =	1		
format conversion factor for tension, K_F =	3.32		
adjusted bolt design value in shear, Z' =	2158	lbs	
knowledge factor, κ =	0.90		
*DCR = V_{sill}/(κ*Z') =*	*2.48*	***NG***	

Drawings do not indicate the use of hold-down hardware to foundation. This is considered a deficiency and will need

Shear wall 3 - Shear Wall Strength Check

wall height, h =	15.5	ft	
shear wall length, L =	4.5	ft	
shear wall ratio, h/L =	3.44	***< 3.5***	
tributary effective seismic shear on shear wall, V_{ueff} =	13.2	kip	
tributary effective seismic moment on shear wall, M_{ueff} =	204.3	kip*ft	
shear wall strength, V_n =	400	lbs/ft	(per AWC SDPWS-2008 Table 4.3B)
expected yield strength, Q_{CE} =	600	lbs/ft	(increased by 1.5 per ASCE 41-17 12.4.3.6.2)
m-factor =	4.15	(interpolated between LS & CP. ASCE 41-17 Table 12-3)	
knowledge factor, κ =	0.90		
wood shear wall strength, κmϕQCE =	10.1	kip	
demand capacity ratio, DCR =	*1.31*	***NG***	

Shear wall 3 Base Plate Anchorage (1/2" expansion anchor @ 4'-0" spacing)

Factor for adjusting action, χ =	1.15	(interpolated between LS & CP)	
C_1C_2 =	1.4		
Force delivery reduction factor, J =	2		
anchor spacing =	4	ft	
Seismic shear force on sill bolt, V_{sill} =	4.81	kip	(Connection considered force-controlled)
Anchor steel shear strength =	5.49	kip	(From Hilti Profis Calculation)
Anchor pryout strength =	10.91	kip	(From Hilti Profis Calculation)
Concrete edge failure strength =	17.84	kip	(From Hilti Profis Calculation)
knowledge factor, κ =	0.90		
steel strength DCR =	*0.97*	***OK***	
pullout strength DCR =	*0.49*	***OK***	
concrete breakout strength DCR =	*0.30*	***OK***	

2x6 Sill Plate check for Shear

Seismic shear force on sill bolt, V_{sill} =	4.81	kip	
eference lateral design value for bolt in single shear, Z =	650	lbs	(1/2"Ø bolt in assumed Douglas Fir-Larch)
wet service factor, C_M =	1		
temperature factor, C_t =	1		
group action factor, C_g =	1		
geometry factor, C_Δ =	1		
end grain factor, C_{eg} =	1		
diaphragm factor, C_{di} =	1		
toe-nail factor, C_{tn} =	1		
format conversion factor for tension, K_F =	3.32		
adjusted bolt design value in shear, Z' =	2158	lbs	
knowledge factor, κ =	0.90		
*DCR = V_{sill}/(κ*Z') =*	*2.48*	***NG***	

Drawings do not indicate the use of hold-down hardware to foundation. This is considered a deficiency and will need

Shear wall 4 - Shear Wall Strength Check

wall height, h =	15.5	ft	
shear wall length, L =	2.5	ft	
shear wall ratio, h/L =	6.20	***> 3.5 (NG)***	

As noted in ASCE 41-17 Table 12-3 footnote b, since the aspect ratio exceeds the maximum ratio this wall cannot be considered to act as part of the lateral resisting system.

Table 4.3B Nominal Unit Shear Capacities for Wood-Frame Shear Walls[1,2,5,6]

Wood Structural Panels Applied over 1/2" or 5/8" Gypsum Wallboard or Gypsum Sheathing Board

Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimum Fastener Penetration in Framing Member or Blocking (in.)	Fastener Type & Size	A SEISMIC Panel Edge Fastener Spacing (in.) 6			4			3			2			B WIND Panel Edge Fastener Spacing (in.) 6	4	3	2
				v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_w (plf)	v_w (plf)	v_w (plf)	v_w (plf)
			Nail (common or galvanized box)		OSB	PLY		OSB	PLY		OSB	PLY		OSB	PLY				
Wood Structural Panels - Structural I[3,4]	5/16	1-1/4	8d	400	13	10	600	18	13	780	23	16	1020	35	22	560	840	1090	1430
	3/8, 7/16, 15/32	1-3/8	10d	560	14	11	860	18	14	1100	24	17	1460	37	23	785	1205	1540	2045
Wood Structural Panels - Sheathing[3,4]	5/16	1-1/4	8d	360	13	9.5	540	18	12	700	24	14	900	37	18	505	755	980	1260
	3/8			400	11	8.5	600	15	11	780	20	13	1020	32	17	560	840	1090	1430
	3/8, 7/16, 15/32	1-3/8	10d	520	13	10	760	19	13	980	25	15	1280	39	20	730	1065	1370	1790
			Nail (galvanized casing)																
Plywood Siding	5/16	1-1/4	8d (2-1/2" x0.113")	280		13	420		16	550		17	720		21	390	590	770	1010
	3/8	1-3/8	10d (3"x0.128")	320		16	480		18	620		20	820		22	450	670	870	1150

1. Nominal unit shear capacities shall be adjusted in accordance with 4.3.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.3.6. For specific requirements, see 4.3.7.1 for wood structural panel shear walls. See Appendix A for common and box nail dimensions.
2. For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = [1-(0.5-G)], where G = Specific Gravity of the framing lumber from the *NDS* (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
3. Apparent shear stiffness values, G_a, are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for shear walls constructed with either OSB or 3 ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, G_a values for plywood shall be permitted to be multiplied by 1.2.
4. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
5. Where panels are applied on both faces of a shear wall and nail spacing is less than 6" on center on either side, panel joints shall be offset to fall on different framing members. Alternatively, the width of the nailed face of framing members shall be 3" nominal or greater at adjoining panel edges and nails at all panel edges shall be staggered.
6. Galvanized nails shall be hot-dipped or tumbled.

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Company:	Carollo Engineers	Page:	1
Address:		Specifier:	B. Stuetzel
Phone I Fax:	|	E-Mail:	
Design:	Workshop - Sill plate anchorage check	Date:	9/17/2021
Fastening point:			

Specifier's comments: City of Wilsonville - Workshop - Shear Wall Sill Plate Anchorage into Concrete Foundation

1 Input data

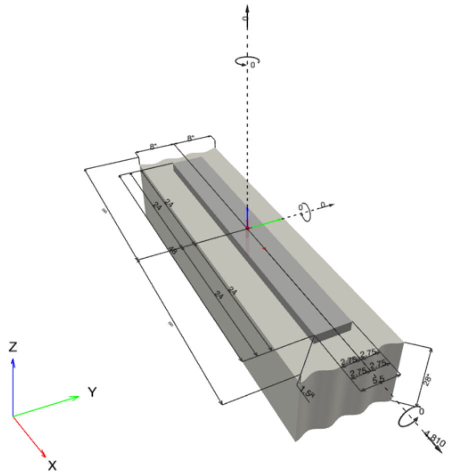
Anchor type and diameter:	**Kwik Bolt TZ - CS 1/2 (3 1/4)**
Item number:	not available
Effective embedment depth:	$h_{ef,act}$ = 3.250 in., h_{nom} = 3.625 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-1917
Issued I Valid:	1/1/2020 | 5/1/2021
Proof:	Design Method ACI 318-14 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.500 in.
Anchor plate^R :	l_x x l_y x t = 48.000 in. x 5.500 in. x 1.500 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 28.000 in.
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d))
	Shear load: yes (17.2.3.5.3 (c))

Note: the Kwik Bolt TZ - CS anchor is in the process of phase-out.

Application also possible with Kwik Bolt TZ2 - CS under the selected boundary conditions.

^{R} - The anchor calculation is based on a rigid anchor plate assumption.

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



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Design:	Workshop - Sill plate anchorage check	Date:	9/17/2021
Fastening point:			

1.1 Design results

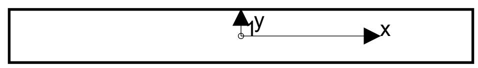
Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 4,810; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	yes	135

2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	4,810	4,810	0



max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Fastening point:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	4,810	5,495	88	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	4,810	10,911	44	OK
Concrete edge failure in direction y+**	4,810	17,840	27	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} [lb]	$\alpha_{V,seis}$	ϕ	ϕ V_{sa} [lb]	V_{ua} [lb]
5,495	1.000	1.000	5,495	4,810

4.2 Pryout Strength

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$c_{a,min}$ [in.]	k_{cp}	c_{ac} [in.]	$\psi_{c,N}$
95.06	95.06	8.000	2	6.000	1.000

$e_{c1,V}$ [in.]	$\psi_{ec1,V}$	$e_{c2,V}$ [in.]	$\psi_{ec2,V}$	$\psi_{ed,N}$	$\psi_{cp,N}$	k_{cr}
0.000	1.000	0.000	1.000	1.000	1.000	17

λ_a	N_b [lb]	ϕ	$\phi_{seismic}$	ϕ V_{cpg} [lb]	V_{ua} [lb]
1.000	5,455	1.000	1.000	10,911	4,810

4.3 Concrete edge failure in direction y+

l_e [in.]	d_a [in.]	c_{a1} [in.]	A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]
3.250	0.500	8.000	288.00	288.00

$\psi_{ed,V}$	$\psi_{parallel,V}$	$e_{c,V}$ [in.]	$\psi_{ec,V}$	$\psi_{c,V}$	$\psi_{h,V}$
1.000	2.000	0.000	1.000	1.000	1.000

λ_a	V_b [lb]	ϕ	$\phi_{seismic}$	ϕ V_{cbg} [lb]	V_{ua} [lb]
1.000	8,920	1.000	1.000	17,840	4,810

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Phone I Fax:	|	E-Mail:	
Design:	Workshop - Sill plate anchorage check	Date:	9/17/2021
Fastening point:			

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2018, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-14, Section 17.8.1.

Fastening meets the design criteria!

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Company:	Carollo Engineers	Page:	5
Address:		Specifier:	B. Stuetzel
Phone I Fax:	|	E-Mail:	
Design:	Workshop - Sill plate anchorage check	Date:	9/17/2021
Fastening point:			

6 Installation data

Profile: no profile

Hole diameter in the fixture: d_f = 0.562 in.

Plate thickness (input): 1.500 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Anchor type and diameter: Kwik Bolt TZ - CS 1/2 (3 1/4)

Item number: not available

Maximum installation torque: 480 in.lb

Hole diameter in the base material: 0.500 in.

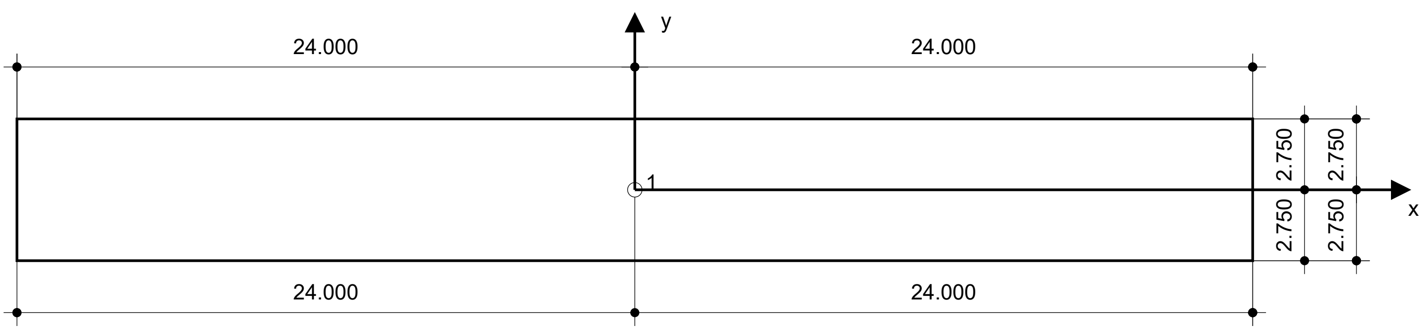
Hole depth in the base material: 4.000 in.

Minimum thickness of the base material: 8.000 in.

Hilti KB-TZ stud anchor with 3.625 in embedment, 1/2 (3 1/4), Carbon steel, installation per ESR-1917

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Torque wrench • Hammer



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-0.000	0.000	-	-	8.000	8.000

7 Remarks; Your Cooperation Duties

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BY:	BS	DATE	Aug-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Workshop	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (CSZ)		

SEISMIC BASE SHEAR FOR WORKSHOP

7.4.1.3.1 Pseudo Seismic Force for LSP. The pseudo lateral force in a given horizontal direction of a building shall be determined using Eq. (7-21). This force shall be used to evaluate or retrofit the vertical elements of the seismic-force-resisting system.

$$V = C_1 C_2 C_m S_a W \qquad (7-21)$$

Table 7-3. Alternate Values for Modification Factors C_1C_2

Fundamental Period	$m_{max} < 2$	$2 \le m_{max} < 6$	$m_{max} \ge 6$
$T \le 0.3$	1.1	1.4	1.8
$0.3 < T \le 1.0$	1.0	1.1	1.2
$T > 1.0$	1.0	1.0	1.1

Table 7-4. Values for Effective Mass Factor C_m

No. of Stories	Concrete Moment Frame	Concrete Shear Wall	Concrete Pier-Spandrel	Steel Moment Frame	Steel Concentrically Braced Frame	Steel Eccentrically Braced Frame	Other
1–2	1.0	1.0	1.0	1.0	1.0	1.0	1.0
3 or more	0.9	0.8	0.8	0.9	0.9	0.9	1.0

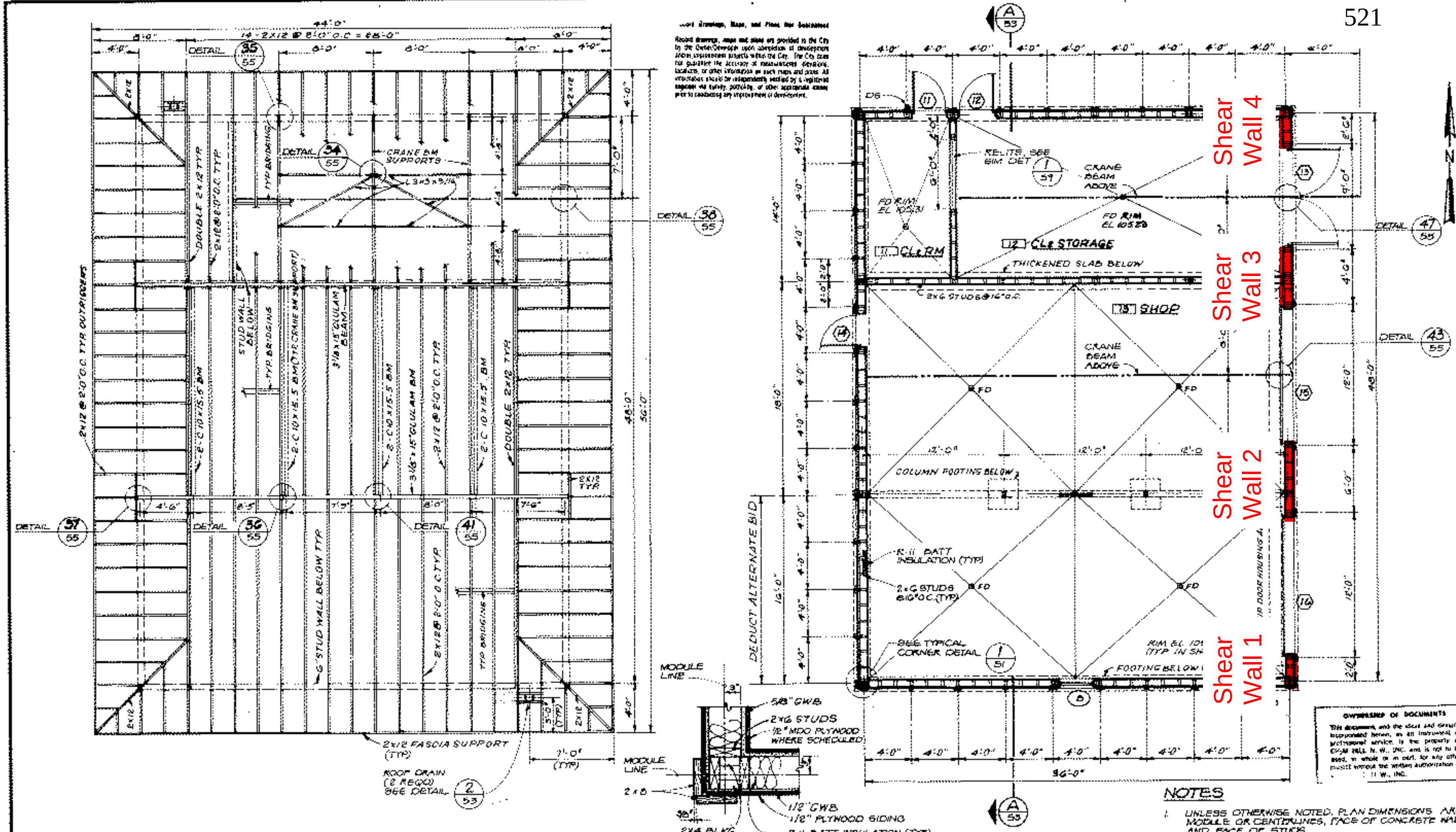
Note: C_m shall be taken as 1.0 if the fundamental period, T, in the direction of response under consideration is greater than 1.0 s.

spectral response acceleration, S_{xs} =	0.446	g	(CSZ seismic hazard)
spectral response acceleration, S_{x1} =	0.332	g	(CSZ seismic hazard)
building period, T =	0.149	s	
response spectrum acceleration, S_a =	0.446	g	
effective seismic weight, W =	59	kip	
C_1C_2 =	1.4		(Table 12-3 for wood structural panels, m=2.75)
effective mass factor, C_m =	1.0		
seismic lateral force, V =	36.8	kip	

SE23

TREATMENT PLANT

79 08 003



ROOF FRAMING PLAN
1/4"=1'-0"

TYP CORNER DETAIL 1 51
1 1/2"=1'-0"

FLOOR PLAN
1/4"=1'-0"

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NOTES

1. UNLESS OTHERWISE NOTED, PLAN DIMENSIONS ARE TO MODULE OR CENTERLINES, FACE OF CONCRETE WALLS, AND FACE OF STUDS
2. DO NOT SCALE DRAWINGS, USE NOTED DIMENSIONS
3. ALL STUDS ARE 2x6 @ 16" O.C. MAX.

79 08 003

CH2M HILL	DES. JJP / DR. / CHK. KLM / APPD. SLD	Record Drawing	CITY OF WILSONVILLE, OREGON PHASE III SEWAGE TREATMENT PLANT EXPANSION
UTILITIES BUILDING ARCHITECTURAL ROOF FRAMING AND FLOOR PLAN	SHEET B1	OF 75	DATE AUG. 1978
DWG. NO. P12508.A			

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BY:	BS	DATE	Sep-21	CLIENT	City of Wilsonville	SHEET	
CHKD BY		DESCRIPTION			Workshop	JOB NO.	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (CSZ)		

NARROW SHEAR WALL CHECK ALONG EAST ELEVATION

5.5.3.6.1 Stucco, Gypsum Wallboard, Plaster, or Narrow Shear Walls. The overturning and shear demands for noncompliant walls shall be calculated in accordance with Section 5.2.4, and the adequacy shall be evaluated in accordance with Section 5.2.5.

12.4.3.6.2 Strength of Wood Structural Panel Sheathing or Siding Shear Walls. The expected strength of wood structural panel shear walls shall be taken as mean maximum strengths obtained experimentally. Expected strengths of wood structural panel shear walls shall be permitted to be based on 1.5 times yield strengths. Yield strengths shall be determined using LRFD procedures contained in AWC SDPWS, except that the resistance factor, ϕ, shall be taken as 1.0 and expected material properties shall be determined in accordance with Section 12.2.2.

Approved allowable stress values for fasteners shall be permitted to be converted in accordance with Section 12.2.2.5.1 where the strength of a shear wall is computed using principles of mechanics.

For existing wood structural panel shear walls framed with 2-in. nominal framing at adjoining panel edges where 3-in. nominal framing is required per AWC SDPWS, the expected strength shall not be taken as greater than 0.90 times the expected strength associated with use of 3-in. nominal framing at adjoining panel edges.

7.5.2 Linear Procedures

7.5.2.1 Forces and Deformations. Component forces and deformations shall be calculated in accordance with linear analysis procedures of Sections 7.4.1 or 7.4.2.

7.5.2.1.1 Deformation-Controlled Actions for LSP or LDP. Deformation-controlled actions, Q_{UD}, shall be calculated in accordance with Eq. (7-34):

$$Q_{UD} = Q_G + Q_E \qquad (7\text{-}34)$$

where

Q_{UD} = Deformation-controlled action caused by gravity loads and earthquake forces.

Q_G = Action caused by gravity loads as defined in Section 7.2.2; and

Q_E = Action caused by the response to the selected Seismic Hazard Level calculated using either Section 7.4.1 or Section 7.4.2;

12.4.3.6.3 Acceptance Criteria for Wood Structural Panel Sheathing or Siding Shear Walls. For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 12-3. For nonlinear procedures, the coordinates of the generalized force–deformation relation, described in Eq. (12-1), and deformation acceptance criteria for primary and secondary components shall be taken from Table 12-4.

12.3.3.1 Wood Construction. Unless otherwise specified in this standard, connections between wood components of a seismic-force-resisting system shall be considered in accordance with this section. Demands on connectors, including nails, screws, lags, bolts, split rings, and shear plates used to link wood components to other wood or metal components shall be considered deformation-controlled actions. Demands on bodies of connections and bodies of connection hardware shall be considered force-controlled actions.

7.5.2.1.2 Force-Controlled Actions for LSP or LDP. Force-controlled actions, Q_{UF}, shall be calculated using one of the following methods:

1. Q_{UF} shall be taken as the maximum action that can be developed in a component based on a limit-state analysis considering the expected strength of the components delivering force to the component under consideration, or the maximum action developed in the component as limited by the nonlinear response of the building.
2. Alternatively, Q_{UF} shall be calculated in accordance with Eq. (7-35).

$$Q_{UF} = Q_G \pm \frac{\chi Q_E}{C_1 C_2 J} \qquad (7\text{-}35)$$

Based on the findings from Tier 1, the east elevation wall is considered to have narrow shear walls. These shear walls will be checked to resist the seismic load on structure. From ASCE 41-17 Section 12.4.3.6.2, the shear walls will be considered deformation-controlled actions. The anchor connections for these shear walls will be considered force-controlled.

Roof seismic load, V =	36.8	kip	
diaphragm span, L =	44.00	ft	
roof tributary width for seismic, T_w =	22	ft	
wall length, L_{wall} =	48	ft	
effective shear wall length, L_{sweff} =	10.5	ft	(wall lengths considered to act as shear walls)
unit roof seismic load on shear wall, v_E =	0.38	kip/ft	
unit effective base seismic load on shear wall, v_{Eeff} =	1.75	kip/ft	

Wall Double Top Plate Check for Tension & Compression

Diaphragm bending moment, M = (V/2)*L_{wall}/4 =	220.8	kip*ft	
Tension/ Compression force on top plate, T_C = M/L =	5.0	kip	
top plate net area, A_{net} =	8.25	in^2	(3-2x6 plates but only one plate effective at joint)
tension/compression stress, f_{t-c} =	608.3	psi	

Double Top Plate Check for Tension

design tension value, F_t =	575.0	psi	(assumed Douglas Fir-Larch No. 2)
wet service factor, C_M =	1.0		

temperature factor, C_t =	1.0	
size factor, C_F =	1.0	
incising factor, C_i =	1.0	
format conversion factor for tension, K_F =	2.7	
adjusted tension design stress, F'_t =	1552.5	psi
knowledge factor, κ =	0.90	
DCR = $f_t/(\kappa * F'_t)$ =	*0.44*	***OK***

Double Top Plate Check for Compression

design compression value perpendicular to grain, F_c =	625.0	psi	(assumed Douglas Fir-Larch No. 2)
wet service factor, C_M =	1.0		
temperature factor, C_t =	1.0		
incising factor, C_i =	1.0		
bearing area factor, C_b =	1.0		
format conversion factor for tension, K_F =	2.4		
adjusted compression design stress, F'_c =	1500.0	psi	
knowledge factor, κ =	0.90		
DCR = $f_c/(\kappa * F'_c)$ =	*0.45*	***OK***	

There is no detail provided to show how the top plates in wall are spliced together. This connection cannot be checked and as such considered deficient. Mitigation is required to provide chord connection.

<u>Shear wall 1 - Shear Wall Strength Check</u>

wall height, h =	15.5	ft
shear wall length, L =	2	ft
shear wall ratio, h/L =	7.75	***> 3.5 (NG)***

As noted in ASCE 41-17 Table 12-3 footnote b, since the aspect ratio exceeds the maximum ratio this wall cannot be considered to act as part of the lateral resisting system.

<u>Shear wall 2 - Shear Wall Strength Check</u>

wall height, h =	15.5	ft	
shear wall length, L =	6	ft	
shear wall ratio, h/L =	2.58	***< 3.5***	
tributary seismic shear on shear wall, Vu =	10.5	kip	
tributary seismic moment on shear wall, Mu =	163.0	kip*ft	
shear wall strength, V_n =	400	lbs/ft	(per AWC SDPWS-2008 Table 4.3B)
expected yield strength, Q_{CE} =	600	lbs/ft	(increased by 1.5 per ASCE 41-17 12.4.3.6.2
m-factor =	2.75	(interpolated between LS & IO. ASCE 41-17 Table 12-3)	
knowledge factor, κ =	0.90		
wood shear wall strength, κmϕQCE =	8.9	kip	
demand capacity ratio, DCR =	*1.18*	***NG***	

<u>Shear wall 2 Base Plate Anchorage (1/2" expansion anchor @ 4'-0" spacing)</u>

Factor for adjusting action, χ =	1.3	(interpolated between LS & IO)	
C_1C_2 =	1.4		
Force delivery reduction factor, J =	2		
anchor spacing =	4	ft	
Seismic shear force on sill bolt, V_{sill} =	3.25	kip	(Connection considered force-controlled)
Anchor steel shear strength =	5.49	kip	(From Hilti Profis Calculation)
Anchor pryout strength =	10.91	kip	(From Hilti Profis Calculation)
Concrete edge failure strength =	17.84	kip	(From Hilti Profis Calculation)
knowledge factor, κ =	0.90		
steel strength DCR =	*0.66*	***OK***	
pullout strength DCR =	*0.33*	***OK***	
concrete breakout strength DCR =	*0.20*	***OK***	

2x6 Sill Base Plate check for Shear

Seismic shear force on sill bolt, V_{sill} =	3.25	kip

eference lateral design value for bolt in single shear, Z =	650	lbs	(1/2"Ø bolt in assumed Douglas Fir-Larch)
wet service factor, C_M =	1		
temperature factor, C_t =	1		
group action factor, C_g =	1		
geometry factor, C_Δ =	1		
end grain factor, C_{eg} =	1		
diaphragm factor, C_{di} =	1		
toe-nail factor, C_{tn} =	1		
format conversion factor for tension, K_F =	3.32		
adjusted bolt design value in shear, Z' =	2158	lbs	
knowledge factor, κ =	0.90		
*DCR = V_{sill}/(κ*Z') =*	*1.68*	***NG***	

Drawings do not indicate the use of hold-down hardware to foundation. This is considered a deficiency and will need

Shear wall 3 - Shear Wall Strength Check

wall height, h =	15.5	ft	
shear wall length, L =	4.5	ft	
shear wall ratio, h/L =	3.44	***< 3.5***	
tributary seismic shear on shear wall, Vu =	7.9	kip	
tributary seismic moment on shear wall, Mu =	122.2	kip*ft	
shear wall strength, V_n =	400	lbs/ft	(per AWC SDPWS-2008 Table 4.3B)
expected yield strength, Q_{CE} =	600	lbs/ft	(increased by 1.5 per ASCE 41-17 12.4.3.6.2
m-factor =	2.75	(interpolated between LS & IO. ASCE 41-17 Table 12-3)	
knowledge factor, κ =	0.90		
wood shear wall strength, κmφQCE =	6.7	kip	
demand capacity ratio, DCR =	*1.18*	***NG***	

Shear wall 3 Base Plate Anchorage (1/2" expansion anchor @ 4'-0" spacing)

Factor for adjusting action, χ =	1.3	(interpolated between LS & IO)	
C_1C_2 =	1.4		
Force delivery reduction factor, J =	2		
anchor spacing =	4	ft	
Seismic shear force on sill bolt, V_{sill} =	3.25	kip	(Connection considered force-controlled)
Anchor steel shear strength =	5.49	kip	(From Hilti Profis Calculation)
Anchor pryout strength =	10.91	kip	(From Hilti Profis Calculation)
Concrete edge failure strength =	17.84	kip	(From Hilti Profis Calculation)
knowledge factor, κ =	0.90		
steel strength DCR =	*0.66*	***OK***	
pullout strength DCR =	*0.33*	***OK***	
concrete breakout strength DCR =	*0.20*	***OK***	

2x6 Sill Plate check for Shear

Seismic shear force on sill bolt, V_{sill} =	3.25	kip	
eference lateral design value for bolt in single shear, Z =	650	lbs	(1/2"Ø bolt in assumed Douglas Fir-Larch)
wet service factor, C_M =	1		
temperature factor, C_t =	1		
group action factor, C_g =	1		
geometry factor, C_Δ =	1		
end grain factor, C_{eg} =	1		
diaphragm factor, C_{di} =	1		
toe-nail factor, C_{tn} =	1		
format conversion factor for tension, K_F =	3.32		
adjusted bolt design value in shear, Z' =	2158	lbs	
knowledge factor, κ =	0.90		
*DCR = V_{sill}/(κ*Z') =*	*1.68*	***NG***	

Drawings do not indicate the use of hold-down hardware to foundation. This is considered a deficiency and will need

Shear wall 4 - Shear Wall Strength Check

wall height, h =	15.5	ft	
shear wall length, L =	2.5	ft	
shear wall ratio, h/L =	6.20	***> 3.5 (NG)***	

As noted in ASCE 41-17 Table 12-3 footnote b, since the aspect ratio exceeds the maximum ratio this wall cannot be considered to act as part of the lateral resisting system.

Table 4.3B Nominal Unit Shear Capacities for Wood-Frame Shear Walls[1,2,5,6]

Wood Structural Panels Applied over 1/2" or 5/8" Gypsum Wallboard or Gypsum Sheathing Board

Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimum Fastener Penetration in Framing Member or Blocking (in.)	Fastener Type & Size	A SEISMIC Panel Edge Fastener Spacing (in.)												B WIND Panel Edge Fastener Spacing (in.)			
				6			4			3			2			6	4	3	2
				v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_w (plf)	v_w (plf)	v_w (plf)	v_w (plf)
			Nail (common or galvanized box)		OSB	PLY		OSB	PLY		OSB	PLY		OSB	PLY				
Wood Structural Panels - Structural I[3,4]	5/16	1-1/4	6d	400	13	10	600	18	13	780	23	16	1020	35	22	560	840	1090	1430
	3/8, 7/16, 15/32	1-3/8	10d	560	14	11	860	18	14	1100	24	17	1460	37	23	785	1205	1540	2045
Wood Structural Panels - Sheathing[3,4]	5/16	1-1/4	6d	360	13	9.5	540	18	12	700	24	14	900	37	18	505	755	980	1260
	3/8			400	11	8.5	600	15	11	780	20	13	1020	32	17	560	840	1090	1430
	3/8, 7/16, 15/32	1-3/8	10d	520	13	10	760	19	13	980	25	15	1280	39	20	730	1065	1370	1790
			Nail (galvanized casing)																
Plywood Siding	5/16	1-1/4	6d (2-1/2" x0.113")	280		13	420		16	550		17	720		21	390	590	770	1010
	3/8	1-3/8	10d (3"x0.128")	320		16	480		18	620		20	820		22	450	670	870	1150

1. Nominal unit shear capacities shall be adjusted in accordance with 4.3.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.3.6. For specific requirements, see 4.3.7.1 for wood structural panel shear walls. See Appendix A for common and box nail dimensions.
2. For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = [1-(0.5-G)], where G = Specific Gravity of the framing lumber from the *NDS* (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
3. Apparent shear stiffness values, G_a, are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for shear walls constructed with either OSB or 3 ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, G_a values for plywood shall be permitted to be multiplied by 1.2.
4. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
5. Where panels are applied on both faces of a shear wall and nail spacing is less than 6" on center on either side, panel joints shall be offset to fall on different framing members. Alternatively, the width of the nailed face of framing members shall be 3" nominal or greater at adjoining panel edges and nails at all panel edges shall be staggered.
6. Galvanized nails shall be hot-dipped or tumbled.

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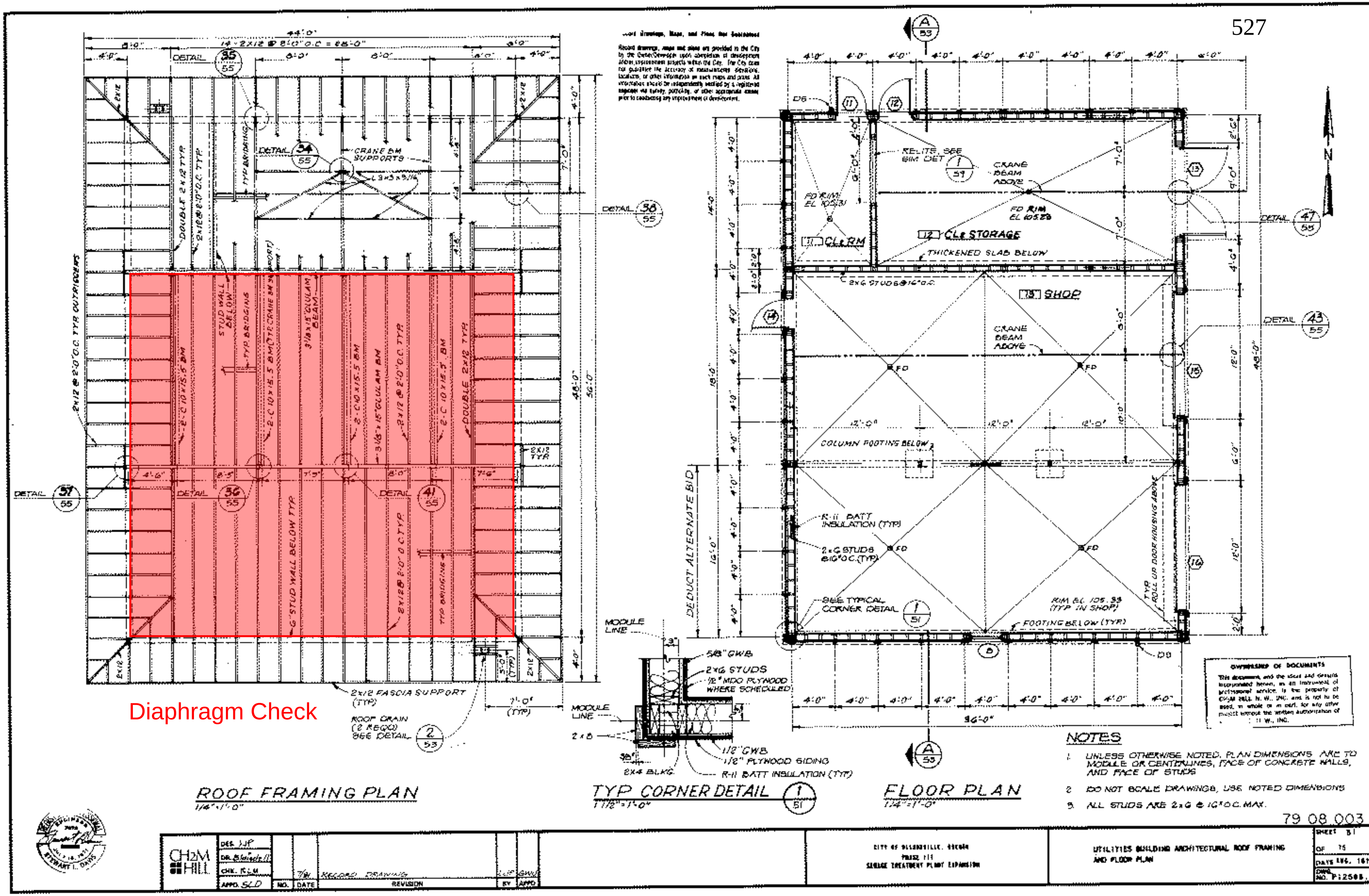

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BY:	BS	**DATE**	Aug-21	**CLIENT**	City of Wilsonville	**SHEET**	
CHKD BY		**DESCRIPTION**			Workshop	**JOB NO.**	11962A.00
DESIGN TASK					ASCE 41-17 - Tier 2 (CSZ)		

WOOD DIAPHRAGM CHECK

12.5.3.6.2 Strength of Wood Structural Panel Sheathing Diaphragms. The expected strength of wood structural panel diaphragms shall be taken as mean maximum strengths obtained experimentally. Expected strengths shall be permitted to be based on 1.5 times yield strengths of wood structural panel diaphragms. Yield strengths shall be determined using LRFD procedures contained in AWC SDPWS, except that the resistance factor, ϕ, shall be taken as 1.0 and expected material properties shall be determined in accordance with Section 12.2.2.

For existing wood structural panel diaphragms framed with 2-in. nominal framing at adjoining panel edges where 3-in. nominal framing is required per AWC SDPWS, the expected strength shall not be taken as greater than 0.80 times the expected strength associated with use of 3-in. nominal framing at adjoining panel edges.

Approved allowable stress values for fasteners shall be permitted to be converted in accordance with Section 12.2.2.5.1 where the strength of a diaphragm is computed using principles of mechanics.

The expected shear capacity of unchorded diaphragms shall be calculated by multiplying the values given for chorded diaphragms by 0.60.

5.6.2 Procedures for Wood Diaphragms. For wood diaphragms with noncompliant spans or aspect ratios, an analysis of the diaphragm shall be performed in accordance with Section 5.2.4, and the adequacy of the diaphragm system shall be evaluated in accordance with Section 5.2.5. The diaphragm deflection shall be calculated, and the adequacy of the vertical-load-carrying elements at the maximum deflection, including P-delta effects, shall be evaluated.

From Tier 1, the roof diaphragm exceeds the 30ft span between lateral resisting members. The diaphragm will be considered unblocked as there is no indication if blocking is used between members. Nailing pattern is assumed 8d@6"oc for 1/2" plywood. Diaphragm is assumed to be deformation-controlled.

Roof seismic load, V =	38.6	kip	
diaphragm span, L =	56.00	ft	
roof unit diaphragm load, v =	0.69	kip/ft	
Roof span between shear walls, L_1 =	34.00	ft	
Roof depth, d =	36.00	ft	
diaphragm shear, v_1 =	0.325	kip/ft	
diaphragm strength, V_N =	360	lbs/ft	(per AWC SDPWS-2008 Table 4.2C)
expected diaphragm strength, Q_{CE} =	540	lbs/ft	(expected strength shall be 1.5x the allowable)
m-factor =	2	(interpolated between LS & IO. ASCE 41-17 Table 12-3)	
knowledge factor, κ =	0.90		
diaphragm strength, $\kappa m\phi Q_{CE}$ =	0.972	kip/ft	
demand capacity ratio, DCR =	*0.33*	***OK***	

Checking diaphragm deflection in E-W direction. Deflection will be calculated per ASCE 41-17 Eq. 12-3.

$$\Delta_y = 5v_yL^3/(8EAb) + v_yL/(4G_d) + \Sigma(\Delta_c X)/(2b) \qquad (12\text{-}3)$$

roof unit diaphragm load, v =	689.3	lb/ft	
diaphragm span, L =	56.00	ft	
modulus of elasticity, E =	1700000	psi	
area of diaphragm chord, A =	34.5	in2	
diaphragm width, b =	44	ft	
diaphragm shear stiffness, G_d =	8000	lb/in	
sum of individual chord splice slip values, Σ(ΔcX) =	1.375	in*ft	(assumed one splice at midspan of wall)
diaphragm deflection, Δy =	1.25	in	

Checking wall adequacy to resist P-delta effects due to deflection calculated. 2x6 stud will be checked.

roof DL =	16.7	psf
tributary length of roof DL =	6	ft

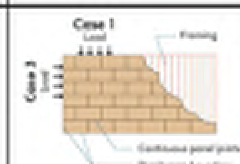
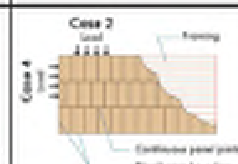
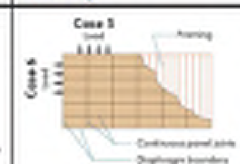
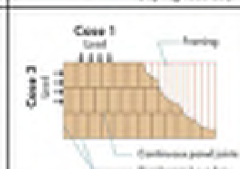
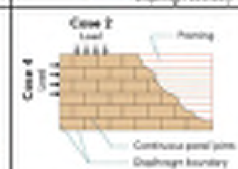
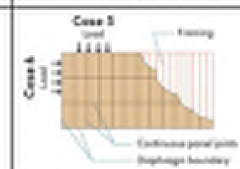
wall stud spacing =	16	in	
unit vertical load on single stud, P_u =	133.6	lbs	
diaphragm deflection, Δy =	1.25	in	
moment due to P-delta effect, P_u*Δy =	167.2	lbs*in	
stud area, A =	8.25	in2	
stud wall height, l_e =	15.5	ft	
stud depth, d =	5.5	in	
l_e/d =	33.8		
modulus of elasticity, E_{min} =	510000	psi	
wet service factor, CM =	1		
temperature factor, Ct =	1		
incising factor, Ci =	1		
adjusted modulus of elasticity, E'_{min} =	510000	psi	
$FcE = 0.822E'_{min}/(l_e/d)^2$ =	366.6	psi	
design value, Fc =	850	psi	(assumed Douglas Fir-Larch)
design value, Fbn =	700	psi	(assumed Douglas Fir-Larch)
format conversion factor for compression, KD =	2.4		
format conversion factor for bending, KD =	2.54		
size factor, CF =	1		
adjusted F"c =	2040	psi	
adjusted F'bn =	1778	psi	
c =	0.8		
FcE/F"c =	0.180		
(1+FcE/F"c)/(2c) =	0.737		
column stability factor, CP =	0.172		
adjusted F'c =	351.9	psi	
axial strength, P_n =	2903.1	lbs	
section modulus, S =	7.6	in3	
bending strength, Mn = F'bn*S =	13446.1	lbs*in	
knowledge factor, κ =	0.90		
DCR = $P_u/(\kappa*P_n)$ =	*0.05*	***OK***	
DCR = $M_u/(\kappa*M_n)$ =	*0.01*	***OK***	
Combined DCR =	*0.06*	***OK***	

Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Unblocked Wood Structural Panel Diaphragms[1,2,3,4,5]

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Supported Edges and Boundaries (in.)	A SEISMIC — 6 in. Nail Spacing at diaphragm boundaries and supported panel edges — Case 1 — v_s (plf)	Case 1 — G_a (kips/in.) OSB	Case 1 — G_a (kips/in.) PLY	Cases 2,3,4,5,6 — v_s (plf)	Cases 2,3,4,5,6 — G_a (kips/in.) OSB	Cases 2,3,4,5,6 — G_a (kips/in.) PLY	B WIND — 6 in. Nail Spacing at diaphragm boundaries and supported panel edges — Case 1 — v_w (plf)	Cases 2,3,4,5,6 — v_w (plf)
Structural I	6d	1-1/4	5/16	2	330	9.0	7.0	250	6.0	4.5	460	350
				3	370	7.0	6.0	280	4.5	4.0	520	390
	8d	1-3/8	3/8	2	480	8.5	7.0	360	6.0	4.5	670	505
				3	530	7.5	6.0	400	5.0	4.0	740	560
	10d	1-1/2	15/32	2	570	14	10	430	9.5	7.0	800	600
				3	640	12	9.0	480	8.0	6.0	895	670
Sheathing and Single-Floor	6d	1-1/4	5/16	2	300	9.0	6.5	220	6.0	4.0	420	310
				3	340	7.0	5.5	250	5.0	3.5	475	350
			3/8	2	330	7.5	5.5	250	5.0	4.0	460	350
				3	370	6.0	4.5	280	4.0	3.0	520	390
	8d	1-3/8	3/8	2	430	9.0	6.5	320	6.0	4.5	600	450
				3	480	7.5	5.5	360	5.0	3.5	670	505
			7/16	2	460	8.5	6.0	340	5.5	4.0	645	475
				3	510	7.0	5.5	380	4.5	3.5	715	530
			15/32	2	480	7.5	5.5	360	5.0	4.0	670	505
				3	530	6.5	5.0	400	4.0	3.5	740	560
	10d	1-1/2	15/32	2	510	15	9.0	380	10	6.0	715	530
				3	580	12	8.0	430	8.0	5.5	810	600
			19/32	2	570	13	8.5	430	8.5	5.5	800	600
				3	640	10	7.5	480	7.0	5.0	895	670

1. Nominal unit shear capacities shall be adjusted in accordance with 4.2.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.6. For specific requirements, see 4.2.7.1 for wood structural panel diaphragms. See Appendix A for common nail dimensions.
2. For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = [1-(0.5-G)], where G = Specific Gravity of the framing lumber from the *NDS* (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
3. Apparent shear stiffness values, G_a, are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for diaphragms constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, G_a values shall be permitted to be multiplied by 1.2.
4. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
5. Diaphragm resistance depends on the direction of continuous panel joints with respect to the loading direction and direction of framing members, and is independent of the panel orientation.

	Cases 1&3: Continuous Panel Joints Perpendicular to Framing	Cases 2&4: Continuous Panel Joints Parallel to Framing	Cases 5&6: Continuous Panel Joints Perpendicular and Parallel to Framing
Long Panel Direction Perpendicular to Supports			
Long Panel Direction Parallel to Supports[a]			

(a) Panel span rating for out-of-plane loads may be lower than the span rating with the long panel direction perpendicular to supports (See Section 3.2.2 and Section 3.2.3)

Appendix C

SEISMIC RETROFIT COST ESTIMATE

Project Name: Operations Building
Project Number: 11962A.00
Project Construction Duration:

Date Prepared: 6/3/2022
Prepared By: B. Stuetzel
Date Accepted:
Accepted By:

				MATERIALS		INSTALLATION		TOTAL	TOTAL	
		QTY.	Unit	Unit Cost	Amount	per UM	Amount	Direct Cost	Direct Cost	Reference
General Conditions	**Mitigation**									
	Temporary trailers for staffing	6	MO			$ 24,000	$ 144,000	$ 144,000		
									$ 144,000	
Deficiency No. S1/S2	**Mitigation**									
No diaphragm ties in the N-S direction to transfer diaphragm forces into the lateral resisting system.	Add new steel beam	80	FT	$ 79	$ 6,320	$ 7	$ 566	$ 6,886		RS Means. Assumed W18x50.
	Add new beam anchorage	8	EA			$ 500	$ 4,000	$ 4,000		$500/connection. Estimate
	Add new steel plate	30	FT	$ 51	$ 6,150	$ 8	$ 225	$ 6,375		RS Means. 25 lbs/ft Gal Steel.
	Epoxy anchors at 6" OC	64	EA	$ 71	$ 4,544	$ 38	$ 2,432	$ 6,976		RS Means
	Field welding of steel plate to beam	4	EA			$ 2,400	$ 9,600	$ 9,600		Estimate
	Construction difficulty, operations and work restrictions	1	LS					$ 33,837		100% of other costs
									$ 67,673	
Deficiency No. NS1	**Mitigation**									
The ceiling edges don't provide adequate gap clearance to wall for movement.	Demo and restoration of interior ceiling system	155	SF			$ 75	$ 11,625	$ 11,625		$75/SF.
	Construction difficulty, operations and work restrictions	1	LS					$ 5,813		50% of other costs
									$ 17,438	
Deficiency No. NS2	**Mitigation**									
Lens covers over lights lack safety devices.	Add security latches	48	EA	$ 10	$ 480		$ -	$ 480		RS Means. Increased cost by 25%
	Construction difficulty, operations and work restrictions	1	LS					$ 240		50% of other costs
									$ 720	
Deficiency No. NS3	**Mitigation**									
Windows above entrance appear to lack proper restraint in frame if cracked or damaged.	Demo existing window	1	LS					$ 3,000		Estimate
	New window frame and glazing	154	SF	$ 72	$ 44,044	$ 8	$ 1,271	$ 45,315		RS Means
	Construction difficulty, operations and work restrictions	1	LS					$ 24,157		50% of other costs
									$ 72,472	
Deficiency No. NS4	**Mitigation**									
Storage racks lack restraint to structure. The laboratory refridgerator lacks restraint if wheels are in locked position.	Add epoxy anchors	6	EA	$ 71	$ 426	$ 38	$ 228	$ 654		RS Means
	Construction difficulty, operations and work restrictions	1	LS					$ 327		50% of other costs
									$ 981	

Deficiency No. NS5	Mitigation									
The laboratory hoods could not be determined if adequate lateral bracing is attached back to structure. The air handling unit lacks anchorage to support structure.	Add diagonal bracing	4	EA			$ 500	$ 2,000	$ 2,000		$500/brace. Estimate
	Add welded connection	4	FT	$ 7	$ 27	$ 102	$ 409	$ 436		RS Means
	Construction difficulty, operations and work restrictions	1	LS					$ 2,436		100% of other costs
									$ 4,872	

	Sub-total		$ 308,156
ENR Index Factor	1.23		
	Sub-total		$ 378,085.39
Project Level Allowance	30%		$ 113,425.62
	Sub-total		$ 491,511
GR / GC	15%		$ 73,726.65
	Sub-total		$ 565,238
Contractor's Profit	10%		$ 56,524
	Sub-total		$ 621,761
Bond	2%		$ 12,435
	Sub-total		$ 634,197
Insurance	2%		$ 12,684
	GRAND TOTAL		**$ 646,881**

CONSTRUCTION COST ONLY

NOTES:

Project Name: Process Gallery
Project Number: 11962A.00
Project Construction Duration:

Date Prepared: 6/3/2022
Prepared By: B. Stuetzel
Date Accepted:
Accepted By:

				MATERIALS		INSTALLATION		TOTAL	TOTAL	
		QTY.	Unit	Unit Cost	Amount	per UM	Amount	Direct Cost	Direct Cost	Reference
Deficiency No. S1	**Mitigation**									
Roof beam aligned with interior shear wall lacks ability to transfer seismic loads into the lateral resisting system.	Add new steel plate	10	FT	$ 51	$ 2,050	$ 8	$ 75	$ 2,125		RS Means
	Epoxy anchors at 6" OC	21	FT	$ 71	$ 1,491	$ 38	$ 798	$ 2,289		RS Means
	Field welding of steel rod to existing joist	1	EA			$ 2,400	$ 2,400	$ 2,400		
	Construction difficulty, operations and work restrictions	1	LS					$ 6,814		100% of other costs
									$ 13,628	
Deficiency No. NS1	**Mitigation**									
Air handling unit lacks anchorage along channel support. The aeration blower pumps in basement lack proper anchorage back to structure (missing nuts).	Add epoxy anchors	2	EA	$ 71	$ 142	$ 38	$ 76	$ 218		RS Means
	Provide nuts for threaded rod anchorage	12	EA	$ 2	$ 24			$ 24		RS Means
	Construction difficulty, operations and work restrictions	1	LS					$ 121		50% of other costs
									$ 363	
Deficiency No. NS2	**Mitigation**									
Multiple pipes lack restraint to Unistrut supports. The compression struts for RAS piping lack diagonal bracing back to structure.	Install pipe straps to unistrut	20	EA	$ 25	$ 500	$ 9	$ 175	$ 675		
	Add diagonal bracing	6	EA			$ 500	$ 3,000	$ 3,000		$500/brace. Estimate
	Construction difficulty, operations and work restrictions	1	LS					$ 3,675		100% of other costs
									$ 7,350	

	Sub-total		$ 21,341
ENR Index Factor	1.23		
	Sub-total		$ 26,183.91
Project Level Allowance	30%		$ 7,855.17
	Sub-total		$ 34,039
GR / GC	15%		$ 5,105.86
	Sub-total		$ 39,145
Contractor's Profit	10%		$ 3,914
	Sub-total		$ 43,059
Bond	2%		$ 861
	Sub-total		$ 43,921
Insurance	2%		$ 878
	GRAND TOTAL		**$ 44,799**

CONSTRUCTION COST ONLY

NOTES:

Project Name: Workshop
Project Number: 11962A.00
Project Construction Duration:

Date Prepared: 6/3/2022
Prepared By: B. Stuetzel
Date Accepted:
Accepted By:

				MATERIALS		INSTALLATION		TOTAL	TOTAL	
		QTY.	Unit	Unit Cost	Amount	per UM	Amount	Direct Cost	Direct Cost	Reference
Deficiency No. S1	**Mitigation**									
Hold-down anchors within east elevation wall lack strength to resist overturning forces due to seismic.	Demo and restoration of finishes	330	SF			$ 75	$ 24,750	$ 24,750		$75/SF. Estimate
	Add additional epoxy anchors	4	EA	$ 71	$ 284	$ 38	$ 152	$ 436		RS Means
	Construction difficulty, operations and work restrictions	1	LS					$ 25,186		100% of other costs
									$ 50,372	
Deficiency No. S2	**Mitigation**									
Shear wall segments along the east elevation do not have sufficient shear capacity to resist the in-plane seismic loads.	Install new plywood overlay for shear walls	165	SF	$ 1	$ 157	$ 3	$ 508	$ 665		RS Means
	Install steel plate w/ connections to top plate splice locations	10	FT	$ 24	$ 240	$ 6	$ 60	$ 300		RS Means. 13 lbs/ft steel
	Construction difficulty, operations and work restrictions	1	LS					$ 965		100% of other costs
									$ 1,930	
Deficiency No. S3	**Mitigation**									
Shear wall segments along the east elevation do not have adequate sill bolt anchorage for resisting the in-plane seismic loads.	Add epoxy anchors	4	EA	$ 71	$ 284	$ 38	$ 152	$ 436		RS Means
	Construction difficulty, operations and work restrictions	1	LS					$ 436		100% of other costs
									$ 872	
Deficiency No. NS1	**Mitigation**									
Storage racks within building lack restraint back to structure. Shelving unit along south elevation lacks anchorage across entire length.	Add epoxy anchors	8	EA	$ 71	$ 568	$ 38	$ 304	$ 872		RS Means
	Construction difficulty, operations and work restrictions	1	LS					$ 436		50% of other costs
									$ 1,308	

NOTES:

	Sub-total		$ 54,482
ENR Index Factor	1.23		
	Sub-total		$ 66,845.47
Project Level Allowance	30%		$ 20,053.64
	Sub-total		$ 86,899
GR / GC	15%		$ 13,034.87
	Sub-total		$ 99,934
Contractor's Profit	10%		$ 9,993
	Sub-total		$ 109,927
Bond	2%		$ 2,199
	Sub-total		$ 112,126
Insurance	2%		$ 2,243
	GRAND TOTAL		**$ 114,368**

CONSTRUCTION COST ONLY

Project Name: Wastewater Treatment Plant (Overall Site Structures)
Project Number: 11962A.00
Project Construction Duration:

Date Prepared: 6/3/2022
Prepared By: B. Stuetzel
Date Accepted:
Accepted By:

				MATERIALS		INSTALLATION		TOTAL	TOTAL	
		QTY.	Unit	Unit Cost	Amount	per UM	Amount	Direct Cost	Direct Cost	Reference
Deficiency No. NS1	**Mitigation**									
Storage racks within the Headworks building	Add epoxy anchors	4	EA	$ 71	$ 284	$ 38	$ 152	$ 436		RS Means
lack anchorage back to structure	Construction difficulty, operations and work restrictions	1	LS					$ 218		50% of other costs
									$ 654	
Deficiency No. NS2	**Mitigation**									
Recirculation pump at Disk Filters lacks	Add weighted sand bags to prevent overturning	4	EA	$ 10	$ 40			$ 40		RS Means
restraint against overturning	Construction difficulty, operations and work restrictions	1	LS					$ 20		50% of other costs
									$ 60	
Deficiency No. NS3	**Mitigation**									
ACCU units near Aeration Basins lack	Add epoxy anchors	8	EA	$ 71	$ 568	$ 38	$ 304	$ 872		RS Means
anchorage to structural pads	Construction difficulty, operations and work restrictions	1	LS					$ 436		50% of other costs
									$ 1,308	

	Sub-total		$ 2,022
ENR Index Factor	1.23		
	Sub-total		$ 2,480.85
Project Level Allowance	30%		$ 744.26
	Sub-total		$ 3,225
GR / GC	15%		$ 483.77
	Sub-total		$ 3,709
Contractor's Profit	10%		$ 371
	Sub-total		$ 4,080
Bond	2%		$ 82
	Sub-total		$ 4,161
Insurance	2%		$ 83
	GRAND TOTAL		**$ 4,245**

CONSTRUCTION COST ONLY

NOTES: