CP-64174
Revision 0

ADDITIONAL QUANTITATIVE ANALYSIS FOR 241-Z-361, PUREX DEEP BED FILTER HOUSING, AND THE REDOX SAND FILTER

Prepared for the U.S. Department of Energy
Assistant Secretary for Environmental Management

Contractor for the U.S. Department of Energy
under Contract DE-AC06-08RL14788

CH2MHILL
Plateau Remediation Company

P.O. Box 1600
Richland, Washington 99352

Approved for Public Release;
Further Dissemination Unlimited

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Program/Project: CPRM

T. E. Bratvold
CH2M HILL Plateau Remediation Company

Date Published
October 2019

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APPROVED
By Julia Raymer at 2:39 pm, Oct 21, 2019

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LPI, Inc. Consulting Engineers

Advanced Analysis & Fitness for Service
Failure & Materials Evaluation
Nondestructive Engineering

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ADDITIONAL ANALYSES: STRUCTURAL ASSESSMENT OF AGING LEGACY BURIED CRIBS AND TANKS AT THE HANFORD CENTRAL PLATEAU

Report No.: LA181779-R-002
Revision No: 0

September 2019

Prepared For
TERRAGRAPHICS ENVIRONMENTAL ENGINEERING, INC.
Pasco, WA

Prepared By
LPI, Inc.

Lucius Pitkin

EXECUTIVE SUMMARY

LPI, Inc. (LPI) performed structural evaluations of below-grade facilities located within the Central Plateau of the Hanford site that were determined to have the highest combined risk of failure [1 and 28]. The structural evaluations in [28] were performed to determine if margin from collapse still exists in the structural members. LPI was requested to perform additional analyses on select buried structures, including a progressive failure analysis of Tank 241-Z-361, and surcharge standoff calculations for the PUREX and REDOX filters (see Table ES-1). Results of these additional analyses are documented herein. The structural stability in the present analyses was assessed using requirements and guidance provided in the American Concrete Institute (ACI) [4], the American Society of Civil Engineers (ASCE) 7 [24], and Braja M. Das' 6th edition book "Principles of Foundation Engineering" [27].

ANSYS ® software was used to perform the progressive failure analysis for Tank 241-Z-361. This software is a general-purpose finite element based-computer code [7] that is widely used in the nuclear industry for thermal and structural analyses of structural and mechanical components and includes wide range of capabilities for simulation and analysis of various material models and loading conditions. This software was validated and verified for the specific computers used in the analysis using a set of verification problems provided by ANSYS. In addition, Mathcad was used to perform the surcharge standoff calculations for the PUREX and REDOX filters and to calculate the reinforced concrete structural capacities.

Information collection and review efforts were conducted to identify the different types of material properties and applied loads to be considered in the progressive failure analysis and surcharge standoff calculations. Most material properties were obtained from publicly available reports, many of which involved previous assessments of Hanford degraded structures including, but not limited to, the PUREX Tunnel 1 Engineering Evaluation [14 and 21], Hanford Double-Shell Tanks [17], and Hanford Single-Shell Tanks [16, and 22].

Consistent with the material properties and applied loads employed in the structural evaluations documented in [28], applied loads in the present progressive failure analysis of Tank 241-Z-361 consist of dead weight, waste pressure, lateral soil pressure, and snow. Seismic load combination was shown not to be limiting in [28]. Live loads were not considered in the progressive failure analysis (also consistent with methods in [28]) given that barricades, such as light-duty post and chain fence, and "Caution Underground Radioactive Material" signage are currently employed at the ground surface of Tank 241-

Z-361 [18]. The progressive failure analysis for Tank 241-Z-361 presented herein addresses the overall structural stability, and should not be used to justify worker safety.

Similar loads were used in the standoff analysis of the PUREX and REDOX filters with the addition of the live load representing surcharge load due to equipment total weight. Furthermore, the present evaluations for Tank 241-Z-361, and the PUREX and REDOX filters also simulated degradation (see Table ES-2) in the structural members by employing reduced concrete thickness and/or reduced rebar surface area based on the documented evidence of subsidence, corrosion, or water intrusion [18].

Standoff calculations show that the PUREX filter walls are within ACI 318 Code [4] design limits for 60,000 lb surcharge (at maximum intensity of 250 lb/ft^2) at 10 ft (lower bound standoff distance) from the east, south and west walls, and an upper bound standoff distance of 24 ft for all plug locations, as shown in Figure ES1-1. Likewise, standoff calculations show that the REDOX filter walls are within the ACI 318 Code [4] design limits for all equipment weights at 24 ft as shown in Figure ES1-2.

The progressive failure analysis (nonlinear analysis) of Tank 241-Z-361 presented is based on current code requirements for load factors, load combinations, and strength reductions for concrete and steel materials. Results of the progressive failure analysis indicate that concrete cracking will occur and progress along the bottom of the long wall. However, multiple redundancies in the tank shape will provide adequate load paths and strength to resist the applied loads without excessive damage.

Table ES-1: Scope of Aging Legacy Buried Structures [28]

Group	Structure	Description	Waste Level[1]	Acceptable (Y/N)	Discussion
T2	241-Z-361	Rectangular concrete tank constructed in 1948; 28 x 15 x 19 ft tall; 2 ft overburden	8 ft	N	Moment and shear limits are exceeded at bottom of long walls. This will likely cause limited local failure. Failure will not progress due to the redundancy available in the box structure.
O1	PUREX deep bed filters	Rectangular concrete vault constructed in 1954; 82 x 52 x 13 ft; 4 ft overburden and 4 in. shotcrete	N/A	Y	-
O2	REDOX sand filter	Rectangular concrete vault constructed in 1950; 85 x 85 x 20 ft; No overburden	N/A	Y	-

[1] Waste level is based on the documents retrieved and summarized in the Risk Assessment Report (Rev. 2) [18].

Table ES-2: Simulated Damage/Degradation [28]

Group	Structure	Simulated Degradation
T2	241-Z-361	Roof and wall thickness were reduced by 1 in., rebar area reduced 10% because steel liner corrosion has been observed [18]
O1	PUREX deep bed filters	Rebar area was reduced by 10% and roof slab and external wall thickness was reduced by 1 in. because water intrusion has been observed [18]
O2	REDOX sand filter	Similar to the water intrusion concerns for the PUREX deep bed filters, concrete cover rebar area was reduced by 10% and roof slab and external wall thickness was reduced by 1 in.

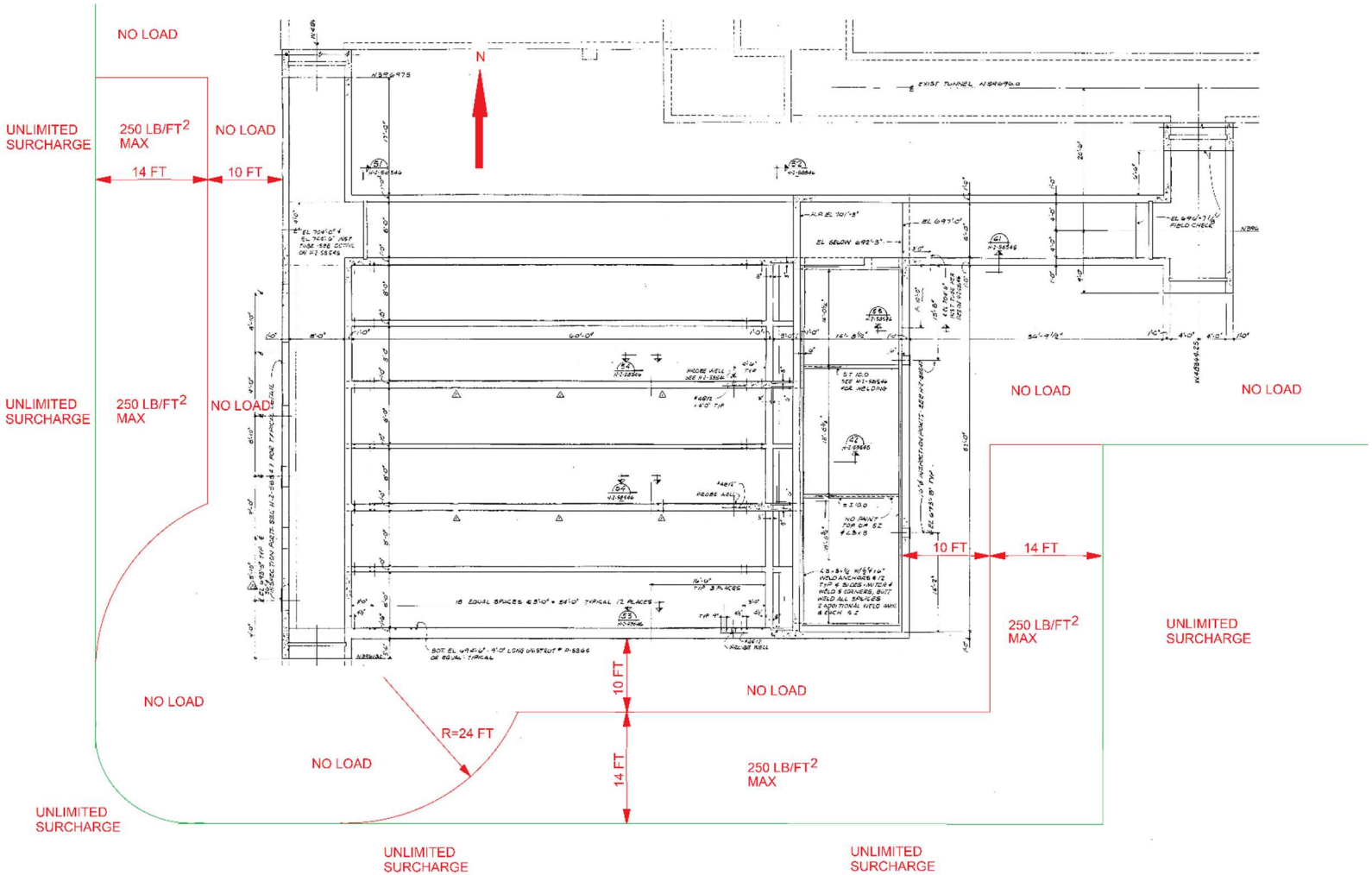

Figure ES1-1: Standoff Distance Map – PUREX Filters (Group O1)

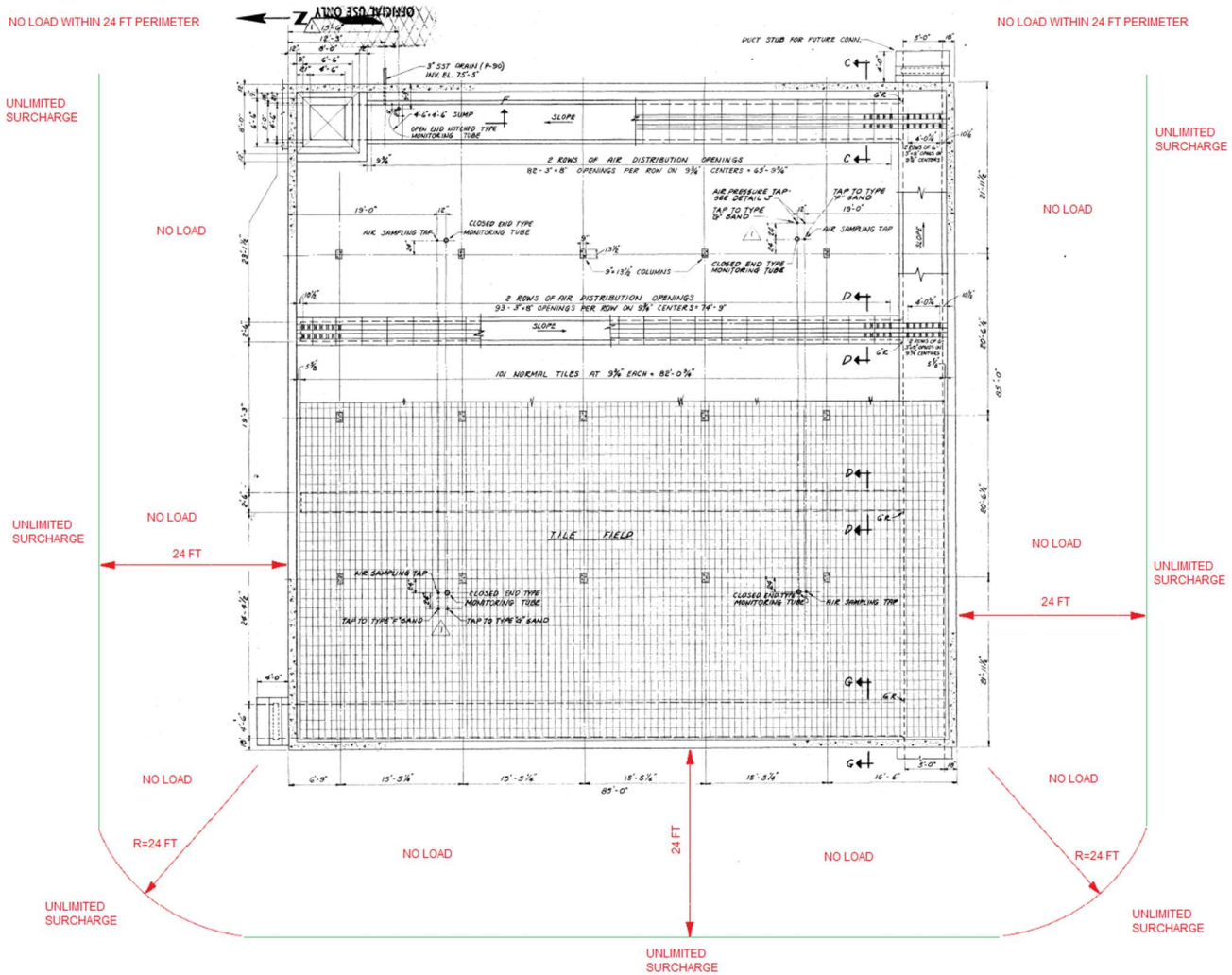

Figure ES1-2: Standoff Distance Map – REDOX Filter (Group O2)

DOCUMENT RECORD

Document Type:	☐ **Calculation** ☒ **Report** ☐ **Procedure**	
Document No:	LA181779-R-002	
Document Title:	Additional Analyses: Structural Assessment of Aging Legacy Buried Cribs and Tanks at the Hanford Central Plateau	
Client:	TerraGraphics Environmental Engineering, Inc.	
Client Facility:	428 W. Shoshone St, Pasco, WA 99301	
Client PO No:	Subcontract No. S18-1055, Task Order 1	
Quality Assurance:	**Nuclear Safety Related?** ☒ **No** ☐ **Yes**	
Computer Software Used:	☐ **No**[1] ☒ **Yes**[2]	1. Check NO when EXCEL, Mathcad and/or similar programs are used since algorithms are explicitly displayed. 2. Include Software Record for each computer program utilized
Instrument Used	☒ **No** ☐ **Yes**[3]	3. Include Document Instrument Record.

Revision	Approval Date	Preparer[5]	Checker[5]	Design Verification[5]	Approver[4,5]
0	09/04/19	*B. Elaidi*	*C. Guzmán-Leong*	*n/a*	*R. S. Vecchio*

[4] The Approver of this document attests that all project examinations, inspections, tests and analysis (as applicable) have been conducted using approved LPI Procedures and are in conformance to the contract/purchase order.

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Page	6	of	59	Total Pages	(include any Title Sheet and Attachments in page count. Document Back Cover, if utilized, not included in page count)

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Form: LPI-3.1-Rev-10-Fig-5-2

Record of Revision

Revision No.	Date	Description of Change	Reason
0	09/04/19	N/A	Original Issue

Form: LPI-3.1-Rev-10-Fig-5-7

Document Software Record[1]

(include separate sheet for each software package used)

1	**Computer Software Used** (Code/Version)	ANSYS 18.0 [7]	
2	**Software Supplier**	ANSYS, Inc.	
3	**Software Update Review**	☒ Error notices; describe: Reviewed error notices on ANSYS website ☒ Other; describe: Refer to [8]	
4	**Nuclear Safety Related Software**	☐ NO ☒ YES[2]	[2] If YES, complete the following: Computer type: Dell Latitude E5550 Laptop Computer S/N: 8L0C362 Computer O/S: Windows 10, 64 V & V (include as ref): [8]
5	**Bases for Application**	Identify the bases that support use of this software for the application herein; may be separately discussed elsewhere in this document (indicate section) and/or may be addressed in the V&V (identify reference number):	See Section 3
6	**Input Listing/Summary[3]**	☐ Input listing and/or summary attached: ☒ Not attached; identify File/Disc ID: Z-361-LC1 – 6-13-2019 14;01;53 AM EST	
7	**Output Data/Identifier(s)[3]**	☒ Output results attached: Selected relevant output is included herein ☐ Not attached; identify File/Disc ID:	
	[3]e.g., run date/time; use for reference, as appropriate, within body of calculation		
8	**Comments**	NA	
9	**Keywords[4]**	SOLID185, LINK180	
	[4]For use in describing software features used in this calculation; use common terms based on software user manual and/or help files.		
10	**Project Manager Name:**	Consuelo Guzmán-Leong	

[1] If computer software is used on project, complete form with required information.
Update the LPI Computer Software Use List per LPI Procedure 13.1 requirements.

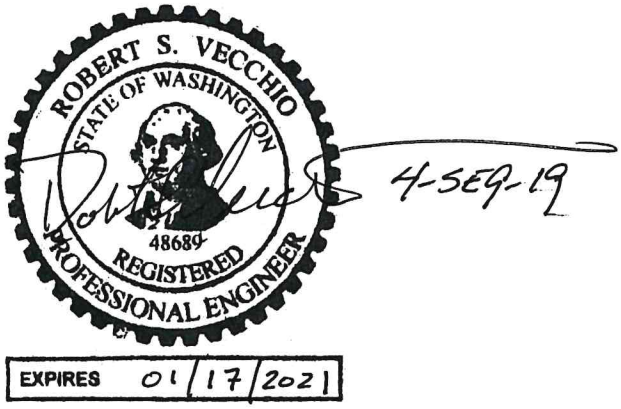
Form: LPI-3.1-Rev-10-Fig-5-8

CERTIFICATION PAGE

I, the undersigned, being a registered Professional Engineer competent in the modeling and structural evaluation of various concrete, steel, and wood structures and acceptance criteria in the ACI 318 and ASCE 7 Codes, and using the drawings cited herein, do hereby certify that to the best of my knowledge and belief the progressive failure analysis and standoff calculations performed and documented in LPI Report No. LA181779-R-002, Rev. 0 are complete and accurate and comply with methods that are universally recognized for analysis of similar structures and determination of strength levels.

Certified by:	Robert S. Vecchio, Ph.D., PE

Registration No.:	48689	State:	Washington



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1. INTRODUCTION

LPI, Inc. (LPI) was requested to provide engineering services for the assessment of below-grade facilities located within the Central Plateau of the Hanford site that were determined to have the highest combined risk of failure [1 and 28]. The LPI report "*Structural Assessment of Aging Legacy Buried Cribs and Tanks at the Hanford Central Plateau*" [28] documents the structural evaluations of selected legacy aging buried cribs and tanks in accordance with the Master Service Agreement with TerraGraphics Environmental Engineering, Inc. [1].

The "RL-40 Aging Structures Risk Assessment" report [18] was conducted to identify aging buried structures most prone to structural degradation or collapse; structural evaluations were then performed and documented in the LPI report [28]. Based on the results of report [28], additional evaluations were performed for select structures and include progressive failure analysis for Tank 241-Z-361, and standoff calculations for the PUREX and REDOX filters. The objective of the progressive failure analysis is to simulate the progression of concrete failure and to investigate the overall structural stability of Tank 241-Z-361 walls using appropriate nonlinear concrete material damage model. The purpose of the standoff calculations was to determine the minimum distance required between heavy equipment (greater than ten tons gross vehicle weight rating) and the perimeter of the PUREX and REDOX filters to ensure ACI 318 [4] design limits are met.

2. SCOPE

Table 2-1 lists the structures included in this study and provides a summary description for each structure; the level of waste stored (as applicable) is also indicated. As shown in Table 2-1, all structures are buried in or surrounded by soil.

Table 2-1: Scope of Aging Legacy Buried Structures

Group	Structure	Description	Waste Level[2]
T2	241-Z-361	Rectangular concrete tank constructed in 1948; 28 x 15 x 19 ft tall; 2 ft overburden	8 ft
O1	PUREX deep bed filters	Rectangular concrete vault constructed in 1954; 82 x 52 x 13 ft; 4 ft overburden and 4 in shotcrete	N/A
O2	REDOX sand filter	Rectangular concrete vault constructed in 1950; 85 x 85 x 20 ft; No overburden	N/A

[2] Waste level is based on the documents retrieved and summarized in the Risk Assessment Report (Rev. 2) [18].

3. PROGRESSIVE FAILURE ANALYSIS FOR TANK 241-Z-361

ANSYS ® software was used to perform the structural evaluation (progressive failure analysis) of Tank 241-Z-361, a below-grade structure located within the Central Plateau of the Hanford site. ANSYS is a general-purpose finite element based-computer code [7] that is widely used in the nuclear industry for thermal and structural analyses of structural and mechanical components, and includes a wide range of capabilities for simulation and analysis of various material models and loading conditions. This software was validated and verified as documented in [10].

The overall dimensions of the 241-Z-361 settling tank are 28 x 15 x 19 ft, with 2 ft of soil overburden. This settling tank contains 8 ft of waste. The walls and roof for this tank are 12 in. thick [18]. Material properties and loading were applied as described in Appendix A of LPI report "*Structural Assessment of Aging Legacy Buried Cribs and Tanks at the Hanford Central Plateau*" [28]. Based on available information of historical events, and the current condition of the concrete indicating some level of deterioration and steel liner corrosion [18], all concrete walls and the roof for the Tank 241-Z-361 structure were reduced in thickness by 1 in., and rebar area was reduced by 10% to simulate observed degradation. The degree of degradation was assumed in the absence of any information that may allow the determination of the actual degradation present in Tank 241-Z-361 [28].

Tank 241-Z-361 was analyzed in LPI report "*Structural Assessment of Aging Legacy Buried Cribs and Tanks at the Hanford Central Plateau*" [28] as a linear elastic structure for current code bounding design basis load combinations (provided in Appendix A [28]). The analysis results in [28, Section 4.7] indicate bending failure at the bottom of the long concrete walls. This failure mode is associated with tensile cracking on the outside face of the wall, increase in tension in the outer vertical rebar, and increase in concrete compression on the inside face.

Based on the computed stress distribution of the linear analysis [28, Figure 4.7-2], high stresses were localized at the bottom at mid-length of the long walls. As the concrete tensile stresses exceeds the rupture limit and cracks develop, concrete tensile stresses are redistributed in adjacent sections causing progression of cracking and potentially yielding of rebar and crushing of concrete on the compression side of the wall. Depending on geometry and loading, progression of cracking will lead to one of two scenarios. In the first scenario, the progression of cracking slows and stops with the tank maintaining its structural integrity. In the second scenario, the bending failure mechanism (described above) propagates to adjacent wall sections until all possible load paths are structurally degraded and a state of catastrophic collapse of the long walls is reached. The purpose of the analysis described herein is to simulate the progression of concrete failure and to investigate the overall structural stability of the tank walls using appropriate nonlinear concrete material damage model.

The nonlinear analysis was performed with a reduced model of the tank shown in Figure

3-1. The model is 1/4 of the tank assuming double symmetry about the two vertical planes along the center of the tank. The height of the model is the average height of the tank. The change in height is small and the approximation in simulating the height is not critical. The foundation slab was not included in the model (for simplification) and the vertical walls were fully restrained along the bottom. This is conservative because the flexibility of the foundation slab is simulated with a rigid boundary resulting in increase of bending stresses.

Concrete members were meshed with ANSYS SOLID185 8-node elements, as shown in Figure 3-2. The rebar in the long wall were modeled with ANSYS LINK180 line elements (Figure 3-3). The modeled rebar includes the vertical and horizontal rebar along the inside and outside faces of the long wall. Since, the roof and short wall were modeled with linear concrete material, modeling of the reinforcement was not required. The concrete roof and wall thicknesses were reduced by 1 in. and the rebar cross section areas were reduced by 10% to simulate degradation due to environmental conditions.

Concrete material behavior was based on an ANSYS Microplane material model and accounts for tensile cracking and compression crushing. This model is based on research by Bażant and Gambarova [29] in which behavior is simulated by uniaxial stress-strain relation on multiple planes, with damage modeled through uniaxial damage laws. The following parameters were specified for the concrete material damage model:

1. Concrete compressive strength = 3.000 psi (Table A2 in Appendix A of [28])
2. Concrete tensile strength = 6.7 $\sqrt{f'c}$ = 367 psi (ACI 318 [4], Section 8.6.1) conservatively use 300 psi
3. Concrete Poisson's ratio = 0.15 (Table A2 in Appendix A of [28])
4. Concrete elastic modulus = 57,000 $\sqrt{f'c}$ = 3.1 x 10^6 psi (ACI 318 [4], Section 8.5.1)
5. Residual strength in tension cracking and compression crushing is 10% of the corresponding specified strength (i.e. post cracking tensile strength is 0.1 x 300 = 30 psi and post crushing compression strength is 0.1 x 3,000 = 300 psi). This small residual strength is intended to preserve numerical stability of the analysis without significantly affecting strength of the structure.

Rebar material was modeled using properties for linear elastic steel; elastic modulus = 29x10^6 psi (ACI 318 [4], Section 8.5.2) and Poisson's ratio = 0.3 (typical for steel material). Stresses in the rebar were verified to be below yield (40 ksi) and, therefore, were bounded by elastic behavior.

Loads applied in the present analysis are consistent with those employed in the linear analysis documented in Section 4.7 of [28] and include the following:

1. Self-weight of concrete based on 150 lb/ft^3 density
2. 2 ft overburden pressure on the roof (Figure 3-4)
3. Roof snow load of 15 lb/ft^2

4. Lateral earth pressure on the outside vertical faces of the tank (Figure 3-4)
5. Waste pressure on the inside faces of the tank based on 8 ft waste depth (Figure 3-4). Since the waste pressure opposes the lateral earth pressure, the waste pressure was conservatively reduced by replacing the specific gravity of the waste of 1.7 with a value of 1.0.

The present analysis of Tank 241-Z-361 was performed based on load combination 1 provided below because it is the most limiting of the two load combinations shown in Appendix A of LPI report "*Structural Assessment of Aging Legacy Buried Cribs and Tanks at the Hanford Central Plateau*" [28]. The total combined loads were applied incrementally using refined load step (2,000 load steps were used) so that time history of cracking can be calculated accurately.

$$U = 1.4\ DW + 1.4\ F + 1.7\ LE + 1.4\ S \qquad \text{Load Combination 1}$$

where:

U is the required strength (axial, bending moment or shear) of the concrete or steel structural component, DW is deadweight, F is waste load, LE is lateral earth load, and S is snow load.

The analysis showed that convergence was attained at every load step, including the final load step. This indicates that global failure of the tank should not occur. Contours of vertical concrete stresses are shown in Figure 3-5[3]. Time history of vertical concrete stresses and strains through thickness at the bottom mid-length of the long wall are shown in Figure 3-6 and Figure 3-7, respectively. Results revealed tensile cracking of concrete through the wall thickness, except at the two nodes on the compression side where the concrete remained in compression below its compression strength. Similar plots are shown in Figure 3-8 and Figure 3-9 for the outside face along the length of the tank, which show the extent of tensile cracking and tensile strain along the length of the tank. A stress-strain plot for the concrete bottom outside face at mid-length of the tank is shown in Figure 3-10, which shows post cracking concrete reduction in strength and stiffness. Figure 3-11[3] shows contour plots of the vertical stress and strain at the bottom of the long wall indicating the extent of cracking and post cracking strains.

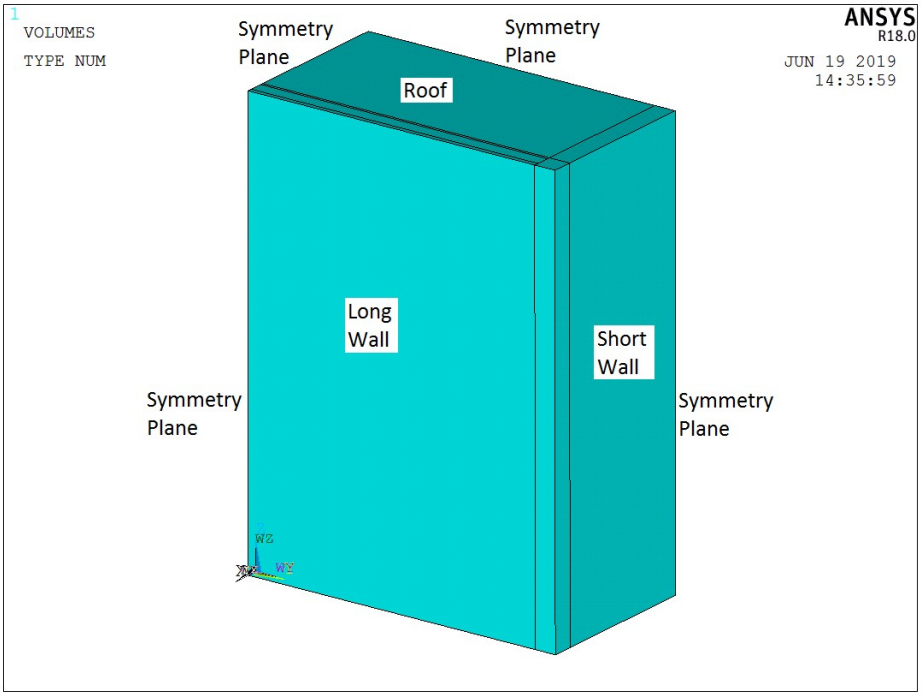
Figure 3-12 shows the time history tensile stresses in the outside face vertical rebar along the bottom of the tank where tension in the rebar is maximum. It is shown that the rebar stresses increase due to concrete cracking but remain below the yield stress of 40 ksi. Total out-of-plane deflection of the long wall is 0.22 in. as indicated by the DMX value shown in the legend (located in the upper left corner) of Figure 3-5[3]. This deflection is small compared to the thickness of the wall and is indicative of structural stability with adequate margin.

In summary, the nonlinear analysis indicates that concrete cracking will occur and progress

[3] Parameters DMX, SMN, and SMX in the legend of this figure (located in the upper left corner) represent the maximum deflection (in.), minimum stress (psi), and maximum stress (psi), respectively.

along the bottom of the long wall. However, multiple redundancies in the tank box shape will provide adequate load paths and strength to resist the applied loads without excessive damage.

Note, units in subsequent figures are psi for pressure, in. for deflection, and in./in. for strain.



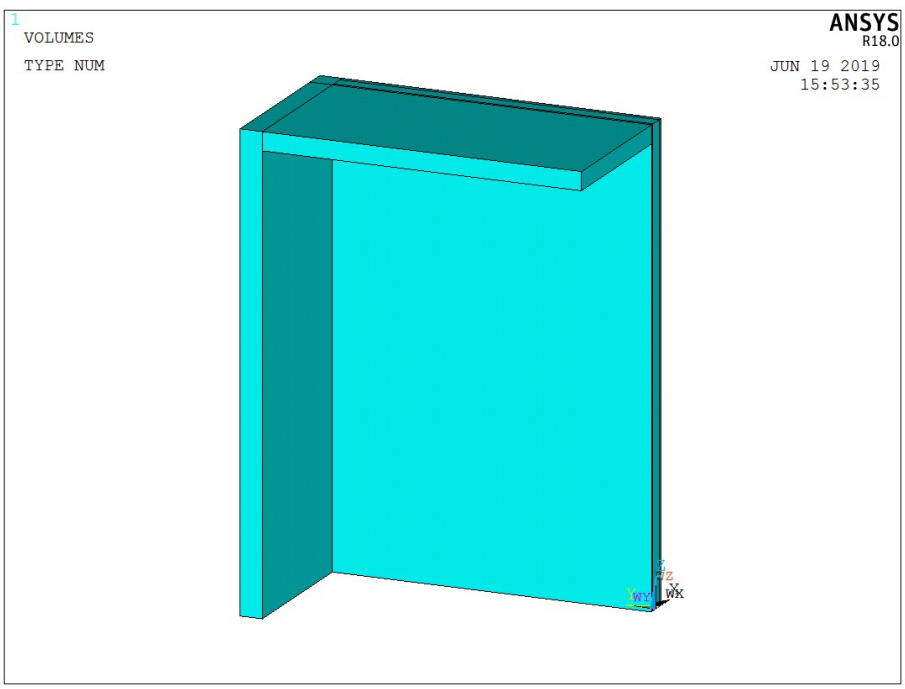


Figure 3-1: General Views of the ANSYS Solid Model of 241-Z-361

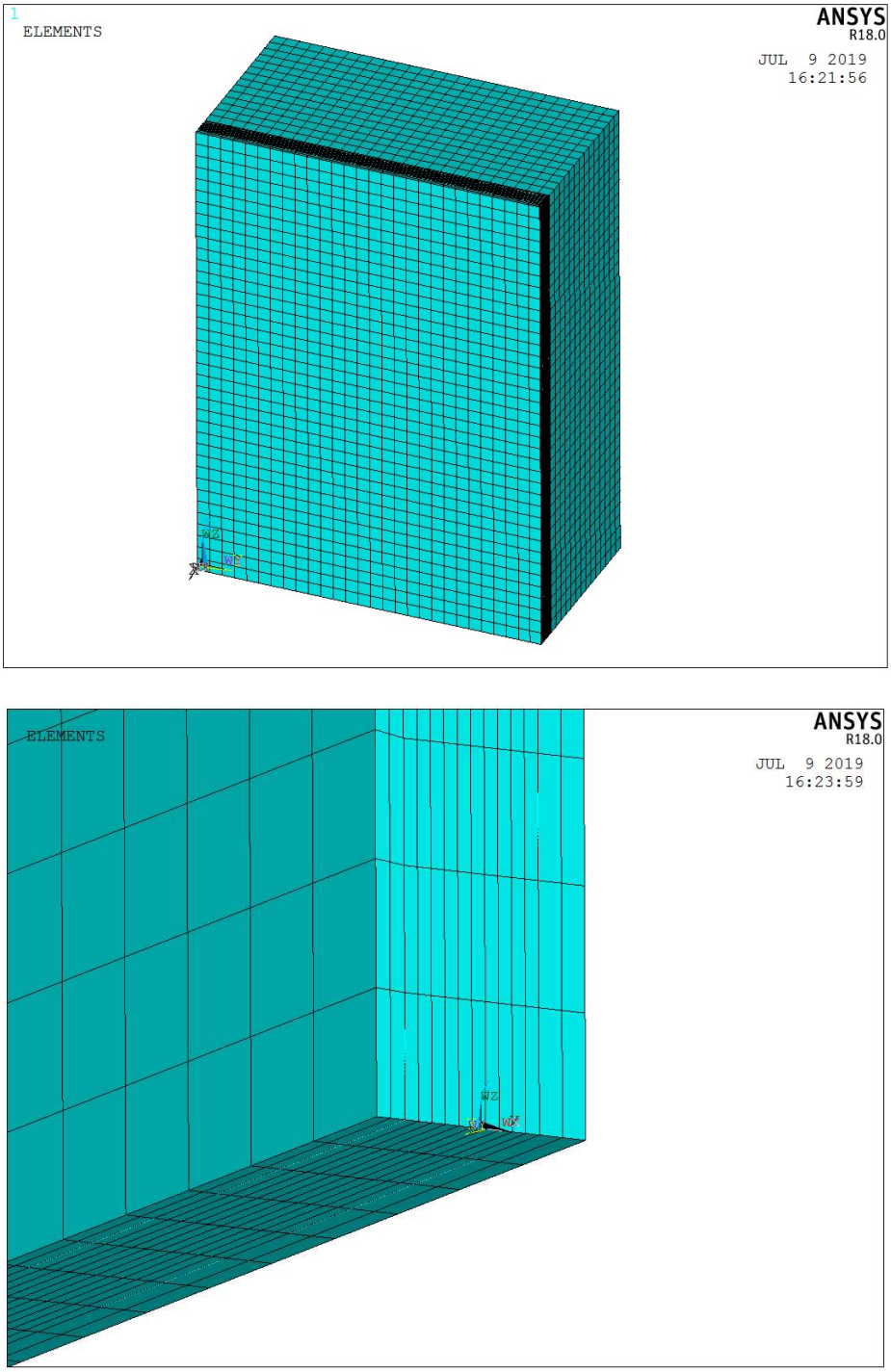
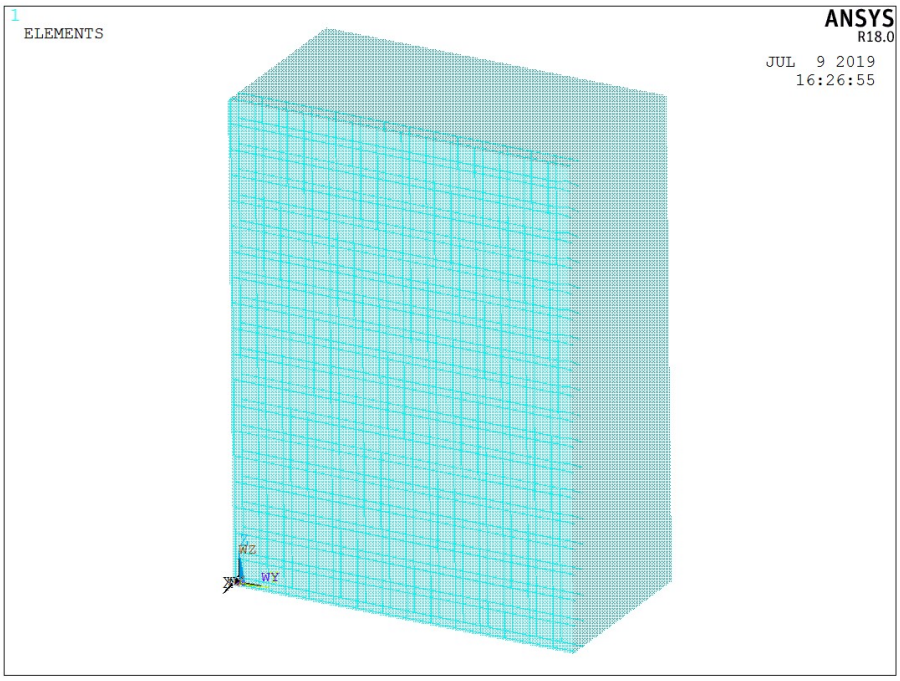


Figure 3-2: General Views of the ANSYS Finite Element Mesh of Concrete Elements Showing Mesh Refinement in the Long Wall



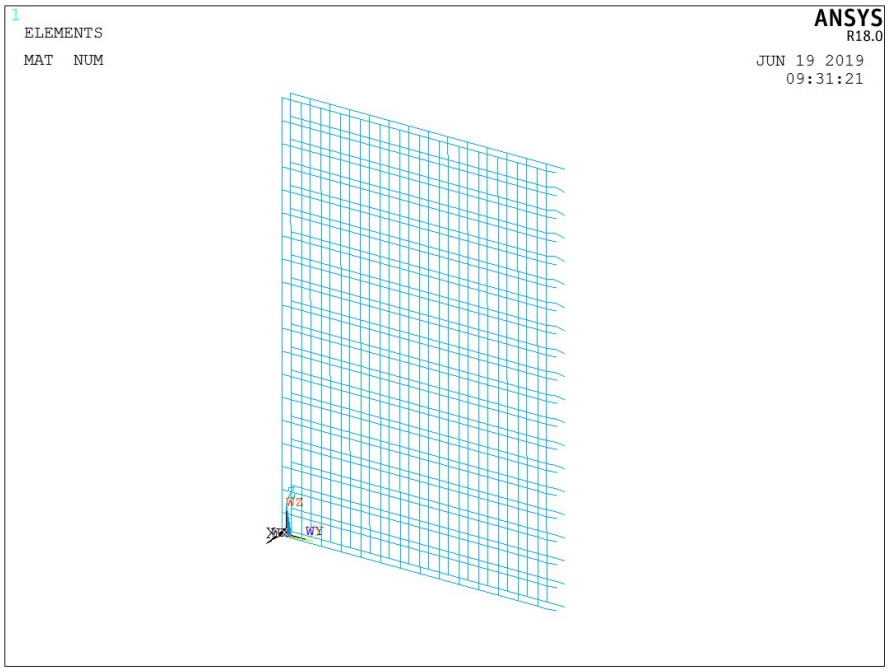
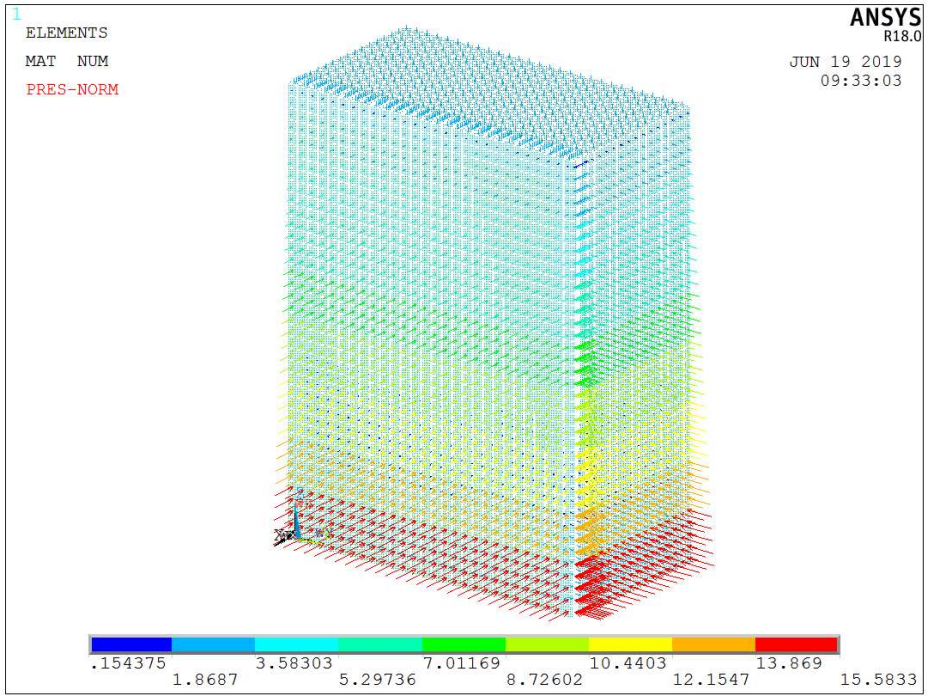


Figure 3-3: General Views of the ANSYS Finite Element Mesh of Rebar in the Long Wall



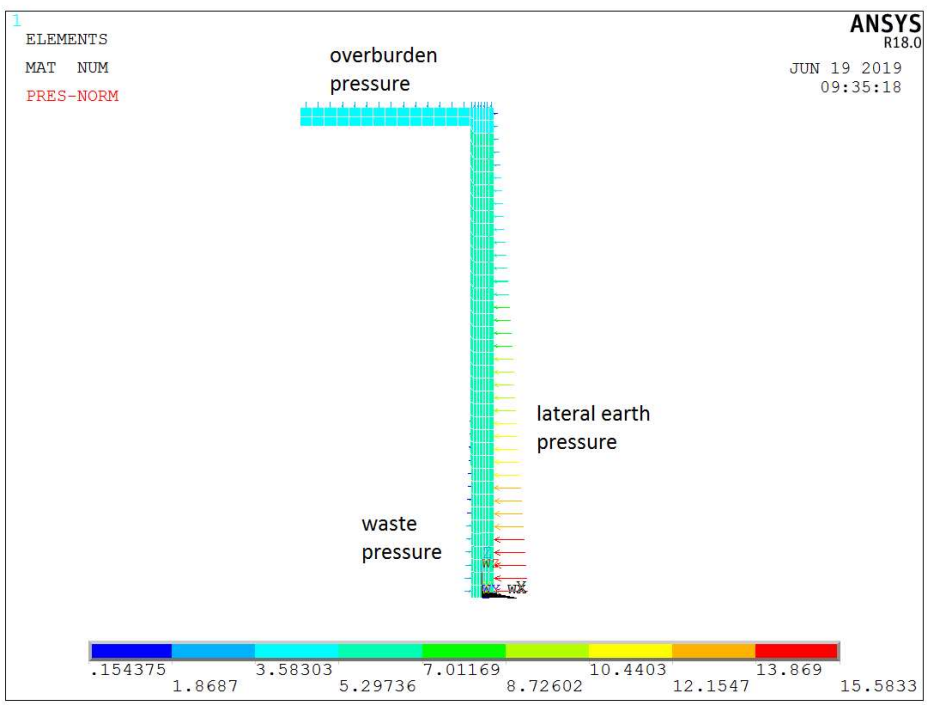


Figure 3-4: Pressure Loads (psi) Applied in ANSYS Analysis

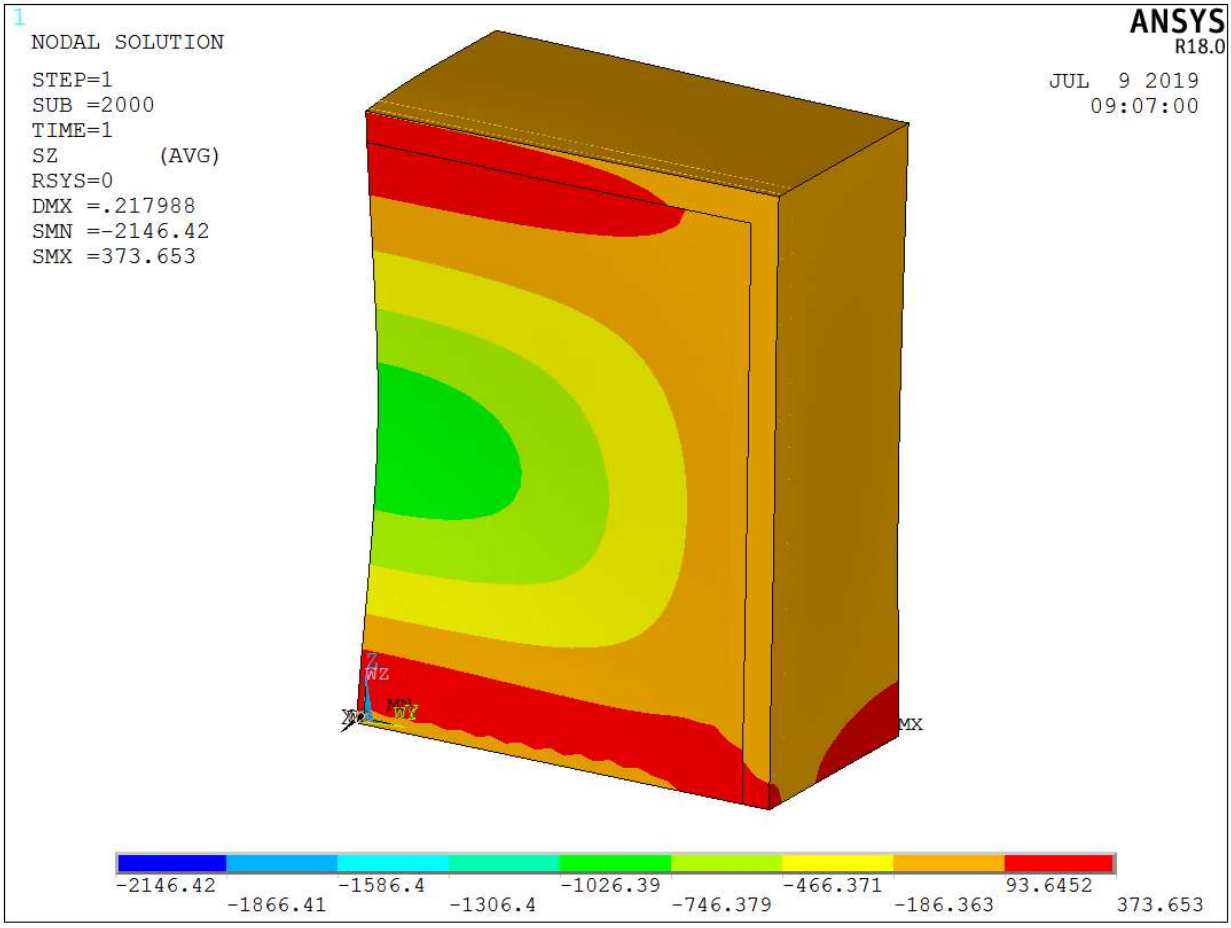
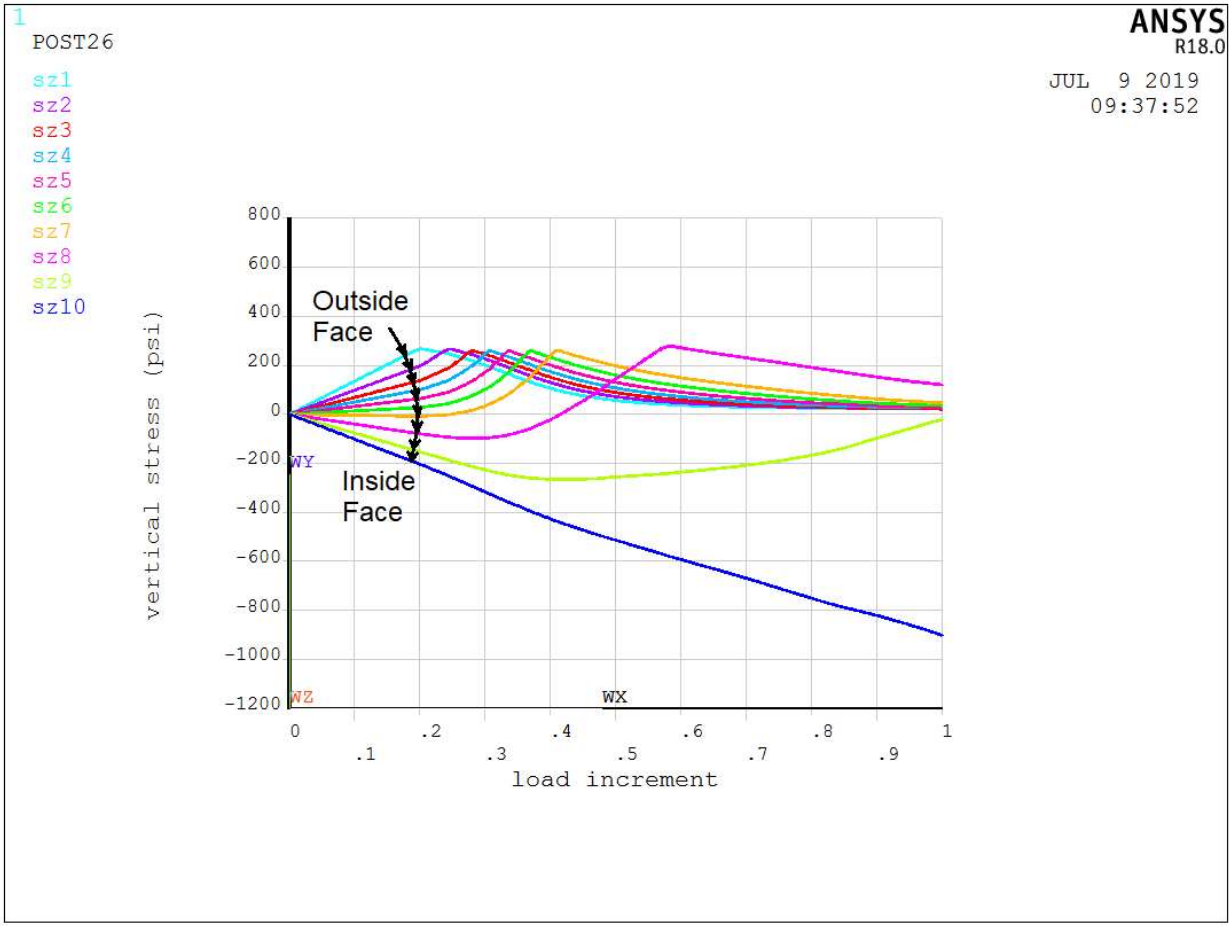


Figure 3-5: Contours of Vertical Concrete Stresses (psi)



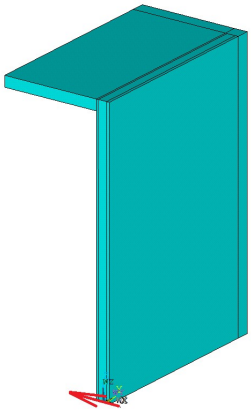
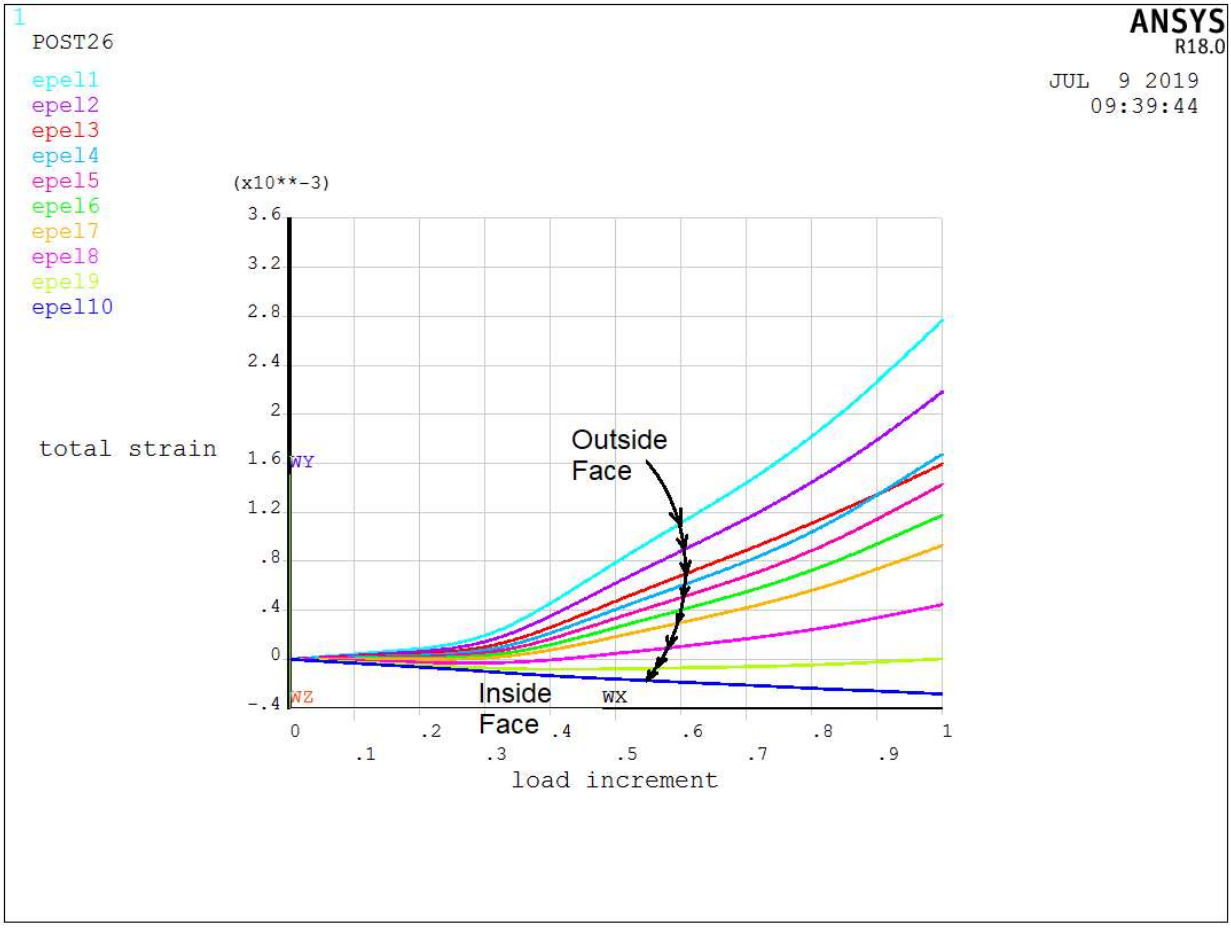


Figure 3-6: Time History of Concrete Vertical Stresses (psi) Through Wall at Section of Maximum Tension



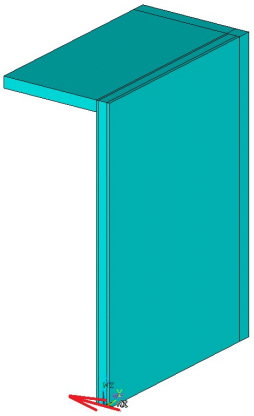
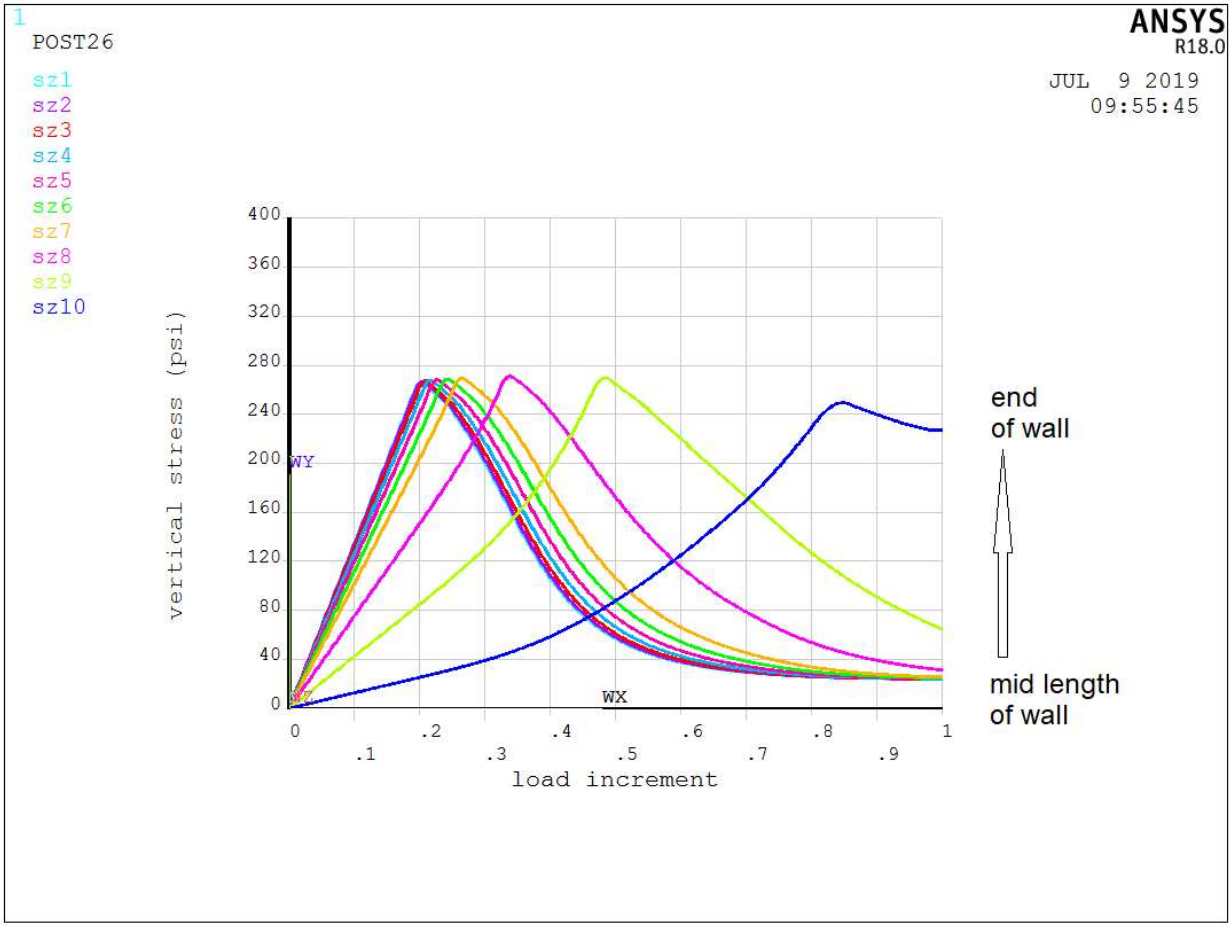


Figure 3-7: Time History of Concrete Vertical Strains (in./in.) Through Wall at Section of Maximum Tension



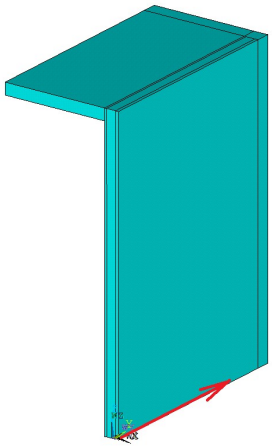
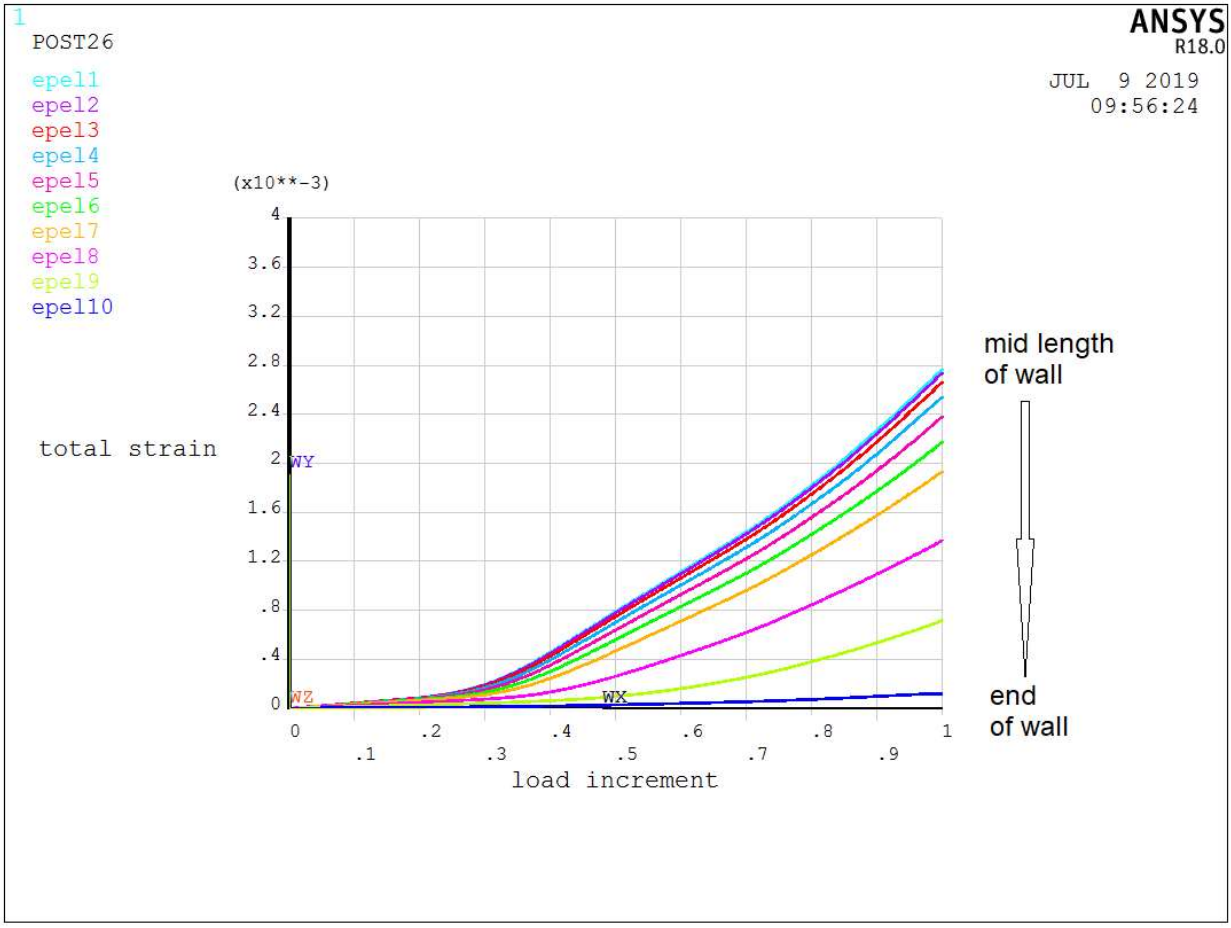


Figure 3-8: Time History of Concrete Vertical Stresses (psi) on the Outside Face Along Wall Length



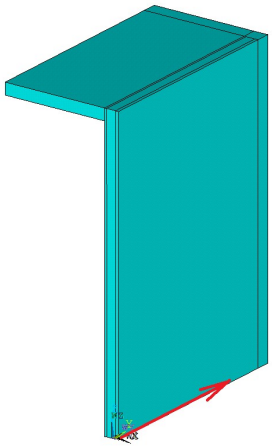


Figure 3-9: Time History of Concrete Vertical Strains (in./in.) on the Outside Face Along Wall Length

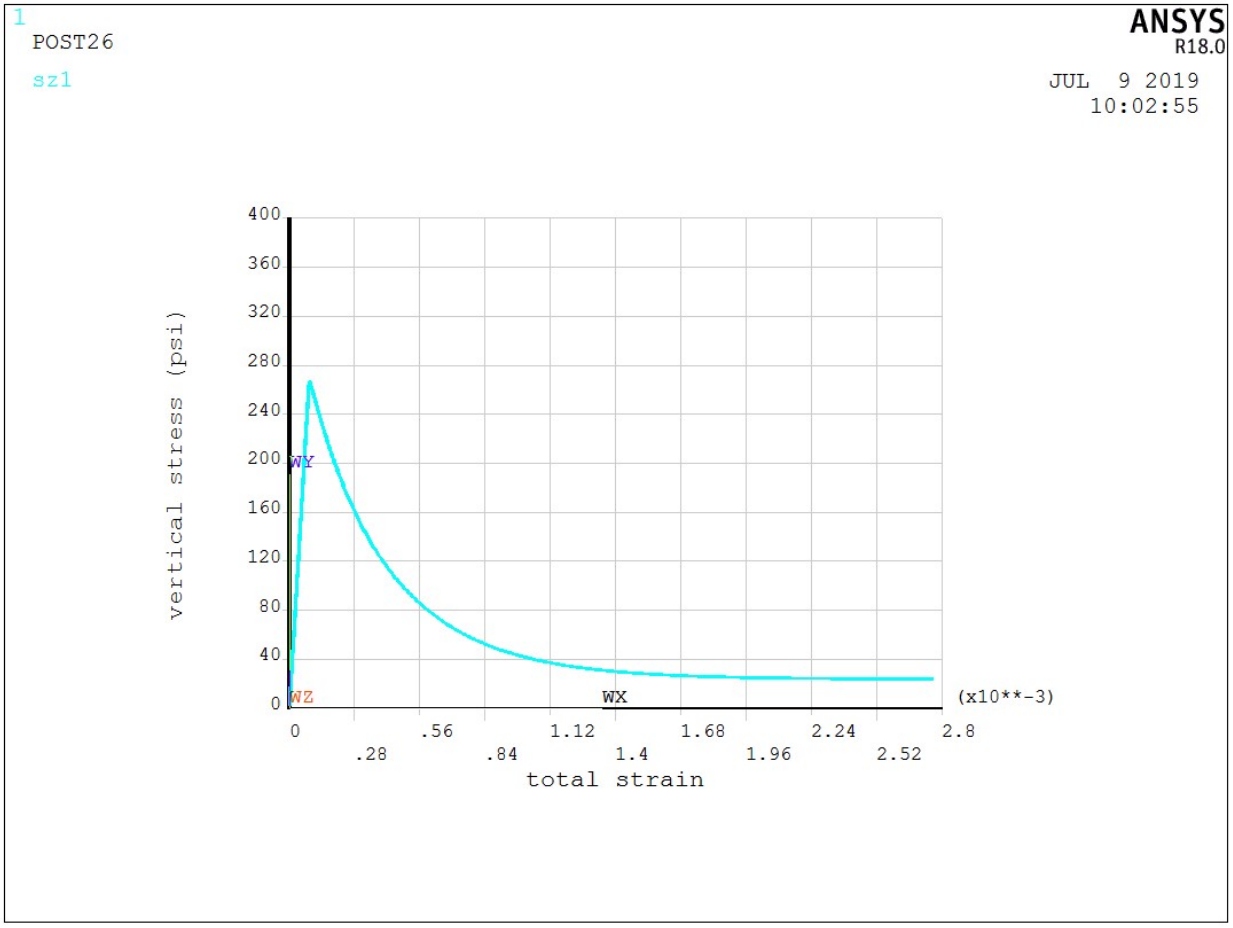
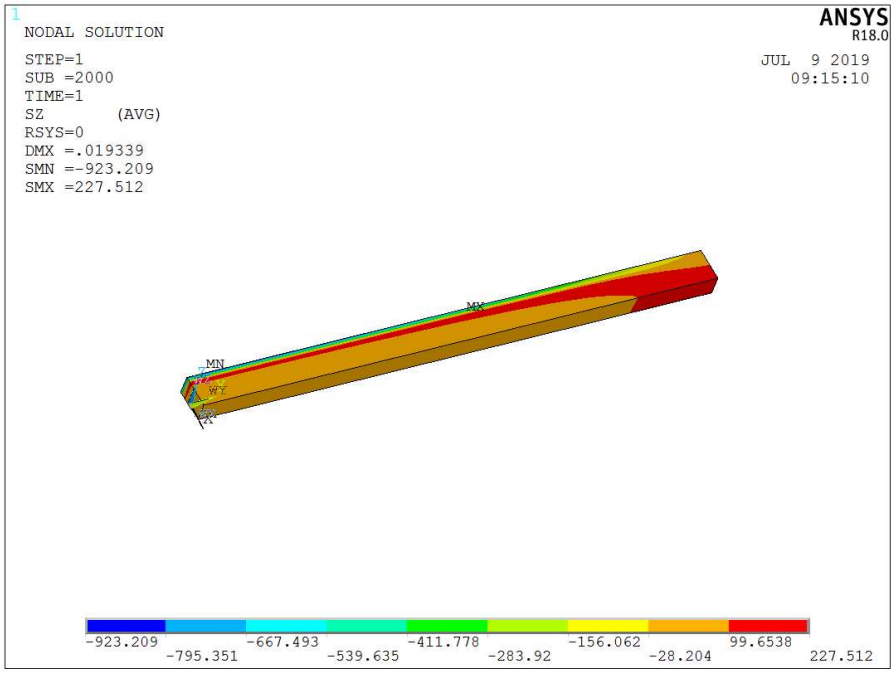


Figure 3-10: Concrete Stress-Strain (psi – in./in.) Graph at Section of Maximum Tensile Cracking (bottom outside face at mid-length)



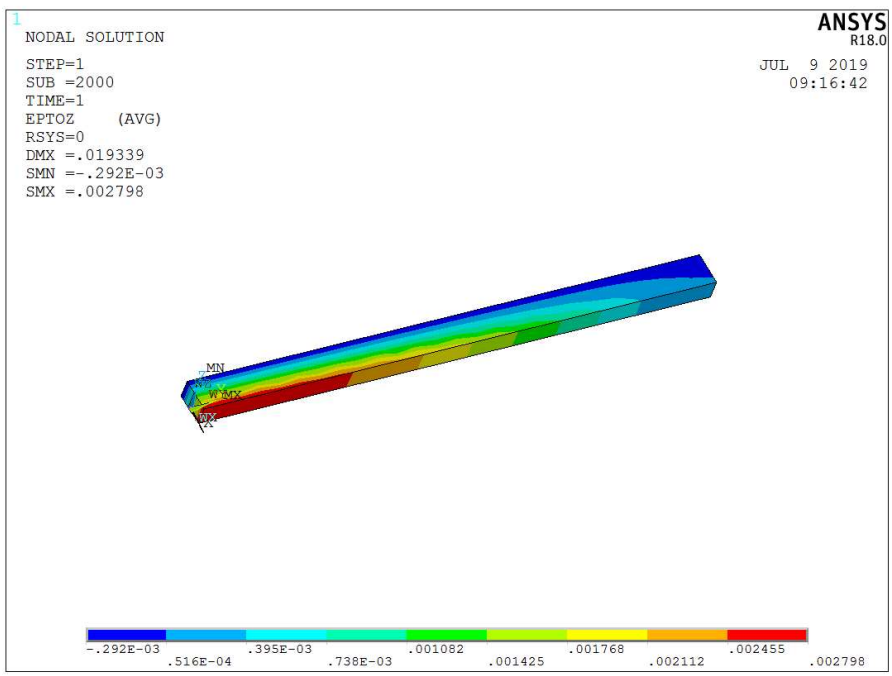


Figure 3-11: Contour Plots of Concrete Vertical Stress (Top, psi) and Strain (Bottom, in./in.) of Section Along Bottom of the Long Wall

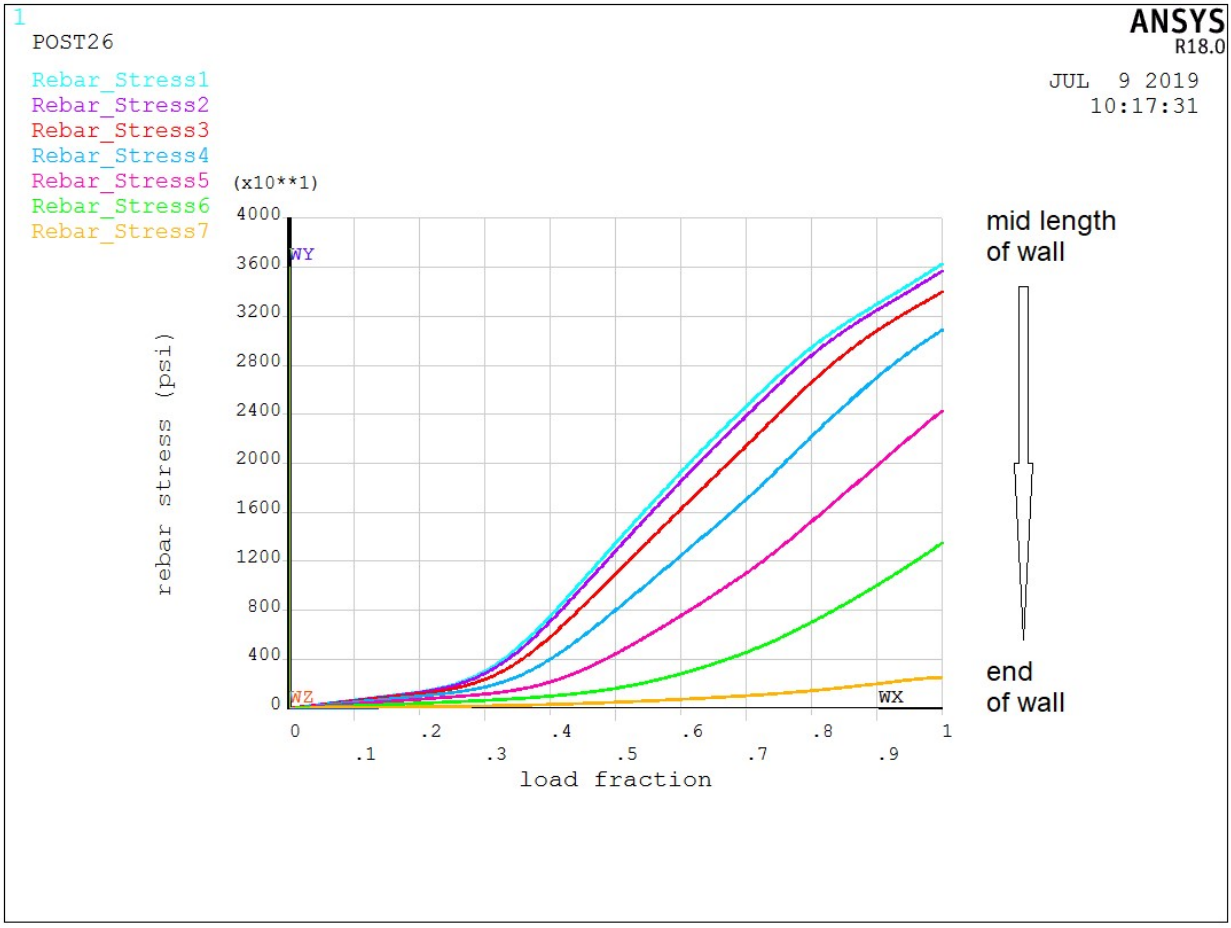


Figure 3-12: Time History of Stresses (psi) in Vertical Outside Face Rebar (over 12 ft length starting from wall mid length)

3.1 Sensitivity to Load Step Size

The total load in the nonlinear analysis is applied incrementally in load steps. Results of non-linear analyses should be independent of the selected load step size. Therefore, the load step size should be sufficiently small and verification of solution independence should be demonstrated. This is done by performing the above analysis with 3,000 load steps instead of the 2,000 steps used for the base analysis. Table 3-1 compares the results for the two load step sizes. It is evident that there is no change in the results. Therefore, the base solution results above are independent of the load step and the selected load step is adequately small.

Table 3-1: Load Step Size Sensitivity

	Base Solution 2,000 load steps	3,000 load steps	% difference
Max Rebar Tension (psi)	36,279	36,279	0.0
Max Concrete Tensile Strain*	0.002798	0.002798	0.0
Max Total Deflection (in.)	0.217988	0.217988	0.0

∗ Obtained 6 in. from base of wall at mid-length in between vertical rebar.

3.2 Sensitivity to Mesh Size

Results of the analysis were also determined to be independent of mesh size. This was assessed by reducing the concrete element sizes through the thickness in the long wall by 50%. Results of the refined mesh are compared to the base results, as shown in Table 3-2. A 50% reduction in element size produces 6.5 % change in the results. Since the acceptance criteria of the analysis is to demonstrate that the reinforced concrete structure of the tank is stable, this change in results is considered within acceptable limits. Therefore, the base solution discussed above is considered acceptable (i.e. the mesh size in the base solution is adequate).

Table 3-2: Mesh Size Sensitivity

	Base solution	Refined Mesh	% difference**
Max Rebar Tension (psi)	36,279	34,216	6.0
Max Concrete Tensile Strain*	0.002798	0.002993	6.5
Max Total Deflection (in.)	0.217988	0.220668	1.2

* Obtained 6 in. from base of wall at mid-length in between vertical rebar.

** With respect to refined mesh solution

4. STANDOFF CALCULATIONS FOR PUREX AND REDOX FILTERS

Calculations were performed to determine the closest distance heavy equipment that can be placed near the PUREX and REDOX filter structures.

The standoff distance evaluation was performed for a conservatively bounding equipment total weight of 60,000 lb. This weight conservatively bounds heavy equipment such as the 10 ton M123/125 trucks. Equipment layout dimensions used to distribute the weight on the ground surface were taken as 24 ft x 10 ft, which bound the length x width dimensions of the M123/M125 trucks. The equipment load is considered as live load and is added to Load Combination 1 which is shown in [28] to be the limiting load combination. A live load factor of 1.7 is applied based on the load combination definition in Section 9.2.1 of [5].

The standoff distance lower bound values are such that the equipment load will not be impacting the roof structure. Only exterior walls may be loaded. Calculations were performed to obtain the lateral earth pressure due to existing earth fill and ground level equivalent surcharge. Principals of statics were used to obtain the resulting bending moment and shear forces in the wall, which were compared to reinforced concrete section strength computed per the ACI 318 Code [4].

Lateral earth pressure on walls was calculated assuming no movement of the walls and thus, the at-rest earth pressure coefficients are used (as opposed to the lower value active earth pressure coefficients that are commonly used in retaining wall analysis). This is conservative because any small wall deflection will tend to reduce the lateral earth pressure below the at-rest values. All soil media were assumed to be sand with internal angle of friction, ϕ, of 30 deg. Likewise, earth fill was assumed to be sand with no cohesion, and a coefficient of friction of at least 30 deg. Thus, the at-rest earth pressure coefficient is obtained as follows:

$$Ko = 1 - \sin(\phi) \qquad \text{[27, eq. 7.3]}$$
$$= 1 - 0.5 = 0.5$$

The lateral earth pressure due to surcharge on a limited area of ground surface is obtained using the following equations:

$$\text{Lateral surcharge pressure} = M\{2\, qequip\, (\beta - \sin(\beta)\cos(2\alpha))/\pi\} \qquad \text{[27, eq. 8.28]}$$

where:

qequip = surcharge (load intensity at ground surface)
= 60,000 lb / (24 ft x 10 ft)
= 250 lb / ft^2
$\alpha = (\theta 2 + \theta 1)/2$ (see Figure 4-1)

$\beta = \theta 2 - \theta 1$ (see Figure 4-1)
$M = 1.4 - 0.4\, b' / 0.14\, H \geq 1.0$ [27, eq. 8.29]
H = depth at bottom of wall (Hb in Figure 4-1)

The surcharge in the above equations is assumed to be a strip load, that is, continuous along the wall length. Any internal pressure on the walls opposes the external lateral earth pressure. Such loads are conservatively not included in the analysis herein.

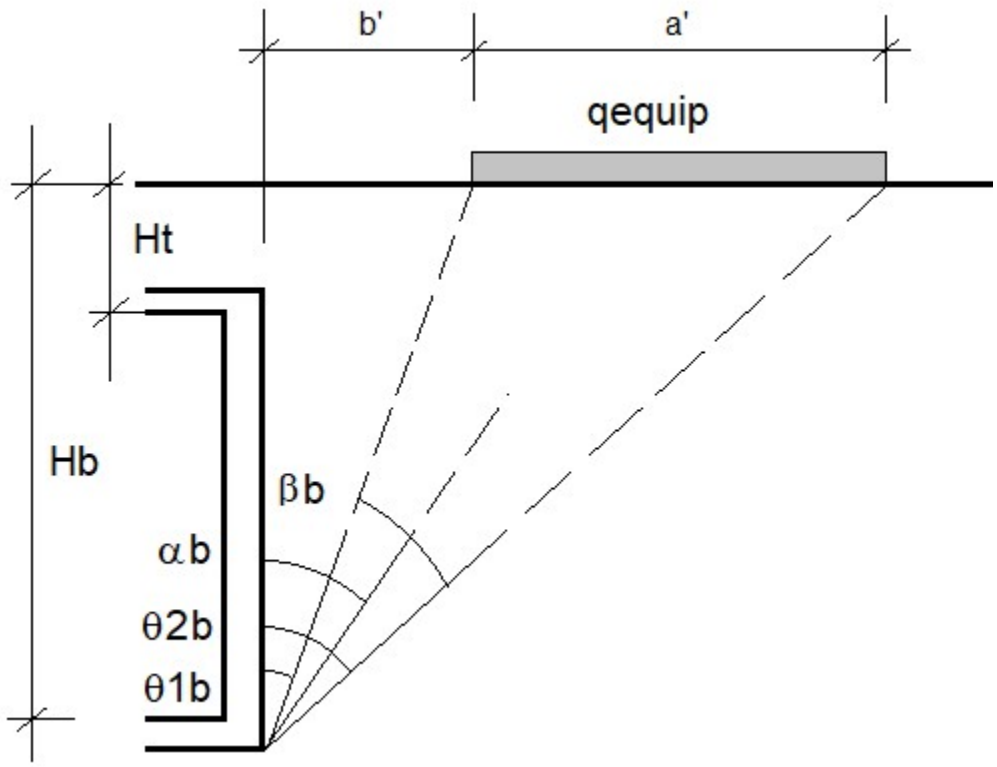


Figure 4-1: Lateral Earth Pressure Due to Surcharge
(note that "b" in the angle labels refers to location at bottom of wall)

An upper bound standoff distance was also calculated on the basis that the surcharge load does not impact the walls. For this calculation the surcharge load distribution through the soil is conservatively taken to follow the failure line for passive earth pressure, which makes an angle of $45^o - \phi/2$ with the horizontal, as shown in Figure 4-2. Using the geometry in this figure, the upper bound standoff distance is as follows:

Upper Bound Standoff Distance = depth of the foundation slab x tan ($45^o + \phi/2$).

Beyond this distance there is not limitation on equipment weight.

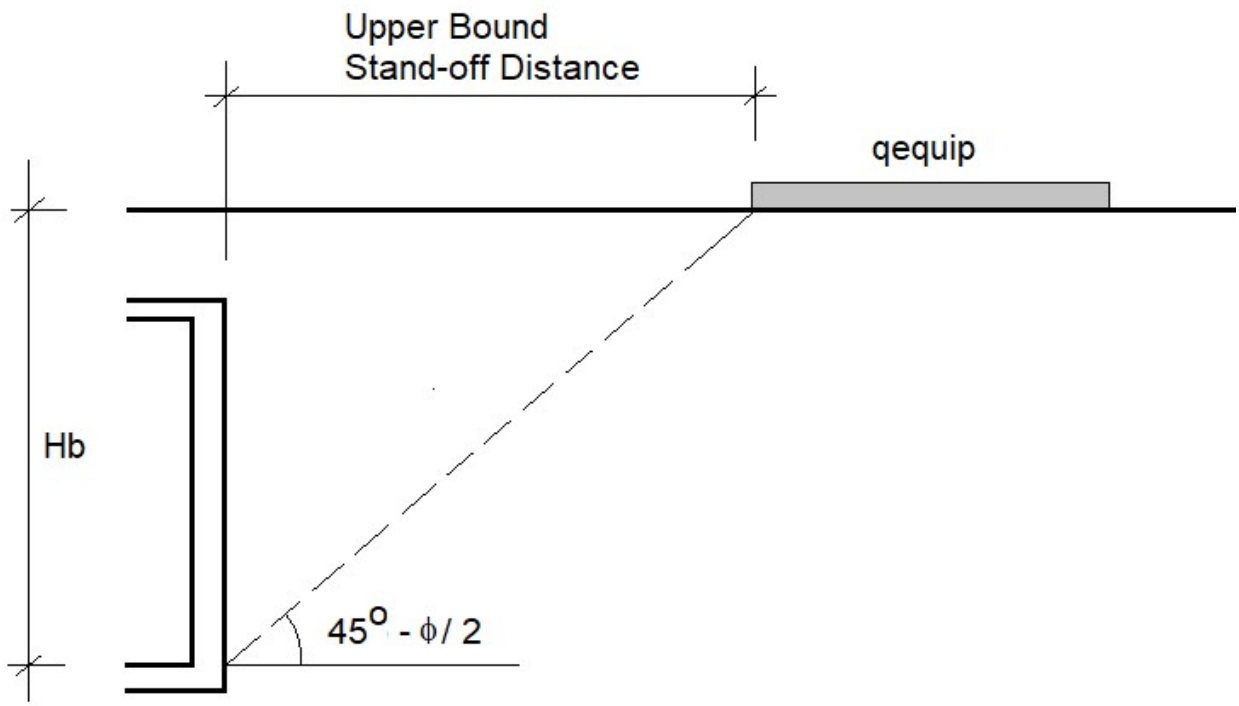


Figure 4-2: Illustration of Upper Bound Stand-off Distance

4.1 Structure PUREX Filters, Group O1

An 8 ft wide x 8 ft high reinforced-concrete exhaust air duct connects the 202-A Building to deep-bed filter 1 (bypassed) and deep-bed filter 2 (active) located in the south yard area and is composed of below-grade concrete exhaust air treatment and discharge equipment. The overall dimensions of deep-bed filter 2 area are 82 x 52 x 13 ft deep. There is 4 ft of overburden on the roof structure and 4 in. of shotcrete to mitigate water intrusion [18]. Degradation caused by water intrusion has been observed and, thus, accounted for by reducing roof slab and walls thicknesses by 1 in. and rebar cross sectional area by 10% [18, 28].

Review of the PUREX filter layout drawings, the equipment access is evaluated on the east, south, and west sides of the PUREX filter #2. The north side of PUREX Filter #2 is against Filter #1 structure and therefore equipment access is not postulated. The design drawings are used to obtain the concrete wall thickness and reinforcement information.

Detailed calculations of the lateral earth loading, demand, and concrete capacities are provided in Appendix A of this document. It is shown that the walls are within the ACI 318 Code [4] design limits for 60,000 lb surcharge at 10 ft (lower bound standoff distance) from the east, south and west walls. The upper bound standoff distance is as follows:

Depth of foundation slab x tan ($45^o + \phi/2$) = (13 ft + 7.5 in.) tan (60°) = 23.6 ft use 24 ft

The duct in the north-east corner of the structure is at a shallower depth (709 ft – (696 ft + 7.125 in.) = 12 ft + 4.875 in.) than the filter structure. However, the difference in depth is small and the same upper bound standoff distance is maintained.

The plug located in the south-west corner of the building and the plug in the north-east duct are not considered adequate to resist equipment loads and are not, therefore, bounded by the lower bound standoff distance calculated above. Accordingly, the calculated upper bound standoff distance of 24 ft is specified for all plug locations. Figure 4-3 summarizes the standoff distance results.

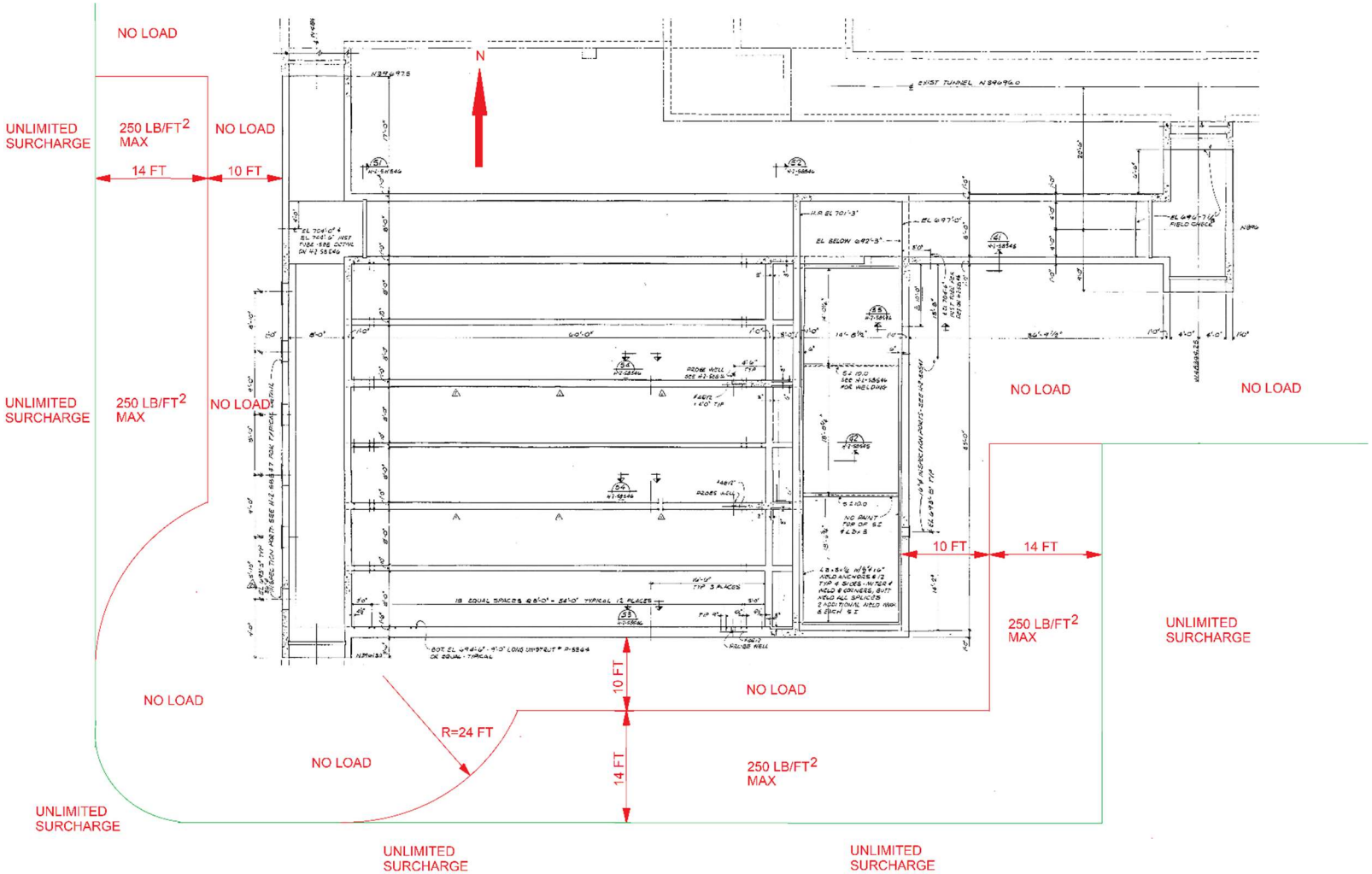


Figure 4-3: Standoff Distance Map – PUREX Filters (Group O1)

4.2 Structure REDOX Filter, Group O2

The REDOX sand filter is a below-grade structure approximately 85 x 85 x 20 ft deep, consisting of approximately 12 ft of sand and 8 ft of air space in a concrete shell. The filter medium decreases in particle size from coarse gravel at the bottom to 30-mesh sand at the top. The roof over the sand filter has been repaired and is in good condition [18]. Degradation caused by water intrusion has been observed, which is accounted for by reducing roof slab and walls thicknesses by 1 in. and rebar cross sectional area by 10% [18, and 28].

Evaluation of the REDOX filter was performed for equipment access near all four sides of the structure due to lack of information of the surroundings. The walls along the four sides are 1 ft thick. Wall reinforcement information for the REDOX concrete was not available; therefore, ACI 318 Code [4] strength calculations were performed using reinforcement details of the PUREX filter 1 ft thick external walls. Air ducts with approximate 1.5 ft wall thickness runs below the foundation slab along three sides of the structure. Reinforcement of the 1 ft thick walls was assumed similar to that in the 1 ft thick walls of the PUREX filters.

Based on the detailed evaluation in Appendix A of this document, the calculated standoff distance for the equipment weight of 250 lb/ft^2 is approximately 24 ft. This is within the range for the upper bound standoff distance calculated as:

$Hb \times \tan(45^o + \phi/2) = (12 \text{ ft} + 5 \text{ in.}) \tan(60^o) = 21.5 \text{ ft, use } 24 \text{ ft}$

Therefore, the standoff distance is considered to be 24 ft at all locations and there is no limitation on equipment weights located 24 ft or more away from the external walls, as shown in Figure 4-4.

This evaluation did not account for the perimeter ducts nor the plugs. The perimeter ducts are approximately 6 ft in height and their walls are approximately 1.5 ft thick. Based on the calculations performed herein, the surcharge effect on the ducts/plugs is expected to be less than 5% of the existing lateral earth pressure for the given standoff distance. Therefore, the standoff distance of 24 ft is applicable everywhere around the REDOX filters structure.

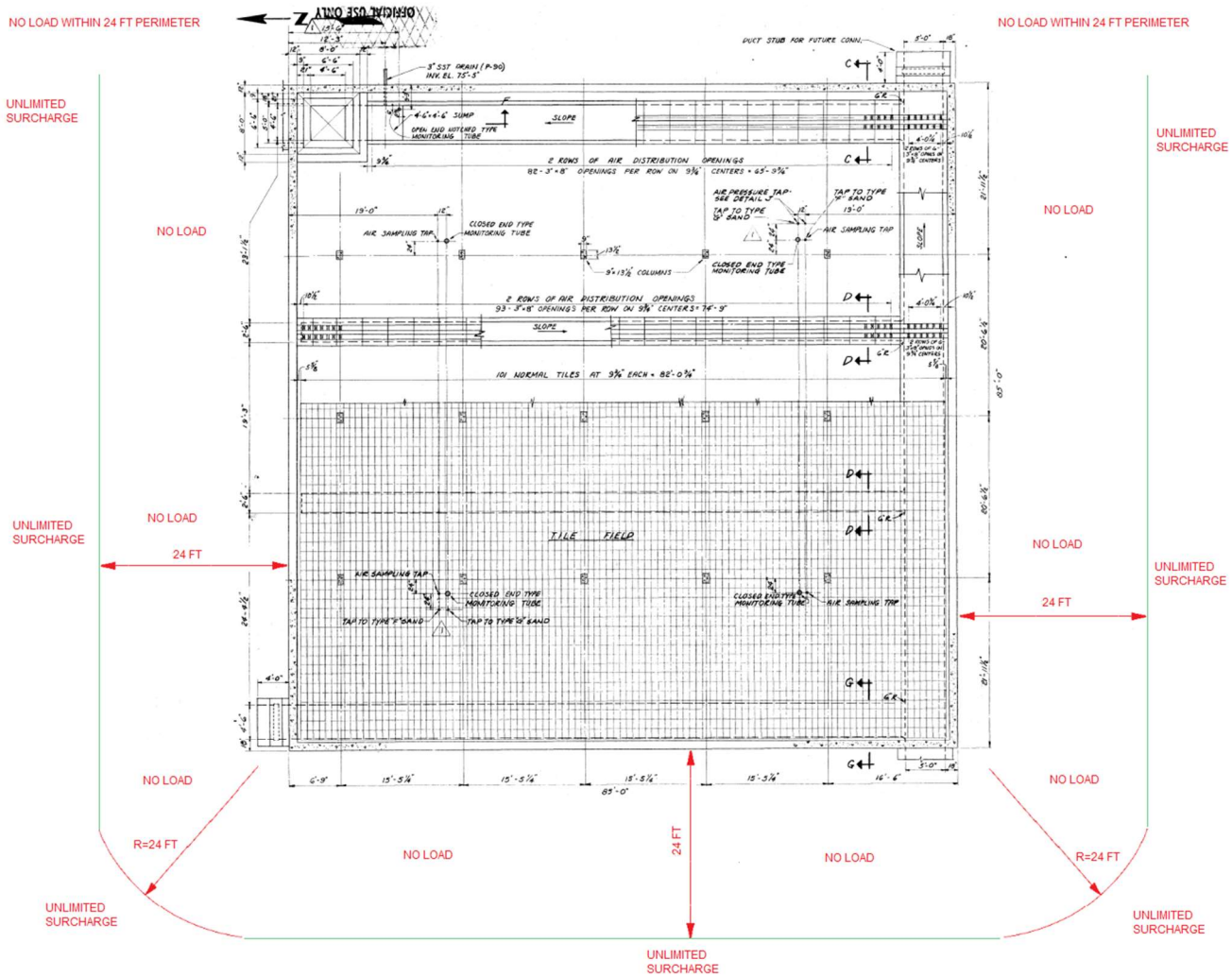


Figure 4-4: Standoff Distance Map – REDOX Filter (Group O2)

5. SUMMARY

LPI performed structural analyses of the facilities, which have the highest combined risk and consequence of failure as reported in the "RL-40 Aging Structures Risk Assessment" report [18, and 28]. LPI was requested to perform additional analyses on select buried structures, including a progressive failure analysis of Tank 241-Z-361, and standoff calculations for the PUREX and REDOX filters. The objective of the progressive failure analysis is to determine if Tank 241-Z-361 maintains structural integrity subject to concrete cracking. The purpose of the standoff calculations was to determine the minimum distance required between heavy equipment (greater than ten tons gross vehicle weight rating) and the perimeter of the PUREX and REDOX filters to ensure ACI 318 [4] design limits are met.

Using a verified and validated version of ANSYS ® software, a progressive failure analysis of Tank 241-Z-361 was performed. The stability analysis employed factored load combination 1 (see Section 3 of this document) and strength limits based on minimum specified material strength limits to investigate the nonlinear behavior of the reinforced concrete and structural stability of the tank. The acceptance strength limits calculated in [28] in accordance with ACI 318, do not represent actual failure limits but, rather, provide some indication of the potential for failure. In addition, Mathcad was used to perform the standoff calculations for the PUREX and REDOX filters.

Results are based on the available information (drawings [2], and "RL-40 Aging Structures Risk Assessment" report [18]) and assumptions reported herein (see Section 3, 4, and Appendix A of [28]). The progressive failure analysis and standoff calculations presented address the overall structural stability of the underground structures in the scope of work and should not be used to justify worker safety. Accordingly, the results presented in this report should be assessed within the context of the available information and applied assumptions.

Standoff calculations show that the PUREX filter walls are within the ACI 318 Code [4] design limits for 60,000 lb surcharge at 10 ft (lower bound standoff distance) from the east, south and west walls, and an upper bound standoff distance of 24 ft for all plug locations. Standoff calculations for the REDOX filter walls show that the minimum standoff distance is 24 ft and that there is no load limitations beyond that distance.

The progressive failure analysis of Tank 241-Z-361 (nonlinear analysis) indicates that concrete cracking will occur and progresses along the bottom of the long wall. However, multiple redundancies in the tank box will provide adequate load paths and strength to resist the applied loads without excessive damage. It was also shown that rebar tensile stresses remain below yield strength of the reinforcement material.

The progressive failure analysis of Tank 241-Z-361 presented herein is based on current code requirements for load factors, load combinations, and strength reductions for concrete and steel materials. The assessment of the original design requirements was not in the scope of work; therefore, the results of this report should not be used to characterize the adequacy of the original design.

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APPENDIX A

STANDOFF CALCULATIONS:

PUREX AND REDOX FILTERS

PUREX FILTERS

Surcharge Load Standoff Distance - PUREX Filters (Group O1):

The calculation presented herein includes evaluation of external walls for the lateral earth pressure due to earth fill and surcharge load of heavy equipment in the proximity of the structure. The surcharge load distance from the structure is such that it will not result in additional loading on the roof and therefore roof evaluation is not included. Heavy equipment are not postulated on the north side of the building because of the presence of the neighboring filter structure. By review of the design drawings, the south and west walls are considered to be the most limiting of all external walls. The lateral earth pressure includes the effect of 4" shotcrete on top of the overburden. Degradation of the reinforced concrete is considered by reducing concrete wall thickness and area of tensile rebar. These reductions are implemented for the inside face of the wall only.

The strength evaluation of the concrete walls is performed per ACI 318 requirements using load combination 1 in LPI report LA181779-R-001. This is the limiting load combination for this scope of work based on review of applicable load combinations in ACI 318 and ASCE 7. The surcharge load is considered as live load. Per ACI 318 and ASCE 7, the bounding live load factor for load combination 1 is 1.7. Internal pressure due to filter materials is ignored since it opposes the external earth pressure and its magnitude is uncertain.

Refer to Section 4.0 of this report for equations for the surcharge load induced lateral earth pressure.

References:

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Calculations:

$H_{sc} := 4 \cdot in$	thickness of top shotcrete layer over the soil overburden
$H_t := 4 \cdot ft$	soil depth at top of wall, above slab
$H_b := 13 \cdot ft + 7.5 \cdot in$	soil depth at bottom of wall

$\gamma_{con} := 150 \cdot \frac{lbf}{ft^3}$ unit weight of concrete, used for shotcrete

$\gamma_{earth} := 125 \cdot \frac{lbf}{ft^3}$ unit weight of earth fill and overburden

$K_o := 0.5$ at-rest lateral earth pressure coefficient (See Section 4.0)

$LL_multiplier := 1.7$ load multiplier for LL for Load Combination 1 [2, 3]

$b' := 10 \cdot ft$ standoff distance from wall, obtained by iteration

$a' := 24 \cdot ft$ width of equipment based on assumed M123/125 truck.

$Pequip := 60000 \cdot lbf$ equipment total weight, obtained by iteration

$qequip := \frac{Pequip}{24 \cdot ft \cdot 10 \cdot ft} = 250 \cdot \frac{lbf}{ft^2}$ equipment load intensity at grade

The lateral earth pressure on the wall is the sum of the lateral earth pressure due to soil fill and overburden and the lateral earth pressure due to the surcharge load.

$\theta 1t := atan\left(\frac{b'}{Ht}\right) = 1.19$ $\theta 2t := atan\left(\frac{b' + a'}{Ht}\right) = 1.454$ geometric angles defining the point at top of wall, see Figure 4-1 in the body of the report

$\beta t := \theta 2t - \theta 1t = 0.263$

$\alpha t := \frac{\theta 2t + \theta 1t}{2} = 1.322$

$sM := max\left(1.4 - \frac{0.4 \cdot b'}{0.14 \cdot Hb}, 1.0\right) = 1$ soil surcharge multiplier

$sequipt := sM \cdot \frac{2qequip}{\pi} \cdot (\beta t - sin(\beta t) \cdot cos(2 \cdot \alpha t)) = 78.334 \cdot \frac{lbf}{ft^2}$ lateral earth pressure at top of wall due to truck weight

$prest := LL_multiplier\left[K_o \cdot \left(\gamma_{earth} \cdot Ht + \gamma_{con} \cdot Hsc\right) + sequipt\right] = 601 \cdot \frac{lbf}{ft^2}$ total lateral earth pressure at top of wall

$\theta 1b := atan\left(\frac{b'}{Hb}\right) = 0.633$ $\theta 2b := atan\left(\frac{b' + a'}{Hb}\right) = 1.19$ geometric angles defining the point at bottom of wall

$\beta b := \theta 2b - \theta 1b = 0.557$

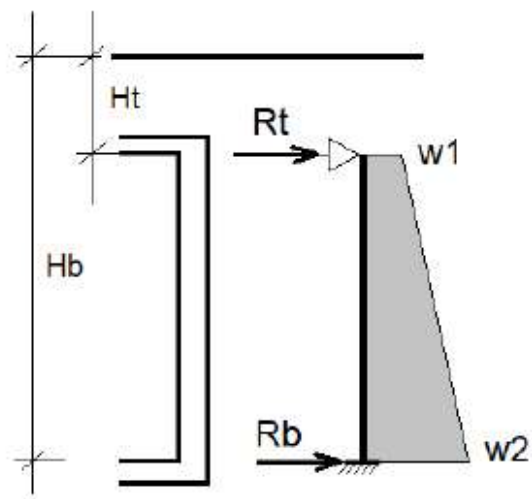
$\alpha b := \frac{\theta 2b + \theta 1b}{2} = 0.911$

$sequipb := sM \cdot \frac{2qequip}{\pi} \cdot (\beta b - \sin(\beta b) \cdot \cos(2 \cdot \alpha b)) = 109.534 \cdot \frac{lbf}{ft^2}$ lateral earth pressure at top of wall due to truck weight

$presb := LL_multiplier\left[K_o \cdot \left(\gamma_{earth} \cdot Hb + \gamma_{con} \cdot Hsc\right) + sequipb\right] = 1676 \cdot \frac{lbf}{ft^2}$ total lateral earth pressure at top of wall

Bending Moment and Shear in South Wall

The wall is assumed pinned at top at floor slab level and fixed at bottom at foundation slab. Loading is approximated with a trapezoidal distribution with w1 intensity at top and w2 intensity at bottom.



$w1 := prest \cdot ft$ total lateral load intensity at top of wall

$w2 := presb \cdot ft$ total lateral load intensity at bottom of wall

$L := Hb - Ht$ span of wall vertical strip

$Rt(w1, w2, L) := \frac{3 \cdot w1 \cdot L}{8} + \frac{(w2 - w1) \cdot L}{10}$ total reaction at top pinned end [5, Table 3, case 2c]

$Rt(w1, w2, L) = 3.203 \times 10^3 \cdot lbf$

The free-body diagram above is used for the derivation of the following forces and moments.

$Rb(w1, w2, L) := \frac{(w2 + w1) \cdot L}{2} - Rt(w1, w2, L)$ total reaction at bottom

$Rb(w1, w2, L) = 7.755 \times 10^3 \cdot lbf$

$Shear(x, w1, w2, L) := Rt(w1, w2, L) - w1 \cdot x - (w2 - w1) \frac{x}{L} \cdot \frac{x}{2}$ shear at distance x from top of wall

$$Mom(x, w1, w2, L) := Rt(w1, w2, L) \cdot x - w1 \cdot x \cdot \frac{x}{2} - (w2 - w1) \cdot \frac{x}{L} \cdot \frac{x}{2} \cdot \frac{x}{3}$$

bending moment at distance x from top of wall

$$Momneg(w1, w2, L) := \left(\frac{-w2}{15} - \frac{7w1}{120}\right) \cdot L^2$$

moment at bottom of wall (x = L)

$$Shearps := Shear(0, w1, w2, L) = 3.203 \times 10^3 \cdot lbf$$

maximum positive shear

$$Shearng := Shear(L, w1, w2, L) = -7.755 \times 10^3 \cdot lbf$$

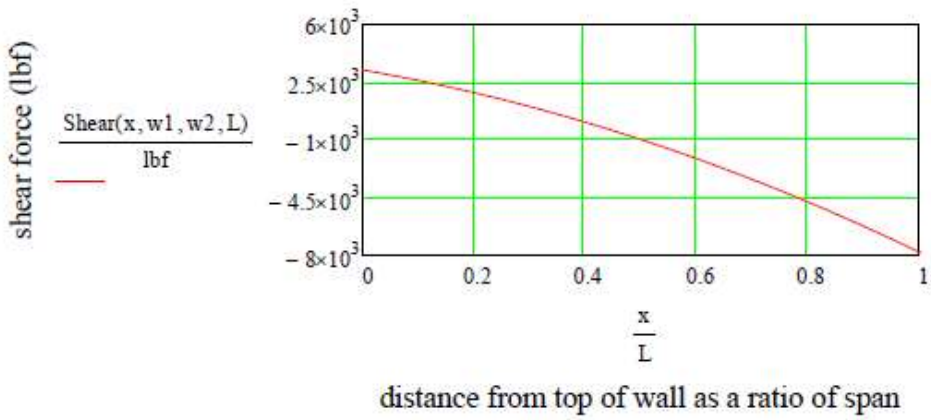
maximum negative shear

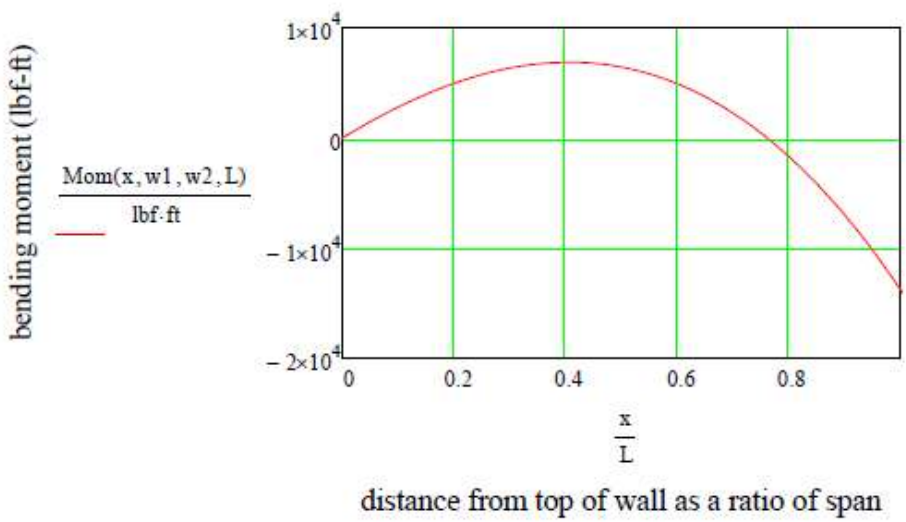
$$Mom(L, w1, w2, L) = -1.36 \times 10^4 \cdot lbf \cdot ft$$

maximum negative wall moment at bottom of wall

$$Momneg(w1, w2, L) = -1.36 \times 10^4 \cdot lbf \cdot ft$$

$$Momng := Momneg(w1, w2, L)$$





Change to unitless expressions to use the Mathcad Maximize function to obtain maximum positive moment. Use consistent lbf and ft units

$$w'1 := \frac{w1}{lbf \cdot ft^{-1}} \quad w'2 := \frac{w2}{lbf \cdot ft^{-1}} \qquad L' := \frac{L}{ft}$$

$$MomU(x') := Rt(w'1, w'2, L') \cdot x' - w'1 \cdot x' \cdot \frac{x'}{2} - (w'2 - w'1) \cdot \frac{x'}{L'} \cdot \frac{x'}{2} \cdot \frac{x'}{3}$$

Unitless moment equation using consistent lbf, ft units

$x' := 0.4 \cdot L'$ initial guess value for the distance at maximum positive moment

Given use MathCad "Given" function to find the distance x at maximum positive moment

$maxx := Maximize(MomU, x')$

$maxx = 3.91$ distance x in ft at max positive moment

$MomU(maxx) = 6.82 \times 10^{3}$ max positive moment in lbf-ft

$Momps := MomU(maxx) \cdot lbf \cdot ft = 8.184 \times 10^{4} \cdot lbf \cdot in$

$Momng = -1.632 \times 10^{5} \cdot lbf \cdot in$

PUREX Filters (Group O1) - South Wall

South wall evaluation for the surcharge load is performed for a section at about mid-height (at maximum positive moment) where the inside face is at tension and at a section at bottom with the tension and compression faces reversed. Reinforcement is per the design drawings. Strength evaluations for bending and shear are per [2, 6].

$b := 12 \cdot in$ wall strip width

Inside face vertical rebar #4 every 12", reduce by 10% for degradation

$d := 18 \cdot in - 1 \cdot in - \frac{0.5 \cdot in}{2}$ depth of inside face vertical rebar

$As4 := 0.2 \cdot in^2$ cross section area of #4 rebar [6]

$As := As4 \cdot \frac{b}{12 \cdot in} \cdot 0.9 = 0.18 \cdot in^2$ degraded rebar area proportioned for strip width

$\phi Pn_SI := 0.7 \cdot 0.8 \cdot [0.85 \cdot fc \cdot (b \cdot d - As) + fy \cdot (As)] = 2.908 \times 10^5 \cdot lbf$ axial force capacity

$\phi Mn_SI := 0.9 \cdot As \cdot fy \cdot \left(d - \frac{As \cdot fy}{2 \cdot 0.85 \cdot fc \cdot b}\right) = 107778 \cdot in \cdot lbf$ bending moment capacity

$\phi \tau n_SI := 0.75 \left(2 \cdot \sqrt{\frac{fc}{psi}}\right) \cdot psi = 82 \cdot psi$ shear stress capacity

$\phi Vn_SI := \phi \tau n_SI \cdot b \cdot d = 1.651 \times 10^4 \cdot lbf$ shear force capacity

Outside face vertical rebar at bottom #10 every 9", not degraded.

$d := 18 \cdot in - 1 \cdot in - \frac{1.25 \cdot in}{2} - 1 \cdot in$ depth of outside face vertical rebar, remove 1" for degradation

$As10 := 1.27 \cdot in^2$ cross section area of #10 rebar [6]

$As := As10 \cdot \frac{b}{9 \cdot in} = 1.693 \cdot in^2$ rebar area proportioned for strip width

$\phi Pn_SO := 0.7 \cdot 0.8 \cdot [0.85 \cdot fc \cdot (b \cdot d - As) + fy \cdot (As)] = 2.99 \times 10^5 \cdot lbf$ axial force capacity

$\phi Mn_SO := 0.9 \cdot As \cdot fy \cdot \left(d - \frac{As \cdot fy}{2 \cdot 0.85 \cdot fc \cdot b}\right) = 869792 \cdot in \cdot lbf$ bending moment capacity

$\phi\tau n_SO := 0.75\left(2\cdot\sqrt{\frac{fc}{psi}}\right)\cdot psi = 82\cdot psi$ shear stress capacity

$\phi Vn_SO := \phi\tau n\cdot b\cdot d = 1.516\times 10^{4}\cdot lbf$ shear force capacity

$IR_Shearps := \frac{Shearps}{\phi Vn_SI} = 0.19$ demand over capacity ratio for shear at top < 1.0 acceptable

$IR_Shearng := \frac{|Shearng|}{\phi Vn_SI} = 0.47$ demand over capacity ratio for shear at bottom < 1.0 acceptable

$IR_Momps := \frac{Momps}{\phi Mn_SI} = 0.76$ demand over capacity ratio for moment at about mid height < 1.0 acceptable

$IR_Momng := \frac{|Momng|}{\phi Mn_SO} = 0.19$ demand over capacity ratio for moment at about mid height < 1.0 acceptable

PUREX Filters (Group O1) - West Wall

West wall evaluation for the surcharge load is performed for a section at about mid-height (at maximum positive moment) where the inside face is at tension and at a section at bottom with the tension and compression faces reversed.

$b := 12 \cdot in$ wall strip width

Inside face vertical rebar #6 every 9", reduce by 10% for degradation

$d := 12 \cdot in - 1 \cdot in - \frac{0.75 \cdot in}{2}$ depth of inside face vertical rebar

$As6 := 0.44 \cdot in^2$ cross section area of #6 rebar [6]

$As := As6 \cdot \frac{b}{9 \cdot in} \cdot 0.9 = 0.528 \cdot in^2$ degraded rebar area proportioned for strip width

$\phi Pn_SI := 0.7 \cdot 0.8 \cdot [0.85 \cdot fc \cdot (b \cdot d - As) + fy \cdot (As)] = 1.931 \times 10^5 \cdot lbf$ axial force capacity

$\phi Mn_SI := 0.9 \cdot As \cdot fy \cdot \left(d - \frac{As \cdot fy}{2 \cdot 0.85 \cdot fc \cdot b}\right) = 195400 \cdot in \cdot lbf$ bending moment capacity

$\phi \tau n_SI := 0.75 \left(2 \cdot \sqrt{\frac{fc}{psi}}\right) \cdot psi = 82 \cdot psi$ shear stress capacity

$\phi Vn_SI := \phi \tau n_SI \cdot b \cdot d = 1.048 \times 10^4 \cdot lbf$ shear force capacity

Outside face vertical rebar at bottom #6 every 9" spliced with #7 every 9". Use average of the two sizes.

$d := 12 \cdot in - 1 \cdot in - \frac{0.75 \cdot in}{2} - 1 \cdot in$ depth of outside face vertical rebar, remove 1" for degradation

$As6 := 0.44 \cdot in^2$ cross section area of #6 rebar [6]

$As7 := 0.60 \cdot in^2$ cross section area of #7 rebar [6]

$As := \frac{As6 + As7}{2} \cdot \frac{b}{9 \cdot in} = 0.693 \cdot in^2$ rebar area proportioned for strip width

$\phi Pn_SO := 0.7 \cdot 0.8 \cdot [0.85 \cdot fc \cdot (b \cdot d - As) + fy \cdot (As)] = 1.795 \times 10^5 \cdot lbf$ axial force capacity

$\phi Mn_SO := 0.9 \cdot As \cdot fy \cdot \left(d - \frac{As \cdot fy}{2 \cdot 0.85 \cdot fc \cdot b}\right) = 228929 \cdot in \cdot lbf$ bending moment capacity

$\phi\tau n_SO := 0.75\left(2\cdot\sqrt{\frac{fc}{psi}}\right)\cdot psi = 82\cdot psi$ shear stress capacity

$\phi Vn_SO := \phi\tau n\cdot b\cdot d = 9.489\times 10^{3}\cdot lbf$ shear force capacity

$IR_Shearps := \frac{Shearps}{\phi Vn_SI} = 0.31$ demand over capacity ratio for shear at top < 1.0 acceptable

$IR_Shearng := \frac{|Shearng|}{\phi Vn_SI} = 0.74$ demand over capacity ratio for shear at bottom < 1.0 acceptable

$IR_Momps := \frac{Momps}{\phi Mn_SI} = 0.42$ demand over capacity ratio for moment at about mid height < 1.0 acceptable

$IR_Momng := \frac{|Momng|}{\phi Mn_SO} = 0.71$ demand over capacity ratio for moment at about mid height < 1.0 acceptable

REDOX FILTERS

Surcharge Load Standoff Distance - REDOX Filters (Group O2):

The calculation presented herein includes evaluation of external walls for the lateral earth pressure due to earth fill and surcharge load of heavy equipment in the proximity of the structure. The surcharge load distance from the structure is such that it will not result in additional loading on the roof and therefore roof evaluation is not included. No information is available for the building surroundings and therefore, heavy equipment are postulated on all sides of the building. By review of the design drawings, all external walls are 1 ft thick but no information is available for wall reinforcement. As such, one case is evaluated with assumed set of rebar. Degradation of the reinforced concrete is considered by reducing concrete wall thickness and area of tensile rebar. These reductions are implemented for the inside face of the wall only.

The strength evaluation of the concrete walls is performed per ACI 318 requirements using load combination 1 in LPI report LA181779-R-001. This is the limiting load combination for this scope of work based on review of applicable load combinations in ACI 318 and ASCE 7. The surcharge load is considered as live load. Per ACI 318 and ASCE 7, the bounding live load factor for load combination 1 is 1.7. Internal pressure due to filter materials is ignored since it opposes the external earth pressure and its magnitude is uncertain.

Refer to Section 4.0 of this report for equations for the surcharge load induced lateral earth pressure.

References:

1. LPI report LA181779-R-001
2. ACI 318
3. ASCE 7
4. Braja M Das, "Principles of Foundation Engineering," 6th Edition, copyright @ 2007 by NELSON, a division of Thomson Canada Limited.
5. R.J. Roark and W.C. Young, "Formulas for Stress and Strain," 5th Edition.
6. Gaylord, E.H. Jr. and Gaylord, C.H., "Structural Engineering Handbook," third Edition.

Calculations:

$Ht := 0.001 \cdot ft$ — soil depth at top of wall, above slab, a small value is used to avoid divide by zero in the geometric calculation below.

$Hb := 12 \cdot ft + 5 \cdot in$ — soil depth at bottom of wall

$\gamma_{earth} := 125 \cdot \frac{lbf}{ft^3}$ unit weight of earth fill and overburden

$K_o := 0.5$ at-rest lateral earth pressure coefficient (See Section 4.0)

$LL_multiplier := 1.7$ load multiplier for LL for Load Combination 1 [2, 3]

$b' := 24 \cdot ft$ standoff distance from wall, obtained by iteration

$a' := 24 \cdot ft$ width of equipment based on assumed M123/125 truck.

$Pequip := 60000 \cdot lbf$ equipment total weight, obtained by iteration

$qequip := \frac{Pequip}{24 \cdot ft \cdot 10 \cdot ft} = 250 \cdot \frac{lbf}{ft^2}$ equipment load intensity at grade

The lateral earth pressure on the wall is the sum of the lateral earth pressure due to soil fill and overburden and the lateral earth pressure due to the surcharge load.

$\theta 1t := atan\left(\frac{b'}{Ht}\right) = 1.571$ $\theta 2t := atan\left(\frac{b' + a'}{Ht}\right) = 1.571$ geometric angles defining the point at top of wall, see Figure 4-1 in the body of the report

$\beta t := \theta 2t - \theta 1t = 2.083 \times 10^{-5}$

$\alpha t := \frac{\theta 2t + \theta 1t}{2} = 1.571$

$sM := max\left(1.4 - \frac{0.4 \cdot b'}{0.14 \cdot Hb}, 1.0\right) = 1$ soil surcharge multiplier

$sequipt := sM \cdot \frac{2qequip}{\pi} \cdot (\beta t - sin(\beta t) \cdot cos(2 \cdot \alpha t)) = 6.631 \times 10^{-3} \cdot \frac{lbf}{ft^2}$ lateral earth pressure at top of wall due to truck weight

$prest := LL_multiplier\left(K_o \cdot \gamma_{earth} \cdot Ht + sequipt\right) = 0.118 \cdot \frac{lbf}{ft^2}$ total lateral earth pressure at top of wall

$\theta 1b := atan\left(\frac{b'}{Hb}\right) = 1.093$ $\theta 2b := atan\left(\frac{b' + a'}{Hb}\right) = 1.318$ geometric angles defining the point at bottom of wall

$\beta b := \theta 2b - \theta 1b = 0.224$

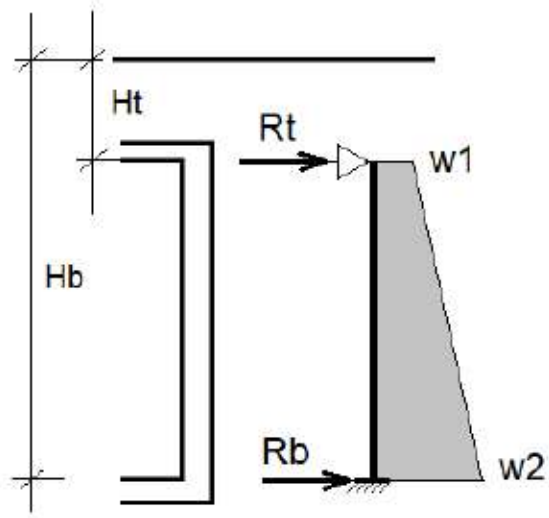
$\alpha b := \frac{\theta 2b + \theta 1b}{2} = 1.206$

$sequipb := sM \cdot \frac{2qequip}{\pi} \cdot (\beta b - \sin(\beta b) \cdot \cos(2 \cdot \alpha b)) = 62.066 \cdot \frac{lbf}{ft^2}$ lateral earth pressure at top of wall due to truck weight

$presb := LL_multiplier\left(K_o \cdot \gamma_{earth} \cdot Hb + sequipb\right) = 1425 \cdot \frac{lbf}{ft^2}$ total lateral earth pressure at top of wall

Bending Moment and Shear in the 1ft Thick Walls

The wall is assumed pinned at top at floor slab level and fixed at bottom at foundation slab and thick duct walls. Loading is approximated with a trapezoidal distribution with w1 intensity at top and w2 intensity at bottom.



$w1 := prest \cdot ft$ total lateral load intensity at top of wall

$w2 := presb \cdot ft$ total lateral load intensity at bottom of wall

$L := Hb - Ht$ span of wall vertical strip

$Rt(w1, w2, L) := \frac{3 \cdot w1 \cdot L}{8} + \frac{(w2 - w1) \cdot L}{10}$ total reaction at top pinned end [5, Table 3, case 2c]

$Rt(w1, w2, L) = 1.769 \times 10^3 \cdot lbf$

The free-body diagram above is used for the derivation of the following forces and moments.

$Rb(w1, w2, L) := \frac{(w2 + w1) \cdot L}{2} - Rt(w1, w2, L)$ total reaction at bottom

$Rb(w1, w2, L) = 7.076 \times 10^3 \cdot lbf$

$Shear(x, w1, w2, L) := Rt(w1, w2, L) - w1 \cdot x - (w2 - w1) \frac{x}{L} \cdot \frac{x}{2}$ shear at distance x from top of wall

$$Mom(x,w1,w2,L) := Rt(w1,w2,L)\cdot x - w1\cdot x\cdot\frac{x}{2} - (w2-w1)\cdot\frac{x}{L}\cdot\frac{x}{2}\cdot\frac{x}{3}$$

bending moment at distance x from top of wall

$$Momneg(w1,w2,L) := \left(\frac{-w2}{15} - \frac{7w1}{120}\right)\cdot L^2$$

moment at bottom of wall (x = L)

$$Shearps := Shear(0,w1,w2,L) = 1.769\times 10^3\cdot lbf$$

maximum positive shear

$$Shearng := Shear(L,w1,w2,L) = -7.076\times 10^3\cdot lbf$$

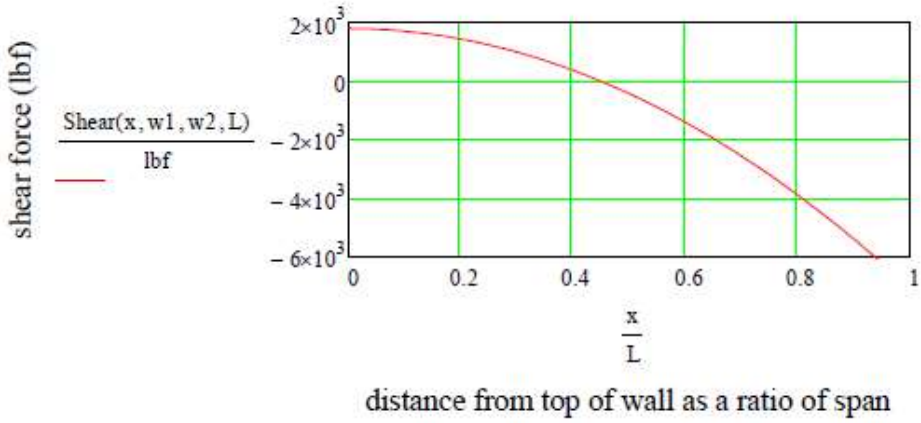
maximum negative shear

$$Mom(L,w1,w2,L) = -1.464\times 10^4\cdot lbf\cdot ft$$

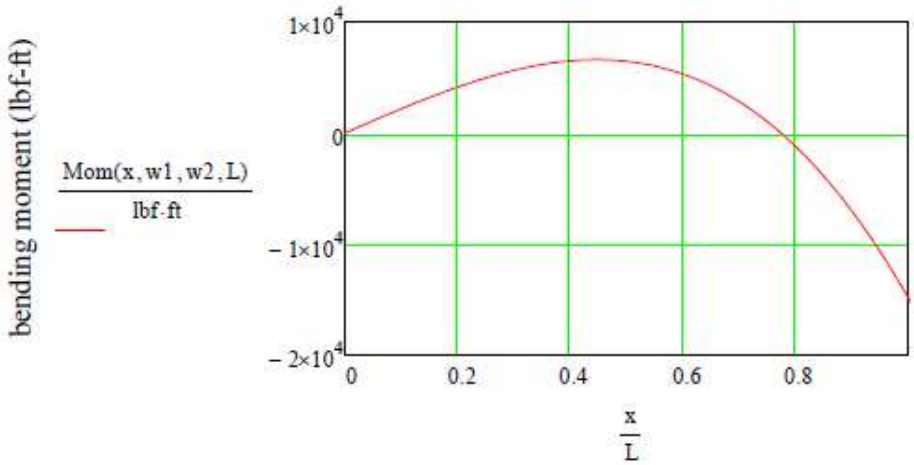
$$Momneg(w1,w2,L) = -1.464\times 10^4\cdot lbf\cdot ft$$

maximum negative wall moment at bottom of wall

$$Momng := Momneg(w1,w2,L)$$



distance from top of wall as a ratio of span



distance from top of wall as a ratio of span

Change to unitless expressions to use the Mathcad Maximize/Minimize functions to obtain maximum positive moment. Use consistent lbf and ft units

$$w'1 := \frac{w1}{lbf \cdot ft^{-1}} \quad w'2 := \frac{w2}{lbf \cdot ft^{-1}} \qquad L' := \frac{L}{ft}$$

$$MomU(x') := Rt(w'1, w'2, L') \cdot x' - w'1 \cdot x' \cdot \frac{x'}{2} - (w'2 - w'1) \cdot \frac{x'}{L'} \cdot \frac{x'}{2} \cdot \frac{x'}{3}$$

Unitless moment equation using consistent lbf, ft units

$x' := 0.4 \cdot L'$ initial guess value for the distance at maximum positive moment

Given use MathCad "Given" function to find the distance x at maximum positive moment

$maxx := Maximize(MomU, x')$

$maxx = 5.552$ distance x in ft at max positive moment

$MomU(maxx) = 6.549 \times 10^{3}$ max positive moment in lbf-ft

$Momps := MomU(maxx) \cdot lbf \cdot ft = 7.858 \times 10^{4} \cdot lbf \cdot in$

$Momng = -1.757 \times 10^{5} \cdot lbf \cdot in$

REDOX Filter (Group O2) - 1ft Thick Wall

All outer walls (excluding duct walls) are 1ft thick. No information is available for the reinforcement. The reinforcement is assumed similar to that in the 1 ft thick walls of the PUREX filter. Wall evaluation for the surcharge load is performed for a section at about mid-height (at maximum positive moment) where the inside face is at tension and at a section at bottom with the tension and compression faces reversed.

$b := 12 \cdot in$ wall strip width

Inside face vertical rebar #6 every 9", reduce by 10% for degradation

$d := 12 \cdot in - 1 \cdot in - \frac{0.75 \cdot in}{2}$ depth of inside face vertical rebar

$As6 := 0.44 \cdot in^2$ cross section area of #6 rebar [6]

$As := As6 \cdot \frac{b}{9 \cdot in} \cdot 0.9 = 0.528 \cdot in^2$

$\phi Pn_SI := 0.7 \cdot 0.8 \cdot [0.85 \cdot fc \cdot (b \cdot d - As) + fy \cdot (As)] = 1.931 \times 10^5 \cdot lbf$ axial force capacity

$\phi Mn_SI := 0.9 \cdot As \cdot fy \cdot \left(d - \frac{As \cdot fy}{2 \cdot 0.85 \cdot fc \cdot b}\right) = 195400 \cdot in \cdot lbf$ bending moment capacity

$\phi \tau n_SI := 0.75 \left(2 \cdot \sqrt{\frac{fc}{psi}}\right) \cdot psi = 82 \cdot psi$ shear stress capacity

$\phi Vn_SI := \phi \tau n_SI \cdot b \cdot d = 1.048 \times 10^4 \cdot lbf$ shear force capacity

Outside face vertical rebar at bottom #6 every 9" spliced with #7 every 9". Use average of the two sizes.

$d := 12 \cdot in - 1 \cdot in - \frac{0.75 \cdot in}{2} - 1 \cdot in$ depth of outside face vertical rebar, remove 1" for degradation

$As6 := 0.44 \cdot in^2$ cross section area of #6 rebar [6]

$As7 := 0.60 \cdot in^2$ cross section area of #7 rebar [6]

$As := \frac{As6 + As7}{2} \cdot \frac{b}{9 \cdot in} = 0.693 \cdot in^2$

$\phi Pn_SO := 0.7 \cdot 0.8 \cdot [0.85 \cdot fc \cdot (b \cdot d - As) + fy \cdot (As)] = 1.795 \times 10^5 \cdot lbf$ axial force capacity

$\phi Mn_SO := 0.9 \cdot As \cdot fy \cdot \left(d - \frac{As \cdot fy}{2 \cdot 0.85 \cdot fc \cdot b}\right) = 228929 \cdot in \cdot lbf$ bending moment capacity

$$\phi\tau n_SO := 0.75\left(2\cdot\sqrt{\frac{fc}{psi}}\right)\cdot psi = 82\cdot psi$$

shear stress capacity

$$\phi Vn_SO := \phi\tau n\cdot b\cdot d = 9.489\times 10^{3}\cdot lbf$$

shear force capacity

$$IR_Shearps := \frac{Shearps}{\phi Vn_SI} = 0.17$$

demand over capacity ratio for shear at top < 1.0 acceptable

$$IR_Shearng := \frac{|Shearng|}{\phi Vn_SI} = 0.68$$

demand over capacity ratio for shear at bottom < 1.0 acceptable

$$IR_Momps := \frac{Momps}{\phi Mn_SI} = 0.4$$

demand over capacity ratio for moment at about mid height < 1.0 acceptable

$$IR_Momng := \frac{|Momng|}{\phi Mn_SO} = 0.77$$

demand over capacity ratio for moment at about mid height < 1.0 acceptable