Structural Integrity of Single Shell Tanks at Hanford – 9491

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ABSTRACT

The 149 Single Shell Tanks at the Hanford Site were constructed between the 1940's and the 1960's. Many of the tanks are either known or suspected to have leaked in the past. While the free liquids have been removed from the tanks, they still contain significant waste volumes. Recently, the tank farm operations contractor established a Single Shell Tank Integrity Program. Structural integrity is one aspect of the program. The structural analysis of the Single Shell Tanks has several challenging factors. There are several tank sizes and configurations that need to be analyzed. Tank capacities range from fifty-five thousand gallons to one million gallons. The smallest tank type is approximately twenty feet in diameter, and the three other tank types are all seventy-five feet in diameter. Within each tank type there are varying concrete strengths, types of steel, tank floor arrangements, in-tank hardware, riser sizes and locations, and other appurtenances that need to be addressed. Furthermore, soil properties vary throughout the tank farms. The Pacific Northwest National Laboratory has been conducting preliminary structural analyses of the various single shell tank types to address these parameters. The preliminary analyses will assess which aspects of the tanks will require further detailed analysis. Evaluation criteria to which the tanks will be analyzed are also being developed for the Single Shell Tank Integrity Program. This information will be reviewed by the Single Shell Tank Integrity Expert Panel that has been formed to issue recommendations to the DOE and to the tank farm operations contractor regarding Single Shell Tank Integrity. This paper provides a summary of the preliminary analysis of the single shell tanks, a summary of the recommendations for the detailed analyses, and the proposed evaluation criteria by which the tanks will be judged.

INTRODUCTION/BACKGROUND

Single shell tanks (SSTs) are underground nuclear waste storage tanks having a single liner of carbon steel housed within a cylindrical reinforced concrete structure. A total of 149 underground SSTs were constructed during the years 1943 through 1964. These SSTs are divided into 12 separate groups (based on their location) referred to as tank farms. The twelve tank farms are identified as A, AX, B, BX, BY, and C in the 200 East Area and S, SX, T, TX, TY, and U in the 200 West Area of the Hanford Site near Richland, Washington. There are 133 large capacity (530,000, 758,000 and 1,000,000 gallons) 100-Series tanks with a 75-foot internal diameter and 16 small capacity (55000 gal) 200-Series tanks with a 20-foot internal diameter. Figure 1 shows four schematics of the 100 and 200-Series SST configurations. The three larger tanks are the 100-Series, and the smaller tank is the 200-Series in the figure. Tank geometry details, construction drawings and specifications are mentioned in greater detail in Han [1].
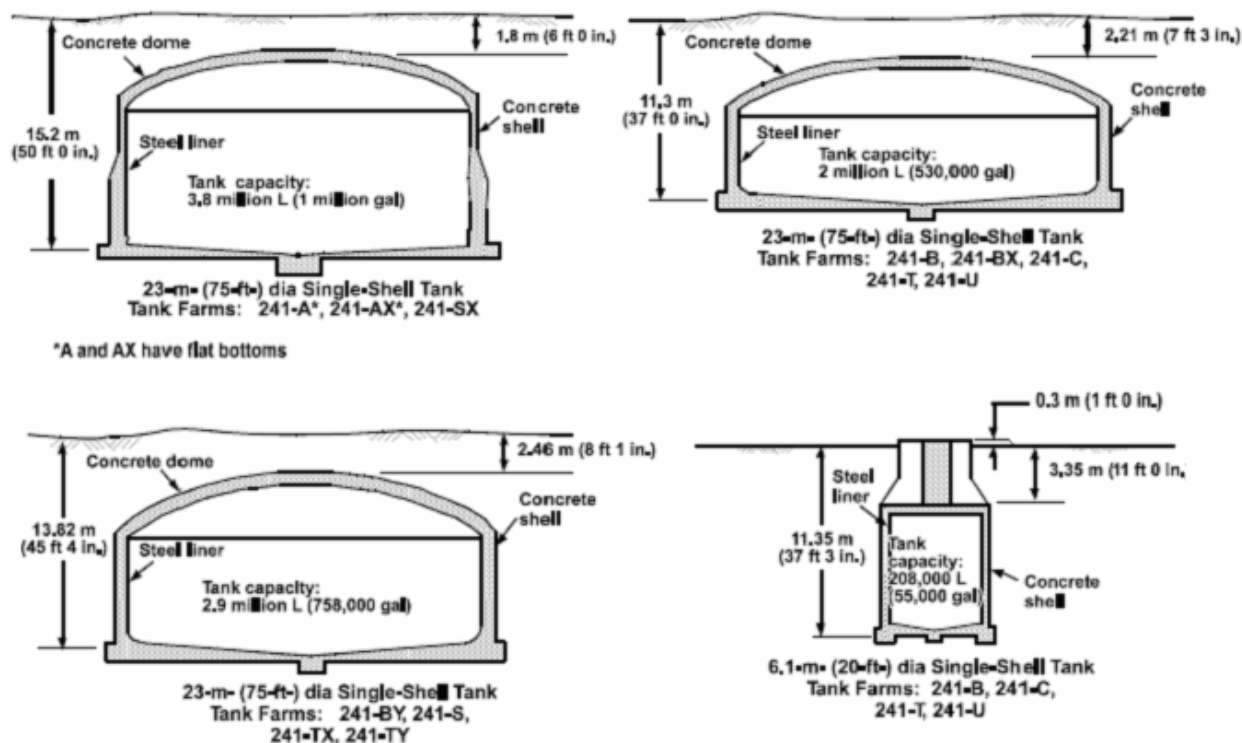
Figure 1: 100- and 200-Series Single-Shell Tank Configurations.

Julyk [2] summarized the construction date, steel liner material, and nominal wall thicknesses for each of the SST designs. The 100-Series steel liners are circumferentially stiffened by 5 x 3-1/2 x 5/16-inch angle rings welded to the inner surface of the vertical cylindrical portion of the liner. The 200-Series steel liner has only one stiffener ring located at the top of the liner. Lead flashing is provided at the top stiffener ring of all the SST liners (except SX and A) to prevent waste condensate from running down between the liner and the concrete wall. The long leg of the top stiffener is sloped, and the lead flashing is wrapped over the stiffener and anchored to the concrete above the stiffener. In the case of the A and SX liners, a formed metal ring is welded to the top of the liner and anchored to the concrete. A one-inch thick layer of cement mortar reinforced with 2x2-in. wire mesh fabric over 3-ply asphaltic membrane waterproofing is provided between the steel liner and the concrete wall of almost all SSTs. Out of 149 SSTs, 67 tanks (45%) have leaked or are assumed to have leaked. Leakage of the liners is postulated to have been caused by nitrate-assisted stress corrosion cracking (SCC), which has been confirmed in similar waste storage tanks at the Savannah River Site (SRS). Continued safe use of these nuclear waste storage tanks is necessary until the tanks are cleaned and decommissioned by the US DOE sometime in the future. The safety of these aging structures during waste remediation campaigns hinges on our understanding of the current structural integrity of these tanks.

ANALYSIS

In order to preliminarily assess the integrity of the SSTs, the finite element analysis (FEA) of tanks is carried out using the commercial FEA software, ANSYS®. Two types of tanks, Type I (55,000 gal; 200-Series) and Type II (530,000 gal; 100 Series) SSTs were chosen for the preliminary analysis covering both 20 ft and 75 ft diameter tanks. The preliminary analyses use two-dimensional (2-D) axi-symmetric models of the tank cross-sections since both the major structural details and the tank loads are generally symmetric about the cylindrical tank geometry. A static, steady state analysis is carried out to simulate

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® ANSYS Version 11.0 General Purpose Finite Element Program, ANSYS Inc., Canonsburg, Pennsylvania

the stresses due to various loads on the tank. Additional seismic analyses will be conducted at a later time.

The structural finite element models of the Type I and II tanks are shown in Figure 2. These two tanks were selected to include the two different dome geometries, and they were also the most complete and detailed data sets (the geometric models, construction and engineering drawings, and design specifications) reported in the literature.
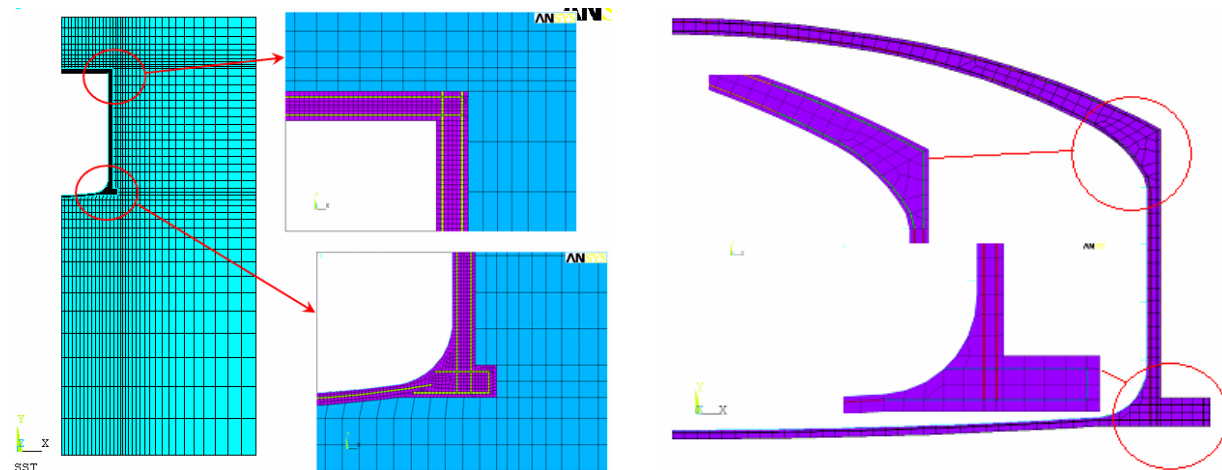

Figure 2: Finite element mesh of Type-I (left) and Type-II (right) concrete tanks with smeared rebar layers.

The 200-Series, Type I tanks are cylindrical, reinforced concrete structures with a flat dome. The Type I tank modeled is approximately 22 ft in diameter and nearly 27 ft tall when over the dome attachments are neglected. The maximum estimated soil cover for these tanks is 12 ft. The 100-Series,Type II tanks are also cylindrical, reinforced concrete structures capped by a curved dome. This tank design is approximately 77 ft in diameter and 33 ft tall. The maximum estimated soil cover (burial depth) for these tanks is 9.25 ft. Both types of tanks are modeled as axi-symmetric structures with axi-symmetric loads. For each tank the concrete and rebar are modeled with 4 node axi-symmetric plane elements (ANSYS element type PLANE42). This element is not able to directly simulate concrete crushing, cracking, or tensile softening behavior. However, the softening effect of concrete crushing and cracking was approximated using the anisotropic (ANISO) elastic-plastic material behavior. The steel reinforcing bar (rebar) in either tank is modeled as a layer of composite material with material properties derived from the volume fractions of steel (corresponding to actual rebar) and concrete in a slab of thickness equal to the rebar diameter. The volume fraction of steel takes into account both meridional and circumferential rebars. Since the number and size of rebars vary depending on their location in the tank (dome, haunch, wall, footing, and slab), the composite rebar layer material properties are varied accordingly.

The steel liner, which is not structurally connected to the concrete tank, is modeled with 2 node axi-symmetric shell elements (SHELL 208) with various thicknesses in the slab (3/8"), knuckle (5/16") and wall (1/4") regions. A corrosion allowance of 1-mil per year (for 60 years) is deducted from the thicknesses of liner sections to account for the corrosion that may have occurred during the life of the tanks. The angled stiffeners that are welded to the inside of the Type II tanks are modeled with axi-symmetric shell elements (SHELL208). The interfaces between the steel liner and the concrete are modeled as contact surfaces with the friction coefficients shown in Table 1. Plane axi-symmetric elements (PLANE42) are used to model the soil surrounding each of the tanks. The contact interfaces between the soil and the concrete tank are modeled with different material properties that account for the variable friction coefficients along the dome, wall, and slab. The friction coefficients used at the soil-concrete

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interfaces are also summarized in Table 1. A very low coefficient of friction is applied at the vertical side wall so that application of the gravity load does not cause unnatural drag-down forces that would not have occurred during the incremental placement of the soil surrounding the tanks.

Table 1: Coefficients of Friction Used in the SST Structural Analysis

Material Interface Description		Coefficient of Friction
Soil-to-Concrete:	Dome	0.3
	Side Walls	0.05
	Base Mat	0.6
Concrete-to-Steel (concrete cast against steel)		0.4
Concrete-to-Steel (slab concrete-to-steel liner)		0.3

The material properties listed in Julyk [2] were used in this analysis. The maximum waste levels and the maximum temperatures that the tanks have experienced were obtained from the design limits and operational history data available in the literature [3]. Temperature dependent material properties were included in the models to account for the degrading effects of the tank concrete being exposed to high temperatures from radioactive decay heating. The tank dead load, soil over burden, and hydrostatic waste loads were applied with the appropriate boundary conditions. To eliminate any influence of the soil remote boundary conditions on the tank, the outer radius of the soil is taken as three times the outside radius of the tank wall for both the Type I and II tanks. A concentrated live load of 200,000 lb (representing a crane or other heavy equipment on the ground) was also applied as a distributed load over a 10 ft radius on the soil surface directly above the dome center.

EVALUATION CRITERIA

The reinforced concrete structure and the steel tank liner were evaluated using criteria specific to these structural elements. First, the ACI-349 code for nuclear safety related concrete structures [4] was used to evaluate the reinforced concrete structure. Second, the requirements of Article CC-3000 of the ASME Boiler & Pressure Vessel Code Section III, Division 2, Subsection CC [5] were used to evaluate the steel liner backed by concrete. These are the same criteria used to evaluate similar components of the Hanford double shell tanks (DSTs) [6]. In the preliminary analysis, the tanks were evaluated for only the normal service loads. Both normal and abnormal loads (such as seismic motion) will be addressed in the subsequent detailed analyses.

RESULTS

The tank deformations, post-yield concrete softening (concrete cracking), tank stresses, soil stresses, and steel liner stresses were calculated for both the Type I and Type II tanks. The results of the Type II tank analysis were found to be in good agreement with the previous structural analysis of the C-106 tank, which is also a Type II single shell tank [7]. Figure 3(a) shows a comparison of the deformed shape and the crack patterns from the preliminary Type II tank model and the C-106 tank analysis. Figure 3(b) shows that the approximate method of using anisotropic yielding to simulate concrete cracking gave post-yield softening behavior that was similar to the cracking predicted in the C-106 analysis. In addition, Figure 4 shows that the trends in the stresses and displacements of the Type I tank results are similar to the Type II analysis results.
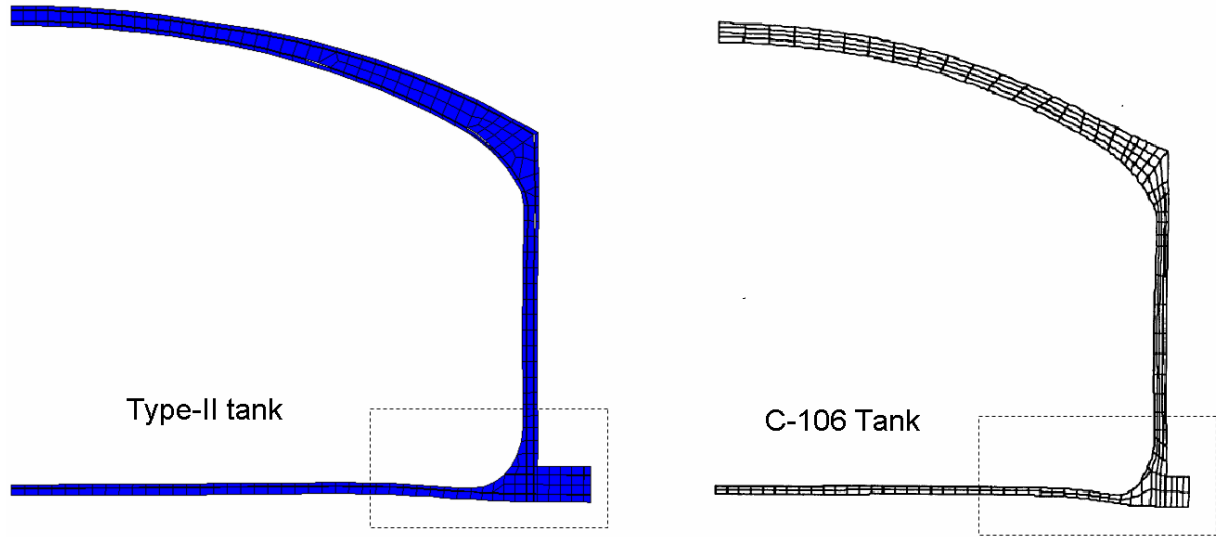
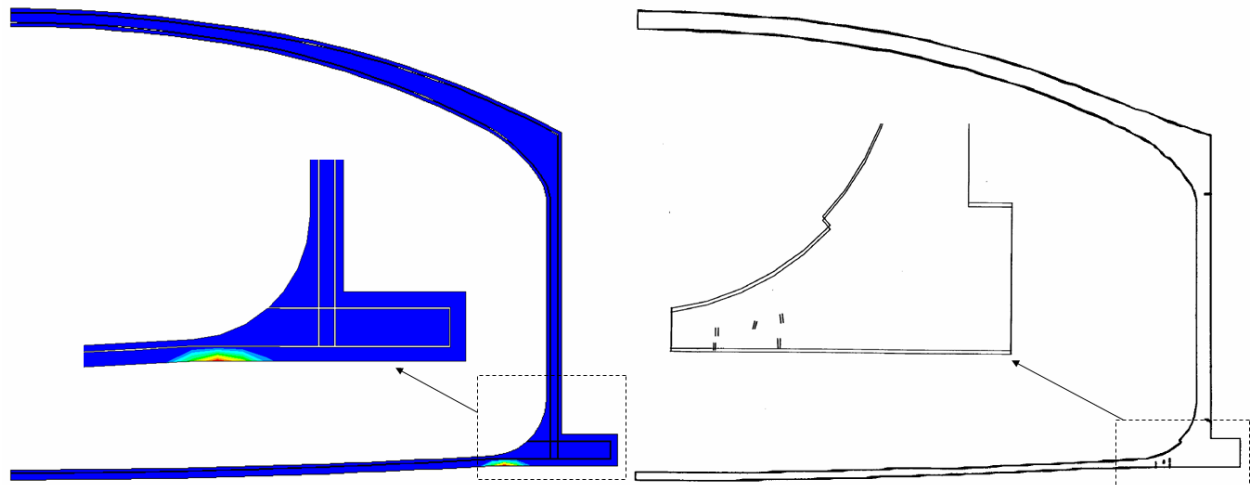Figure (a) Displaced shape of Type II and C-106 Tank

Figure (b) Crack pattern of Type II and C-106 Tank

Figure 3: Tank deformation and crack pattern comparisons of Type-II tank and C-106 Tank

The SST concrete structures were evaluated to the ACI-349 code standards following the procedure described in the DST analysis report [6]. The method involves comparing the predicted net section moment, normal force, and shear force (i.e., the structural "demands") with the allowable values (the structural "capacities") calculated using the code-based methods and the reinforced section geometry and the specified rebar and concrete strengths at the operating temperature. No thermal stresses were included in the current preliminary analysis. However, thermally degraded strength and stiffness properties were included in the models to account for the high temperature operating history of these tanks. A total of nine concrete sections were chosen beginning at the center of the dome and traversing through the haunch, down the wall, and back toward the center of the slab. Figures 5(a) and 5(b) show the locations of these sections including the section meridional, circumferential, and shear demand/capacity (D/C) ratios for the Type I and Type II tanks, respectively. The demand/capacity ratio is defined as the ratio of the vector length from the origin to the force-moment demand coordinate to the vector length from the origin to the force-moment capacity curve assuming the same ratio of force to moment. A demand/capacity ratio exceeding 1.0 indicates failure to meet the ACI requirements. It is evident from

the preliminary results presented in figure 5(b) that the meridional demands at the footing region exceeded 1.0 for the Type II tank. This indicates the possibility of some concrete cracking at the footing region for the empty tank condition. This same result was also noted in the C-106 tank analysis. Both analyses also show that the steel liner (the waste containment boundary) is subjected to very small stresses at this location.
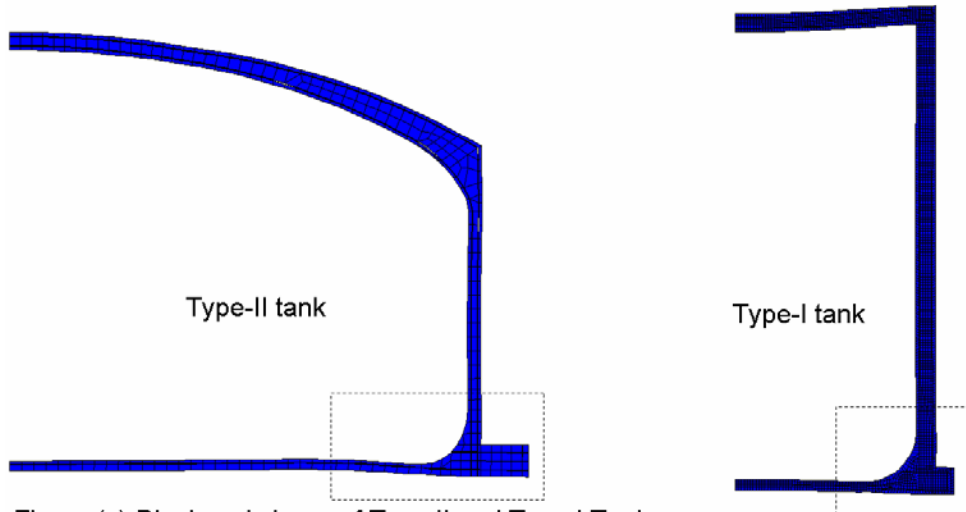

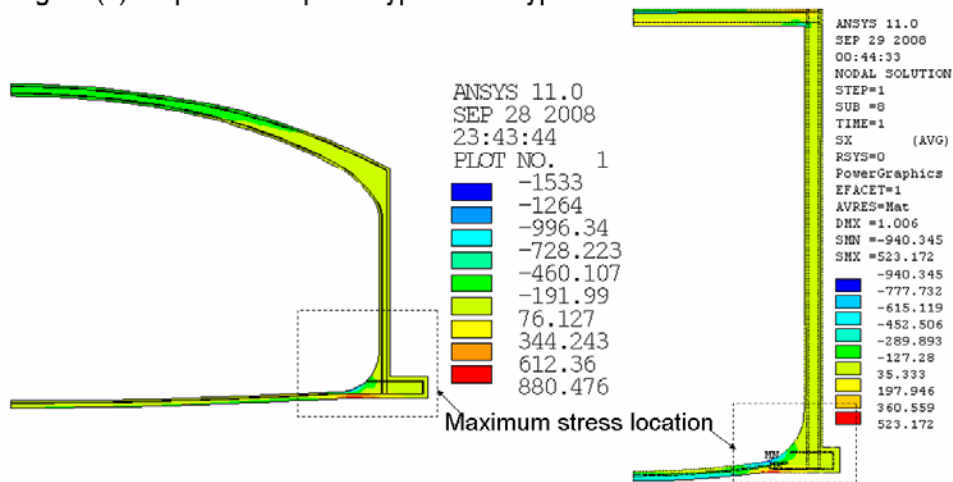Figure (a) Displaced shape of Type II and Type I Tank

Figure (b) Stress Contours (S$_x$) for Type II and Type I Tank with no waste (empty tank)

Figure 4: Comparison of the deformations and stresses from the Type 1 and II single shell tank models.
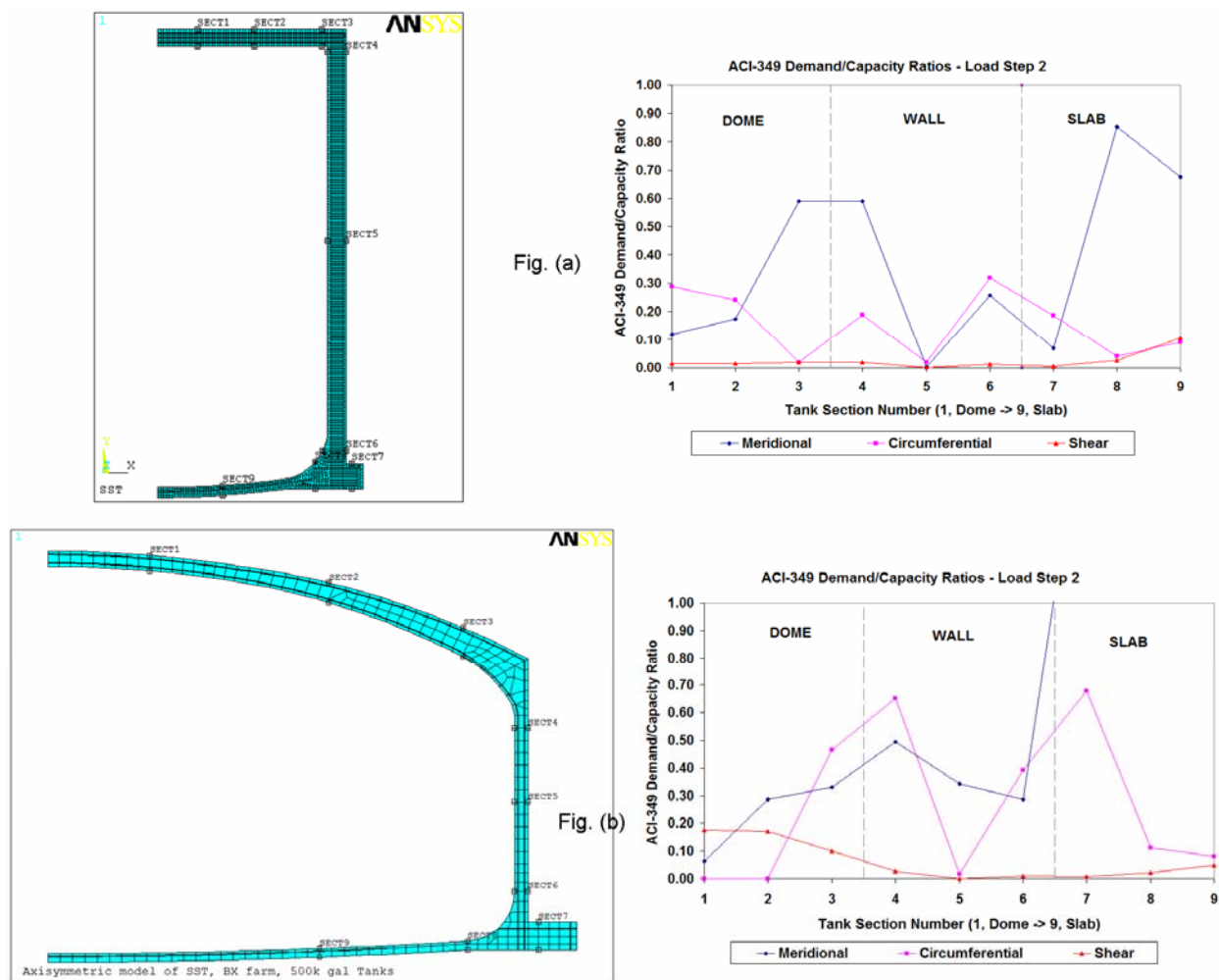
Figure 5: Tank Sections and D/C ratio plots for ACI evaluation (a) Type I tank (b) Type II tank

The steel liner was evaluated using the strain-based criteria in Article CC-3000 of the ASME code, Section III, Division II [5] for steel liners backed by concrete. This criteria was also recommended by Day et al [8] for evaluating the secondary liner of the Hanford double shell tanks. The steel liner allowable strains are shown in Table 2. The finite element analyses of both the Type I and II tanks predicted that the stresses in the steel liner were very small compared to the yield strength of material. Therefore, the accompanying strains were also well below the allowable limits.

CONCLUSIONS

The simplified finite element models used in this preliminary analysis are adequate to predict the general deformation trends and the locations of maximum stresses in the tank. Figure 3 shows that the deformed shape obtained for the Type II tank is in agreement with the deformed tank shape plotted in the C-106 report.

Table 2: Liner allowables

Strain Type	Strain Allowable (in/in)
Service Load Category	
Membrane	$\varepsilon_{sc} = 0.002$
	$\varepsilon_{st} = 0.002$
Membrane plus Bending	$\varepsilon_{sc} = 0.004$
	$\varepsilon_{st} = 0.004$
Factored Load Category	
Membrane	$\varepsilon_{sc} = 0.005$
	$\varepsilon_{st} = 0.003$
Membrane plus Bending	$\varepsilon_{sc} = 0.014$
	$\varepsilon_{st} = 0.010$
Where: ε_{sc} - Compressive Strain	
ε_{st} - Tensile Strain	

The anisotropic material model with different compressive and tensile yields used in the analysis is able to predict the locations of initial crack formation in the reinforced concrete. Realistically, once the cracks form in concrete sections, the tensile stresses will be relieved in the concrete and the load will be taken by reinforcement. This process may reduce further formation of cracks in the vicinity of existing cracks. However, this kind of behavior cannot be simulated with the material model used in this analysis, which may result in somewhat higher tensile stresses as seen in the radial stresses. The steel liner stresses are very small compared to the yield strength of material. This provides evidence that the presence of the liner does not affect the concrete stresses or the code evaluation of the concrete tank. The ASME code evaluations for the liner indicated sufficient margins for the liner strains.

The tensile stresses generated at the footing region in both types of tanks under gravity loads (self weight and soil over burden) indicated that an empty tank is the worst case for footing failure. The additional hydrostatic loads due to the waste tend to relieve the tensile stresses in the footing region. This result was also noted in the C-106 tank analysis.

RECOMMENDATIONS FOR FURTHER DETAILED ANALYSIS

The preliminary analysis assumed that the tanks are axi-symmetric structures with axi-symmetric loads. However, this may not be true if the loads on the dome due to attachments (risers and other concrete structures) are considerable. A tank-specific analysis may be necessary for those cases. Although the simplified concrete material model used in this preliminary analysis is adequate to predict the general trend of deformations and cracking locations in the tank, a more rigorous 3-D slice model using the ANSYS 3-D concrete elements would provide a more realistic representation of the concrete material behavior. This model must also incorporate tension response and cracking, compressive behavior and crushing, and thermal aging effects on property degradation and creep.

More comprehensive material models are also required for the soil to predict more realistic deformations and stresses. The soil model should account for variation in properties such as density to be consistent

with the back fill properties and to improve soil-structure interactions. In addition, detailed thermal models are also needed to predict realistic temperature profiles in the concrete and the surrounding soil in order to accurately predict the current state of the tanks.

The buckling stability of these tanks should also be addressed considering the slight vacuum maintained in the vapor space of some tanks plus the additional concentrated loads above the buried tanks that may be due to retrieval activities. A seismic analysis is also necessary to include the dynamic soil structure interaction loads that may occur during the long-term cleanup and decommissioning campaign.

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