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7. Abstract

Accelerated Safety Analyses - Phase I (ASA-Phase I) have been conducted to assess the appropriateness of existing tank farm operational controls and/or limits as now stipulated in the Operational Safety Requirements (OSRs) and Operating Specification Documents, and to establish a technical basis for the waste tank operating safety envelope. Structural sensitivity analyses were performed to assess the response of the different waste tank configurations to variations in loading conditions, uncertainties in loading parameters, and uncertainties in material characteristics. Extensive documentation of the sensitivity analyses conducted and results obtained are provided in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analyses - Phase I*. This document provides a summary of the accelerated safety analyses sensitivity evaluations and the resulting findings.

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ACCELERATED SAFETY ANALYSES
STRUCTURAL ANALYSES PHASE I
STRUCTURAL SENSITIVITY EVALUATIONS OF
SINGLE- AND DOUBLE-SHELL WASTE STORAGE TANKS

November 1994

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GENERAL PEER REVIEW CHECKLIST

Document Reviewed: WHC-SD-WM-SARR-012, Rev 1 - Accelerated Safety Analysis - Structural Analysis Phase I - Structural Sensitivity Evaluations of Single- and Double-Shell Waste Storage Tanks

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Scope of Review: This SA&E review is being performed in addition to the Central Engineering peer review. The purpose of this additional review is for consistency with other SARR Topical reports. It should be noted however that this report does not contain any hazard or accident analysis. The scope of this report is limited to structural analysis. The scope of this peer review is limited to ensuring that those safety peer review checklist items that are applicable are adequately addressed. The scope of revision one is editorial in a nature.

Yes No^{*} NA

☐	☐	☒	Accident scenarios explicitly stated and supported.
☒	☐	☐	Conclusions consistent with analytical results.
☐	☐	☒	Safety issues adequately defined.
☐	☐	☒	Hazardous material inventories defined and/or stated.
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GENERAL PEER REVIEW CHECKLIST

Yes No^{*} NA

☐ ☐ ☒	Hazardous systems, components, or materials consistently emphasized and stated throughout the document.
☒ ☐ ☐	Appendices adequately support the document.
☒ ☐ ☐	References adequately support the document.
☒ ☐ ☐	All qualitative and quantitative analyses peer reviewed and approved.
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Comments:

L.F. Dougherty *L.F. Dougherty*
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11/23/94
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TYPICAL CHECKLIST FOR INDEPENDENT REVIEW

Document Reviewed ACCELERATED SAFETY ANALYSIS - STRUCTURAL ANALYSES
PHASE I

Author D. L. BECKER

Yes No N/A

☒	☐	☐	Problem completely defined.
☒	☐	☐	Necessary assumptions explicitly stated and supported.
☐	☐	☒	Computer codes and data files documented.
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☒	☐	☐	Data checked for consistency with original source information as applicable.
☐	☐	☒	Mathematical derivations checked including dimensional consistency of results.
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☐	☐	☒	Hand calculations checked for errors.
☐	☐	☒	Code run streams correct and consistent with analysis documentation.
☐	☐	☒	Code output consistent with input and with results reported in analysis documentation.
☐	☐	☒	Acceptability limits on analytical results applicable and supported. Limits checked against sources.
☐	☐	☒	Safety margins consistent with good engineering practices.
☒	☐	☐	Conclusions consistent with analytical results and applicable limits.
☒	☐	☐	Results and conclusions address all points required in the problem statement.

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ACCELERATED SAFETY ANALYSES
STRUCTURAL ANALYSES PHASE I
STRUCTURAL SENSITIVITY EVALUATIONS OF
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1.0 INTRODUCTION

1.1 GENERAL STRUCTURAL ANALYSIS APPROACH

The overall plan for the structural analysis of waste storage tanks consists of various logical phases that are generic to any standard structural analysis approach. First, analysis models for normal and operating loads must be developed. The output of these models is dependent on the accuracy of the data incorporated into the analyses and the assumptions. The next logical step is to perform sensitivity analyses. Sensitivity analyses can be used to identify the changes in stress as a result of variations in loading conditions, uncertainties in loading parameters, and uncertainties in material characteristics. Loading conditions include the expected normal and operating load range as well as the range of assumptions used to calculate the loads (e.g., soil density). Although the purpose of the sensitivity analyses is not intended to determine the total combined stress in the tank or typically account for degraded conditions, they do provide valuable insight into what assumptions and variations the models are sensitive to, and also provide insight into the sensitivity of the tanks/models to various loads.

Once sensitivity analyses are completed, the assumptions that the models are sensitive to can be verified and used as feedback to refine the model. This helps ensure that the model is realistic and defensible. After the models are refined they can be used to perform qualifying analyses for the tanks.

To qualify the tanks for normal and operating loads, the bounding normal and operating loads (not accident loads) are incorporated into the analysis process to determine the maximum combined stresses in the tank. Degraded material properties and structural elements are accounted for in the analysis process. The maximum stresses are then normally compared to appropriate acceptance criteria (e.g., American Society of Mechanical Engineers Boiler and Pressure Vessel Code or American Concrete Institute based allowable stress criteria) to ensure that adequate factors of safety exist in the tanks.

Once the tanks are qualified to the specified maximum normal and operating loads, accident analysis can be performed. Seismic accident analysis will use these same normal and operating loads but with different analysis techniques, which include seismic loads, to calculate the stresses and stress sensitivities. In addition, different acceptance criteria are used to determine if the stresses due to an earthquake are acceptable while the maximum normal and operating load conditions are imposed.

An assessment of other accident loads can be postulated by linearly extrapolating then Accelerated Safety Analyses - Phase I (ASA-Phase I) stress

results for load values above the normal and operating limits, and comparing the resultant stresses to accident loading acceptance criteria. However, because the behavior of the tank structure under combined loading conditions is nonlinear, and the accident loads are typically considerably higher than the normal and operating loads, a linear extrapolation of the stress results for accident conditions may introduce inaccuracies into the predicted results. Even though some inaccuracies may be introduced into the accident analyses predictions, it should be possible to determine at what load level and in what area of the tank failure will most likely occur. The results of these accident predictions, combined with the seismic and sensitivity analyses, can be used as a more defensible basis for establishing operating safety requirements and technical safety requirements (OSRs/TSRs).

In terms of these normal phases of structural analysis, this topical is limited to the development of the models and completion of the sensitivity analyses for a range of normal and operating loads. Confirmation of the sensitivity analysis assumptions, additional model development, qualifying analyses and accident analyses are part of the next phase(s) of analysis and are not addressed by this topical.

The purpose of this topical is to support the Accelerated Safety Analysis (ASA) by ensuring that the current safety envelope, as defined by the *Hanford Site Tank Farm Facility Interim Safety Basis (ISB)* (Leach 1993), has appropriate controls for normal and operating loads. The three main tasks of this ASA-Phase I effort were to:

- 1) Perform analyses and evaluations necessary to arrive at controls, limits and restrictions on day-to-day operations (which exclude accident events) aimed at the following recommended ISB safety objective (Leach 1993):

"Reduce or curtail the risk of limiting the intended/desired life and function of the tank farm facilities." (For example, use the sensitivity analysis to determine where controls are necessary to maintain tank integrity under normal and operating load conditions).

- 2) Develop the methodology and set up the tools necessary to analyze some of the more benign accident loads (excluding seismic events) and support USQ evaluations (i.e., develop models that can be refined and used to complete the structural analyses).
- 3) Provide sensitivity analyses results that can be used to assess the appropriateness of the current operating controls, limits and restrictions delineated or referenced in the ISB that relate to material and structural integrity of the waste tanks, as well as the potential need for implementing new or modified controls and limits.

This ASA-Phase I effort addresses the loads that are either considered part of normal operations or that need to be accounted for to re-evaluate the necessity and effectiveness of existing controls, limits and restrictions. A tabulation of these loads and the range of load values considered are given in Table 1. Certain specific loads that may be beyond the original design bases for the tanks are being identified and addressed in other ASA topical reports.

Loads not addressed in this ASA-Phase I structural analysis task, are being addressed in separate parallel tasks, or have been scheduled for completion after Phase I of the ASA. This topical report does not include accident analysis, including seismic analysis, nor the justification for deletion of existing IOSRs but rather an assessment that the tank is not sensitive to loads that are not already covered by IOSRs.

1.2 ACCELERATED SAFETY ANALYSES - PHASE I

Accelerated Safety Analyses - Phase I have been conducted to assess the appropriateness of existing tank farm operational controls and/or limits as now stipulated in the Operational Safety Requirements (OSR) (Aguirre 1994, Dougherty 1994 and Huebach 1994) and Operating Specification Documents (OSD) (OSD 1989, OSD 1992a and OSD 1992b) and to establish a technical basis for the waste tank operating safety envelope. Structural sensitivity analyses were performed to assess the response of the different waste tank configurations to variations in loading conditions, uncertainties in loading parameters, and uncertainties in material characteristics. Extensive documentation of the sensitivity analyses conducted and results obtained are provided in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analyses-Phase I* (Fisher et al. 1994). This document is a summary of the accelerated safety analyses sensitivity evaluations and the resulting findings (Fisher et al. 1994). For a thorough discussion of the ASA-Phase I analyses performed, and a reporting of the results obtained, the reader should refer to Fisher et al. 1994.

The ASA-Phase I effort addressed all existing Hanford Site double- and single-shell underground radioactive waste storage tanks (DST and SST), except for the 200 series tanks which have a storage capacity of 55,000 gal, for the types of loadings experienced during normal operations. Hence, no consideration was given within the ASA-Phase I effort to address postulated accident loadings.

The evaluation of combined load cases and comparison of the results to an acceptance criteria and the evaluation of postulated accident loadings are to be performed under follow-on work phases.

2.0 BACKGROUND

The *Hanford Site Tank Farm Facilities Interim Safety Basis (ISB)* (Leach 1993) provides a reference to the tank farm safety envelope for applicable tank farm facilities and installations. The ISB is intended to facilitate understanding of the safety envelope and its corresponding justification until formal safety analysis report upgrades can be completed. The safety envelope includes the technical basis for safe operation and maintenance of tank farm facilities, equipment, and processes.

During preparation of the ISB, tank structural integrity concerns were identified as a key element in defining the safety envelope. Hence, activities were conducted, as an integral effort for development of the ISB,

to assess adequacy of the technical bases that substantiate the elements of the safety envelope associated with structural integrity. The results of these activities are documented in Appendix E of the ISB.

The ISB studies identified several deficiencies in the technical bases associated with the structural integrity issues and the corresponding operational limits/controls specified for conduct of normal tank farm operations. Consequently, a plan was initiated to upgrade the waste tank safety envelope technical bases.

In consonance with this plan, ASA-Phase I evaluations were performed to assess the appropriateness of the current OSR and OSD controls/limits and the effects of variations in the operating loads on waste tank performance. The results of these analyses fulfill the following objectives.

- Provide the technical bases for recommendations to upgrade the OSRs and OSDs
- Provide detailed analytical models for conducting follow-on structural analyses activities
- Provide information to identify specific combinations of loading conditions (temperature, thermal gradients, etc.) to be addressed by future safety-related structural analyses efforts

3.0 SUMMARY

One of the primary objectives of the ASA-Phase I objectives was to develop analytical models for the waste tanks that could be used for conducting follow-on analyses activities. This goal has been achieved. The detailed analytical models of the waste tanks are now available and can be used for performing additional generic analyses of the SSTs and DSTs, or they can easily be modified to perform tank-specific analyses. The models have been developed using state-of-the-art modeling techniques, and flexibility has been incorporated into the loading profiles to easily accommodate alternative loading profiles. The electronic files can be modified as necessary using the same or similar state-of-the-art modeling techniques and the I-DEAS (SDRC 1993) modeling software package. The models and all related data formulate a database that has been thoroughly checked for consistency with the design drawings and specifications and should be established as a baseline database for performing additional waste tank analyses. Accordingly, Facility Stress Analysis now has a team of seasoned and experienced analysts that are familiar with use of the models and can efficiently conduct follow-on waste tanks analyses activities.

Potential areas for application of the tank models for performing additional analyses include support of tank farm operations, tank structural integrity assessment studies, and development of life management plans. For example, an evaluation of tank behavior with predamaged sections could be carried out; sections with through cracks to simulate leakage zones, rebar corrosion, and construction joints could be investigated; and possible damage of refractory concrete could be evaluated for its effect on the primary tank

stresses. In addition, three-dimensional tank models could be developed for investigating seismic, asymmetrical loads, and asymmetrical geometric features (e.g., risers and dome penetrations). The existing tank models can be easily revolved to create three-dimensional half-tank models or full-tank models.

Because the ASA-Phase I effort consisted of performing sensitivity evaluations and was not intended to provide the detailed analytical results required to structurally qualify the waste tanks, sufficient data was not developed from the ASA-Phase I evaluations to technically justify modification of the current OSR and OSD controls or limits. However, based on the sensitivity analyses results, the following observations are noted.

- Controls are in place for the maximum allowable liquid level and specific gravity. Control of these two parameters will also control hydrostatic head. Control of all three parameters makes control of one of the parameters redundant.
- Some of the controls now in place are somewhat complicated (i.e., the control on vapor space pressure and annulus pressure). Through coordination with tank farm operations personnel these controls could be simplified.
- Several of the surveillance frequencies show inconsistencies in monitoring tank control parameters. Through coordination of tank farm operations, a review of the surveillance frequencies could be conducted to establish more efficient and consistent processes.

Although the ASA-Phase I effort does not provide the detailed analytical results required to structurally qualify the waste tanks, the sensitivity study results do provide a valuable tool for predicting the structural response of the waste tanks to variations of the specific loading values or combinations thereof. Through examination of the sensitivity study results, and performing linear extrapolation or interpolation of the calculated values, a prediction of the effects of variations in the normal operating loading conditions, either within or outside of the currently defined operating controls/limits, can be quickly obtained. This capability can be used to support day-to-day operations of the tank farm facilities or to obtain an expedient assessment of an abnormal loading condition. In addition, the sensitivity study results can be used to determine the worst-case load combination for any specific area of the tank and/or for each individual structural element (e.g., concrete, reinforcing steel, primary tank steel, secondary tank liner steel). Since the sensitivity evaluations are based on nonlinear analyses, it must be recognized that the predictions made by application of linear analysis techniques may be different than the results obtained by performing a detailed nonlinear analysis.

Tank component stress sensitivity findings are summarized in Sections 9.1.1 through 9.1.4. The concrete compressive stresses are below the lowest tank farms design strength of 3,000 lbf/in². The worst-case compressive stresses are below the thermally degraded strength values and crushing does not occur. Maximum load case combinations can be developed from the sensitivity results and maximum load case stress results can be estimated using equations developed from the sensitivity results. These estimates should then be verified with actual finite-element solutions.

For some load cases the tank foundation shows significant damage through the section and high reinforcing steel stresses. This is especially true for the 500,000-gal tank. These regions should be reviewed for modeling enhancements and mesh density requirements. Regions of high stress gradient should have higher mesh densities so that stress distribution can be more accurately predicted. In addition, the 500,000-gal tank foundation should be evaluated for shear-induced section failures. The resulting crack width and leakage potential could be estimated.

The soil stiffness greatly affects the foundation stress results. Stiff soils result in very little or no foundation cracking while flexible soils result in significant cracking through the section. The foundation cracking is driven by the differential settlement between the floor portion and the outside footing portion of the tank base, which are subjected to very different stress levels. The worst cracking occurs when the tank is empty as this results in a minimum load applied to the tank floor and maximum pressure difference between the inside floor area and outer footing area. The soil weight and external loads are primarily carried by the outer portion of the tank footing. When the tank is filled with waste, the cracks tend to close as the tank now has additional load on the floor region. Whether the tank floor cracks will close enough to prevent or reduce further leaking is an unanswered question. Accordingly, whether the cracks will reopen when the hydrostatic load is reduced is also an unknown. Additional analyses and modeling enhancements are recommended to further assess the behavior of the tanks in the footing/floor region. Alternatives include: (1) developing detailed soil models; (2) performing foundation interaction studies, and (3) verifying the simplified soil stiffness models for use with detailed tank models.

Three areas of special interest were included in the ASA-Phase I evaluations: the effects of through-wall thermal gradients on the reinforced concrete sections, the effects of concrete creep on waste tank behavior, and corrosion. Each of these subjects was investigated as a separate task, and the results of the investigations are presented and summarized in Sections 9.2, 9.3, and 10.1.5, respectively. The effects of these concerns were not factored into the stress sensitivity calculations. Recommendations resulting from the ASA-Phase I evaluations are provided in Section 10.0 Recommendations.

4.0 DESCRIPTION OF TANKS

Five basic tank configurations have been utilized in the waste storage program conducted at the Hanford Site. Four of these designs are SSTs which represent the oldest tank design and system of buried waste storage tanks. The fifth standard configuration is represented by the DSTs. Typical tank geometries of the SSTs and DSTs evaluated by the ASA-Phase I effort are shown in Figure 1. The 55,000-gal tank configuration (200 series tanks) were not evaluated and are not illustrated in Figure 1.

The tanks are located in seventeen separate groupings referred to as tank farms. These SST and DST farms are located on the Hanford Site in the 200 East and 200 West Areas. These facilities are operated by the Westinghouse Hanford Company (WHC) for the U.S. Department of Energy (DOE).

4.1 SINGLE-SHELL TANKS

A total of 149 SSTs were constructed between the years 1943 and 1964. These SSTs are located in twelve separate tank farms. One hundred thirty-three tanks have an internal diameter of 75 ft and either a nominal storage capacity of 500,000, 750,000, or 1,000,000 gal of radioactive waste depending on the height of the tank wall. They are cylindrical in shape with a dome roof. The tanks, which are buried, are constructed of reinforced concrete with a single internal steel liner which extends across the tank basemat and up the cylindrical sides. Structurally, the basemat reinforcing steel is extended into the cylindrical concrete tank wall. The height of the dome above the basemat varies from approximately 30 to 43 ft according to the capacity of the tanks. The minimum soil depth at the dome crown is 5 ft. The welded carbon steel liner is independent of the reinforced concrete tank and was designed to provide containment of the stored liquid waste. Surface live loads, static and dynamic soil loads, dead loads, and hydrostatic and hydrodynamic loads are carried by the reinforced concrete tank and dome. The remaining 16 SSTs (the 200 series tanks) are of a different configuration with a smaller capacity of 55,000 gal and have been interim stabilized and interim isolated.

4.2 DOUBLE-SHELL TANKS

The twenty eight DSTs were constructed between 1968 and 1986. These tanks are located in six separate tank farms. Four of these tanks, located two each in the 241 AY and AZ tank farms have a design capacity of 1,000,000 gal. The remaining twenty-four tanks have design capacities of 1,144,000 gal. Geometrically, all of the DSTs are the same. Structurally, they are identical within any single tank farm grouping. Between tank farms, the tank designs are structurally similar with some variation in concrete and rebar strength and in rebar arrangement in the haunch region. The DSTs are 83 ft in external diameter and constructed of reinforced concrete with an internal primary tank and a secondary steel liner. These buried tanks are cylindrical in shape with a dome roof and buried. The height of the dome is approximately 47 ft above the basemat, and the minimum soil overburden is 6 1/2 ft at the crown. The welded primary carbon steel tank consists of a base which rests on an approximately 8-in.-thick layer of refractory insulating grout. The primary tank provides the containment for the stored liquid wastes. The secondary welded carbon steel liner is structurally integral with the inner surface of the cylindrical reinforced-concrete tank wall and rests on the concrete basemat. The primary tank dome is connected to the concrete dome; the primary tank floor is separated from the secondary liner floor by an 8-in. layer of insulating refractory grout. The primary tank and secondary liner are structurally independent within the tanks but interact with the concrete.

The primary steel tank provides containment for the stored liquid waste and the secondary steel liner/reinforced concrete tank establishes a redundant leakage barrier. There is a nominal 30-in. annulus between the primary tank cylindrical wall and the steel-lined reinforced concrete tank cylindrical wall. The reinforced concrete tank and dome carry the surface live loads, the static and dynamic soil loads, and the dead loads. The primary steel tank

resists the hydrostatic and hydrodynamic liquid waste loads and the internal pressure.

5.0 ANALYSIS METHODOLOGY

The analysis approach that was used for conducting the ASA-Phase I sensitivity evaluations involved the development of four generic finite-element tank models and the application of varying loading conditions to these models using the general purpose finite-element structural analysis code, ANSYS (Swanson 1992). Generic tank models and conservative bounding assumptions were used to represent the different tank design configurations thus allowing cost effective analyses to be performed and avoiding the need to evaluate each tank independently. The ANSYS Code (Swanson 1992) was used for conducting these analyses because it provides the following capabilities:

- Excellent pre-processing
- Excellent model checking
- Excellent post-processing
- Good nonlinear concrete material modeling
- Concrete creep and temperature degradation modeling
- Electronic interface with I-DEAS (SDRC 1993) pre-processor.

Four generic finite-element models were developed to obtain representative structural models of the basic waste tank design configurations, except for the 55,000-gal tanks. A generic model of the 55,000-gal tanks was not developed; effort and resources for evaluation of these tanks were not included in the ASA-Phase I scope of work. The 55,000-gal tanks are at low temperatures, interim stabilized, robust design configurations and judged to pose relatively low safety concerns. Development of the four generic models was based on tank capacity and similarity of the structural features and attributes. Discussion of the modeling details is provided below in Section 6.0. The selection of structural features and attributes represented in the models is based on information contained in the Fisher and Shank 1994.

The tank models were analyzed for axisymmetric geometry and loading conditions. A 15-degree pie-shaped model, which is actually a three-dimensional model, was used instead of a two-dimensional axisymmetric model. Use of this model was necessary because the ANSYS concrete element is a three-dimensional element. Axisymmetric behavior was enforced on the three-dimensional pie-shaped model by employing a proper set of boundary conditions and displacement coupling constraints. The techniques used to enforce axisymmetric behavior on a three-dimensional model, though well recognized in the industry, were verified through a test case.

At the onset of the ASA-Phase I activities it was decided to separate the investigations of the sensitivity of the tanks to concrete creep from the sensitivity of the tanks to applied normal and operation loadings. This decision was based on the premise that sensitivity of the tanks to normal and operational loadings could be determined by considering these loads as static loading conditions even though the analyses accounted for the nonlinear

effects, and the response of the tanks to normal and operational loads is not significantly affected by the concrete creep.

Loading conditions were applied to each of the tank models in incremental steps. The basic loading process attempted to simulate the historical stages of tank usage. Specific loading processes and parameters were evaluated included backfilling of the soil around and over the tanks after construction, changes in soil density and characteristics, filling of the tanks with waste, changes in waste specific gravity, increase of the waste temperature, application of surface loads, and application of pressure loadings. Further discussion of loadings is provided below in Section 8.0. Specific loadings and load values to be considered were established, with cooperation from Tank Farm Safety Analysis Documentation and Tank Waste Remediation System personnel, prior to performing the finite-element analyses and are documented in Becker 1994.

5.1 ANALYSIS ASSUMPTIONS

Various assumptions have been made in the course of the analysis. These assumptions deal with issues relating to geometry, material properties, loading methodology, the processing of computer analyses to accommodate variations in load values, and other analysis variables. A summary listing of the load values and analysis variables is provided in the following table.

ASA-Phase I Sensitivity Study Range of Analysis Variables.

Sensitivity Study Variable	Single-Shell Tanks	Double-Shell Tanks
Soil Depth (at tank dome apex)	5 to 10 ft	6 to 9 ft
Soil Density	101 to 130 lbf/ft ³	101 to 130 lbf/ft ³
Rankine Coefficient	0.5	0.5 to 0.7
Subgrade Modulus	400 lbf/in/in ²	200, 300, and 497 lbf/in/in ²
Waste Specific Gravity	1.0 to 2.0	1.0 to 2.0
Waste Depth	Empty to full	Empty to full
Waste Temperature	70 to 350 °F	70 to 350 °F
Vapor Pressure	0 to +60 in. wg.	0 to +60 in.wg.
Annulus Pressure	Not Applicable	0 to -20 in. wg.
Uniform Live Load	0 to 100 lbf/ft ²	0 to 100 lbf/ft ²
Concentrated Live Load	0 to 100 tons	0 to 100 tons
Creep Time	0 years	0 to 50 years

The range of normal and operating load values to be considered in the ASA-Phase I evaluations was determined at the onset of the analyses effort, in cooperation with Tank Farm Safety Analysis Documentation and Tank Waste Remediation System personnel. These load values are documented in the load report, *Single- and Double-Shell Tank Load Report for Accelerated Safety Analyses* (Becker 1994); a listing of the load values is provided in Table 1.

Design drawing dimensions were used in developing the geometry of the models. These dimensions were selected from the various tank configuration drawings to generate a generic tank model for each tank configuration which would be conservative from a structural analysis point of view. As-built dimensions were not used.

Pre-damaged material properties or tank sections were not used. This consideration may be important as tanks with known leakage problems may have degraded section or material properties as compared to non-leaking tanks. Corrosion of rebar, if any, was not considered. Tank steel corrosion for the generic DST was not included. The tank steel for the SSTs was not modeled as it was assumed that the reinforced concrete carried all the load with no help from the steel tank. The design drawing minimum concrete strength of 3,000 lbf/in² was used for all tank analyses. The strength of concrete can increase significantly with age. Conservatively, the strength increase of concrete with age was not included in the analysis. The strength and modulus of the concrete were degraded due to thermal effects (Peterson 1994).

Soil depth was specified as a constant value measured from the apex of the tank dome. Soil density was specified as constant over the tank dome. The normal (to the tank dome) component of the soil weight was applied to the tank dome. The transverse component parallel to the tank dome was neglected. The soil above the tank dome and outside the tank diameter was assumed not to cause any additional load on the tank dome. The soil was assumed to load the tank wall in a linear pressure variation as a function of soil depth (depth multiplied by the weight density multiplied by the Rankine lateral soil load coefficient). Horizontal soil springs along the vertical tank wall were not used. A vertical soil spring was used to support the tank floor. This soil spring was assumed to be linear and constant along the floor of the tank. Tank sensitivity to soil spring (subgrade modulus) values was evaluated. The concentrated load placed at ground level is assumed to be directly transferred to the tank dome. The effect of load-intensity reduction due to soil depth was conservatively ignored.

Tank waste was assumed to have a constant distribution of density. The waste produced a linear varying hydrostatic load which varied linearly with liquid depth. Waste density stratification was not considered.

Temperature gradients along the wall of the tank were considered. The temperature of the tank below the waste level was assumed to be at the maximum waste temperature. The vapor space above the liquid waste was assumed to be at the specified dome temperature. A linear temperature gradient over a 6-ft length of wall section was imposed and provided the transition from waste temperature to dome temperature. This assumed distribution is not tank specific. Tank-specific temperature distributions depend on many factors. The simplified approach attempts to utilize temperature distributions and gradients which tend to be conservative. Temperature gradients through the

wall of the tank were evaluated as a separate task but were not factored into the stress calculations. Evaluation of stresses resulting from through-wall gradients should be addressed in future work.

Vapor-pressure load in the vapor space between the dome and liquid waste surface was considered to have a uniform distribution. The annulus pressure between the primary and secondary tanks in the DST was also considered to have a uniform distribution.

6.0 DESCRIPTION OF ANALYTICAL MODELS

The waste tanks are axisymmetric structures except for some off-center penetrations in the domes and side walls. The effects of these off-center penetrations are local in nature and secondary to the overall structural response of the tanks. The loads applied to the tanks for the ASA-Phase I evaluations are also axisymmetric. Hence, axisymmetric modeling techniques have been employed to develop the generic tank analytical models. The effects of the penetrations, which may cause local stress increases, were not quantified. However, it should be emphasized that these local stress increases could be evaluated by expanding the generic models to develop quarter- or half-size models and incorporating the necessary features to appropriately simulate the penetrations.

The tanks have steel reinforcement (rebar) of various sizes and spacings. In the 500,000-gal and 750,000-gal SSTs, reinforcing steel is placed radially and circumferentially (hoop) in each face of the footing/lower knuckle, walls, haunch, and dome (except for a 20-ft-square area centered at the dome apex). For these tanks, rebar placed in a rectangular pattern reinforces both faces of the dome center region and one face of the floor. For the 750,000-gal SSTs, rebar intersecting the penetration at the center of the dome is welded to the manhole.

In the 1,000,000-gal SSTs, reinforcing steel is placed radially and circumferentially (hoop) in each face of the footing/lower knuckle, walls, haunch, and dome (except for a 20-ft-square area centered at the apex of the dome of tanks 241-A and 241-SX). Rebar placed in a rectangular pattern reinforces both faces of the dome center region (except for the 241-AX tanks) and one face of the floor. In the 241-AX tanks the dome is reinforced on both faces with radial and circumferential steel.

In the 1,000,000-gal DSTs, reinforcing steel is placed radially and circumferentially (hoop) in each face of the floor/footing, wall, haunch, and dome. In all tanks the haunch is reinforced with an additional interior layer of hoop and radial steel at about mid-depth, with ties between the top and bottom rebars.

The finite-element models of the tanks were generated with the SDRC computer program I-DEAS (SDRC 1993). The analyses were performed using the ANSYS general purpose finite-element code (Swanson 1992). A 15-degree pie-section was modeled with axisymmetric boundary conditions to accommodate the ANSYS concrete element. The SOLID65 element was used to model the three-dimensional solid concrete tank section. These elements are defined by eight

nodes with three-degrees of translational freedom each. Rebar was not included within the element. Element SOLID65 accommodates nonlinear material properties so the concrete is capable of cracking in tension (in three orthogonal directions), crushing in compression, plastic deformation, and creep. The element was configured to allow complete or partial loss of shear transfer for open cracks while permitting full shear transfer on closed cracks.

The reinforcing steel (radial, hoop and diagonal) in the dome, haunch and wall was explicitly modeled with orthotropic-elastic shell elements (SHELL63) using membrane stiffness only. Separate elements, attached to concrete nodes, were used to represent the radial rebar, and membrane stiffness was entered in the radial direction only. Similarly, the hoop rebar was simulated with membrane stiffness in the hoop direction only, and the diagonal reinforcing steel was simulated with stiffness in the diagonal direction only. The floor rebar in the SSTs is an exception. The floor of the SSTs is constructed with a rectangular pattern of equal-size rebar in both directions that was modeled with single isotropic-elastic shell elements (SHELL63) having membrane stiffness only. For the DSTs, which have radial and circumferential steel on both faces of the floor, separate elements attached to concrete nodes were used to represent the radial and hoop rebar with membrane stiffness entered only in the radial and hoop directions, respectively. The transfer of shear between steel and concrete is accounted for by modeling the steel as separate elements attached to concrete nodes, thus maintaining strain compatibility.

Using this technique rather than smearing the rebar in the concrete element has several practical advantages: the rebar can be located accurately within the structure, particularly in complex areas such as the haunch and the footing/lower knuckle; the rebar elements may be grouped by type (radial or hoop) or by physical properties (dome, haunch, wall, etc.) to permit clear displays for verifying the model and more important, for viewing, evaluating, and documenting the results; and changes to the rebar, which may be required to accommodate future tank-specific analyses, can be performed conveniently and quickly.

The generic SST models do not include the inner carbon steel liner and its attendant asphaltic membrane and grout layers. The extent of load transfer between the liner and concrete is a function of the compressive stiffness of the interfacing membrane/grout layers. Since there are no data to quantify this stiffness, it is conservative to neglect these items and apply the internal tank loads directly to the concrete. An additional benefit of this approach is that it addresses the potential loss of structural integrity of the liner.

The soil was modeled under the floor and footing of the tank as soil springs with one layer of three-dimensional solid elements (SOLID45). These elements are similar to the concrete element (SOLID65) except that they do not provide cracking and crushing capabilities. In addition to imposing axisymmetric boundary conditions on these elements, they provided the vertical restraint for the model.

Unique features of each of the generic tank models are discussed below.

6.1 UNIQUE FEATURES OF THE 500,000-GALLON SST GENERIC MODEL

The 500,000-gal SSTs are located within five tank farms (241-B, -C, -T, -U and -BX). The configurations of the tanks in each of these farms are essentially the same as shown in Fisher and Shank 1994. Because the tanks in farm 241-BX were constructed with somewhat less reinforcing steel in their domes, this design was selected as the generic 500,000-gal SST.

The tanks are buried 77.2-ft-diameter, cylindrical, reinforced concrete structures 33-ft high. Their 12-in.-thick walls are supported by 2-ft-thick footings/lower knuckles approximately 6-ft wide. The cylinder is enclosed on top by a curved dome 15-in. thick and on the bottom by a curved floor 6-in. thick. The inside radius of the dome varies from 95 ft at its center, to 60 ft, and then to 10 ft, where it blends smoothly into the wall at the haunch. The inside radius of the curved floor is 570 ft. A single internal carbon steel liner extends across the floor and up the cylindrical walls. It varies in thickness from 3/8 in. on the floor to 1/4 in. on the wall. A 1-in. space between the liner and the wall is filled with 3/8 in. of asphaltic waterproof membrane and 5/8 in. of grout or mortar. The membrane also extends under the floor where the grout thickness increases to 2 in.

Six concrete elements were modeled through the 15-in. thickness of the dome to capture both bending and shear responses. Four elements were used between the inner and outer layers of rebar, and one outside each layer. Five elements were modeled through the 12-in. wall and three through the 6-in. floor, giving approximately the same element shapes in each region. Plots of the 500,000-gal generic SST model are shown in Figures 2, 3, and 4.

6.2 UNIQUE FEATURES OF THE 750,000-GALLON SST GENERIC MODEL

The 750,000-gal SSTs are located within four tank farms (241-BY, -TX, -TY, and -S). The configurations of the tanks in each of these farms are identical as shown in Fisher and Shank 1994. The drawings of tank farm 241-TY were selected for the generic 750,000-gal SST.

The tanks are buried 77.7-ft-diameter, cylindrical, reinforced concrete structures 39-ft high. Their 15-in.-thick walls are supported by 3-ft-thick footings/lower knuckle approximately 8-ft wide. The cylinder is enclosed on top by a curved dome 15-in. thick and on the bottom by a curved floor varying in thickness from 18 in. at the footing/lower knuckle to 6 in. at the center. The inside radius of the dome varies from 95 ft at its center, to 60 ft, and then to 10 ft where it blends smoothly into the wall at the haunch. A penetration with an outside radius of 22 in. is located at the center of the dome. The inside radius of the curved floor is 570 ft. A single internal carbon steel liner extends across the floor and up the cylindrical walls. It varies in thickness from 3/8 in. on the floor to 1/4 in. on the wall. A 1-in. space between the liner and the wall is filled with 3/8 to 1/2 in. of asphaltic membrane waterproofing and 1/2 to 5/8 in. of gunite (grout). The membrane also extends under the floor where the grout thickness increases to 2 in.

Six concrete elements were modeled through the 15-in. thickness of the dome to capture both bending and shear responses. Four elements were used

between the inner and outer layers of rebar, and one outside each layer. Five elements were modeled through the wall. The floor was modeled with a varying number of elements through the thickness starting with six at the footing/lower knuckle and decreasing to three at the center. This arrangement gave approximately the same element shapes in each region. Plots of the 750,000-gal generic SST model are shown in Figures 5, 6 and 7.

6.3 UNIQUE FEATURES OF THE 1,000,000-GALLON SST GENERIC MODEL

The 1,000,000-gal SSTs are located within three tank farms (241-A, -SX and -AX). The design features of the tanks in each of these tank farms are different as shown by Fisher and Shank (1994). The generic model envelops the three designs by combining the minimum structural features from each.

The tanks are buried 77.6-ft-diameter, cylindrical, reinforced concrete structures. Overall heights vary from 46.5 ft (241-SX) to 48.8 ft (241-AX). In all of the 1,000,000-gal SSTs, the upper part of the wall is 15-in. thick while the lower part is 24-in. thick. The cylindrical walls are enclosed on top by a curved dome 15-in. thick. The inside radius of the dome of the 241-AX tanks varies from 117.5 ft at the center, to 70 ft, and then to 26.5 ft, where it blends smoothly into the wall at the haunch. The inside radii of the 241-A and 241-SX domes are the same as those of the 500,000- and 750,000-gal SSTs (95, 60 and 10 ft, respectively). The bottom of the cylindrical walls is enclosed by the footing/lower knuckle and floor. Footings/lower knuckles of 241-AX tanks are 3-ft thick and 14-ft wide and the flat floor is 18-in. thick. Those of the 241-A tanks are 2-ft thick and 11.5-ft wide with a 6-in.-thick flat floor. Footings/lower knuckles of the 241-SX tanks are 23-in. thick under the walls and blend into the curved floors, which are 8-in. thick at the tank center. The inside radius of the curved floor is 570 ft. A single 3/8-in.-thick internal carbon steel liner extends across the floor and up the cylindrical walls of each tank. The liner bears directly on the concrete except in the 241-A and 241-SX tanks. In these tanks a space between the liner and floor is filled with 3/8 in. of asphaltic membrane waterproofing and 2-in. of grout.

The generic 1,000,000-gal SST model was assembled from the above geometry as follows.

- **DOME/HAUNCH** - The dome of the 241-AX tank design was selected because it is the flattest (has the largest radii), which makes it the most flexible structurally. Further, the 241-A and 241-SX domes are like those of the 500,000- and 750,000-gal tanks, which are evaluated with those tanks. The thickness of all the domes is the same, and the reinforcing is nearly identical, except for the 241-AX tank where it is somewhat less.
- **WALL** - The wall from 241-SX was selected. The lower 24-in.-thick region of its wall is about 5-ft shorter than the wall of the other tanks. The reinforcing in this region is on each face only, while tanks 241-A and 241-AX have one or two rows of additional interior hoop rebar. Both of these considerations make the 241-SX wall more flexible structurally.

- FOOTING/LOWER KNUCKLE AND FLOOR - Structurally, tank 241-A has the minimum footing/lower knuckle and floor. The footing/lower knuckle is thinner and narrower than that of 241-AX, and the floor is thinner than that in either of the other tanks. It has a single layer of reinforcing steel; 241-AX has layers on each face, as does 241-SX, outside of a 23.5-ft radius.

Seven concrete elements were modeled through the 15-in. thickness of the dome to capture both bending and shear responses. Five elements were used between the inner and outer layers of rebar, and one outside each layer. Six elements were modeled through the wall and footing/lower knuckle. The floor has four elements through the 6-in. thickness. This arrangement gave approximately the same element shapes in each region. Plots of the generic 1,000,000-gal SST model are shown in Figures 8, 9 and 10.

6.4 UNIQUE FEATURES OF THE 1,000,000-GALLON DST GENERIC MODEL

The 1,000,000-gal DSTs are located within six tank farms (241-AN, -AP, -AW, -AY, -AZ, and -SY). The design features of these DSTs are similar, with only a few exceptions, as shown by Fisher and Shank (1994). The generic model envelops the designs by combining the minimum structural features from each.

The tanks are buried 83-ft-diameter, cylindrical, reinforced concrete structures about 51-ft high. Their 18-in.-thick walls are supported by, but not connected to, 2-ft-thick footings that vary in width from about 15 to 22 ft. The floor connecting the footings is 8- to 9-in. thick. The cylindrical walls are enclosed on top by a curved dome 15-in. thick. The inside of the dome is an 80- by 30-ft ellipse. An internal carbon steel secondary liner extends across the floor and up the cylindrical walls and the haunch to the dome. This liner bears on a 2.5-in. layer of concrete (waffle slab) placed on top of the floor and slotted with a rectangular pattern of 5-in.-wide drainage passages. It is attached to the wall and haunch with 1/4-in. J-bolts located on 2-ft centers. The 75-ft-diameter carbon steel primary tank rests on an 8-in. layer of refractory concrete placed between it and the secondary liner. It becomes tangent with the upper end of the secondary liner in the haunch region and is then attached to the dome concrete with 1/2-in. J-bolts, also on 2-ft centers, for the remainder of its length. There is an 18-in. annulus between the primary tank and secondary liner.

The concrete portions of the 1,000,000-gal DSTs are very similar. The outside radii of the 241-AP domes are 5- to 6-ft larger than those of the other tanks, but the inside dome dimensions are the same in all tanks. So those with smaller outside radii conservatively provide a somewhat thinner dome and haunch. The most significant geometric variation among the tanks is in the footing/floor configurations: tanks 241-SY and 241-AY have the minimum sections structurally. The reinforcing steel in all the tanks is also very similar. Rebar areas from tank 241-SY were used everywhere except in the upper part of the wall and the footing. The quantity of 241-AP rebar was minimal in the upper wall and that of 241-AY was minimal in the footing. The plate thicknesses of the secondary liner and primary tank are minimal in the 241-AY tanks; hence, these were used in the model.

Six concrete elements were modeled through the 15-in.-thickness of the dome to capture both bending and shear responses. Four elements were used between the inner and outer layers of rebar, and one outside each layer. Five elements were modeled through the wall and three through the floor. This arrangement gave approximately the same element shapes in each region. Plots of the generic 1,000,000-gal DST model are shown in Figures 11, 12 and 13.

The carbon steel secondary liner and the primary tank were modeled with isotropic elastic shell elements (SHELL63). The refractory concrete, located between the liner and primary tank, was modeled with one layer of orthotropic three-dimensional solid elements (SOLID45) as was the waffle slab located beneath the liner. The SOLID45 elements are similar to the concrete elements (SOLID65) except that they do not provide cracking and crushing capabilities. The SOLID45 elements were given vertical stiffness only.

The shear stiffness of the J-bolts connecting the steel secondary liner to the surface of the concrete wall was modeled with nonlinear spring elements (COMBIN39). The shear stiffness of the J-bolts connecting the liner to the haunch and the primary tank to the dome was modeled with three-dimensional spar elements (LINK8). Nonlinear shear stiffness was accounted for by using bilinear material properties, to capture the effect of J-bolt yielding and to permit the redistribution of load to adjacent J-bolts. Axial stiffness of the J-bolts between the steel secondary liner/primary tank dome and the concrete surface was modeled with uniaxial linear spring elements (COMBIN14). Axial yielding of these bolts did not occur. The portion of the J-bolts embedded in the concrete was modeled with orthotropic-plastic shell elements (SHELL43). These elements extended from the concrete surface to the first layer of rebar. Since the J-bolts were cast-in-place in the concrete they are not considered to be preloaded.

Three-dimensional point-to-point surface contact elements (CONTAC52) were modeled between the steel secondary liner/primary tank dome and the concrete surface at intermediate node locations between J-bolts. These contact elements, which are capable of supporting compression only, were given zero initial gaps. The interfaces between the primary tank and refractory concrete and between the secondary liner and waffle slab in the area near the wall were modeled in the same manner. The same type of element was used to represent the lower corner radii of the secondary tank steel liner and the primary tank but positive initial gaps, calculated from the geometry, were included. The interface between the wall and footing was also modeled with contact elements in the vertical direction and laterally to the outside footing shear ledge.

7.0 CURRENT OPERATING SAFETY REQUIREMENTS

As noted in Section 1.0 above, one of the purposes of the ASA-Phase I effort is to assess the appropriateness of the existing tank farm operational controls/limits as now stipulated in the OSRs and OSDs, and where appropriate, to recommend changes or modification to these existing controls/limits. To accomplish this task, a detailed review was made to identify all of the current structurally-related OSR and OSD tank farm operational controls/limits. The IOSRs and OSDs reviewed include the following:

- WHC-SD-WM-OSR-005, *Single-Shell Tank Interim Operational Safety Requirements* (Dougherty 1994).
- OSD- T-151-00013, *Operating Specifications for Single-Shell Waste Storage Tanks* (OSD 1992a).
- WHC-SD-WM-OSR-016, *Double-Shell Tank Interim Operational Safety Requirements* (Heuback 1994).
- OSD-T-151-00007, *Operating Specifications for the 241-AN, AP, AW, AY, AZ and SY Tank Farms* (OSD 1992b).
- WHC-SD-WM-OSR-004, *Aging Waste Facility Interim Operational Safety Requirements* (Aguirre 1994).
- OSD-T-151-00017, *Operating Specifications for Aging-Waste Operations In 241-AY and 241-AZ* (OSD 1989).

Based on this review the following structurally-related controls/limits and loading mechanisms were identified for consideration in the ASA-Phase I evaluations:

- Maximum waste level
- Maximum waste temperature
- Vapor space pressure
- Corrosion
- Hydrostatic load
- Live load
- Soil loads
- Dome deflection
- Heat-up/cool-down rates
- Seismic
- Concrete tank temperatures

Although the SST OSR and OSD contain entries for the 55,000-gal SSTs, these tanks were not evaluated in the ASA-Phase I effort. Hence, no consideration of the 55,000-gal tanks (200 series tanks) is provided in this document.

8.0 DISCUSSION OF LOADINGS

Loads that must be resisted by the waste tanks consist of the following types:

- Construction loads
- Normal loads
- Operating loads
- Natural-hazard loads
- Postulated accident loads.

A brief definition of specific types of loads included in each of the above designations is provided below (see Sections 8.1 through 8.5).The

sensitivities of the tanks to loadings other than normal and operating loads, were not included in the ASA-Phase I effort. Other loadings will be addressed in subsequent qualification and accident analyses.

To facilitate performing the ASA-Phase I evaluations and to ensure that appropriate loading variations were addressed, the loading document, *Single- and Double-Shell Tanks Load Report for Accelerated Safety Analyses* (Becker 1994) was prepared in conjunction with the Tank Farm Safety Analysis Documentation and Tank Waste Remediation System personnel. It identifies all loadings and the variations in loading values that were considered in performing the ASA-Phase I evaluations.

It is recognized that a major concern associated with structural performance of the waste tanks is exposure of the tanks to elevated temperature service conditions. For reinforced concrete structures, it is well documented that repeated changes in load and temperature, particularly for temperatures above 200 °F, lead to in situ structural conditions that are critical to future structural performance. These conditions include the long-term effects of creep and temperature degradation of mechanical properties, structure ratcheting due to crack opening and closing, and the residual stresses to which all these phenomena and the loading itself contribute. For this reason, an extensive effort was conducted to identify the historical service conditions for all SSTs and DSTs, in terms of waste level and waste temperature. The results of this effort, documented in Huisingh, et al. (1994), were used to establish the upper bounds of the thermal loading evaluated in the ASA-Phase I sensitivity evaluations (see Becker 1994). A summary tabulation of the loadings and the values considered in the ASA-Phase I effort is provided in Table 1.

Huisingh et al. (1994), reports that according to the records located to date a few SSTs have been exposed to waste temperatures in excess of 350 °F. Additional SSTs are also suspected to have had waste temperatures exceed 350 °F. In Becker (1994) the concrete was conservatively assumed to reach the same temperature as the waste. The maximum waste/concrete temperature to be addressed in the ASA-Phase I evaluations was 350 °F for the SST and 250 °F for the DSTs. Additional evaluations beyond the ASA-Phase I effort will be required to investigate those SSTs that have experienced temperatures greater than 350 °F.

8.1 CONSTRUCTION LOADS

Construction loads include the effects of short-term construction loads and equipment, e.g., lifting loads, trucks, cranes, and girders, which usually must be resisted by a partially constructed structure. Leak testing and pressure testing may also be considered as construction loads. Wind loading is sometimes considered at various stages during the construction process. As noted above, construction loads were not considered during the ASA-Phase I effort.

8.2 NORMAL LOADS

Normal loads are those encountered during normal operation. These include gravity, soil, hydrostatic, and thermal loads.

- Gravity loads consist of dead loads, which include the weight of permanent construction (e.g., the secondary reinforced-concrete tank, the secondary tank steel liner, and the primary steel tank), liquid waste material, and fixed equipment (e.g., pump pits and motors), live loads, which include the weight of movable equipment, and other loads that vary in intensity and occurrence.
- Hydrostatic loads result from the liquid waste exerting hydrostatic pressure on the primary tank floor and wall.
- Thermal loads result from heat generation within the radioactive waste material. Depending upon the tank contents, heat load generation rates, and tank cooling, a varying time-dependent temperature distribution may exist.
- Soil loads result from soil overburden on top of the tank dome and lateral confinement along the outside of the wall. Soil loads are functions of the soil backfilling placement procedures during construction and depend on placement compaction and soil properties such as cohesion and friction angle.

8.3 OPERATING LOADS

Operating loads consist of pressure and loads resulting from conducting operational activities (modified operations).

- Pressure loads are a function of pressure differentials between the tank inner vapor space and the atmospheric conditions. Pressure loads are governed by operation of the tank ventilation system and capacity. In the case of the DSTs, the pressure within the annulus space contributes to the overall pressure loading of the tank.
- Modified operations loads account for an abnormal surcharge due to trucks, cranes, or other construction equipment being placed over the tank dome.

8.4 NATURAL-HAZARD LOADS

Natural-hazard loads include flood loads, wind loads, and seismic loads. Current provisions for design to resist natural hazards are stipulated in DOE-RL 1993. These provisions have been determined from the guidance document UCRL-15910 (Kennedy et al. 1988).

As noted previously, natural-hazard loads are not included in the ASA-Phase I sensitivity evaluations. During initial planning of the ASA-Phase I effort it was anticipated that the results of planned parallel, seismic

analyses of tank C-106 would be available at the time of this report was written, and that these results could be referenced; thus, duplicate efforts to evaluate the waste tanks for resistance to seismic loadings could be avoided. However, the C-106 seismic analyses have been delayed and the results are not available for incorporation into this report. Evaluation for accident loadings are anticipated to be performed under a separate scope of work.

8.4.1 Flood Loads

The areas of the Hanford Site containing the tank farms are located above the highest elevation that would be reached by Columbia River flood waters during the probable maximum flood. Flood scenarios have also been reported for seismically induced failures of upriver dams on the Columbia River. Maximum flood water elevations for these scenarios are also below the elevation of the 200 Area plateau.

8.4.2 Wind Loads

Wind loads for surface structures shall be in accordance with the requirements of SDC-4.1 (DOE-RL 1989). Since the waste tanks are completely below grade, wind loads do not need to be considered during design. However, wind loads may need to be considered during construction.

8.4.3 Seismic Loads

The Hanford Site is located in the Pasco Basin in the Intermontane Plateau Physiographic Province. The Site is an area of low-magnitude seismicity that has been monitored instrumentally since 1969. The historical record of felt earthquakes goes back to about 1850; however, these early records are anecdotal. Seismotectonic studies were completed for the design and construction of the Fast Flux Test Facility (FFTF). These studies culminated in a site-specific seismic design response spectra anchored at 0.25 g peak horizontal ground motion. The FFTF was not licensed by the U.S. Nuclear Regulatory Commission (NRC); however, the NRC did review the final safety analysis report (NUREG 1978). The FFTF seismic design criteria were incorporated into the Hanford Plant Standards *Architectural-Civil Design Criteria*, SDC 4.1, Revision 6, in November 1973 (DOE-RL 1973).

The FFTF seismic design criteria were used for seismic qualification of safety-related equipment at the Hanford Site until September 9, 1989, when SDC 4.1 was revised to incorporate recent DOE design guidance (DOE-RL 1989 and Kennedy et al. 1988). A graded approach to the design of DOE nonreactor facilities was recommended, based upon the hazard level of the facility. The postulated seismic load for a high-hazard non-reactor facility is equivalent to the 5,000-year return-period earthquake and, for a moderate hazard, is equivalent to the 1,000-year return-period earthquake. A median Newmark and Hall spectral shape is recommended by the DOE design guidance.

A probabilistic seismic hazard study was completed for the various facility locations on the Hanford Site (WHC 1989) to determine the 5,000- and 1,000-year return-period ground motions. Revision 11 of SDC 4.1 (DOE-RL 1989)

incorporates the DOE guidance and the results of the seismic hazard study. The response spectra for high- and moderate-hazard facilities are anchored to 0.20 g and 0.12 g peak horizontal ground motion, respectively.

8.5 POSTULATED ACCIDENT LOADS

Postulated accident condition loads are imposed on the tanks from various abnormal conditions identified in either the functional design criteria or the safety analysis report. These loading conditions typically involve the dissipation of large amounts of energy, are dynamic in nature, and usually result from temperature and pressure excursions or dynamic and impulsive loadings. Where postulated accident conditions are identified in functional design criteria, appropriate acceptance criteria should also be provided. If, however, the postulated accident conditions result from safety analyses performed subsequent to design, appropriate evaluation criteria must then be identified or developed. First-step evaluations normally involve an examination of the original design methods, margins, and criteria. If insufficient design margin exists to provide a positive margin for resisting the postulated-accident conditions, then additional analyses using more sophisticated analyses methods can be performed, and/or more appropriate acceptance criteria based on a positive margin of safety against deformation or failure can be considered. Postulated accident conditions were not included in the ASA-Phase I scope of work; hence, they are not addressed in this report.

8.6 ASA-PHASE I LOADING EVALUATIONS

For the ASA-Phase I, the waste tanks have been evaluated for normal loads and operational loads that consist of internal loads and external loads as well as those caused by seismic motions. The normal loadings and operational loadings that were considered are identified in loads document *Single- and Double-Shell Tanks Load Report for Accelerated Safety Analyses* (Becker 1994). A listing of these loads is provided in Table 1. The loads addressed in this evaluation were classified as internal, external, thermal and dynamic. Body loads such as the weight of steel reinforcement and concrete, are transparent yet included in the evaluation. Loading profiles, sequences and steps of the specific conditions evaluated are documented in Fisher et al. 1994.

8.6.1 External Loads

The external loads are the loads applied to the outside of the underground waste storage tanks. These loads include soil overburden or dome loadings, lateral soil loads applied to the cylindrical shell wall, uniform distributed loading, and concentrated loading. The uniform distributed loading and the concentrated loading are applied to all waste tanks to represent tank farm aboveground operations. Unlike the uniform and concentrated loadings, the soil overburden loading varies from tank farm to tank farm and even from tank to tank on a tank farm. Hence, the evaluation effort has focused on determining the sensitivity of a waste tank structure to variations in soil height above the apex of the waste tank dome.

8.6.1.1 Single-Shell Tanks. The SSTs were evaluated for overburden soil depths ranging from a minimum of 5 to a maximum of 10 ft and for soil density variations from 101 to 130 lbf/ft³. The lateral soil pressure acting on the tank wall is a function of density, depth of soil and Rankine coefficient. The Rankine coefficient used in the evaluations was 0.5. A loading profile was established for each of the single-shell tanks showing soil height and density variations.

A uniform load of 100 lbf/ft² and a 100-ton concentrated load over a 10-ft-radius area were evaluated. The 100 lbf/ft² uniform loading allows 50 lbf/ft² for contingencies such as concrete slabs, and snow and ash load. A loading profile was established which includes waste specific gravity and vapor space pressure in addition to the 100 lbf/ft² uniform loading and 100-ton concentrated loading.

8.6.1.2 Double-Shell Tanks. The DSTs were evaluated for an overburden soil depth ranging from 6 to 9 ft. A range in density variation from 101 to 130 lbf/ft³ was also considered. The lateral soil pressure acting on the tank wall is a function of density, depth of soil, and Rankine coefficient. The Rankine coefficient used in the evaluations was 0.5. In addition, a case was evaluated for a Rankine coefficient of 0.7 to determine its significance on the tank structure. A uniform load of 100 lbf/ft² and a 100-ton concentrated load over a 10-ft-radius area were evaluated. A loading profile was established for the DST showing soil height and density variations.

8.6.2 Internal Loads

The internal loads are those within the waste storage tank such as waste loads and vapor space pressures. For the DSTs, there is an annulus pressure loading between the primary steel tank and secondary steel tank.

8.6.2.1 Single-Shell Tanks. The SSTs were evaluated at waste levels of 1/4 full, 2/3 full, and full. The full waste level was applied with combined loadings that represent a reference loading. Successive load steps were made representing an empty tank condition, a 1/4-full condition and a 2/3-full condition.

The waste specific gravity was considered to be within a range of 1.0 to 2.0. Hence, a load case established to evaluate both specific gravities was included in the external loading profile. Likewise, the vapor space pressure loading was included in the external loading profile.

8.6.2.2 Double-Shell Tanks. The DSTs were evaluated at levels of 1/4-full, 2/3-full and full waste level. The full waste level was initiated with combined loadings that represent a reference loading. Successive load steps were made representing a 1/4-full condition and a 2/3-full condition along with a range of waste specific gravities from 1.0 to 2.0.

8.6.3 Thermal Loads

The thermal loads considered in this evaluation are a uniform waste temperature loading and a dome vapor-space temperature loading. Because past

thermal analyses are limited in number and scope or address only specific tanks, the temperature is assumed to be constant through the thickness. A thermal transition length of 6 ft was assumed along the tank wall to attenuate the waste temperature.

8.6.3.1 Single-Shell Tanks. A maximum waste temperature of 350 °F was evaluated based on current operating controls. Also, a combined temperature loading having a waste temperature of 300 °F and a dome temperature of 250 °F was evaluated. A loading profile was established to attain the specified temperatures by ramping the increasing temperatures at full waste height from a reference temperature of 70 °F. The temperatures were applied to the model assuming that the concrete structure is in contact with the waste and is at waste temperature, and the dome is at vapor-space temperature.

8.6.3.2 Double-Shell Tanks. The DSTs were evaluated for a maximum waste temperature of 250 °F, as noted in Becker (1994) and a dome temperature of 200 °F. Therefore, a loading profile was established for the specified temperatures at both full-level waste and 1/4-level waste. The loading profile ramps the increasing temperatures representing waste and the vapor space from a reference temperature of 70 °F. The temperatures are applied to the model assuming the concrete structure in contact with the waste is at waste temperature and the dome is at vapor-space temperature with a 6-ft transition in length.

8.6.4 Dynamic Loads

Dynamic loads include seismic loading and accident-related loadings. Accident-related loadings were not included in the ASA-Phase I scope of work. Seismic evaluations are discussed in Section 9.4.

8.6.5 Subgrade Modulus

The effect of soil subgrade modulus on the response of the tank footing and wall was evaluated using the generic DST model. Soil subgrade moduli values of 200, 300 and 497 lbf/in.² of area were considered with a constant value assumed for both the floor and the footing.

9.0 ANALYSIS SENSITIVITY FINDINGS

9.1 OPERATING LOADS

The models were divided into five components for reviewing stress output: the dome, the haunch, the cylindrical wall, the footing/lower knuckle, and the floor. Stresses in each component were reviewed to determine the maximum stresses for each load case evaluated. Additional components examined on the DSTs include the J-bolts in the dome, upper haunch, and wall; primary steel tank; and the secondary tank steel liner.

To determine the loading sensitivity effect on the tank structure, a summary stress change per unit load was developed for each tank region. These

loading sensitivities, which are documented in Fisher et al. 1994, were then used to develop summaries of the predicted stress changes due to loading (or model parameter) changes. Tables summarizing these predicted stress changes are provided as discussed below. Estimated maximum variations were used to convert the sensitivities to stress changes.

Terms that have coefficients which are positive reduce the compressive stress while terms that have coefficients which are negative increase the compressive stress. The temperature and vapor pressure decrease the compressive stress of the haunch. The temperature effect must be evaluated with respect to any strength degradation that may occur due to the temperature of the component.

The stress, sensitivity, and stress-change tables can be used to develop load cases that approximate maximum stress levels in the various components. Care must be exercised when developing these load cases as some load (or model) parameters cause stress increases in some components and stress decreases in other components. Also, large-magnitude changes in loading (or model) parameters from those used in the study may cause additional nonlinearities and stress redistribution which will affect the sensitivities reported herein.

9.1.1 500,000-Gallon Single-Shell Tanks

In the 500,000-gal SSTs, the calculated maximum concrete stress of 1,289 lbf/in² in compression occurred in the haunch section from a 10-ft soil overburden loading at a density of 101 lbf/ft³ (see Table 2). The calculated concrete stress change (sensitivity) for a 29 lbf/ft³ change in soil density is 271 lbf/in² (see Table 5). The predicted maximum compressive concrete stress for a 10-ft soil loading with a density of 130 lbf/ft³ is 1,560 lbf/in² (1,289 plus 271). This value is well below the maximum design compressive strength of 3,000 lbf/in², but slightly above the thermally degraded strength of 1,417 lbf/in² (Peterson 1994). The reinforcing steel has a maximum calculated stress is 26,195 lbf/in² which occurs in the cylindrical wall from a 10-ft soil overburden loading for a soil density of 101 lbf/ft³ (see Table 3). The calculated steel stress change (sensitivity) for a 29 lbf/ft³ change in soil density is 7,639 lbf/in² (see Table 6). Thus, the predicted maximum steel stress for a 10-ft soil loading with a density of 130 lbf/ft³ is 33,834 lbf/in² (26,195 plus 7,639). This value is below the yield stress of 40,000 lbf/in². The maximum stresses are summarized in Tables 2, 3 and 4 for concrete and steel reinforcement. Maximum concrete and reinforcing steel stresses result from loading case S3, as tabulated in Table 30. The detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994) gives the load cases for the other stress values listed in Tables 2, 3 and 4.

Tables 5, 6 and 7 summarize the stress changes due to loading (or model) parameter changes. Estimated maximum variations were used to convert the sensitivities to stress changes. Table 5 shows that the maximum concrete principal stress changes are due to changes in the soil depth, soil density, uniform load, concentrated loading, waste temperature, and waste level.

Specific gravity is less significant. Increasing vapor pressure produces a reduction in compressive stresses.

Tables 6 and 7 show that the steel reinforcement also experiences significant stress changes due to soil depth, soil density, uniform load, concentrated loading, waste temperature and waste levels.

9.1.2 750,000-Gallon Single-Shell Tanks

In the 750,000-gal SSTs, the maximum concrete stress of 1,420 lbf/in² in compression occurred in the haunch section from soil overburden loading. This value is well below the maximum design compressive stress of 3,000 lbf/in². But is higher than the degraded strength of 1,417 lbf/in². Since the temperature in the haunch region is below 350 °F, the ANSYS stress solution indicated that crushing did not occur. The reinforcing steel experienced a maximum stress of 34,548 lbf/in², which occurred in the cylindrical wall from soil overburden loading. This value is below the yield stress of 40,000 lbf/in². The maximum stresses are summarized in Tables 8, 9, and 10 for concrete and steel reinforcement. Maximum concrete and reinforcing steel stresses result from loading case S5, as tabulated in Table 30. The detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994) gives the load cases for the other stress values listed in Tables 8, 9 and 10.

Tables 11, 12 and 13 summarize the stress changes due to loading (or model) parameter changes. Table 8 shows that the maximum concrete principal stress changes are due to changes in the soil depth, soil density, uniform load, concentrated loading, temperature, and waste level. Specific gravity is less significant. Vapor-pressure changes produce a reduction in compressive stresses.

Tables 12 and 13 show that the steel reinforcement also experiences significant stress changes due to soil depth, soil density, uniform load, concentrated loading, temperature, and waste levels.

9.1.3 1,000,000-Gallon Single-Shell Tanks

In the 1,000,000-gal DSTs, the maximum concrete stress of 1,251 lbf/in² in compression occurred in the lower knuckle/floor section from soil overburden loading. This is well below the maximum design compressive stress of 3,000 lbf/in² and below the 1,417 lbf/in² degraded strength. The reinforcing steel experienced a maximum stress of 6,733 lbf/in², which occurred in the haunch component from soil overburden loading. The maximum stresses are summarized in Tables 14, 15 and 16 for concrete and steel reinforcement. Maximum concrete and reinforcing steel stresses correspond to load case S5, as shown in Table 30. The detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994) gives the load cases for the other stress values listed in Tables 14, 15 and 16.

Tables 17, 18 and 19 summarize the stress changes due to loading (or model) parameter changes. The tables show that the maximum concrete principal stress changes are due to changes in the soil depth, soil density, uniform load, concentrated loading, waste temperature and waste level. Specific gravity is less significant. Increasing vapor pressure produces a reduction in compressive stresses in the concrete.

Tables 18 and 19 show that the steel reinforcement also experiences significant stress changes due to soil depth, soil density, uniform load, concentrated loading, waste temperature and waste levels.

9.1.4 1,000,000-Gallon Double-Shell Tanks

In the 1,000,000-gal DSTs, a maximum concrete compression stress of 806 lbf/in² occurred in the wall section under soil loading. This value is well below the maximum design compressive strength of 3,000 lbf/in² and the degraded value of 2,524 lbf/in² (Peterson 1994). A maximum reinforcing steel stress of 23,838 lbf/in² occurred in the radial reinforcing steel. This value is below the minimum rebar yield stress of 40,000 lbf/in². This stress occurred in the floor section after concrete cracking and subsequent stress redistribution. Maximum stresses for the primary and secondary steel tanks were 16,765 and 16,793 lbf/in², respectively. These are below the minimum yield stress of 32,000 lbf/in². J-bolt forces in the dome, upper haunch, and wall showed that shear yielding occurred. After J-bolt yielding any additional loads will be distributed to adjacent J-bolts. This minimizes the stress concentration in the tank walls. Maximum stress and force values are summarized in Tables 20 through 24. Maximum stresses for the concrete, reinforcing steel, primary tank and secondary steel tank liner correspond to load cases S5, L5, L1, and S5, respectively, as shown in Table 30. The detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994) gives the load cases for the other stress values listed in Tables 20 through 24.

Tables 25 through 29 summarize the force and stress changes due to loading (and model) parameter changes. Table 15 indicates that maximum concrete principal stress changes in concrete are due to changes in the lateral soil load (Rankine) coefficient, the soil depth, the soil density, the concentrated load application, and the waste-level reduction. Vapor pressure and temperature produced the next highest stress changes. Uniform load, specific gravity, annulus pressure, and subgrade modulus are seen to have very little effect on concrete principal compressive stress levels.

Table 26 shows that radial steel reinforcing stress changes are primarily affected by the waste level and subgrade modulus. This particular steel is located in the floor region; for these loading cases exhibited significant concrete cracking and subsequent stress redistribution to the steel (see crack plots in Appendix I of Fisher et al. 1994). Temperature, concentrated load application, creep and soil depth contributed to the next largest effects. All other loadings had very small effects on steel stress changes.

Table 27 shows that hoop steel reinforcing stress changes are primarily affected by temperature changes and creep. Concentrated load application, soil depth, soil density, and the lateral soil loading coefficient have the next biggest effects. All other parameter variations produced small changes in hoop steel stress.

Table 28 summarizes primary and secondary tank steel equivalent stress changes. Secondary tank stress changes are largest for the lateral soil loading coefficient, temperature, soil depth, soil density variations and creep. The primary tank shows the largest changes for waste specific gravity and creep.

Table 29 shows the force changes in the J-bolts. Most of the load cases cause yielding in specific J-bolts where force values do not increase due to load or model changes. Therefore, the sensitivities for these cases were listed as zero.

9.2 THERMAL GRADIENT ANALYSIS

Temperature gradients through the tank concrete wall resulting from different thermal transient loadings from the tank waste were investigated using both closed-form theoretical solutions and finite-element analysis. The concrete wall was assumed to be subjected to various degrees of surface temperature change per day until it reached a maximum temperature of 350 °F. This assumption conservatively simulates the worst temperature condition the inside wall of the tank might possibly have experienced regardless of the amount of heat generated in the waste.

Conservative calculations determined the worst temperature gradient through the wall that is expected to have resulted in the highest thermal stress in the concrete wall. Detailed results of the thermal gradient comparisons are contained in Fisher et al. 1994; a summary of the results is provided below.

Different heating rates ranging from 2 °F per day to 20 °F per day, from an initial temperature of 70 °F up to a maximum of 350 °F, were applied at the tank inside wall. Different through-wall temperature gradients were calculated. Theoretical calculations and finite-element modeling were performed to determine the temperature gradients through the concrete wall under different rates of loadings. The results of theoretical calculations were essentially to verify those from the numerical computations by finite-element method. The results were in excellent agreement. A temperature gradient of 18.8 °F was calculated for a heating rate of 3 °F per day up to 350 °F. The worst temperature gradient is 51 °F for a heating rate of 20 °F per day reaching up to 350 °F for a period of 14 days.

9.3 CONCRETE CREEP REVIEW FOR HANFORD APPLICATIONS

An important consideration in the design, remediation, and safety evaluation of reinforced concrete nuclear waste storage tanks is the creep behavior of concrete subjected to stresses, elevated temperatures, and various moisture or curing conditions. As part of the Hanford Waste Tank Accelerated

Safety Assessment Program, previous concrete creep research work was investigated to assess the effects of creep on long-term structural integrity and load capacity of existing underground storage tanks. An overview of the research and experimental work reviewed is given below along with a summary of the results of an assessment of the creep effects on the 1,000,000-gal generic DST model.

9.3.1 Creep And Concrete As A Structural Material

Although this investigation originated with a conservative safety concern over the effects of concrete creep on Hanford waste tanks, the phenomenon of concrete creep has been found overall to contribute to the success of concrete as a structural material (Neville 1970). Concrete is a complex nonlinear material; its failure strain limits depend on the strain rate and the time during which any increment of load acts.

Although cracks visible in reinforced concrete members may imply that concrete is a brittle material that fails at low strains, it is not. Large strains are possible before failure, with the result that reinforced concrete can fail in a ductile manner; and reinforced concrete can develop large rotations leading to a considerable redistribution of stress and an energy-absorbing ductile failure. This comparative ductile failure of reinforced concrete which contributed to its success as a structural material, can be attributed to its ability to undergo creep.

The effect of concrete creep is to redistribute structural load from regions of high stresses to regions of minimal stress. This load redistribution allows more load-carrying capability before failure. Although this is the case; it must be remembered that conservative reinforced concrete practice (such as defined by ACI), with large built-in margins, may not always view these redistributions as meeting accepted design guidelines.

Although reinforced concrete creep can be beneficial, such is not always the case (Neville 1970). The creep can harm prestressed concrete members. Creep also can have harmful effects on massive concrete structures that undergo a thermal cycle that produces a thermal gradient. The initial thermal stresses are relieved by creep, but cooling produces an undesirable reversal of stress and the result may be cracking.

The Hanford waste tanks do not employ any prestressed concrete construction; thus, the first harmful effect is not important for the Hanford Site. Although the Hanford C-106 thermal creep studies addressed thermal cycling, the temperature contours were derived with model assumptions that gave maximum temperatures but not necessarily maximum possible thermal gradients. Additional evaluation should be planned to investigate thermal cycling of waste tanks with a range of thermal gradients.

Specific technically related issues that were investigated and are addressed in considerable detail in Fisher et al. 1994 are as follows.

- Creep under uniaxial stress
- Creep compliance function
- Influence of humidity on creep
- Influence of concrete curing on creep

- Influence of temperature on creep
- Time dependency and long-term creep of concrete
- Creep under multiaxial states of stress
- Creep Poisson's Ratio
- Influence of aggregate on creep
- Aggregate content and creep
- Influence on creep of stress-strength ratio

9.3.2 Creep Analysis Results of 1,000,000-Gallon Single-Shell Tank

A concrete thermal creep analysis of the 1,000,000-gal SST generic model was performed for a waste level of one-quarter with a waste temperature of 350 °F. The creep relation was defined with existing ANSYS capability for creep modeling. The creep magnitudes-versus-temperature were specified to give creep strains at 50 years, as defined by the Rashid thermal creep model of Hanford concrete (James and Rashid 1993). Although, the literature review has shown possible large variations in creep magnitude, it is important to relate the initial results to past Hanford work.

The concrete in the 1,000,000-gal SST at the end of 50 years showed a significant unloading of compression stress. The maximum concrete compressive stress in the upper haunch decreased from 598 to 375 lbf/in². The creep-induced concrete stress reduction corresponded to an increase in steel stresses.

The most significant stress change for the SST steel occurred in the wall vertical rebar. As the tank concrete wall creeps over 50 years, the motion of the concrete tends to apply additional stress to the rebar. The maximum induced compression stress increases from 10,280 to 24,887 lbf/in² (Yield of the steel rebar is 40,000 lbf/in²).

The second significant stress change for the SST rebar occurred in the upper haunch. As the tank concrete dome and haunch creeps over 50 years, the maximum haunch rebar compression stress increases from 7,869 to 23,785 lbf/in².

9.3.3 Creep Analysis Results Of 1,000,000-Gallon Double-Shell Tank

A concrete thermal creep analysis of the 241-SY tank design was performed for a waste level of one-quarter with a waste temperature of 250 °F. The creep relation was defined using existing ANSYS capability for creep modeling. The creep magnitudes-versus-temperature were specified to give creep strains at 50 years, the same as defined by the Rashid (James and Rashid 1993) thermal creep model of Hanford concrete. Although the research literature review has shown possible large variations in creep magnitude, it is important to relate the initial results to past Hanford work.

The concrete in the 1,000,000-gal DST at the end of 50 years showed a significant unloading of compression stress. The maximum concrete compressive stress in the wall decreased from 551 to 244 lbf/in². The creep-induced concrete stress reduction corresponded to an increase in steel stresses.

The most significant stress change for the double-shell tank steel was in the secondary liner in the lower knuckle. The secondary liner is anchored to the concrete wall above the knuckle with the full weight of the waste constraining the bottom of the liner to the floor. As the tank concrete wall creeps over 50 years, the motion of the concrete tends to apply additional bending to the secondary liner knuckle radius region. The maximum bending induced stress increases from 15,624 to 31,455 lbf/in² (Yield of the steel liner is 33,000 lbf/in²).

9.3.4 Conclusions And Concerns For Hanford Waste Tank Thermal Creep

Conclusions and concerns relative to concrete creep as applied to Hanford waste tanks are as follows:

- The main effect of concrete creep is to redistribute structural load within the concrete from regions of high concentrated stresses to regions of minimal stress. This load redistribution is beneficial and allows more load carrying capability before failure. The 1,000,000-gal DSTs concrete maximum compressive stresses decreased over a 50 year life as a result of creep.
- The 1,000,000-gal SST concrete maximum compressive stress of 598 lbf/in² is well within the absolute proportional creep limits for creep prediction. Over 50 years the concrete maximum compressive stress decreased from 598 to 375 lbf/in².
- The 1,000,000-gal SST maximum rebar stress increases from 10,280 to 24,887 lbf/in² over a 50 year period.
- The 1,000,000-gal DST concrete maximum compressive stress of 551 lbf/in² is well within the absolute proportional creep limits for creep prediction. Over 50 years the concrete maximum compressive stress decreased from 551 to 244 lbf/in².
- The 1,000,000-gal DST secondary liner steel increases in stress over time to 31,455 lbf/in² at 50 years. This maximum stress is a result of bending at the lower haunch radius due to the creep motion of the tank walls.
- Although the creep analyses documented in this report can be related to past Hanford concrete creep studies, research literature shows that there can be a wide variation in creep magnitude. Additional work is recommended to investigate the creep bounds with respect to Hanford tanks.
- Creep can have harmful effects on massive concrete structures that undergo a thermal cycle that produces a thermal gradient. The initial thermal stresses are relieved by creep, but cooling produces an undesirable reversal of stress and the result may be cracking. It is recommended that additional evaluation be planned to investigate thermal cycling of waste tanks with a range of thermal gradients.

- Concrete creep has been found to vary significantly with the moisture content and test chamber humidity. Moisture content and changing moisture content in concrete should be addressed initially by employing existing sealed and unsealed concrete creep tests to define lower and upper bounds for thermal creep sensitivity studies. Although Fisher et al. 1994 contains test results comparing sealed and unsealed concrete test results, it probably is necessary to also review and compare additional data to establish the bounds.
- Long-term creep structural studies (1 to 60 years) should address the creep variation bounds as defined by concrete test results. ANACAP-U software (James and Rashid 1993) was fit to test data for a maximum of two years, while limited test data exists for up to 30 years.
- Creep tests results currently used for thermal creep analyses are for stress limits close to or less than the absolute proportional creep limit. The phenomenon of creep at stresses approaching concrete crushing has not been addressed experimentally for a range of temperatures. The data do not exist that would define the creep influence on concrete failure bounds. The creep as currently characterized can address only stress and strain redistribution with time and temperature. The failure of reinforcing steel as a result of creep redistribution can be predicted; but if the concrete stresses exceed the absolute proportional creep limits, the results should be used with proper engineering caution.

9.4 SEISMIC EVALUATIONS

A summary status of the seismic evaluations of the waste tanks is provided in Becker et al. 1990. With respect to the SSTs, Becker et al. 1990 notes that only the tanks in the A and AX Tank Farms have been explicitly evaluated to 0.25 g (DOE-RL 1973). However, because all SSTs were designed to the codes and standards of their day and because the largest capacity tanks, A and AX, have been seismically evaluated for 0.25 g, it is probable that analyses would quantify the remaining SST designs to the current high-hazard design specification of 0.20 g (DOE-RL 1989). The above-referenced seismic evaluations of the A and AX tanks are documented in ARH 1972 and RHO 1978a, respectively.

With respect to the DSTs, Becker et al. 1990 concludes that all DSTs have been analyzed for 0.25 g (except the AY tanks). Only the steel primary tank of the AY tanks was evaluated to 0.25 g. However, most of the key structural components of the AY tanks are of similar design to the other DSTs and are judged to be seismically qualified to the current seismic spectra (DOE-RL 1989). Seismic evaluations of the DSTs are documented in HN 1968, ARH 1971, ARH 1974, ARH 1976, T-3036 1983, RHO 1978b, RHO 1981a, RHO 1981b, RHO 1982, and KEH 1982.

The most comprehensive of these analyses, KEH 1982, reports that the maximum seismically induced concrete stress in the secondary tank is

856.3 lbf/in². The seismic analysis considered soil-to-tank interaction of a single tank and tank-to-tank interaction of adjacent tanks. The dynamic response increase due to tank-to-tank interaction was computed to be 39 percent of the response due to soil-to-tank interaction.

At the conception of the scope of work for performing the ASA-Phase I analyses in early fiscal year 1994, it was anticipated that the results of seismic analyses planned for tank C-106 would be available for incorporation into these ASA-Phase I sensitivity evaluation. Hence, in preparation of the ASA-Phase I scope of work it was envisioned that the sensitivity of the buried waste tanks could be determined from the tank C-106 seismic analyses, so the effort to perform seismic sensitivity evaluations was not included in the ASA-Phase I effort. However, at this time tank C-106 seismic analyses results are not anticipated to be available until later in fiscal year 1994.

In addition, several technical issues relative to the determination of seismically induced loadings have recently been expressed by knowledgeable experts and external reviewers. These concerns involve both philosophical and technical issues, and will most likely not be resolved in the near future. Consequently, it is deemed prudent not to undertake additional detailed seismic evaluations of the buried waste tanks until results of the C-106 seismic analyses are available and critical technical issues are resolved.

9.5 SOIL SPRING MODELING

The ASA-Phase I evaluations of the generic tanks have employed soil springs to simulate the behavior of the soil stiffness. Soil stiffness values were predicted using the following two different approaches:

- The derivation of a simple relation for the variation of unload-reload elastic modulus with confining pressure or depth using Hanford Site test data
- The Schmertmann procedure for estimating the elastic settlement of the tank foundations using an empirical developed by Schmertmann (Schmertmann 1970, Schmertmann 1978, and Holtz 1991).

If the Hanford waste tanks are considered to have a full-diameter rigid foundation and the corresponding uniform distributed load, the first approach for estimating soil foundation stiffness results in springs of 100 lbf/in. for each square inch of foundation area (Note: This value compares to Dames & Moore recommendations). Initial ANSYS finite-element analyses with the spring value given above experienced nonlinear numerical problems and large-scale concrete cracking and crushing in the tank floor area.

The tank footings carry the soil overburden, dome, and wall weight, while the center floor region carries only the waste load. The tank floor and footing do not act as a single rigid foundation for Hanford tanks. The footing region carried the largest load (Note: The load is greater for the footing than the overburden load at the foundation depth). Calculations of soil springs assuming only the footing region as foundation resulted in estimated soil springs of 330 to 380 lbf/in. (depending on confining pressures estimations for variation) for each square inch of area. Additional ANSYS

analyses were run with soil springs of 497, 300, and 200 lbf/in for each square inch of area. The numerical problems were controlled with proper force equilibrium. Although the higher spring values seemed to solve a numerical problem, the question still remains: What are the proper equivalent foundation and floor springs for modelling Hanford waste tanks?

The alternative investigation employed the Schmertmann method using superposition of two problems. A rigid foundation with a uniform pressure equivalent to the center floor pressure gives an elastic settlement consistent with foundation springs of 100 lbf/in. for each square inch of area. The delta pressure for the footing region also gives additional settlement, if only the active footing width is applied. The two settlements were added, and the sum was divided into total footing pressure to obtain foundation springs of around 200 lbf/in. for each square inch of area. An ANSYS analysis with soil springs for the center floor and footing of 100 and 200 lbf/in. per square inch of area, respectively, yielded a solution with numerical problems; analysis activities using this approach were stopped in order to complete the ASA-Phase I evaluations in accordance with scheduled needs. The numerical problems might be overcome by using a finite stiffness value for concrete unloading after cracking.

Based on the results of these investigations it is concluded that the current ANSYS detailed modeling of reinforced concrete tanks has shown that the Hanford tank designs are sensitive to the soil modeling and the interaction of the tank floor and footing region with foundation soil needs further evaluation.

10.0 RECOMMENDATIONS

One of the primary goals of the ASA-Phase I evaluations was to assess the appropriateness of the existing IOSR and OSD controls/limits being used for operation of the tank farms and, where appropriate, to recommend changes to the current controls/limits that relate to structural integrity of the waste tanks. These recommendations are provided below in Section 10.1 OSR- AND OSD- RELATED RECOMMENDATIONS. In addition to these recommendations, several other recommendations that relate to the need for conducting additional analyses/studies are provided in Section 10.2, ADDITIONAL ANALYSES RECOMMENDATIONS.

10.1 OSR- AND OSD-RELATED RECOMMENDATIONS

10.1.1 Waste Level

The ASA-Phase I evaluations considered a waste level range from empty to full for each tank configuration. The results of these evaluations are documented in Fisher et al. 1994. The waste level considered for each of the tank configurations is as shown in Table 1. These evaluations indicate that, without consideration of thermal effects, the most severe loading condition exists for an empty tank. Under this condition the soil under the tank footing area is compressed from the heavy soil loading being transmitted

through the tank dome and wall into the footing. At the same time the soil under the tank floor supporting the waste is only compressed slightly; as a result there is a large difference in settlement of the footing with respect to the tank floor, and high bending moments occur in the tank floor adjacent to the footing. As waste is added to the tank, the relative settlements become less and the bending moments diminish accordingly. In addition, the waste hydrostatic pressure on the wall of the tank offsets the external soil pressure. The ASA-Phase I evaluations do not indicate a need to impose different liquid-level controls/limits from those now specified in the OSRs.

10.1.2 Waste Temperature

The ASA-Phase I evaluations considered a maximum waste temperature of 350 and 250 °F for the SSTs and DSTs, respectively (see Table 1). The use of these temperature values as upper bounds for conducting the sensitivity analyses was established by Becker 1994. The results of these evaluations are documented in Fisher et al. 1994. The results of the ASA-Phase I evaluations do not provide any information to suggest that these temperature limits are structurally unacceptable. Hence, it is recommended that the maximum temperature limits for the SSTs and DSTs be set at 350 and 250 °F, respectively.

Other temperature-dependent controls/limits, such as maximum tank bottom center temperature, maximum tank side wall/footing temperature, maximum dome temperature, maximum average solution temperature, and maximum average sludge temperature, are related and are functions of the maximum waste temperature. Hence, the specified maximum waste temperature control/limit is sufficient to control other temperature-related parameters within acceptable limits.

Temperature gradients resulting from changes in waste temperature or changes in heat transfer characteristics of the tank heat balance system were evaluated in Fisher et al. 1994. Based on these evaluations and because of the uncertainties associated with available thermal analyses, it is recommended that the current maximum rate of change of the waste temperature, 3 °F/day, not be changed.

Review of the OSRs and OSDs shows that current requirements for surveillance of waste temperatures varies considerably for the different waste tank configurations. A more consistent surveillance frequency could be implemented to accommodate the nonstructurally related concerns.

10.1.3 Concrete Temperature Program

Concrete temperature is a function of the waste temperature. A limit to control maximum waste temperature will also control the concrete temperature. In the ASA-Phase I evaluations, it was conservatively assumed that all of the concrete below the level of the waste was at the same temperature as the waste. The results of these evaluations are reported in Fisher et al. 1994. This conservative assumption was made to facilitate performing the structural analyses and because of the lack of definitive thermal analyses for concrete temperatures. It is recognized that detailed thermal analyses could be performed to predict more precise temperatures of the concrete in the waste

tanks; however, considerable uncertainty could still be associated with these analyses.

The actual temperature distribution within the concrete and other tank components is influenced by several thermal model parameters. Thermal conductivity of the waste, concrete and soil, layering of the waste, distribution of the heat source within the waste are but a few. Due to the uncertainties of the nature or value of these parameters it is recommended that thermal sensitivity analyses be conducted to determine their effects on the temperatures of tank components. These sensitivity results when coupled with structural evaluations will reveal which of the thermal model parameters govern the stress results. If sufficient uncertainty exists in the nature or value of the critical thermal model parameters then testing may be required to further define them.

10.1.4 Vapor-Space/Annulus Pressure Limits

The range of vapor-space pressures evaluated in the ASA-Phase I effort are identified in Table 1. This evaluation was performed by considering a vapor-space pressure of +60 in. water gauge for the SSTs and +60 in. water gauge in the primary tank in combination with -20 in. water gage in the annulus for the DSTs. These conditions resulted in the maximum pressure loadings, hence the maximum stresses. Analyses of the tanks to resist buckling were not performed as part of the ASA-Phase I effort. The detailed results of the ASA-Phase I evaluations are provided in Fisher et al. 1994.

Results of the ASA-Phase I evaluations do not indicate that the current limits on vapor-space pressure, and or annulus pressure, in the case of the DSTs, should be modified. The primary structural concern associated with the vapor space pressure limits is failure of the tank floor due to uplift or buckling of the DST primary steel tank floor or the bottom of the SST secondary steel liner. Such failures are precluded by current controls/limits which require a minimum waste level to be maintained within the tank. These current limits are considered to be appropriate; however, some simplification may be in order to facilitate tank farm operations.

10.1.5 Corrosion Control

A review has been made concerning the liner/primary tank corrosion control limits for SST and DST systems. Chemistry controls are required for water additions to SSTs for purposes of cooling and for hydrotesting and flushing of transfer lines. Water chemistry controls are also required for sluice-removal of waste if the resulting waste compositions are different than those presently used for DST corrosion control. It is also proposed that the water chemistry controls applied to significant quantities of water added to DSTs be reviewed to ensure that the corrosiveness of the overall waste chemistry as well as the surface waste chemistry is not increased.

Technical bases for corrosion control limits for both SSTs and DSTs have been examined including the most probable SST and DST corrosion degradation mechanisms. Little direct evidence of corrosion in the tanks is available. Thus, corrosion control limits must be based on laboratory studies, literature

data, sparse waste chemistry information, and some early in-tank (SST) corrosion coupon data.

Technical concerns regarding both water and waste corrosion control limits have been reviewed with respect to uninhibited water additions to tanks, the use of uninhibited water for hydrotesting and flushing of transfer lines, and possible limitations in the waste chemistries and testing conditions used to establish the corrosion data base. Methods for resolving potential corrosion concerns regarding the past use of uninhibited water in SST and DST systems center on inspections and selected corrosion test programs. A detailed discussion of the corrosion assessment is provided in Appendix C of Fisher et al. 1994.

Based on the results of the corrosion assessment it is recommended that the current corrosion control limits not be changed. However, reassessment of these limits should be made as new data become available.

Results of limited testing (Daniel et al. 1982) suggest concrete and reinforcing steel will not degrade due to chemical attack when in contact with tank wastes. This test program exposed concrete and reinforcing steel to a simulated DST slurry and to simulated salt cake. The tests were conducted at 180 °F for periods of up to three years. Post-test examination of the concrete and reinforcing steel showed no evidence of rusting, cracking, disruption of mill scale or loss of strength. However, these tests did not evaluate the condition of tank waste flowing past the test items as may be the case if a tank is leaking. Also, the effects of corrosion in a leak zone may be similar to those occurring in a splash zone where accelerated chemical attack may occur. Without more definitive testing the condition of reinforcing steel in a tank leak zone can not be determined.

10.1.6 Hydrostatic Load

The current specification limit for hydrostatic load is based on two waste parameters: waste level and specific gravity. These two parameters must be multiplied together to obtain an equivalent hydrostatic pressure loading with the units in inches water gauge. This equivalent hydrostatic pressure value is compared to the operating limits.

Because the waste level in the tanks is measured and monitored and because the specific gravity of the waste in the tanks is controlled, the requirement to control all three of these parameters appears to be unnecessary. No reason can be identified, based on structurally related concerns, to retain all three requirements. Accordingly, since hydrostatic head is not measured there is no means of verifying that the requirement is being met. The current limits on the liquid level and specific gravity of the waste contents in the tanks should not be changed.

10.1.7 Dome Loads

Because the ASA-Phase I evaluations involved sensitivity studies and were not intended to provide analyses that constitute qualification of the tanks, and because of other ongoing contemplated analyses related to tank dome

loadings, and activities being conducted to better define tank overburden soil properties, it is considered prudent at this time to not change the current tank dome loading limits. However, based on the ASA-Phase I evaluations, it is judged that the currently specified limits for concentrated loads can be used as the bases for an equivalent total uniform live load. Also, soil loading can be interchanged with uniform live loading, or vice versa, providing that proper allowances are made for the appropriate load factors. It should be noted, however, the uniform loadings (e.g., soil and uniform live loads) cannot be used as the basis for additional concentrated loadings. The asymmetric loading imposed by a crane moving across the tank dome has not been studied.

Current operating requirements for the SSTs dictate that surveys for dome deflection measurements be conducted at specified time intervals or as a result of specific operational activities. This practice is considered to be very prudent and provides valuable data on performance of the tank structure. Accordingly, because of the value of this data with respect to structural performance of the tanks with time, it is recommended that a similar program be implemented to perform dome surveys of the DSTs.

10.2 ADDITIONAL ANALYSES RECOMMENDATIONS

In the process of performing the ASA-Phase I evaluations several areas of concern were identified that could be investigated. The following is a listing of the areas of concern and a brief discussion of the issues involved.

10.2.1 Additional Soil/Foundation Interaction Analyses

While performing the ASA-Phase I evaluations, it was found that structural response of the waste tanks in the footing areas is sensitive to the soil stiffness. Difficulty was experienced in getting the ANSYS Code to converge for soil stiffness values that are consistent with those recommended for design of new structures on the Hanford Site. Higher values were used in the ASA-Phase I analyses to facilitate the completion of these studies. Hence, additional analyses should be performed to more fully assess the sensitivity of the waste tank footings to soil stiffness.

10.2.2 Evaluation Of Tanks That Have Experienced Temperatures Higher Than 350 °F

The ASA-Phase I evaluations consisted of performing sensitivity analyses of the four generic tank configurations for bounding loading conditions as specified in Becker 1994. In Becker 1994, the upper-bound temperature condition for assessment of the SSTs is 350 °F. This upper-bound temperature was selected based on SST operating thermal history data presented in Huisingh et al. 1994, along with consideration of the current SST OSR (Dougherty 1994). From a review of the SST thermal history records located to date (Huisingh et al. 1994) a few SSTs have experienced waste temperatures in excess of 350 °F and other SSTs are suspected to have had waste above 350 °F. Hence, analysis of these tanks on a case-by-case basis, with consideration given to the actual thermal history in conjunction with the other in situ conditions and historical loadings, is recommended. Accordingly, because concrete properties

are affected by increased temperatures and because the reinforced concrete in the tanks is not necessarily exposed to the same temperature as the waste, these tank-specific analyses should be accompanied by detailed tank-specific thermal analyses.

10.2.3 Criteria Development For Evaluation Of Thermal Stresses

Thermal stresses in mechanical and civil structures are normally considered as secondary stresses or as being unimportant. Current experimental results for Hanford concrete has shown that its thermal expansion coefficient is lower than the thermal- expansion of the steel reinforcing.

Additional work assessing the impact of the differences of the concrete and reinforcing thermal expansion should be continued. Studies of tanks and simple reinforced concrete problems could address the significance of thermally induced reinforcing and concrete interaction stresses. Debonding of rebar would be considered as well as concrete tension cracking. Studies would be conducted to obtain guidelines of importance for different temperature ranges.

10.2.4 Evaluation Of Defined Load Combinations

The normal SST and DST operating loads have been addressed in (Becker 1994). Those loads, when combined, form load combinations that can be applied to the tanks for evaluation; the results can be compared to allowable loads and/or stresses. The load combination(s) must be in accordance with acceptable steel and reinforced concrete code criteria. Load factors must be appropriately applied to each load type in the load combination as specified by the steel and reinforced concrete code criteria. It is recommended that a code evaluation should be performed to determine the structural adequacy of the both SSTs and DSTs.

10.2.5 Additional Thermal Analyses

Thermal analyses of the SSTs and DSTs have typically been limited to tanks having above-normal waste temperatures. Those tanks experiencing abnormal temperatures are few in comparison with the total number of SSTs and DSTs. Consequently, thermal analyses should be performed on SSTs and DSTs which represent normal operating temperatures of the majority of tanks. Having thermal analyses representative of normal operating temperatures provides thermal gradients through the steel and concrete structures and longitudinal thermal gradients that are required for a more rigorous structural evaluation..

10.2.6 Assessment Of Stresses Resulting from Differential Thermal Expansion Of Steel And Concrete

The coefficient of thermal expansion of steel is considerably greater than that of concrete. For Hanford concrete, the coefficient of thermal expansion has been found by testing to be lower than that of ordinary

concrete, or approximately 60 percent of the coefficient of thermal expansion of steel. The ASA-Phase I thermal analyses used equal thermal expansion coefficients for steel and concrete. The thermal stresses are generally considered to be self-equilibrating and cause little concrete compressive stress or steel-reinforcing-bar stress in the SSTs. Because concrete cracking is usually present in concrete structures, structural integrity should not rely on concrete tensile strengths. In fact the analysis should specify zero concrete tensile strength; past numerical difficulties have prevented using this assumption. Thermal expansion stresses for the DST may be more severe compared to those for SSTs because in the DSTs secondary and primary liners are structurally attached to the concrete with J-bolts. Because of schedule limitations only limited effort was expended during the ASA-Phase I evaluations to assess the effects of the different rates of thermal expansion on behavior of the tanks. Additional efforts are required to fully address this concern.

10.2.7 Non-Symmetric Loadings On The Dome

A concentrated load representing aboveground crane operations is a large nonsymmetric loading that typically has been evaluated as a symmetric dome loading using two-dimensional axisymmetric finite-element modelling methods. This method assumes the load is axisymmetrically applied at the apex of the dome and within a 10-ft radius. Analyses should be performed assuming the crane load is traversing across the dome at ground level using three-dimensional finite-element methods; the non-symmetric load may result in highly stressed areas in the haunch region due to shear or potential tank wall buckling. Other non-symmetric loadings or design appurtenances (e.g., penetrations) may be incorporated using three-dimensional models that produce high locally stressed conditions.

10.2.8 Additional Seismic Analyses

Seismic studies of existing Hanford waste tanks and new tank designs have shown that uncertainties exist in the state of the art for proper modeling and analysis methods for assessment of fully embedded structures (tanks) under dynamic loading. It is recommended that seismic analyses of the generic tank models developed herein be coordinated with other tank seismic study schedules to take advantage of the experience gained during resolution of these technical issues.

10.2.9 Additional Analyses To Evaluate J-Bolts

The J-bolt finite-element properties used in the ASA-Phase I evaluations were developed using local concrete stiffness and shear strength capacity estimates. These properties should be verified with detailed finite-element models and test data. Also, local J-bolt interaction with the secondary liner and resulting localized strain levels should be investigated.

10.2.10 Comparisons Of Analyses Results With Those From Other Codes

The ANSYS tank models used for conducting the ASA-Phase I should be benchmarked with alternate evaluation computer programs such as ABAQUS (HKS 1989). Simplified reinforced concrete test cases should be modeled with both ANSYS and ABAQUS for comparison and verification of Hanford concrete modeling and to quantify limitations of the analytical methods. Estimates of solution accuracy should also be developed.

10.2.11 Structural Degradation Mechanisms

Thermal degradation of concrete properties was considered in the ASA-Phase I sensitivity evaluations. Other structural degradation mechanisms were not considered. Hence, it is recommended that other types of structural degradation mechanisms be accounted for in the process of performing subsequent qualifying structural analyses. In situ testing may be necessary to quantify the effects of specific degradation mechanisms.

11.0 REFERENCES

- Aguirre, H. Jr., 1994, *Aging Waste Facility Interim Operational Safety Requirements*, WHC-SD-WM-OSR-004, Rev. 0, Westinghouse Hanford Company, Richland, Washington.
- ARH, 1971, *Seismic Analysis of Underground Waste Storage Tanks 241-AZ-101 and -102 at Hanford, Washington*, ARH-R-85, John A. Blume and Associates, Engineers, for the Atlantic-Richfield Hanford Company, Richland, Washington.
- ARH, 1972, *Final Report, Strength and Stress Analysis for Waste Tank Structures at Hanford, Washington*, ARH-R-120, K. P. Milbrandt, Illinois Institute of Technology, Atlantic-Richfield Hanford Company, Richland, Washington.
- ARH, 1974, *Analysis of Underground Waste Storage Tanks 241-SY at Hanford, Washington*, ARH-R-172, URS/John A. Blume and Associates, Engineers, Atlantic-Richfield Hanford Company, Richland, Washington.
- ARH, 1976, *Analysis of Underground Waste Storage Tanks 241-AW at Hanford, Washington*, ARH-R-219, URS/John A. Blume and Associates, Engineers, Vitro Engineering, Richland, Washington.
- Becker, D. L., R. A. Marusich, J. A. Ryan, D. A. Smith, A. M. Tallman, G. R. Wagenblast, 1990, *Summary Status on the Seismic Evaluations of Hanford Site Radioactive Waste Storage Tanks*, WHC-EP-0373, Westinghouse Hanford Company, Richland, Washington.
- Becker, D. L., 1994, *Single- and Double-Shell Tanks Load Report for Accelerated Safety Analysis*, WHC-SD-ES-286, Rev. 0, Westinghouse Hanford Company, Richland, Washington.

- Daniel, J. I., D. C. Stark, P. H. Kaar, 1982, *Long-Term Effects of Waste Solutions on Concrete and Reinforcing Steel*, RHO-RE-CR-8 P, Construction Technology Laboratories, Portland Cement Association, Skokie, Illinois.
- Dougherty, L. R., 1994, *Single-Shell Tank Interim Operational Safety Requirements*, WHC-SD-WM-OSR-005, Rev. 0, Westinghouse Hanford Company, Richland, Washington.
- DOE-RL, 1973, "Standard Design Criteria (SDC) 4.1," *Hanford Plant Standards*, Rev. 6, DOE-RL Order 6430.1B, U.S. Department of Energy-Richland Operations Office, Richland, Washington.
- DOE-RL, 1989, "Standard Design Criteria (SDC) 4.1," *Hanford Plant Standards*, Rev. 11, DOE-RL Order 6430.1B, U.S. Department of Energy-Richland, Operations Office, Richland, Washington.
- DOE-RL, 1993, "Standard Design Criteria (SDC) 4.1," *Hanford Plant Standards*, Rev. 12, DOE-RL Order 6430.1B, U. S. Department of Energy-Richland, Operations Office, Richland, Washington.
- Fisher, T. W., L. L. Hyde, C. J. Moore, S. W. Peterson, W. W. Chen, 1994, *Structural Sensitivity Evaluations of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I*, WHC-SD-WM-DA-150, Rev. 0, Westinghouse Hanford Company, Richland, Washington.
- Fisher, T. W., and D. J. Shank, 1994, *Single- and Double-Shell Waste Tank Design Comparisons at Hanford, Washington*, WHC-SD-WM-TI-598, Rev. 0, Westinghouse Hanford Company, Richland, Washington.
- Heubach, E. C., II, 1994, *Double-Shell Tank Interim Operational Safety Requirements*, WHC-SD-WM-OSR-016, Rev. 0, Westinghouse Hanford Company, Richland, Washington.
- Holtz, R. D., 1991, "Stress Distribution and Settlement of Shallow Foundations," *Foundation Engineering Handbook*, Second Edition, H. Y. Fang, Editor, Chapter 5, Van Nostrand Reinhold, New York, N. Y. pp. 179-180.
- HN, 1968, *Final Report, Strength and Stress Analysis for Waste Tank Structures at Hanford, Washington*, HN-197, Holmes and Narver, Inc., Atlantic-Richfield Hanford Company, Richland, Washington.
- Huisingh, J. S., N. D. Ha, and B. D. Flanagan, 1994, *Maximum Surface Level and Temperature Histories for Hanford Waste Tanks*, WHC-SD-WM-TI-591, Rev. 0, Westinghouse Hanford Company, Richland, Washington.
- James, R. J., and Y. R. Rashid, 1993, *ANACAP-U Theory Manual Supplement For Hanford Concrete*, ANATECH Research Corporation, ANA-QA-124, Rev. 2, Westinghouse Hanford Company, Richland, Washington.
- KEH, 1982, *Stress Analysis and Structural Analysis 241-AP Tank Farm*, KEH R-82-21, Kaiser Engineers Hanford Company, Richland, Washington.

- Leach, C. E., 1993, *Hanford Site Tank Farm Facilities Interim Safety Basis*, WHC-SD-WM-ISB-001, Rev. 0, Westinghouse Hanford Company, Richland, Washington.
- NUREG, 1978, *Safety Evaluation Report for the United States Department of Energy Fast Flux Test Facility*, NUREG-0358, U.S. Nuclear Regulatory Commission, Washington, D.C.
- Neville, A. M., 1970, *Creep of Concrete: Plain, Reinforced, and Prestressed*, American Elsevier Publishing Company, New York, New York.
- OSD, 1989, *Operating Specifications for Aging-Waste Operations In 241-AY and 241-AZ*, OSD-T-151-00017, Revision D-0, Westinghouse Hanford Company, Richland, Washington.
- OSD, 1992a, *Operating Specifications for Single-Shell Waste Storage Tanks*, OSD-T-151-00013, Revision D-1, Westinghouse Hanford Company, Richland, Washington.
- OSD, 1992b, *Operating Specifications for the 241-AN, AP, AW, AY, AZ, and SY Tank Farms*, OSD-T-151-00007, Revision D-1, Westinghouse Hanford Company, Richland, Washington.
- Peterson, W.S., 1994, *Evaluation of Strength and Modulus Degradation due to Temperature Effects on Hanford Concrete*, WHC-SD-WM-DA-153, Westinghouse Hanford Company, Richland, Washington.
- RHO, 1978a, *Analysis of Underground Waste Storage Tank 241-AX at Hanford, Washington*, RHO-R-6, URS/John A. Blume and Associates, Engineers, Vitro Engineering, Richland, Washington.
- RHO, 1978b, *Additional Analyses of Underground Waste Storage Tanks 241-AW, Hanford, Washington*, RHO-C-17, URS/John A. Blume and Associates, Engineers, Vitro Engineering, Richland, Washington.
- RHO, 1981, *Additional Analysis of Underground Waste Storage Tanks 241-SY, Hanford, Washington*, RHO-C-59, URS/John A. Blume and Associates, Engineers, Vitro Engineering, Richland, Washington.
- RHO, 1981, *A Comprehensive Summary of the Analysis of the 241-AW Underground Waste Storage Tanks, Hanford, Washington*, RHO-C-60, URS/John A. Blume and Associates, Engineers, Vitro Engineering, Richland, Washington.
- RHO, 1982, *Thermal Creep and Ultimate Load Analyses of the 241-AY/AZ Reinforced Concrete Underground Waste Storage Tanks*, RHO-SD-RE-TI-041, F. R. Vollert, Rockwell Hanford Operations, Richland, Washington.
- Schmertmann, J. H., 1970, "Static Cone to Compute Static Settlement Over Sand," *Journal of Soil Mechanics and Foundation Division*, ASCE, 96, No. SM3, pp. 1011-1043, New York, New York.
- Schmertmann, J. H., 1978, *Guidelines For Cone Penetration Test Performance, and Design*, Federal Highway Administration, Report FHWA-TS-78-209, Washington, D.C.

SDRC, 1993, *I-DEAS Master Series 1.1*, Structural Dynamics Research Corporation, Milford, Ohio.

Swanson, 1992, *ANSYS User's Manual For Revision 5.0*, Updo DN-R300:50-1, Swanson Analysis System, Inc., Houston, Pennsylvania.

T-3036, 1983, *Manufacturer's Design Report for Kaiser Engineers Hanford Company, Richland, Washington, and the United States Department of Energy Concrete Encased Steel Tanks for Tank Farm No. 241-AP, 200-East Area, Hanford Works, Richland, Washington, American Bridge Division, Engineering United States Steel Corporation, Pittsburgh, Pennsylvania.*

WHC, 1989, *Evaluation of Seismic Hazard for Non-Reactor Facilities, Hanford Reservation, Hanford Washington, WHC-MR-0023*, Woodward-Clyde Consultants, Westinghouse Hanford Company, Richland, Washington.

Figure 1. Typical Waste Tank Configurations Evaluated
by Accelerated Safety Analyses - Phase I.

500,000 Gallon Single-Shell Tank
Tank Farms: B, BX, C, T, U

750,000 Gallon Single-Shell Tank
Tank Farms: BY, S, TX, TY

1,000,000 Gallon Single-Shell Tank
Tank Farms: A*, AX*, SX

1,000,000 Gallon Double-Shell Tank
Tank Farms: AN, AP, AW, AY, AZ, SY

* A and AX have flat bottoms

Figure 2. Finite-Element Model of the 500,000-Gallon
Single-Shell Tank.

Figure 3. Finite-Element Model of the 500,000-Gallon
Single-Shell Tank Haunch Region.

Figure 4. Finite-Element Model of the 500,000-Gallon
Single-Shell Tank Wall/Footing Region.

Figure 5. Finite-Element Model of the 750,000-Gallon
Single-Shell Tank.

Figure 6. Finite-Element Model of the 750,000-Gallon
Single-Shell Tank Haunch Region.

Figure 7. Finite-Element Model of the 750,000-Gallon
Single-Shell Tank Wall/Footing Region.

Figure 8. Finite-Element Model of the 1,000,000-Gallon
Single-Shell Tank.

Figure 9. Finite-Element Model of the 1,000,000-Gallon
Single-Shell Tank Haunch Region.

Figure 10. Finite-Element Model of the 1,000,000-Gallon
Single-Shell Tank Wall/Footing Region.

Figure 11. Finite-Element Model of the 1,000,000-Gallon
Double-Shell Tank.

Figure 12. Finite-Element Model of the 1,000,000-Gallon
Double-Shell Tank Haunch Region.

Figure 13. Finite-Element Model of the 1,000,000-Gallon
Double-Shell Tank Wall/Footing Region.

Table 1. Range of Load Values For Waste Tank Accelerated
Safety Analyses - Phase I.

Loading Type	Tank Configuration			
	Single-Shell Tanks			Double-Shell Tanks
	500,000 Gallon	750,000 Gallon	1,000,000 Gallon	1,000,000 Gallon
Soil Cover at Dome Apex (ft)	5 - 10	5 - 10	5 - 10	6 - 9
Soil Density (lb/ft ³)	101	101 - 130	101 - 130	101 - 130
Uniform Live Load ⁽¹⁾ (lb/ft ²)	100	100	100	100
Concentrated Live ⁽²⁾ Load (tons)	100	100	100	100
Vapor Space Pressure (in. wg.)	-15 +60	-15 +60	-15 +60	-12 ⁽³⁾⁽⁴⁾ +60 ⁽³⁾⁽⁴⁾
Waste Level	Empty-Full	Empty-Full	Empty-Full	Empty-Full
Waste Specific Gravity	1.0 - 2.0	1.0 - 2.0	1.0 - 2.0	1.0 - 2.0
Maximum Waste Temperature (°F)	350	350	350	250
Seismic Load (Peak Free Field Horizontal and Ground Acceleration)	0.20 g	0.20 g	0.20 g	0.20 g

- Note: 1. This load is to be distributed over the vertically projected surface area of the tank dome.
 2. This load is to be distributed over a 10-ft radius area concentric to the apex of the dome.
 3. To be considered in conjunction with a pressure in the annulus of 0 to -20 in. wg. with respect to atmospheric.
 4. Double-shell primary tank vapor space pressure is with respect to the annulus pressure.

Table 2. 500,000-Gallon Single-Shell Tank
Maximum Concrete Results.

MAXIMUM CONCRETE PRINCIPAL STRESS (lbf/in ²)					
LOAD STUDY CASE	COMPONENT				
	Dome	Upper Haunch	Wall	Lower Knuckle	Floor
Soil	-496	-1289*	-942	-780	-507
Uniform/Concentrated	-425	-934	-653	-552	-335
Vapor Pressure	-416	-1053	-724	-651	-387
Waste Level	-272	-744	-516	-657	-404
Temperature	-366	-714	-493	-615	-345

Note: The maximum concrete stress (*) results from loading case S3 (a 10-ft soil depth and 101 lbf/ft³ density) as tabulated in Table 30. Load cases corresponding to other stress values are given in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994).

Table 3. 500,000-Gallon Single-Shell Tank
Maximum Radial Steel Results.

MAXIMUM RADIAL STEEL EQUIVALENT STRESS (lbf/in ²)				
LOAD STUDY CASE	COMPONENT			
	Dome	Upper Haunch	Wall	Lower Knuckle
Soil	3285	10407	26195 *	10139
Uniform/Concentrated	2678	7258	19826	7355
Vapor Pressure	2642	8280	22053	8604
Waste Level	2007	4438	12536	8199
Temperature	3693	5053	11499	8199

Note: The maximum reinforcing steel stress (*) results from loading case S3 (a 10-ft soil depth and 101 lbf/ft³ density) as tabulated in Table 30. Load cases corresponding to other stress values are given in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994).

Table 4. 500,000-Gallon Single-Shell Tank
Maximum Hoop Steel Results.

MAXIMUM HOOP STEEL EQUIVALENT STRESS (1bf/in ²)			
LOAD STUDY CASE	COMPONENT		
	Dome	Upper Haunch	Wall
Soil	4349	6974	6938
Uniform/Concentrated	2667	4181	4138
Vapor Pressure	2507	4316	4252
Waste Level	1372	1831	1784
Temperature	3440	7332	7721

Note: The maximum reinforcing steel stress (see Table 3) results from loading case S3 (a 10-ft soil depth and 101 lbf/ft³ density) as tabulated in Table 30. Load cases corresponding to other stress values are given in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994).

Table 5. 500,000-Gallon Single-Shell Tank
Concrete Principal Stress Change.

CONCRETE PRINCIPAL STRESS CHANGE (lbf/in ²)					
SENSITIVITY VARIATION/LOADING CHANGE	COMPONENT				
	Dome	Upper Haunch	Wall	Lower Knuckle	Floor
Soil Depth (+ 10 ft)	-480	-1240	-1110	-440	-430
Soil Density (+29 lbf/ft ³)	-107	-271	-173	-139	-109
Uniform Load (+100 lbf/ft ²)	-32	-89	-64	-33	-17
Concentrated Load (+100 tons)	-226	-41	-31	-18	-90
Vapor Pressure (+2.2 lbf/in ²)	-53	70	79	64	51
Waste Level (+6 ft)	-2	-24	-36	-50	-37
Specific Gravity (+1.0)	-6	-3	-6	18	29
Temperature (+50 °F)	-73	0	-102	3	4

Table 6. 500,000-Gallon Single-Shell Tank
Radial Steel Equivalent Stress Change.

RADIAL STEEL EQUIVALENT STRESS CHANGE (lbf/in ²)				
SENSITIVITY VARIATION/LOADING CHANGE	COMPONENT			
	Dome	Upper Haunch	Wall	Lower Knuckle/Floor
Soil Depth (+ 10 ft)	3640	25990	20600	16540
Soil Density (+29 lbf/ft ³)	666	5463	7639	1365
Uniform Load (+100 lbf/ft ²)	215	684	1967	475
Concentrated Load (+100 tons)	1317	296	846	256
Vapor Pressure (+2.2 lbf/in ²)	-433	-664	-547	-90
Waste Level (+6 ft)	14	235	926	4117
Specific Gravity (+1.0)	37	27	182	65
Temperature (+50 °F)	+755	1223 ⁽¹⁾	1134	184

(1) This change resulted from a maximum sensitivity change for 200 °F

Table 7. 500,000-Gallon Single-Shell Tank
Hoop Steel Equivalent Stress Change.

HOOP STEEL EQUIVALENT STRESS CHANGE (lbf/in ²)			
SENSITIVITY VARIATION/LOADING CHANGE	COMPONENT		
	Dome	Upper Haunch	Wall
Soil Depth (+10 ft)	9290	13610	13750
Soil Density (+29 lbf/ft ³)	798	1417	1380
Uniform Load (+100 lbf/ft ²)	275	466	469
Concentrated Load (+100 tons)	1300	300	300
Vapor Pressure (+2.2 lbf/in ²)	-396	-847	-431
Waste Level (+6 ft)	0	-3	507
Specific Gravity (+1.0)	96	187	285
Temperature (+50 °F)	1090	2547	2779

Table 8. 750,000-Gallon Single-Shell Tank
Maximum Concrete Results.

MAXIMUM CONCRETE PRINCIPAL STRESS (1bf/in ²)					
LOAD STUDY CASE	COMPONENT				
	Dome	Upper Haunch	Wall	Lower Knuckle	Floor
Soil	-599	-1420*	-853	-964	-441
Uniform/Concentrated	-629	-1070	-696	-548	-228
Vapor Pressure	-466	-1147	-728	-626	-251
Waste Level	-295	-633	-592	-669	-301
Temperature	-359	-537	-488	-507	-213

Note: The maximum concrete stress (*) results from loading case S5 as tabulated in Table 30. Load cases corresponding to other stress values are given in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994).

Table 9. 750,000-Gallon Single-Shell Tank
Maximum Radial Steel Results.

MAXIMUM RADIAL STEEL EQUIVALENT STRESS (1bf/in ²)					
LOAD STUDY CASE	COMPONENT				
	Dome	Upper Haunch	Wall	Lower Knuckle	Floor
Soil	4352	15729	34548*	6232	3026
Uniform/Concentrated	2882	10253	28501	1087	1071
Vapor Pressure	3012	10950	29583	1157	1309
Waste Level	2251	6744	5579	1155	1691
Temperature	3623	4288	5632	1642	1727

Note: The maximum reinforcing steel stress (*) results from loading case S5 as tabulated in Table 30. Load cases corresponding to other stress values are given in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994).

Table 10. 750,000-Gallon Single-Shell Tank
Maximum Hoop Steel Results.

MAXIMUM HOOP STEEL EQUIVALENT STRESS (1bf/in ²)			
LOAD STUDY CASE	COMPONENT		
	Dome	Upper Haunch	Wall
Soil	4839	7125	6232
Uniform/Concentrated	4652	5735	5503
Vapor Pressure	3356	5375	7508
Waste Level	2095	1374	2006
Temperature	3233	7425	8076

Note: The maximum reinforcing steel stress (see Table 9) results from loading case S5 as tabulated in Table 30. Load cases corresponding to other stress values are given in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994).

Table 11. 750,000-Gallon Single-Shell Tank
Concrete Principal Stress Change.

CONCRETE PRINCIPAL STRESS CHANGE (lbf/in ²)					
SENSITIVITY VARIATION/LOADING CHANGE	COMPONENT				
	Dome	Upper Haunch	Wall	Lower Knuckle	Floor
Soil Depth (+ 10 ft)	-530	-1320	-620	-370	-160
Soil Density (+29 lbf/ft ³)	-66	-141	-86	-214	-87
Uniform Load (+100 lbf/ft ²)	-40	-90	-60	-30	-10
Concentrated Load (+100 tons)	-300	-70	-50	-10	+5
Vapor Pressure (+2.2 lbf/in ²)	-68	78	62	53	26
Waste Level (+9 ft)	0	0	32	24	14
Specific Gravity (+1.0)	-80	-235	-149	34	28
Temperature (+50 °F)	-85	17	-97	14	11

Table 12. 750,000-Gallon Single-Shell Tank
Radial Steel Equivalent Stress Change.

RADIAL STEEL EQUIVALENT STRESS CHANGE (lbf/in ²)					
SENSITIVITY VARIATION/LOADING CHANGE	COMPONENT				
	Dome	Upper Haunch	Wall	Lower Knuckle	Floor
Soil Depth (+10 ft)	5070	12030	10550	1660	1340
Soil Density (+29 lbf/ft ³)	513	3043	2930	314	29647
Uniform Load (+100 lbf/ft ²)	220	890	2690	60	70
Concentrated Load (+100 tons)	1150	720	2390	40	40
Vapor Pressure (+2.2 lbf/in ²)	-510	-450	-455	-60	-125
Waste Level (+9 ft)	10	-7	316	61	-101
Specific Gravity (+1.0)	527	8176	13277	35	-235
Temperature (+50 °F)	856	245	607	188	121

Table 13. 750,000-Gallon Single-Shell Tank
Hoop Steel Equivalent Stress Change.

HOOP STEEL EQUIVALENT STRESS CHANGE (lbf/in ²)			
SENSITIVITY VARIATION/LOADING CHANGE	COMPONENT		
	Dome	Upper Haunch	Wall
Soil Depth (+10 ft)	9990	1270	11310
Soil Density (+29 lbf/ft ³)	474	262	664
Uniform Load (+100 lbf/ft ²)	330	520	500
Concentrated Load (+100 tons)	2160	750	750
Vapor Pressure (+2.2 lbf/in ²)	-537	-890	-237
Waste Level (+9 ft)	0	111	345
Specific Gravity (+1.0)	1636	2886	2872
Temperature (+50 °F)	1155	2985	3479

Table 14. 1,000,000-Gallon Single-Shell Tank
Maximum Concrete Results.

MAXIMUM CONCRETE PRINCIPAL STRESS (lbf/in ²)				
LOAD STUDY CASE	COMPONENT			
	Dome	Upper Haunch	Wall	Lower Knuckle/Floor
Soil	-589	-859	-976	-1251*
Uniform/Concentrated	-559	-505	-414	-453
Vapor Pressure	-395	-587	-476	-545
Waste Level	-282	-509	-653	-825
Temperature	-285	-482	-521	-634

Note: The maximum concrete stress (*) results from loading case S5 as tabulated in Table 30. Load cases corresponding to other stress values are given in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994).

Table 15. 1,000,000-Gallon Single-Shell Tank
Maximum Radial Steel Results.

MAXIMUM RADIAL STEEL EQUIVALENT STRESS (lbf/in ²)					
LOAD STUDY CASE	COMPONENT				
	Dome	Upper Haunch	Wall	Lower Knuckle	Floor
Soil	4069	6733*	5963	5470	1392
Uniform/Concentrated	3687	3987	3520	1847	211
Vapor Pressure	2583	4502	3936	2218	283
Waste Level	1832	3642	3501	3391	881
Temperature	2998	3474	4854	2846	1030

Note: The maximum reinforcing steel stress (*) results from loading case S5 as tabulated in Table 30. Load cases corresponding to other stress values are given in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994).

Table 16. 1,000,000-Gallon Single-Shell Tank
Maximum Hoop Steel Results.

MAXIMUM HOOP STEEL EQUIVALENT STRESS (1bf/in ²)				
LOAD STUDY CASE	COMPONENT			
	Dome	Upper Haunch	Wall	Lower Knuckle
Soil	4098	2269	3115	2617
Uniform/Concentrated	3711	1608	1547	913
Vapor Pressure	2600	1634	1548	1057
Waste Level	1804	1164	2066	1674
Temperature	2737	5066	6337	1578

Note: The maximum reinforcing steel stress (see Table 15) results from loading case S5 as tabulated in Table 30. Load cases corresponding to other stress values are given in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994).

Table 17. 1,000,000-Gallon Single-Shell Tank
Concrete Principal Stress Change.

CONCRETE PRINCIPAL STRESS CHANGE (lbf/in ²)				
SENSITIVITY VARIATION/LOADING CHANGE	COMPONENT			
	Dome	Upper Haunch	Wall	Lower Knuckle
Soil Depth (+10 ft)	-515	-512	-414	-446
Soil Density (+29 lbf/ft ³)	-60	-97	-133	-164
Uniform Load (+100 lbf/ft ²)	-39	-42	-35	-28
Concentrated Load (+100 tons)	-253	-12	-9	-13
Vapor Pressure (+2.2 lbf/in ²)	99	109	37	38
Waste Level (+12 ft)	0	-48	-224	-320
Specific Gravity (+1.0)	0	18	8	70
Temperature (+50 °F)	-29	-233	-135	-20

Table 18. 1,000,000-Gallon Single-Shell Tank
Radial Steel Equivalent Stress Change.

RADIAL STEEL EQUIVALENT STRESS CHANGE (lbf/in ²)					
SENSITIVITY VARIATION/LOADING CHANGE	COMPONENT				
	Dome	Upper Haunch	Wall	Lower Knuckle	Floor
Soil Depth (+10 ft)	15990	4620	4120	2160	310
Soil Density (+29 lbf/ft ³)	390	812	823	-815	277
Uniform Load (+100 lbf/ft ²)	250	301	271	116	2
Concentrated Load (+100 tons)	1600	100	80	50	0
Vapor Pressure (+2.2 lbf/in ²)	-669	-1016	-221	-218	-60
Waste Level (+12 ft)	0	256	918	-199	562
Specific Gravity (+1.0)	-1.6	-150	288	-323	-395
Temperature (+50 °F)	301	569	2659	145	136

Table 19. 1,000,000-Gallon Single-Shell Tank
Hoop Steel Equivalent Stress Change.

HOOP STEEL EQUIVALENT STRESS CHANGE (lbf/in ²)				
SENSITIVITY VARIATION/LOADING CHANGE	COMPONENT			
	Dome	Upper Haunch	Wall	Lower Knuckle
Soil Depth (+10 ft)	3950	2020	910	790
Soil Density (+29 lbf/ft ³)	379	171	605	427
Uniform Load (+100 lbf/ft ²)	263	189	188	34
Concentrated Load (+100 tons)	1600	100	90	10
Vapor Pressure (+2.2 lbf/in ²)	-776	-302	-145	-158
Waste Level (+12 ft)	5	65	1007	420
Specific Gravity (+1.0)	-3.7	60	150	-466
Temperature (+50 °F)	235	1652	1695	81

Table 20. 1,000,000-Gallon Double-Shell Tank
Maximum Concrete Stress.

MAXIMUM CONCRETE STRESS (1bf/in ²)					
LOAD STUDY CASE	COMPONENT				
	Dome	Upper Haunch	Wall	Floor	Refrac Slab
Soil	-425	-519	-806 *	-353	-55
Uniform/Concentrated Load	-319	-144	-159	-177	-64
Vapor/Annulus Pressure	-323	-428	-564	-277	-52
Waste Level	-326	-499	-556	-378	-77
Temperature	-441	-580	-650	-288	-65
Creep Years (50 years)	-159	-221	-244	-234	-54
Subgrade Modulus	-258	-386	-529	-365	-59

Note: The maximum stress for the concrete (*), corresponds to load case S5 as tabulated in Table 30. Load cases corresponding to other stress values are given in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994).

Table 21. 1,000,000-Gallon Double-Shell Tank
Maximum Radial Steel Stress.

MAXIMUM RADIAL STEEL EQUIVALENT STRESS (1bf/in ²)				
LOAD STUDY CASE	COMPONENT			
	Dome	Upper Haunch	Wall	Floor
Soil	2842	3641	1916	1908
Uniform/Concentrated Load	1960	1074	1134	811
Vapor/Annulus Pressure	2125	2871	1562	1669
Waste Level	2152	3059	1648	23838*
Temperature	3899	5187	3023	2442
Creep Years (50 years)	7109	10626	5592	7838
Subgrade Modulus	1707	2515	1389	19565

Note: The maximum stress for the reinforcing steel (*) corresponds to load case L5 as tabulated in Table 30. Load cases corresponding to other stress values are given in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994).

Table 22. 1,000,000-Gallon Double-Shell Tank
Maximum Hoop Steel Stress.

MAXIMUM HOOP STEEL EQUIVALENT STRESS (lbf/in ²)				
LOAD STUDY CASE	COMPONENT			
	Dome	Upper Haunch	Wall	Floor
Soil	2443	1406	7075	957
Uniform/Concentrated Load	1958	869	768	189
Vapor/Annulus Pressure	1789	1005	4617	741
Waste Level	1798	1133	4491	847
Temperature	2691	4356	8001	977
Creep (50 years)	6160	3334	17678	1833
Subgrade Modulus	1415	771	4195	882

Note: The maximum stresses for the reinforcing steel (see Table 21) corresponds to load cases L5, as tabulated in Table 30. Load cases corresponding to other stress values are given in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994).

Table 23. 1,000,000-Gallon Double-Shell Tank
Maximum Tank Steel Stress.

MAXIMUM TANK STEEL EQUIVALENT STRESS (lbf/in ²)		
LOAD STUDY CASE	COMPONENT	
	Primary Tank	Secondary Tank
Soil	8699	16793**
Uniform/Concentrated Load	16705	1338
Vapor/Annulus Pressure	10902	12127
Waste Level	16765*	12439
Temperature	10182	15095
Creep (50 years)	10705	31455
Subgrade Modulus	15921	12197

Note: The maximum stresses for the primary tank (*) and secondary steel tank liner (**) correspond to load cases L1 and S5, respectively, as tabulated in Table 30. Load cases corresponding to other stress values are given in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994).

Table 24. 1,000,000 Gallon Double-Shell Tank
Maximum J-Bolt Forces.

J-BOLT FORCE (kips)				
LOAD STUDY CASE	COMPONENT			
	J-Bolt Axial Force	J-Bolt Shear Force		
		Dome	Haunch	Wall
Soil	-12.2	41.5	-18.5	18.5
Uniform/Concentrated Load	8.6	33.1	-16.9	8.1
Vapor/Annulus Pressure	-8.4	41.5	-18.5	18.5
Waste Level	-8.5	41.5	-18.5	18.5
Temperature	-8.5	41.5	-18.5	18.5
Creep (50 years)	-17.2	41.5	-18.5	18.5
Subgrade Modulus	-7.5	41.5	-18.5	18.5

- Notes: 1. J-bolt yield force = 41.5 kips.
 2. Load cases corresponding to other stress values are given in the detailed ASA-Phase I report, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Tanks for Accelerated Safety Analysis - Phase I* (Fisher et al. 1994).

Table 25. 1,000,000-Gallon Double-Shell Tank
Concrete Principal Stress Change.

CONCRETE PRINCIPAL STRESS CHANGE ⁽¹⁾					
LOADING CHANGE	COMPONENT				
	Dome	Upper Haunch	Wall	Floor	Refrac Slab
Soil Depth (+9 ft)	-324	-252	-81	-180	-11
Soil Density (+30 lbf/ft ³)	-61	-84	-127	-58	-3
Uniform Load (+100 lbf/ft ²)	-28	-22	-13	-16	0
Concentrated Load (+100 tons)	-233	-9	-5	-8	-1
Vapor Pressure (+60 in wg.)	-81	-51	+1	-43	-10
Waste Level (-30 ft)	+2	+18	-5	-110	+22
Specific Gravity (+1.0)	-3	-20	6	-7	-25
Temperature (+180 °F)	-83	-92	-86	+10	-13
Creep (+50 year)	129	148	307	36	-19
Annulus Pressure (-20 in. wg.)	+1	-3	-13	-5	+3
Rankine Coefficient (+.2)	-2	-23	-231	-14	0
Subgrade Modulus (-300 lbf/in ³)	2	5	-19	-30	-5

(1) Stress (lbf/in²)

Table 26. 1,000,000-Gallon Double-Shell Tank
Radial Steel Stress Change.

RADIAL STEEL EQUIVALENT STRESS CHANGE ⁽¹⁾				
LOADING CHANGE	COMPONENT			
	Dome	Upper Haunch	Wall	Floor
Soil Depth (+9 ft)	2151	2295	1062	720
Soil Density (+30 lbf/ft ³)	402	594	255	384
Uniform Load (+100 lbf/ft ²)	147	171	89	90
Concentrated Load (+100 tons)	1360	93	40	42
Vapor Pressure (-60 in. wg.)	504	402	240	87
Waste Level (-30 ft)	-11	-132	-82	18900
Specific Gravity (+1.0)	22	149	89	-252
Temperature (+180 °F)	1775	2070	1462	459
Creep (+50 year)	4107	6574	2826	5258
Annulus Pressure (-20 in. wg.)	-7	+2	-7	-39
Rankine Coefficient (+.2)	0	6	-136	-91
Subgrade Modulus (-300 lbf/in ³)	-9	-42	-21	18300

(1) Stress (lbf/in²)

Table 27. 1,000,000-Gallon Double-Shell Tank
Hoop Steel Stress Change.

HOOP STEEL EQUIVALENT STRESS CHANGE ⁽¹⁾				
LOADING CHANGE	COMPONENT			
	Dome	Upper Haunch	Wall	Floor
Soil Depth (+9 ft)	1908	1206	639	270
Soil Density (+30 lbf/ft ³)	342	129	1260	177
Uniform Load (+100 lbf/ft ²)	189	133	118	-5
Concentrated Load (+100 tons)	1395	76	51	-3
Vapor Pressure (-60 in. wg.)	468	360	-66	50
Waste Level (-30 ft)	-5	-93	60	72
Specific Gravity (+1.0)	7	123	-67	-136
Temperature (+180 °F)	902	3348	3528	130
Creep (+50 year)	3479	2164	9164	822
Annulus Pressure (-20 in. wg.)	-1	-40	127	0
Rankine Coefficient (+.2)	17	-303	2440	-126
Subgrade Modulus (-300 lbf/in ³)	-2	-29	171	330

(1) Stress (lbf/in²)

Table 28. 1,000,000-Gallon Double-Shell Tank
Tank Steel Equivalent Stress Change.

TANK STEEL EQUIVALENT STRESS CHANGE ⁽¹⁾		
Loading Change	COMPONENT	
	Primary Tank	Secondary Tank
Soil Depth (+9 ft)	828	3798
Soil Density (+30 lbf/ft ³)	144	2766
Uniform Load (+100 lbf/ft ²)	.06	205
Concentrated Load (+100 tons)	2	65
Vapor Pressure (-60 in. wg.)	-1698	420
Waste Level (-30 ft)	-1290	267
Specific Gravity (+1.0)	8181	-305
Temperature (+180 °F)	1602	2970
Creep (+50 year)	3622	15831
Annulus Pressure (-20 in. wg.)	624	-92
Rankine Coefficient (+.2)	-153	3402
Subgrade Modulus (-300 lbf/in ³)	33	1650

(1) Stress (lbf/in.²)

Table 29. 1,000,000-Gallon Double-Shell Tank
J-Bolt Force Change.

J-BOLT FORCE CHANGE ⁽¹⁾				
Load Change	COMPONENT			
	J-Bolt Axial Force	J-Bolt Shear Force		
		Dome	Haunch	Wall
Soil Depth (+9 ft)	-2.25	0 (Yield)	0 (Yield)	0 (Yield)
Soil Density (+30 lbf/ft ³)	-1.97	0 (Yield)	0 (Yield)	0 (Yield)
Uniform Load (+100 lbf/ft ²)	-.1	2	-3	.9
Concentrated Load (100 ton)	.1	1.2	-.8	.5
Vapor Pressure (-60 in. w. g.)	-.18	+6	-1.32	2.1
Waste Level (-30 ft)	-.18	-10.2	-.9	0 (Yield)
Specific Gravity (+1.0)	-.1	11.5	-.5	0 (Yield)
Temperature (+180 °F)	1.4	-28.8	0 (Yield)	0 (Yield)
Creep (+50 year)	-10.8	0	0	0
Annulus Pressure (-20 in.wg.)	+0.6	1.7	-.7	.3
Rankine Coefficient (+.2)	-3.2	0 (Yield)	0 (Yield)	0 (Yield)
Subgrade Modulus (-300 lbf/in ³)	-.7	0 (Yield)	0 (Yield)	0 (Yield)

(1) Force (kips)

Table 30. Maximum Stress Load Cases.

Tank	Load Case	Load								
		Waste Level (ft)	Waste Specific Gravity	Waste Temperature (°F)	Soil ⁽¹⁾ Overburden (ft)	Soil Density (lb/ft ³)	Dome Vapor Space Temperature (°F)	Vapor Pressure (in. water gauge)	Uniform Load (lb/ft ²)	Concentrated Load (ton)
500,000 SST	S3 ⁽³⁾	16	1.0	70	10	101	70	0	0	0
750,000 SST	S5 ⁽³⁾	0	1.0	70	10	130	70	0	0	0
1,000,000 SST	S5 ⁽³⁾	0	1.0	70	10	130	70	0	0	0
1,000,000 DST	L1 ⁽²⁾	35.17	2.0	70	6	130	70	0	0	0
1,000,000 DST	L5 ⁽²⁾	8.75	0.0	70	6	130	70	0	0	0
1,000,000 DST	S5 ⁽²⁾	35.17	1.0	70	9	130	70	0	0	0

⁽¹⁾ Lateral soil pressure (Rankine coefficient = 0.5)

⁽²⁾ Subgrade Modulus = 497 lbf/in³
Refractory Pressure (RP) = 0
Annulus Pressure (AP) = 0

⁽³⁾ Subgrade modulus 400 lbf/in³