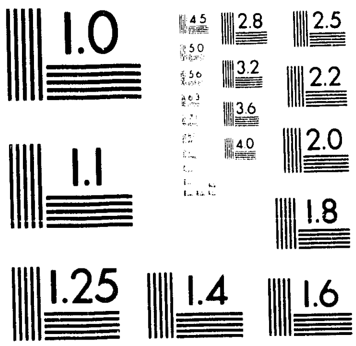
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Structural Analysis of Hanford's Single-Shell 241-C-106 Tank

A First Step Toward Waste-Tank Remediation

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BIOGRAPHICAL INFORMATION

This information will be used by the session chairman to introduce your paper.

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A First Step Toward Waste-Tank Remediation

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STRUCTURAL ANALYSIS OF HANFORD'S SINGLE-SHELL 241-C-106 TANK
A FIRST STEP TOWARD WASTE-TANK REMEDIATION

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ABSTRACT

The buried single-shell waste tank 241-C-106, located at the U.S. Department of Energy's Hanford Site, has been a repository for various liquid radioactive waste materials since its construction in 1943. A first step toward waste tank remediation is demonstrating that remediation activities can be performed safely. Determination of the current structural capacity of this high-heat tank is an important element in this assessment. A structural finite-element model of tank 241-C-106 has been developed to assess the tank's structural integrity with respect to in situ conditions and additional remediation surface loads. To predict structural integrity realistically, the model appropriately addresses two complex issues: (1) surrounding soil-tank interaction associated with thermal expansion cycling and surcharge load distribution and (2) concrete-property degradation and creep resulting from exposure to high temperatures generated by the waste. This paper describes the development of the 241-C-106 structural model, analysis methodology, and tank-specific structural acceptance criteria.

BACKGROUND

Tank 241-C-106 is a 530,000-gal capacity single-shell underground nuclear waste-storage tank located in the C Tank Farm in the 200 East Area of the U.S. Department of Energy's Hanford Site. The tank has been used for radioactive waste storage since 1947. A program to recover strontium and cesium from high level waste began during the 1960's . While these operations were being conducted, problems with the sludge washing and decant process caused the transfer of 197,000 gal of strontium-loaded sludge to tank 241-C-106. In 1971, temperatures in the sludge exceeded 212 °F. The current radioactive-decay heat-generation rate has been calculated at between 90,000 and 130,000 Btu/h. To prevent the sludge from drying out and the tank from overheating, approximately 6,000 gal of cooling water are added to the tank each month. These additions would have to continue until the year 2045 because of the half-life of the strontium. At that time, the structural integrity of the 100-year-old tank may no longer be acceptable.

In response to Public Law 101-510, Section 3137, Westinghouse Hanford Company submitted to the United States Department of Energy (DOE) the document *A Plan to Implement Remediation of Waste Tank Safety Issues at the Hanford Site* [1] as part of a report to Congress. The report ranked and set priorities for all the waste tank safety concerns. Tank 241-C-106 ranked fourth with a priority of one. The Tri-Party Agreement (TPA) mandates the interim stabilization of tank 241-C-106 by September of 1996 and the interim isolation of all 149 single-shell tanks by the same date. Interim stabilization and isolation criteria require the cessation of the current practice of adding water to the tank as an evaporative cooling mechanism. Retrieval of the high-heat waste by sluicing is the most expedient technique available for achieving the interim stabilization and isolation of tank 241-C-106. Once the heat-generating sludge is removed, cooling water additions can cease. DOE and the Washington State Department of Ecology designated tank 241-C-106 to be a retrieval demonstration tank. A first step toward waste tank remediation is demonstrating that the associated activities can be performed safely. Determining the current structural capacity of the high-heat tank 241-C-106 is an important element in the safety assessment.

INTRODUCTION

Tank 241-C-106, shown in Figure 1, is a cylindrical reinforced concrete structure capped by a dome with a rise of 12 ft from the inside of the haunch. The tank stands 33-ft tall with an outside diameter of 77 ft. It has a reinforced concrete wall thickness of 1 ft and a minimum dome thickness of 1¼ ft. The floor of the tank is a 6-in.-thick reinforced concrete dished-bottom slab covered with a 2-in. layer of grout reinforced with wire mesh. The tank is buried in sandy soil with a minimum soil cover of approximately 7 ft at the dome apex. The waste is retained within a ¼-in.-thick welded steel-plate liner with an open top. The liner is contained within and separated from the concrete by ¾-in.-thick three-ply asphaltic waterproofing.

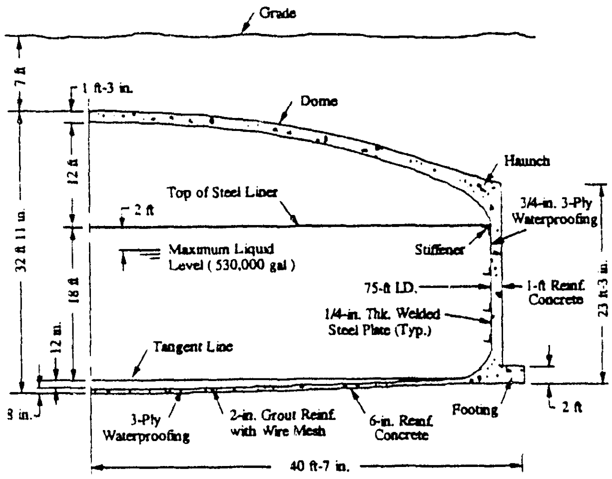

Figure 1. Schematic of Tank 214-C-106.

A structural model of the 241-C-106 tank and surrounding soil has been developed that can be used to judge the tank's structural integrity with respect to in situ conditions, as well as additional surface loads. To predict structural integrity, the model must address several complex issues associated with the aging effects of a buried reinforced-concrete tank exposed to temperatures above 200 °F. Creep and material property degradation resulting from time and high-temperature exposure are important considerations in characterizing the in situ condition of the tank.

Conventional design-based methods of evaluation need to be augmented to address conditions that are beyond the original design basis. A useful measure of the tank's structural integrity is the margin against collapse. Collapse-load analysis requires a robust concrete constitutive model. Large surface-applied loads associated with the collapse analysis impose similar demands on the soil-plasticity algorithm. Cyclic temperature changes associated with tank operations result in thermal expansion and contraction of the tank against the soil that requires a soil constitutive model that characterizes the cohesionless mechanical behavior of the sandy soil surrounding the tank.

The 241-C-106 structural model was used to predict the present and future in situ conditions of the tank, evaluate its structural integrity, and determine the reserve capacity. This paper describes the development of the 241-C-106 structural model, analysis methodology, and tank-specific structural acceptance criteria. Two companion papers detail the development of the concrete property degradation relations [2] and the soil modeling effort [3], respectively. An overview of the structural evaluation process is shown in Figure 2.

CONSTITUTIVE MODELS

The ABAQUS [4] finite-element program was chosen as the analytical tool because of its reputation in solving nonlinear problems, its extensive material library, and its versatility in accepting user-defined external subroutines. In the 241-C-106 structural model, the surrounding soil is modeled explicitly as a continuum by using the ABAQUS extended Drucker-Prager granular soil-plasticity material model option. The Drucker-Prager soil constitutive model has a pressure-dependent yield function. The elastic properties also can be made pressure dependent through the ABAQUS field variable option. The field variable represents the confining pressure expected to develop during the analysis at a given soil element in the model [3].

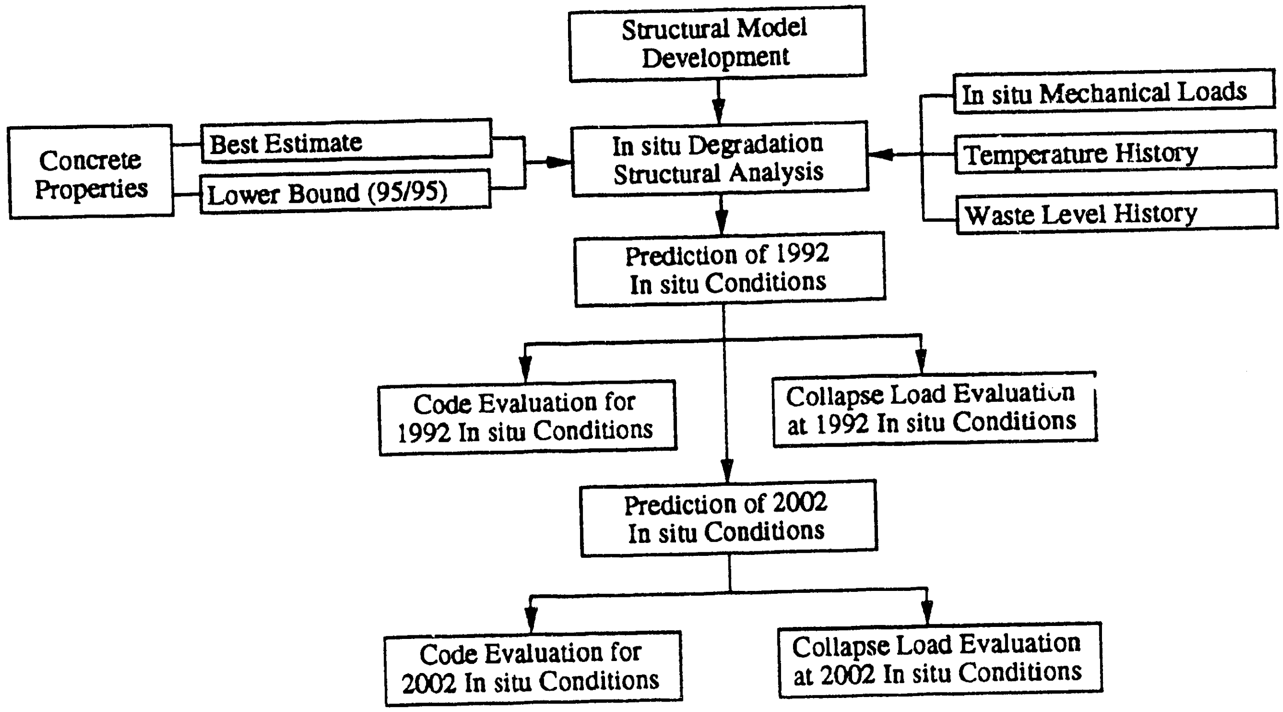

Figure 2. Tank 241-C-106 In situ Structural Integrity Evaluation Flow Chart.

The application of a material subroutine provided in ANACAP-U [5] enables analysts to address several issues relating to the nonlinear behavior of concrete:

- Irreversible degradation of compressive strength, tensile strength, and elastic modulus as a function of time and temperature [2]
- Cracking
- Compressive strain hardening with strain softening beyond the peak stress
- Thermally activated creep.

The concrete material subroutine is based on the smeared crack approach introduced by Rashid [6].

ANALYSIS METHODOLOGY

THERMAL ANALYSIS

Best-estimate and upper-bound thermal analyses were performed to determine the temporal and spatial temperature distribution in the 241-C-106 tank from initial operation in 1947 to the year 2002. The following information was analyzed to construct an approximate heat source for the thermal analyses:

- Historical records of waste material additions and extractions
- Recorded levels and level changes of both liquid and sludge layers
- Tank total heat load based on psychrometric analyses of evaporation rates and the isotopic chemical makeup of the contained wastes
- Tank thermocouple data in the air space, the liquid layer, and the sludge layer
- Operating history of the tank ventilation system.

A time-dependent heat source in the sludge waste layers was established to account for changes in the volume of waste material and the radioactive decay of the primary constituent. An axisymmetric thermal model of the tank and surrounding soil was used (see Figure 3) with appropriate boundary conditions to obtain the temperature history throughout the tank structure.

The best-estimate thermal analysis, calibrated to reliable thermocouple data obtained during the 1992 ventilation system failure, predicts a maximum temperature of approximately 320 °F in the waste sludge, 318 °F in the concrete foundation slab, and 162 °F in the tank dome. The upper-bound thermal

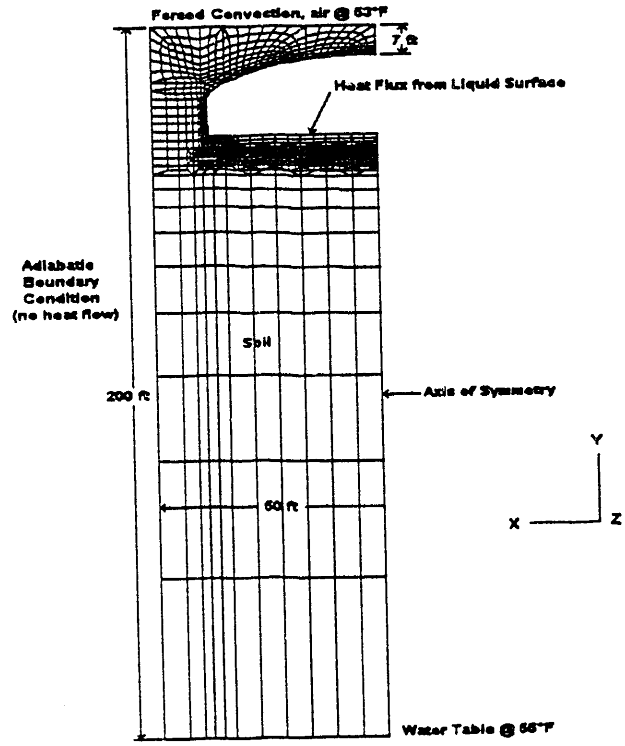

Figure 3. Thermal Model for Tank 241-C-106.

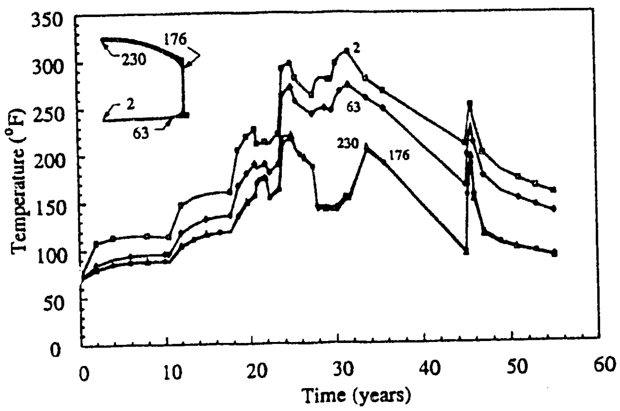

Figure 4. Upper-Bound Temperature History at Selected Locations in the Tank 241-C-106.

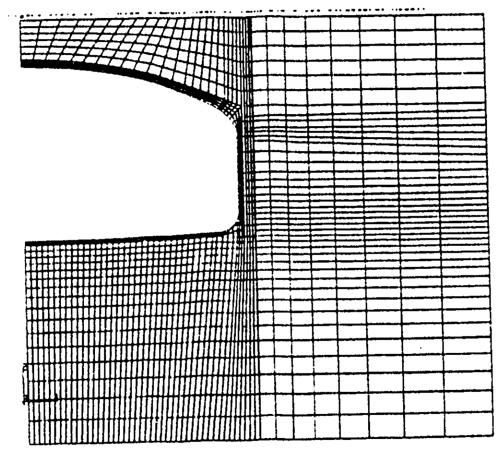

Figure 5. Axisymmetric Finite-Element Structural Model of Tank 241-C-106 and Surrounding Soil.

analysis, based on heat-generation rates comparable to estimates of previous investigators, predicts a maximum temperature of 312 °F in the waste sludge, 310 °F in the slab, and 220 °F in the tank dome. The degrading effect of time at temperature on concrete properties for temperatures above 200 °F is an important consideration in the structural analysis. Hence, the upper-bound thermal history is the primary focus in the structural analysis because of the higher predicted temperatures in the critical dome region. A 55-year upper-bound thermal history (see Figure 4) was developed and applied in the creep analysis to establish the 1992 and 2002 in situ conditions.

STRUCTURAL MODEL

The structural model of the 241-C-106 tank is an axisymmetric ABAQUS finite-element model. The tank and the surrounding soil are modeled with quadratic axisymmetric finite elements (see Figure 5). The tank elements are reinforced with rebar sub-elements where necessary. The soil regions contain special features for the development of the initial geostatic stress state, as discussed in [3]. Contact between the soil and the tank is enforced through nonlinear interface springs that are very stiff in one direction and very compliant in the other. These interface springs reduce the numerical complexity associated with contact (gap) elements. The tank liner is modeled with three-noded axisymmetric shell elements. The liner interacts with the tank through spring elements similar to those between the soil and the tank.

The steel rebar and liner are modeled with the ABAQUS elastic-plastic constitutive options. The non-structural grout layer between the bottom of the steel liner and the foundation slab is modeled as a soft elastic material. The concrete constitutive model in ANACAP-U interfaces with ABAQUS through the user-defined material subroutine (*UMAT) option provided in ABAQUS. The ANACAP-U elastic-perfectly plastic concrete model with time-and-temperature property degradation is used in the initial static and creep analysis of the tank to determine its degraded in situ condition. The concrete material model is switched to the more robust ANACAP-U strain-hardening/softening model in evaluating the post-creep state of the structure relative to code-based evaluation criteria or when determining the collapse-load capacity of the tank. All analyses are conducted with the ABAQUS large-displacement option selected.

STRUCTURAL-MODEL LOADING

Loads applied to the structural model include dead-weight, hydrostatic pressure induced by the waste, thermal history, and surcharge pressure. Gravity first is applied to the tank and the soil below the tank. The soil region around and above the tank is constructed by adding a series of strain-free layers [3]. The weight of the soil is transmitted to the tank through the nonlinear interface springs described above. The waste-induced pressure is applied to the liner as a hydrostatic pressure.

The complete 55-year thermal-distribution history is applied as a series of load steps. The waste-induced hydrostatic pressure is modified during the application of the thermal history to simulate the mechanical effects of the addition and extraction of the waste during the tank operational history. The response state of the tank is saved at the end of each load step. The state of the concrete is described by a set of internal state variables at each concrete element integration point. The state variables store the values of the degraded material properties, as well as the position, orientation, and open/closed status of any cracks that form.

Collapse load analyses are performed from selected post-creep states by application of a uniformly distributed load at the soil surface over a 10-ft radius about the center of the tank with the ANACAP-U strain-hardening/softening concrete model selected. The load then is increased until the structure offers little resistance to additional load or until the solution becomes unstable.

Code-based service-load checks are performed from the degraded, post-creep state of the tank. The factored-load cases are described in detail in the following section.

STRUCTURAL ACCEPTANCE CRITERIA

The structural acceptance criteria combine applicable sections of American Concrete Institute (ACI) Code 349-90, *Code Requirements for Nuclear Safety-Related Concrete Structures* [7], and the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel (B&PV) Code [8] design-by-analysis philosophy, with supplementary criteria to deal with conditions not specifically covered by the code provisions. The design-by-analysis approach comprises two independent strategies for evaluating the tank's structural integrity. The first approach is based on the current ACI 349-90 requirements, except that load factors are applied through a nonlinear analysis of the tank from its degraded, post-creep state. Section forces and moments are calculated by summing across the section thickness the nodal forces determined in the nonlinear analysis. This approach provides a measure of structural integrity in terms of the provisions of ACI 349. The second approach relies on the intrinsic ability of the finite-element model to predict the time-dependent structural response to the in situ loads and the structural response associated with conditions near collapse. The results of the collapse-load analysis, starting from the predicted in situ state, provide a basis for determining the reserve capacity of the tank.

DESIGN CODE APPROACH

Table 1 lists the various load types and the corresponding minimum and maximum load factors consistent with ACI 349-90, Section 9.2. The live load L_2 is a 100-ton vertical load distributed uniformly over a 77-ft diameter at the soil surface. All of the tabulated loads except L_2 are known to be present in the in situ condition. Therefore, only L_2 is assigned a minimum load factor less than 0.9. Because the

Table 1. Service Load Type and Corresponding Load Factor.

Load Symbol	Description of Load	Minimum Load Factor	Maximum Load Factor
D	Dead load of tank	0.9	1.4
F	Hydrostatic load from waste	0.9	1.7
H	Lateral earth pressure	0.9	1.7
L_1	Soil overburden	0.9	1.4
L_2	100-ton live load	-0-	1.7
T	Temperature	0.9	1.4

vertical component of earth pressure is more certain than the lateral component, the load factor for soil overburden is 1.4 as opposed to 1.7 for the side soil-induced loading. Conversely, because the specific gravity of the waste is uncertain, a load factor larger than the 1.4 specified by the code is used for F.

The loads D, F, L_1, and L_2 tend to maximize vertical dome displacement and/or radial wall displacement, while lateral earth pressure H has the opposite effect. Thermal growth tends to increase the effect of lateral earth pressure. ACI 349-90 allows a 25-percent reduction on the load factors in load combinations that include the thermal load T; however, the effect of temperature is small in comparison to the effects of other loads. Therefore, the load factor reduction is not considered. The following two load combinations envelop service load combinations applicable to an ACI Code evaluation of the tank:

Load Case 1:

$$U = 1.4D + 1.7F + 1.4L_l + 1.7L_2 + 0.9H + 0.9T \quad (1)$$

Load Case 2:

$$U = 0.9D + 0.9F + 0.9L_l + 1.7H + 1.4T. \quad (2)$$

The ACI 349-90 Code addresses safety considerations with load factors and strength reduction ϕ-factors. Load factors address the possibility that prescribed service loads may be exceeded. The ϕ-factors address variations in materials, construction dimensions, and calculation approximations. In a conventional code evaluation, the nominal-load response parameters (forces, moments and shears) are factored. An alternative approach considers the response from factored loads. The two approaches are equivalent only when linear elastic analysis is used. The second approach is more appropriate when the structural response is nonlinear.

Response from one load cannot be isolated from the response of another load in a nonlinear analysis. Behavior is path dependent and superposition is invalid. Therefore, load factors cannot be applied to the response. Instead, the load factors can be applied with confidence directly to the loads. Paragraph 19.2.1 of ACI 349-90 states that "methods of analyses which are based on accepted principles of engineering mechanics and applicable to the geometry of the structure may be used." *Commentary on Code Requirements for Nuclear Safety-Related Concrete Structures* [7] offers strong support for the applicability of nonlinear analysis techniques in assessing the redistribution of loads in cracked, reinforced-concrete shell structures. For structures experiencing significant thermal loads, an ACI 349 Committee Report 349.IR-91, *Reinforced Concrete Design for Thermal Effects on Nuclear Power Plant Structures* [9], clearly states a preference for nonlinear analysis. Furthermore, simply factoring the response parameters at critical structural sections may not be conservative because of the load redistribution that occurs within the structure.

MARGIN ASSESSMENT APPROACH

An alternate method of measuring the in situ structural integrity of the 241-C-106 tank is to develop a model capable of accurately predicting the nonlinear structural behavior. Such a model would account for actual tank details and could be used to predict the approach to failure. As a measure of the structure's collapse-load capacity, the collapse load is determined for a uniformly distributed load applied over a local region about the center of the tank in an axisymmetric analysis. An equivalent in situ load is defined as the corresponding central load that, when acting alone, gives the same center deflection of the dome as that obtained in the in situ condition. The total collapse load divided by the equivalent in situ load is the computed safety factor ($SF_{computed}$) against collapse in the in situ state, i.e.,

$$SF_{computed} = \frac{\textit{Total collapse load}}{\textit{Equivalent in situ load}} \quad (3)$$

where

Total collapse load = Central load at collapse + Equivalent in situ load.

Another way of measuring the in situ structural integrity is by determining the reserve capacity of the structure where

$$\textit{Reserve capacity} = \frac{\textit{Total collapse load}}{SF_{required}} - \textit{Equivalent in situ load.} \quad (4)$$

The ASME B&PV Code definition of collapse load applicable to experimental stress analysis is applied to the calculated load-deflection curve to define the collapse load. On a load-deflection curve an angle θ defines the angle between the best-fit linear regression of the linear portion of the load-deflection curve and the load axis. A second straight line, hereafter called the collapse limit line, is drawn through the displacement axis intercept at an angle ϕ with the load axis, where $\phi = \tan^{-1}(2\tan\theta)$. The collapse load is defined as the intersection of the load-deflection curve with the collapse limit line, as shown in Figure II-1430-1, ASME B&PV Code, Section III, NB-3213.25 [8]. The ASME technique can give a grossly conservative collapse load when the linear portion of the load-displacement curve is interrupted by a "jump" in displacement without an appreciable change in load and then returns to a relatively linear behavior just after the "jump." In determining the collapse load, the ASME technique may be augmented to take into consideration the jump discontinuity. Displacement data beyond the jump discontinuity may be shifted down by a constant amount to eliminate the discontinuity. The ASME method described above then is applied to the shifted data as shown in Figure 6.

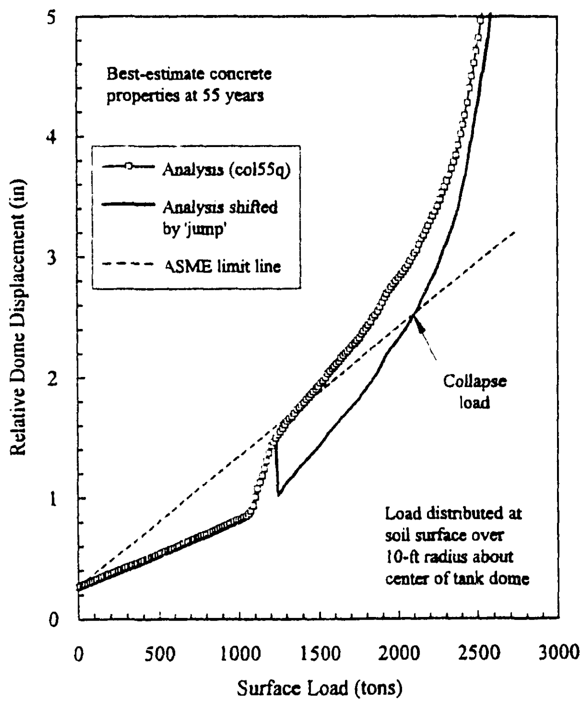

Figure 6. Tank 241-C-106 55-Year Post-Creep Load-Deflection Curve for Central Load over 10-ft Radius (Best-Estimate Concrete Properties and Upper-Bound Thermal History).

SAFETY FACTOR

The ACI 349 [7] and 318 [10] codes use a combination of load factors and capacity reduction, or ϕ-factors. While the magnitudes of the safety factors are based on some acceptable level of risk, the actual values are determined by committee consensus. The ACI Code attempts to reduce the risk of failure to 1 in 100,000, i.e., 1 in 1,000 on load demand and 1 in 100 on structural capacity [11]. Dividing the ACI load factor by the appropriate ϕ-factor provides the expected factor of safety against exceeding the local section capacity.

A definition better suited to protect against global structural instability is given by the ASME B&PV code (NB-3213.25), which allows plastic analysis to determine the collapse load for normal service conditions. For concrete shell structures, ACI 349 refers to ACI 318 for applicable commentary on structural stability. ACI 318 allows the use of inelastic analysis where it can be shown to provide a safe basis for design. While ACI 318 requires the designer to consider general instability, it gives no specific guidance. Instead, it refers to recommended design practices for domes used in industrial applications as given in a 1970 ACI Committee 344 report, *Design and Construction of Circular Prestressed Concrete Structures with Circumferential Tendons* [12] and refers to design approaches for other shells as given in ACI SP-67, *Concrete Shell Buckling* [13].

The ACI Committee 344 report provides a minimum shell-thickness equation based on linear elasticity that contains multiplicative reduction factors to account for geometric imperfections, creep, nonlinearities, concrete cracking, and material-strength uncertainties. Much of what these reduction factors are intended to account for is addressed explicitly by the ABAQUS/ANACAP-U finite-element model. The ABAQUS large-displacement option is invoked to determine the load-displacement curve. The potential for concrete crushing is characterized specifically within the ANACAP-U material subroutine. Whether the failure of the dome by concrete crushing is abrupt will be evident from the analysis results. Rebar bond failure is not modeled in the concrete constitutive model and must be checked separately. Although local bond failure may not result in a state of general structural instability, there would be no defensible basis for accepting loads that caused bond failure.

The empirical buckling equation from ACI Committee 344 [12] is

$$p_{cr} = \frac{\phi\ \beta_i\ \beta_c\ E_c}{1.5}\left(\frac{t_{\min}}{R_{\max}}\right)^2, \qquad (5)$$

where

p_{cr} = critical buckling pressure
E_c = elastic modulus of concrete
t_{min} = minimum shell thickness
R_{max} = maximum radius of curvature of shell,

and the following parameters were introduced to account for material uncertainties,

$$\phi = 0.7, \tag{6}$$

creep, nonlinearities, and cracking,

$$\beta_c = 0.44 + 0.005 t_{min} - 0.046, \tag{7}$$

and imperfections,

$$\beta_i = \left(\frac{R_{max}}{R_i}\right)^2, \tag{8}$$

where R_i is the radius of curvature of the imperfection.

The collapse analysis for C-106 addresses all of the issues to some degree except for material uncertainties. Therefore, the only applicable knockdown factor is 0.7/1.5 = 0.467. An ACI report on *Buckling of Thin Concrete Domes* [14] suggests that the critical buckling load be required to exceed the ACI-factored loads so that it would be more consistent with the *International Association for Shell and Spatial Structures* (IASS) recommendation as discussed in ACI SP-67 [13]. Gravity loading of the soil overburden dominates the loads that contribute to dome deflection in the in situ condition. Therefore, the applicable load factor would be 1.4. Hence, the total required factor of safety to protect against in situ short-term buckling is 1.4/0.467, essentially 3.0.

REBAR BOND SLIP/SPLICE FAILURE

Failure criteria for rebar bond and splices must be addressed to establish limits that assure the ability of the finite-element analysis to predict global structural behavior. Deformed bars depend primarily on mechanical interlocking for enhanced bonding. With little confinement, the bond failure of large deformed bars may manifest itself by the splitting of the concrete along the plane of the bar. This type of failure depends primarily on the load on the concrete, with bar stress and bar perimeter being of little importance. As confinement around a bar improves, by virtue of increased cover or transverse reinforcement, the ultimate load depends increasingly on the bar perimeter. Therefore, small bars, top-cast bars, or bars that are confined to the extent that bond failure occurs by shear failure of the concrete lugs between bar deformations instead of splitting, will carry a maximum unit load proportional to the bar perimeter. Failures of the non-splitting type tend to be localized and not critical to the global integrity of the structure.

Splitting bond failure is caused by a system of forces developed as a result of the bearing of the rebar lugs on the concrete. The resulting system of forces is directed outwardly around the bar to form a hollow, truncated cone of pressure. These outward forces, like water pressure in a pipe, lead to splitting on weak planes in the concrete along the bar. A well established theory for splitting bond failure, first proposed in Orangun 1977 [15], addresses the dependency of bond stress on such parameters as development length, bar size, bar cover, and clear bar spacings.

Two-dimensional finite-element analyses were conducted for a concrete cross section for square rebar having sizes, cover, and clear spacing representative of the 241-C-106 tank construction. Three adjacent rebar in a layer (i.e., at the same "cover" distance from the face of the concrete) were considered in each analysis case (see Figure 7).

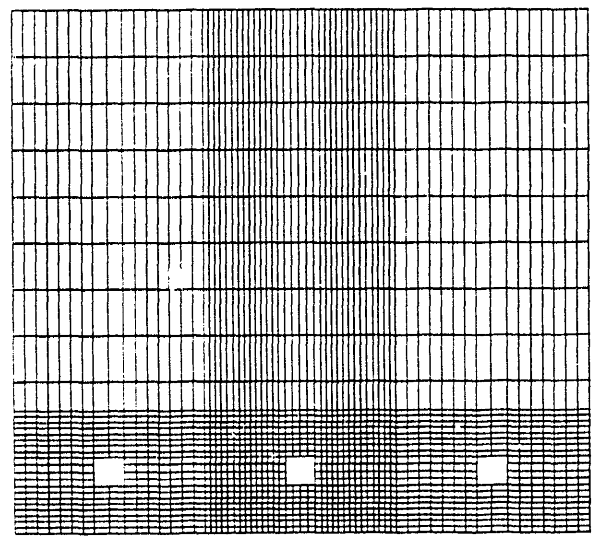

Figure 7. Finite-Element Model of Concrete with 1¼-in. Square Rebar.

The concrete model mesh refinement and boundary conditions were established with an emphasis on the accurate prediction of principal tensile stresses in the critical "cover" region. Square voids having the dimensions of the rebar were modeled, and a uniform pressure was applied to the vertical and lateral surfaces of the voids. Two cases were addressed:

(1) 1-in. square rebar, 8-in. center-to-center spacing, and a 3-in. cover

(2) 1¼-in. square rebar, 8-in. center-to-center spacing and a 2-in. cover.

Corresponding models for round rebar of the same characteristic dimensions (i.e., 1-in. and 1¼-in. diameter) were also developed. Although the distribution of the concrete principal tensile stresses for a square rebar varies appreciably from that for an "equal" size round rebar, the maximum principal tensile stress for the square rebar is no more than 11 percent higher than it is for the "equal" size round rebar. Therefore, for the C-106 tank square-rebar bond evaluation, it is conservative to establish their capacities at 80 percent of that proposed for round rebar.

241-C-106 ANALYSIS RESULTS

The design-by-analysis ACI-based evaluations serve as an indicator of the structural integrity of the 241-C-106 tank, but are not relied on to characterize the actual capacity of the structure. All of the current ACI 349-90 detailing requirements are met, with the exception of a minor violation in the minimum rebar cover at the bottom of the footing. After the in situ post-creep condition of the structure was established, two ACI-based evaluations were performed corresponding to current best estimate (45-year post-creep state determined with best-estimate material properties and upper-bound thermal history) and future worst-case (55-year post-creep state determined with lower-bound concrete properties and upper-bound thermal history) conditions.

Resulting shear, and combined bending and axial load at various sections throughout the tank were evaluated with factored-load combinations. Shear was not the controlling mechanism. The worst-case condition shows that the location most susceptible to exceeding the combined bending and axial load capacity are the wall just below the haunch and the footing. Even though the ACI criteria are met, there is essentially no margin in the wall just below the haunch and only a 10-percent margin in the footing. All other locations have margins of greater than 50 percent.

Collapse analyses were performed with both the best-estimate and lower-bound Hanford-concrete properties as summarized in Table 2. In spite of the lack of significant margin as indicated by the ACI-based code evaluation, the 55-year (2002) post-creep collapse load with the best-estimate Hanford-concrete properties is predicted to be 2,110 tons. The collapse load predicted with the best-estimate thermal history shows very little difference relative to the collapse load predicted with the upper-bound thermal history. The 55-year post-creep collapse load for the lower-bound Hanford-concrete properties is predicted to be 1,670 tons. This value produces a safety factor against collapse of 7 compared to a required safety factor of 3.

CONCLUSION

With the aid of modern computers and sophisticated finite-element numerical methods, it is now possible to simulate the complex nonlinear behavior of reinforced concrete structures. The application of these methods can give a better estimate of the response of the structure to time-dependent and temperature-dependent loading. However, such analyses require robust solution strategies embedded in a well-developed nonlinear finite-element computer program that adequately captures nonlinear geometric effects.

The ANACAP-U concrete constitutive model with Hanford-specific material properties that account for property degradation as a function of time and temperature was linked to the general purpose ABAQUS finite-element computer program to determine the structural response of the 241-C-106 waste-storage tank to its operational load history. The structural integrity of the degraded post-creep state of the tank is assessed through application of the ACI 349 code criteria by a continuation of the nonlinear analysis from the post-creep state with code-based factored service loads. The in situ reserve capacity of the tank as determined through a collapse load analysis is predicted to be significantly greater than indicated by the results of the ACI code-based evaluation.

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Table 2. Tank 241-C-106 In Situ Reserve Capacity.

Time in Service (years)	Concrete Properties	Calculated Thermal History	Collapse Load (tons)	Equivalent In Situ Surface Load (tons)	Total Collapse Load (tons)	Calculated Safety Factor	Required Safety Factor	Reserve Capacity (tons)
45	Lower-bound	Upper-bound	1,690	256	1,946	7.60	3	393
45	Best-estimate	Upper-bound	2,120	256	2,376	9.28	3	537
55	Lower-bound	Upper-bound	1,670	256	1,926	7.52	3	386
55	Best-estimate	Upper-bound	2,110	256	2,366	9.24	3	533

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