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FOR HANFORD SITE WASTE TANK EVALUATION

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7. Abstract

As Hanford Site contractors address maintenance and future structural demands on nuclear waste tanks built as early as 1943, it is necessary to address their current safety margins and ensure safe margins are maintained. Although the current civil engineering practice has building codes for reinforced concrete design guidelines, the tanks were not constructed to today's building codes and future demands potentially result in loads and modifications to the tanks that are outside the original design basis and current practice.

The Hanford Site engineering staff has embraced nonlinear finite-element modeling of concrete in an effort to obtain a more accurate understanding of the actual tank margins. This document brings together and integrates past Hanford Site nonlinear reinforced concrete analysis methods, past Hanford Site concrete testing, public domain research testing, and current concrete research directions. This document, including future revisions, provides the structural engineering overview (or survey) for a consistent, accurate approach to nonlinear finite-element modeling of reinforced concrete for Hanford Site waste storage tanks.

This report addresses concrete strength and modulus degradation with temperature, creep, shrinkage, long-term sustained loads, and temperature degradation of rebar and concrete bonds. Recommendations are given for parameter studies and evaluation techniques for review of nonlinear finite-element analysis of concrete.

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CONCRETE STRUCTURAL ANALYSIS
TOOLS AND PROPERTIES
FOR HANFORD SITE
WASTE TANK EVALUATION

September 1995

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**CONCRETE STRUCTURAL ANALYSIS
TOOLS AND PROPERTIES
FOR HANFORD SITE
WASTE TANK EVALUATION**

September 1995

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CONCRETE STRUCTURAL ANALYSIS TOOLS
AND PROPERTIES FOR HANFORD SITE
WASTE TANK EVALUATION

1.0 INTRODUCTION

As Hanford Site contractors address maintenance and future structural demands on nuclear waste tanks built as early as 1943, it is necessary to address their current safety margins and ensure safe margins are maintained. Although the current civil engineering practice has building codes for reinforced concrete design guidelines, the tanks were not constructed to today's building codes and future demands potentially result in loads and modifications to the tanks that are outside the original design basis and current practice.

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2.0 CONCRETE FAILURE CONSTITUTIVE MODELING

Concrete materials show a great variety of load carrying capabilities when subjected to different conditions. The behavior and strength of concrete is complicated and is influenced by the mechanical and physical properties of the aggregate and cement paste and the nature of the boundary conditions and loading. It is necessary for practical applications to make idealizations in the mathematical modeling of strength characteristics (Chen and Saleeb 1994).

Many different proposed failure criteria for concrete can be found in literature and literature reviews (Nadai 1950, Jaeger 1956, Paul 1968, Argyris et al. 1974, Eibl and Ivanyi 1976, Chen 1981). Although there are a number of proposed failure models, the most important at this time are the Mohr-Coulomb and the five-parameter Willam-Warnke model. These two failure models are reviewed in the following section.

At this time at the Hanford Site, software is available with three different concrete models. The three software packages with concrete models are ABAQUS, ANSYS, and ANACON¹ (also previously known as ANACAP-U). The concrete crushing or failure models in the three software packages are also summarized in the following based on the software theoretical documentation. The concrete analyst is advised to run simple test models and check error reports to verify proper software predictions.

2.1 MOHR-COULOMB CONCRETE FAILURE MODELING WITH TENSION CUTOFF

The Mohr-Coulomb plasticity failure model defines a pressure or hydrostatic stress dependent failure surface or envelope. In the principal stress space, the Mohr-Coulomb failure surface represents an irregular hexagonal pyramid, with straight meridians (Chen and Saleeb 1994, see Figure 2.1-1). The failure of concrete under general stress states has been based on the Mohr-Coulomb failure criterion combined with a tension cutoff, (see Figure 2.1-2). This combined criterion provides a fair first approximation to the failure of concrete in the post-failure range. According to this combined criterion, failure can be

¹ABAQUS is software developed by Hibbitt, Karlsson & Sorensen, Inc., Pawtucket, Rhode Island.

ANSYS is a registered trademark of Swanson Analysis Systems, Inc., Houston, Pennsylvania.

ANACON and ANACAP-U are software developed by ANATECH Research Corporation of San Diego, California.

Figure 2.1-1. Mohr-Coulomb Criterion. (Chen and Saleeb 1994)
(a) Principal Stress Space
(b) Mohr's Diagram

Figure 2.1-2. Mohr-Coulomb Maximum Principle Stress. (Chen and Saleeb 1994)
Note: in the π -plane ($\sigma_1 + \sigma_2 + \sigma_3 = 0$) (a) Mohr-Coulomb Criterion.
(b) Maximum Tensile Stress Criterion (Tension Cutoff).

(a)

(b)

divided into two failure mechanisms. Tensile or cracking failure mechanism and compressive or shear failure mechanism. In view of its extreme simplicity, many routine stress analyses are still based on the Mohr-Coulomb criterion with a tension cutoff (Chen and Saleeb 1994).

In the tension cutoff range, the tensile mechanism of cleavage fracture occurs because of splitting or cracking. It is controlled by the criterion of a constant maximum tensile stress. The cracking occurs generally in the direction perpendicular to the maximum tensile stress.

The compressive stress range failure occurs because of shear slips, and it is controlled by the Mohr-Coulomb maximum shear stress. If all principal stresses are different in magnitude, there are two equally probable planes of slip. Under biaxial compressive stresses, the test specimens have been found to split perpendicular to the direction of the greatest principal strain. This strongly suggests that a dual failure criterion expressed in terms of strains as well as stresses is needed. The Mohr-Coulomb criterion combined with maximum tensile stress or strain cutoffs, achieves a common approximate dual representation.

The cracking type of failure is characterized by splitting perpendicular to the direction of maximum tensile strain. Based on experimental results for different stress states the value of maximum tensile strain at failure changes. Although inconsistent, the value of maximum tensile strain is assumed constant for this model.

The following is a number of conclusions that can be made concerning the limitations and advantages of the combined failure criterion of Mohr-Coulomb with a tension cutoff (Chen and Saleeb 1994).

- The failure criterion is only based on minimum and maximum principal stress with no influence of the intermediate principal stress. This implies that the maximum biaxial compressive strength of concrete is the same as the uniaxial compressive strength which is counter to observed data. For example, a maximum strength increase of about 25 percent is observed in biaxial compression tests for a second principal stress that is 0.5 of the maximum principal stress.
- The Mohr-Coulomb failure surface meridians are straight lines. Figures 2.1-3 and 2.1-4 show that this approximation becomes poorer as the hydrostatic pressures become larger.
- The failure cross sections in the deviatoric plane are similar (affine) curves with a constant ratio of deviatoric tension to deviatoric compression. The failure cross section and the ratio of deviatoric tension to compression experimentally changes with hydrostatic pressure.

Figure 2.1-3. General Character of Meridians Based on the Experiments. (Chen and Saleeb 1994)

Figure 2.1-4. Typical Test Results Along Compressive and Tensile Meridians. (Chen and Saleeb 1994)

- The failure surface is not a smooth surface. The sharp corners (singularities) are known to cause numerical problems.
- The combined criterion provides a partial explanation concerning the modes of failure in concrete, tensile (cracking) type and compressive (shear slip) type.
- Considering the simplicity of the Mohr-Coulomb criterion with tension cutoffs, it is a fair first approximation in many cases (In the author's opinion, the model may require adjustment or tuning for the stress state based on experimental data.).

2.2 FIVE-PARAMETER WILLAM-WARNKE CONCRETE FAILURE MODELING

At this time (July 1995) the best concrete failure model appears to be the five parameter model developed by Willam and Warnke (1974). In the five parameter model the shape of the failure surface in the principal stress space can be defined in contrast to the Mohr-Coulomb model.

The Willam-Warnke model has curved meridians radiating out from the hydrostatic stress axis as opposed to the straight meridian of the Mohr-Coulomb model. The Willam-Warnke curved meridians are described by second-order parabolic expressions. In the deviatoric plane normal to the hydrostatic stress axis the Willam-Warnke failure surface is defined by an elliptic curve for each part $0 \leq \text{angle } \theta \leq 60$ degrees (see Figure 2.2-1). The failure surface is defined by approximating a tensile and compressive meridian with two second order parabolas which are connected by an ellipsoidal surface with the deviatoric curve as the basis (Willam and Warnke 1974, Chen and Saleeb 1994). The Willam-Warnke model is periodic in the deviatoric plane with a period of 120 degrees and it possesses a 60 degree symmetry. It has a noncircular trace in the deviatoric plane with its shape changing from nearly triangular to nearly circular with increasing hydrostatic pressure.

The following five failure stress state failure points are required to define the material parameters. They include three simple tests and two arbitrary strength points in the high compression regime.

1. Uniaxial compression strength
2. Uniaxial tensile strength
3. Equal biaxial compression strength
4. High compressive stress point on the tensile meridian
5. High compressive stress point on the compression meridian

The five parameter Willam-Warnke is valid for all stress combinations in the range of most practical applications including tensile stresses, and it gives a reliable estimate to the experimental tests (Chen and Saleeb 1994). The results obtained by Willam and Warnke (1974) using the five parameter model are compared to test data of Launay and Gachon (1972) in Figure 2.2-2. Close agreement is observed in both the hydrostatic (meridian) and the deviatoric sections shown in Figure 2.2-2 a and b. In Figure 2.2-2b the deviatoric curves can be seen to become more circular with increasing hydrostatic pressure.

2.3 CRACK MODELING

There are three approaches for crack modeling that have been employed with finite-element studies of concrete structures. These are smeared cracking modeling, discrete cracking modeling, and fracture mechanics modeling. Each model may be employed for different purposes. The smeared crack modeling is probably the best choice if a structural system load-deflection behavior is desired. If a detailed local behavior is of interest then discrete cracking modeling may be required. For the special class of problems in which fracture mechanics is the appropriate tool, a specialized fracture model may be necessary (Chen and Saleeb 1994).

In the smeared cracking approach, the cracked concrete is assumed to remain as a continuum. The cracks are smeared out in a continuous fashion. The cracking is assumed to form planes perpendicular to the maximum principal stress direction. Before cracking concrete is assumed to be an isotropic linear elastic material. After the first crack the cracked concrete becomes orthotropic with one axis of the material principal directions following the crack orientation. The tangent stiffness in the direction normal to the crack goes to zero with an open crack and back to the original value with a strain condition representing closed cracks. In a similar manner the shear modulus goes to zero with large crack openings and back to the original uncracked shear modulus with strains that would close the cracks.

In the discrete cracking models a predefined crack is built into the finite-element mesh. As the crack propagates, the finite-element mesh must be updated with the cracking. One difficulty of this method is the crack location restrictions imposed by the preselected finite-element mesh. Also as the cracking progresses the solution effort increases due to expanding degrees-of-freedom. The use of discrete cracking has had limited acceptance in general structural applications.

In the current stage of development of fracture mechanics cracking modeling of concrete much remains to be done (Kesler et al. 1972). This area has been studied by Hillemier and Hilsdorf (1977) and Bazant and Cedolin (1979).

Figure 2.2-1. Five-Parameter Willam-Warnke, Deviatoric Section of the Failure Surface with Threefold Symmetry. (Chen and Saleeb 1994)

FIVE-PARAMETER FAILURE MODEL

Figure 2.2-2. Comparison of the Five-Parameter Model Results and Triaxial Test Data. (Launay et al. 1972, Willam and Warnke 1974, Chen and Saleeb 1994) Dots Denote Experimental Results.
(a) Hydrostatic Section. (b) Deviatoric Sections.

LINEAR ELASTIC-FRACTURE MODELS FOR CONCRETE

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2.4 ANSYS CONCRETE CONSTITUTIVE MODELING

The ANSYS concrete constitutive model permits cracking in three orthogonal directions at each integration point. The cracking is treated as a smeared band of cracks, rather than discrete cracks. The concrete is assumed to be initially isotropic. Any reinforcement is assumed to be smeared throughout the element. In addition to cracking and crushing, the concrete may undergo plasticity, with the Drucker-Prager most commonly used (This plasticity is before failure crushing.). The crushing failure model is the Willam-Warnke with defaults for three of the five parameters. The default parameters only apply for stress situations with a low hydrostatic stress component. All five failure parameters should be specified when a large hydrostatic stress component is expected (ANSYS User's Manual 1994). The ANSYS concrete model makes no attempt to include prediction of cyclic response, or of the reduction in the elastic stiffness caused by inelastic straining.

2.5 ABAQUS CONCRETE CONSTITUTIVE MODELING

The ABAQUS 5.4 concrete model is intended as a model of concrete behavior for relatively monotonic loadings under fairly low confining pressures (less than four to five times the largest compressive stress that can be carried by concrete in uniaxial compression) (ABAQUS Theory Manual 1994). Cracking is assumed to be the most important aspect of the behavior. Cracking is a smeared crack model with constitutive calculations performed at the solid element integration points. The ABAQUS concrete model includes the cracking induced anisotropy. When the principal stress components are dominantly compressive the response of the concrete is modeled by an elastic-plastic theory, using a simple relationship for the yield surface written in terms of the first two stress invariants. Associative flow and isotropic hardening are used. This model significantly simplifies the actual behavior. The associative flow generally overpredicts the inelastic volume strain. The simple compressive yield surface does not match all data very accurately (ABAQUS Theory Manual 1994). The ABAQUS concrete model makes no attempt to include prediction of cyclic response, or of the reduction in the elastic stiffness caused by inelastic straining.

2.6 ANACAP-U AND ANACON CONCRETE CONSTITUTIVE MODELING

The ANACAP-U/ANACON concrete constitutive modeling is a smeared crack model with a Mohr-Coulomb failure condition at compressive failure (James and Rashid 1993). This concrete yield or failure model is used together with Prager's kinematic hardening rule extended to include isotropic hardening and the general nonlinear stress-strain curves.

3.0 CAUTIONS AND COMMENTS ON NONLINEAR ANALYSES

Nonlinear numerical analysis has a number of pitfalls inherent in the process that are dependent on the actual physics, the numerical approximation selected, and the computer numerical accuracy. These potential pit falls can compromise a structural analysis with concrete materials.

Linear analysis has been a standard for engineering design and analysis, because for a given structure a single unique solution can be obtained for any load case. In the nonlinear regime of structural analysis with nonlinear materials and large deflection theory, the nature of the applicable equations or phenomena is such that more than one solution is possible for any given loading condition. In nonlinear analysis the selection of the best or most probable solution is history dependent. At this point the numerical computer analysis world and the actual physical world can select different solutions or answers.

In the numerical world simplifying assumptions, the numerical methods, and the computer may push the analysis to predict a solution different from the actual result. Also, a lack of understanding of all of the important phenomena may lead to inappropriate simplification. Inappropriate simplification can make test results appear random in comparison to the analysis results.

Nonlinear solutions can have bifurcation or branch points that can lead the numerical analysis to predict a different solution than the observed test result. But under varying conditions both solution paths may be valid.

In summary nonlinear analysis requires extra work and understanding versus linear analysis. The advent of low cost computing has made nonlinear analysis more affordable but current numerical methods require a lot of engineering manipulation to obtain any solution. A greater technical expertise is required to obtain a valid solution. In short, today nonlinear engineering analysis is a highly technical endeavor that requires careful evaluation and checking.

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4.0 CONCRETE COMPRESSIVE STRENGTH AND MODULUS WITH THERMAL DEGRADATION

The exposure of concrete to elevated temperatures deteriorates the physical properties (strength and stiffness) of concrete. The strength of concrete and the level of degradation with time at temperature is influenced by many variables such as the following:

- Water-to-cement ratio and extent of compaction: The compressive strength of fully compacted concrete increases with a decreasing water-cement ratio. However, insufficiently compacted concrete can reduce the expected strength of concrete with a low water-to-cement ratio because of the presence of gross voids (Neville 1971).
- Type and size of aggregate and mixture proportions: Limestone aggregates degrade less than siliceous aggregates, and degradation increases with an increasing cement-to-aggregate ratio (RHO 1981).
- Conditions of testing: There is less degradation in laboratory specimens tested while still in the "hot" condition compared to those tested after being allowed to cool (RHO 1981).
- Extent of curing before testing: The longer a structure or specimen is cured before exposure to high temperatures, the higher both its initial strength at the start of heating and its subsequent degraded strength during heating (RHO 1981).
- Extent of moisture retention: Sealed specimens lose more strength than unsealed specimens. The moisture in the concrete of the Hanford Site underground waste-storage tanks is not effectively sealed, but is free to diffuse into the surrounding soil or atmosphere, although some protective coatings were applied in the initial construction. Mass concrete in nuclear power plants is essentially sealed. Hence much of the research has concentrated on investigating the effects of elevated temperature on sealed concrete, such research generally will not be applicable to the Hanford Site tanks (RHO 1981).
- Loading condition: There is less degradation in laboratory specimens subjected to external loads during heating than in those that are not (RHO 1981).

4.1 HANFORD SITE CONCRETE TESTING AND HENAGER EMPIRICAL RELATIONS (ANACAP-U AND ANACON)

To quantify the behavior of Hanford Site concrete at elevated temperatures, standard 6- x 12-in. unsealed cylindrical specimens were cast and tested by Portland Cement Association (PCA) at CTL for concrete mix designs with nominal 28-day design compressive strengths of 3,000 and 4,500 lbf/in² as used in the construction of the Hanford Site waste-storage tanks. Hanford Site concrete was fabricated from basalt aggregates and American Society of Testing and Material (ASTM) Designation C150 Type II (low alkali) portland cement. To closely simulate the compositions and properties of the concretes used in the construction of the Hanford Site waste-storage tanks, test specimens were fabricated from the same mix proportions that were specified in the original construction. Cements, aggregates, and admixtures came from the same sources as materials used in the Hanford Site tank construction: sand and gravel from the Hanford Site batch plant and ASTM Designation C150 Type II portland cement from Lonestar Industries, Seattle, Washington. Specimens were moist-cured for 200 to 300 days before heating and then continuously heated at 250, 350, and 450 °F for up to 1,304 days. Strength and modulus tests were performed while the concrete was essentially in the "hot" condition. A statistical analysis of the PCA/CTL test data (RHO 1981) was conducted by Pacific Northwest Laboratory (PNL) (Henager et al. 1988) to establish concrete-property equations and uncertainty bands for Hanford Site concrete. The following empirical expressions were developed from the PCA/CTL high-temperature test data for Hanford Site concrete and incorporated into ANACAP-U, an ABAQUS user defined subroutine (James and Rashid 1993).

$$E = 5.3947 + 0.1233 S - 0.006751 T - 0.1786 \log_e(t+1) I + E_b \quad (4-1)$$

$$f_c = 4416.338 + 490.919 S - 4.714 T_A + (1.273 T_B - 230.241) \log_e(t+1) I + f_{cb} \quad (4-2)$$

$$f_t = 448.1758 + 23.7436 S - 0.6077 T_A - 18.4341 \log_e(t+1) I + f_{tb} \quad (4-3)$$

where

- E = modulus of elasticity (10⁶ lbf/in²)
- f_c = compressive strength (lbf/in²)
- f_t = splitting tensile strength (lbf/in²)
- S = nominal 28-day compressive strength (10³ lbf/in²)
(valid range 3 ≤ S ≤ 4.5)

T	= constant value of temperature ($^{\circ}\text{F}$)
T_A	= $\max [0, T-350]$ ($^{\circ}\text{F}$)
T_B	= $\max [0, 350-T]$ ($^{\circ}\text{F}$)
t	= time at constant temperature (days)
I	= 0 for $T \leq 200$ $^{\circ}\text{F}$ = $(T - 200)/50$ for 200 $^{\circ}\text{F} < T < 250$ $^{\circ}\text{F}$ = 1 for $T \geq 250$ $^{\circ}\text{F}$
E_b	= uncertainty band width for modulus (10^6 lbf/in 2) = 0.00 for best fit
f_{cb}	= uncertainty band width for compressive strength (lbf/in 2) = 0 for best fit
f_{tb}	= uncertainty band width for tensile strength (lbf/in 2) = 0 for best fit

The database used in establishing these empirical equations is limited to a temperature range of 250 to 450 $^{\circ}\text{F}$. Extrapolation to temperatures significantly above 450 $^{\circ}\text{F}$ is not recommended. However, adapting these empirical equations to a general constitutive model for practical applications requires accommodating a lower range of temperatures. Because exposure of concrete to constant temperatures below 200 $^{\circ}\text{F}$ generally causes insignificant loss in strength and modulus with exposure time, a cut-off region is defined so that no time-dependent degradation is assumed to occur below 200 $^{\circ}\text{F}$, and a linear transition is applied between 200 and 250 $^{\circ}\text{F}$. This cut-off and transition condition is implemented through the 'I' term applied to the time-dependent terms in the above equations. The equation for modulus of elasticity still yields decreasing modulus values with increasing temperature below 200 $^{\circ}\text{F}$, independent of time.

These equations are derived from data obtained from test specimens held at constant, discrete temperatures. A second assumption involves applying these property equations to variable temperature histories. Because the mechanisms causing deterioration are not reversible (i.e., a return to lower temperatures does not replace lost moisture and cannot heal microcracks), the loss in properties is assumed not recoverable on return to lower temperatures. Also assumed is that no new microcracking will form during the next temperature rise until the current temperature exceeds the maximum temperature experienced by the material. At the same time it is clear that degradation will not suddenly stop if the temperature drops somewhat during a soak period, i.e., if the material at 300 $^{\circ}\text{F}$ experiences a certain rate of degradation, then degradation should continue if the temperature drops to 275 or to 250 $^{\circ}\text{F}$. Thus, no recovery of material properties occurs during cooldown, but the time-degradation term in the equations remains active as long as the material is at temperatures above 200 $^{\circ}\text{F}$. However, on temperature increase, the temperature-dependence term is not active until the temperature exceeds the previous maximum temperature experienced by the material. Physically, this interpretation can be justified as follows: the temperature-dependence term is associated with degradation from microcracking, and the time term is associated with degradation from water movement.

ANACAP-U/ANACON starts with a Hanford concrete strength that reflects a testing value that is significantly greater than the minimum specified value. In order to do analysis with this code the structural analyst should carefully review the strength formulation to ensure that the strength is as intended. The analyst may also consider reducing the specified strength to do analysis with this formulation.

4.2 PETERSON'S REEVALUATION OF HANFORD SITE CONCRETE TESTING AND DEGRADATION EQUATIONS

Test data relating concrete degradation to time and temperature were developed (RHO 1979, PNL 1979, RHO 1981). The database was used to generate concrete degradation equations as a function of time, temperature, and concrete strength (PNL 1986, PNL 1988). Several limitations in the test data reduction process were highlighted (PNL 1986, PNL 1988, Peterson 1994).

An approach was developed to address several of the more important limitations (Peterson 1994). Essential features of the approach included the following:

- Correcting batch reference strength/modulus data to a common fog room time so that a consistent strength reduction ratio due to temperature degradation could be computed.
- Combining the 3k and 4.5k mix data sets so as to double the effective database used in the linear regression analysis.
- Dividing up the degradation into short term and long term parts (this was essential in getting the correct modulus degradation ratios).
- Non-dimesionalizing the degradation as a ratio of the degraded strength to the undegraded strength so that the equations could be used for any specified strength concrete.

The major limitations that could not be addressed include the following:

- The lack of a statistical sample size so that the large scatter present in the data could be statistically quantified.
- The existence of data points that showed significant increases in strength or modulus as a function of time rather than a decrease (directly related to the scatter in the data).

This effort resulted in a new set of degradation equations that were non-dimensionalized with respect to undegraded strength and modulus values where:

$$\text{property degradation ratio} = A \cdot x + B, \quad x = \ln(\text{day} + 1)$$

short term degradation ($0 < \text{day} < 28$)

strength degradation at 250 °F $A = -.01251$, $B = 1.0$

modulus degradation at 250 °F $A = -.10841$, $B = 1.0$

strength degradation at 350 °F $A = -.03497$, $B = 1.0$

modulus degradation at 350 °F $A = -.14500$, $B = 1.0$

long term degradation ($28 < \text{day}$)

strength degradation at 250 °F $A = -.01809$, $B = 1.01881$

modulus degradation at 250 °F $A = -.00176$, $B = 0.64089$

strength degradation at 350 °F $A = -.03518$, $B = 1.00073$

modulus degradation at 350 °F $A = -.00611$, $B = 0.53231$

This results in the following degradation values for 50 years at constant temperature:

concrete strength degradation at 250 °F = .8413

modulus degradation at 250 °F = .6236

concrete strength degradation at 350 °F = .6555

modulus degradation at 350 °F = .4723

The separation of the long term high temperature curve to short and long term segments is a good improvement compared to the PNL curves used in ANACAP-U/ANACON as documented in Section 4.1. The temperature degradation data in this section can be input directly into ANSYS in a temperature dependent table format. The ANACAP-U/ANACON code would have to be revised to let the user define the degradation curve to use this data.

4.2.1 Concrete Stress-Strain Curves

Concrete stress-strain curve shapes for different temperatures and degradations were not available from the test data. Therefore an assumed stress-strain curve (Peterson 1994) has been used where the peak strain at the failure stress was assumed to be 0.0025 in/in independent of the degradation and temperature effects. Peterson (1994) recommends that experimental stress-strain curves for various time/temperature degradation levels should be compared to the idealized stress-strain curves used in the analysis and modified if significant differences exist.

4.2.2 Concrete Property Values for Tank Evaluation

The database used in this data reduction effort shows considerable scatter especially at the longer time periods. In general, from an engineering judgement standpoint, one would expect the concrete property to degrade with time at temperature. However, some data show the property to increase with time. This is contrary to the perceived behavior. The database used to develop the degradation equations must be viewed with caution. Also any extrapolation beyond the actual test time of 5 years may be in question. Due to the database uncertainty it is highly recommended that a range in concrete material properties be considered in any evaluation (Peterson 1994). An appropriate range for strength and modulus is indicated in Figures 4.2-1 and 4.2-2 (Kassir et al. 1993).

The concrete strength and modulus are used to develop stress-strain curves for use in the analysis. The tank structures are redundant structures and therefore the distribution of the stress in the tank and between tank components depends on the stiffness of the components. If the concrete modulus is selected to have a low value then stress will be re-distributed to the steel. On the other hand, if the concrete modulus is selected too high then the stress is re-distributed to the concrete. Therefore, the more severe case depends on whether the steel or the concrete is the critical component. Both high and low modulus range values must be considered.

Concrete strength value affects the ACI code evaluations: lower strength results in less capacity. Unlike the modulus, it is obvious that the lower strength produces the most severe case for evaluation purposes.

Figure 4.2-1. Reduction of Compressive Strength of Concrete at Elevated Temperatures. (Kassir et al. 1993)

Figure 4.2-2. Variation of the Modulus of Elasticity of Concrete with Temperature. (Kassir et al. 1993)

5.0 COEFFICIENT OF THERMAL EXPANSION AND THERMAL CONDUCTIVITY

5.1 THERMAL EXPANSION COEFFICIENT

The Hanford Site concrete test results (CTL 1981) reported an average expansion coefficient of 3.29×10^{-6} , with a range of 2.12 to 3.91×10^{-6} . The relatively low magnitude of these values, plus some questionable testing techniques, has resulted in the accuracy having been questioned by Hanford Site analysts.

To help resolve the Hanford Site concrete expansion coefficient issue, additional tests were initiated in 1993. The first tests were run on cored specimens from an older Hanford Site structure (T Plant, 1944 vintage). As indicated by the results reported in Appendix B of Winkel (1994), the average coefficient for 12 specimens was $4.0 \times 10^{-6}/^{\circ}\text{F}$, with a range of 3.57 to 4.36×10^{-6} . Testing was performed on both oven-dried and fully saturated specimens. Moisture was found to have a very small effect on the results, this is somewhat contrary to Section 2.9.2 of ACI 209R (ACI 1993A). The temperature range for the test data were about 50°F . The resulting expansion coefficient is assumed to be independent of temperature, that is based upon the experience of the Bureau of Reclamation (testing agency). This position is also consistent with Figure 16.5 of Neville (1971) that indicates a constant concrete expansion coefficient up to a temperature of 600°F .

In addition to being old, the T Plant concrete is relatively low strength ($f'_c = 2,500 \text{ lb/in}^2$). Therefore, recently poured cylinders from the HWVP project were obtained ($f'_c = 5,000 \text{ lb/in}^2$) for additional thermal expansion testing. The results of the HWVP concrete testing are included in Appendix B of Winkel (1994). The HWVP test results predicted a mean value of $3.7 \times 10^{-6}/^{\circ}\text{F}$, with a 2 sigma range of 3.2×10^{-6} to 4.2×10^{-6} .

Since the expansion coefficient of concrete is strongly influenced by the expansion coefficient of the aggregate, there is reason for concern that Hanford Site concrete aggregate variability may increase the uncertainty in the expansion coefficient value. The T Plant concrete cores, discussed above, were all taken from the same vicinity and are not likely to cover the full range of Hanford Site aggregate variability. As discussed in Section 3.2 of Winkel (1994), Hanford Site aggregate appears to be principally basalt. According to Table 2.9.3 of ACI 1993A, the expansion coefficient of basalt aggregate can vary from 2.2 to $5.4 \times 10^{-6}/^{\circ}\text{F}$. (It is also noted that this same table presents a range for basaltic concrete of 4.4 to 5.8×10^{-6} , that is higher than the range of the T Plant measurements.) Although "Hanford Site basalt" should have a narrower range, there is a potential for variability. The relatively narrow data range for both the T Plant and HWVP test data, plus the small difference between the HWVP and T Plant concrete test results, provide some evidence of the uniformity of the Hanford Site concrete aggregate, at least relative to the thermal expansion coefficient.

5.2 THERMAL CONDUCTIVITY

The coefficient of thermal conductivity of concrete is .435 BTU/h-ft-°F (Juryk et al. 1993, P/THERMAL 1991).

6.0 CONCRETE CREEP WITH THERMAL EFFECTS

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 Site 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. Concrete creep analyses and studies performed recently by WHC have used ABAQUS (HKS 1989) (a nonlinear finite-element program for structural analysis and research) with work done by ANATECH Research Corporation (James and Rashid 1993). The ANATECH work was incorporated into a concrete material subroutine written for ABAQUS. This concrete model was used to evaluate the condition of the high-heat Hanford Site C-106 waste tank (Juryk et al. 1993).

The fact that a relatively large amount of work has been done on concrete creep, makes it desirable to begin with an overview of the research and experimental work reviewed for this document and then follow with an application to Hanford Site waste tank analyses.

6.1 RESEARCH REVIEW OF CONCRETE CREEP

Although this investigation originated with a conservative safety concern over the effects of concrete creep on Hanford Site 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 it 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 has contributed to its success as a structural material. This success can be attributed to its ability to undergo creep.

The effect of concrete creep is to redistribute structural load from regions of high concentrated 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 design practice with large built-in margins may not always view these redistributions as meeting the accepted design guidelines.

Although reinforced concrete creep can be beneficial, such is not always the case (Neville 1970). The creep can have harmful effects on 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 excessive cracking.

The Hanford Site waste tanks do not employ any prestressed concrete construction, thus, the first harmful effect is not important for the Hanford Site waste tanks. Although the Hanford Site 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.

It is recommended that additional evaluation be planned to investigate thermal cycling of waste tanks with a range of thermal gradients.

6.1.1 Creep Under Uniaxial Stress

Research has been done (Rusch 1960, Chuang et al. 1972) on a general flexural theory for structural concrete that incorporates creep behavior. This flexural theory is demonstrated in Figure 6.1-1 for concrete subjected to a uniaxial compression load.

Curve OA of Figure 6.1-1 is the instantaneous stress-strain relationship for concrete in compression. The curve OA shows the classic strain hardening observed in many engineering materials as failure is approached (on line AB). The instantaneous failure stress corresponds to the ACI f'_c . To the right of curve OA there are additional stress-strain curves that incorporate additional strain resulting from creep at a specified time. Curve OB gives the ultimate stress-strain relation, for an infinite time under a sustained load. The failure strength decreases with creep and time until it intersects line OB at the absolute sustained load strength. The stress-strain curves with and without creep have a linear region starting at the origin and become nonlinear above the absolute creep proportional limit. The ANATECH concrete software used for the Hanford Site C-106 analyses addressed only the linear creep region below the proportional limit. Collapse analyses with creep must consider the nonlinear effects of stress on creep above the proportional stress limit.

If only creep is considered, Figure 6.1-2 shows that a similar relationship between stress and creep strain can be developed by subtracting the instantaneous strain from the total strain. Examination of Figure 6.1-2 indicates that the relationship between stress and creep strain is linear at stresses below the creep proportional limit. The slope of the stress creep-strain curves decreases with increased time under load, an indication that creep is a function of time.

Figure 6.1-1. Stress-Strain-Time Relationship for Concrete Subjected to a Sustained Uniaxial Load.

Figure 6.1-2. Stress Creep-Strain Time Relationship for Concrete Subjected to a Sustained Uniaxial Load.

6.1.2 Creep Compliance Function

The linear relation of creep strain to stress level below the creep proportional limit leads to the current theoretical and numerical usage of a creep compliance function. The creep compliance function is equal to the creep strain divided by the stress. Thus, the creep strain is equal to the stress times the creep compliance term (Note that some references use the term "specific creep", while others "unit creep"). In this idealization, creep compliance can be a function of time, concrete mix, concrete curing, humidity, and temperature. At stress levels greater than the creep proportional limit, creep compliance becomes also a function of stress:

$$\epsilon_c = J(\)\sigma \quad (6-1)$$

where

ϵ_c = creep strain
 $J(\)$ = creep compliance function
 σ = stress.

For example: In Figure 6.1-4, the creep strain at 2 years for 50 percent relative humidity can be seen to be approximately 0.001 mm/mm (in/in) with loading stress of 1,420 lbf/in². The creep compliance would be obtained by dividing 0.001 in/in by 1,420 lbf/in² giving a compliance of 0.7 millionths per lbf/in².

6.1.3 Influence of Humidity on Creep

Neville (1970) considers humidity to be one of the most important environmental influences on concrete creep. Experimental work has shown that drying concrete creeps at a higher rate and achieves higher ultimate creep than concrete that remains wet or remains dry. The influence of relative humidity is great, at a relative humidity of 50 percent, creep may be two to three times greater than at a relative humidity of 100 percent (Troxell et al. see Figure 6.1-3). The effects of humidity decrease with the increase in the size of the concrete specimen (Keeton 1960).

L'Hermite and Mamillan (1968) found experimentally a linear relation between creep and relative humidity for specimens loaded 200 days or more (see Figure 6.1-4). de la Peña (1959) found that the effect of relative humidity on creep is proportionately the same for lean or rich mixes of concrete.

Creep depends not only on the loss of water under load but also on the actual amount of water present when the load is applied. The relative humidity during the curing process is of interest as well. Dutron (1957, see Table 6.1-1) was responsible for one of the earliest experimental investigations involving variable humidity both before and after loading. Hansen's (1960, see Table 6.1-2) experimental work produced similar results.

Figure 6.1-3. Influence of Relative Humidity of Storage on Creep of Concrete
(4 x 14 in. [0 x 35 cm]) Cylinders, 1:5.67 Mix with a Water-Cement
Ratio of 0.59, Moist-Cured for 28 Days and then Loaded
at the Humidity Indicated. (Troxell et al. 1958)

Figure 6.1-4. Relation Between Relative Humidity and Creep for
2.8 x 2.8 x 11 in. (7 x 7 x 28 cm) Prisms
Under a Stress of 1420 lbf/in² (100 kg/cm²). (L'Hermite and Mamillan 1968)

Table 6.1-1. Creep for Different Relative Humidities Before and After Application of Load. (Datron 1957)

Storage condition*		Relative creep
Before loading	Under load	
Water	Water	1.0
Dry air	Water	1.7 to 2.2
Moist air	Moist air	2.1 to 2.7
Dry air	Dry air	3.1 to 3.4
Water	Dry air	4.0 to 4.5

*Dry air: 45-50 percent relative humidity, moist air: 65-70 percent relative humidity.

Table 6.1-2. Creep for Different Storage Conditions Before and After Application of Load. (Hansen 1960)

Test No.	Relative humidity of storage (percent)		Age at loading (days)	Relative creep
	Before loading	Under load		
1	70	60		1.70
2	70	60		1.39
3	70	70		1.29
4	70	Water	28	1.31
5	Water	Water		1.00
6	70	Sealed		0.70
7	10	Sealed		0.58
1a	70	50		0.68
2a	70	60		0.59
3a	70	70		0.57
4a	70	Water	128	
5a	Water	Water		1.00
6a	70	Sealed		0.48
7a	10	Sealed		0.44

Clearly, humidity can play a major role in the magnitude of concrete creep. The variability of humidity during concrete curing and long-term loading can cause a large variation in experimental creep test results. Not only does humidity play a major role in creep, but it has added complexity and confusion to interpretation of test results. In the structural design phase humidity-induced creep variation may be evaluated best by evaluating the sensitivity of the design to variation in concrete creep.

6.1.4 Influence of Concrete Curing on Creep

Concrete curing can have a significant effect on the magnitude of creep. Low-pressure steam curing has been found to reduce concrete creep (ACI Committee 1963). For example, steam curing at 65 °C (150 °F) for 13 hours can reduce creep by as much as 50 percent (ACI Committee 1963, see Figure 6.1-5). The steam cure reduction in creep is due partially to accelerated hydration of cement, and partly to moisture loss when the hot specimens are removed to a drier atmosphere (Neville 1970).

6.1.5 Influence of Temperature on Creep

Temperature is a major environmental influence on concrete creep (Neville 1970). The temperature effects on concrete are of major importance for nuclear structures involving operational temperatures as high as 93 °C (200 °F) and with temperatures possibly as high as 177 °C (350 °F) for some Hanford Site waste tanks.

In the past, Hanford Site engineering studies of concrete creep have employed a special-purpose ABAQUS user-defined material subroutine. The material subroutine is called ANACAP-U by the developers at ANATECH Research Corporation. Figure 6.1-6 documents the ANACAP-U creep compliance for a temperature range of 27 to 71 °C (80 to 160 °F). The ANACAP-U software was fit to the Hannant (1967) experimental data for creep variation versus temperature.

The Hannant (1967) experimental data had a limited temperature range of only 27 to 77 °C (81 to 170 °F). The ANATECH Research Corporation assumed a continuous monotonic temperature relation for creep and used the curve fit to extrapolate creep predictions into the higher temperatures [James and Rashid 1993]. Experimental work reported by Marechal (1969) and Neville (1970) indicates that temperature behavior above 71 °C (160 °F) is not continuously monotonic. The moisture content and the drying at higher temperatures is capable of causing major shifts in the creep results. Figure 6.1-7 shows a comparison of how ANACAP-U varies creep compliance with temperature to experimental tests by Nasser and Marzouk (1981) who addressed moisture by testing unsealed and sealed concrete specimens.

Figure 6.1-5. Specific Creep of Steam-Cured and Normally Cured Concrete Loaded When Its Strength is 4,000 lbf/in² (280 kg/cm²). (ACI Committee 1968)

Figure 6.1-6. ANACAP-U Concrete Creep, 80 °F to 160 °F.

Figure 6.1-7. Concrete Creep Temperature Dependence.

Figure 6.1-7 shows creep compliance documented at 180 days for concrete at a range of temperatures. The heavy solid line reflects the ANACAP-U variation of creep with temperature. The heavy dashed lines show the Nasser and Marzouk (1981) test data for sealed concrete. The lighter dashed lines show the test data for unsealed concrete.

Although, the ANACAP-U mathematical model shows positive comparisons to some test points for most temperatures, the moisture variation range between sealed and unsealed concrete results in a wide band of creep variation.

6.1.6 Time Dependency and Long-Term Creep of Concrete

The variation of concrete creep with time is a very important consideration for structural creep evaluation. Figure 6.1-8 shows that ANACAP-U modeling of creep time dependency compares quite well to experimental results of unsealed concrete at 21 °C (70 °F) for times shorter than a year. The original form of the ANATECH creep mathematical model (ANACAP-2D) included terms to improve the very short-term comparison (less than 200 days).

Although short-term time trend comparisons can be selected to look quite good, there are large creep magnitude variations for sealed versus unsealed concrete test specimens.

The Hanford Site concrete mix aggregate is classified as largely of basaltic origin. Figure 6.1-9 compares ANACAP-U long-term predictions to experimental testing of basalt concrete (Troxell et al. 1958, Kordina 1960). The heavy solid line on Figure 6.1-9 is the ANACAP-U trend prediction (based on experimental data for 2 years). The thin solid line is reduced test data from Kordina (1960) that reached a maximum time of eight years. The circles represent test data points documented by Troxell et al. (1958) for tests approaching 30 years' time.

The actual long-term creep test results for basalt-aggregate concrete and the ANACAP-U predictions show a very large range of variation.

6.1.7 Creep Under Multiaxial States of Stress

Although past studies have differed with regard to applying superposition to creep behavior (Duke and Davis 1944, Gopalakrishnan et al. 1969, Hannant 1966, Ross 1964), superposition is currently used for multiaxial stress analyses. Applying superposition for multiaxial creep strain states has been shown experimentally (Chuang et al. 1970) to be reasonably valid at stresses below or close to the creep proportional limit.

Figure 6.1-8. Concrete Creep at 70 °F, at 2,500 lb/in² Stress.

Figure 6.1-9. ANACAP-U Concrete Creep Versus Testing.

For a multiaxial state of stress, if the x, y, and z loads are applied simultaneously, the creep strains in the three directions are defined simply as functions of creep compliance, the stresses, and creep Poisson's Ratio:

$$(\epsilon_c)_x = J() [\sigma_x - \nu_c \sigma_y - \nu_c \sigma_z] \quad (6-2)$$

$$(\epsilon_c)_y = J() [\sigma_y - \nu_c \sigma_x - \nu_c \sigma_z] \quad (6-3)$$

$$(\epsilon_c)_z = J() [\sigma_z - \nu_c \sigma_x - \nu_c \sigma_y] \quad (6-4)$$

where

- $(\epsilon_c)_x$ = creep strain in x direction
- $(\epsilon_c)_y$ = creep strain in y direction
- $(\epsilon_c)_z$ = creep strain in z direction
- σ_x = stress in x direction
- σ_y = stress in y direction
- σ_z = stress in z direction
- $J()$ = creep compliance function
- ν_c = creep Poisson's Ratio.

If the loads or stresses are applied at different times, then superposition is again assumed and the creep strains for each load are summed. The time dependency of the creep compliance results in different compliance values for each load having a different time of application.

It is common for experimental creep testing of concrete to be conducted on cylindrical specimens with only axial and radial stresses involved. If the axial and radial loads are applied at the same time, the creep strains in the radial and axial directions can be defined as

$$(\epsilon_c)_a = J() [\sigma_a - 2 \nu_c \sigma_r] \quad (6-5)$$

$$(\epsilon_c)_r = J() [\sigma_r - \nu_c (\sigma_a + \sigma_r)] \quad (6-6)$$

where

- $(\epsilon_c)_a$ = creep strain in the axial direction
- $(\epsilon_c)_r$ = creep strain in the radial direction
- σ_a = stress in the axial direction
- σ_r = stress in the radial direction
- $J()$ = creep compliance function
- ν_c = creep Poisson's Ratio.

6.1.8 Creep Poisson's Ratio

The following expression (York et al. 1972), derived from the theory of elasticity, was used to calculate the creep Poisson's Ratio for specimens subjected to sustained uniaxial and biaxial states of stress levels that are less than the creep proportional limit:

$$v_c = \frac{\sigma_a (\epsilon_c)_r - \sigma_r (\epsilon_c)_a}{2 \sigma_r (\epsilon_c)_r - (\epsilon_c)_a (\sigma_r - \sigma_a)} \quad (6-7)$$

where

- v_c = creep Poisson's Ratio
- σ_a = axial stress
- σ_r = radial stress
- $(\epsilon_c)_a$ = axial creep strain
- $(\epsilon_c)_r$ = radial creep strain.

The creep Poisson's Ratio was derived simply by defining Poisson's Ratio for a cylinder in terms of radial and axial strains and then substituting radial and axial creep strain. The creep Poisson's Ratio for a compressive test of a cylinder of concrete is calculated by employing the test-measured axial and radial creep strains.

Test results have been obtained by Chuang et al. (1970). The Chuang test results at 75 °F for creep Poisson's Ratio were for as-cast specimens, 0.15, and for air-dried specimens, 0.108. At 150 °F the creep Poisson's Ratio results were 0.149 for as-cast specimens and 0.140 for air-dried specimens.

6.1.9 Influence of Aggregate on Creep

It was recognized early that the mineralogical character of the aggregate is a factor in creep of concrete, though there is some doubt about the interpretation of this effect (Neville 1970). The aggregate also affects the elastic deformations of concrete due to its composite nature.

This current Hanford Site review of past testing of the influence of aggregate on concrete creep had hoped to show that the relationship of basalt to other aggregate can consistently be correlated. Although the magnitude of concrete creep of basalt concrete is very similar to aggregates of river gravel, granite and quartz the long term trend was not completely consistent. In the future it may be of benefit to correlate Hanford Site basalt aggregate concrete to other concrete aggregate to help expand the useful research database.

Two test series studying the change in concrete creep resulting from different aggregates documented by Neville (1970), conducted by Davis and Davis (1931) and Kordina (1960). These two tests are considered here because they also included basalt as a test aggregate.

The experimental work done by Davis and Davis (1931) was probably the first to study the influence of aggregate on concrete creep. Figure 6.1-10 shows a summary of their final test results as reported by Troxell et al. (1958). Figure 6.1-11 shows the general trends observed during the creep tests by Kordina (1960).

Figure 6.1-10. Creep of Concrete Made With Different Aggregates
(Aggregate-Cement Ratio 5.67, Water-Cement
Ratio 0.59, Stress 800 lbf/in² (56 kg/cm²)). (Troxell et al. 1958)

Figure 6.1-11. Specific Creep of Concretes Made With
Different Normal Weight Aggregates. (Kordina 1960)

6.1.10 Aggregate Content and Creep

Using concrete work by Pickett (1956), Neville derived a relation between creep and the aggregate content in concrete. The basis for this derivation was that when a compressive strain in the cement paste under a sustained load increases with time, the actual contraction of concrete is restrained by the presence of the aggregate. The following equation relates the creep of the concrete mix to the creep of neat cement paste, the elastic moduli, and the volume concentration of aggregate:

$$\log \frac{c_p}{c} = \alpha \log \frac{1}{1 - g - u} \quad (6-8)$$

where

$$\alpha = \frac{3 (1 - \nu)}{1 + \nu + 2 (1 - 2\nu_a) \frac{E}{E_a}} \quad (6-9)$$

and

- c_p = creep of neat cement paste
- c = creep of concrete mix
- g = volume concentration of aggregate
- E = modulus of elasticity of surrounding material (concrete)
- E_a = modulus of elasticity of aggregate
- ν = Poisson's Ratio of surrounding material (concrete)
- ν_a = Poisson's Ratio of aggregate
- u = volume content of unhydrated cement.

Equation 8, for the influence of aggregate content on creep, has been shown to perform well for concrete creep testing at conditions of 95 percent humidity (Neville 1970).

6.1.11 Influence on Creep of Stress-Strength Ratio

Neville (1959) developed an approximate stress-strength ratio rule stating that, for constant mix proportions and the same type of aggregate, creep is proportional to the applied stress and inversely proportional to the strength at the time of load application. Experimental results supply substantial evidence of a linear relation between creep and the applied stress. An exception to the linear creep relation is loading one to three day old concrete. The upper limit of this relation is called the absolute creep proportional limit.

Neville (1959) found that severe internal microcracking takes place in a concrete compression specimen at a stress-strength ratio of 0.4 to 0.6. The accelerated cracking corresponds to a shift in the creep behavior that coincides with the absolute creep proportional limit. Above the proportional limit, creep increases with stress at an increasing rate (Jones and Richart 1936, Jensen and Richart 1938, Gvozdev 1966). Figure 6.1-12 shows a sample plot of the stress-strength ratio versus creep.

6.1.12 Conclusions and Concerns for Hanford Site Waste Tank Thermal Creep

The following conclusions about concrete creep apply to Hanford Site waste tanks:

- The main effect of concrete creep is to redistribute structural load from regions of high concentrated stresses to regions of minimal stress. This load redistribution is beneficial and allows more load carrying capability before failure. For example: The maximum compressive stresses in a generic 1,000,000-gallon double-shell tank (DST) concrete decreased over a 50-year life as a result of creep (Hyde et al. 1994).
- The normal operational loads for the Hanford Site waste tanks are well within the absolute proportional creep limit for concrete creep prediction. For example: The maximum compressive stress 551 lbf/in² in the concrete of the generic 1,000,000-gallon DST. Over the 50 years the maximum compressive stress in the concrete decreased from 551 to 244 lbf/in² (Hyde et al. 1994).
- Concrete creep in single-shelled and double-shelled tanks can add significant load to the rebar. In double-shelled tanks creep can add a significant load to steel studs and the steel liners. For example: The secondary liner steel in the generic 1,000,000-gal DST increases in stress over time to 31,455 lbf/in² at 50 years. This maximum stress is due to bending at the lower haunch radius that results from the creep motion of the tank walls (Hyde et al. 1994).
- Although the creep analyses documented in this report can be related to past Hanford Site concrete creep studies, the literature has shown that there can be a wide variation in creep magnitude. The creep bounds with respect to Hanford Site tanks should be investigated.
- Creep can harm 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, that may result in additional cracking. Investigation of thermal cycling of waste tanks with a range of thermal gradients should be addressed.
- Creep has been found to vary significantly with the moisture content of the concrete and the humidity of the test chamber. 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 this document compares sealed and unsealed concrete test results, it probably is necessary also to 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 was fit to test data for a maximum of two years, while limited test data exist 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 to define the creep influence on concrete failure bounds do not exist. 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.
- ~ The variability of concrete creep with environmental effects (temperature, humidity, etc.) makes prediction of Hanford Site long term tank stresses subject to uncertainty. Bounding studies are strongly recommended for areas of engineering concern with acceptable safety factors for evaluation.

Figure 6.1-12. Relation Between Creep After 1 Minute Under Load and Stress-Strength Ratio For 1:4:5 Concrete, Loaded At The Age of 7 Days. (Neville 1971)

6.2 ANACAP-U AND ANACON CREEP MODELING

In ANACAP-U the creep strain for concrete is calculated from an incremental form of the hereditary integral

$$\epsilon^c(t) = \int_0^t J(\xi - \xi') \frac{d\sigma(\xi')}{d\xi'} d\xi', \quad (6-10)$$

where J is the uniaxial creep compliance that is independent of stress, σ is the stress, and ξ' and ξ are the equivalent times that account for temperature variation with time through the relations

$$\xi = \int_0^t \phi(T(\tau)) d\tau, \quad \xi' = \int_0^{\tau'} \phi(T(\tau)) d\tau, \quad (6-11)$$

where $\phi(T)$ is the temperature shift factor. The *ANACAP-U Theory Manual* (James 1992) and the *Supplement for Hanford Concrete* (James and Rashid 1993) provide detailed derivations of the creep formulation resident in the ANACAP-U constitutive model for concrete. The following discussion explains how the Hanford Site concrete creep equation was developed.

The ANACAP-U creep formulation requires that the creep data defining the creep response of Hanford Site concrete be represented in terms of a uniaxial creep-compliance (strain per unit of stress) function that is a function of time and temperature. In addition to the material property measurements discussed in Section 4.1, the PCA/CTL program (RHO 1981) tested six 4,500 lbf/in² Hanford Site concrete cylinders to obtain creep strain data at 250 and 350 °F under an applied stress of 500 and 1,500 lbf/in². Because of the limited nature of these data and apparent inconsistencies when extrapolated to lower temperatures with other data in the literature, an effort was made to base the Hanford Site concrete creep response on the broader database available through the literature. Browne (1968) and Hannant (1967) have reported data in the temperature range from 68 to 200 °F and for hold-times up to approximately 1,000 days. Marechal (1969) reports data in the temperature range of 68 to 482 °F for 100 days. There is fair agreement in these data for some temperatures and significant differences at other temperatures. Browne and Marechal show relative agreement for temperatures of 68 to 158 °F, while Browne and Hannant (1967) show better agreement at approximately 122 °F. However, Marechal's 100-day creep strain is about twice that of Hannant (1967) for this temperature. Marechal shows higher creep strains at 100 days for 122 °F than for 158 or 482 °F and higher creep strains for 68 °F than for 203 or 221 °F. Such differences indicate that the creep rate of concrete

may be non-monotonic with temperature. This non-monotonic behavior also is indicated by results presented by Neville (1973). However, the data reported by Browne (1968) and Hannant (1967) contradict this non-monotonic behavior of the creep rate of concrete with temperature. Many elements in the experiments may account for these differences, but the major reasons appear to be differences in mix design and whether test specimens are sealed or unsealed.

In an effort to be conservative, the creep rate for Hanford Site concrete is assumed to be monotonic with temperature. The Hannant (1967) data appear to be the most consistent qualitatively and are used as the basis for the specific creep function for Hanford Site concrete. On the basis of these data, the creep compliance J is assumed to be a function of time and temperature only and can be represented as:

$$J(T, t) = \phi(T) C(t) , \quad (6-12)$$

where $C(t)$ is the specific creep function versus time, and $\phi(T)$ is a temperature enhancement factor. With the Hannant (1968) data from 167 °F, a four-term exponential series is fit to the data as:

$$C(t) = 0.1936 (1 - e^{-0.069t}) + 0.280 (1 - e^{-0.0069t}) \\ + 0.375 (1 - e^{-0.00069t}) + 0.348 (1 - e^{-0.000069t}) , \quad (6-13)$$

where time is in days and $C(t)$ is in units of 10^{-6} in/in per lbf/in². The temperature shift then is fit from the other Hannant (1967) data with a power series as

$$\phi(T) = 226.09 - 0.00429 T + 147.52 T^{-0.367} - 309.26 T^{-0.044} , \quad (6-14)$$

where T is temperature in °F and $\phi(T)$ is unitless. Figure 6.2-1 shows the resulting creep-compliance curve versus the Hannant (1967) data. Figures 6.2-2 and 6.2-3 compare the fitted equation with the Browne (1968) data (multiple specimens) at 149 and 200 °F, respectively.

Figure 6.2-1. Creep-Compliance Curve Fit of ANACAP-U Hanford
Concrete vs. Hannant Data.

Figure 6.2-2. Creep-Compliance Curve Fit of ANACAP-U Hanford Concrete vs. Browne Data at 65 °C (149 °F).

Figure 6.2-3. Creep-Compliance Curve Fit of ANACAP-U Hanford Concrete vs. Browne Data at 93 °C (200 °F).

As presented in RHO (1981), the creep strains reported from Hanford Site data are contaminated by a time-dependent component of the elastic strains because of thermal degradation of the elastic modulus. Equation 4-1 enables us to remove these time-dependent elastic strains from the data. The apparent creep-strains, $\bar{\epsilon}$, reported for Hanford Site data are determined as:

$$\bar{\epsilon} = \epsilon^T - \epsilon^0, \quad (6-15)$$

where ϵ^T is the measured total strain (excluding thermal expansion) and ϵ^0 is the initial strain. However, the "true" creep strain is:

$$\epsilon^C = J\sigma = \epsilon^T - \epsilon^e \quad (6-16)$$

where J is the "true" creep compliance, σ is the stress, and ϵ^e is the time-varying elastic strain. Combining these equations and solving for the creep compliance yields:

$$J = \frac{\epsilon^0}{\sigma} + \frac{\bar{\epsilon}}{\sigma} - \frac{\epsilon^e}{\sigma}. \quad (6-17)$$

Now, $\frac{\epsilon^0}{\sigma} = \frac{1}{E(0, T)}$ and $\frac{\epsilon^e}{\sigma} = \frac{1}{E(t, T)}$ so that:

$$J = \frac{\bar{\epsilon}}{\sigma} + \frac{1}{E(0, T)} - \frac{1}{E(t, T)} \quad (6-18)$$

is the corrected creep compliance calculated from the reported creep strains $\bar{\epsilon}$ adjusted with the time-and-temperature-dependent modulus.

The following equations for creep compliance based on Hanford Site data are determined by averaging values calculated for reference times of 100 days and 1,000 days.

$$J(t, 350\text{ }^{\circ}\text{F}, 500\text{ lbf/in}^2) = 0.645 \log_{10}(t+1) \quad (6-19a)$$

$$J(t, 250\text{ }^{\circ}\text{F}, 1,500\text{ lbf/in}^2) = 0.205 \log_{10}(t+1) \quad (6-19b)$$

$$J(t, 350\text{ }^{\circ}\text{F}, 1,500\text{ lbf/in}^2) = 0.345 \log_{10}(t+1) \quad (6-19c)$$

Figure 6.2-4 compares the fitted equation with these corrected Hanford Site data and shows reasonably good agreement for the 350 °F curve and conservative predictions for the 250 °F curve. The resulting creep-compliance function given by Equations 6-12 through 6-14 is shown in Figure 6.2-5 as a function of time and temperature.

A useful measure of creep strain is given by the creep coefficient, C_{ct} , defined as the ratio of the creep strain to the initial, instantaneous elastic strain when the load is first applied at temperature, that is:

$$C_{ct} = \frac{\epsilon^c}{\epsilon^e} = \frac{J \sigma}{\sigma/E} = J E \quad (6-20)$$

where J is given by Equation 6-12 and E is given by Equation 4-1 at the time of initial loading. The resulting creep coefficient is shown in Figure 6.2-6 as a function of time and temperature. The ultimate creep coefficient, C_{cu} , is defined as the creep coefficient with the creep strain taken as the final asymptotic value for the design life of the structure. A typical value for 3,000 lbf/in² concrete at normal building temperatures is 3.1 (Nilson and Winter 1991). From Figure 6.2-6 the ultimate creep coefficient ranges from 2 at 100 °F to approximately 10.5 at 350 and 450 °F.

Figure 6.2-4. Creep-Compliance Curve Fit of ANACAP-U
Hanford Concrete vs. Hanford Concrete Creep Data.

Figure 6.2-5. Creep Compliance of ANACAP-U Hanford
Concrete vs. Time at Temperature.

Figure 6.2-6. Creep Coefficient of ANACAP-U Hanford Concrete vs. (a) Time and (b) Temperature.

Creep Coefficient

Creep Coefficient

6.3 RECOMMENDATIONS FOR ANSYS INPUT

Evaluation of the bounds of possible creep has been ever expanding as public domain experimental literature has been reviewed. Although the data are interesting, it shows a wide variability and may be of limited applicability to Hanford Site concrete. This issue is currently under review by the Tank Structural Integrity Panel, but this issue will hopefully be resolved in the future.

The current recommendation is to input creep using current ANSYS creep equations with magnitudes adjusted to compare with the Hannant data (1968, 1960) as fit by James and Rashid (1993) as shown in Figures 6.1-7 and 6.1-9 for ANACAP-U. The creep equations are established in a tabular form for a range of temperatures. It is recommended that a parameter or trend study be completed with these creep magnitudes being factored.

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7.0 SHRINKAGE

Concrete stored in air of less than 100 percent humidity experiences drying shrinkage. This shrinkage is caused by water withdrawal. A part of this water movement is irreversible and should be distinguished from the reversible moisture movement caused by alternating storage under wet and dry conditions.

The most important influence is exerted by the aggregate. The aggregate restrains the amount of shrinkage that can actually be realized. The ratio of shrinkage of concrete to shrinkage of neat paste depends on the aggregate content in the concrete. Figure 7-1 (Pickett 1956) shows typical results.

Even within a range of ordinary aggregates there is a considerable variation in shrinkage, refer to Figure 7-2 (Troxell et al. 1958). If one compares shrinkage shown in Figure 7-2 to creep from the same test series show in Figure 6.1-10, it can be seen that both phenomena are of comparable magnitudes. This result would imply that if creep studies are necessary then shrinkage should also be considered.

Many investigations have in fact indicated an influence of the size of the specimen on shrinkage. The observed shrinkage decreases with an increase in the size of the concrete specimen. The shape of the specimen also appears to enter the picture but as a first approximation shrinkage can be expressed as a function of the volume/surface ratio of the specimen. Figure 7-3 shows there is a linear relation between this ratio and the logarithm of the shrinkage (Hansen and Mattock 1966).

Figure 7-1. Influence of Aggregate Content in Concrete (by volume)
on the Ratio of Shrinkage of Concrete
to the Shrinkage of Neat Cement Paste. (Pickett 1956)

Figure 7-2. Shrinkage of Concretes of Fixed Mix Proportions but Made With Different Aggregates. (Troxell et al. 1958)
Stored in Air at 21 °C (70 °F) and a Relative Humidity of 50%
Time Reckoned Since End of Wet Curing at the Age of 28 days

Figure 7-3. Relation Between Ultimate Shrinkage and Volume/Surface Ratio. (Hansen and Mattock 1966)

8.0 STRESS-STRAIN RELATION

Figures 8-1 and 8-2 show typical non-linear compressive stress-strain curves for concrete (Kottas 1979, Bertero 1972, Kassir 1993). Both the compressive strength and modulus are seen to degrade with temperature. Specific degradation equations have been developed for Hanford Site concrete (see Section 4.0). These specific degradation equations have been utilized in the Accelerated Safety Analysis (ASA) Phase I (Fisher 1994) and Phase II (Scott 1995) waste tank evaluations.

The peak compressive strain (the compressive strain at the maximum compressive stress) is seen to decrease or increase depending on which experimental data set is chosen (see Figures 8-1 and 8-2). The stress-strain curves that exhibit a decreasing peak strain with temperature produce a more brittle and non-ductile behavior in the structure. Therefore it is envisioned that using the data will be more conservative than using the data in which the peak strain increases with temperature (exhibiting increased ductility).

Figure 8-1. Stress-Strain Curves Versus Temperature
With Increasing Peak Strain Behavior.
(Kottas 1979, Bertero 1972, Kassir 1993)

(a) Sealed Specimens

**Stress-Strain Curves vs Temperature
with Increasing Peak Strain Behavior**

Figure 8-2. Stress-Strain Curves Versus Temperature
With Increasing Peak Strain Behavior.
(Kottas 1979, Bertero 1972, Kassir 1993)

Stress-Strain Curves vs Temperature
with Increasing Peak Strain Behavior

Various mathematical expressions have been used to generate the uniaxial non-linear compressive stress-strain curves for use in analysis (Peterson 1974, Peterson 1975, Peterson 1994). The mathematical expressions can be used to generate specific stress-strain values required for input into analysis code material models (see Figure 8-3). It is not envisioned that the choice of a particular mathematical expression will have any significant effect on the analytical results just as long as modulus degradation, strength degradation, and peak strain embrittlement with temperature is captured.

Figure 8-3. Analytic Stress-Strain Curves Versus Temperature
With Decreasing Peak Strain Behavior.

Analytic Stress-Strain Curves vs Temperature
with Decreasing Peak Strain Behavior

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9.0 TENSION STRENGTH AND POISSON'S RATIO

Tension strength is a secondary concern for concrete analysis. The tension strength of concrete cannot be considered to add any significant amount of structural capacity to the reinforced concrete structure (ACI 349, 1990). If it is suspected that the tension strength of concrete has numerically added any additional capacity to the concrete structure then the tension strength must be reduced until the intent of the code (ACI) is met. The analysis methodology must allow for concrete cracking and stress redistribution (Peterson 1995). The tensile strength is currently selected as being 10 percent of the degraded compressive strength. However, as mentioned previously, if the intent of the code is not met, then this value should be reduced until it can be demonstrated that the specified tensile strength does not significantly contribute to the capacity of the structure.

Poisson's ratio is also a parameter that has a secondary effect on the structural capacity. Typical values that have been used in analysis range from 0.15 to 0.20 for both degraded and undegraded concrete. Thus the specific temperature dependency for Poisson's Ratio need not be considered in waste tank analysis unless it is shown to have a significant effect on the structural capacity.

Poisson's ratio for high compressive stress ratios (compressive stress/uniaxial short term compressive strength ≥ 0.75) under sustained loading may range from 0.3 to 1.0 (Linse 1972): failure is eminent for these highly stressed specimens.

10.0 SUSTAINED LOADING

Concrete exhibits a lower apparent strength when subjected to sustained loadings. Strength reductions have been recorded for compressive stress states as low as 0.70 of the short term compressive strength (Stockl 1972, see Figure 10-1). Recent experimental tests have indicated reduced strength effects for stress ratios between 0.75 to 0.80 of the short term compressive strength (Smadi 1985, see Figures 10-2 and 10-3). Sustained loading of concrete in biaxial compression has also indicated a reduced strength effect where the sustained loading strength is 0.84 of the short term compressive strength (Linse 1972, see Figure 10-4). Experimental data that includes both high temperature and sustained loading effects have not been discovered in the available literature. There may be an interaction between temperature degradation and sustained loading affects where the stress ratio limit is further reduced.

Figure 10-1. Strength of Long-Time Loaded Prisms Related to Strength in a 20-Minute Test. (Stockl 1972)

Fig. 4: Strength of long-time loaded prisms related to strength in a 20-min-test

Figure 10-2. Sustained Loading Failure Limits
(Compressive Strength 3030-3540 psi) (Smadi 1985)

Sustained loading Failure Limits
(compressive strength 3030-3540 psi)

Figure 10-3. Sustained Loading Failure Limits
(Compressive Strength 5000-5790 psi) (Smadi 1985)

**Sustained loading Failure Limits
(compressive strength 5000-5790 psi)**

Figure 10-4. Ultimate Strengths of Biaxially Loaded Specimens (Linse 1972).

It has been hypothesized (Smadi 1985) that the limiting mechanism under sustained loading maybe strain dependent, i.e. there is a strain limit when failure will occur. Since creep strains accelerate under higher temperatures, the interaction of sustained loading and temperature will result in larger strains occurring sooner in time for a given stress. Therefore the material will reach the strain limit at a reduced stress ratio due to accelerated creep. The strain limit concept has not been verified by experiment, but if true, the concept will further reduce the allowable stress ratio limit for sustained loading under high temperatures.

The sustained loading compressive strength for concrete will be selected as 0.75 of the short term compressive strength. Compressive stress ratios above 0.75 will not be allowed. A typical allowable axial load-moment (P-M) interaction envelope is shown in Figure 10-5. A maximum compressive stress ratio (stress/uniaxial compressive strength) of 0.78 occurs at the balance point on the P-M envelope. This is above a stress ratio limit of 0.75. A 0.75 stress ratio limit will further reduce the allowable P-M envelope. The additional reduced P-M envelope is indicated in the figure.

Smadi et. al. (1985) implies that there is a critical strain limit (composed of the instantaneous strain and creep strain components) when the incipient failure occurs. Attempts have been made to use this strain limit concept to define the critical stress below at which sustained loading will not cause failure. This resulted in allowable stresses considerably lower than the 0.75 limit proposed herein. The stress limit based on a critical strain limit that includes a creep component shows a greater dependency on temperature since creep rates increase dramatically with increases in temperature. Since the critical strain limit concept needs further experimental work to define the basis of the critical strain limit, explicit use of this criteria will not be made at this time.

Figure 10-5 stress ratio limit line of 0.75 was developed using the initial (nearly linear) stress-strain curve shape without any modification due to creep which occurs when there are sustained loading effects (see Figures 10-2 and 10-3). If creep is allowed, the stress-strain curve up to a stress of 0.75 of f_c' becomes more like the shape of the instantaneous stress-strain curve up to ultimate stress: the shape changes from a linear shape to a more parabolic shape. This parabolic shape can be used to compute the reduced ultimate PM diagram and also a reduced code allowable PM diagram. The reduced PM capacity curves are computed just like the original un-reduced curves. The Whitney stress block method can be used since the shape of the stress-strain curve with sustained loading effects (creep) is basically parabolic in shape just as was the case for the initial stress-strain (without sustained loading effects) curve up to f_c' . This concept must be verified via test.

Figure 10-5. ACI Code and Stress Limit Capacity P-M Curves.

Figure 10-6 shows the ultimate and code allowable PM curves computed without sustained loading effects and the reduced ultimate PM curves computed with sustained loading effects. A 0.75 reduction factor was used for concrete strength. If a section reached a PM value that landed on the ultimate PM capacity curve then that section would fail under instantaneous loading. If a section reached a PM value that landed on the reduced ultimate PM capacity curve then that section may fail under sustained loading. The code allowable curve provides a safety margin to section failure. The fact that the reduced ultimate PM curve is close to the code allowable curve implies that the safety margin is less for sustained loading. Additional safety margin to section failure is provided by the use of the ACI load factors (1.4,1.7), the use of f_c' (a lower bound strength), and by ignoring concrete strength increase due to aging.

If the concrete analysis model or parameter study is using a best estimate strength it may be necessary to look at long term sustained load capability and the resulting margins. In general most evaluation analyses should use the minimum specified 28 day strength. In general the real concrete strength is much higher than the specified strength. The concrete strength continues to increase after the 28 day period.

Figure 10-6. Reduced Capacity Curves.

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11.0 BOND STRENGTH DEGRADATION

Table 11-1 summarizes bond strength reductions due to sustained loading, temperature, cyclic loading, and transverse tensile strain to be discussed herein.

The bond stress data that was found in the public domain literature are of limited scope (due consideration must be given to the aggregate, concrete mix, coefficient of thermal expansion, temperature range). The experimental results were based on a limited range of temperatures. The bond strength variation with a temperature range of interest for Hanford Site waste tank evaluation was conservatively set to the degradation at 300°C (as shown in Table 11.1). A very important parameter for consideration in bond degradation is the differential in thermal expansion between steel and concrete. Hanford Site concrete has a coefficient of thermal expansion of 3.7×10^{-6} in/in/°F. Bond degradation tests did not report the coefficient of thermal expansion of the concrete mix and Table 11-1 results may not be conservative or directly applicable to the Hanford Site concrete.

Table 11-1. Allowable Bond Strength Limits for Temperatures
Between 150° F to 572°F.

LOAD CONDITION	BOND STRESS LIMIT	COMMENTS
sustained load	$R \leq 0.67-0.75$	3,000 hr max
short term load with temperature	$R \leq 1.0$ (limestone) $R \leq 0.77$ (basalt) $R \leq 0.75$ (quartz)	300 °C max
sustained load with temperature	$R \leq 0.5-0.6$	300 °C max
cyclic load	$R \leq 0.5$	16 cycles max
cyclic load with temperature	$R \leq 0.375$ estimated by (0.5×0.75)	300 °C max
transverse strain effect	$(R,et) = (1.,0)$ $(R,et) = (0.,.00475)$	linear reduction

Notes:

R is the bond stress or strength ratio defined as the degraded strength divided by the undegraded strength

et is the transverse tensile strain

Bond Degradation due to Sustained Loading

Bond failure did not occur if the bond stress ratio was less than 0.67 to 0.75 for sustained loadings of up to 3,000 hours (Bartos 1982). The bond stress ratio or strength ratio is defined as the degraded bond strength divided by the short term undegraded bond strength. The upper limit of 0.75 is equal to the reduction for sustained loading of Section 10.0.

Bond Degradation due to Short Term Loading with Temperature

Bond strength has been studied for short term loadings combined with temperature. Figure 11-1 shows the relative bond strength as a function of the applied temperature (Bartos 1982). Figure 11-1 indicates that short term bond failure will not occur for temperatures up to 300 °C for limestone aggregate concrete but quartz aggregate concrete failed at a bond stress ratio greater than 0.75 at approximately 150°C (302°F) (Bartos 1982).

Figure 11-2 shows the bond stress plotted on the vertical axis as a function of bond slip plotted on the horizontal axis and temperature for basalt aggregate. Figure 11-2 indicates that at 300°C short term bond failure will occur at a bond stress ratio of 0.77 (22.1/28.8) for basalt aggregate concrete (Bartos 1982). If the bond stress is kept below 0.77 of the original undegraded stress capability (bond stress ratio) then failure will not occur for temperatures between 20 °C and 300 °C.

Beam specimens that were subjected to heat treatment before cooling and then load tested showed increase in strength properties from 20 °C (68 °F) to 150 °C (302 °F) and then a rapid decrease in strength for higher temperatures (Khan 1986). Most failures were associated with the development of one bold crack in the shear zone. Failure of the beams was in bond-shear (abrupt and without warning) for temperature exposures below 600 °C (1,112 °F). Above 600 °C, the failure mode was in flexure due to the larger relative reduction in compressive strength of the concrete for these higher temperatures (Khan 1986). The beam flexure test results are directly opposite to the bar pullout tests that showed a decrease in strength properties with increasing temperatures (see Figures 11-1 and 11-2). Additional work needs to be done to identify the controlling test and material parameters affecting the test. The conservative approach of reducing bond strength with increasing temperature will be taken.

Figure 11-1. Relative Bond Failure Stress as a Function of Temperature (Bartos 1982).

Relative Bond Failure Stress as a Function of
Temperature

Figure 11-2. Bond Stress-Slip Relationship for Basalt Aggregate (Bartos 1982).

Bond Stress- Slip Relationship for Basalt Aggregate

Bond Degradation due to Sustained Loading with Temperature

The effects of sustained loading plus temperature was also experimentally studied (Bartos 1982). The experimental data indicated that if the bond stress ratio is kept 0.5 to 0.6 or below, then bond failure would not occur for temperatures up 300 °C. Figures 11-3 and 11-4 shows bond failures for concretes with different aggregates and compressive strengths when subjected to sustained loadings combined with high temperature. Temperature is plotted on the vertical axis, while rebar bond slip is plotted on the horizontal axis. As the bond stress ratio is increased, the temperature when bond failure occurs decreases: a bond stress ratio of 0.25 results in a sustained temperature capacity of around 600 to 800 °C, while a bond stress ratio of 0.70 results in a sustained temperature capacity below 200 °C. It is interesting to note that the combined sustained loading and temperature bond stress ratio limit of 0.50 is approximately equal to 0.67 (sustained loading limit with temperature) times 0.75 (temperature limit with short term loading).

Bond Degradation due to Cyclic Loading

Figure 11-5 shows the bond slip as a function of load cycle and bond stress ratio (R). The fast rising slip curves for bond stress ratios of 0.95, 0.75, 0.70, and 0.62 indicate that bond failure will soon occur. The bond slip curve for a bond stress ratio of 0.50 shows a comparatively stable response as load cycles are applied. Bond failures due to cyclic loading will tend not to occur if the bond stress is below 50 percent of the undegraded strength (Bartos 1982, see Figure 11-5). This test series only had 16 cycles of load application so due caution is in order.

Bond Degradation due to Cyclic Loading with Temperature

Bond failure limits for cyclic load combined with temperature has not been discovered in the available literature. An estimate of the bond failure limit may be calculated by multiplying the cyclic load limit (0.5) with the temperature limit of (0.75). This procedure resulted in the correct limit for the sustained load with temperature as discussed earlier. This results in an allowable bond stress ratio limit for cyclic load plus temperature of 0.375 (0.5×0.75). The 0.375 bond stress limit is an estimate that has not been verified with experimental tests. This estimate will be modified if and when experimental data can be located in the literature.

Bond Degradation due to Transverse Loading

Transverse tensile strain (normal to the rebar) has been shown to reduce the apparent bond strength (Bartos 1982). Figure 11-6 shows the reduction in bond strength as a function of the transverse tensile strain for different covers. The reduction in strength has been plotted as a series of straight lines. Bond strength is 1.0 (no degradation) for zero transverse tensile strain and zero (complete degradation) for a transverse tensile strain of .004750 in./in. Degradation in bond strength for rebar with different covers are plotted as straight lines with different degradation slopes.

Figure 11-3. Temperature Displacement Behavior
Under Constant Load for Different Aggregates (Bartos 1982).

Temperature Displacement Behaviour Under Constant
Load for Different Aggregates

Figure 11-4. Temperature Displacement Behavior Under Constant Load for Different Concrete compressive Strengths (Bartos 1982).

Temperature Displacement Behaviour Under Constant Load for Different Concrete Compressive Strengths

Figure 11-5. Increase in Slip Due to Load Cycles (Bartos 1982).

Increase in Slip due to Load Cycles

Figure 11-6. Bond Stress Capacity Ratio as a Function of Transverse Tensile Strain. (Bartos 1982)

Bond Stress Capacity Ratio as a Function of
Transverse Tensile Strain

Summary

The effect of rebar bond degradation can be considered using different approaches. The actual bond stress between the concrete and rebar can be computed and then compared with the allowable degraded bond stress. Also the allowable bond stress ratio can be viewed in terms of a required anchorage or splice length. If the structure was initially designed so as to develop the full yield strength of the rebar with no bond degradation consideration and a reduction of 0.5 due to bond degradation is desired, then a steel stress limit of $0.5 \cdot f_y$ is allowable.

Figures 11-7 and 11-8 compare the reduced ultimate capacity curves which have been reduced for both sustained loading effects and bond strength degradation. Figure 11-7 corresponds to a section with 0.59 percent tensile reinforcement and Figure 11-8 corresponds to a section with 0.18 percent tensile reinforcement. The reduced ultimate PM capacity curves are close to the ACI code allowable curves. The concept of developing PM capacity curves based on the Whitney stress block method to capture sustained loading (creep) and/or bond degradation effects at high temperatures must be verified via testing.

The above high temperature steel stress limit of $0.5 \cdot f_y$ does not have a proper testing and practice basis for code usage. As a result this limit will be used as a discriminator to identify tank sections that require additional engineering evaluation. The rebar-concrete interaction and failure have been characterized by more than one approach.

In the late 60's, the ACI code started to use the developed length instead of the bond theory and during the same time the code increased rebar lap length tremendously. The bond stress reduction due to high temperature would be most important at the lap region. The bond stress reduction is dependent on the actual lap length and the configuration of the rebars in the tank.

Finally, the main concern for bond strength shown herein is based on the temperature environment of the waste tanks and the paucity of relevant data. No contradiction with code practice is intended. Considerable work remains to relate test results with the tank environment in a manner consistent with code practice. In particular, it has been noted that the use of peak bond strength data herein may not be consistent with a code basis relating to lower load plateaus beyond the peaks.

Figure 11-7. Reduced Capacity Curves.

ULTIMATE, CODE, AND REDUCED ULTIMATE CAPACITY CURVES
 SUSTAINED LOAD REDUCTION FACTOR OF $0.75 \cdot F_C$
 BOND STRENGTH REDUCTION FACTOR OF $0.50 \cdot F_Y$
 ($F_C = 3000$ PSI, $70F$, $F_Y = 40$ KSI)
 (0.18% TENSION STEEL)

△ ULTIMATE
 + CODE
 × REDUCED ULTIMATE ($0.75 \cdot F_C$, $0.50 \cdot F_Y$)

Figure 11-8. Reduced Capacity Curves.

12.0 RECOMMENDATIONS FOR CONCRETE FAILURE EVALUATION

In this document Section 2.0 addresses failure surfaces for finite-element modeling. The typically used criteria Mohr-Coulomb has its limitations, and even the best state-of-the-art five parameter Willam-Warnke failure model is no better than the input data. Please read and review the finite-element failure model limitations documented in Section 2.0. But now that the finite-element simulation is producing numerical results, the question is "What parameter studies are necessary and how should the results be checked and evaluated for a safe condition?"

For temperatures less than 150 °F, it is recommended that the forces and moments in the tank concrete sections be checked against the ACI criteria with load factors. It is also recommended that ACI criteria for rebar bond failure be checked. In concrete analyses that include concrete creep the stresses should be checked to see that the maximum concrete stress is less than 0.50 of f_c' (the stress proportional creep limit).

Section 7 shows that concrete shrinkage is of comparable magnitude to concrete creep. It is recommended that shrinkage be considered combined with creep for Hanford Site waste tank analyses.

The concrete failure strength is a very important parameter for all of the finite-element failure surface criteria. It is recommended for nonlinear analysis that a minimum design specification failure strength be used for final safety and safety margin evaluation with a range of concrete stiffnesses that reflects the stiffness variation envelope. If test cylinder data is available for a tank then a potentially less conservative statistical basis may be established for f_c' , following ACI guidelines. Although the final evaluation must be conservative as above, nonlinear analysis trend studies should include a set of best estimate material properties. The important best estimate nonlinear failure mechanisms should be checked against the lower bound evaluation case.

For temperature histories of greater than 150 °F, it is recommended that the concrete strength and modulus be degraded and checked against the ACI section criteria with factored loads. At this time it is recommended that a bond stress factor of 0.5 on rebar yield stress be used as a discriminator to flag tank sections of concern for extra engineering evaluation (please refer to report Section 11.0). The bond stress factor would be combined with the ACI concrete section criteria (PM curves) as was done in report Section 11.0.

The structure is acceptable if the tank structure can meet the conditions of the ACI section criteria with factored loads as above. But if the tank does not meet ACI criteria or the load is beyond the design basis it may be necessary to assess the risk in the form of a margin or safety factor. The calculation of an acceptable basis for safety factor requires additional considerations.

Concrete exhibits a lower strength under long term sustained loads as compared to the strength corresponding to short term (instantaneous) loads if the applied stress level is above .75 to .80 of the short term strength, (see Figures 10-2 and 10-3 [Smadi et al. 1985]). Smadi et al. (1985) showed in Table 12-1 that test specimens at 0.80 of instantaneous failure strength (f_c') can fail in one to three hours. The sustained loading strength at high temperature (150 to 350 °F) will also be selected as 0.75 of the degraded concrete strength evaluated at the appropriate time and temperature. Data to support this value of 0.75 for high temperature operation has yet to be verified with experimental tests. It is recommended that the sustained loading strength of concrete be similarly assigned a value of .75 of the short term best estimate strength (f_c) under normal temperature operation. It is recommended that tank collapse finite element load studies also include analysis results with the failure strength factored down by 0.75. Please note that if minimum specification concrete strength (f_c') is used for the basis of the safety factor calculation it may not be necessary to factor the strength down by multiplying by 0.75. The strength can increase with age after 28 days.

Table 12-1. Summary of Time-Dependent Strain Data for Low-Strength Concrete.

Series	Ratio of Stress to f'_c	psi (MPa)	Age at loading (days)	Total strain $\mu\text{in.}/\text{in.}$	Initial strain, $\mu\text{in.}/\text{in.}$	Creep strain $\mu\text{in.}/\text{in.}$	Creep coefficient	Specific creep, $\mu\text{in.}/\text{in.}/\text{psi}$ ($\mu\text{cm}/\text{cm}/\text{MPa}$)	Time at failure (min)
L-95	0.95	3470 (23.9)	28	2615	1803	813	0.451	0.247 (35.8)	0.38
L-90	0.90	3030 (20.9)	28	3166	1405	1761	1.253	0.646 (93.7)	2.25
L-85	0.85	3030 (20.9)	28	4249	1245	3004	2.413	1.166 (169.2)	35.00
LM-80*	0.80	4070 (28.1)	28	3532	1279	2253	1.762	0.692 (100.4)	52.00
LM-80*	0.80	4070 (28.1)	28	3613	1315	2298	1.748	0.706 (102.4)	151.00' 180.00'

*Lies between low- and medium-strength concretes.

'One specimen failed 151 min after beginning of sustained loading. The other did not fail at 3 hr after loading. Load was then increased to failure. Each value is the average of two specimens.

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13.0 REFERENCES

- ACI Committee 517, 1963, *Low Pressure Steam Curing*, Journal of the American Concrete Institute, Proc. 60, pp. 953-986.
- ACI, 1989, *Building Code Requirements for Reinforced Concrete*, ACI 318-89, American Concrete Institute, Detroit, Michigan.
- ACI, 1990, *Code Requirements for Nuclear Safety-Related Concrete Structures*, ACI 349-90, American Concrete Institute, Detroit, Michigan.
- ACI, 1993A, "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures", ACI 209R-82, *ACI Manual of Concrete Practice Part 1*, American Concrete Institute, Detroit, Michigan.
- Argyris, J. H., G. Faust, J. Szimmat, E. P. Warnke, and K. J. Willam, 1974, Recent Developments in Finite Element Analysis of Prestressed Concrete Reactor Vessels, " *Nuclear Engineering and Design*, Vol. 28, pp. 42-75.
- Bartos, P., 1982, *Bond in Concrete*, edited by P. Bartos, Proceedings of The International Congress on Bond in Concrete, Dept. of Civil Engineering, Paisley College of Technology, Scotland, June 1982.
- Bertero, V. V., and M. Polivka, 1972, *Influence of Thermal Exposures on Mechanical Characteristics of Concrete*, Paper SP 34-28, Vol. 1, Concrete for Nuclear Reactors, American Concrete Institute, Detroit.
- Browne, R. D., 1968, "Properties of Concrete in Reactor Vessels," *Conference on Prestressed Concrete Pressure Vessels at Church House, Westminster*, The Institute of Civil Engineers, London, 1968, pp. 131-152.
- Chen, W. F., 1981, *Plasticity in Reinforced Concrete*, McGraw-Hill Book Company, Inc., New York.
- Chen, W. F. and A. F. Saleeb, 1994, *Constitutive Equations for Engineering Materials, Volume 1: Elasticity and Modeling*, Second revised edition, Elsevier Science B. V., The Netherlands.
- Chuang, J. W., T. W. Kennedy, and E. S. Perry, *An Approach to Estimating Long-Term Multiaxial Creep Behavior From Short-Term Uniaxial Creep Results*, Union Carbide Report No. 2864-3, The University of Texas, Austin, Texas.

- Chuang, J. W., T. W. Kennedy, E. S. Perry, and J. N. Thompson, 1972, *Prediction of Multiaxial Creep From Uniaxial Creep Tests*, ACI Special Publication SP-34 Concrete For Nuclear Reactors Vol. II., Paper SP 34-35.
- CTL, 1981, *Effects of Long-Term Exposure to Elevated Temperatures on the Mechanical Properties of Hanford Concrete*, RHO-C-54, Construction Technologies Laboratories for Rockwell Hanford Operations, Richland, Washington.
- Davis, R. E., and H. E. Davis, 1931, *Flow of Concrete Under the Action of Sustained Loads*, ACI Journal, Proc. 27, pp. 410-417.
- de la Pena, C., *Shrinkage and Creep of Specimens of Thin Section*, RILEM Bulletin No. 3, pp. 60-70.
- Duke, C. M., and H. E. Davis, 1944, *Some Properties of Concrete Under Sustained Combined Stresses*, Proceedings of the American Society for Testing and Materials, Vol 44, pp. 888-896.
- Dutron, R., 1957, *Creep of Concrete. The Influence of Variations in the Humidity of the Ambient Atmosphere*, Sixth Congress of the IABSE, Stockholm, Preliminary Publication, pp. 57-65.
- Eibl, J. and G. Ivanyi, 1976, Studie zum Tragund Verformungs-verhalten von Stahlbeton (in German with English summary), *Deutscher Ausschuss für Stahlbeton*, Heft 260, Berlin.
- Fisher T. W., L. L. Hyde, C. J. Moore, W. S. 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.
- Gopalakrishnan, K. S., A. M. Neville and A. Ghali, 1969, *Creep Poisson's Ratio of Concrete Under Multiaxial Compression*, Journal of the American Concrete Institute, Title No. 66-90, pp. 1008-1019.
- Gvozdev, A. A., 1966, *Creep of Concrete*, in *Mekhanika Tverdogo Tela*, Moscow, pp. 137-152.
- Hannant, D. J., 1966, *The Strain Behavior of Concrete Under Compressive Stress at Elevated Temperatures*, Research and Development Report, Central Electricity Research Laboratories, London, England.
- Hannant, D. J., 1968, *Strain Behavior of Concrete Up To 95 °C Under Compressive Stresses*, Conference on Prestressed Concrete Pressure Vessels at Church House, Westminster, The Institute of Civil Engineers, London, pp. 177-192.

- Hansen, T. C., 1960, *Creep of Concrete. The Influence of Variations in the Humidity of the Ambient Atmosphere*, Sixth Congress of the IABSE, Stockholm, Preliminary Publication, pp. 57-65.
- Hansen, T. C. and A. H. Mattock, 1966, The influence of size and shape of member on the shrinkage and creep of concrete, *J. Amer. Concr. Inst.*, 63, pp. 267-90, Feb.
- Henager, C. H., G. F. Piepel, W. E. Anderson, P. L. Koehmstedt, and F. A. Simonen, 1988, *Modeling of Time-Variant Concrete Properties at Elevated Temperatures*, PNL-7779, Pacific Northwest Laboratory, Richland, Washington.
- HKS, 1989, *ABAQUS User's Manual*, Version 4.8 with 4.9 Supplement, Hibbitt, Karlsson & Sorensen, Inc., Pawtucket, Rhode Island.
- Jaeger, J. C., 1956, *Elasticity, Fracture and Flow*, Methuen and Company, Limited, Lobdon, England.
- 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.
- Jensen, R. S., and F. E. Richart, 1938, *Short-Time Creep Tests Of Concrete In Compression*, ASTM Proc. 38, Part 2, pp. 410-417.
- Jones, P. G., and F. E. Richart, 1936, *The Effect Of Testing Speed On Strength And Elastic Properties Of Concrete*, ASTM Proc. 36, Part 2, pp. 380-391.
- Julyk, L. J., T. J. Bander, J. P. Day, A. D. Dyrness, R. S. Marlow, C. J. Moore, E. W. Pianka, and M. P. Weis, 1993, *Tank 241-C-106 Structural Integrity Evaluation For In Situ Conditions*, WHC-SD-W320-ANAL-001, Westinghouse Hanford Company, Richland, Washington.
- Kassir, M. K., K.K. Bandyopadhyay, and M. Reich, 1993, *Thermal Degradation of Concrete in the Temperature Range from Ambient to 315 °C (600 °F)*, BNL-52384, UC-406, Engineering Research & Applications Division, Department of Nuclear Energy, Brookhaven National Laboratory, Associated Universities, Inc., Upton, N.Y., June 1993.
- Keeton, J. R., 1960, *Time-Dependent Deformations of Plain Concrete*, Highway Research Board, Proc. 39, pp. 310-335.
- Khan, M. R. and R. Royles, 1986, *Post Heat Exposure Behaviour of Reinforced Concrete Beams*, Magazine of Concrete Research, Vol. 38, No. 135, June 1986.

- Kordina, K., 1960, *Experiments On The Influence Of The Mineralogical Character Of Aggregates On The Creep Of Concrete*, RILEM Bulletin No. 6, pp. 7-22.
- Kottas, R., et al., 1979, *Strength Properties of Concrete in the Temperature Range of 20 °C to 200 °C*, Paper H1/2, Vol H, Session 1, Transaction of the 5th International Conference on Structural Mechanics in Reactor Technology, Berlin, Germany, August 1979.
- L'Hermite, R. G. and M. Mamillan, 1968, *Further Results of Shrinkage and Creep Tests*, Intern. Conf. on the Structure of Concrete, London, Cement and Concrete Association, pp. 423-433.
- Linse, D., 1972, *Strength of Concrete Under Biaxial Sustained Load*, Paper SP 34-17, Vol. 1, Concrete for Nuclear Reactors, American Concrete Institute, Detroit.
- MacGregor, J. G., 1992, *Reinforced Concrete: Mechanics and Design*
- Marechal, J. E., 1969, *Le Fluage du Beton en Fonction de la Temperature*, Material and Structures (RILEM), Vol. 2 No. 8, pp. 111-115.
- Nadai, A., 1950, *Theory of Flow and Fracture of Solids*, Vol. 1, McGraw-Hill Book Company, Inc., New York.
- Nasser, K. W. and H. M. Mazouk, 1981, *Creep of Concrete at Temperatures from 70 to 450 °F Under Atmospheric Pressure*, Journal of the American Concrete Institute, Proc. 60, pp. 953-986.
- Neville, A. M., 1959, *Role Of Cement In The Creep Of Mortar*, Journal of the American Concrete Institute, Proc. 55, pp. 963-984.
- Neville, A. M., 1970, *Creep of Concrete: Plain, Reinforced, and Prestressed*, American Elsevier Publishing Company.
- Neville, A. M., 1971, *Hardened Concrete: Physical and Mechanical Aspects*, ACI Monograph No. 6, American Concrete Institute, Detroit, Michigan.
- Neville, A. M., 1973, *Properties of Concrete*, John Wiley & Sons, New York, New York.
- Paul, B., 1968, "Macroscopic Criteria for Plastic Flow and Brittle Fracture," *Fracture, an Advanced Treatise*, Vol.2, H. Liebowitz, Editor, Academic Press, New, York.
- Peterson, W. S., 1974, *The Inelastic Analysis of Reinforced Concrete Slabs*, Lehigh University, Fritz Engineering Laboratory, Report No. 378B.3, Bethlehem, Pennsylvania.

- Peterson , W. S., 1975, *The Inelastic Analysis of Reinforced and Prestressed Concrete Beam-Slab Highway Bridge Superstructures*, Ph.D. Dissertation, Lehigh University, Fritz Engineering Laboratory, Report No. 378B.3, Bethlehem, Pennsylvania.
- 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.
- Peterson, W. S., R. S. Marlow, M. P. Weiss, 1995, *Ansys and Abaqus Finite Element Code Collapse Load Prediction, Comparison and Verification*, WHC-SD-ANAL-044, Rev. 0, Westinghouse Hanford Co., Richland, Washington.
- Peterson, W. S., 1995, Tank Structural Integrity Panel, presentation, Westinghouse Hanford, Richland, Wa., Aug 1995.
- Pickett, G., 1956, *Effect Of Aggregate On Shrinkage Of Concrete And A Hypothesis Concerning Shrinkage*, Journal of the American Concrete Institute, Proc. 52, pp. 581-590.
- PNL, 1979, *Strength and Elastic Properties of 1580-Day Hanford Concrete Cylinders at Room Temperature and 350 F*, Pacific Northwest Laboratory, Construction Technology Laboratories, A Division of the Portland Cement Association, Skokie, Illinois.
- RHO, 1979, *Elastic and Strength Properties of Hanford Concrete Mixes at Room and Elevated Temperatures*, RHO-C-28, Construction Technology Laboratories, Skokie, Illinois.
- RHO, 1981, *Effects of Long-Term Exposure to Elevated Temperature on The Mechanical Properties of Hanford Concrete*, RHO-C-54, Rockwell Hanford Operations, Richland, Washington.
- Ross, A. D., 1964, *Experiments On The Creep Of Concrete Under Two-Dimensional Stressing*, Magazine of Concrete Research, Vol 16, No. 46.
- Rusch, H., 1960, *Research Toward A General Flexural Theory For Structural Concrete*, Journal of the American Concrete Institute, Title No. 57-1, pp. 1-28.
- Scott, M. A., W. S. Peterson, 1995, *Accelerated Safety Analysis Structural Analyses Phase II ACI Code Evaluations of the Maximum Loads in the Concrete Dome, Haunch, and Upper Wall of the Double-Shell Waste Storage Tanks*, WHC-SD-WM-SARR-032, Rev. 0, Westinghouse Hanford Company, Richland, Washington.

- Smadi, M. M., F. O. Slate, and A. H. Nilson, 1985, *High-, Medium-, and Low-Strength Concretes Subjected to Sustained Overloads- Strains, Strengths, and Failure Mechanisms*, ACI Journal, Title No. 82-58, Vol. September-October 1985.
- Stockl, S., 1972, *Strength of Concrete Under Uniaxial Sustained Loading*, SP 34-16, Vol. 1, Concrete for Nuclear Reactors, American Concrete Institute, Detroit.
- Troxell, G. E., J. M. Raphael, and R. E. Davis, 1958, *Long-Time Creep and Shrinkage Tests of Plain and Reinforced Concrete*, ASTM Proc. 58, pp. 1101-1120.
- Winkel, R. V., 1994, *Concrete Material Characterization, Reinforced Concrete Tank Structure, Multi-function Waste Tank Facility*, WHC-SD-W236A-ER-013, Westinghouse Hanford Company, Richland, Washington.
- York, G. P., T. W. Kennedy, E. S. Perry, and J. N. Thompson, 1972, *Experimental Determination Of Poisson's Ratio For Creep*, ACI Special Publication SP-34 Concrete for Nuclear Reactors Vol I, Paper SP 34-29.