MAR 03 1995

ENGINEERING DATA TRANSMITTAL

Page 1 of 1

1. EDT

609359

2. To: (Receiving Organization) Multi-Function Waste Tank Facility Project W236A		3. From: (Originating Organization) Structural Assessments		4. Related EDT No.: N/A							
5. Proj./Prog./Dept./Div.: MWTF		6. Cog. Engr.: B. V. Winkel		7. Purchase Order No.: N/A							
8. Originator Remarks: This report documents the Multi-Function Waste Tank Facility Project position on the concrete mechanical properties needed to perform design/analysis calculations for the MWTF secondary concrete structure				9. Equip./Component No.: N/A							
				10. System/Bldg./Facility: N/A							
11. Receiver Remarks:				12. Major Assm. Dwg. No.: N/A							
				13. Permit/Permit Application No.: N/A							
				14. Required Response Date: 11/30/94							
15. DATA TRANSMITTED											
(A) Item No.	(B) Document/Drawing No.	(C) Sheet No.	(D) Rev. No.	(E) Title or Description of Data Transmitted	(F) Impact Level	(G) Reason for Trans- mittal	(H) Orig- inator Dispo- sition	(I) Receiv- er Dispo- sition			
1	WHC-SD-W236A-ER-013	---	0	Concrete Material Characterization Reinforced Concrete Tank Structure Multi- Function Waste Tank Facility	3Q	1	1				
16. KEY											
Impact Level (F)		Reason for Transmittal (G)			Disposition (H) & (I)						
1, 2, 3, or 4 (see MRP 5.43)		1. Approval 2. Release 3. Information 4. Review 5. Post-Review 6. Dist. (Receipt Acknow. Required)			1. Approved 2. Approved w/comment 3. Disapproved w/comment 4. Reviewed no/comment 5. Reviewed w/comment 6. Receipt acknowledged						
(G)	(H)	17. SIGNATURE/DISTRIBUTION (See Impact Level for required signatures)						(G)	(H)		
Reason	Disp.	(J) Name	(K) Signature	(L) Date	(M) MSIN	(J) Name	(K) Signature	(L) Date	(M) MSIN	Reason	Disp.
1	1	Cog. Eng. B.V. Winkel	<i>B. Winkel</i>	12-21-94	H5-57	K.C. Tu	<i>K.C. Tu</i>	1/30/95	E6-50	4	
1	1	Cog. Mgr. S.K. Farnworth	<i>S.K. Farnworth</i>	2/2/95	H5-55	M.D. Stine	<i>M.D. Stine</i>	2/7/95	E6-50	3	
1	1	QA A.K. Sharma	<i>A. Sharma</i>	2/2/95	H5-60						
		Safety									
		Env.									
18.		19.		20.		21. DOE APPROVAL (if required) Ltr. No. N/A					
B. V. Winkel <i>B. Winkel</i> 12-21-94 Signature of EDT Originator		J. M. Light <i>J. M. Light</i> 2/2/95 Authorized Representative for Receiving Organization		S. K. Farnworth <i>S.K. Farnworth</i> 1/19/95 Cognizant/Project Engineer's Manager		☐ Approved ☐ Approved w/comments ☐ Disapproved w/comments					

SUPPORTING DOCUMENT		1. Total Pages 05-89
2. Title Concrete Material Characterization Reinforced Concrete Tank Structure Multi-Function Waste Tank Facility	3. Number WHC-SD-W236A-ER-013	4. Rev No. 0
5. Key Words concrete, multi-function waste tank, material properties, double-shell tanks	6. Author Name: B. V. Winkel Signature Organization/Charge Code 8D440/DTTC3	
7. Abstract This report documents the Multi-Function Waste Tank Facility Project position on the concrete mechanical properties needed to perform design/analysis calculations for the MWTF secondary concrete structure <div style="text-align: center;">DISCLAIMER</div> <p>This report was prepared as an account of work sponsored by an agency of the United States Government. Neither the United States Government nor any agency thereof, nor any of their employees, makes any warranty, express or implied, or assumes any legal liability or responsibility for the accuracy, completeness, or usefulness of any information, apparatus, product, or process disclosed, or represents that its use would not infringe privately owned rights. Reference herein to any specific commercial product, process, or service by trade name, trademark, manufacturer, or otherwise does not necessarily constitute or imply its endorsement, recommendation, or favoring by the United States Government or any agency thereof. The views and opinions of authors expressed herein do not necessarily state or reflect those of the United States Government or any agency thereof.</p>		
		8. RELEASE STAMP <div style="border: 1px solid black; padding: 10px; text-align: center;">OFFICIAL RELEASE BY W-236A IMT DATE MAR 03 1995 STATION (36) CLERK (77)</div>

DISCLAIMER

Portions of this document may be illegible in electronic image products. Images are produced from the best available original document.

RELEASE AUTHORIZATION

Document Number: WHC-SD-W236A-ER-013, Rev. 0

Document Title: Concrete Material Characterization Reinforced
Concrete Tank Structure Multi-Function Waste Tank
Facility

Release Date: 03/01/95

**This document was reviewed following the
procedures described in WHC-CM-3-4 and is:**

APPROVED FOR PUBLIC RELEASE

WHC Information Release Administration Specialist:

V.L. Birkland
V.L. Birkland

3/1/95

TRADEMARK DISCLAIMER. Reference herein to any specific commercial product, process, or service by trade name, trademark, manufacturer, or otherwise, does not necessarily constitute or imply its endorsement, recommendation, or favoring by the United States Government or any agency thereof or its contractors or subcontractors.

This report has been reproduced from the best available copy. Available in paper copy and microfiche. Printed in the United States of America. Available to the U.S. Department of Energy and its contractors from:

U.S. Department of Energy
Office of Scientific and Technical Information (OSTI)
P.O. Box 62
Oak Ridge, TN 37831
Telephone: (615) 576-8401

Available to the public from:

U.S. Department of Commerce
National Technical Information Service (NTIS)
5285 Port Royal Road
Springfield, VA 22161
Telephone: (703) 487-4650

TABLE OF CONTENTS

1.0	Introduction	1
2.0	Summary and Recommendations	2
2.1	MWTF Concrete Mechanical Property Recommendations	2
2.2	ACI Code	4
2.3	Effect of Moisture on Properties	4
2.4	Use of ANACAP-U Software	5
2.5	Effect of Mix Design	6
2.6	Hanford/Literature Concrete Property Comparisons	7
3.0	MWTF Concrete Mix Design	7
3.1	Mix Design Compatibility with Data Base	7
3.2	Effects of Aggregates	8
4.0	Effects of Temperature on Concrete Properties	9
4.1	Compressive Strength	10
4.2	Modulus of Elasticity	14
4.3	Tensile Strength	17
4.4	Creep	17
4.5	Thermal Expansion	18
4.6	Poisson's Ratio	19
5.0	ACI 349 Code Design Issues	19
5.1	Design Strength	19
5.2	Modulus of Elasticity	21
5.3	Thermal Expansion Coefficient	21
5.4	ACI 359 Issues	21
6.0	References	23
	Appendix A: Strength and Modulus of Elasticity Calculations	A-1
	Appendix B: Thermal Expansion Coefficient Test Results	B-1
	Appendix C: PNL Correlations Revised to Incorporate 70° Data	C-1
	Appendix D: ORNL Review Comments and Comment Resolutions	D-1

LIST OF FIGURES

4-1	70 °F Strength Data Versus Time	12
4-2	Residual Strengths, as Predicted by the PNL Equations	12
4-3	Reduction of Compressive Strength of Concrete at Elevated Temperatures.	13
4-4	Room Temperature Modulus of Elasticity Versus Time.	15
4-5	Residual Modulus as Predicted by the PNL Equations.	15
4-6	Variation of the Modulus of Elasticity of Concrete with Temperature (Hot and Cold Testing).	16
5-1	Compressive Strength Degradation Versus Time Predictions at 200 °F.	20
5-2	Modulus of Elasticity Predictions for Hanford Concrete.	22

LIST OF TABLES

2-1	Summary of Recommended MWTF Concrete Mechanical Properties (Age > 6 Mos., Dry Concrete)	3
2-2	Effects of Moisture on the Mechanical Properties of Concrete . .	6
3-1	Comparison of CTL 4,500 lb/in ² and HWVP 5,000 lb/in ² Concrete Design Mixes	8

CONCRETE MATERIAL CHARACTERIZATION
REINFORCED CONCRETE TANK STRUCTURE
MULTI-FUNCTION WASTE TANK FACILITY

PROJECT W-236A

Issued by
WESTINGHOUSE HANFORD COMPANY
OCTOBER 1994

for the
U.S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
RICHLAND, WASHINGTON

Prepared by:	<u>B. V. Winkel</u> B. V. Winkel, Fellow Engineer Facility Stress Analysis	<u>11-17-94</u> Date
Reviewed by:	<u>L. D. Blackburn</u> L. D. Blackburn, Fellow Engineer Materials and Welding Engineering	<u>11-17-94</u> Date
Approved by:	<u>Susan K. Farnworth</u> S. K. Farnworth, Manager Structural Assessments	<u>1-19-95</u> Date

CONCRETE MATERIAL CHARACTERIZATION
REINFORCED CONCRETE TANK STRUCTURE
MULTI-FUNCTION WASTE TANK FACILITY

1.0 INTRODUCTION

The purpose of this report is to document the Multi-Function Waste Tank Facility (MWTF) Project position on the concrete mechanical properties needed to perform design/analysis calculations for the MWTF secondary concrete structure. This report provides a position on MWTF concrete properties for the Title I and Title II calculations. The scope of the report is limited to mechanical properties and does not include the thermophysical properties of concrete needed to perform heat transfer calculations.

In the 1970's, a comprehensive series of tests were performed at Construction Technology Laboratories (CTL) on two different "Hanford concrete" mix designs (CTL 1981). Statistical correlations of the CTL data were later generated by Pacific Northwest Laboratories (PNL) (Henager et al. 1988). These test results and property correlations have been utilized in various design/analysis efforts of Hanford waste tanks. However, due to changes in the concrete design mix and the lower range of MWTF operating temperatures, plus uncertainties in the CTL data and PNL correlations, it was prudent to evaluate the CTL data base and PNL correlations, relative to the MWTF application, and develop a defensible position.

The CTL test program for Hanford concrete involved two different mix designs: a 3 kip/in² mix and a 4.5 kip/in² mix. The proposed 28-day design strength for the MWTF tanks is 5 kip/in². In addition to this design strength difference, there are also differences between the CTL and MWTF mix design details. Also of interest, are the appropriate application of the MWTF concrete properties in performing calculations demonstrating ACI Code compliance. Mix design details and ACI Code issues are addressed in Sections 3.0 and 5.0, respectively.

The concrete design temperature for the MWTF concrete structure is 200 °F. The CTL test program and PNL data correlations focused on a temperature range of 250 to 450 °F. The temperature range of interest for the MWTF tank concrete application is 70 to 200 °F.

This concrete characterization document was reviewed internally and also by Dan Naus from Oak Ridge National Laboratory (ORNL). The ORNL review comments and comment resolutions are provided in Appendix D.

2.0 SUMMARY AND RECOMMENDATIONS

2.1 MECHANICAL PROPERTY RECOMMENDATIONS

A summary of the recommended mechanical properties at 70 °F and 200 °F for the MWTF secondary concrete tank design is provided in Table 2-1. Because of property uncertainties, ranges of values are given. Detailed discussions of each property are provided in the section referenced in the Comments column. A summary discussion is provided in this section. The range of values reported in Table 2-1 are based upon laboratory tests. Greater variations are likely to occur in the field, as discussed in Section 2.3. However, since there is insufficient data to address variations in the field, the Table 2-1 ranges are recommended for the design calculations. For temperatures between 70 and 200 °F, a linear interpolation may be performed. A "low temperature" time/temperature correlation is also available, as discussed in Section 4.1 and Appendix C.

As indicated in the table caption, the Table 2-1 mechanical properties represent dry concrete, with an age greater than six months. The "dry" condition comes from the fact that the test database involved oven-dried cylinders, with the exception of the 70 °F specimens, which were "tested wet." As discussed in Section 2.3, below, an attempt was made to appropriately modify the 70 °F test data, to obtain a consistent "tested dry" data base. The dry condition is expected to be closer to field conditions than a wet condition. The data base test specimens were all greater than six months in age when tested. It is expected that the tank concrete will reach six months of age prior to being placed in service.

When performing structural calculations on the MWTF secondary concrete structure, it is important to distinguish between (1) structural analysis predictions for the various prescribed load combinations and (2) structural adequacy evaluations. In general, structural analysis predictions should be based upon best-estimate or mean mechanical properties. Having obtained the best estimate stress/strain (section load/deformations), the structural adequacy evaluation should generally be based upon minimum strength. For ACI Code adequacy evaluations, the minimum compressive strength is defined by f'_c . As discussed in Section 2.2, an appropriate temperature degradation must be applied to the minimum strength for temperatures above 150 °F.

There are exceptions to the use of best-estimate material properties. For some applications, it may be obvious that an appropriate combination of minimums and maximums is more conservative and therefore should be used. For example, to obtain conservative thermal stresses, the maximum thermal expansion coefficients and elastic moduli should be used.

Concerning "thermal creep" analyses of the MWTF concrete structure, Section 6.1.2.3 of the MWTF design guidelines (Atalay et al. 1993) specifies four distinct results: (1) ACI 349, Section 9.2.1, Eq. (9) check, (2) Determination of the secondary and primary tank anchorage displacements, (3) Beginning and end-of-life cracked section properties for seismic evaluation, (4) A collapse check for creep buckling. For the first three

Table 2-1. Summary of Recommended MMTF Concrete Mechanical Properties (age > 6 mos., dry concrete).

Property	Temp. (°F)	Time (yr)	Range			Comments
			Min.	Mean	Max.	
ACI Code Nominal Design Strength	70 to 150 151 to 200	0 to 50 "	N/A	5,000 4,000	N/A	A 20% thermal degradation factor is assumed for a 200 °F design temp. See Section 5.1.
Best Estimate Compressive Strength, f_c (lb/in ²)	70 200	0 50 0 50	6,460 8,290 5,930 5,530	7,750 9,580 7,220 6,820	9,040 10,870 8,510 8,110	Mean values from new "low temperature" data correlation. See Section 4.1.1. Min./Max. from PNL 95/95 tolerance band.
Modulus of Elasticity, E (10 ⁶ lb/in ²)	70 200	0 50 0 50	3.2 3.7 3.2 2.4	4.0 4.5 4.0 3.2	4.8 5.3 4.8 4.0	Mean values from new "low temperature" correlation. See Section 4.1.2. Min./Max. from PNL 95/95 tolerance band.
Splitting Tensile Strength, f_t (lb/in ²)	70 200	0 50 0 50	370 685 430 390	490 805 550 510	610 925 670 630	Mean values from new "low temperature" correlation. See Section 4.1.3. Min./Max. from PNL 95/95 tolerance band.
Thermal Expansion Coefficient (10 ⁻⁶ in/in/°F)	70 to 200	0 to 50	3.2	3.7	4.2	Based upon testing of specimens using the HWVP 5 kip/in ² mix. See Section 4.1.5. Min./Max. from 2 sigma variation in the HWVP Concrete test data.
Poisson's Ratio	70 200	0 to 50	.15 .10	.18 .13	.21 .16	Based upon a statistical evaluation of the CTL test results at 70 °F and 250 °F. Min/max are 2σ var.

results, mean properties are appropriate. However, for the collapse analysis, it is more appropriate to use minimum strengths to assure a conservative collapse prediction.

2.2 ACI CODE

From the ACI Code viewpoint, the most important concrete property is the "specified compressive strength of concrete", f'_c . The currently specified f'_c for the MWTF concrete is 5,000 lb/in². Normally, the definition of f'_c is limited to concrete temperatures up to 150 °F. Since the MWTF concrete tank design temperature of 200 °F exceeds the 150 °F limit, it is necessary to apply an appropriate reduction factor, based upon test data, as discussed in Appendix A of ACI 349. As demonstrated in Section 5.1, for a design life of 50 years at 200 °F, a design strength reduction of 20% is a reasonably conservative estimate of the potential thermal degradation. This is the basis for the recommended high-temperature design strength value of 4,000 lb/in², for performing ACI design verification calculations involving concrete temperatures above 150 °F. It is emphasized that this high temperature strength reduction does not alter the room temperature nominal design mix value of 5,000 lb/in² for f'_c , which will be specified in the construction specification.

As discussed in Section 5.0, ACI 349 does provide quantitative definitions of elastic modulus, thermal expansion coefficient, and effective moment of inertia. However, it is obvious in Chapter 9 of ACI 349 that structural calculations are not limited to the code values. For example, in Section 9.5 it states that a "more comprehensive analysis or test" may be performed in deflection calculations using "accepted engineering data." Thus, for comprehensive analyses (e.g., nonlinear finite element analyses) it appears to be permissible to use material correlations in performing the calculations. Recommendations relative to the use of these material correlations, which have been incorporated into the ANACAP-U software (James and Rashid 1993), are provided in Section 2.4.

Other than the reduction in the f'_c for ACI design calculations above 150 °F and the thermal expansion coefficient, the concrete properties shown in Table 2-1 are based upon material correlations of the CTL data (CTL 1981). The mean values are based upon modifications of correlations, originally developed by PNL (Henager et al. 1988). The modifications were intended to improve the accuracy for temperatures below 250 °F (see Appendix C). In addition to including data from 70 °F tests, the "low temperature" modification to the material correlation attempted to account for moisture differences between the 70 °F "tested wet" and the high-temperature data, which involved oven-dried specimens, as discussed in Section 2.3.

2.3 EFFECT OF MOISTURE ON PROPERTIES

One concern in adding the 70 °F data to the higher temperature data is that the 70 °F specimens were moist, whereas the higher temperature specimens were dry, due to heating in the oven. As discussed in Section 4.0 and summarized in Table 2-2, significant differences can exist between "tested wet" and "tested dry" specimens. For example, the compressive strength of

"wet-cured, tested-dry" concrete is 20 to 40% higher than for "wet-cured, tested-wet" concrete. Since during operations the tanks are closer to the "tested dry" condition, the Table 2-1 values are presented as "tested dry" properties, which necessitated an adjustment to the 70 °F data, which was "tested wet." Note also that the Table 2-1 concrete properties are for "mature concrete" (age > 6 months). The CTL high temperature specimens were all cured for at least six months prior to testing. Thus, the Table 2-1 properties at time = 0 are for six-month-old concrete, with the exception of the 28-day minimum design strength, f_c' .

Another potentially significant aspect of moisture is the effect of moisture during the curing process. As shown, for example, in Figure 10.7 of Troxell and Davis 1956, the compressive strength of concrete can be significantly higher for moist-cured specimens than for "air-cured" specimens. Moisture wise, the curing conditions in the field vary considerably. Most curing specifications require a minimum moist cure, in the field, of seven days. Since this is considerably less than the six-month moist cure associated with the CTL data (CTL 1981), there is concern that the CTL database may not be representative of the field-cured concrete in the Hanford waste tanks. For the waste tank concrete, the inside liners are used as the permanent inside forms of the walls and dome. After removal of the outside forms, the 18-in-thick walls are subjected to drying on the outside. Thus, long-term moist curing is not likely, at least for the outside of the walls and dome.

Some evidence of the potential impact of field curing is presented in Appendix A, which compares the correlation predictions with actual compressive strength data obtained from testing cores taken from Hanford concrete structures. The concrete core data indicate that the mean field strength is comparable to the laboratory correlation predictions. However, the field data are considerably more scattered. Therefore, the Table 2-1 range is probably too small for concrete properties in the field.

2.4 USE OF ANACAP-U SOFTWARE

The PNL Hanford concrete material correlations (Henager et al. 1988) have been incorporated into the ANACAP-U software (James and Rashid 1993) which has been used in various waste tank computer analyses in the past few years. For temperatures above 200 °F, ANACAP-U uses the PNL correlations as prescribed by PNL. For temperatures below 200 °F, ANACAP-U used trends obtained from the literature. For the MWTF concrete temperature range of interest (70 to 200 °F), the accuracy of the ANACAP-U material representation could be improved.

This need for improving ANACAP-U for the lower temperature range, as well as other improvement needs, has been discussed with ANATECH Corporation. Modifications by ANATECH to the ANACAP-U software was initiated in the summer of 1994, but, at the time of this report writing, had not been completed due to lack of funding.

Revision 0

Table 2-2. Effects of Moisture on the Mechanical Properties of Concrete.

Property	Effect of Moisture	Comments
Compressive Strength	Concrete tested dry exhibits 20 to 40% higher compressive strength than concrete tested wet.	See Section 4.1.1 for discussion. Due to anticipated operating temperatures, MWTF concrete is expected to be closer to the dry condition.
Tensile Strength	Similar effect as compressive strength, but not quantitized in the literature reviewed.	See Section 4.1.3 for discussion.
Modulus of Elasticity	Dry specimens test 15 to 30% lower than wet specimens (opposite to moisture effect on strength).	See Section 4.1.2 for discussion. 70 °F modulus data decreased by 20% to approximate the dry condition modulus.
Poisson's Ratio	No discussions found, in literature reviewed, on effect of moisture on Poisson's Ratio	No adjustments for moisture made for Poisson's ratio values.
Creep	Creep in 0% ambient relative humidity approximately double the creep in 100% ambient humidity.	See Section 4.1.4 for discussion. ANACAP-U room temp. creep correlation is likely unconservative because it was probably derived from moist concrete testing.
Thermal Expansion Coefficient	Little difference between the tested dry and tested wet results for Hanford concrete.	See Section 4.1.4 for discussion and Appendix B for test results. No adjustment made to values associated with moisture.

2.5 EFFECT OF MIX DESIGN

Since the CTL test program involved specific concrete mix designs that are different from the anticipated MWTF mix design, there is some concern relative to the adequacy of the PNL correlations for the MWTF concrete. This concern is discussed in Section 3.0. The discussion in Section 3.0 is based upon the assumption that the MWTF design mix will correspond to the HWVP 5,000 lb/in² design mix. The primary difference in the mix designs is the use of admixtures in the HWVP design, specifically fly ash and superplasticizer. The effect of these admixtures on the high temperature properties of concrete is

addressed in Nasser and Chakraborty 1985. No detrimental effects of these admixtures were reported. On the contrary, the fly ash admixture appeared to improve the high temperature strength relative to mix designs without fly ash. Cement/aggregate differences between the HWVP and CTL mixes, however, do increase the uncertainty in the PNL correlations when applied to the MWTF project.

2.6 HANFORD/LITERATURE CONCRETE PROPERTY COMPARISONS

The effects of elevated temperature on the mechanical properties of concrete has received periodic attention in the literature for the past few decades. A recent compilation of the literature data is documented in (Kassir et al. 1993). In Sections 4.1.1 and 4.1.2 of this report, compressive strength and modulus of elasticity versus temperature comparisons are made between the "Hanford concrete" correlation and the data found in the literature. In general, the literature does not address the effect of time at temperature. In the Hanford concrete correlations, the time parameter is significant, as illustrated in Figures 4-3 and 4-6. As demonstrated in these figures, the 10-day and one-year Hanford concrete predictions, fall within the bounds of the literature data. However, the 50-year Hanford concrete predictions drop below the literature data. The 50-year predictions come from an extrapolation of the Hanford concrete data base, which includes data for up to 3.5 years of time at temperature. Since no 50-year data is available, the accuracy of the extrapolation cannot be assessed. Since the long-term extrapolations are lower than the literature data, for most applications the Hanford concrete correlation is conservative relative to the literature.

3.0 MWTF CONCRETE MIX DESIGN

3.1 MIX DESIGN COMPATIBILITY WITH DATA BASE

Current plans are to specify a 28-day design strength of 5,000 lb/in² for the MWTF tanks. The MWTF project is planning on utilizing the HWVP project batch plant, using either the MC5P or MC6P 5,000 lb/in² mix design from HWVP specification B-595-A-A900. The only difference between the MC5P and MC6P mix designs is an air entrainment additive in the MC5P mix. The CTL test program for "Hanford concrete" (CTL 1981) involved two mix designs, a 3,000 lb/in² mix and a 4500 lb/in² design mix. A comparison of the CTL 4500 lb/in² mix design details with the HWVP 5,000 lb/in² design mixes is provided in Table 3-1.

Note that there are differences in the mix designs, the most significant of which is the reduction in the amount of cement in the current mix (Fluor Daniel 1993). This reduction in cement follows a trend of the past few decades and results, in part, from newer cements having higher earlier strengths. On the other hand, the HWVP mix uses fly ash, which retards the rate of strength gain. Thus, it is expected that reduced cement/fly ash effect should have a minor effect on the strength-time relationship relative to the CTL mix data.

Table 3-1. Comparison of CTL 4,500 lb/in² and HWVP 5,000 lb/in² concrete design mixes.

Batch Ingredients	Quantity (lb/yd ³)	
	CTL Mix Design (4,500 lb/in ²)	HWVP Mix Design (MC5P, 5,000 lb/in ²)
Portland Cement, Type II	653	455
Pozzolan (Fly Ash)		80
Aggregate	1,880	1,975
Sand	1,240	1,400
Water	286	240
Admixtures	Darex, 2% Soln., 4.5 oz.	Superplasticizer (HWR)
Air (% by vol.)	5	2
Water/Cement Ratio	0.43	0.45

Another concern is the elevated temperature effect of the HWVP use of flyash and high water reducers (HWR, also called superplasticizers). The effect of these admixtures on the high temperature properties of concrete was investigated by (Nasser and Chakraborty 1985). No evidence of detrimental effects of these admixtures was reported by (Nasser and Chakraborty 1985). In fact, it appeared that the fly ash actually improved the effect of temperature on the compressive strength. It should be mentioned, however, that a specific HWR was used in the test program by (Nasser and Chakraborty 1985), namely a "naphthalene" superplasticizer. The HWVP concrete specification specifies the HWR by acceptable brand names, and may not be a naphthalene type.

Due, in part, to the differences between the CTL database mix design and the proposed HWVP design mix for the MWTf tanks, current plans (Winkel 1994) call for the placement of test cylinders in the annulus regions between the primary and secondary liners. By periodically removing cylinders from the tank environment, and testing the cylinders for compressive strength, etc., the adequacy of using the CTL data base for the MWTf concrete can be evaluated.

3.2 EFFECTS OF AGGREGATES

As indicated in Table 3-1, about 75% of the concrete weight is in the sand and gravel. Thus, for evaluating data consistency, it is necessary to address sand and gravel effects when making concrete comparisons. Due to the geological/flooding history of the Hanford Site, significant variations in sand and gravel can exist among the various sand/gravel sources. Due, in part, to this site aggregate variability, a precise, quantitative approach is avoided in the discussions below. A second justification for a qualitative approach, is the fact that Hanford concrete aggregate is generally a secondary

player in concrete structural issues. This statement assumes that the aggregates used are sound, clean, compatible with the cement, and properly graded. For important structures on the Hanford site, this assumption is reasonable, as indicated by a review of several Hanford concrete construction specifications. The HWVP standard mix designs included prequalified gravel pits (Century West Engineering 1993) as the aggregate source. These gravel pits were used in the development of the HWVP mix designs and are expected to be used for the MWTF concrete.

Aggregates affect different concrete properties in different ways. In general, for normal weight aggregates, the type of aggregate has a "minor effect" on the concrete strength (Troxell and Davis 1956). The shape and stiffness of the aggregate can have some effect on the modulus of elasticity (Neville 1971). Also, the thermal expansion coefficient of concrete is directly related to the thermal expansion properties of the aggregate (ACI 1993A).

The sand and gravel for the CTL test cylinders (CTL 1981) originated from the "Hanford batch plan." CTL performed petrographic studies, which are reported in Appendix J of (CTL 1981). The coarse aggregates are described as being "principally basalt, dolerite, and gabbro, with lesser amounts of quartz, plagioclase, and biotite gneiss." The fine aggregates are described as "fragments of the above-mentioned rocks; and in addition, metasandstones of various types, metaquartzite, quartz diorite, and argillite."

A recent report (Century West Engineering 1993) presents the results of aggregate evaluations from two different Hanford Site pits. A petrographic examination of the aggregate makeup of the pits (two samples per pit) were similar, stating that "basalt is the dominant rock type identified." A relatively recent petrographic analysis of a gravel sample from the ACME pit in south Richland (Chen-Northern 1989) also indicated that the gravel was dominated by basalt.

As indicated by the above discussions, there is some consistency in the aggregate type in the Hanford area, i.e. the aggregate is principally basalt. Since aggregate tends to be a secondary player in the important concrete properties, it is reasonable to assume that aggregate variations are not a major concern when discussing variability of "Hanford concrete." The one possible exception to this assumption is the thermal expansion coefficient, which is discussed in Section 5.3.

4.0 EFFECTS OF TEMPERATURE ON CONCRETE PROPERTIES

Although the MWTF concrete tank design temperature of 200 °F is low relative to some of the existing tank applications, it is above the ACI temperature effects threshold of 150 °F, and therefore it is necessary to consider temperature effects on the concrete properties. The effects of temperature on waste tank concrete was addressed in a relatively comprehensive test program performed by Construction Technologies Laboratory (CTL) (CTL 1981). The CTL data was later statistically evaluated by Pacific Northwest Laboratories (PNL) (Henager et al. 1988). The subsections below address the application of the CTL/PNL data/correlations to the MWTF project.

Utilizing the data in (CTL 1981), PNL developed "Hanford Concrete" material correlations as a function of time, t , temperature, T , and nominal strength, S . There are some concerns associated with using the PNL correlations for the MWTF application: (1) The data base used by PNL included data from the 250 °F to 450 °F temperature range. For the MWTF application, the range of interest is the 70 °F to 200 °F temperature range. The 70 °F data, reported by CTL, were not included in the correlations. (2) PNL did not specifically address the effects of aging of the concrete in the material correlations. For the MWTF temperature range, aging is more significant and should be more carefully addressed. (3) Neither CTL nor PNL addresses the effect of moisture on the concrete properties. Moisture losses due to heating of the moist specimens can have a significant impact on the concrete mechanical properties, as demonstrated in the subsections below. The moisture issue may be relevant when "wet" 70 °F concrete test results are correlated with "dry" higher temperature data. The effect of moisture during curing (laboratory versus field curing) can also be significant, as discussed in Section 2.3.

To address the above weaknesses in the original PNL correlations, additional correlations were developed that included the 70 °F data, the effects of moisture, and aging. The results of this PNL correlation revision are provided in Appendix C and are discussed in the subsections below.

4.1 COMPRESSIVE STRENGTH

The CTL data (CTL 1981) and subsequent PNL compressive strength correlation (Henager et al. 1988) were input into Mathcad 4.0 and evaluated relative to the MWTF tank application. The evaluation details are provided in Appendix A. A summary of the evaluation findings is provided below.

The effect of the moisture on the compressive strength of concrete is significant. Section 10.17 of Troxell and Davis 1956 states that a "compression specimen tested in the air-dry condition will exhibit 20 to 40% higher strength than that of corresponding concrete tested in a saturated condition." In order to have a consistent data correlation, the 70 °F "tested wet" data should be adjusted to correlate properly with the "tested dry" high-temperature data, since the high-temperature specimens were exposed to drying in the oven. This was done by increasing the 70 °F compressive strength data for the revised low temperature strength correlation. Since the waste tanks operate at elevated temperatures, the "tested dry" condition should be more representative of the in-service condition of the tanks. The 70 °F compressive strength estimates, given in Table 2-1, were obtained from 70 °F data curve fitting, discussed below, with a +20% adjustment to represent a "tested dry" condition.

Concrete strength evaluations available in the literature are generally reported in the form of "residual strength," which is defined as the measured strength divided by an appropriate "initial" or "reference" strength such as the 28-day design strength. The definition of the denominator (reference strength) can make a significant difference in the resulting residual strength. For the high-temperature Hanford concrete information reported in Henager et al. 1988 and CTL 1981, the data and correlations cover a relatively long-time period. This added complexity of time suggested that the reference

strength be defined as a time-dependent variable, to account for aging at room temperature. Another significant factor in defining the reference strength is whether to use "tested wet" or "tested dry" strengths. Since the standard room temperature test involves moist concrete, the reference strengths used in Appendix A and reported below were for moist concrete.

Since considerable scatter was evident in the reported room temperature data, it was necessary to develop a best-fit reference strength curve, as shown in Figure 4-1. The data shown is from the 4.5 kip/in² design mix correlation. The 3.0 kip/in² design mix data was not utilized since the 4.5 kip/in² data is closer to the 5.0 kip/in² design strength currently specified for the MWTF tanks. From the curves shown, the "70F Curve Fit" curve was judged to be the best estimate of the 70 °F strength, and was used as the reference strength in developing residual strength data. Although the 70 °F data for the 3.0 kip/in² design mix was not used in the 70F curve fit, it was utilized in the low temperature time/temperature/strength correlations described in Appendix C.

Using the PNL compressive strength correlation, residual strength curves were generated for times of ten days, one year, and 50 years, as shown in Figure 4-2. There are a large number of papers in the literature that address the effects of temperature on concrete properties. A recent report (Kassir et al. 1993) summarizes data from several sources. Typically, as in the Kassir et al. 1993 report, the effect of time at temperature is not addressed.

In order to provide a comparison of the "Hanford concrete" residual strength results with strength-temperature data reported in the literature, the Figure 4-2 residual strength curves were superimposed onto a similar plot containing upper and lower bound strength data from several testing programs (Kassir et al. 1993) in Figure 4-3. Since the Hanford concrete was "tested hot," the applicable lower bound is the "tested hot" curve. Since the time at temperature is not emphasized in the Kassir et al. 1993 report, and is simply reported as "varying periods" for the Kassir et al. 1993 upper and lower bound curves, it is highly probable that the maximum time at temperature is relatively short. Typical test programs, reported in the literature, indicate bake times ranging from a few days to a few weeks. The PNL correlations were developed from data with oven times ranging from 3 to 1304 days (3-1/2 years). Note that the ten-day and one-year PNL curves fall between the upper and lower bound curves, a fact that lends some credibility to the PNL strength correlations. The 50-year prediction from the PNL strength equation involves a 46.5-year time extrapolation and is conservative relative to the composite data reported in Kassir et al. 1993.

Literature discussions of the significance of time at temperature, as indicated by the PNL correlation, are very limited. This absence of addressing test duration is likely due to the fact that most all test data bases cover relatively short times and the significance of the time parameter was lost in the data scatter. In the review of several papers, only two were found which reported temperature effects testing beyond a month (Nasser and Chakraborty 1983 and Carette and Malhotra 1985). In both of these cases, the duration of testing was much less than for the Hanford concrete testing (four months for Carette and six months for Nasser). Thus, the validity of the significance of time, indicated by the CTL data and PNL correlation, is

Figure 4-1 70 degF strength data and curve fitting results.

Note: The time scale shown is time beyond 193 days to be consistent with the high temperature data, which was cured for six months prior to testing. See Appendix A.

T = 200, 250, 350

Figure 4-2 Residual strengths, as predicted by the PNL correlations

Figure 4-3. Reduction of Compressive strength of Concrete at Elevated Temperature.

difficult to quantitatively verify in the literature. However, nothing was found in the literature which conflicted with the CTL/PNL data/correlations.

As discussed in the beginning of Section 4.1, the PNL correlations were revised in Appendix A to cover the MWTf temperature range of interest (70 to 200 °F). The revised mean compressive strength equation is

$$f_c = 4222 + 861.91*S - 1.199*(165.00 - T)*\ln(t+1) - 1.313*S*T$$

where f_c = compressive strength, lb/in²
 S = nominal strength, kip/in²
 T = temperature, °F ($T < 350$ °F)
 t = time at temperature, days.

This revised correlation was used to obtain the 200 °F mean compressive strength estimates in Table 2-1. The upper and lower bound estimates were obtained by assuming a 1,290 lb/in² 95/95 tolerance band, which came from the PNL statistical evaluation.

4.2 MODULUS OF ELASTICITY

The CTL data (CTL 1981) and subsequent PNL modulus of elasticity correlation (Henager et al. 1988) were input into Mathcad 4.0 and evaluated relative to the MWTf tank application. The evaluation details are provided in Appendix A. A summary of the evaluation findings is provided below.

The concrete moisture content can have a significant effect on the modulus of elasticity, but is opposite in effect than for compressive strength. That is, the "tested wet" results are higher than the "tested dry" results. From Figure 13.5 of Troxell and Davis 1956, the wet specimens tested from 15 to 30% higher than the dry specimens. In order to have a consistent "dry" data set, the 70 °F data moduli were reduced by 20% for the revised lower temperature data correlation.

The PNL correlation prediction for elastic modulus versus time at 70 °F is shown in Figure 4-4. Figure 4-4 illustrates the inadequacy of the PNL extrapolation of elastic modulus for temperatures well below 250 °F. Note that a significant reduction of modulus with time is predicted by the PNL correlation (33% at 50 years, as shown in Appendix A). As discussed in Section 4.1, the room temperature modulus should increase with time. A more reasonable room temperature correlation was obtained by assuming a linear with log time, which is shown as the "70F Fit" curve in Figure 4-4.

Using the "Log Fit" correlation (tested wet) as the room temperature reference modulus, the "residual modulus" curves shown in Figure 4-5 were obtained. The wet reference strength was used to be consistent with the approach typically used in the literature. These residual modulus curves were transposed onto a comparable figure (Figure 4-6) containing typical data from the literature (Kassir et al. 1993). Note that the 50-year predictions predict a larger thermal degradation, relative to the composite data reported by Nasser et al. 1993, particularly at temperatures below 200 °F.

Figure 4-4 Room temperature modulus of elasticity versus time.

Note: See Fig. 4-1 for time scale explanation.

Figure 4-5 Residual Modulus as predicted by the PNL correlations

Figure 4-6. Variation of the Modulus of Elasticity of Concrete with Temperature (Hot and Cold Testing)

--- HANFORD CONCRETE CORRELATION

As discussed in the beginning of Section 4.1, the PNL correlations were revised in Appendix C to cover the MWTf temperature range of interest (70 °F to 200 °F). The revised mean elastic modulus equation is

$$E = 3.329 + .126*S + .001*(124.82 - T)*\ln(t+1),$$

where E = elastic modulus, 1×10^{-6} lb/in²
 S = nominal strength, kip/in²
 T = temperature, °F ($T < 250$ °F)
 t = time at temperature, days.

This revised correlation was used to obtain the mean elastic modulus estimates in Table 2-1. The upper and lower bound estimates were obtained by assuming a 0.76 kip/in² 95/95 tolerance band, which came from the PNL statistical evaluation. This low temperature correlation also includes a 20% decrease in the 70 °F data to represent a "tested dry" condition, as discussed in Section 2.3.

4.3 TENSILE STRENGTH

As discussed in the beginning of Section 4.1, the PNL correlations were revised in Appendix C to cover the MWTf temperature range of interest (70 °F to 200 °F). The revised mean splitting tensile strength equation is

$$f_t = 310.7 + .46675*T + 29.288*S + .2808*(184.34 - T)*\ln(t+1)$$

where f_t = splitting tensile strength, lb/in²
 S = nominal strength, kip/in²
 T = temperature, °F ($T < 250$ °F)
 t = time at temperature, days.

This revised correlation was used to obtain the 200 °F mean splitting tensile strength estimates in Table 2-1. The upper and lower bound estimates were obtained by assuming a 120 lb/in² 95/95 tolerance band, which came from the PNL statistical evaluation. This correlation also included a 20% increase in the 70 °F data to account for the "tested dry" condition.

4.4 CREEP

Three duplicate creep tests were run on 4.5 kip/in² concrete cylinders by CTL (CTL 1981). One pair of tests were run at 250 °F and the other two at 350 °F. A statistical review of this data was performed by PNL (Henager et al. 1988), using this limited data. The creep test specimens were kept moist prior to testing. Total strains were reported, with no apparent separation of shrinkage strains or elastic strains due to temperature degradation of the elastic modulus. Later ANATECH (James and Rashid 1993) attempted to "correct" the CTL creep results to account for modulus degradation. As discussed in Section 5.0, ANATECH developed a creep formulation from literature data, which was compared to the Hanford concrete data.

From the brief review performed for this report, it was obvious that there is significant uncertainty in the Hanford concrete creep. ANATECH states (James and Rashid 1993) that the ANACAP-U formulation is "conservative," relative to the very limited 250 °F CTL data. The ANATECH definition of conservative means that the ANACAP-U formulation over predicted the CTL data creep strains. However, for the ACI required strength calculations, lower bound creep may be more conservative, because of potential peak stress reduction, due to creep-related stress redistribution.

4.5 THERMAL EXPANSION COEFFICIENT

The Hanford 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 analysts.

To help resolve the Hanford concrete expansion coefficient issue, additional tests were initiated in 1993. The first tests were run on cored specimens from an older Hanford structure (T Plant, 1944 vintage). As indicated by the results reported in Appendix B, 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, which is somewhat contrary to Section 2.9.2 of ACI 209R (ACI 1993A). The temperature range for the test data was about 50 °F, approximately the lower third of the MWTF temperature range of interest. The resulting expansion coefficient is assumed to be independent of temperature, which is based upon the experience of the Bureau of Reclamation (testing agency). This position is also consistent with Figure 16.5 of Neville 1971 which 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 ($f'_c = 2,500 \text{ lb/in}^2$ versus $5,000 \text{ lb/in}^2$ for the MWTF tanks). 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. As shown in Appendix B, 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} . These HWVP test results are recommended for the MWTF design and are included in Table 2-1.

Since the expansion coefficient of concrete is strongly influenced by the expansion coefficient of the aggregate, there is reason for concern that Hanford 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 aggregate variability. As discussed in Section 3.2, Hanford 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} , which is higher than the range of the T Plant measurements.) Although "Hanford 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, provides some evidence of the uniformity of the Hanford concrete aggregate, at least relative to the thermal expansion coefficient.

4.6 POISSON'S RATIO

Table B-2 of (CTL 1981) lists Poisson's ratio test results for both the 3.0 and the 4.5 kip/in² mix designs. Excluding batches 1 and 2 (see 2nd page of Appendix A) and using the 4.5 kip/in² results, mean values of 0.175 and 0.129 were obtained for temperatures of 70 and 250 °F, respectively. The corresponding standard deviations were 0.0145 and 0.0150, respectively. Based upon a review of the data, test duration did not appear to have a significant effect on the Poisson's ratio values.

5.0 ACI 349 CODE DESIGN ISSUES

5.1 DESIGN STRENGTH

The normal ACI 349-93 Code (ACI 1993B), hereafter referred to as "the code", design approach is to specify a 28-day, room-temperature compressive strength, f'_c , which is used in the "required strength" calculations in Section 9.2 of the code. The standard approach for demonstrating concrete strength requirements is to perform strength tests on moist-cured specimens which are tested moist at room temperature. The normal code operating temperature limit is 150 °F. However, Section A.4.3 of the code states that higher temperatures are allowed if "tests are provided to evaluate the reduction in strength and this reduction is applied to design allowables." Test details are not addressed.

A logical first step for addressing the effects of temperature above 150 °F is to perform 28-day, moist-specimen tests at room temperature and the maximum operating temperature (200 °F). As discussed in Section 4.1, the existing Hanford concrete database consists of a combination of room-temperature, tested-wet and elevated-temperature, tested-dry results. Also, the elevated-temperature tests were performed on concrete with a minimum age of six months. In Appendix C, an effort is made to develop time/temperature correlations for "dry" concrete. The predicted temperature dependence from the Appendix C compressive strength correlation is shown in Figure 5-1. Note that at time zero (age $\approx$ 6 months), the 200 °F strength is about 89% of the room temperature strength, indicating a 200 °F temperature degradation of about 11%.

The effect of time at temperature also needs to be addressed. Note that Figure 5-1 illustrates that the ratio of 200 to 70 °F strength decreases with time. This decrease is significant because the 200 °F strength is diminishing and the 70 °F strength is increasing, due to aging. Since the code does not permit taking credit for strength increases beyond 28 days, it is not reasonable to compare the 50-year room-temperature strength to the 50-year strength at the design temperature. A more reasonable approach for addressing the effects of time on the elevated-temperature design strength is to compare

Figure 5-1 Compressive strength degradation versus time predictions at 200 F.

(a) PNL mean and lower bound predictions

t := 0, 100.. 18000

(b) PNL residual strength prediction

the short-term room-temperature strength to the 50-year, 200 °F strength. As indicated in Figure 5-1, this comparison indicates that the 50-year thermal degradation is about 16% less than the short-term room-temperature strength. Thus, a reasonable degradation on the specified f'_c of 5,000 lb/in² is 20%, resulting in a design strength of 4,000 lb/in² at 200 °F.

The adequacy of a 4,000 lb/in² design strength for a 200 °F/50-year design temperature/design life can be further demonstrated by considering a lower bound strength prediction (95/95 tolerance band). As indicated in Figure 5-1, the 200 °F/50-year lower bound strength prediction is 4,410 lb/in², which exceeds the 4,000 lb/in² design strength. This lower bound prediction also includes a 20% reduction to correspond to a "tested wet" strength, to be consistent with the ACI requirement for design strength based upon moist concrete.

This 20% reduction is consistent with the position taken on the HWVP project. As discussed in Van Geem 1992, CTL recommended a 20% reduction in design strength for an operating temperature of 200 °F, for the HWVP project. This position was based upon the observation that the maximum decrease noted from all of the 250 °F test data was 20%.

5.2 MODULUS OF ELASTICITY

For design/analysis purposes, a lower bound modulus is not necessarily conservative. For example, thermal stresses are proportional to the modulus and therefore a lower bound modulus would be unconservative. Also, for soil-structure interaction calculations, a higher modulus (stiffer structure) may yield higher stresses. Therefore, for code design purposes, a range of modulus values will be discussed.

The code states that the modulus of elasticity for concrete may be taken as 57,000 times the square root of f'_c . Since Hanford concrete modulus data is available, more accurate values than this approximation can be utilized. Predicted elastic modulus values for Hanford concrete at 70 °F and 200 °F are shown in Figure 5-2, including upper and lower bound predictions. The 70 °F predictions are based upon curve fitting of room-temperature data, presented in Appendix A. The 200 °F predictions are from the PNL correlations. Note that a relatively large range of modulus values is predicted.

5.3 THERMAL EXPANSION COEFFICIENT

Section A.3.3.d of the code specifies that the "coefficient of thermal expansion may be taken as 5.5×10^{-6} per °F unless other values are substantiated by tests." As discussed in Section 4.1.5, recent tests have indicated a 2σ range of thermal expansion values of 3.2 to 4.2×10^{-6} /°F, with a mean value of 3.7×10^{-6} /°F.

5.4 ACI 359 ISSUES

The requirements for concrete property testing in Chapter 3 of ACI 349 (ACI 1993B) are relatively sparse. There is a joint ASME-ACI committee

Figure 5-2 Modulus of Elasticity Predictions for Hanford concrete.

(a) Low temperature correlation, predictions at 200 F.

(b) 70 F Log fit predictions

(ACI 359) that is responsible for the concrete reactor vessel and containment code, which is published in ASME Section III, Division 2 (ASME 1993). Section CC-2230 of ASME III-2 presents a much more comprehensive list of concrete material properties. The material properties discussed in CC-2230 are more compatible with the high temperature waste tank programs than are the requirements of ACI 349. Thus, some consideration should be given to utilizing some of the requirements of Subsection CC of ASME III-2.

6.0 REFERENCES

- 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.
- ACI 1993B, "Code Requirements for Nuclear Safety Related Concrete Structures," ACI 349-90, *ACI Manual of Concrete Practice, Part 4*, American Concrete Institute, Detroit, Michigan.
- ASME 1993, *ASME Boiler and Pressure Vessel Code, Section III, Rules for Construction of Nuclear Power Plant Components, Division 2, Code for Concrete Reactor Vessels and Containments*, American Society of Mechanical Engineers, New York, New York.
- Atalay, B. M., et al., 1993, *Structural Design Guidelines for Multi-Function Waste Tank Facility Underground Tanks and Piping for Project W-236A*, WHC-SD-W236A-DGS-001, Westinghouse Hanford Company, Richland, Washington.
- Bertero, V. V., and M. Polivka, 1971, "Influence of Thermal Exposures on Mechanical Characteristics of Concrete," *Concrete for Nuclear Reactors, Volume 1*, SP-34, American Concrete Institute, Detroit, Michigan.
- Carette, G. C., and Halhotra, V. M., 1985, "Performance of Dolostone and Limestone Concretes at Sustained High Temperatures," *Temperature Effects on Concrete*, ASTM STP-858, American Society of Testing Materials, Philadelphia, Pennsylvania.
- Century West Engineering, 1993, *Aggregate Source Evaluation, Sites 23 and 30, Hanford Nuclear Reservation for the Hanford Waste Vitrification Project*, Century West Engineering Corp. for UE&C Catalytic, Inc., Richland, Washington.
- Chen-Northern, 1989, *Results of Petrographic Testing Performed by Twin City Testing For Kaiser Grout Storage Facility Project No. B-714*, Chen-Northern, Inc. to ACME Concrete Company, Richland, Washington.
- 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.
- Fluor Daniel, 1993, *Ready-Mixed Concrete Production and Delivery*,

B-595-A-A900, Fluor Daniel Advanced Technology Division for the U.S. Department of Energy, Richland Operations Office, Richland, Washington.

- Gillen, M., 1979, *Strength and Elastic Properties of 1580-Day Hanford Concrete Cylinders at Room Temperature and 350F*, Construction Technology Laboratories for Battelle, Pacific Northwest Laboratories, Richland, Washington.
- Henager, C. H., et al., 1988, *Modeling of Time-Variant Concrete Properties at Elevated Temperatures*, Pacific Northwest Laboratory, Richland, Washington.
- James, R. J., and Y. R. Rashid, 1993, *ANACAP-U Theory Manual Supplement for Hanford Concrete*, ANA-QA-124, Rev. 2, ANATECH Research Corporation, LaJolla, California.
- Julyk, L. J., 1994, *Tank 241-C-106 Structural Integrity Evaluation for In Situ Conditions*, WHC-SD-W320-ANAL-001, Westinghouse Hanford Company, Richland, Washington.
- Kassir, M. K., et al., 1993, *Thermal Degradation of Concrete in the Temperature Range from Ambient to 315 °C(600 °F)*, Brookhaven National Laboratory, Upton, New York.
- Nasser, K. W., and M. Chakraborty, 1983, "Temperature Effects on Strength and Elasticity of Concrete Containing Admixtures", *Temperature Effects on Concrete*, ASTM STP-858, American Society of Testing Materials, Philadelphia, Pennsylvania.
- Neville, A. M., *Hardened Concrete: Physical and Mechanical Aspects*, American Concrete Institute Monograph No. 6, Detroit, Michigan.
- Troxell, G. E. and H. E. Davis, 1956, *Composition and Properties of Concrete*, McGraw-Hill Book Company, Inc., New York, New York.
- Van Geem, M. G., 1992, *January 1992 Update on Evaluating Predicted Long-Term Properties of Concrete at Elevated Temperatures* (letter SFF-008 to R. P. Kumar of Fluor Daniel, Inc.), Construction Technology Laboratories, Skokie, Illinois.
- Winkel, B. V., 1991, *105-KE/105-KW Irradiated Fuel Storage Basins Concrete and Reinforcing Bar Characterization, Interim Report*, WHC-SD-NR-SA-023, Westinghouse Hanford Company, Richland, Washington.
- Winkel, B. V., 1994, *In-Service Inspection Recommendations, Reinforced Concrete Tank Structure, Multi-Function Waste Tank Facility*, WHC-SD-W236A-013, Westinghouse Hanford Company, Richland, Washington.

APPENDIX A

STRENGTH AND MODULUS OF ELASTICITY CALCULATIONS

INTRODUCTION

The Hanford concrete strength and modulus equations, developed by PNL, were entered into the Mathcad program in order to develop tabular/graphic data for the purpose of comparing to appropriate data in the literature. Data was generated for ten days, one year, and 50 year bake periods. For the strength data, temperatures above 350 degF were not considered because the PNL equation form changes at 350 deg. and the MWTF tank design temperature is 200 degF.

The PNL equations include the dimension of time, which adds complexity to the terms "residual strength" and "residual modulus". The "residual" values calculated below are defined as the ratio of the value divided by the room temperature value for the same time. This time dependency of the denominator is necessary because the 70 deg. values also change magnitude with time, due to aging, as illustrated below. Although, all concrete cylinders were aged prior to testing for at least 193 days, aging effects beyond the initial curing time appears to be a significant factor when utilizing the PNL data and correlations.

The 70F 28-day design strength for the MWTF tanks is 5 ksi. The PNL correlations include a design strength factor, S, which is 5 for the MWTF application. As shown below, the 70F, 4.5 ksi CTL data was utilized, which required that a 5 ksi "correction factor" be applied, when comparing to actual data for 4.5 ksi concrete, based upon the PNL design strength factor, S.

PNL Correlation Equations:

$$S = 4.5$$

$$f_c(t, T) = (4416.338 + 490.919 \cdot S) - 230.241 \cdot \ln(t + 1) - 1.273 \cdot (350 - T) \cdot \ln(t + 1) \quad f_c(0, 200) = 6.625 \cdot 10^3$$

$$Rf_c(t, T) = \frac{f_c(t, T)}{f_c(t, 70)} \quad (\text{Residual Strength definition})$$

$$E(t, T) = 5.3947 + 0.1233 \cdot S - 0.006751 \cdot T - 0.1786 \cdot \ln(t + 1)$$

$$RE(t, T) = \frac{E(t, T)}{E(t, 70)} \quad (\text{Residual Modulus definition})$$

$$t = 0, 100 \dots 18300$$

Note: $t = 18,250$ days = 50 yr. (tank design life)

These 70 degF estimates are based upon correlations with a minimum temperature of 250 degF, and are not likely to be accurate when extrapolated to 70F. The inadequacy of the extrapolation is very obvious for the modulus prediction, which shows a significant decrease with time, when it should be increasing. The data base available to PNL included 70 degF data, which was utilized below to evaluate and improve the 70 degF correlations.

70 Degree Data Utilization:

Strength Data

$i := 1..10$

$t4_i := (\text{READ}(\text{time4kr})) - 193$

Note: There are 15, 70F data points. However, 3 points were removed (batches 1 & 2) because a different cement was used (all the rest were "Lonestar" cement. By removing these 3, the correlation value, corr, was significantly improved (from .5 to .8, 1 is best). The 193 day time shift was used to be consistent with the high temperature data which was cured for > 6 months, prior to testing. The last two data points were taken from a separate CTL report (Gillen 1979).

$fc4_i := (\text{READ}(\text{strn4kr}))$ $\ln4_i := \ln(t4_i)$

Linear Fit:

$a4 := \text{slope}(t4, fc4)$ $a4 = 1.028$

$b4 := \text{intercept}(t4, fc4)$ $b4 = 6.355 \cdot 10^3$

$c4 := \text{corr}(t4, fc4)$ $c4 = 0.728$

$fcA_i := 6625$ ANACAPU value for $T < 200$ degF & $S = 4.5$

Log Fit:

$fcC_i := fc(t4_i, 70)$ $\ln4 := \text{slope}(\ln4, fc4)$ $\ln4 = 155.326$

$\ln4 := \text{intercept}(\ln4, fc4)$ $\ln4 = 6.214 \cdot 10^3$

$fc\ln_i := 155.3 \cdot \ln(t4_i) + 6214$

$\ln4 := \text{corr}(\ln4, fc4)$ $\ln4 = 0.559$

$t4_i$	$fc4_i$	$fc\ln_i$
1	$6.45 \cdot 10^3$	$6.214 \cdot 10^3$
1	$6.32 \cdot 10^3$	$6.214 \cdot 10^3$
47	$6.53 \cdot 10^3$	$6.812 \cdot 10^3$
168	$6.79 \cdot 10^3$	$7.01 \cdot 10^3$
486	$6.64 \cdot 10^3$	$7.175 \cdot 10^3$
687	$5.95 \cdot 10^3$	$7.228 \cdot 10^3$
$1.011 \cdot 10^3$	$7.725 \cdot 10^3$	$7.288 \cdot 10^3$
$1.011 \cdot 10^3$	$8.12 \cdot 10^3$	$7.288 \cdot 10^3$
$1.388 \cdot 10^3$	$7.14 \cdot 10^3$	$7.338 \cdot 10^3$
$1.388 \cdot 10^3$	$8.24 \cdot 10^3$	$7.338 \cdot 10^3$

Elastic Modulus Data

$E4_i := \text{READ}(\text{mod4r})$

$a4 := \text{slope}(t4, E4)$ $a4 = 7.61 \cdot 10^{-4}$

$b4 := \text{intercept}(t4, E4)$ $b4 = 4.632$

$c4 := \text{corr}(t4, E4)$ $c4 = 0.792$

Using the above linear regression curve fitting parameters, the linear fit and modulus data are shown below. The linear fit is obviously inadequate for a 50-year time period. The ACI Code defines the Modulus of normal weight concrete as 57,000 times the square root of the strength. Using the ACI definition, plus a log curve fit, more reasonable 70 degF estimates of the modulus were obtained:

$$A := \frac{57000}{1000000} \quad (\text{ACI Code coefficient})$$

$$aEln4 := \text{slope}(tln4, E4) \quad aEln4 = 0.14$$

$$A = 0.057$$

$$bEln4 := \text{intercept}(tln4, E4) \quad bEln4 = 4.403$$

$$A := .057$$

$$Eln4_i := 0.33 \cdot \ln(t4_i) + 3.24$$

$$E70_i := A \cdot \sqrt{fc_{ln_i}}$$

$$cIn4 := \text{corr}(tln4, E4) \quad cIn4 = 0.74$$

$t4_i$	$E4_i$	$E70_i$	$Eln4_i$
1	4.32	4.493	3.24
1	4.58	4.493	3.24
47	5	4.704	4.511
168	5.02	4.772	4.931
486	5.03	4.828	5.281
687	4.46	4.846	5.396
$1.011 \cdot 10^3$	5.64	4.866	5.523
$1.011 \cdot 10^3$	5.71	4.866	5.523
$1.388 \cdot 10^3$	5.48	4.883	5.628
$1.388 \cdot 10^3$	5.79	4.883	5.628

Note that the square-root-of-the-strength approach does not fit the data as well as the "Log Fit". Therefore, the log fit was utilized in the calculations which follow. The 70F data fit is for a 4.5ksi nominal mix. To account for the small increase associated with a 5 ksi design mix (MWTF nominal strength), correction factors of $cfs = 245$ and $cfm = 0.062$ were applied to the 70F curve fits, as shown below. These correction factors are the differences in the predictions between $S=4.5$ and $S=5$ in the PNL correlations for a temperature of 200F.

Although the above 70 degF approximate correlations for strength and modulus could be improved, they appear to be adequate for obtaining at least preliminary estimates of the "residual" values. Thus, the following 70 degF correlations were used below for developing residual strength and moduli curves. Time zero for the above 70 degF correlations corresponds to an age of approximately six months. This time shift was utilized to be consistent with the high temperature data which used $t = 0$ corresponding to oven placement (min. age at test was 193 days). The 2nd expressions shown, $fc70d(t)$ and $E70d(t)$, include 20% adjustments to estimate the results if the 70F specimens were "tested dry". See report text for differences between "tested wet" and "tested dry".

$$cfs = 490.919 \cdot (5 - 4.5) \quad cfs = 245.459$$

$$cfm = .1233 \cdot (5 - 4.5) \quad cfm = 0.062$$

$$fc70(t) := (6214 + 155.3 \cdot \ln(t + 1) - cfs)$$

$$fc70d(t) := fc70(t) \cdot 1.2$$

$$fc70d(0) = 7.751 \cdot 10^3$$

$$fc70d(18250) = 9.58 \cdot 10^3$$

$$E70(t) := (3.24 + 0.33 \cdot \ln(t + 1) - cfm)$$

$$E70d(t) := E70(t) \cdot 0.8$$

$$E70d(0) = 2.641$$

$$E70d(18250) = 5.232$$

RESIDUAL CONCRETE STRENGTH PREDICTIONS

Residual strength versus temperature curves were generated for bake times of ten days, one year, and 50 years, as shown below. The "tested wet" 70F correlations were used as the reference strength to be consistent with the literature approach.

Ten Day Strength Degradation:

$$t := 10 \quad S := 5$$

$$fc(t, T) := (4416.338 + 490.919 \cdot S) - 230.241 \cdot \ln(t + 1) + 1.273 \cdot (350 - T) \cdot \ln(t + 1)$$

$$T := 200, 250 \dots 350$$

$$Rfc(t, T) := \frac{fc(t, T)}{fc_{70}(t)}$$

T	Rfc(t, T)
200	0.992
250	0.97
300	0.947
350	0.925

(Definition of residual strength)

One Year Strength Degradation:

$$t := 365 \quad S := 5 \quad T := 70$$

$$T := 200, 250 \dots 350$$

$$Rfc(t, T) := \frac{fc(t, T)}{fc_{70}(t)} \quad (\text{Definition of residual strength})$$

T	Rfc(t, T)
200	0.9
250	0.849
300	0.798
350	0.747

50 Year Strength Degradation

$$t := 18250 \quad S := 5$$

$$T := 200, 250 \dots 350$$

$$Rfc(t, T) := \frac{fc(t, T)}{fc70(t)} \quad (\text{Definition of residual strength})$$

T	Rfc(t, T)
200	0.812
250	0.734
300	0.656
350	0.578

MODULUS OF ELASTICITY EQUATIONS

Ten Day Modulus Degradation:

$$t = 10 \quad S = 5$$

$$E(t, T) = 5.3947 + 0.1233 \cdot S - 0.006751 \cdot T - 0.1786 \cdot \ln(t + 1)$$

$$RE(T) := \frac{E(t, T)}{E70(t)}$$

$$T := 200, 300 \dots 500$$

T	RE(T)
200	1.034
300	0.869
400	0.704
500	0.539

One Year Modulus Degradation:

$t := 365 \quad S := 5$

$E(t, T) := 5.3947 + 0.1233 \cdot S - 0.006751 \cdot T - 0.1786 \cdot \ln(t + 1)$

$RE(T) := \frac{E(t, T)}{E70(t)}$

$T := 200, 300 \dots 500$

T	RE(T)
200	0.687
300	0.558
400	0.43
500	0.301

Fifty Year Modulus Degradation:

$t := 18250 \quad S := 5$

$E(t, T) := 5.3947 + 0.1233 \cdot S - 0.006751 \cdot T - 0.1786 \cdot \ln(t + 1)$

$RE(t, T) := \frac{E(t, T)}{E70(t)}$

$T := 200, 300 \dots 500$

T	RE(t, T)
200	0.445
300	0.342
400	0.238
500	0.135

Note that the 500 F, 50 yr. prediction is very low. The 50-yr. extrapolation appears to be overly conservative.

200 Degree Degradation with Time:

The 200 deg. degradation is particularly significant to the MWTF tank program because 200 deg. is the concrete design temperature. The data below was generated for the 200 deg. design temperature.

Strength Degradation

$S = 5 \quad T = 200$

Note: Since 70F reference strength generally involves specimens "tested wet", the residual strength ratio uses the $fc70(t)$ term, rather than the $fc70d(t)$.

$$Rfc(t, T) = \frac{fc(t, T)}{fc70(t)} \quad Rfc(0, 200) = 1.064 \quad Rfc(18250, 200) = 0.812 \quad \frac{.588}{.864} = 0.681$$

$$RfcC(t, T) = \frac{fc(t, T)}{fc(t, 70)} \quad RfcC(0, 200) = 1 \quad RfcC(18250, 200) = 0.8$$

$t = 30, 60 \dots 18200$

200 DegF Modulus Degradation

$$RE(t, T) = \frac{E(t, T)}{E70(t)}$$

$t = 1, 100 \dots 18000$

200 DegF Comparison with ANACAPU

The ANACAPU approach assumes no degradation with time for temperatures equal or less than 200 degF. As show below, ANACAPU values at 200 degF, at long times, are significantly higher than the PNL correlation values, and are lower than the 70 degF correlations.

$$fcAN(t) = fc(0,200) \quad fc(0,200) = 6.871 \cdot 10^3 \quad EA200(t) := E(0,200)$$

$$fcLB(t,T) := fc(t,T) - 1292 \quad (1292 \text{ psi is the } 95/95 \text{ tolerance band for compressive strength})$$

$$ELB(t,T) := E(t,T) - 0.76 \quad (0.76 \text{ is the } 95/95 \text{ tolerance band for elastic modulus})$$

$$t := 0, 100 \dots 18000 \quad fc(9125,250) = 5.932 \cdot 10^3 \quad fc(18250,350) = 4.612 \cdot 10^3$$

Comparison with Hanford Concrete Core Strength Data:

There have been a number of existing structure core strength tests performed at Hanford (Winkel 1991, see report references). This concrete core testing information provides an indication of the properties of actual Hanford concrete structures, subjected to field curing and long term environment. For these core tests, the average core strength divided by the design strength was calculated to be 2.28. This result is compared to the 70F correlations below. Note that for a 5 ksi mix, the core test result strength ratios are actually higher than the correlation predictions. This is somewhat surprising, because field conditions would be expected to produce lower strength ratios. This apparent anomaly was apparently due to the fact that all of the core test results involved relatively low strength concrete (2.5 and 3.0 ksi mix designs). The CTL test results indicate that the higher strength mixes produce lower strength ratios than the lower strength mixes. Therefore, the 3 ksi mix comparison is more meaningful.

Another item of interest is the data scatter in both the CTL data and the Hanford structures core strength data. For the CTL data, PNL (Henager et al. 1988) reported a 95/95 tolerance band of 1292 psi for compressive strength. A comparable figure for the concrete core data can be obtained by from a 2-sigma, normal variation distribution. A 2-sigma strength ratio of 1.24 was reported in (Winkel 1991). For a 3 ksi mix, this corresponds to a strength variation of $1.24 \times 3000 = 3720$ psi, which is about three times the scatter of the CTL data.

$$R_{fcave}(t) = 2.28 \quad t := 0, 100 \dots 20000 \quad S := 3 \quad f_c(0, 70) = 6.871 \cdot 10^3$$

$$f_c(t, T) := (4416.338 + 490.919 \cdot S) - 230.241 \cdot \ln(t + 1) + 1.273 \cdot (350 - T) \cdot \ln(t + 1) \quad \text{PNL Comp. Strength Corr.}$$

$$\frac{f_c(18250, 70)}{3000} = 2.376$$

PNL Correlation Extended to Include 70F Data

Strength Correlation:

$$S = 5$$

$$fcl(t, T) := 4222.2 + 861.91 \cdot S + 1.199 \cdot (165.00 - T) \cdot \ln(t + 1) - 1.313 \cdot S \cdot T \quad (\text{from Appendix C})$$

$$fcl(0, 200) = 7.219 \cdot 10^3 \quad fcl(18250, 200) = 6.807 \cdot 10^3$$

$$t := 1, 10 \dots 1000$$

$$t := 0, 5000 \dots 18250$$

$fcl(t, 70)$	$fcl(t, 200)$
$8.072 \cdot 10^3$	$7.219 \cdot 10^3$
$9.042 \cdot 10^3$	$6.861 \cdot 10^3$
$9.121 \cdot 10^3$	$6.832 \cdot 10^3$
$9.167 \cdot 10^3$	$6.815 \cdot 10^3$

$$T := 50, 100 \dots 200$$

$$t := 0, 100 \dots 20000$$

Modulus Correlation:

$$E(t, T) := 3.329 + .126 \cdot S + 0.001 \cdot (124.82 - T) \cdot \ln(t + 1) \quad (\text{from Appendix C})$$

$$E(0, 200) = 3.959 \quad E(18250, 200) = 3.221$$

$$EC_i := E(t_4, 70) \quad EIC_i := \frac{E(t_4, 70)}{0.8}$$

Tensile Strength Correlation

PNL Splitting Tensile Strength Correlation:

$$S := 4.5$$

$$ft(t, T) := 448.1758 + 23.7436 \cdot S - 18.4341 \cdot \ln(t + 1)$$

$$ftA := ft(0, 200)$$

$$ftA = 555.022$$

$$i := 1..10$$

$$ftA_i := ftA$$

$$tt_i := \text{READ}(\text{timed})$$

$$ft4_i := \text{READ}(\text{strnt})$$

$$ft(0, 200) = 555.022$$

$$ftC_i := ft(tt_i, T)$$

$$ft(18250, 200) = 374.147$$

$$ttln_i := \ln(tt_i)$$

$$a4 := \text{slope}(tt, ft4) \quad a4 = 0.061$$

$$aln4 := \text{slope}(ttln, ft4)$$

$$aln4 = 25.506$$

$$b4 := \text{intercept}(tt, ft4)$$

$$bln4 := \text{intercept}(ttln, ft4)$$

$$bln4 = 423.67$$

$$ftln_i = 25.51 \cdot \ln(tt_i) + 423.7$$

$$T = 70$$

$$ftl(t, T, S) := 310.72 - .46675 \cdot T + 29.288 \cdot S + .2808 \cdot (184.34 - T) \cdot \ln(t - 1) \quad \text{Low Temp. correlation for dry concrete.}$$

See Appendix C.

$$ftCl_i := ftl(tt_i, T, 4.5)$$

$$ft70(t) := 423.7 + 25.51 \cdot \ln(t + 1)$$

$$\frac{ftl(0, 70, 5)}{ftl(0, 70, 4.5)} = 1.031$$

$$ft70d(t) := 1.03 \cdot 1.20 \cdot ft70(t)$$

- 70F Data
- PNL Correlation; 100..20000
- - - Log Fit
- ... Linear Fit
- . - ANACAPU
- - - Low Temp. Corr., .8 for wet test

tt_i	$ft4_i$
30	457
30	522
194	622
194	644
240	550
361	525
679	525
880	592
$1.204 \cdot 10^3$	591
$1.204 \cdot 10^3$	642

$$f(t, T) = 310.72 + .46675 \cdot T + 29.288 \cdot S + .2808 \cdot (184.34 - T) \cdot \ln(t + 1)$$

Low Temp. correlation for dry concrete.
See Appendix C.

$$f(0, 200) = 535.866 \quad f(18250, 200) = 492.72$$

T = 50, 100, 200

Review of ANACAP-U Creep Equations Relative to ACI 209R

$$\gamma_{la}(t_{la}) = 1.25 \cdot t_{la}^{-0.118} \quad \gamma_{la}(200) = 0.669 \quad \gamma_{la}(7) = 0.994$$

$$\varepsilon(t) := \frac{t^{0.6}}{10 + t^{0.60}} \frac{2.35}{E70(0)} \gamma_{la}(200) \quad \frac{1}{E70(0)} = 0.247$$

$$\varepsilon_{lb}(t) := \frac{t^{0.6}}{10 + t^{0.60}} \frac{1.30}{E70(0)} \gamma_{la}(200)$$

$$\varepsilon_{sub}(t) := \frac{t^{0.6}}{10 + t^{0.60}} \frac{4.15}{E70(0)} \gamma_{la}(200)$$

$$t = 1, 100 \dots 20000$$

200F Predictions

$$\varepsilon_{lbT}(t) := \frac{t^{0.6}}{10 + t^{0.60}} \frac{1.30}{E(0, 200)} \gamma_{la}(200) \cdot 4$$

$$\varepsilon_{subT}(t) := \frac{t^{0.6}}{10 + t^{0.60}} \frac{4.15}{E(0, 200)} \gamma_{la}(200) \cdot 6$$

Effect of Ambient Relative Humidity

$$\lambda = 40$$

$$hf(\lambda) := 1.27 - 0.0067 \cdot \lambda \quad hf(0) = 1.27$$

$$\varepsilon_0(t) := \frac{t^{0.6}}{10 + t^{0.60}} \cdot \frac{2.35}{E70(0)} \cdot \gamma_{la}(200) \cdot hf(0)$$

$$\varepsilon_{100}(t) := \frac{t^{0.6}}{10 + t^{0.60}} \cdot \frac{2.35}{E70(0)} \cdot \gamma_{la}(200) \cdot hf(100)$$

APPENDIX B

THERMAL EXPANSION COEFFICIENT TEST RESULTS

HWVP CONCRETE THERMAL EXPANSION TEST DATA

January 1994

SERVICE CONTRACT, U.S. BUREAU OF RECLAMATION, CONCRETE & STRUCTURAL BRANCH

STATEMENT OF WORK

Thermal expansion properties of "Hanford concrete" are part of the mechanical properties data base needed by the MWTF tank design team. The existing limited test data for thermal expansion of Hanford concrete conflicts with textbook/literature data for the aggregate types common to the Hanford area. Additional thermal expansion test data for concrete cores obtained from one of the Hanford concrete structures (T Plant), should help resolve this conflict.

A standard thermal expansion test for concrete cores (copy attached) has been developed by the Bureau of Reclamation and is routinely used to obtain concrete thermal expansion coefficient data. This standard test utilizes six, four-inch-long specimens. Westinghouse Hanford Co. will supply the required six specimens, plus two spares, in accordance with the Bureau specifications. The supplied specimens will be shipped in sealed containers to maintain the in situ moisture content. Following removal from the sealed containers, the specimens will be weighed to permit the later determination of the in situ saturation percentage. Expansion coefficient data will be developed for both the fully saturated and fully dry condition for a standard set of six specimens. Test results will be documented in accordance with the Bureau standard format and transmitted to Westinghouse Hanford Co. The documentation should include evidence that the "calibration and standardization" of the test equipment was performed, as required by Section 6. of the test procedure.

Note: The above statement of work was attached to the purchase requisition sent to the U.S. Bureau of Reclamation, which performed the thermal expansion testing. The attached test procedure had previously been reviewed by WHC and judged to be an acceptable procedure for the project needs.

UNITED STATES DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

USBR 4910-89

PROCEDURE FOR
COEFFICIENT OF LINEAR THERMAL EXPANSION
OF CONCRETE

INTRODUCTION

✓ This test procedure is under the jurisdiction of the Concrete and Structural Branch, code D-3730, Research and Laboratory Services Division, Denver Office, Denver, Colorado. The procedure is issued under the fixed designation USBR 4910; the number immediately following the designation indicates year of original adoption or year of last revision.

1. Scope

1.1 Most unrestrained engineering materials expand when heated and contract when cooled. The strain due to a 1°-temperature change is known as the coefficient of thermal expansion. This coefficient is approximately constant for a considerable range of temperatures, and generally increases with an increase in temperature. For a homogeneous, isotropic material, the coefficient applies to all dimensions in all directions. This test designation covers a procedure for determining the thermal coefficient of expansion for hardened concrete in a saturated, intermediate, or dry-moisture condition.

✓ 2. Applicable Documents

2.1 *USBR Procedures:*

- ✓ 4042 Obtaining and Testing Drilled Cores and Sawed Beams or Cubes of Concrete and Shotcrete
- ✓ 4192 Making and Curing Concrete Test Specimens in Laboratory

3. Apparatus

3.1 *Holding Tank.*—An insulated, copper-lined tank to hold circulating water.

3.2 *Water Tank.*—A water tank containing electric immersion heaters and refrigeration coils capable of maintaining circulating water at constant temperatures between 35 and 90 °F (1.7 and 32.2 °C).

3.3 *Recorder.*—A thermocouple recorder to record water temperature versus time.

3.4 *Refrigeration Unit.*—A freon refrigeration unit for lowering water temperature.

3.5 *Controller.*—A controller for regulating water temperature. Controller should automatically turn heater and refrigeration units on and off.

3.6 *Steel Frames.*—Invar steel, level frames for holding concrete test specimens as they are lowered into water (six required).

3.7 *Thermometer.*—A thermometer with a range of 35 to 90 °F (1.7 to 32.2 °C), and with an accuracy of 0.1 °F (0.16 °C).

3.8 *Transformers.*—For measuring length change during testing, six LVDT's (linear variable differential transformers) are required.

3.9 *Transducer Indicators.*—Two amplifier transducer indicators are required. These indicators are high-sensitivity, differential transformer, input modules for reading summation of length change occurring in three of the test specimens. The normal length change range selection for concrete is set so that a full scale division is 0.01 inch (0.25 mm); the scale is divided into 100 divisions.

3.10 Other types of sensing, controlling, and recording equipment and instrumentation can also provide satisfactory results. This system is being described somewhat in detail for the benefit of Bureau personnel who will be conducting the test with this equipment.

4. Precautions

4.1 This test procedure may involve hazardous materials, operations, and equipment, and does not claim to address all safety problems associated with its use. It is the responsibility of the user to consult and establish appropriate safety and health practices and determine applicability of regulatory limitations prior to use.

5. Sampling and Test Specimens

5.1 Six concrete test specimens are sawed as 2- by 2- by 4-inch (50- by 50- by 100-mm) prisms obtained in accordance with USBR 4042, or cast as 2- by 4-inch (50- by 100-mm) cylinders made in accordance with USBR 4192. ✓ Invar buttons are then epoxied onto the ends of the specimens. The buttons are recessed to accommodate the tips of the holding frame during testing. Length measurements are made to nearest 0.01 inch (0.25 mm), and testing normally occurs in a 100-percent vacuum saturated condition, as close to zero load as possible.

5.2 In our previous test procedures, mass concrete has been tested at three different moisture conditions to determine difference in thermal coefficients at each state. These three conditions are 100 percent dry, 100 percent vacuum saturated, and 75 percent vacuum saturated. If testing by these conditions is requested, 12 specimens are prepared in 2 groups of 6 each. All specimens are oven

dried at 190 °F (87.8 °C) to initiate testing under equal conditions.

5.2.1 During initial drying of first set of specimens, mass is determined until no loss of the mass is observed. Specimens are immediately dipped in heated paraffin wax to sustain this 100 percent dry condition.

5.2.2 After initial drying, second set of specimens are vacuum saturated and their mass determined until no increase in mass is observed. Specimens are immediately submerged in a water bath of constant temperature to sustain this 100 percent saturated condition.

5.2.3 The second set of specimens, which were previously tested in a fully saturated state, are also used for obtaining data in a partially saturated condition. The fully saturated specimens are dried and their mass determined until they reach a condition of 75 percent saturation. To remain in this state, specimens are immediately dipped in heated paraffin wax. The highest values for coefficient of thermal expansion are normally obtained in this 75 percent saturated condition.

6. Calibration and Standardization

6.1 The calibration and standardization of miscellaneous equipment or apparatus used in performing the tests listed under the Applicable Documents of section 2 are covered under that particular procedure.

6.2 The LVDT's are calibrated on the thermal expansion testing apparatus as follows:

Step 1.—Place test specimens in tank.

Step 2.—Set ABC selector switch to A of TAI (Transducer Amplifier-Indicator).

Step 3.—Set range selector to STANDBY. Adjust meter screw for zero reading.

Step 4.—Turn sensitivity to maximum (clockwise) and set zero knobs to center point (arrows point upward).

Step 5.—Repeat steps 1 through 4 for second TAI.

Step 6.—Turn range selector to NULL.

Step 7.—Unlock adjustment knob on frame and set LVDT for zero meter reading, lock LVDT, and adjust with "mechanical adjustment knob" to bring needle as close to zero as possible. Lock manual adjustment knob.

Step 8.—Fine adjust to zero with proper null screw.

Step 9.—If necessary, alternate mechanical and null adjustment to bring needle as close to zero as possible.

Step 10.—Set ABC selector switch to B, and repeat steps 7 through 9.

Step 11.—Set ABC selector switch to C, and repeat steps 7 through 9.

Step 12.—Turn second TAI range selector to NULL.

Step 13.—Repeat steps 7 through 11 for second amplitude.

Step 14.—Set range selector to 10.

Step 15.—Adjust zero for A with fine-zero controls. Repeat for B and C.

Step 16.—Check zero of A+B+C.

Step 17.—Repeat steps 14 through 16 with second TAI.

Step 18.—Set ABC selector switch to A.

Step 19.—Insert 0.01-inch (0.25-mm) shim between first LVDT and mechanical adjuster with a sawing motion.

Step 20.—Adjust sensitivity to bring needle to 100.

Step 21.—Set range selector to CAL and bring needle to 100 with CAL SET adjustment.

Step 22.—Bring range selector back to 10.

Step 23.—Set ABC selector switch to B. Insert shim.

Step 24.—Adjust needle to 100 with span B adjustment screw.

Step 25.—Set ABC selector switch to C. Insert shim.

Step 26.—Repeat step 24 with span C screw.

Step 27.—Check zero of A+B+C.

Step 28.—Repeat steps 18 through 27 with second amplitude.

Step 29.—With range selector at 10 and ABC selector switch at A+B+C, begin test by moving rheostat to about 55 and turning on all other switches. Turn recorder to "select-o-print."

Step 30.—Temperature is adjusted with knob on temperature controller.

6.3 Temperature control shall be verified as conforming with section 3 by adequate thermometers certified by manufacturer.

7. Conditioning

7.1 Tests should be conducted in a room environment where temperature change is held to a minimum. Other conditioning is covered under section 5.

8. Procedure

8.1 Test specimens in a fully dry, fully saturated, or partially saturated state are placed in their holding frames and lowered into a water bath of constant temperature 1 day prior to testing. This allows all components to reach a temperature equilibrium.

8.2 Fill in heading of Data and Computation Sheet (fig. 1) and begin test.

8.3 The initial time, water temperature, and amplified meter readings are taken and temperature controller is set to 70 or 80 °F (21.1 or 26.7 °C) depending upon initial room temperature.

8.4 When water reaches set temperature, hold temperature constant for 5 to 7 minutes while entries are made on data sheet of length changes, temperatures, and time.

8.5 Increase temperature to 90 °F (32.2 °C) and repeat data entries.

8.6 Lower temperature to 85 °F (29.4 °C) and read in 10 °F (5.6 °C) increments down to 35 °F (1.7 °C). Increase temperature to 40 °F (4.4 °C) and read in 10 °F increments back to the initial set temperature. In this manner, data points are obtained for every 5 °F (2.8 °C) interval between 35 and 90 °F as temperature is fluctuated.

8.7 Meter versus temperature readings are plotted and a linear curve is drawn (fig. 2).

9. Calculations

9.1 Figure 1 shows a typical data and calculation form.

9.2 Figure 2 shows the curve discussed in section 8.7. The length or projected meter readings are determined

from the curve at the intersect point of 35 °F (1.7 °C) and 90 °F (32.2 °C).

9.2.1 Calculate the coefficient of thermal expansion as follows:

$$CE = \frac{y_2 - y_1}{0.55 S} \quad (1)$$

where:

CE = coefficient of thermal expansion, in (in/in)/°F × 10⁻⁶ or (cm/cm)/°C × 10⁻⁶;

y₁ = intercept of line at 35 °F (1.7 °C);

y₂ = intercept of line at 90 °F (32.2 °C); and

S = summation of gauge lengths, in inches (millimeters). *gauge*

9.2.2 A more precise method of calculating the coefficient of thermal expansion is by the least squares method of calculation for determining the slope of a line (fig. 3):

$$CE = \frac{y_2 - y_1}{0.55 S}$$

$$y_1 = -c + 35b$$

$$y_2 = -c + 90b$$

$$c = a - \bar{y} \quad (4)$$

$$b = \frac{\sum (x - \bar{x})(y - \bar{y})}{\sum (x - \bar{x})^2} \quad (5)$$

$$\bar{x} = \frac{\sum x}{n} \quad (6)$$

where:

b = slope of line,

x = temperature in °F (°C),

$\bar{x}$ = average temperature in °F (°C),

y = measurement in inches per inch

(centimeters per centimeter),

$\bar{y}$ = average measurement in inches per

inch (centimeters per centimeter), and

CE, S, y₁, and y₂ = as previously defined.

10. Report

10.1 A cover letter along with figures 1 and 2 or figure 3 should serve as a report for this procedure.

11. Precision and Bias

11.1 The precision and bias for this procedure have not been established.

a = a constant derived from the average temperature multiplied by the slope of the line
c = a constant derived from "a" in the average meter reading

IN REPLY
REFER TO:
D-3731
RES-3.40

United States Department of the Interior

BUREAU OF RECLAMATION

DENVER OFFICE

P O BOX 25007
BUILDING 67, DENVER FEDERAL CENTER
DENVER, COLORADO 80225-0007

January 12, 1994

DRAFT

MEMORANDUM

To: G. W. DePuy
Head, Concrete Technology Team, D-3731

From: Dennis Arney
D-3731

Subject: Report of Test Results: Westinghouse Hanford Company
Their Purchase Order Reference: M386853, USBR WOID ER352

Materials Engineering Branch Referral Memorandum No. 3731-94-2

Tests were performed on concrete core samples supplied by Westinghouse Hanford. These cores were 1.5 inches in diameter and approximately 4 inches long. They were received sealed in plastic bags.

The average moisture content of the core samples received was 6.58 percent. This percent moisture content is based on the oven dry weight.

Thermal coefficient of expansion tests were performed on two sets of core. The first set was at a fully saturated moisture condition and the second set was at a fully oven dry condition sealed in wax.

Testing was performed in accordance with Bureau of Reclamation Designation 4910.92. Each LVDT was calibrated prior to testing as required by Section 6 of the test procedure. The temperature range was from 36° F to 90° F.

The test results were as follows:

Oven Dry	3.66 millionths in/in/° F
Saturated	3.67 millionths in/in/° F.

cc: D-3730
D-3731 (Arney, files)

WHC-SD-W236A-ER-013
Revision 0

January 11, 1994

WESTINGHOUSE HANFORD

Concrete Core Identification

Labels on plastic bags as received Four cores in each bag	Assigned core numbers
8-5-93 MC 6P 93-306-5	1, 2, 3, 4
8-6-93 MC 6P 93-315-5	17, 18, 19, 20
8-25-93 MC 6P 93-310-5	13, 14, 15, 16
8-26-93 MC 6P 93-314-5	5, 6, 7 8
8-26-93 MC 6P 93-318-5	21, 22, 23, 24
9-25-93 MC 6P 93-308-5	9, 10, 11, 12
Oven dry cores: 1, 2, 5, 7, 8, 10	
Saturated cores: 14, 16, 17, 20, 22, 24	

COEFFICIENT OF THERMAL EXPANSION
WESTINGHOUSE HANFORD - OVEN DRY SEALED - 1/7/94

$Y = 63.9 + .86X / \text{COEFFICIENT OF EXPANSION} = 3.66 \times 10^{-6} \text{ IN/IN/F}$

REV

JAN-14-94 FRI 9:06

METER READING (0.01 X 100 INCHES)

WHC-SD-W236A-ER-013
Revision 0COEFFICIENT OF THERMAL EXPANSION
WESTINGHOUSE HANFORD - OVEN DRY SEALED - 1/7/94

PAGE 1

POINT	TEMPERATURE (DEGREES F)	METER READING (0.01 X 100 INCHES)
1	37.60000	-36.20
2	37.60000	-26.80
3	40.00000	-31.60
4	40.00000	-27.60
5	45.20000	-27.40
6	45.20000	-24.20
7	50.00000	-21.90
8	50.00000	-20.10
9	54.80000	-18.20
10	54.80000	-15.00
11	60.20000	-13.00
12	60.20000	-11.00
13	65.20000	-7.70
14	65.20000	-6.70
15	70.00000	-3.20
16	70.00000	-2.60
17	75.20000	0.00
18	75.20000	0.00
19	80.20000	3.70
20	80.20000	3.90
21	85.20000	8.90
22	85.20000	9.30
23	89.80000	13.80
24	89.80000	13.90

JAN-14-94 FRI 9:06

WHC-SD-W236A-ER-013
Revision 0

P.06

WAF

COEFFICIENT OF THERMAL EXPANSION
WESTINGHOUSE HANFORD - SATURATED - 1/5/94

$\gamma = -51.6 + .66X$ / COEFFICIENT OF EXPANSION -3.67×10^{-6} IN/IN/F

B-11

METER READING (2.01 X 102 INCHES)

COEFFICIENT OF THERMAL EXPANSION
WESTINGHOUSE HANFORD - SATURATED - 1/5/94

PAGE 1

POINT

TEMPERATURE (DEGREES F) METER READING (0.01 X 100 INCHES)

1	36.60000	-17.30
2	36.60000	-16.00
3	40.00000	-17.60
4	40.00000	-16.60
5	45.00000	-14.20
6	45.00000	-11.80
7	50.30000	-9.70
8	50.30000	-9.10
9	55.20000	-4.30
10	55.20000	-4.10
11	60.20000	-3.00
12	60.20000	-1.20
13	63.40000	0.00
14	63.40000	0.00
15	70.20000	9.00
16	70.20000	9.40
17	75.50000	11.90
18	75.50000	14.10
19	79.70000	17.00
20	79.70000	19.60
21	85.20000	21.00
22	85.20000	23.00
23	90.00000	25.20
24	90.00000	29.70

January 11, 1994

JAN-14-94 FRI 9:07

WESTINGHOUSE HANFORD

Amplifier - Meter Readings

Thermal Coefficient of Expansion - Oven Dry

RECEIVED

Core No.	Amplifier 1			Summary 1, 2, 3	Amplifier 2			Summary 4, 5, 6	Average
	100.95 1 306-5	103.40 2 306-5	99.25 5 314-5		99.30 7 314-5	96.00 8 314-5	98.55 10 308-5		
Temp °F	3.077	4.032	3.907		3.909	3.780	3.880		
75.2	0	0	0	0	0	0	0	0	0
80.2	+1.2	+1.5	+1.0	+3.7	+1.2	+1.3	+1.4	+3.9	+3.8
85.2	+2.8	+3.5	+3.1	+9.3	+3.0	+3.0	+3.1	+8.9	+9.1
89.8	+4.6	+4.8	+4.5	+13.8	+4.5	+4.7	+4.6	+13.9	+13.8
65.2	-2.2	-2.4	-2.1	-6.7	-2.6	-2.8	-2.7	-7.7	-7.2
54.8	-5.0	-5.3	-4.9	-15.0	-5.7	-6.4	-6.1	-18.2	-16.6
40.0	-9.4	-9.1	-9.1	-27.6	-10.5	-10.5	-10.6	-31.6	-29.6
37.6	-8.8	-8.9	-9.1	-26.8	-12.0	-12.1	-12.2	-36.2	-31.5
45.2	-8.0	-8.1	-8.3	-24.2	-8.9	-9.4	-9.1	-27.4	-25.3
50.0	-6.5	-6.9	-6.7	-20.1	-7.2	-7.4	-7.3	-21.9	-21.0
60.2	-3.9	-3.7	-3.7	-11.0	-4.4	-4.3	-4.3	-13.0	-12.0
70.0	-0.8	-1.0	-0.9	-2.8	-1.0	-1.1	-1.2	-3.2	-3.0

B-13

MHC-SD-M236A-ER-013
Revision 0

P. 08

January 11, 19

WESTINGHOUSE HANFORD

Amplifier - Meter Readings

Thermal Coefficient of Expansion - Saturated

RECEIVED

Core No.	Amplifier 1			Summary 1, 2, 3	Amplifier 2			Summary 4, 5, 6	Average
	97.45 14 310-5	101.25 16 310-5	100.40 17 315-5		97.25 20 315-5	97.50 22 318-5	100.00 24 318-5		
(Gage Len (in)) Temp °F	3.937	3.986	3.953		3.907	3.839	3.937		
63.4	0	0	0	0	0	0	0	0	0
70.2	+3.2	+3.1	+3.0	+9.4	+3.0	+3.1	+2.9	+9.0	+9.2
85.2	+7.8	+7.7	+7.9	+23.0	+7.0	+7.0	+7.2	+21.0	+22.0
79.7	+6.6	+6.5	+6.5	+19.6	+5.7	+5.8	+5.8	+17.0	+18.3
75.5	+4.0	+3.9	+4.0	+11.9	+4.6	+4.7	+4.9	+14.1	+13.0
90.0	+8.4	+8.5	+8.4	+25.2	+9.8	+10.0	+9.9	+29.7	+27.4
55.2	-1.4	-1.4	-1.6	-4.3	-1.4	-1.5	-1.3	-4.1	-4.2
50.3	-3.3	-3.2	-3.3	-9.7	-3.0	-3.0	-3.1	-9.1	-9.4
60.2	-1.0	-1.1	-1.0	-3.0	-0.4	-0.5	-0.3	-1.2	-2.1
45.0	-3.9	-3.8	-3.9	-11.8	-4.7	-4.8	-4.6	-14.2	-13.0
36.6	-5.3	-5.4	-5.4	-16.0	-5.8	-5.9	-5.8	-17.3	-16.6
40.0	-5.5	-5.6	-5.5	-16.6	-5.9	-6.0	-5.9	-17.6	-17.1

B-14

WHC-SD-W236A-ER-013
Revision 0

P. 83

INDIVIDUAL EXPANSION COEFFICIENT CALCULATIONS

(Performed by WHC using Bur. of Reclam. data)

OVEN-DRIED DATA

ORIGIN := 1

i := 1..12

TD_i := READ(TEMPD)

D1D_i := READ(D1D)

a1 := slope(TD, D1D) a1 = 0.263

b1 := intercept(TD, D1D) b1 = -19.523

$\alpha_1 := \frac{a1}{2.39740} \quad \alpha_1 = 3.311 \cdot 10^{-6}$

D2D_i := READ(D2D) D3D_i := READ(D3D) D4D_i := READ(D4D) D5D_i := READ(D5D) D6D_i := READ(D6D)

a2 := slope(TD, D2D) a3 := slope(TD, D3D) a4 := slope(TD, D4D) a5 := slope(TD, D5D) a6 := slope(TD, D6D)

$\alpha_2 := \frac{a2}{2.40320}$

$\alpha_3 := \frac{a3}{2.39070}$

$\alpha_4 := \frac{a4}{2.39090}$

$\alpha_5 := \frac{a5}{2.37800}$

$\alpha_6 := \frac{a6}{2.38800}$

j := 1..6

α_j
$3.311 \cdot 10^{-6}$
$3.367 \cdot 10^{-6}$
$3.395 \cdot 10^{-6}$
$3.874 \cdot 10^{-6}$
$4.103 \cdot 10^{-6}$
$3.984 \cdot 10^{-6}$

$\alpha_{ave} := \text{mean}(\alpha) \quad \alpha_{ave} = 3.672 \cdot 10^{-6}$

$\sigma := \text{stdev}(\alpha) \quad \sigma = 3.223 \cdot 10^{-7}$

$\alpha_{min} := \alpha_{ave} - 2 \cdot \sigma \quad \alpha_{min} = 3.028 \cdot 10^{-6}$

$\alpha_{max} := \alpha_{ave} + 2 \cdot \sigma \quad \alpha_{max} = 4.317 \cdot 10^{-6}$

SATURATED DATA

i := 1..12

TS_i := READ(TEMPS)

D1S_i := READ(D1S)

a1 := slope(TS,D1S) a1 = 0.281

b1 := intercept(TS,D1S) b1 = -16.782

$$\alpha s_1 := \frac{a1}{2.38370} \quad \alpha s_1 = 3.659 \cdot 10^{-6}$$

D2S_i := READ(D2S) D3S_i := READ(D3S) D4S_i := READ(D4S) D5S_i := READ(D5S) D6S_i := READ(D6S)

a2 := slope(TS,D2S) a3 := slope(TS,D3S) a4 := slope(TS,D4S) a5 := slope(TS,D5S) a6 := slope(TS,D6S)

$$\alpha s_2 := \frac{a2}{2.39860}$$

$$\alpha s_3 := \frac{a3}{2.39530}$$

$$\alpha s_4 := \frac{a4}{2.39070}$$

$$\alpha s_5 := \frac{a5}{2.38390}$$

$$\alpha s_6 := \frac{a6}{2.39370}$$

αs_j
$3.659 \cdot 10^{-6}$
$3.518 \cdot 10^{-6}$
$3.563 \cdot 10^{-6}$
$3.748 \cdot 10^{-6}$
$3.878 \cdot 10^{-6}$
$3.758 \cdot 10^{-6}$

$$\alpha s_{ave} := \text{mean}(\alpha s) \quad \alpha s_{ave} = 3.688 \cdot 10^{-6}$$

j := 1..6

$$\alpha t_j := \alpha_j \quad \alpha t_{j+6} := \alpha s_j$$

$$\alpha t_{ave} := \text{mean}(\alpha t) \quad \alpha t_{ave} = 3.68 \cdot 10^{-6}$$

$$\sigma := \text{stdev}(\alpha t) \quad \sigma = 2.439 \cdot 10^{-7}$$

$$\alpha t_{min} := \alpha t_{ave} - 2 \cdot \sigma \quad \alpha t_{min} = 3.192 \cdot 10^{-6}$$

$$\alpha t_{max} := \alpha t_{ave} + 2 \cdot \sigma \quad \alpha t_{max} = 4.168 \cdot 10^{-6}$$

$N := 15$

$j := 1..N$

$k := 1..N - 1$

$\text{intervals}_j := \left(3 + \frac{j}{10}\right) \cdot 10^{-6}$

$f := \text{hist}(\text{intervals}, \alpha)$

WHC-SD-W236A-ER-013
Revision 0

APPENDIX C

PNL CORRELATIONS REVISED TO INCORPORATE 70°F DATA .

PREPARED BY: Steve Wilmarth
Steve Wilmarth, Statistician
Process Laboratories and Technology

STATISTICAL ANALYSIS OF CONCRETE DATA

Regression equations relating compressive strength, splitting tensile strength, and modulus to time at temperature were computed using data from concrete samples held at 70° F for 3K and 4.5K mix type concrete. The general form of these regression equations is given in equation (1).

$$(1) \text{ Property} = a + b * \log(t+1)$$

where

t	is the time in days at constant temperature
log	is the natural logarithm
Property	is compressive strength, splitting tensile strength or modulus.

Regression equations of this form had previously been computed for concrete samples heated to 250, 350, and 450° F (Henager et al 1988). See body of report for references, Section 7.0.

The data for concrete samples held at 70° F was obtained from the same experiment as the 250, 350 and 450° F samples. The heated samples, however, were aged at room temperature for at least 194 days before being put in the oven. Thus the 70° F results were adjusted by subtracting 194 days from the time at temperature, t, given in the experiment. This adjustment was made so that the "initial strength" (strength before being put in the oven or fog room) of the heated and 70° F concrete samples was the same.

After being aged, heated samples were put in an oven, while the 70° F samples were kept in a high humidity fog room until being tested. Hence the 70° F samples were "tested wet" while the 250, 350 and 450° samples were "tested dry". Because of this difference, the modulus, compressive strength, and splitting tensile strength properties of the 70° F samples were adjusted to make the fog room measurements comparable to those that were kept in an oven. This adjustment was done by making the compressive strength and splitting tensile strength properties 20% higher, while the modulus property was made 20% lower. See Section 2.3 of the body of this report.

Table 1 contains estimates of the intercept, a, and slope, b, for each of the temperature and mix type combinations. The standard errors of these estimates is given in parenthesis. Estimates are given for both the original and the adjusted data.

Multiple regression equations relating compressive strength, splitting tensile strength, and modulus to temperature, mix type, and time at temperature are given in Table 2. The equations were computed using every temperature in the experiment. However, the equations given in Table 2 are only valid for temperatures less than or equal to 250° F. Two regression equations are given for each property. The first set of equations are based on data that was not adjusted for the units that were "tested wet". These equations contain a

variable, W, that accounts for the effect of being "tested wet". The second set of equations are based on data that has already been adjusted for this effect.

Note that the estimated coefficients of the $\log(t+1)$ terms in Table 2, depend on temperature. For example, the first compressive strength equation contains the term:

$$1.226*(160.19 - \text{Temp})*\log(t+1)$$

This term changes from positive to negative as the temperature rises above 160.19° F. Thus the predicted compressive strength decreases with time when the temperature is above this level.

Previous predictive equations have ignored potential decreases in modulus and strength for concrete below 200° F. The equations in Table 2 suggest that over long periods of time, significant decreases in strength or modulus may occur at temperatures below 200° F.

Attachment 1 contains graphs of the the best fit lines for the 3K and 4.5K mix types at each of the four temperatures. Attachment 2 contains plots of the unadjusted 3K and 4.5K data with the predicted lines from the multiple regression equations. Note that for the compressive strength and modulus properties, the spacing between the 3K and 4.5K regression lines changes for different temperatures. These multiple regression equations contain a $S*\text{Temp}$ "interaction" term. This interaction term indicates that the difference between the initial strength of the 3K and 4.5K samples depends on temperature.

Table 1. Least Square Estimates of Equation (1). Page 2 of 2.

<u>MODULUS</u>			
<u>Mix</u>	<u>Temp.</u>	<u>Intercept</u>	<u>Slope</u>
3	70	4.6565 (.1408)	.0387 (.024)
3	250	4.0338 (.1016)	-.1854 (.021)
3	350	3.2623 (.1323)	-.1476 (.026)
3	450	2.7073 (.0918)	-.1622 (.019)
4.5	70	4.8555 (.2115)	.0979 (.036)
4.5	250	4.2464 (.1922)	-.1617 (.037)
4.5	350	3.6143 (.1368)	-.1795 (.026)
4.5	450	3.1196 (.0732)	-.2396 (.015)

<u>ADJUSTED MODULUS</u>			
<u>Mix</u>	<u>Temp.</u>	<u>Intercept</u>	<u>Slope</u>
3	70	3.7252 (.1126)	.0310 (.019)
3	250	4.0338 (.1016)	-.1854 (.021)
3	350	3.2623 (.1323)	-.1476 (.026)
3	450	2.7073 (.0918)	-.1622 (.019)
4.5	70	3.8844 (.1692)	.0783 (.029)
4.5	250	4.2464 (.1922)	-.1617 (.037)
4.5	350	3.6143 (.1368)	-.1795 (.026)
4.5	450	3.1196 (.0732)	-.2396 (.015)

* The number in () is the standard error of the estimate.

TABLE 2. Multiple Regression Equations. (Page 1 of 2).**COMPRESSIVE STRENGTH**

$$(1) \hat{f}_c = 4440.067 + 960.49*S + 1.226*(160.19 - \text{Temp})*\log(t+1) - 1.878*S*\text{Temp}$$

$$(2) \hat{f}_{c2} = 4440.067 + 490.99*S - 4.84*TA + 1.226*TB*\log(t+1) - 232.65*\log(t+1)$$

$$(3) \hat{f}_{cp} = 4416.338 + 490.92*S - 4.71*TA + 1.273*TB*\log(t+1) - 230.24*\log(t+1)$$

SPLITTING TENSILE STRENGTH

$$(1) \hat{f}_{st} = 409.306 + 27.79*S + 0.251*(176.55 - \text{Temp})*\log(t+1)$$

$$(2) \hat{f}_{st2} = 433.01 + 27.29*S - 0.608*TA - 18.44*\log(t+1)$$

$$(3) \hat{f}_{stp} = 448.18 + 23.74*S - 0.608*TA - 18.43*\log(t+1)$$

where

$\hat{f}_c$ = predicted value of compressive strength (psi)
for Temp $\leq$ 250 °F

$\hat{f}_{c2}$ = predicted value of the compressive strength (psi)
for Temp $\geq$ 250 °F

$\hat{f}_{cp}$ = PNL compressive strength equation (psi)

$\hat{f}_{st}$ = predicted value of splitting tensile strength (psi)
for Temp $\leq$ 250 °F

$\hat{f}_{st2}$ = predicted value of splitting tensile strength (psi)
for Temp $\geq$ 250 °F

$\hat{f}_{stp}$ = PNL splitting tensile strength equation (psi)

S = nominal initial compressive strength (10^3 psi)

Temp = constant value of temperature (°F)

TA = max[0, 350-Temp] °F

TB = max[0, Temp-350] °F

t = time at constant temperature

WHC-SD-W236A-ER-013
Revision 0TABLE 2. (Page 2 of 2).

MODULUS

$$(1) \hat{E} = 3.873 + 0.426*S - 0.00307*Temp - 0.000921*S*Temp \\ + 0.00108*(134.38 - Temp)*\log(t+1)$$

$$(2) \hat{E}_2 = 3.873 + 0.426*S - 0.0031*Temp - 0.125*\log(t+1) - 0.00092*S*Temp$$

$$(3) \hat{E}_p = 5.39 + 0.123*S - 0.0068*Temp - 0.179*\log(t+1)$$

where,

 $\hat{E}$ = predicted value of modulus (10^6 psi)
for Temp $\leq$ 250 °F $\hat{E}_2$ = predicted value of modulus (10^6 psi)
for Temp $\geq$ 250 °F $\hat{E}_p$ = PNL modulus equation (10^6 psi)S = nominal initial compressive strength (10^3 psi)

Temp = constant value of temperature (°F)

TA = max[0, 350-Temp] °F

TB = max[0, Temp-350] °F

t = time at constant temperature

ATTACHMENT 1

GRAPHS OF
COMPRESSIVE STRENGTH
SPLITTING TENSILE STRENGTH
AND MODULUS

PAGE 1 OF 4

TEMP = 250 F

TEMP = 450 F

TEMP = 70 F

TEMP = 350 F

TEMP = 250 F

TEMP = 450 F

TEMP = 70 F

TEMP = 350 F

TEMP = 250 F

TEMP = 450 F

TEMP = 70 F

TEMP = 350 F

ATTACHMENT 2

PLOTS OF MULTIPLE REGRESSION EQUATIONS
FOR COMPRESSIVE STRENGTH
SPLITTING TENSILE STRENGTH
AND MODULUS

PAGE 1 OF 4

TEMP = 70 F

TEMP = 350 F

TEMP = 250 F

TEMP = 450 F

TEMP = 70 F

SPLITTING TENSILE STRENGTH

log(DAYS + 1)

TEMP = 350 F

SPLITTING TENSILE STRENGTH

200 300 400 500 600 700

log(DAYS + 1)

TEMP = 250 F

SPLITTING TENSILE STRENGTH

200 300 400 500 600 700

log(DAYS + 1)

TEMP = 450 F

SPLITTING TENSILE STRENGTH

200 300 400 500 600 700

log(DAYS + 1)

TEMP = 70 F

TEMP = 250 F

TEMP = 350 F

TEMP = 450 F

APPENDIX D

ORNL REVIEW COMMENTS AND COMMENT RESOLUTIONS

5. Document Number(s)/Title(s) /Preliminary MWTf Tank Concrete Characterization	6. Program/Project/ Building Number	7. Reviewer Dan J. Naus	8. Organization/Group	9. Location/Phone
--	--	--------------------------------	-----------------------	-------------------

17. Comment Submittal Approval:

T. J. Conrads

Organization Manager (Optional)

10. Agreement with indicated comment disposition(s)

11. CLOSED

10-4-94
Date

B. V. Winkel/

Reviewer/Point of Contact

B. V. Winkel
Author/Originator9/29/94
Date

Dan J. Naus

Reviewer/Point of Contact

T. J. Conrads
Author/Originator

12. Item	13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)	14. Hold Point	15. Disposition (Provide justification if NOT accepted.)	16. Status
D-2 1	Table 2.1, Under Actual F'_c : The strength gains estimated in going from 0 (28 days or 193 days) to 50 years may be somewhat optimistic if data from concretes such as University of Wisconsin study were utilized to develop the estimate. Cement chemistry and fineness have changed somewhat in the last 50 years to produce concretes that exhibit more rapid initial strength gains (i.e., long-term strength gains will not be as great as exhibited by University of Wisconsin study). As noted later in the appendix, a strength gain of up to 2.28 relative to the design strength was obtained from cores. Did this factor enter into predicted strength gain?		A basic assumption of the report is that the mechanical property trends exhibited by the Hanford concrete, tested by CTL 15 to 20 years ago (3000 psi and 4500 psi mixes), is a reasonable predictor of the behavior of the 5000 psi new tank concrete. Your question relative to the effect of concrete chemistry and fineness changes occurring over the past 50 years is reasonable. However, since precise chemistry and fineness data is not available, it is difficult to quantitize the effect of these parameters. Also, as you mentioned in a later comment, the fly ash additive of the	

REVIEW COMMENT RECORD (RCR)

1. Date

8-31-94

2. Review No.

3. Project No.

are

4. Page

2 of 13

12. Item	13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)	14. Hold Point	15. Disposition (Provide justification if NOT accepted.)	16. Status
1	(Continued)		new 5000 psi mix results in a retardation of the strength gain. This would tend to compensate for the more rapid initial strength gains exhibited by the more modern cements. The 2.28 core strength ratio, discussed in Appendix A, was not used for the new tank concrete property estimates, but was mentioned only as an example of Hanford concrete field data on the effects of aging on strength. As noted in the comments column, the compressive strength property estimates given in Table 2-1 were taken from the Hanford concrete time/temperature correlations derived from the CTL-generated data, as discussed in the Appendices.	

D-3

REVIEW COMMENT RECORD (RCR)

1. Date

8-31-94

2. Review No.

3. Project No.

are

4. Page

3 of 13

12. Item	13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)	14. Hold Point	15. Disposition (Provide justification if NOT accepted.)	16. Status
2	<p>Section 2.2:</p> <p>a. Wouldn't it be more correct to say recommended concrete design strength is 5000 psi. To account for degradation of concrete due to long-term elevated temperature exposure, it is recommended that 4000 psi be used in the design calculations. -- As stated, it implies a 4000 psi concrete mix.</p> <p>b. Except for preliminary calculations, material correlations are not valid because they were developed from limited data obtained from different concretes than the mix proposed. In order to be used for design purposes, the correlations need to be validated for the actual mix.</p> <p>c. ACI Code design strength? What if meant by ACI code design strength?</p> <p>d. Use of a 20% reduction in compressive strength may be very conservative. The enclosed Taywood Engineering report indicates that their results showed no long-term deterioration at temperatures up to 95°C for specimens stored in a stable moisture condition (sealed).</p>		<p>a. Section 2.2 has been modified to clarify this concern.</p> <p>b. Since the new tank concrete design temperatures are relatively low ($<200^{\circ}\text{F}$), it is difficult to justify a comprehensive testing program for the new tank concrete design mix. Tentative plans are to place concrete test cylinders in the annulus between the primary and secondary tanks, to be tested periodically to verify the long-term strength adequacy of the new mix design.</p> <p>c. I agree that "ACI Code design strength" is questionable terminology. The report has been changed accordingly.</p> <p>d. As you know, there is considerable scatter in the literature relative to the effect of temperature on concrete compressive strength. For example, the Figure 4-3 "Lower Bound Strength" curve indicates a 25% reduction at 200°F. Our current position, recommending a 20% reduction is judged to be a reasonably conservative position. If, at some later date, circumstances warrant a review of this position, the 20% value could be modified.</p>	

D-4

REVIEW COMMENT RECORD (RCR)

1. Date

8-31-94

2. Review No.

3. Project No.

are

4. Page

4 of 13

12.
Item

13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)

14.
Hold
Point

15. Disposition (Provide justification if NOT accepted.)

16.
Status

3

2.3, 2nd ¶:
Another difference is that the heat cycle the field concrete goes through will be different than the CTL data because of the more massive nature of the field concrete. When the prestressed concrete reactor vessels were constructed in England, they embedded thermocouples and used the output to control the temperatures of the water bath in which the specimens were placed (heat cycle). In the enclosed Taywood report, results are presented for concrete specimens cast in conjunction with several of the UK prestressed concrete reactor vessels. Except for the Wylfa and Harlepool specimens, the specimens were subjected to the heat cycle cure. Prior to testing, the specimens had been stored continuously in a sealed, stable moisture state at temperatures from 10° to 95° C, with some having been under sustained loading. The long-term properties (ages ranging from 4 to 24 years) were determined. Relative to 28-day reference compressive strength values, increases in strength for the Wylfa (23.8 years) and Heysham I (23.6 years) concretes were 55% and 59%, respectively, for sealed specimens maintained at 10° to 20° C. Although results are limited, gains in strength by samples obtained from U.S. nuclear facilities where water was not available for continued cement hydration also were relatively low (e.g., specimens from the fuel cycle facility at ANL-West exhibited a strength increase of 41% in going from 28-days to 30 years, and specimens from shielding concretes exhibited even lower strength gains).

The waste tank geometry is probably much less massive than the UK prestressed concrete reactor vessels. The waste tank wall thickness varies from 15 inches at the dome center to 18 inches at the base of the wall. I would expect that the heat cycle effects during curing to be a secondary effect.

You seem to be concerned that we may overpredicting the strength gain with age. You mentioned earlier a concern that we may have used a 2.2 aging factor, based upon core testing. This factor of 2.2 is not actually being used in the new tank design. The high factor is due more to conservatism in the design mix than aging. The aging that we are using is much less than this. Note, for example in Table 2-1, that a 70°F mean strength gain of 7750 to 9580 psi is specified (24% gain, 6 months to 50 years).

D-5

REVIEW COMMENT RECORD (RCR)

1. Date

8-31-94

2. Review No.

3. Project No.

are

4. Page

5 of 13

12. Item	13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)	14. Hold Point	15. Disposition (Provide justification if NOT accepted.)	16. Status
D-6 4	Additional information on thermal expansion of wet (Berkeley) versus dry (CTL) results for limestone concrete is contained in the CRBRP report ORNL/BRP-81/1/R1, which is enclosed. I would expect moisture to have an effect because of shrinkage due to moisture loss on heating. At elevated temperature, cracking also can be involved so Berkeley reported results in terms of total length change. Also, CTL only used 0.5-in. diameter specimens for the 0.75-in. maximum size aggregate concrete, whereas Berkeley used 6 by 12 inch cylinders. I have also included results from Berkeley where they looked at the effect of preventing moisture transfer.		<p>The expansion coefficients reported in this report are for constant moisture conditions. Volume changes due to drying shrinkage are assumed to be uncoupled from thermally-induced volume changes. The recently performed Bureau of Reclamation tests were run on both oven-dried and saturated sealed specimens. The Bureau reported very little difference between the oven-dried and saturated specimens (Appendix B). This was somewhat surprising, because the thermal expansion equations given in Section 2.9.2 of ACI-209R predicts that the degree of saturation has a noticeable effect on the thermal expansion coefficient.</p> <p>The Bureau tests were run at relatively low temperatures, since the new tank design effort is not concerned with temperatures above 200°F. The CRBRP testing done by Berkeley and the CTL tests were concerned with much higher temperature ranges. In our judgement, the standard, low temperature tests performed by the Bureau of Reclamation are adequate for our new waste tank application.</p>	

REVIEW COMMENT RECORD (RCR)

1. Date 8-31-94	2. Review No.
3. Project No. are	4. Page 6 of 13

12. Item	13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)	14. Hold Point	15. Disposition (Provide justification if NOT accepted.)	16. Status
5 D-7	<p>Section 2.5:</p> <p>Nesser and Chakraborty used Type V cement and Type C fly ash. General conclusion involving use of fly ash and extended range water reducers is valid. However, you should do some validation using actual materials. Would a Type F or Type C fly ash be used at Hanford? Apparent benefit derived from fly ash relative to elevated temperature exposure is probably due to delayed nature of strength gain, so 29-day value would be lower than for a comparable mix without fly ash. Compatibility between particular extended range water reducer and cement (cement plus fly ash) also needs to be evaluated as well as dosage to provide required workability characteristics (slump and time). In the past, I think a combination of regular and extended range water reducers has been utilized to provide the required workability (e.g., counteract relatively rapid loss of workability with time that had been exhibited by many of the extended range water reducers). Masler Builders or an equivalent organization could provide this information. Have trial batches and properties been developed?</p>		<p>The concrete mix slated for use on the new tank project is a pre-qualified 5000 psi mix developed for another Hanford project (HWVP). A Type I or II cement and a Type F fly ash are specified. This particular mix design is beyond the trial batch stage, and has been used in actual construction. The 5000 psi concrete thermal expansion test data (Appendix B) was obtained from cylinders generated for the HWVP project.</p>	
6	<p>Section 3.1:</p> <p>If you are developing a mix in the laboratory or when sufficient data are not available to establish a standard deviation, ACI 359 (ASME Section III, Division 2 - Table CC-2233r2.2-1) under laboratory conditions requires an average compressive strength of $f'_c + 1200$ psi (for 5000 psi concrete). Are ASME requirements being utilized? Again, trial mixes need to be developed in the lab and at proposed batch plant to demonstrate desired properties are achievable.</p>		<p>As mentioned above, the new tank concrete mix design is planned to come from a currently operating batch plant using an existing mix design. The current specifications include only ACI and ASTM requirements (no ASME requirements).</p>	

REVIEW COMMENT RECORD (RCR)

1. Date

8-31-94

2. Review No.

3. Project No.

are

4. Page

7 of 13

12.
Item

13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)

14.
Hold
Point

15. Disposition (Provide justification if NOT accepted.)

16.
Status

7

Table 3.1:
Total cementitious material is less for 5000 psi mix than CTL mix. Some results are indicating that where strength was used as the primary design parameter problems can develop later due to durability. Using reduced cement contents will produce desired strengths, but the permeability and resulting durability may be compromised because of lower cementitious materials content. Due to environments that concrete may see at Hanford, the concrete permeability may not be as important as it would be for design of a structure such as a bridge.

Based upon over 50 years of experience with concrete structures at Hanford, concrete durability has not been a significant issue. Other than a few isolated cases of degradation due to an adverse chemical exposure, durability/permeability has not been an issue for the concrete structures at Hanford. Specifically, for the new tank environment, there is a double liner between the waste and the concrete. The tanks will be buried and therefore not exposed to weathering. To date, no evidence of sulphate degradation from the Hanford soil environment has been found. Thus, durability/permeability should not be a significant concern for the new tank concrete.

D-8

REVIEW COMMENT RECORD (RCR)

1. Date

8-31-94

2. Review No.

3. Project No.

are

4. Page

8 of 13

12. Item	13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)	14. Hold Point	15. Disposition (Provide justification if NOT accepted.)	16. Status
8 D-9	<p>Section 3.2:</p> <p>Aggregate probably has greater significance with respect to its influence on thermal properties. Aggregate affects both the plastic and hardened concrete properties (strength, abrasion, resistance, dimensional stability, etc.). Since aggregate will occupy 75% (by weight) of concrete - what do you mean by aggregate being a secondary player in structural issues? Aggregate affects concrete properties which in turn affect ability of a structure to meet its performance requirements. Do you mean variability in aggregate from the various sources is of secondary importance relative to influencing resulting concrete properties? Same comment as above applies to later statement in section that aggregate tends to be a secondary player in the important concrete properties. Aggregate materials are important and not secondary.</p>		<p>The wording has been changed to "Hanford concrete aggregate is generally a secondary player in concrete structural issues". Although there is some variability in the Hanford gravel mineralogy, significant aggregate-related variability in the structural properties or incompatibility with the cement has not been observed in the past at Hanford. The gravel source used in the currently qualified 5000 psi design mix is planned to be used for the MWTF concrete.</p> <p>The primary purpose of Section 3.2 of the report is to address potential concrete aggregate effects on the Hanford concrete mechanical property data base. Based upon our review of this issue, it was concluded that we would not expect the Hanford aggregate variability to have a significant effect on the concrete mechanical properties. The one possible exception to this was the thermal expansion coefficient. This is one reason why we had thermal expansion testing done using the actual 5000 psi mix, which used aggregate from the gravel pits expected to be used for the new tank (MWTF) project.</p>	

REVIEW COMMENT RECORD (RCR)

1. Date

8-31-94

2. Review No.

3. Project No.

are

4. Page

9 of 13

12.
Item

13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)

14.
Hold
Point

15. Disposition (Provide justification if NOT accepted.)

16.
Status

9

Section 4.0:
General comment is related applicability of CTL data. You noted this earlier. Aggregates are essentially similar, but mixes and cementitious materials are different. Concretes will be different so use of CTL derived data should be done with caution, i.e., strength gain and response to elevated temperatures will be somewhat different due to use of fly ash.

Ideally, we should enter into an extensive test program for the intended 5000 psi design mix. However, with the relatively low concrete design temperature of 200°F, it is difficult to justify. Our current position is that the limited testing, mentioned in the Comment 2b response above, is sufficient.

D-10

REVIEW COMMENT RECORD (RCR)

1. Date	8-31-94	2. Review No.	
3. Project No.	are	4. Page	10 of 13

12. Item	13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)	14. Hold Point	15. Disposition (Provide justification if NOT accepted.)	16. Status
10	<p>Section 4.1.1:</p> <p>a. I'm a little confused in your adjustment of moist cured data to equivalent air-dry results. In the Troxell reference (Fig. 10.10), the 20% increase in strength relative to standard moist cure and tested moist was for specimens cured up to 3 months moist and then air-dried just prior to test. Results represent a locus of points and not a strength versus time relation. If specimens were moist cured for 1 month and then air-dried, the locus of strength results obtained from the air-dried specimens would be higher up to an age of about 4 months, and then within about 5% the strength of the moist cured test specimens at later ages. In hindsight, it probably would have been useful for CTL to have cast sealed specimens to simulate mass concrete conditions and provide reference strengths. Troxell notes that specimens air-dried and tested in air-dry condition were one-quarter to one-third stronger than specimens exposed to air and then saturated just prior to testing. In addition, Troxell data for moist followed by air dried only went out to 3 months initial moist cure, whereas CTL went 200-300 days moist cure prior to testing. Results presented by Troxell out to 12 months probably assume similar behavior. As you noted, there is also the affect of aging which should be taken into account. Continuous moist curing represents a control condition and indicates the strength that is achievable for the particular concrete mix (if temperature is not a factor). When air-drying starts, hydration slows until it is eventually arrested. Furthermore, when exposed to elevated temperature, hydration will initially increase (accelerated aging), cease, and then</p>		<p>a. We don't appear to be looking at the same Figure. My Troxell/Davis reference came from Figure 10.7 (shown below). Note that after 3 months the "moist-cured and air-dried prior to test" consistently runs about 20% higher than the "standard curing, continuously moist and moist at test" curve. When I first reviewed the CTL data and discovered that the 70°F data hadn't been included in the correlation development, my first attempts to incorporate the 70°F data yielding puzzling results. It then dawned on me that no attempt was made to account for the drying out which occurred when the wet specimens were heated up. Some of the short-term data actually showed increases in strength in heating from 70 to 250°F. Surprisingly, neither CTL, nor Battelle (correlation developer) even acknowledge moisture as a parameter of interest.</p>

FIG. 10.7. Effect of moist-curing conditions at 70°F and moisture content of concrete at time of test on compressive strength of concrete. (From Gilkey [876].)

REVIEW COMMENT RECORD (RCR)

1. Date

8-31-94

2. Review No.

3. Project No.

are

4. Page

11 of 13

12.
Item

13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)

14.
Hold
Point

15. Disposition (Provide justification if NOT accepted.)

16.
Status

D-12

Section 4.1.1 (Cont'd):
strength will decrease. This scenario is influenced by both rate of heat application and magnitude of temperature. It seems a more defensible approach for the compressive strength value would be to use reference continuous moist cure, air-cured and dry at test, or somewhere in middle (sealed) which probably would be more representative of the tanks. Moisture condition of concrete at time of heating is also important.

b. Compressive strength relation for temperature range of 50° to 200° F was used to develop compressive strength estimates in Table 2-1. It was noted in the table that 70° F estimates were from "dry" data fit, but earlier in this section it was noted that strengths in Appendix A and reported for others were for moist concrete. This is somewhat confusing. Why did you not reference results to moist cure as others have done?

c. Trying to separate temperature effects from aging is difficult. By not making the 20% adjustment, you can still establish the residual strength after thermal exposure relative to 28-day moist-cured specimen results. This can then be used to indicate the likely change in concrete strength. Also, if the concrete is exposed to a temperature of 200° F, or less, the effect of thermal exposure would probably be secondary to aging effects, particularly if the concrete would be in place for an extended period of time prior to being subjected to elevated temperature. Using the mean value for strengths at 50 years for 70° and 200° F, the ~29% decrease in strength seems somewhat high.

I found that by applying a 20% factor to account for the "tested dry" vs. "tested moist" differences between the high and low temperature data, it appeared to account for the data anomalies and produced a more reasonable data correlation. However, the data scatter makes it difficult to prove. Although the 20% factor approach is pretty rough, I believe it is an improvement over ignoring the effect of drying, which was previously done. The effect of moisture was one of the issues to be addressed by ANATECH CORP. (Joe Rashid) this past summer in their upgrading of the ANACAP-U software for Hanford. However, funding was cut and it hasn't been done yet. Hopefully, it will be looked at in FY95.

b. One of the objectives of Section 4.1 was to show that the Hanford concrete data correlations are reasonable relative to data reported in the literature (e.g. Figs. 4-3 & 4-6). To be consistent with the literature data, moist-cured reference values were used. The Table 2-1 values are intended to be best estimate values for field conditions. Since the field environment is 200°F concrete in dry sand, the dry condition appeared to be most appropriate.

c. Since we have no 50-year, 200°F data, our best estimate comes from extrapolating the CTL data. CTL did not develop 28-day data. We have attempted to be consistent with the existing correlations and software.

REVIEW COMMENT RECORD (RCR)

1. Date 8-31-94	2. Review No.
3. Project No. are	4. Page 12 of 13

12. Item	13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)	14. Hold Point	15. Disposition (Provide justification if NOT accepted.)	16. Status
11	Section 4.1.4: Part of last line on page is repeated at top of page (3 pages later).		This error has been corrected.	
12	Section 4.1.5: Tests were performed using oven-dried and fully saturated conditions. I assume specimens were saturated just prior to testing. In ACI 209 (Sec. 2.9.2), if I am not mistaken, these factors apply to moisture conditions to which the structures were subjected to for a period of time and not merely the condition at time of test. Neville & Brooks- (Table 13.3, 1991 edition) indicates slightly smaller thermal coefficient expansion for specimens moist cured as opposed to those which were cured by air drying (e.g., difference for granite aggregate concrete was about 10%). These results correspond to information presented in ACI 209.		The Bureau of Reclamation standard tests are performed in a water bath using specimens sealed in paraffin. It isn't clear to me why the time at the moisture condition would make a significant difference.	
13	Section 5.4, Figure 5-4: Are symbols properly identified in figure?		Figure 5-5 has been replaced with a corrected figure.	
14	Section 6.1: Adequacy of a 4000 psi design strength for a 200°F/50 year design temperature/design life can be In Table 2-1, the 5000 psi is specified as the nominal design strength. The 4000 psi would be the concrete design strength used in calculations since it accounts for 20% degradation due to long-term elevated temperature exposure. To be consistent, the 5000 psi should be the minimum specified 28-day strength and the design strength the 4000 psi value.		I agree. Sections 2.1 and 6.1 have been modified with the intent of distinguishing between the room temperature f'_c value of 5000 psi and the high temperature design strength of 4000 psi.	

D-13

MHC-SD-M236A-ER-013
Revision 0

REVIEW COMMENT RECORD (RCR)

1. Date 8-31-94	2. Review No.
3. Project No. are	4. Page 13 of 13

12. Item	13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)	14. Hold Point	15. Disposition (Provide justification if NOT accepted.)	16. Status
15	App. A, Residual Concrete Strength Predictions, Comparison with Hanford Core Data: Core test results were related to design strength. It would be much more meaningful to present study to compare core results to reference strengths if these results are available to indicate a trend. Our results also indicate that when using 28-day results as the reference strength, the ratio of core strength to reference strength will be larger for lower strength concretes and concretes that have access to moisture for continued hydration.		Unfortunately, the core testing involved older structures and 28-day reference strengths are generally not available.	

D-14