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LOAD REQUIREMENTS FOR MAINTAINING STRUCTURAL INTEGRITY OF HANFORD SINGLE-SHELL TANKS DURING WASTE FEED DELIVERY AND RETRIEVAL ACTIVITIES

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Abstract: This document provides structural load requirements and their basis for maintaining the structural integrity of the Hanford Single-Shell Tanks during waste feed delivery and retrieval activities. The requirements are based on a review of previous requirements and their basis documents as well as load histories with particular emphasis on the proposed lead transfer feed tanks for the privatized vitrification plant.

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July, 1999

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LOAD REQUIREMENTS FOR MAINTAINING STRUCTURAL INTEGRITY OF HANFORD SINGLE-SHELL TANKS DURING WASTE FEED DELIVERY AND RETRIEVAL ACTIVITIES

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EXECUTIVE SUMMARY

The purpose of this document is to provide a sound basis for the equipment protection requirements found in the HNF-3912 (1999), *System Specification for the Single-Shell Tank System*, under Section 3.3.6.2 for Tank Temperature Limits, Tank Liquid Waste Levels, Single-Shell Tank Pressure Limits, and Dome Loading. The goal is to specify the required minimum structural-related constraints necessary to maintain the structural integrity (load-carrying capacity) of the single-shell tanks (SSTs) for the retrieval and waste feed delivery (WFD) to the privatized vitrification plant. Although maintaining leak tightness is important in protecting the environment, many of the SSTs have already leaked. Thus, the emphasis here is on maintaining adequate load-carrying capacity. Although the recommended limits apply to all SSTs, emphasis is given to the proposed Phase-1 lead-transfer feed tanks 241-C-102 and -104.

The initial preliminary assessment given in HNF-4047 (1999), *Engineering Basis Document Review Supporting the SST System Specification Development,* was based on a detailed review and screening of the information contained in the Operational Specification Documents (OSDs) and the Technical Safety Requirements (TSRs). The recommended structural integrity requirements reported herein were developed through an expanded review of existing requirements and their technical baseline documents (see Appendix A), as well as, the load histories of the proposed lead-transfer feed tanks (see Appendix B).

Current TSRs controls are based on the preventive and mitigative features determined to be essential in HNF-SD-WM-BIO-001 (1998), *Tank Waste Remediation System Basis for Interim Operation*. The BIO is based on hazard analyses to ensure adequate protection to the onsite workers and the public. Though adequate controls bound safety analysis hazards, additional conservative controls are required to maintain code-based design margins to ensure adequate equipment protection. The recommended load limits provided herein are based on structural integrity considerations; other operating factors could further restrict these limits.

Because of uncertainties in the current structural condition of the SSTs as well as uncertainties in the supporting structural analysis technical baseline, any relaxation of the current OSD and TSR structural related constraints is not recommended. The recommended load limits are consistent with the current OSD and TSR constraints except for the heat-up/cool-down rate. A reduction in the heat-up/cool-down rate from 20 °F/day to 3 °F/day is recommended until the adequacy of the higher value can be confirmed. Overall uncertainty in the conservatism of the SST technical baseline led to the TSR imposed constraint against additional soil loading on the SSTs in order to minimize the risk of structural damage. Additional analyses have been proposed to provide a sounder technical baseline.

The load history for the 241-C-106 high-heat tank bounds the load histories of the Phase-1 lead-transfer feed tanks 241-C-102 and -104. Hence, the Phase-1 lead-transfer feed tanks can be considered structurally adequate based on the recent seismic and structural evaluations of the 241-C-106 tank for its bounding load history.

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1.0 INTRODUCTION

This document supports the development of structural integrity requirements for the Hanford Site single-shell tanks (SSTs) in an effort to ensure continued adequate structural integrity of the SST system during the Phase 1 and Phase 2 retrieval and transfer of waste feed material to the privatized vitrification plant. These requirements are to be included in HNF-3912 (1999), *System Specification for the Single-Shell Tank System*, which supports the Waste Feed Delivery (WFD) and retrieval mission described in HNF-SD-WM-MAR-008 (1999), *Tank Waste Remediation System Mission Analysis Report*. Only structural integrity (load bearing) related requirements are addressed herein—liquid integrity (functional) requirements are not explicitly addressed. Structural integrity, as defined herein, refers to the ability of the structure to maintain a stable geometry under applied loads with sufficient margin. Structural failure (collapse) is defined as the calculated inability of the structure to withstand normal operating loads or normal operating plus seismic loads. The reinforced concrete vault structure is the primary load-bearing component of the SSTs.

Liquid integrity of the SSTs is associated primarily with the leak tightness of the steel liner and secondarily with the leak tightness of the concrete vault in the event of a liner failure. Hence, leakage of a tank does not imply loss of tank structural integrity (load-carrying capacity) but it does lead to a loss of function (containment of waste) with resulting potential environmental consequences. The magnitude of the environmental consequences is a function of the net volume of waste that could be released before the remaining liquid content can be retrieved. Loss of leak tightness has already occurred in about 45 percent of the SSTs—to date 67 of 149 of the SSTs have been declared "known or assumed leakers" (HNF-EP-0182-129, 1999). Continued leak tightness cannot be assured in the remaining SSTs with any high degree of certainty. Although on-going interim stabilization efforts reduce the available liquid volume that could be leaked during the extended storage of the remaining waste material, application of certain retrieval or waste feed transfer options, such as sluicing, can increase the available liquid volume.

This report expands on the review screening of existing Operational Specification Documents (OSDs) given in HNF-4047 (1999), *Engineering Basis Document Review Supporting the SST System Specification Development*. The resulting recommended structural integrity requirements reported herein were developed through a review of existing requirements and their basis documents (see Appendix A) as well as load histories with emphasis on the proposed lead-transfer feed tanks 241-C-102 and -104 for the privatized vitrification plant (see Appendix B). Recommended load limits and potential improvements in the technical baseline are identified.

2.0 BACKGROUND

The SSTs consist of a group of underground 133 large capacity (530, 758, and 1,000 Kgal) 100-Series tanks with a 75-foot internal diameter and 16 small capacity (55 Kgal) 200-Series tanks with a 20-foot internal diameter. See Figure 1 for a schematic of the 100- and 200-Series SST configurations. The tanks are clustered within twelve Tank Farms identified as A, AX, B, BX, BY, and C in the 200 East Area and S, SX, T, TX, TY, and U in the 200 West Area of the Hanford Site near Richland, Washington. The tanks provide interim storage of high-level radioactive waste until future processing and permanent disposal options become available. The tanks were built from 1943 to 1964 with a presumed design life of 20 to 25 years. The preliminary functional design criteria for the AX tanks (HW-70529, 1961) lists a design life of 25 years with a 1/16-inch corrosion allowance for the steel liner. Hence, the current age of the SSTs varies from 35 to 56 years, which is well beyond their design life. In addition, some of the 100-Series SSTs exceeded their original design temperatures. All SSTs were removed from service (no waste receipts or transfers except for removing liquids) by November 1980. As of December 1998, 119 of the 149 SSTs have been interim stabilized (less than 50 Kgal of drainable interstitial liquid and less than 5 Kgal of supernatant liquid remain in the tank). Ongoing comprehensive maintenance and surveillance programs have ensured continued safe operations to date.

The SSTs are underground, steel-lined, reinforced concrete structures. The reinforced concrete vault provides a secondary waste containment barrier and is the primary load support structure that resist internal hydrostatic loads from the stored waste, external soil loads, and equipment loads. The 100-Series type vault is a reinforced concrete structure consisting of a dished- or flat-bottom circular slab, vertical cylindrical wall, and a shallow (12-foot rise) elliptically shaped dome with a stiffened haunch region. The bottom slab extends beyond the wall to act as a wall footing with continuous reinforcing steel extending from the slab into the wall. The vertical concrete wall for the 530- and 758-Kgal tanks is 12- and 15-inches thick, respectively. For the 1,000-Kgal tanks, the wall thickness is 24 inches in the lower 2/3 of the wall and transitions to15 inches in the upper wall region. The dome is 15-inches thick at the center and transitions to the thicker haunch region.

The thin walled (1/4 to 3/8-inch thick) carbon steel liner is the primary waste containment barrier. The steel liner is not structurally attached to the concrete vault but is retained within the concrete vault in an open-cup arrangement. A layer of cement mortar reinforced with a 2x2-inch square wire mesh fabric over a 3-ply asphaltic membrane waterproofing is provided between the steel liner and the concrete vault structure (except A, AX and SX tanks). The bottom steel liner rests on a 2-inch wet-grout layer (except AX tanks, which have drain slots) which is supported by the reinforced concrete bottom slab. The steel liner is circumferentially stiffened by ring shaped angles welded to the inside face of the liner vertical wall of the 100-Series SSTs. The SSTs were put into service with the steel liners in the as-welded (no post-weld stress relief) condition. Although not verified, this lack of post-weld stress relief has been assumed to be a major contributor in the leakage of the SSTs due to stress-corrosion cracking (SCC) when coupled with an adverse waste chemistry and high temperature. General corrosion and local pitting corrosion can also lead to leakage of the SSTs.

Table 1 summarizes the material and design specification data for the 100-Series SSTs. The wall thickness of the steel liner is also provided for each of the various SST designs. A more detailed comparison of design details between the SSTs is given in WHC-SD-WM-TI-598 (1994). The tanks are listed in Table 1 in the chronological order of construction. The 530- and 758-Kgal tanks were not designed for the storage of self-boiling waste; their original design temperature was 220 °F. The 1,000-Kgal tanks were designed for the storage of self-boiling waste; their original design temperature was 250 °F (WHC-SD-WM-TI-648, 1994). Some of the data listed in Table 1 is not readily available (NRA) and has been so indicated. In the case of the seismic qualification, HW-37519 (1955) states that the tanks meet the earthquake requirements of the Uniform Building Code (UBC) applicable at the time of construction and can be considered earthquake resistant. However, HW-37519 does not provide any supporting references.

Tables 2 and 3 summarize the status of the 100- and 200-Series tanks, respectively. In addition to certain design features listed, these tables list the construction, in-service, and out-of-service dates;

leak status; historical peak temperature and date of occurrence; current temperature and current total waste volume; watch-list status; and stabilization and isolation status.

3.0 SINGLE-SHELL TANK STRUCTURAL INTEGRITY REQUIREMENTS

Internal and external loads affect the structural integrity of the SSTs. Normal loads associated with in-place soil loading; internal vapor pressure from active ventilation system operation and transient pressures from spontaneous gas releases or boiling; hydrostatic pressure from stored waste; dead loads from in-place equipment; and live loads from crane or support vehicle activities over or near the tanks are of primary consideration.

In addition, exposure to temperatures above 150 °F over time can reduce the concrete strength and modulus of elasticity, as well as increase the creep rate of the concrete. Cyclic loads due to fill and drain cycles and the associated thermal transient loads can adversely affect the performance of the concrete structure. The effect of elevated temperature on the concrete is non-recoverable. Hence, the thermal load history experienced by each of the SSTs is important in assessing the current structural integrity of the SSTs. The potential degradation of the concrete is one of the larger uncertainties in assessing the current structural integrity of the SSTs.

The tank structure must also have sufficient reserve capacity to sustain natural phenomena and accident type loads. Natural phenomena loads of interest include snow, volcanic ashfall, and seismic induced loads. Accident loads include loads from potential hydrogen deflagration, organic-salt nitrate reaction, organic solvent or gasoline fires, or equipment load drops. Seismic induced ground motion is likely to cause a spontaneous release of retained gases in those tanks that tend to accumulate gas in the waste. If the composition of the released gaseous mixture is within its flammability range and a spark of sufficient energy is generated, the flammable gas mixture could be ignited resulting in an internal pressure transient within the tank. A spark could be generated by metal-to-metal impacts resulting from the seismic induced relative motion between in-tank equipment and its support riser. Hence, a seismic event could also be the initiator for a hydrogen deflagration (HNF-SD-WM-BIO-001, 1998).

The liquid integrity of the SSTs is affected by waste chemistry (NO_3 and OH + NO_2 relative concentration), stress state of steel liner including weld induced residual stress, and waste temperature. These elements affect the corrosion characteristics of the steel liner (WHC-EP-0772, 1994). In addition, for actively ventilated tanks, a high ventilation induced vacuum coupled with a low tank waste level could lead to an uplift of the bottom of the steel liner that could result in tank leakage of the remaining liquid waste.

Current requirements have been established to restrict operations in order to mitigate the consequences or prevent the occurrence of adverse events from a safety (hazard/consequence) perspective. The safety basis for these requirements/controls is HNF-SD-WM-BIO-001 (1998), *Tank Waste Remediation System Basis for Interim Operation*. Additional controls have also been established in Operating Specification Documents (OSDs) to ensure continued usage of the tanks consistent with their mission.

3.1 CURRENT REQUIREMENTS

Current technical safety requirements (TSRs) for single-shell tanks (SSTs) are given in HNF-SD-WM-TSR-006 (1998), *Tank Waste Remediation System Technical Safety Requirements*, on the basis of HNF-SD-WM-BIO-001 (1998) until the BIO is replaced by a fully compliant Final Safety Analysis Report (FSAR). The TSRs are based on the preventive and mitigative features determined to be essential in the BIO. An Addendum to the TSR document contains Transitional Requirements – controls that have been directed by DOE to be retained in the TWRS Authorization Basis. Implementation procedures are contained in lower tier documents, such as HNF-IP-1266 (1997), *Tank Farms Operations Administrative Controls*. The TSR and BIO define acceptable conditions, safe boundaries, and bases thereof, and management or administrative controls required to ensure safe operation during waste storage, transfer, and characterization. The TSR and BIO do not specifically cover environmental regulatory requirements. Operating Specification Documents (OSDs), such as, OSD-T-151-00013 (1998), *Unclassified Operating Specifications for Single-Shell Waste Storage Tanks*, OSD-T-151-00030 (1998), *Operating Specification for Watch List Tanks*, and OSD-T-151-00031 (1998), *Operating Specifications for Tank Farm Leak Detection and Single-Shell Tank Intrusion Detection*, impose additional technical limits and controls on processes or operations associated with the SSTs which, if violated, could damage equipment or facilities, hamper operations, jeopardize compliance with environmental requirements, or adversely affect product quality. These lower tier documents provide more restrictive limits and controls to assure continued usage in order to meet programmatic needs until final retrieval and closure has been achieved.

3.2 NEW REQUIREMENTS

The SST waste retrieval and transfer feed activities may require changes to the current requirements in order to meet the demands of the proposed waste retrieval and transfer feed effort while still maintaining safety and environmental requirements. These new requirements have not been established to date and may require additional analysis and/or testing to demonstrate compliance.

3.3 STRATEGY FOR DEVELOPMENT OF STRUCTURAL INTEGRITY REQUIREMENTS

The strategy for determining the limiting requirements to be imposed on future waste feed and retrieval efforts as they affect SST continued structural and leak integrity is based on a review and assessment of the supporting structural analyses, past and current restrictions, operating history to date, and future needs. The scope of the review and assessment herein includes all SSTs; however, its focus is on the Phase 1 WFD activity for the proposed lead-transfer feed tanks 241-C-102 and -104. This allows direct comparison with tank 241-C-106 (current high-heat lead-retrieval tank) operating history and its recent retrieval activity supporting analyses. The following tasks were completed in support of this effort:

1) *Review mission/system level documents that contain structural integrity requirements (e.g., WAC codes, TPA, PHMC contract, privatization contract). Summarize existing requirements and commitments regarding structural integrity requirements.*

The following documents were reviewed regarding structural integrity (load-bearing capacity) requirements.

- HNF-2944 (1998), *Single Shell Tank Retrieval Program Mission Analysis Report.*
- HNF-2826 (1998), *Single-Shell Tank System Functional Analysis.*
- HNF-SD-WM-MAR-008 (1999), *Tank Waste Remediation System Mission Analysis Report.*
- HNF-2919 (1999), *Constraints for System Specifications for the Double-Shell and Single-Shell Tank Systems.*
- WAC (1998), *Dangerous Waste Regulations, Chapter 173-303.*

2) *Complete an industry survey, including DOE, of sites that have successfully addressed the requirement issues on integrity of underground radioactive and non-radioactive waste storage tanks. Determine and summarize the applicability of this work and its results to C102/C104.*

The scope of this task was limited to a review of the following document:

- BNL-52527 (1997), *Guidelines for Development of Structural Integrity Programs for DOE High-Level Waste Storage Tanks.*

This document provides general guideline for the development of structural integrity programs for DOE high-level waste storage tanks. Although elements of the guidelines are applicable to SSTs, the scope of the document is focused on double-shell tanks (DSTs). See Appendix A (under Codes and Standards) for additional review comments.

3) *Review the documentation used for justifying the structural integrity of C-106 to support the current sluicing program.*

Documents reviewed under this task include:

- HW-1946 (1943), *Specification for Composite Storage Tanks – Bldg. #241 at Hanford Engineer Works Project 9536* (original specification for the C Tank Farm).
- ARH-CD-948 (1977), *History and Status of Tanks 241-C-105 and 241-C-106.*
- RHO-CD-638 (1979), *Engineering Study on Tanks 105-C and 106-C for Long Term Structural Integrity.*
- WHC-SD-W340-ES-001 (1993), *Project W-340 Manipulator Retrieval System Tank 241-C-106.*
- WHC-SD-W320-ANAL-001 (1995), *Tank 241-C-106 Structural Integrity Evaluation for In situ Conditions.*
- WHC-SD-W320-ANAL-002 (1995), *Seismic Evaluation of Tank 241C106 in Support of Retrieval Activities.*
- WHC-SD-W320-ANAL-003 (1995), *Tank 241C106 Structural Evaluation in Support of Project W320 Retrieval,*

WHC-SD-W320-ANAL-001, -002 and -003 document results of an extensive state-of-the-art evaluation of the structural integrity of the 241-C-106 high-heat tank to historical operating loads, a 0.20-g earthquake based on HPS-SDC 4.1 (1993) load criteria for non-nuclear Safety Class 1 structures, and retrieval operational loads, respectively. See review of these reports in Appendix A. The 241-C-106 analyses also bound the 241-C-102 and -104 tanks since the load history of the 241-C-106 tank is bounding (see Table 2 and Appendix B) for this group of tanks. Because temperature in the concrete did not exceed 150 °F for at least 15 years after construction, the concrete strength was assumed to be higher than the 28-day minimum-specified design value of 3,000 lbf/in^2 due to aging. An undegraded compressive strength of 4,600 lbf/in^2 was estimated based on the lower-bound 95% confidence-band relation developed from Hanford-mix concrete lab-test data. Corresponding concrete strength and modulus degradation relations with time at temperature were applied in the thermal-creep analysis of the 241-C-106 tank for its estimated upper-bound thermal history (WHC-SD-W320-ANAL-001, 1995).

4) *Review the structural calculations supporting the OSDs and TSRs for C102/104 and determine the compliance with current criteria in HNF-PRO-097.*

Documents reviewed under this task include:

- HNF-PRO-097 (1997), *Project Hanford Policy and Procedure System: Engineering Design and Evaluation.*
- HNF-SD-WM-TSR-006 (1998), *Tank Waste Remediation System Technical Safety Requirements.*
- HNF-SD-WM-BIO-001 (1998), *Tank Waste Remediation System Basis for Interim Operation.*
- OSD-T-151-00013 (1998), *Unclassified Operating Specifications for Single-Shell Waste Storage Tanks.*
- OSD-T-151-00030 (1998), *Operating Specification for Watch List Tanks.*
- OSD-T-151-00031 (1998), *Operating Specifications for Tank Farm Leak Detection and Single-Shell Tank Intrusion Detection.*
- BNL-52361 (1995), *Seismic Design and Evaluation Guidelines for Department of Energy High-Level Waste Storage Tanks and Appurtenances.*
- WHC-SD-TWR-RPT-002 (1996), *Structural Integrity and Potential Failure Modes of the Hanford High-Level Waste Tanks.*
- WHC-SD-TWR-RPT-003 (1996), *DELPHI Expert Panel Evaluation of Hanford High Level Waste Tank Failure Modes and Release Quantities.*
- SD-RE-TI-012 (1983), *Single-Shell Waste Tanks Load Sensitivity Study.*
- WHC-SD-WM-TI-623 (1994), *Static Internal Pressure Capacity of Hanford Single-Shell Waste Tanks.*
- Selected documents from bibliography of single-shell tank related analyses were also reviewed (see Appendix A).

The hazard classification for the SSTs is specified as Hazard Category 2 in HNF-SD-WM-BIO-001 but their Safety Classification is not specified in the BIO. SSTs are designated as Safety-Class structures in HNF-SD-WM-SAR-067 (1999), *Tank Waste Remediation System Final Safety Analysis Report*, for the Natural Phenomena—Seismic accidents. This leads to a Performance Category (PC) 3 classification for SSTs under seismic loading in accordance with HNF-PRO-097 (1997) which is driven by DOE Order DOE 5480.28 (1993), *Natural Phenomena Hazards Mitigation*. For existing PC 3 type structures located in the 200 Area of the Hanford Site, the evaluation basis horizontal peak ground acceleration is 0.18 and 0.19 g for the East and West Area, respectively (HNF-PRO-097).

Section 3.4.2.12.3 of HNF-SD-WM-SAR-067 shows that a seismic event at the tank farm could damage tank farm facilities. However, tank failure or collapse is not expected for the evaluation basis earthquake based on available analyses [RHO-R-6 (1978), WHC-SD-W320-ANAL-002 (1995), WHC-SD-TWR-RPT-002 (1996), WHC-SD-TWR-RPT-003 (1996), and RLCA (1996)] and worldwide experience of typical damage to structures as a function of seismic acceleration level (see Table A-3 in Appendix A reproduced from HNF-SD-WM-SAR-067). The tank functions to confine materials sufficiently to mitigate onsite and offsite consequences. The safety function of the waste tanks is to maintain gross tank structural integrity, which averts tank collapse and limits the release of waste during and after the evaluation basis earthquake, thus decreasing the consequences of the Natural Phenomena—Seismic accident.

Strict compliance to HNF-PRO-097 has not been shown because all SST seismic analyses predated HNF-PRO-097, which introduced new design response spectra for Hanford Site facilities. However, based on available seismic analyses of SSTs, compliance to HNF-PRO-097 could likely be demonstrated by analysis. The seismic load capacity of SSTs was also evaluated in RLCA (1996) which concluded that the SSTs would not fail until about 0.8 g. The RLCA analysis conservatively neglected soil-structure interaction effects but followed the guidelines, methods, and criteria of DOE Standard DOE-STD-1020-94 (1994), *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities* and BNL 52361 (1995), *Seismic Design and Evaluation Guidelines for The Department of Energy High-Level Waste Storage Tanks and Appurtenances*.

Although the snow and ashfall load requirements in HNF-PRO-097 are not explicitly addressed in the SST structural analyses, the dome load controls in Chapter 5.16 of HNF-IP-1266 (1997) require that a reserve for snow and ashfall be included in the total applied distributed load for comparison to the qualified distributed load. This meets the intent of HNF-PRO-097.

5) *Review the history of usage (e.g. temperature history, fill/drain cycles, and the corresponding temperature transients) for C102/C104. Prepare a load histogram from this data and compare to C106.*

Documents reviewed under this task include:

- WHC-SD-WM-ER-313 (1996), *Supporting Document for the Historical Tank Content Estimate for C Tank Farm.*
- Tank dome elevation survey data for C102 and C104 in comparison to C106.

See Table 2 for comparison of peak temperatures and Appendix B for comparison of historical waste levels, temperatures, and dome elevation survey data. The waste level and temperature data in Appendix B was obtained from WHC-SD-WM-ER-313 (1996). The temperature data reproduced from WHC-SD-WM-ER-313 only goes back to 1974. Table 2 included limited available data from RHO-CD-1172 (1981) back to 1961. A complete record of the temperature histories of the SSTs is not currently available. However, based on the available data for C102, C104, and C106, it is clear that the C106 thermal history bounds the thermal histories of tanks C102 and C104. Hence, the structural integrity of tanks 241-C-102 and -104 is expected to be better than that of 241-C-106.

6) *In light of the above information, identify structural failure modes and the mechanisms that may initiate them.*

Documents reviewed under this task include:

- RHO-CD-1485 (1981), *Description of Potential Failure Modes for Single-Shell Waste Tanks.*
- WHC-SD-TWR-RPT-002 (1996), *Structural Integrity and Potential Failure Modes of the Hanford High-Level Waste Tanks.*
- WHC-SD-TWR-RPT-003 (1996), *DELPHI Expert Panel Evaluation of Hanford High Level Waste Tank Failure Modes and Release Quantities.*
- RLCA (1996), *Evaluation of Hanford High Level Waste Tank Failure Modes for Seismic Loading.*

7) *Using the above background, propose structural integrity requirements (i.e. temperature, temperature rate of change, temperature differential, tank pressure, pressure rate of change and dome loading) for the SSTs.*

Recommended load restrictions to maintain structural integrity are summarized in Section 3.4 below.

3.4 RECOMMENDED STRUCTURAL INTEGRITY REQUIREMENTS

Based on above reviews (see Table 4 and Appendix A), proposed structural integrity requirements (i.e. temperature, temperature rate of change, temperature differential, tank pressure, pressure rate of change, and dome loading) were developed. The following limits are recommended for inclusion in HNF-3912 to provide adequate equipment protection against loss of structural integrity. These limits assume that the tank is structurally sound currently, has not leaked, and that the tank has not been subjected to conditions outside these requirements during its previous history. If the tank has previously leaked, the structural integrity of the concrete and reinforcement that has been exposed to the waste may have been degraded. Although lab tests subjecting concrete and reinforcing steel specimens to simulated double-shell slurry and simulated salt cake solutions at 180 °F did not indicate any adverse effects (RHO-RE-CR-8, 1982), degradation of the concrete and reinforcing steel cannot be ruled out. It is important to realize that the structural load limits are not independent. The recommended limits are based on structural integrity considerations; other operating factors could further restrict these limits. Table 4 provides an overview of the evolution and basis documents for the following recommended requirements.

Tank Temperature Limits

The system shall maintain waste temperature in each 100-Series SST within the following limits:

Maximum 300 °F for waste
Maximum 250 °F for dome
Maximum change of 3 °F/day for bulk waste temperature condition in tank.

These requirements are based on SD-RE-TI-012 (1983) and technical bases listed in SD-RE-TI-035 (1985). Preference is given to the more conservative requirements in SD-RE-TI-035 because of uncertainties in the adequacy of the SD-RE-TI-012 supporting analyses (see review in Appendix A).

The maximum temperature limits are consistent with the structural assessments of the existing SSTs. Although SD-RE-TI-012 (1983) proposed a maximum concrete wall temperature at the tank base of 380 °F, it is not clear that the SD-RE-TI-012 analysis adequately bracketed the potential degradation of the concrete compressive strength and elastic modulus at high temperatures. The 300 °F operating limit is considered acceptable per OSD-T-151-00013 (1998). Note, however, that some tanks have experienced temperature in excess of 300 °F, up to 600 °F (see Table 2). In addition, a complete record of the thermal history of each tank is not available. As the effects of elevated temperature on the strength and modulus of the concrete are non-recoverable, the temperature history of the tank is important in evaluating the current structural integrity of the tank. Note that current HNF-SD-WM-TSR-006 requirements limit the waste temperature of tanks C-106, SX-103, SX-107 through -112, and SX-114 to $\leq$ 250 °F (safety limit) or $\leq$ 205 °F (operating limit) to prevent salt-nitrate reactions and tank bumps for tanks containing sludge with estimated heat loads > 26,000 Btu/h.

Although a 20 °F/day rate of temperature change is supported by the SD-RE-TI-012 (1993) sensitivity structural integrity assessment, this value may be potentially too high and should be restricted to not exceed 3 °F/day until the adequacy of the higher value can be confirmed (WHC-SD-WM-OSR-005, 1994). All analyses before SD-RE-TI-012 (1983) restricted the heat-up/cool-down temperature rate to approximately 3 °F/day. In the case of SD-RE-TI-012, the stress analyses are based on steady-state heat transfer results rather than thermal transient analyses and hence may under predict the thermal induced stresses. In the 1:10-scale model test (ARH-R-47, 1969) the test tank was subjected to two cycles of severe thermal transient resulting in a nominal 100 °F through-wall thermal gradient over a one-hour heat-up period. Although the soil loading and restraint of the bottom section of the scale-model tank was not representative of the prototype tank conditions, delamination of the inside concrete cover from the first inside layer of reinforcement just above the footing was observed. In addition, extensive random cracking was observed in the dome and sidewall of the concrete structure. More recent generic SST stress analysis in WHC-SD-WM-DA-150 (1994) using ANSYS® did not address thermal transient induced stress effects because of numerical stability problems in the concrete constitutive model when attempting to include the 20 °F/day heat-up condition in the Phase I effort. In the WHC-SD-WM-DA-150 local wall-model *heat transfer* analysis, a maximum nonlinear through-wall transient temperature gradient of 18.8 and 51 °F was predicted for a heat-up from ambient to 350 °F at 3 and 20 °F/day, respectively.

SD-RE-TI-035 also listed through wall and meridional thermal gradient limits for the concrete wall (see temperature limits in Table 4 for summary of these thermal gradient limits). As these additional concrete thermal gradient requirements cannot be monitored, the 3 °F/day requirement is assumed to adequately limit the thermal gradients in the concrete wall. Through wall and along the wall (meridional) thermal gradients result in thermal induced stresses that if cycled can reduce the effective strength of the concrete. However, thermal induced cracking from thermal transients are not expected to decrease the ultimate load capacity of the structure unless delamination between the concrete and reinforcing steel occurs at a critical location, leading to a reduction in the reinforcement bond strength.

The 200-Series SSTs have not been evaluated for elevated temperatures because these tanks are not typically exposed to elevated temperatures (see Table 3).

Tank Liquid Waste Levels

The system shall prevent liquid waste levels from exceeding the following limits with a maximum waste specific gravity (SpG) of 2.0:

Tank Identifier	Maximum Waste Level (in.)
A, AX, SX	360
B, C, T, U (200-Series)	280
B, BX, C, T, U (100-Series)	185
BY, S, TX, TY	275

These requirements prevent waste overflow as well as limit the hydrostatic head induced stresses in the tank. These requirements are based on SD-RE-TI-012 (1983). Note that previous elastic analyses resulted in more restrictive conservatively based limits (see discussion in Table 4 under Hydrostatic Head).

SST Internal Pressure Limits

The static internal vapor pressure in SSTs shall not exceed the following limits:

Maximum pressure: +60 in. water gauge (w.g.)
Minimum pressure: -15 in. w.g. with waste level ≧15 in.
≧0 in. w.g. with waste level less than 15 in.

Note that only SSTs containing self-boiling waste are actively ventilated continuously (see Table 3) to limit the maximum temperature of the tank. However, temporary active ventilation can be imposed during certain activities, such as, waste characterization activities. The maximum pressure limit provides an adequate margin against dome failure due to a gradual positive internal pressure increase. The +60-in. w.g. value is a historical value and corresponds to the design relief tank pressure for the water seal in the vent system. An upper limit of +130 in. w.g. was recommended in Letter, 1982, *Vapor Pressure, Single-Shell Tanks,* on the basis of the maximum pressure required to just counteract the minimum soil cover load on an SST. The static tank failure pressure was estimated to be greater than 10 psig in HW-37519 (1955) and WHC-SD-WM-TI-623 (1994).

The minimum pressure requirements prevent buckling of the steel liner sidewall (-15 in. w.g. limit includes 1.4 safety factor against sidewall buckling), prevent uplift of the steel liner bottom plate, and service to limit the total load on the dome when combined with the soil and live load restrictions (Letter 1982). The historic minimum vapor pressure value was -6 in. w.g. and corresponds to the six-inch water seal in the tank farm vent header. The waste level restrictions associated with the minimum pressure prevent uplift of the steel liner bottom plate by maintaining a net positive (downward) pressure on the inside surface of the liner bottom plate.

The above restrictions on the minimum vapor pressure have been simplified and are conservative because they neglect the added pressure from the waste when the bulk SpG of the waste is greater than 1.0. The bulk SpG of the waste ranges from 1.0 to 2.0. A more accurate limit on the vapor pressure is given by

If waste level is ≧ 15 inches divided by the SpG of the waste then
-15 in. w.g. ≤ vapor pressure ≤ +60 in. w.g.

If waste level is < 15 inches divided by the SpG of the waste then
-(waste level) · (SpG of waste) ≤ vapor pressure ≤ +60 in. w.g.

Dome Load Limits

Dome loading on SSTs shall not exceed the maximum loading specified in Chapter 5.16, "Dome Load Controls" of HNF-IP-1266, *Tank Farms Operations Administrative Controls*.

These requirements prohibit the addition of soil over any SST; limit the total concentrated load to 200,000 lbs (100 tons) and 100,000 lbs (50 tons) over 100- and 200-Series SSTs, respectively; and limit the lift height of large equipment over 100-Series SSTs to not exceed 20 feet above the surface grade or pit floor of the tank. These limits were derived from HNF-SD-WM-TSR-006 (1998) and SD-RE-TI-012 (1983).

The above recommended structural integrity related load limits for SSTs are consistent with current restrictions except for the heat-up/cool-down rate of temperature change, which is more restrictive than current restrictions based on SD-RE-TI-012 (1983) because of uncertainties in the SD-RE-TI-012 supporting analyses. Deviations from these load restrictions would require a case-by-case evaluation of the affected SST.

4.0 DISCUSSION

The above load requirements on SSTs are based on a review of previous requirements and their basis documents. The historical structural requirements are summarized and compared in Table 4. These requirements were established through various analyses conducted at different times with varying degrees of complexity. The original design calculations for the SSTs could not be retrieved and may no longer exist. The changing needs of tank operations (higher temperatures and temperature rates, and increased specific gravity as a result of waste self concentration and evaporation campaigns) has resulted in periodic reassessments of the structural capacity of the various tank designs to assure maximum utilization of the existing storage tanks. These post-construction evaluations varied from simplified hand calculations to detailed finite-element analysis techniques, which paralleled corresponding changes in the state-of-the-art of structural analysis techniques. In an effort to meet the changing needs of the SSTs, allowable stresses were increased [see review of HW-37519 (1955) in Appendix A]. Hence, the design envelope has been expanded beyond normal design-code practice and beyond the anticipated design life of 20 to 25 years for the SSTs.

An added complication is the uncertainties in material properties of the concrete under elevated temperatures, as well as, the uncertainty in the soil thermal conductivity and the thermal history of the waste during storage and fill/drain cycles. Exposure of concrete to high temperatures and rapid changes in temperature can lead to a loss in concrete strength and a reduction in modulus as well as introduce unfavorable thermal stresses and accelerated creep behavior. The uncertainty in thermal conductivity of the soil affects the resulting predicted temperature and thermal gradient distribution in the concrete structure.

Different assumptions on material properties were introduced as results became available from an extensive concrete testing program that began in the mid-1970s to determine the effects of elevated temperature on concrete properties. Most analyses (except ARH-C-11, 1976 and RHO-SA-108, 1979) assumed that the concrete compressive strength of the tank concrete structure had not been degraded below its design specification 28-day strength. This was justified in SD-RE-TI-012 (1983) based on a statistical evaluation of limited concrete-core test data and extensive lab-test data for Hanford-mix concrete. However, only temperatures up to 250 °F were considered in the SD-RE-TI-012 data analysis to determine the evaluation-basis concrete compressive strength of 3,200 lbf/in^2 that was used in the SD-RE-TI-012 evaluations. Some tanks had experienced temperatures in excess of 300 °F up to 600 °F (see Table 2). The use of the 3,200 lbf/in^2 compressive strength value is not justified at these higher temperatures. Although most design Codes do not take advantage typically of the expected increase in concrete strength beyond the 28-day specified strength, concrete does increase in strength with age. As most of the SSTs were not exposed to high temperature waste immediately after construction, there was sufficient time to achieve near maximum aged-enhanced strength. Exposure to high temperatures would then degrade the strength and modulus from the aged strength condition. However, without the confirming concrete core test results the conservative approach is to not include any aged-strength increase and degrade the concrete strength from its 28-day specified strength.

Retrieval activities associated with the single-shell waste storage tanks located at the Hanford Site may introduce additional loading on the tanks. Some of these tanks have been exposed to high-heat generating sources from the stored radioactive waste material. At least five of the single-shell tanks were exposed to temperatures in excess of 350 °F (see Table 2). In addition to high temperature exposure, excessive thermal transients associated with fill/drain cycles (heat-up/cool-down rates) may have contributed to a degradation of the concrete material strength properties. All of the single-shell tanks are well beyond their original design life and as many as 67 of the 149 single-shell tanks have or are believed to have leaked. Exposure of the concrete to the waste leakage may have degraded the concrete strength and the reinforcement bond strength. Thus, there is a general concern about maintaining the structural stability of these tanks under their soil overburden load and any additional loading associated with retrieval activities.

This concern is discussed in the WHC-SD-TWR-RPT-002 (1996) report which provides a summary overview of the structural integrity and potential failure modes of the single- and double-shell, underground, waste storage tanks at Hanford. The report addresses the effects of design and actual operating loads based on existing analyses, as well as, postulated and beyond-design-basis loads based on results from the Delphi expert panel which were reported in WHC-SD-TWR-RPT-003 (1996).

The Delphi report presents the results of a two-day meeting of experts to predict failure modes and radiation release quantities in support of the safety analysis effort for the Hanford Site underground waste storage tanks. Dome collapse within the constraints of expected loading was not considered a likely failure mode based on existing analyses and results from the 241-A-105 1:10-scale model test.

In the 1:10-scale model test, dome collapse was estimated to occur at an equivalent uniform external dome pressure of 5,400 lbf/ft^2 (see review of ARH-R-47, 1969 in Appendix A under Concrete Properties and Test Program). This is equivalent to a soil overburden height relative to the dome apex of 41.5 and 47.4 feet for a soil unit weight of 125 and 110 lbf/ft^3, respectively. The ARH-R-47 equation for the equivalent uniform soil pressure was applied in the above to obtain the equivalent soil heights. Analytical simulations of the 1:10-scale model test predicted a failure pressure of 4,100 (ARH-R-120, 1972), 3,900 (WHC-SD-W320-ANAL-001, 1995), and 4,644 lbf/ft^2 (Report No. 941101-001, 1994) using the finite-element computer programs ASOLID®, ABAQUS® (with ANACAP-U® concrete constitutive model), and ADINA®, respectively. The corresponding predicted soil height at failure would be 31.0, 29.5, and 35.4 feet, respectively for a soil unit weight of 125 lbf/ft^3. Hence, the analytical models appear to conservatively under predict the test results. This might be due to a higher concrete compressive strength in the model test than assumed in the analytical models.

Thermal-creep ultimate-load analyses of the 100-Series SSTs using the SAFE-CRACK® computer program predicted a minimum collapse-load soil height of 20 feet for an assumed soil unit weight of 115 lbf/ft^3 (SD-RE-TI-012, 1983). However, in SD-RE-TI-012 the *total* soil load above the dome, including soil between the dome outer surface and horizontal plane at the dome apex and an initial nominal 7 feet of soil overburden above the dome apex, was factored to obtain the collapse-load soil height. This results in an increasing parabolic soil pressure load on the dome with increasing load factor since the soil between the dome outer surface and horizontal plane at the dome apex is factored also. An equivalent uniform soil height, h_{eq}, above the dome apex can be determined with the aid of ARH-R-47 [see review of ARH-R-47 (1969) in Appendix A under Concrete Properties and Test Program] by equating the equivalent uniform pressure from the factored total soil load to the equivalent uniform pressure obtained by increasing the soil height uniformly, i.e.,

$$\alpha \gamma_{115} (h_i + 188/110) = \gamma (h_{eq} + 188/110)$$

where

α = 20 ft / 7 ft = 2.86 (load factor at predicted onset to collapse based on soil overburden of 7 feet with soil unit weight of 115 lbf/ft^3)
γ_{115} = 115 lbf/ft^3 (soil unit weight used in analysis)
h_i = 7 feet (initial soil height above dome apex used in analysis)
γ = actual unit weight of soil
h_{eq} = equivalent uniform soil height (ft) above dome apex.

Solving for h_{eq} gives an equivalent minimum collapse-load soil height of 23.2 and 21.2 feet at a soil unit weight of 115 and 125 lbf/ft^3, respectively.

The recommended *allowable* maximum soil height for all 100-Series SSTs in SD-RE-TI-012 (1983) was 10 feet for an assumed soil unit weight of 115 lbf/ft^3. This was based on simplified analyses of the SST footings. The corresponding allowable soil height at the 125 lbf/ft^3 bounding soil unit weight would be 9.2 feet. However, there are a number of uncertainties associated with the analyses reported in

SD-RE-TI-012 that make it difficult to ascertain whether the net result is conservative or unconservative (see review of SD-RE-TI-012 in Appendix A).

Tank failure from concentrated loads associated with large equipment such as crawler-mounted cranes is a function of total weight, foot print at soil surface, position relative to tank center, and distribution of load over foot print during lift operations. Such concentrated loads have been idealized as loads applied uniformly on the soil surface at the tank center over a 20-foot diameter area. In the Delphi report, tank dome failure for such concentrated loads is estimated at from 300 to 600 tons for the 100-Series SSTs. These failure loads are much greater than the authorized concentrated load of 100 tons for these tanks and exceed the weight of the largest crane used on Site. However, it must be emphasized that these are *estimated failure* loads. These failure loads must be guarded against by providing an adequate safety margin consistent with national codes and standards to protect workers and the public from resulting consequences. Detailed analysis would be required to qualify the SSTs for loads in excess of the limits given in Section 3.4 above. Hence, the equivalent total applied load acting on the dome must be controlled to stay within the design basis analyses of record. In particular, concentrated loads due to cranes and equipment can result in greater induced stresses than uniform loads of the same total weight, hence concentrated loads must be controlled separately as required in HNF-IP-1266.

Despite these shortcomings, the load limits given in Section 3.4 are considered adequate for the SSTs that have not exceeded these limits in the past. In particular, these load limits are adequate for the proposed lead-transfer feed tanks 241-C-102 and -104 for the Phase 1 WFD activity. These tanks are bounded by the analyses of the 241-C-106 tank, which includes recent finite element evaluations for the bounding 241-C-106 thermal load history and soil-structure interaction analysis for a 0.20-g earthquake based on HPS-SDC 4.1 (1993) requirements.

5.0 RESOLUTION OF TECHNICAL BASELINE ISSUES

Despite the perceived robustness of the SSTs, errors, omissions, and combinations of conservative and unconservative assumptions have been identified in the available structural technical baseline documents (see Appendix A document review). These uncertainties make it difficult to accurately determine the true performance margin of the SSTs, particularly when attempting to justify the acceptability of additional loading on the tanks. This has led to the restriction on adding any additional soil overburden to the existing SSTs (HNF-SD-WM-TSR-006). Though adequate controls bound safety analysis hazards, additional conservative controls were imposed on dome loading to minimize the risk of structural damage and potential environmental releases. The inability to observe the condition of and to make structural repairs on these tank structures, either internally or externally, necessitates the use of recognized design codes to limit operating loads to assure an adequate safety margin. Plans for resolution of the SST technical baseline issues have been developed and submitted as part of the multi-year work planning activity under Technical Basis Review (TRB), 190.S48, *Resolve SST Dome Loading Technical Baseline Issue*, dated November 17, 1998. However, priority constraints have postponed start of work on this activity, thus far.

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Figure 1. 100- and 200-Series Single-Shell Tank Configurations.

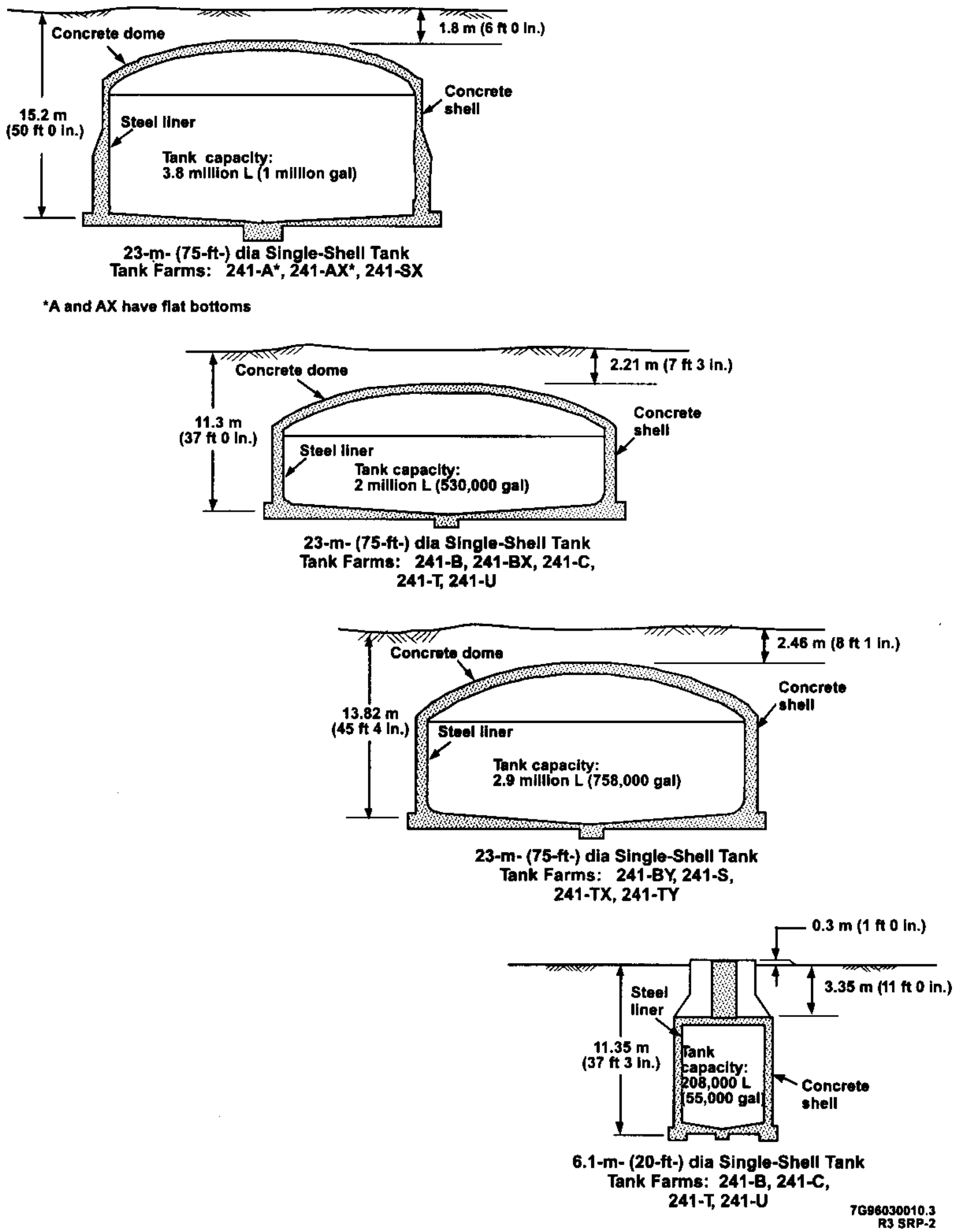

Table 1. Summary of 100-Series (75-ft Internal Diameter) Single-Shell Tank Construction Materials and Design Specifications.

Tank Farm	Storage Volume (Kgal)	Constructed	Hanford Construction Specification	Steel Liner (no structural tie between concrete and steel liner and no post-weld stress relief)[1]									Concrete Specification and 28-day Compressive Strength (ksi)				Reinforcing Steel	Seismic Criteria[3]
				Steel Design Code	Tank Steel ASTM Spec. (Repl. ASTM) Yield Strength (ksi)	Wall Thickness (in.)				Ref. Drawing	Weld Specification						Rebar & Cross-ties (ASTM) Yield Strength (ksi)	
						Bottom	Bottom Knuckle	Vertical Wall			Proc.	Qual.	Specification	Dome[2]	Wall	FDN		
B,C T,U	530	1943-44	HWS-1946	AWWA	A7-39 (A283, Gr. D) $F_Y = 33$	1/4	5/16 (4-ft radius)	1/4	Circumferentially stiffened by welding of 5 x 3-1/2 x 5/16-in. angle (ASTM A7, replacement ASTM A36) rings to inner surface of liner vertical wall	W 71387	AWS	AWS	PCA ST-58 & 57	3	3	3	A15-39 F_Y=40	UNSP
BX	530	1946-47	NRA	AWWA	A7-39 (A283, Gr. D) $F_Y = 33$	3/8	5/16 (4-ft radius)	1/4		H-2-602	AWS	AWS	PCA ST-55 & 57	3	3	3	A15-39 F_Y=40	NRA
TX	758	1947-48	HW-3061	AWWA	A285-46 Grade Unknown F_Y=24-30	3/8	3/8 (4-ft radius)	5/16 (bot. 6 ft) ¼ (top 9 ft)		H-2-809	AWS	AWS	PCA ST-55 & 57	3	3	3	A15-39 F_Y=40	NRA
BY	758	1948-49	HW-3783	ASME Sect. VIII 1946	A283-46T or A285-46 Grade A,B, or C F_Y=24-30					H-2-1313	ASME Sect. VIII	ASME Sect. IX	PCA ST-55 & 57	3	3	3	A15-39 F_Y=40 or A16-35 F_Y=33	NRA
S	758	1950-51	HW-3937	NRA	A283-46T Grade B F_Y=27					H-2-1784	ASME Sect. VIII	ASME Sect. IX	PCA ST-55 & 57	3	3	3	A15-39 F_Y=40	NRA
TY	758	1951-52	HW-4696 HW-4585	ASME Sect. VIII 1946	A283-49T Grade B F_Y=27					H-2-2245	ASME Sect. VIII	HW-4595 ASME Sect. IX	PCA ST-55 & 57	3	3	3	A15-39 F_Y=40	NRA
SX	1,000	1953-54	HW-4957	NRA	A283-52T Grade A or B F_Y=24-27	3/8	None (90°)	3/8		H-2-39511	HW-4925-S	HW-4900-S	PCA ST-55 & 57	3	3	3	A15-50T F_Y=40	NRA
A	1,000	1954-55	HWS-5614 HWS-4799-S	ASME	A283-52T or A285-52aT Grade B or C F_Y=27-30					H-2-55911	HW-4924-S HW-4926-S	HW-4900-S	PCA ST-55 & 57	3	3	3	A15-50T F_Y=40	NRA
AX	1,000	1963-64	HWS-4798-S HWS-8237	ASME Sect. VIII 1962	A201-61T, Gr. A (A515 Gr. 55) $F_Y = 30$	3/8	3/8 (4- to 8-in. radius)	3/8		H-2-44562	ASME Sect. VIII	ASME Sect. IX	PCA ST-55 & 57 ACI 318-56	4	4	4	A15-58T F_Y=40	UBC (0.25g H 0.167g V)

ACI = American Concrete Institute
ASME = American Society of Mechanical Engineers
ASTM = American Society for Testing and Materials
AWS = American Welding Society
AWWA = American Water Works Association
FDN = Foundation basemat
HWS = Hanford Works Specification
NRA = not readily available
PCA = Portland Cement Association
UBC = Uniform Building Code
UNSP = Unspecified
H = Horizontal
V = Vertical

[1] One-inch thick layer of cement motor reinforced with 2x2-in. wire mesh fabric over 3-ply asphaltic membrane waterproofing is provided between steel liner and concrete wall of all 100-Series SSTs except SX, A, and AX. Two-inch thick grout layer reinforced with 2x2-in. wire mesh fabric over 3/8-in. thick 3-ply asphaltic membrane waterproofing is provided between steel liner and concrete foundation of all 100-Series SSTs except AX which has drain slots provided in foundation under liner.

[2] A 3/4-inch layer of cement motor reinforced with 2x2-in. wire mesh fabric over 3/8-in. thick 3-ply asphaltic membrane waterproofing was applied on outer surface of concrete dome and haunch region of all 100-Series SSTs except AX.

[3] HW-37519 (1955) states that the tanks meet the earthquake requirements of the UBC and can be considered earthquake resistant but no supporting details or references were provided therein.

Table 2. 100-Series (75-ft Internal Diameter) Single-Shell Tank Data Summary.

Tank Farm	Tank	Storage Volume (Kgal)	Constructed	In Service	Bottom Shape	Design for Self-Boiling Waste	Current Ventilation	Each Tank: Airlift Circulators	Each Tank: Lateral Wells Under Tank	Each Tank: Drainage Slots Beneath Steel Liner	External Dry Wells	Out of Service	Integrity Category	Temperature (°F): Historic Peak (date)	Temperature (°F): Current (as of 1994)	Total Waste Volume (Kgal)	Watch List Status	Stabilization / Isolation
B	101	530	1943-44	1945-46	Dish	No	Passive	0	0	No	5	1974	ASMD LKR	137 (77)	109	113	-	IS/IP
	102										6	1978	Sound	108 (89)	63	32		
	103										3	1977	ASMD LKR	83 (76)	61	59		
	104										2	1972	Sound	122 (89)	65	371		
	105										1	1977	ASMD LKR	107 (89)	65	306		
	106										4	1977	Sound	86 (82)	63	117		
	107										5	1969	ASMD LKR	124 (89)	60	165		
	108										5	1977	Sound	105 (89)	64	94		
	109										5	1977	Sound	105 (89)	63	127		
	110										4	1971	ASMD LKR	121 (89)	63	246		
	111										1	1976	ASMD LKR	98 (79)	86	237		
	112										5	1977	ASMD LKR	101 (89)	64	33		
C	101	530	1943-44	1946-47	Dish	No	Passive	0	0	No	4	1970	ASMD LKR	112 (80)	66	88	-	IS/IP
	102										0	1976	Sound	106 (78)	96	316	Organic Salts	
	103										5	1979	Sound	168 (77)	120	202		/PI
	104										7	1980	Sound	195 (82)[1]	85	295	-	IS/IP
	105						Active				4	1979	Sound	156 (76)[1]	80	134		IS/PI
	106										6	1979	Sound	216 (94)[1]	153	229	High Heat	/PI
	107						Passive				7	1978	Sound	168 (88)[1]	124	237	-	IS/IP
	108										8	1976	Sound	99 (80)[1]	77	66		
	109										6	1976	Sound	160 (63)	76	66		
	110										4	1976	ASMD LKR	118 (85)[1]	66	178		
	111										5	1978	ASMD LKR	190 (64)	72	57		
	112										4	1978	Sound	160 (61)	79	104		

[1] WHC-SD-WM-ER-313, 1996

Table 2. 100-Series (75-ft Internal Diameter) Single-Shell Tank Data Summary (Continued).

Tank Farm	Tank	Storage Volume (Kgal)	Constructed	In Service	Bottom Shape	Design for Self-Boiling Waste	Current Ventilation	Each Tank: Airlift Circulators	Each Tank: Lateral Wells Under Tank	Each Tank: Drainage Slots Beneath Steel Liner	External Dry Wells	Out of Service	Integrity Category	Temperature (°F): Historic Peak (date)	Temperature (°F): Current (as of 1994)	Total Waste Volume (Kgal)	Watch List Status	Stabilization / Isolation
T	101	530	1943-44	1945-46	Dish	No	Passive	0	0	No	5	1979	ASMD LKR	103 (88)	72	102	-	IS/PI
	102										6	1976	Sound	94 (76)	68	32		IS/IP
	103										6	1974	ASMD LKR	96 (76)	62	27		
	104										5	1974	Sound	90 (78)	62	343		/PI
	105										3	1976	Sound	93 (85)	O/S	98		IS/IP
	106										9	1973	ASMD LKR	93 (79)	60	21		
	107										3	1976	ASMD LKR	114 (81)	69	173		IS/PI
	108										6	1974	ASMD LKR	90 (78)	57	44		IS/IP
	109										6	1974	ASMD LKR	91 (78)	O/S(75)	58		
	110										3	1976	Sound	91 (76)	63	369	Hydrogen	/PI
	111										5	1974	ASMD LKR	98 (81)	63	446	-	IS/PI
	112										3	1977	Sound	87 (78)	60	67		IS/IP
U	101	530	1943-44	1946	Dish	No	Passive	0	0	No	2	1960	ASMD LKR	92 (77)	67	25	-	IS/IP
	102			1946							6	1979	Sound	134 (78)	85	375		/PI
	103			1947							5	1978	Sound	132 (77)	87	468	Hydrogen	
	104			1947							4	1951	ASMD LKR	78 (76)	O/S	122	-	IS/IP
	105			1947							5	1978	Sound	146 (77)	90	418	Hydrogen	/PI
	106			1948							4	1977	Sound	122 (76)	81	226	-	
	107			1948							4	1980	Sound	122 (76)	78	406	Hydrogen	
	108			1949							4	1979	Sound	130 (80)	87	468		
	109			1949							4	1978	Sound	120 (77)	85	463		
	110			1946							5	1975	ASMD LKR	140 (56)	76	186	-	IS/PI
	111			1947							5	1980	Sound	130 (56)	80	329		/PI
	112			1947							5	1970	ASMD LKR	160* (56)	63	49		IS/IP
BX	101	530	1946-47	1948	Dish	No	Passive	0	0	No	2	1972	ASMD LKR	86 (77)	O/S(74)	43	-	IS/IP/CCS
	102			1948							6	1971	ASMD LKR	83 (77)	66	96		
	103			1948							5	1977	Sound	99 (79)	O/S(77)	68		
	104			1949							6	1980	Sound	124 (77)	O/S(87)	99		
	105			1949							6	1980	Sound	126 (77)	66	51		
	106			1949							4	1971	Sound	115 (74)	67	38		
	107			1948							2	1977	Sound	88 (77)	O/S(69)	345		
	108			1949							7	1974	ASMD LKR	90 (80)	65	26		
	109			1950							4	1974	Sound	77 (93)	O/S(77)	193		
	110			1949							5	1977	ASMD LKR	104 (74)	74	207		
	111			1950							6	1977	ASMD LKR	111 (77)	69	162		
	112			1950							5	1977	Sound	90 (80)	65	165		

Table 2. 100-Series (75-ft Internal Diameter) Single-Shell Tank Data Summary (Continued).

Tank Farm	Tank	Storage Volume (Kgal)	Constructed	In Service	Bottom Shape	Design for Self-Boiling Waste	Current Ventilation	Each Tank: Airlift Circulators	Each Tank: Lateral Wells Under Tank	Each Tank: Drainage Slots Beneath Steel Liner	External Dry Wells	Out of Service	Integrity Category	Temperature (°F): Historic Peak (date)	Temperature (°F): Current (as of 1994)	Total Waste Volume (Kgal)	Watch List Status	Stabilization / Isolation
TX	101	758	1947-48	1949	Dish	No	Passive	0	0	No	5	1980	Sound	128 (8/82)	O/S	87	-	IS/IP/CCS
	102			1950							5	1977	Sound	136 (4/76)	O/S	217		
	103			1950							6	1980	Sound	112 (12/76)	71	157		
	104			1950							6	1977	Sound	128 (1/77)	65	65		
	105			1951							6	1977	ASMD LKR	238 (1/52)	104	609		
	106			1951							5	1977	Sound	138 (11/75)	78	453		
	107			1950							7	1977	ASMD LKR	110 (12/76)	66	36		
	108			1950							3	1977	Sound	116 (3/77)	68	134		
	109			1950							5	1977	Sound	168 (11/75)	94	384		
	110			1950							6	1977	ASMD LKR	153 (11/77)	O/S	462		
	111			1949							5	1977	Sound	147 (11/75)	79	370		
	112			1950							6	1974	Sound	112 (4/76)[2]	67	649		
	113			1950							3	1971	ASMD LKR	120 (3/77)	72	607		
	114			1951							3	1971	ASMD LKR	102 (12/76)	O/S	535		
	115			1951							6	1977	ASMD LKR	123 (8/76)	70	640		
	116			1951							3	1969	ASMD LKR	117 (11/75)	O/S	631		
	117			1951							6	1969	ASMD LKR	191*(11/87)	O/S	626		
	118			1951							7	1980	Sound	118 (1/76)[2]	77	347		
BY	101	758	1948-49	1950	Dish	No	Passive	0	0	No	5	1971	Sound	115 (11/74)	75	387	-	IS/IP
	102			1950							5	1977	Sound	155 (11/74)	O/S(72)	277		IS/PI
	103			1950							8	1973	ASMD LKR	137 (7/79)	82	414		
	104			1951							5	1977	Sound	237 (1/75)	128	406		IS/IP
	105			1951							3	1974	ASMD LKR	180 (1/75)	120	504		/PI
	106			1950							5	1977	ASMD LKR	199 (1/75)	129	562		
	107			1950							6	1974	ASMD LKR	125 (1/75)	94	266		IS/IP
	108			1951							7	1972	ASMD LKR	154 (12/76)	110	228		
	109			1950							6	1979	Sound	138 (6/76)	OS	290		IS/PI
	110			1951							4	1979	Sound	205 (4/75)	118	398		IS/IP
	111			1951							4	1977	Sound	124 (11/75)	88	459		
	112			1951							7	1978	Sound	163 (11/75)	90	291		

[2] HNF-SD-WM-ER-321, 1997

Table 2. 100-Series (75-ft Internal Diameter) Single-Shell Tank Data Summary (Continued).

Tank Farm	Tank	Storage Volume (Kgal)	Constructed	In Service	Bottom Shape	Design for Self-Boiling Waste	Current Ventilation	Each Tank: Airlift Circulators	Each Tank: Lateral Wells Under Tank	Each Tank: Drainage Slots Beneath Steel Liner	External Dry Wells	Out of Service	Integrity Category	Temperature (°F): Historic Peak (date)	Temperature (°F): Current (as of 1994)	Total Waste Volume (Kgal)	Watch List Status	Stabilization / Isolation
S	101	758	1950-51	1953	Dish	No	Passive	0	0	No	5	1980	Sound	300 (11/53)	101	427	-	/PI
	102										8	1980	Sound	140 (9/79)	108	549		
	103										7	1980	Sound	130 (9/79)	87	248		
	104										4	1968	ASMD LKR	300 (7/53)	108	294		IS/IP
	105										5	1974	Sound	125 (1/80)	78	456		
	106										6	1979	Sound	144 (2/76)	81	479		/PI
	107			1952							6	1980	Sound	240 (11/52)	110	376		
	108										6	1979	Sound	195 (1/82)	89	450		IS/PI
	109										5	1979	Sound	150 (10/74)	68	507		/PI
	110										7	1979	Sound	240 (10/52)	117	390		IS/PI
	111										5	1972	Sound	169 (2/76)	93	540		/PI
	112										5	1974	Sound	141 (1/78)	88	523		
TY	101	758	1951-52	1953	Dish	No	Passive	0	0	No	3	1973	ASMD LKR	83 (12/76)	66	118	-	IS/IP/CCS
	102										5	1979	Sound	82 (8/77)	60	64		
	103										3	1976	ASMD LKR	86 (8/77)	69	162		
	104										5	1974	ASMD LKR	114 (8/76)	66	46		
	105										1	1960	ASMD LKR	112 (8/76)	79	231		
	106										5	1977	ASMD LKR	106 (12/76)	59	17		

Table 2. 100-Series (75-ft Internal Diameter) Single-Shell Tank Data Summary (Continued).

Tank Farm	Tank	Storage Volume (Kgal)	Constructed	In Service	Bottom Shape	Design for Self-Boiling Waste	Current Ventilation	Each Tank: Airlift Circulators	Each Tank: Lateral Wells Under Tank	Each Tank: Drainage Slots Beneath Steel Liner	External Dry Wells	Out of Service	Integrity Category	Temperature (°F): Historic Peak (date)	Temperature (°F): Current (as of 1994)	Total Waste Volume (Kgal)	Watch List Status	Stabilization / Isolation
SX	101	1,000	1953-54	1954	Dish	Yes	Active	0	3**	No	7	1980	Sound	320 (11/57)	139	442	Hydrogen	/PI
	102			1954							3	1980	Sound	212 (8/85)	153	543		
	103			1954							6	1980	Sound	225 (8/85)	177	651		
	104			1955							6	1980	ASMD LKR	300 (12/56)	170	584		
	105			1955							7	1980	Sound	330 (11/75)	176	683		
	106			1954							6	1980	Sound	195 (10/63)	112	538		
	107			1956							7	1964	ASMD LKR	390 (2/58)	174	104	-	IS/IP
	108			1955							6	1962	ASMD LKR	320 (9/58)	201	87		
	109			1955							7	1965	ASMD LKR	295 (9/62)	153	244	Hydrogen	
	110			1960							8	1976	ASMD LKR	310 (5/66)	177	62	-	
	111			1956							7	1974	ASMD LKR	320 (11/65)	195	125		
	112			1956							7	1969	ASMD LKR	315 (3/62)	162	92		
	113			1958			Passive				1	1958	ASMD LKR	255 (7/58)	77	26		
	114			1956			Active				7	1972	ASMD LKR	335 (8/58)	191	181		
	115			1958			Passive				6	1965	ASMD LKR	260 (9/60)	O/S	12		
A	101	1,000	1954-55	1956	Flat	Yes	Exhauster N/O	4	3**	No	10	1980	Sound	399 (5/61)	153	953	Hydrogen	/PI
	102			1956							7	1980	Sound	420 (8/61)	87	41	-	IS/PI
	103			1956			Exhauster O/S				7	1980	ASMD LKR	300 (6/60)	114	371		IS/IP
	104			1959							7	1975	ASMD LKR	430 (2/63)	191	28		
	105			1962							7	1963	ASMD LKR	325 (3/63)	140	19		
	106			1957							7	1980	Sound	594 (5/63)	135	125		
AX	101	1,000	1963-64	1965	Flat	Yes	Passive	22	0	Yes	8	1980	Sound	260 (6/71)	136	748	Hydrogen	/PI
	102			1966							10	1980	ASMD LKR	250 (12/70)	71	39	-	IS/IP
	103			1965							6	1980	Sound	330 (5/66)	117	112	Hydrogen	
	104			1966							7	1974	ASMD LKR	320 (5/70)	92	7	-	

xxx Current high heat load tank (> 40,000 Btu/h)

References WHC-SD-WM-TI-648 (1994) and HNF-EP-0182-129, December 31, 1998 and WHC-SD-WM-TI-591 (1994) for historic peak temperature data except as noted.
Anticipated design life of the SSTs was 20-25 years (WHC-SD-WM-TI-648).

ASMD LKR – Assumed leaker.
PI – Partial interim isolated (completion of the physical effort required for interim isolation except for isolation of risers and piping that is required for jet pumping or other methods of stabilization).
IP – Intrusion prevention completed (completion of physical effort required to minimize the addition of liquids).
IS – Interim stabilized (removal of supernatant and interstitial liquid).
CCS – Controlled, clean, and stable (provide remote monitoring for required instrumentation, remove surface soil contamination and abandoned equipment, remove pumpable liquids, and isolate tank.
N/O – Not currently operating. O/S – Out of service. * Suspect temperature data.
** There are currently no functioning laterals and no plan to prepare them for use (HNF-EP-0182-129, December 31, 1998).

Table 3. 200-Series (20-ft Internal Diameter) Single-Shell Tank Data Summary.

								Each Tank						Temperature (°F)				
Tank Farm	Tank	Storage Volume (Kgal)	Constructed	In Service	Bottom Shape	Design for Self-Boiling Waste	Current Ventilation	Airlift Circulators	Lateral Wells Under Tank	Drainage Slots Beneath Steel Liner	External Dry Wells	Out of Service	Integrity Category	Historic Peak (date)	Current (as of 1994)	Total Waste Volume (Kgal)	Watch List Status	Stabilization / Isolation
B	201	55	1943-44	1952	Dish	No	Passive	0	0	No	0	1971	ASMD LKR	112 (7/89)[1]	60	29	-	IS/IP
	202											1977	Sound	74 (5/75)[1]	60	27		
	203												ASMD LKR	110(7/89)[1]	61	51		
	204												ASMD LKR	220(7/89)[1]	61	50		
C	201			1947-48								1977	ASMD LKR	81 (2/78)[2]	56	2		
	202												ASMD LKR	80 (2/78)[2]	60	1		
	203												ASMD LKR	83 (2/78)[2]	59	5		
	204												ASMD LKR	-	O/S	3		
T	201			1952								1976	Sound	81 (10/76)[3]	60	29		
	202												Sound	73 (10/94)[3]	62	21		
	203												Sound	79 (7/88)[3]	64	35		
	204												Sound	77 (10/76)[3]	63	38		
U	201			1956								1977	Sound	78 (1/77)[4]	61	5		
	202												Sound	67 (9/95)[4]	61	5		
	203												Sound	82 (2/77)[4]	60	3		
	204			1954									Sound	77 (2/77)[4]	65	3		

[1] WHC-SD-WM-ER-310, 1997
[2] WHC-SD-WM-ER-313, 1996
[2] HNF-SD-WM-ER-320, 1997
[2] HNF-SD-WM-ER-325, 1997

Reference WHC-SD-WM-TI-648 (1994) and HNF-EP-0182-129, December 31, 1998.
Anticipated design life of the SSTs was 20-25 years. (WHC-SD-WM-TI-648).
Specified 28-day concrete compressive strength was 2,500 lbf/in^2 for all 200-Series SSTs (Drawing HW-72417).
Specified wall thickness for steel liner was ¼ inch (Drawing HW-72417).

ASMD LKR – Assumed leaker.
IP – Intrusion prevention completed (completion of physical effort required to minimize the addition of liquids).
IS – Interim stabilized (removal of supernatant and interstitial liquid).
O/S – Out of service.

Table 4. Single-Shell Tank Structural Requirements Summary.

Title/Description	Requirement				Basis	Reference	Comment/Adequacy
Tank Content Composition	None				Corrosion control for leak tightness of SST steel liner	OSD-T-151-00013 (Sect. 13.2.1.A)	Not technically feasible to adjust composition of waste within the in-active (no waste addition allowed) SSTs. Past history of waste composition is important in assessing current and future leak tightness of SSTs.
Waste Level	Tanks	Capacity (Kgal)	Max. Depth (in.)		Overflow control (limited by the location at which the side-fill process lines enter the tank).	OSD-T- 151-00013 (Sect. 13.2.1.B) WHC-SD-WM-OSR-005 (Sect. 2.1) (cancelled, replaced by HNF-SD-WM-TSR-006) SD-RE-TI-035, Rev. 1 (1985) (pg. 12)	TSR-006 does not impose any restriction since no waste addition is allowed in SSTs. However, waste retrieval options may require water or diluted waste additions. The maximum allowable depths assume that the tank is not a leaker and that the bulk specific gravity of the waste is within hydrostatic head limits given below.
			OSD	OSR			
	A, AX, SX	1,000	360	365			
	BY, S, TX, TY	758	275	281			
	B. BX, C, T, U	530	185	189			
	B, C, T, U (200 Series)	55	280	285			

Table 4. Single-Shell Tank Structural Requirements Summary (Continued).

Title/Description	Requirement: Tanks	Maximum Waste Specific Gravity at Full Tank Capacity: Original Design SpG	SD-RE-TI-035: SpG	SD-RE-TI-035: Vapor Pressure (psig)	SD-RE-TI-012 SpG	Basis	Reference	Comment/Adequacy
Hydrostatic Head	A, AX	2	2	6.9		Although structural collapse due to the hydrostatic head is difficult to conceive due to the lateral soil restraint, exceeding the limiting values could lead to leakage through wide cracks in the concrete if steel liner has breached.	SD-RE-TI-035, Rev. 1 (1985) (pg. 12) based on HW-37519 (1955) which applied simplified hand calculations in accordance with Portland Cement Association ST-57 but with increased allowable working stresses than normally considered at design stage. RHO-R-6 (1977) (for AX) SD-RE-TI-012 (1983) Based on simplified hand and detailed finite element calculations.	SD-RE-TI-035 values are based on HW-37519 that allowed the reinforcing steel to approach 20 ksi for sustained hydrostatic presure and 27 ksi when transient vapor pressures were included. This compares to normal design stress allowable of 14 ksi. A through wall thermal gradient of 23 °F/ft of wall thickness was assumed which required that the heat-up rate not exceed *2 °F/day*. Reductions or elimination of the positive vapor pressure will not increase the maximum allowable SpG listed under SD-RE-TI-035 because the SpG is controlled by the 20 ksi allowable and SpG plus vapor pressure is controlled by 27 ksi allowable.
	SX	1.35	1.5	4.8				
	BY, S, TX, TY	1.25	1.2	1.8				
	B, BX, C, T, U	1.2	1.9	2.5	2			
	B, C, T, U (200 Series)		1.9					

The tank loading limits recommended in HW-37519 (1955) were based on allowable working stresses considerably above those recommended by the Portland Cement Association (PCA) and the American Concrete Institute (ACI) in an attempt to achieve maximum utilization of the existing waste storage tanks. In HW-59919 (1959) safe load curves were established from stress analyses conducted in accordance with PCA Bulletins ST-32 and -57 and the ACI-318-56 design codes. However, the reinforcing steel allowable stress in ring tension was increased from the 10 to 12 ksi typically used in cylindrical concrete storage tanks to 16 ksi, which is less than the 20 ksi assumed in HW-37519. In both HW-37519 and HW-59919, a maximum through wall thermal gradient of 23 °F/ft of wall thickness was assumed which required that the average heat-up rate not exceed *2 °F/day*. In addition, the maximum incremental increase in temperature should not exceed 40 °F at any one time, although this increment could be fast (HW-59919). A minimum time interval of four weeks is required to permit the outer temperature of the wall to approach the temperature of the inner surface for this maximum increment in temperature. For the self-boiling tanks the resulting transient vapor pressure was equated to an equivalent static pressure with a dynamic load factor of one (slow vapor pressure loading) and was assumed to be 15 to 30 percent of the hydrostatic head of the contained waste. The results from HW-59919 were more restrictive than obtained from HW-37519. As an example of the results from the safe load curves for the maximum expected waste specific gravity and vapor pressure, HW-59919 gave the allowable liquid levels shown in Appendix A, Table A-5, which are significantly less than the full capacity liquid levels.

In 1965 these requirements were again reassessed in RL-UPO-12 in support of the waste solidification program. The waste solidification program involved operation of tank storage at higher temperatures and higher liquid specific gravities than had been required in the waste management program up to that time. RL-UPO-12 applied the same analysis procedure used in HW-59919 (16 ksi allowable working stress for reinforcement steel) but assumed that the vapor pressure above the liquid was no greater than atmospheric. Vapor pressures greater than atmospheric require additional reductions in the allowable liquid depths. In addition, the effect of the through wall thermal gradient under steady state storage (10 °F/ft) and transient fill (20 °F/ft) conditions was considered. The results are summarized below in Appendix A, Table A-6. Clearly these older restrictions are more restrictive than either SD-RE-TI-035 (1985) or SD-RE-TI-012 (1983).

Table 4. Single-Shell Tank Structural Requirements Summary (Continued).

<table>
<tr><th colspan="2">Title/Description</th><th colspan="5">Requirement</th><th>Basis</th><th>Reference</th><th>Comment/Adequacy</th></tr>
<tr><td rowspan="15">Dome Loading</td><td rowspan="8">Soil and Concentrated Live Load</td><td colspan="2" rowspan="2">Tank Series</td><td colspan="3">Additional Allowable Load Above Baseline</td><td rowspan="4">Structural damage control (prevent structural failure, collapse)</td><td rowspan="4">HNF-SD-WM-TSR-006
HNF-SD-WM-BIO-001</td><td rowspan="4">TSR-006 requires establishment of dome loading program (HNF-IP-1266, Chapter 5.16) that manages soil and concentrated load above tanks. Baseline loading corresponds to existing soil height and net concentrated load in-place as of effective date of TSR.</td></tr>
<tr><td colspan="2">Soil (ft)</td><td>Concentrated (tons)</td></tr>
<tr><td colspan="2">100</td><td colspan="2">None</td><td>100</td></tr>
<tr><td colspan="2">200</td><td colspan="2">None</td><td>50</td></tr>
<tr><td rowspan="2">Tank Series</td><td rowspan="2">Dia. (ft)</td><td colspan="2">Soil Cover (ft)</td><td rowspan="2">Live Load (tons)</td><td rowspan="4">Radiation shielding (min. soil cover) and structural damage control (maintain design Code margins).</td><td rowspan="4">SD-RE-TI-012 (1983)
Allowable soil height is based on assumed soil density of 115 lb/ft^3.</td><td rowspan="4">Actual soil density range is 100 to 125 lb/ft^3. Total soil and concentrated (live) load is administratively controlled by HNF-IP-1266, Chapter 5.16.</td></tr>
<tr><td>Min.</td><td>Max.</td></tr>
<tr><td>100</td><td>75</td><td>5</td><td>10</td><td>100</td></tr>
<tr><td>200</td><td>20</td><td>5</td><td>12</td><td>50</td></tr>
<tr><td rowspan="3">Pressure</td><td colspan="2" rowspan="2">Tank Series</td><td colspan="3">Vapor Pressure (inches water gauge/psig)</td><td rowspan="3">Structural damage control (maintain design Code margins).</td><td rowspan="3">SD-RE-TI-035, Rev. 1 (1985)

The positive vapor pressure load leading to tank failure (collapse) was estimated at 10 psig (all SSTs) from HW-37519 and 11.6 (A, AX, and SX Tanks) and 14 psig (B. BX, C, T, and U Tanks) from WHC-SD-WM-TI-623, assuming a 7 ft soil overburden.</td><td rowspan="3">Total dome load administratively controlled by HNF-IP-1266, Chapter 5.16

The net resultant gauge pressure from waste and vapor pressure must be ≥ zero to prevent uplift of tank steel liner bottom in non-boiling tanks and > vaporization pressure from moisture present in 2-inch layer of grout between steel liner and the asphaltic membrane if the temperature of the waste exceeds boiling.</td></tr>
<tr><td colspan="2">Minimum</td><td>Maximum</td></tr>
<tr><td colspan="2">100</td><td colspan="2">-15/-0.54</td><td>130/4.7</td></tr>
<tr><td rowspan="4">Deflection</td><td colspan="2" rowspan="3">SST Tank Series</td><td rowspan="3">Dome Deflection Screening Limit (ft)</td><td colspan="2">Survey Frequency (months)</td><td rowspan="4">Deflection greater than limit may indicate excessive dome loading or possible structural failure.</td><td rowspan="2">OSD-T-151-00013
SD-RE-TI-035, Rev. 1</td><td rowspan="2">Limit provides a stop point requiring additional investigation.</td></tr>
<tr><td colspan="2">Tank with Airlift Circulators*</td></tr>
<tr><td>Yes</td><td>No</td><td colspan="2" rowspan="2">Salt cake encrustations on in-tank equipment can lead to additional unaccounted for dome loading particularly when not buoyed by the interstitial liquid removed during jet pumping. Additional surveying is required during jet pumping operations.</td></tr>
<tr><td colspan="2">100</td><td>0.02</td><td>12 ±1</td><td>24 ±2</td></tr>
</table>

*OSD-T-151-00013 lists the following 19 tanks that contain dome suspended airlift circulators:

241-AX-101	241-BY-101	241-BY-107	241-TX-102	241-TX-113
241-AX-102	241-BY-103	241-BY-108	241-TX-106	241-TX-114
241-AX-103	241-BY-105	241-BY-110	241-TX-110	241-TX-115
241-AX-104	241-BY-106	241-BY-111	241-TX-111	

Table 4. Single-Shell Tank Structural Requirements Summary (Continued).

Title/Description	Requirement			Basis	Reference	Comment/Adequacy
100-Series SST Temperature (see Table below also)	Maximum	OSD	OSR	Temperatures and temperature changes in excess of these limits may lead to reductions in strength and modulus and severe structural stresses.	OSD-T-151-00013 (Sect. 13.2.1.E) WHC-SD-WM-OSR-005 (Sect. 2.2) (cancelled, replaced by HNF-SD-WM-TSR-006) SD-RE-TI-012 (1983)	200-Series SSTs have not been evaluated for elevated temperatures because these tank typically are not exposed to high temperatures (see Table 3). The 20 °F/day maximum heat-up/cool-down is potentially too high and rates limited to 3 °F/day are preferable (WHC-SD-WM-OSR-005, pg. B-81). Current HNF-SD-WM-TSR-006 requirements limit the waste temperature of tanks C-106, SX-103, SX-107 through -112, and SX-114 to ≤ 250 °F (safety limit) or ≤ 205 °F (operating limit) to prevent salt-nitrate reactions and tank bumps for tanks containing sludge with estimated heat loads > 26,000 Btu/h.
	Waste Temp. (°F)	300	350			
	Dome Temp. (°F)	250				
	Waste Temp. Change (°F/day)	20				
	Through Wall Gradient (°F)					
	Meridional Gradient (°F/ft)					

SD-RE-TI-035 (1985) lists the following more restrictive temperature limits on 100-Series SSTs (see Table A-2 for supporting references):

Tank	Waste Maximum Temperature (°F)	Waste Temperature Change (°F/day)	Maximum Through Wall Gradient (°F)	Meridional Thermal Gradient (°F/ft)			
				Steady State		Transient	
				Along Dome	Along Wall	Along Dome	Along Wall
A, SX	300	2	10	29	14	44	21
AX	350	3					
BY	280	3.7 (up to 250 °F) 1.5 (above 250 °F)					
S, TX, TY	280	2					
B, BX, C, T, U	330	2					

Grayed values are identified as candidate limits.

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APPENDIX A

SUPPORTING DOCUMENT REVIEW
IN
CHRONOLOGICAL ORDER

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CONTENTS

LIST OF FIGURES

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HISTORICAL

HAN-10970, 1945, *Construction of Hanford Engineer Works: History of the Project*, E.I. du Pont de Nemours and Company, Wilmington, Delaware.

IN 6263, 1945, *Design and Procurement History of Hanford Engineer Works, Volumes I and II*, E.I. du Pont de Nemours and Company, Wilmington, Delaware.

HAN-73214, 1946, *History of the Operation of Hanford Engineer Works*, E. I. du Pont de Nemours and Company, Wilmington, Delaware.

ARH-CD-948, 1977, *History and Status of Tanks 241-C-105 and 241-C-106*, C. M. Walker, Atlantic Richfield Hanford Company, Richland, Washington.

WHC-SD-WM-TI-648, 1994, *Tank Characterization Reference Guide*, Rev. 0A, prepared by Los Alamos Technical Associates, Inc., Kennewick, Washington for Westinghouse Hanford Company, Richland, Washington.

This document provides a broad background of information relating to the characterization of the Hanford Site single-shell and double-shell waste storage tanks. This is an excellent summary document and includes general descriptions of all tank farms, process and waste generation histories, sampling and analytical methods, and regulatory, safety, and technology development driven characterization needs. However, the tank status summary data is no longer current. The reader should refer to the current monthly Waste Tank Summary Report (such as HNF-EP-0182-129) for a more current status of the tanks.

HNF-EP-0182-129, 1999, *Waste Tank Summary Report for Month Ending December 31, 1998*, Fluor Daniel Hanford, Inc., Richland, Washington.

This series of monthly reports is the source document on the general status of each of the Hanford Site large underground waste storage tanks with regards to waste inventories, space utilization, safety issues, anomalies, and ongoing investigations.

CODES AND STANDARDS

PCA Bulletin ST-32, 1953, *Effects of Long Exposure of Concrete to High Temperature*, Portland Cement Association, Chicago, Illinois.

PCA Bulletin ST-55, 1954, *Design of Circular Domes*, Portland Cement Association, Chicago, Illinois.

PCA Bulletin ST-57, 1954, *Circular Concrete Tanks without Prestressing*, Portland Cement Association, Chicago, Illinois.

ACI-318-56, 1956, *Building Code Requirements for Reinforced Concrete*, American Concrete Institute, Skokie, Illinois.

BNL-52361, 1995, *Seismic Design and Evaluation Guidelines for Department of Energy High-Level Waste Storage Tanks and Appurtenances*, K., Bandyopadhyay, et al., Brookhaven National Laboratory, Upton, New York.

BNL-52527, 1997, *Guidelines for Development of Structural Integrity Programs for DOE High-Level Waste Storage Tanks*, K. Bandyopadhyay, et al., Brookhaven National Laboratory, Upton, New York.

This document provides general guideline for the development of site-specific structural integrity programs for DOE high-level waste storage tanks. The structural integrity program should consist of:

- Definition of appropriate loads in accordance to applicable national codes and standards.
- Collection of data for possible material and geometry changes indicative of a loss in structural capacity (this includes both leak integrity and structural load carrying capacity).
- Performance of structural integrity assessment of the tank focusing on potential material degradation over time and assessment of consequences.

The most important elements of the structural integrity program include implementation of a leak detection system and performance of reliable non-destructive examinations (NDE). The desirability of controlling waste chemistry to minimize degradation of tank materials and of monitoring for corrosion-induced degradation is also stressed.

The document stresses the importance of assessing the actual condition of the concrete and steel elements of the tanks by direct examination were feasible. The effects of degradation in strength and modulus of concrete and enhanced creep with elevated temperature exposure need to be assessed. However, any gain in concrete strength through aging also needs to be considered so as to not unrealistically penalize the tank.

Although elements of the guidelines are applicable to SSTs, the scope of the document is focused on double-shell tanks (DSTs). Unlike the single-shell tanks, the DSTs are accessible, through the annulus space, to nondestructive examination of the primary tank and secondary steel liner. Nondestructive examination of SST liners is not feasible, except for remote visual examination when the liner is exposed for view as the waste level is lowered. The control of chemistry in SSTs is less practical because of the current restriction that prohibits the addition of material to the SSTs and because the stabilization effort results in the removal of the liquid portion of the waste. Although the removal of the liquid also prevents the monitoring of the liquid waste level as a means of detecting leaks, it does reduce the likelihood of a significant leak. The SSTs do have numerous dry well monitors for detecting leaks and the underside of the SST dome can be visually inspected remotely for signs of distress. In addition, changes in the dome elevation are currently monitored for signs of distress. However, dome elevation survey data can be masked by changing thermal conditions within the tank or the environment making the interpretation of the data difficult.

In addition to an extensive laboratory testing program (PNL-7779, 1988) to assess the properties of Hanford-mix concrete to elevated temperature, concrete cores have been removed from tank 241-SX-115 and tested (RHO-RE-CR-2, 1982) to determine in-situ properties. The long-term effects of waste solutions on concrete and reinforcing steel have also been investigated (RHO-RE-CR-8, 1982). These elements provide input to the development of a SST structural integrity program.

HPS-SDC 4.1, 1993, *Hanford Plant Standards, HPS-SDC 4.1, Rev. 12, Standard Arch-Civil Design Criteria Design Loads for Facilities*, U.S. Department of Energy-Richland Operations Office, Richland, Washington.

HNF-PRO-097, 1997, *Project Hanford Policy and Procedure System: Engineering Design and Evaluation*, Fluor Daniel Hanford, Inc., Richland, Washington.

This document supercedes HPS-SDC 4.1 (1993). It provides the current general and specific design requirements, including natural phenomena hazards, for Hanford Site facilities except as noted therein.

WAC, 1998, *Dangerous Waste Regulations, Chapter 173-303*, Washington State Department of Ecology, Olympia, Washington.

SPECIFICATIONS

HW-1946, 1943, *Specification for Composite Storage Tanks – Bldg. #241 at Hanford Engineer Works Project 9536,* Hanford Engineer Works, A Division of General Electric, Richland, Washington.

Construction specification for the 241-B, -C, -T, and U Tank Farms.

HW-70529, 1961, *Basis for Process Design Engineering PUREX Tank Farm – 241-AX*, by H. W. Stivers, Hanford Atomic Products Operation, General Electric Company, Richland, Washington.

This report documents the preliminary engineering basis for the 241-AX Tank Farm. The requirements are summarized below but do not necessarily represent the final requirements.

Table A-1. 241-AX Tank Farm Design Requirements.

Item and Description	Requirement	Quantity	Remark
1. Earthquake	Earthquake resistance	Zone 2 (0.25 g)	Uniform Building Code
2. Earth cover	Radiation shield	6 ft minimum	Provide minimum 6-ft earth cover over apex of dome. Unit weight of soil 120 lb/ft^3.
3. Live load on tank dome	Support construction or operating equipment	40 lb/ft^2 plus 50 tons	40-lb/ft^2 uniform load on projected dome area plus 50-ton concentrated equipment load.
4. Steel shell	Leak tight barrier	Carbon steel	Type and construction method to minimize corrosion attack.
5. Thermal barrier	Absorb transient thermal gradients. Allow independent thermal deformation of steel shell		Material, thickness, and construction method to be determined during process design engineering.
6. Concrete shell	Exterior support and secondary containment		Could be pre-stressed or conventional depending on structural integrity and economics.
7. Strain gages and thermocouples	Stress computation and heat transfer validation.		Required on all tanks
8. Usable liquid volume	3 to 4 Mgal		Provision for expansion to 6 to 8 Mgal total.
9. Vapor space	Surface area and expansion		Do not fill beyond springing line of dome.
10. Vapor pressure	Absorb intermittant pressure surges and vacuum of off-gas system	13.7 to 17.7 psig	
11. pH	Alkaline	8 to 10	Acid waste will be neutralized in process building
12. Specific gravity	Waste concentration	1.8	Tanks may eventually be used for solidification and storage of non-boiling wastes.

Item and Description	Requirement	Quantity	Remark
13. Settleable solids	Minimum practicable	~15% by volume	Not practical to dilute until no settleable solids exist.
14. Design temperature Vapor Liquid Sludge	Maximum probable	 222 °F 280 °F 350 °F	Including the incremental temperature change.
15. Incremental temperature change in 24 hours	Absorb transient thermal gradients	100 °F in conical bottom 60 °F in vertical wall 40 °F in vapor space	The maximum bulk temperature rise in 24 hours.
16. Rate of fill Raw water Waste from process Waste condensate	Cooling storage	 300 gpm @ 70 °F 75 gpm @ 200 °F 100 gpm @ 170 °F	
17. Corrosion allowance		1/16 inch	
18. Design life	Usable life of tank for design conditions	25 years	

HW-8237, 1962, *Specification for the 241-AX Tank Farm, Project CAC-945,* Hanford Engineer Works, A Division of General Electric, Richland, Washington.

Construction specification for the 241-AX Tank Farms.

SD-RE-TI-035, 1985, *Technical Basis for Single-Shell Tank Operating Specifications*, Rev. 1, Rockwell Hanford Operations, Richland, Washington.

This document contains technical basis documentation for the OSD-T-151-00013 (1998) single-shell tank operating specification. The following (Table A-2) structural related interim operating limits were given until replaced by revised requirements based on SD-RE-TI-012 (1983).

Table A-2. Structural Related Interim Operating Limits per SD-RE-TI-035.

	Single-Shell Tank	Limit			Reference
Dome Loads	**Soil Cover (ft)**				
	B, BX, C, T, U (100-Series)	7			ARH-C-19 (1977)
	BY, S, TX, TY	7			ARH-2883 (1973)
	SX	7			ARH-R-45 (1969)
	A	8			ARH-R-120 (1972)
	AX	8			ARH-C-11 (1976) RHO-R-6 (1977)
	B, C, T, U (200-Series)	No reference available			
	Live Load				
	B, BX, C, T, U (100-Series)	17 tons over 1 ft^2 or			Letter (Dec. 1, 1961)
	S, TY	30 tons over 30 ft^2			Letter (Jan. 4, 1962)
	BY, TX	No additional live load allowed			Letter (Jan. 4, 1962)
	SX	No reference available			
	A	11 tons over 1 ft^2 or 19 tons over 30 ft^2			Letter (Dec. 1, 1961) Letter (Jan. 4, 1962)
	AX	100 tons			ARH-C-11 (1976) RHO-R-6 (1977)
	B, C, T, U (200-Series)	No reference available			
	Live load limits are assumed to occur on the soil over the center of the tank. Higher loads are acceptable at the periphery of the tanks, but should be evaluated on a case-by-case basis.				
Thermal Loads	200-Series SSTs do not have thermal loads, have not been evaluated for elevated temperatures and are therefore not covered in this section.				
	Waste Maximum Temperature (°F)				
	B, BX, C, T, U (100-Series)	330			ARH-C-19 (1977)
	BY, S, TX, TY	280			ARH-2883 (1973) HW-37519 (1955)
	A, SX	300			HW-59919 (1959) RL-UPO-12 (1965)
	AX	350			ARH-R-120 (1972) RHO-R-6 (1977)
	Temperature Transients (Heat-up/Cool-down Rates)				
	B, BX, C, T, U (100-Series) S, TX, TY A, SX	2 °F/day			HW-59919 (1959)
	BY	3.7 °F/day up to 250 °F 1.5 °F/day above 250 °F			ARH-2883 (1972)
	AX	3 °F/day			ARH-C-11 (1976)
	Thermal Gradients				
	B, BX, C, T, U (100-Series)	10 °F	Through wall		RL-UPO-12 (1965)
	BY, S, TX, TY	29 °F/ft	Along dome	Steady state	RL-UPO-12 (1965) NW-47087 (1957)
	SX	14 °F/ft	Along wall		
	A	44 °F/ft	Along dome	Transient	
	AX	21 °F/ft	Along wall		
Hydrostatic	**Maximum Liquid Level/Specific Gravity**				
		Liquid Level (in.)	Specific Gravity	Maximum Head (psig)	
	B, BX, C, T, U (100-Series)	189	1.9	13.0	HW-37519 (1955)
	BY, S, TX, TY	281	1.2	12.2	
	SX	368	1.5	19.9	
	A, AX	365	2	26.2	HW-37519 (1955) RHO-R-6 (1977)
	B, C, T, U (200-Series)	285	1.9	19.5	HW-37519 (1955)
Vapor Pressure	**Vapor Pressure (inches of water gauge)**				
	B, BX, C, T, U (100-Series) BY, S, TX, TY A, AX, SX B, C, T, U (200-Series)	+130 in. w.g. maximum for structure (ventilation system may require lower limit) -15 in. w.g. minimum (with 15 in. of water or equivalent in tank bottom)			Letter (June 24 , 1982)

WHC-SD-WM-OSR-005, 1994, *Single-Shell Tank Interim Operational Safety Requirements*, Rev. 0 (Cancelled), Westinghouse Hanford Company, Richland, Washington.

This document provides operational safety requirements and bases prior to BIO implementation and is of interest in establishing the evolution and adequacy of current requirements.

OSD-T-151-00013, 1998, *Unclassified Operating Specifications for Single-Shell Tanks*, Rev. D-16, Lockheed Martin Hanford Company, Richland, Washington.

Contains the operating specifications and technical bases thereof for all SSTs.

OSD-T-151-00030, 1998, *Operating Specification for Watch List Tanks*, Rev. B-27, Lockheed Martin Hanford Company, Richland, Washington.

OSD-T-151-00031, 1998, *Operating Specifications for Tank Farm Leak Detection and Single-Shell Tank Intrusion Detection*, Rev. C-0, Lockheed Martin Hanford Company, Richland, Washington.

HNF-IP-1266, 1997, "Dome Loading Controls," Chapter 5.16, Rev. 1a, *Tank Farms Operations Administrative Controls*, Lockheed Martin Hanford Company, Richland, Washington.

Chapter 5.16 of HNF-IP-1266 documents the dome load administrative control of the Hanford Site underground waste storage tanks in accordance with HNF-SD-WM-TSR-006 (1998) requirements based on HNF-SD-WM-BIO-001 (1998) and defense-in-depth requirements.

HNF-SD-WM-TSR-006, 1998, Tank Waste Remediation System Technical Safety Requirements, Rev. 0-S, Fluor Daniel Hanford, Inc., Richland, Washington.

Contains current technical safety requirements (TSRs) for single-shell tanks, as well as other TWRS facilities, based on HNF-SD-WM-BIO-001 (1998). The TSRs are based on the preventive and mitigative features determined to be essential in the BIO. An Addendum to the TSR document contains Transitional Requirements – controls that have been directed by DOE to be retained in the TWRS Authorization Basis. The temperature of nine selected SSTs (241-C-106, prior to start of waste retrieval sluicing operations, 241-SX-103, 241-SX-107 through –112, and –114) with heat loads greater than 26,000 Btu/h are restricted to a maximum waste temperature of 250 °F (safety limit) or 205 °F (operating limit) in order to prevent potential salt-nitrate reactions and "tank bumps."

SAFETY ANALYSES

RHO-LD-55, 1980, *An assessment of the Risks Associated with Continued Storage of High-Level Waste in Single-Shell Tanks at Hanford*, prepared by D. J. Quinn and P. C. McNamee of SRI International, Menlo Park, California and R. G. Baca and D. E. Wood of Rockwell Hanford Operations, Richland, Washington.

WHC-SD-SAR-006, 1989, *Single-Shell Tank Isolation Safety Analysis Report*, Rev. 2, G. L. Borsheim, Westinghouse Hanford Company, Richland, Washington.

LA-UR-95-1900, 1995, *Probabilistic Safety Assessment for Hanford High-Level Waste Tanks*, D. R. MacFarlane, et al., Los Alamos National Laboratory, Los Alamos, New Mexico.

HNF-SD-WM-BIO-001, 1998, *Tank Waste Remediation System Basis for Interim Operation*, Rev. 1, Fluor Daniel Hanford, Inc., Richland, Washington.

Documents the safety interim authorization basis for operation of Hanford Tank Waste Remediation System (TWRS) facilities until the Basis for Interim Operation (BIO) is replaced by a fully compliant Final Safety Analysis Report (FSAR). Note that no existing TWRS safety class (SC) structures, systems, and components (SSCs) in the BIO were identified to need any safety class structural design attributes to perform or maintain a safety function during or after evaluation basis natural phenomena events.

HNF-SD-WM-SAR-067, 1999, *Tank Waste Remediation System Final Safety Analysis Report*, Rev. 0, Fluor Daniel Hanford, Inc., Richland, Washington.

Seismic events at the tank farms pose a natural phenomenon hazard because they could initiate an accident or initiate multiple accidents from a common cause. The hazard classification for the SSTs is specified as Hazard Category 2 in HNF-SD-WM-BIO-001 but their Safety Classification is not specified in the BIO. SSTs are designated as Safety-Class structures in HNF-SD-WM-SAR-067 for the Natural Phenomena—Seismic accidents. This leads to a Performance Category (PC) 3 classification for SSTs under seismic loading in accordance with HNF-PRO-097 (1997) which is driven by DOE Order DOE 5480.28 (1993), *Natural Phenomena Hazards Mitigation*. For existing PC 3 type structures located in the 200 Area of the Hanford Site, the evaluation basis horizontal peak ground acceleration is 0.19 g with a return frequency of 10^{-3}/yr (HNF-PRO-097).

Section 3.4.2.12.3 of HNF-SD-WM-SAR-067 shows that a seismic event at the tank farm could damage tank farm facilities. However, tank failure or collapse would not be expected for the evaluation basis earthquake based on available analyses [RHO-R-6 (1978), WHC-SD-W320-ANAL-002 (1995), WHC-SD-TWR-RPT-002 (1996), WHC-SD-TWR-RPT-003 (1996), and RLCA (1996)] and worldwide experience of typical damage to structures as a function of seismic acceleration level (see Table A-3 from HNF-SD-WM-SAR-067).

Table A-3. Seismic Accelerations, Magnitudes, Expected Frequencies, and Effects.

Peak horizontal ground acceleration	Likelihood of occurrence per year[a]	Description[b]
		"Anticipated" Events
0.05 *g*	> 1.0 E-02	Threshold acceleration above which emergency response actions are implemented. Characteristic of MMI VI. Felt by all. Adobe and weak plaster may crack.
		"Unlikely" Events
0.12 *g*	2.0 E-03	Design criterion for existing Safety-Class 1 facilities per HPS-SDC 4.1. Threshold for evacuation of nonessential personnel. Characteristic of MMI VII. Difficult to stand. Ordinary masonry may crack, weak chimneys may fall.
0.19 *g*	1.0 E-03	Design criterion for existing Performance Category 3 equipment (HNF-PRO-097, 1997); selected as the FSAR evaluation basis acceleration. Characteristic of MMI VII. Noticed by vehicle drivers. Waves appear on ponds, water turbid with mud.
0.24 *g*	5.0 E-04	Design criterion for new Performance Category 3 equipment for the 200 East Area (HNF-PRO-097, 1997). Characteristic of MMI VIII. Collapse of ordinary nonreinforced masonry walls and chimneys; collapse of towers and stacks.
0.26 *g*	5.0 E-04	As above except for the 200 West Area.
0.3 *g*	3.0 E-04	Characteristic of MMI IX. Threshold acceleration for damage to ordinary foundations, potential underground pipe breaks.
0.43 *g*	1.5 E-04	High confidence, low probability of gross leakage of underground storage tank (LA-UR-95-1900, 1995). Characteristic of MMI IX as above.
		"Extremely Unlikely" Events
0.6 *g*	5.0 E-05	Median acceleration for gross leakage of SSTs per WHC-SD-TWR-RPT-002. Characteristic of MMI X. Most ordinary masonry and frame buildings are destroyed.
0.8 *g*	2.0 E-05	Median acceleration for gross leakage of DSTs per WHC-SD-TWR-RPT-002. Characteristic of MMI X. Most ordinary masonry and frame buildings are destroyed.

Sources: WHC-SD-TWR-RPT-002, 1996, *Structural Integrity and Potential Failure Modes of the Hanford High-Level Waste Tanks*, Rev. 0, Westinghouse Hanford Company, Richland, Washington.
HPS-SDC 4.1, *Hanford Plant Standards*, Rev. 12, U.S. Department of Energy, Richland Operations Office, Richland, Washington.
[a]Frequency of occurrence based on WHC-SD-W236A-TI-002, 1996, *Probabilistic Seismic Hazard Analysis, DOE Hanford Site, Washington*, Rev. 1, Westinghouse Hanford Company, Richland, Washington.
[b]Modified Mercalli Intensity levels and effects based on ORNL-NSIC-28, 1970, *Earthquakes and Nuclear Power Plant Designs*, Oak Ridge National Laboratory, Oak Ridge, Tennessee.
DST = double-shell tank. MMI = Modified Mercalli Intensity.
FSAR = Final Safety Analysis Report. SST = single-shell tank..

The tank functions to confine materials sufficiently to mitigate onsite and offsite consequences. The safety function of the waste tanks is to maintain gross tank structural integrity, which averts tank collapse and limits the release of waste during and after the evaluation basis earthquake, thus decreasing the consequences of the Natural Phenomena—Seismic accident.

CONCRETE PROPERTIES AND TEST PROGRAM

ARH-R-47, 1969, *Model Tests of Waste Disposal Tanks*, prepared by D. McHenry and O. C. Guedelhoefer, Wiss, Janney, Elstner and Associates, Northbrook, Illinois for Atlantic Richfield Hanford Company, Richland, Washington.

This document reports the results of a micro-concrete 1:10-scale model load test of the 241-A-105 single-shell tank to ultimate load. The objective of the test was to establish the ultimate load capacity of the dome and to investigate the overall performance of the tank by observing strains, displacements, and cracking behavior during thermal and load testing. No attempt was made to reproduce precisely, either the applied load or the thermal conditions of the prototype since the actual scale-model conditions could be simulated in computer analysis, which could then be extended to actual prototype conditions. Hence, the lateral soil loading on the cylindrical wall of the prototype tank was not simulated. Also, the bottom slab and steel liner were not included in the scale model. Holes in the dome for riser penetrations in the prototype were modeled. The yield strength of the scale-model reinforcing steel varied from 31.4 to 37.4 ksi compared to the specified minimum yield strength of 40 ksi for the prototype ASTM A15 reinforcing steel. The 28-day compressive strength of the cylindrical wall and dome of the scale-model micro-concrete was 2,860 and 4,920 lbf/in^2, respectively as compared to the specified 3,000 lbf/in^2 for the prototype.

In the thermal load phase of the test, the objective was to assess the effect of a 100 °F through wall thermal gradient obtained over a one-hour heat-up period. In the load test, a uniform increasing load was applied to the dome until failure. The performance of the tank was monitored by displacement and strain gauge instrumentation.

After the initial shakedown load test of the experimental model to 3,000 lbf/ft^2, the dome was unloaded and the concrete was exposed to steam during the thermal test. In the thermal test a through wall thermal gradient of 126 °F in the dome and 84 °F in the cylindrical wall was achieved in about 52 minutes. The maximum temperature at the inside surface of the dome and cylindrical wall approached 180 °F. Two cycles of thermal loading were conducted with crack mapping after each cycle. Most cracks closed after the scale model was returned to ambient. However, there was also an observation of delamination of the inside concrete cover from the first inside layer of rebar just above the footing. Both the initial load test to 3,000 lbf/ft^2 and subsequent thermal test induced extensive cracking in the dome and sidewall of the concrete structure. However, the thermal induced cracking is not expected to decrease the ultimate load capacity of the structure.

During the load test, noticeable non-linearity in the wall lateral displacement was observed at 3,500 to 3,900 lbf/ft^2. General yielding of the hoop reinforcement in the vicinity of the construction joint just below the dome haunch was indicated at a load level of 3,100 lbf/ft^2. The failure load of the experimental model was 5,400 lbf/ft^2 with a dome center deflection of approximately 0.4 inches (corresponding to 4 inches for prototype) just prior to failure. The mode of failure was a simple combined bending and axial compression failure of the cylindrical wall or "slabbing" as it is sometimes referred to, just below the dome haunch and was fairly uniform around the model. The dome did not buckle. However, the failure was sudden with little or no advance warning. Most of the energy of the failure was dissipated in the northwest quadrant section of the wall with little damage to the opposite quadrant for no apparent reason.

In ARH-R-47 a safety factor against dome collapse was estimated at 5.65 for an assumed soil overburden of 7 feet with a soil unit weight of 110 lbf/ft^3. This assumes that the equivalent uniform load from the soil between a horizontal plane at the dome apex and the top surface of the dome is 188 lbf/ft^2, which when combined with the soil cover load above the dome apex of 770 lbf/ft^2, results in a total equivalent soil service load of 958 lbf/ft^2. *If the soil between the dome apex and the top surface of the dome were uniformly spread over the dome, the resulting uniform load would be approximately 484 lbf/ft^2. This is about a factor of 2.6 times 188 lbf/ft^2. No supporting information was provided in ARH-R-47 to justify the 188 lbf/ft^2 value. However, the 188 lbf/ft^2 equivalent uniform load might be justified by comparing stress resultants from a dome loaded uniformly with corresponding values from a dome load simulating the soil load distribution between the horizontal plane at the dome apex and the top surface of the dome.*

As a simplified estimate for this equivalent uniform load from the soil between a horizontal plane at the dome apex and the top surface of the dome, assume the pressure varies linearly increasing from the center to the outer radius of the tank. Taking the ratio of moments induced in a circular flat plat with fixed edges, for a uniform and linearly increasing pressure load gives

	Uniform Pressure Load (Case 10b from Young)	Linearly Increasing Pressure Load (Case 11b from Young)	Ratio of Moments from Uniform Pressure Load to Linearly Increasing Pressure Load
Center Moment	$q\,a^2\,(1+\nu)/16$	$q\,a^2\,(1+\nu)/45$	2.812
Edge Moment	$-q\,a^2\,/\,8$	$-q\,a^2\,/\,15$	1.875

Young, W. C., 1989, Roark's Formulas for Stress & Strain, 6th Edition, McGraw-Hill, New York, New York.

Hence, the equivalent uniform load used in ARH-R-47 appears reasonable.

Accepting the 188 lbf/ft^2 as the equivalent uniform load for the soil between a horizontal plane at the dome apex and the top surface of the dome, the total equivalent uniform soil load for other soil overburden conditions on SSTs can be estimated from the following relation:

$$p_{soil} = \gamma_{soil} \cdot \left[h + \frac{188 \cdot lbf / ft^2}{110 \cdot lbf / ft^3} \right]$$

where

p_{soil} = *equivalent uniform soil pressure load*
h = *actual soil cover height above dome apex*
γ_{soil} = *actual in-place unit weight of soil.*

It must be recognized that the scale-model test does not determine the actual failure load of the SSTs under in-service conditions. The failure load depends on the actual compressive strength of the in-place concrete and the yield and strain-hardening behavior of the reinforcing steel. The strength of the in-place concrete is affected by its initial strength, age, and thermal load history. In addition, the actual soil loading of the underground tank is more complex than idealized in the simplified test, which did not account for the lateral soil pressure loading. In addition, concentrated loads may cause more distress than a uniform load of equal magnitude depending on the location of the concentrated load relative to the center of the tank.

The test does provide data to benchmark computer models which can then be extended to evaluate actual load histories as was done in ARH-R-120 (1972), WHC-SD-W320-ANAL-001 (1995), and Report No. 941101-001 (1994). The ultimate load results for the experimental model was 5,400 lbf/ft^2 while these analyses of the scale-model test predicted an ultimate load of 4,100, 3,900 and 4,644 lbf/ft^2, respectively using three different finite element computer programs. ARH-R-120 used a modified version of ASOLID®, a thick-shell computer program written by E. Wilson (University of California, Berkeley). WHC-SD-W320-ANAL-001 used the general purpose ABAQUS® finite-element computer program written by Hibbitt, Karlsson, and Sorensen, Inc. with the nonlinear

concrete constitutive model (ANACAP-U®) provided by Y. R. Rashid of ANATECH Research Corporation. Report No. 941101-001 used the ADINA® finite-element computer program.

RL-SEP-630, 1965, *105-A Waste Storage Tank Model Test*, D. D. Wodrich, General Electric Hanford Atomic Products Operation, Richland, Washington.

This report discusses results of a 1:10-scale model test of the 241-A-105 waste-tank steel liner to simulate the observed 8.5-foot upward bulge of the bottom of the 241-A-105 tank liner. The original bulge was believed to have been caused by the formation of steam from residual water in the cement grout between the liner and the concrete basemat because of the heat from the stored waste. The scale model was pressurized on the bottom until it bulged and failed. The differential pressure at failure was 0.63 lbf/in^2 and the maximum bulge height was 4.625 inches, corresponding to 4 feet in the actual tank. The final failure was a rupture of about 2 inches at the weld joint which joins the bottom plate to the side wall of the tank liner at 90°. There is no knuckle radius transitioning the bottom plate and side wall of the liner in the SX- and A-Tank designs.

ARH-R-217, 1976, *Final Report: Concrete Testing Program*, D. Stark, Atlantic Richfield Hanford Company, Richland, Washington.

RHO-C-21, 1978, *Expansion of Hanford Concrete*, prepared by M. P. Gillen, Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois for Rockwell Hanford Operations, Richland, Washington.

RHO-C-22, 1978, *Strength and Elastic Properties of Concretes From Waste Tank Farms*, prepared by M. P. Gillen, Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois for Rockwell Hanford Operations, Richland, Washington.

RHO-C-23, 1978, *Effects of Temperature Cycling on Strength and Elastic Properties of Hanford Concrete*, prepared by M. P. Gillen, Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois for Rockwell Hanford Operations, Richland, Washington.

RHO-C-27, 1979, *Creep and Cycling Tests - Thermal Properties of Hanford Concrete*, prepared by M. P. Gillen, Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois for Rockwell Hanford Operations, Richland, Washington.

RHO-C-28, 1979, *Elastic and Strength Properties of Hanford Concrete Mixes at Room and Elevated Temperatures*, prepared by M. S. Abrams, M. Gillen, and D. H. Campbell, Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois for Rockwell Hanford Operations, Richland, Washington.

RHO-C-50, 1980, *Final Report on Long-Term Creep of Hanford Concrete at 250 °F and 350 °F*, prepared by M. P. Gillen, Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois for Rockwell Hanford Operations, Richland, Washington.

ORNL/BRP-80/5, 1980, *Final Report of Comprehensive Testing Program for Concrete at Elevated Temperatures*, C. B. Oland, D. J. Naus, and G. C. Robinson, Oak Ridge National Laboratory, Oak Ridge, Tennessee.

RHO-RE-CR-4, 1981, *Effects of Moisture Loss Due to Radiolysis on Concrete Strength*, prepared by M. P. Gillen, Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois for Rockwell Hanford Operations, Richland, Washington.

RHO-C-52, 1981, *Interim Report on the Effects of Waste Solutions on Reinforced Hanford Concrete*, P. H. Kaar, and D. C. Stark, Rockwell Hanford Operations, Richland, Washington.

RHO-C-54, 1981, *Effects of Long-Term Exposure to Elevated Temperature on the Mechanical Properties of Hanford Concrete*, prepared by Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois for Rockwell Hanford Operations, Richland, Washington.

RHO-RE-CR-2, 1982, *Strength and Elastic Properties Tests of Hanford Concrete Cores - 241-SX-115 Tank and 202-A PUREX Canyon Building*, prepared by M. P. Gillen, Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois for Rockwell Hanford Operations, Richland, Washington.

RHO-RE-CR-6, 1982, *Durability & Estimated Lifetime of Hanford Concrete*, prepared by M. P. Gillen, Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois for Rockwell Hanford Operations, Richland, Washington.

This report presents the results of a power-function regression analysis of strength and modulus test data from Hanford-mix concrete exposed to a 350 °F elevated temperature for up to 3.5 years. The maximum strength reduction observed was in the range of from 20 to 25 percent. It was noted that the compressive strength of concrete cores taken from 241-A tank farms, after 20 years of service, when tested at 250 °F was well above the initial minimum specified 28-day strength. See PNL-7779 (1988) for analysis of the complete PCA database of the Hanford-mix concrete test data including exposures to elevated temperatures up to 450 °F for up to 3.5 years.

RHO-RE-CR-8, 1982, *Long-Term Effects of Waste Solutions on Concrete and Reinforcing Steel*, prepared by J. I. Daniel, D. C. Stark, and P. H. Kaar, of Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois for Rockwell Hanford Operations, Richland, Washington.

This report presents the results of four years of concrete degradation studies that exposed concrete and reinforcing steel, under load and at 180 °F, to simulated double-shell slurry, simulated salt cake solution, and a control solution. Exposure time varied from three to thirty-six months. In all cases, examination of the concrete and reinforcing steel at the end of the exposure indicated there was no evidence of adverse attack—no evidence of rusting, cracking, disruption of mill scale or loss of strength.

PNL-7779, 1988, *Modeling of Time-Variant Concrete Properties at Elevated Temperatures*, C. H. Henager, G. F. Piepel, W. E. Anderson, P. L. Koehmstedt, and F. A. Simonen, Pacific Northwest Laboratory, Richland, Washington.

This report presents the analysis of the complete PCA database of the Hanford-mix concrete test data including exposures to elevated temperatures of 250, 350, and 450 °F for up to 1,300 days (3.5 years). The PCA database included lab test results for modulus of elasticity, compressive strength, splitting tensile strength, and Poisson's ratio of 3,000 and 4,500 lbf/in^2 Hanford-mix concrete. Limited creep strain data for 4,500 lbf/in^2 Hanford-mix concrete at 250 and 350 °F for up to 650 days was also available. Since the concrete property equations used in previous applications of the SAFE-CRACK® computer program in structural evaluations of the Hanford underground waste

storage tanks were developed before completion of the PCA study, the SAFE-CRACK® property equations were re-evaluated based on the full PCA database. Although there were differences between the previous SAFE-CRACK® property equations and the results obtained from the analysis of the full PCA database, they were in reasonable agreement. The use of a wider database in the development of the SAFE-CRACK® creep equations was justified because of the limited nature of the PCA creep data. See WHC-SD-WM-DA-153 (1994) for a recent re-assessment of the Hanford-mix concrete strength and modulus test data.

BNL-52384, 1993, *Thermal Degradation of Concrete in the Temperature Range from Ambient to 315 °C (600 °F)*, M. K. Kassir, Bandyopadhyay, K. K., and M. Reich, Brookhaven National Laboratory, Upton, New York.

This document presents the results of an independent literature review of the effects of elevated temperature on the properties of concrete. The compressive strength and modulus of elasticity tend to decrease over a large range with increasing temperature. Because of differences in the coefficients of expansion between concrete and the reinforcement steel, the bond strength between concrete and the reinforcement steel tends to decrease with increasing temperature. Thermal cycling causes progressive degradation of concrete with increasing number of cycles though most of the damage occurs in the first few cycles.

WHC-SD-WM-DA-153, 1994, *Evaluation of Strength and Modulus Degradation due to Temperature Effects on Hanford Concrete*, Rev. 0, W. S. Peterson, Westinghouse Hanford Company, Richland, Washington.

This report documents a recent re-assessment of the Hanford-mix concrete test data [RHO-C-28 (1979), RHO-C-54 (1981), PNL (1986) and PNL-7779 (1988)] relating concrete degradation with time at elevated temperature. The results from the re-assessment are more in line with the long-term lower bound residual strength and modulus relations given in BNL-52384 (1993) which were based on a broader database. The re-assessed degradation in compressive strength with time at temperature was consistent with the PNL-7779 correlation, however, the degradation in elastic modulus was not. The PNL-7779 correlation predicts a lower elastic modulus (about 50% lower) at long times than was predicted by the re-assessment. Hence, the application of the PNL-7779 correlation would lead to an under prediction of thermal stress and an over prediction of deflections.

At elevated temperatures, a direct result of the decrease in strength and modulus is a reduction in the load carrying capacity and the induced thermal loads. However, it must be recognized that there is a large scatter band for the compressive strength and residual modulus of concrete with increasing temperature. More rationally, the response of the waste tanks to their load history may be bracketed by comparing the response using the upper bound strength and modulus time-at-temperature relations with the corresponding response using the lower bound strength and modulus relations. For long times at temperature, the strength and modulus relation as a function of temperature given in BNL-52384 could be used to bracket the tank response. The BNL-52384 bounding strength and modulus relations are a function of temperature but not time at temperature, assumed valid for long-time exposure to elevated temperature.

WHC-SD-WM-DA-207, 1995, *Concrete Structural Analysis Tools and Properties for Hanford Site Waste Tank Evaluation*, Rev. 0, prepared by C. J. Moore and W. S. Peterson, ICF Kaiser Hanford Company, Richland, Washington for Westinghouse Hanford Company, Richland, Washington.

This documents provides an overview of concrete strength and modulus degradation with time at temperature, creep, shrinkage, effect of long-term sustained loads on failure limits, and temperature degradation of the bond strength between rebar and concrete. The report also reviews the nonlinear concrete constitutive models available in general purpose finite-element computer programs.

SOIL PROPERTIES

ISO-R-83, 1967, *Investigation of Earth Pressures and Settlement of Waste Tank Structures at Hanford, Washington*, prepared by E. Vey and R. D. Nelson, Department of Civil Engineering, Illinois Institute of Technology for Isochem, Inc., Richland, Washington.

This document reports the results of soil pressures investigations on the tank walls due to expansion of tank walls from temperature fluctuations. In situ penetrometer tests were conducted in horizontal boreholes in the soil surrounding the 241-A-106 and 241-SX-110 tanks. Measurements were made at depths of roughly 24 and 37 ft below the surface in both the radial and tangential directions around the tanks. Tangential measurements were made at several radial distances from the tank walls. In-situ soil temperatures were measured and the physical samples were preserved for laboratory testing. Soil moisture contents were mapped as a function of sample location relative to the tank wall. Re-molded laboratory samples were compacted to densities corresponding to in-situ conditions and then triaxial tests were performed to establish load deformation properties of the soil. Cyclic triaxial tests were conducted to simulate the cyclic thermal expansion of the tank sidewalls.

ARH-LD-132, 1976, *Geology of the 241-C Tank Farm*, W. H. Price and K. R. Fecht, Atlantic Richfield Hanford Company, Richland, Washington.

WHC-SD-GN-ER-33009 1992, *Bibliography and Summary of Geotechnical Studies at the Hanford Site*, Rev. 0, R. A. Giller, Westinghouse Hanford Company, Richland, Washington.

WHC-SD-WM-SOIL-001 1994, *Soil Weight at Hanford Waste Storage Tank Locations*, Rev. 0, prepared by E. W. Pianka, ADVENT Engineering, Services, Inc., San Ramon, California for Westinghouse Hanford Company, Richland, Washington.

Contains complication of soil density data for backfill soil above Hanford Site single- and double-shell tanks.

WHC-SD-WM-DA-208, 1995, *Soil Structural Analysis Tools and Properties for Hanford Site Waste Tank Evaluation*, Rev. 0, prepared by C. J. Moore, R. D. Holtz (University of Washington) and G. R. Wagenblast, ICF Kaiser Hanford Company, Richland, Washington for Westinghouse Hanford Company, Richland, Washington.

This documents provides an overview of Hanford soil properties data, available finite element soil constitutive models, and required soil parameters.

WHC-SD-WM-TI-665, 1995, *Soil Load Above Hanford Waste Storage Tanks*, Rev. 0, OA and OB, prepared by E. W. Pianka, ADVENT Engineering, Services, Inc., San Ramon, California for Westinghouse Hanford Company, Richland, Washington.

Contains complication of soil elevation data for backfill soil above Hanford Site single- and double-shell tanks.

THERMAL AND WASTE LEVEL HISTORY

RHO-CD-1172, 1981, *Survey of the Single-Shell Tank Thermal Histories*, P. F. Mercier, M. D. Wonacott, and C. DeFigh-Price, Rockwell International, Rockwell Hanford Operations Energy Systems Group, Richland, Washington.

This report documents the reconstruction of the early thermal histories of the SSTs prior to 1972 from data found in personal files and reports since much of the early surveillance records were believed to be irretrievably lost. The report emphasizes the need to retain this data due to its critical importance in assessing the long-term structural integrity of the SSTs because laboratory test data has indicated that exposure to elevated temperature is a dominant factor in the reduction of concrete strength and modulus.

The report also points out that the thermal expansion of Hanford concrete is approximately one-half that of steel unlike the thermal expansion of typical construction concrete which is usually within the range of that of steel. This mismatch in thermal expansion leads to additional thermal induced stresses in the concrete and could adversely affect the bound strength between the reinforcing steel and the concrete. However, consistent with normal design practice, the original design analyses of SSTs did not address this mismatch in thermal expansion. *This mismatch in thermal expansion was addressed in post design reassessments, such as in SD-RE-TI-012 (1983) for the SX tank analysis and in WHC-SD-W320-ANAL-001 (1995) for Tank 241-C-106.*

WHC-SD-WM-TI-591, 1994, *Maximum Surface Level and Temperature Histories for Hanford Waste Tanks*, Rev. 0, by J. S. Huisingh, N. D. HA, and B. D. Flanagan, Westinghouse Hanford Company, Richland, Washington.

This report summarizes the available waste surface level and waste temperature history data for the 100-Series SSTs and the DSTs from 1943 through 1994. Surface level and temperature data were summarized from logbooks, various reports, and from the Surveillance Analysis Computer System (SACS) database. Much of the data from 1974 to the present is available from the SACS database. Although some of the data may have been archived, data prior to 1974 was not documented or stored in a consistent and retrievable manner and may be irretrievably lost. However, much of this early data has been summarized in various other reports. For example, early thermal histories for the SST are summarized in RHO-CD-1172 (1981). This data was included in the WHC-SD-WM-TI-591 thermal history summary for the SSTs.

A more up-to-date summary of the surface level and temperature data for the Hanford Site tanks is now contained in the series of *Supporting Documents for the Historical Tank Content Estimate* for each of the tank farms; such as, WHC-SD-WM-ER-313 for the C Tank Farm. However, these more recent documents do not include tank temperature data prior to 1974 and hence do not provide a complete thermal history of the tank waste which is essential for assessing the current structural condition of the tanks.

WHC-SD-WM-ER-313, 1996, *Supporting Document for the Historical Tank Content Estimate for C Tank Farm*, Rev. 1, prepared by ICF Kaiser Hanford Company, Richland, Washington for Westinghouse Hanford Company, Richland, Washington.

WHC-SD-WM-ER-310, 1997, *Supporting Document for the Northeast Quadrant Historical Tank Content Estimate Report for B-Tank Farm*, Rev. 1b, prepared by Fluor Daniel Northwest, Inc., Richland, Washington for Lockheed Hanford Corporation, Richland, Washington.

HNF-SD-WM-ER-320, 1997, *Supporting Document for the Historical Tank Content Estimate for T-Tank Farm*, Rev. 1, prepared by Fluor Daniel Northwest, Inc., Richland, Washington for Lockheed Hanford Corporation, Richland, Washington.

HNF-SD-WM-ER-321, 1997, *Supporting Document for the Historical Tank Content Estimate for TX-Tank Farm*, Rev. 1, prepared by Fluor Daniel Northwest, Inc., Richland, Washington for Lockheed Hanford Corporation, Richland, Washington.

HNF-SD-WM-ER-325, 1997, *Supporting Document for the Historical Tank Content Estimate for U-Tank Farm*, Rev. 1, prepared by Fluor Daniel Northwest, Inc., Richland, Washington for Lockheed Hanford Corporation, Richland, Washington.

SUPPORTING ANALYSES

HW-37519, 1955, *Structural Evaluation Underground Waste Storage Tanks*, by E. F. Smith, Hanford Atomic Products Operation, General Electric Company, Richland, Washington.

This reports presents results of a structural re-evaluation of the then existing 100-Series SSTs (includes all 100-Series SSTs except AX which had not been built yet) to determine the limiting values of internal pressure and effective liquid specific gravity, resulting from waste self-concentration, that would permit maximum utilization of waste storage capacity. The original design criteria for these tanks did not envision any serious temperature problems, nor was any consideration given to possible transient internal vapor pressure in the tanks.

To withstand increased long-term hydrostatic loads the circumferential reinforcing steel was permitted to approach an increased allowable ring-tensile stress of 20,000 lbf/in^2. This is 43% greater than the design ring-tension value of 14,000 lbf/in^2 recommended by the Portland Cement Association (PCA), ST-57 (1954). The increased allowable stress is rationalized on the following bases:

- The increased allowable stress is no more than the usual design stress for steel in other types of concrete structures and it is within the percentage increase permitted by certain codes in rating *existing* structures. The PCA recommended allowable stress for reinforcing steel other than for ring tension was 20,000 lbf/in^2 [see Section 16 of PCA Bulletin ST-57 (1954), *Circular Concrete Tanks without Prestressing*].

- A lower ring-tension allowable helps to control crack size but there are other considerations, such as the bond between the concrete and reinforcement, that need to be considered. A lower allowable steel stress may require larger size bars. Since the bond resistance is a function of the amount of surface contact between the concrete and steel, the use of larger size bar will result in less surface contact area per unit of cross-sectional steel area provided. The net result is less bond resistance, which can result in larger cracks.

- Creep of the concrete with time will tend to reduce the internal shrinkage forces in the concrete. This means that if the tank is initially under a hydrostatic load of relatively low specific gravity, as the internal shrinkage forces are reduced due to creep, the specific gravity of the liquid may be increased accordingly. Or equivalently, the allowable stress may be increased by the corresponding reduction in shrinkage forces. This mode of operation is consistent with the addition of waste with waste self-concentration, thus allowing maximum utilization of the waste storage capacity. The corresponding increase in allowable due to the reduction in shrinkage forces was estimated at 1,500 lbf/in^2.

- Although it is common practice to ignore the effect of soil pressure assistance in supporting any part of the hydrostatic loading, it does exist to some degree, except perhaps after a tank has expanded against the soil during a period of increasing waste temperature and then later shrinks away from the soil during a period of decreasing temperature. Any soil pressure load that is present will reduce the effect of the hydrostatic loading. In addition, the passive resistance of the soil will resist ultimate collapse of the wall.

- The average specific gravity of the waste (average weight per unit volume) does not necessarily reflect the true value causing hydrostatic loading on the wall. Heavy particles in suspension may have but little effect on the lateral hydrostatic pressure produced by the liquid. The settling of such solids to the bottom of the tank would have a similar effect in reducing the apparent specific gravity acting on the wall.

To withstand the transient internal vapor pressure load in combination with the sustained hydrostatic pressure load the circumferential reinforcing steel was permitted to approach an increased allowable ring-tensile stress of 27,000 lbf/in^2. This is close to the estimated at temperature yield point (approximately 30,000 lbf/in^2) for the reinforcing steel. The higher allowable with the transient pressure loading included was justified in analog with the usual practice of permitting an increased allowable approaching yield for wind or earthquake type transient loading in other structures. In addition, it was opinioned that this stress would not likely be reached because the transient vapor pressures are present at a time when the passive resistance of the soil is bearing on the tank wall due to radial expansion of the tank from the increasing elevated temperature of the stored waste.

The ring-tension stress was shown to be controlling and its magnitude is reduced by the beneficial beam-action resistance to the hydrostatic load as a result of the rigid nature of the concrete wall of the tank. The effectiveness of the beam action is affected by the actual boundary conditions of the cylindrical tank wall at the upper haunch and at the wall-to-foundation interface. Because the wall-to-foundation interface is in reality some where between hinged and fixed, the maximum ring tension occurs not at the bottom where the hydrostatic pressure is greatest but at some other point in the wall and at a reduced magnitude.

Although the details of the calculations are not provided in the summary report, they rely on the methodology of PCA ST-57. The resulting maximum effective specific gravity with simultaneous allowable transient internal vapor pressure for liquid waste at *elevated temperature* is summarized below for each of the existing tank farm types at their corresponding full waste volume capacity.

Table A-4. Recommended Limiting Values of Effective Waste Specific Gravity and Transient Vapor Pressure for Existing 100-Series SSTs to Achieve Maximum Utilization of Waste Storage Capacity.

Tank Farm	Storage Volume (Kgal)	Original Design		Recommended Limiting Values	
		Specific Gravity	Transient Vapor Pressure (psig)	Specific Gravity	Transient Vapor Pressure (psig)
B, BX, C, T, U	530	1.2	0	1.9	2.5
BY, S, TX, TY	758	1.25		1.2	1.8
SX	1,000	1.35		1.5	4.8
A	1,000	2.0		2.2	6.9

In the calculation, a conservative thermal gradient of 23 °F/ft of wall thickness was assumed. *However, the magnitude of the assumed maximum elevated temperature of the waste for each tank type was not given in the report but is assumed to be greater than the original design basis (WHC-SD-WM-TI-648) of 220 °F for the non-boiling waste type tanks (B, BX, C, T, U, BY, S, TX, and TY) and 250 °F for tanks (SX and A) designed to accommodate self-boiling waste.*

If the actual specific gravity is less, an increased transient vapor pressure would be permitted up to a limit of 10 psig beyond which the dome of the tank would be in jeopardy. It was recognized that there are uncertainties in the estimated limiting values, both pro and con. The use of higher allowable ring-tensile stress does permit more and wider cracks than would be permitted with the usual lower design allowable stress. However, the limiting values given were believed to be sufficient to ensure that the structural stability of the tank is not endangered. Although actual structural collapse due to hydrostatic head is difficult to conceive, it was believed that the limiting values presented cannot be exceeded without endangering the integrity of the concrete structure due to excessive crack openings, thus permitting leakage through wide cracks in the concrete if the steel liner should leak.

HW-47087, 1957, *Waste Tank Temperature Studies*, by M. W. Cook and J. M. Gerhart, Hanford Atomic Products Operation, General Electric Company, Richland, Washington.

This report provides the results of waste tank temperature studies for a proposed SST designed for the storage of high-heat self-boiling waste. Although waste entering a tank is relatively cool, in the high-heat waste tanks designed for storage of self-boiling waste, the temperature rises rather rapidly to the boiling point due to radioactive decay as filling proceeds and then the temperature continues to rise as self-concentration increases the boiling point of the waste. The increase in temperature during filling and subsequent long term storage results in transient and steady-state thermal gradients along and through the wall that induced thermal stresses in the concrete of the tank structure.

Although the reported results were for a proposed 1.25-Mgal SST design geometry that was never built and for specific operating conditions, the report does provide an overview of the importance of thermal related issues for the waste storage tanks and insights as to the magnitude of temperatures and temperature gradients to be expected in the tank concrete structure. Both transient and steady-state thermal conditions were addressed. *Note that the thickness of the concrete wall for the proposed tank design was 40 inches in the lower 2/3-section of the vertical wall as compared to 24 inches in subsequent one-Mgal SSTs (SX, A, and AX) that were built for the storage of self-boiling waste.*

Although it was recognized that there is uncertainty in both the concrete and soil thermal properties, with the largest uncertainty in the soil properties, the thermal properties of the concrete and soil were assumed to be equal for simplicity. This simplification leads to higher thermal gradients than would be predicted for a more typical dry soil condition where the thermal conductivity of the soil is less than that of the concrete. The heat transfer solution was facilitated by dividing the problem into three time periods. An early transient time period during which the waste temperature increased from ambient to boiling; an intermediate transient time period during which the temperature in the tank, the tank wall, and the surrounding soil change from the initial boiling conditions and approach steady state; and finally, the steady state period where temperature does not vary with time anywhere within the tank and the surrounding soil. For the early transient time period, an analytical solution for a semi infinite slab was assumed to apply. A resistive network analogue (see HW-47088) was constructed and used to obtain the steady-state solution. The solution for the intermediate time period was obtained through an adaptation of the well known Schmidt graphical method.

The results showed that the temperature gradients reach maximum values just as the boiling begins and then decrease gradually as steady state is approached, except at locations that were previously immersed in steam and were then exposed to the higher temperature of the waste solution as the liquid level rises. Dilution of the waste was shown to be a viable means for minimizing temperature gradients, especially during the early part of the tank life.

HW-47088, 1957, *The Design and Application of a Heat Transfer Analogue for Radially Symmetrical Problems*, by M. W. Cook, Hanford Atomic Products Operation, General Electric Company, Richland, Washington.

This report describes the design, construction, and trial application of an electrical analogue solution method for estimating the steady-state temperature field in a SST designed for the storage of high-heat self-boiling waste. An electrically conductive paper was used to construct the electrical analogue model with approximately 3,000 individual resistance elements. The results from the application of the electrical analogue solution model for a proposed tank design are reported in HW-47087.

HW-57274, 1958, *Instability of Steel Bottoms in Waste Storage Tanks*, by L. E Brownell, General Electric-Hanford Atomic Products Operation, Richland, Washington.

Reports results of steel tank liner bottom buckling study. The bottom of 241-SX-113 upwardly dished to a height of 4 feet in June 1958 and then gradually returned to a horizontal position within the month. The report presents a structural theory for the buckling and presents proposed design changes.

HW-59658, 1959, *Heat Transfer Study for Self-Boiling Radioactive Wastes*, by H. W. Stivers and G. R. Taylor, Hanford Atomic Products Operation, General Electric Company, Richland, Washington.

The purpose of this study was to determine the sensitivity of heat transfer parameters on the resulting thermal gradient across the side wall of existing Hanford Mgal SSTs (A and SX) designed for the storage of self-boiling waste, as a function of temperature rise of the stored wastes from ambient ground temperature (70 °F) to boiling (230 °F). Results are summarized in figures for the range of variations in the thermal parameters considered. The thermal parameters considered include the thermal conductivity of the soil, the rate of rise of the liquid waste temperature, the soil temperature seven feet from the wall, the film coefficient of heat conduction of the stored waste, the effect of various insulating materials, and the effect of incremental rate of temperature rise.

The results of this study were recommended as guidance for revising the then current operating procedures for existing tanks storing self-boiling waste and as guidance in future designs for selecting insulating media between the steel liner and the concrete wall. It was also recommended that new tanks incorporate replaceable temperature elements within the extreme wall surfaces as an aid in verifying the predicted results.

HW-59919, 1959, *Limitations for Existing Storage Tanks for Radioactive Wastes from Separation Plants*, by E. Doud and H. W. Stivers, Hanford Atomic Products Operation, General Electric Company, Richland, Washington.

Safe load curves were established from stress analyses conducted in accordance with PCA Bulletins ST-32 (1953) and -57 (1954) and the ACI-318-56 (1956) design codes. However, the reinforcing steel allowable stress in ring tension was increased from the normal 10,000 to 12,000 lbf/in^2 typically used in cylindrical concrete storage tanks to16,000 lbf/in^2, which is significantly less than the 20,000 lbf/in^2 assumed in HW-37519 (1955). A maximum through wall thermal gradient of 23 °F/ft of wall thickness was assumed which required that the average heat-up rate not exceed 2 °F/day, as was assumed in HW-37519. In addition, the maximum incremental increase in temperature should not exceed 40 °F at any one time, although this increment could be fast. A minimum time interval of four weeks is required to permit the outer temperature of the wall to approach the temperature of the inner surface for this maximum increment in temperature. For the self-boiling tanks the resulting transient vapor pressure was equated to an equivalent static pressure with a dynamic load factor of one (slow vapor pressure loading) and was assumed to be 15 to 30 percent of the hydrostatic head of the contained waste. The results from HW-59919 were more restrictive than obtained from HW-37519. As an example of the results from the safe load curves for the maximum expected waste specific gravity and vapor pressure indicated, HW-59919 gave the following allowable liquid levels which are significantly less than the full capacity liquid levels.

Table A-5. Allowable Liquid Levels (in.) for Indicated Maximum Expected Waste Specific Gravity and Vapor Pressure.

Tanks	Storage Volume (Kgal)	Waste Type	SpG	Vapor Pressure (psig)	Max. Liquid Level (in.)	
					Allowed	Reference Full Capacity
A	1,000	Self-boiling	1.37	3.0	258	365
SX	1,000	Self-boiling	1.68	2.5	192	365
BY, S, TX, TY	758	Non-boiling	1.8	0	180	281
B. BX, C, T, U	530	Non-boiling	1.8	0	168	189

Letter, 1961, *Live Load on 241 Waste Tanks* (internal letter E. F. Smith to H. W. Stivers, December 1), Hanford Atomic Products Operation, General Electric Company, Richland, Washington.

Basis letter for live load limits specified in SD-RE-TI-035 (1985).

Letter, 1962, *Live Load on 241-Waste Tanks* (internal letter E. F. Smith to H. W. Stivers, January 4), Hanford Atomic Products Operation, General Electric Company, Richland, Washington.

Basis letter for live load limits specified in SD-RE-TI-035 (1985).

RL-UPO-12, 1965, *Structural Evaluation of Existing 241 Waste Storage Tanks for Waste Solidification Program*, by E. F. Smith, Hanford Atomic Products Operation, General Electric Company, Richland, Washington.

RL-UPO-12 applied the same empirical methods from PCA ST-57 (1954) as used in HW-59919 (16 ksi ring-tension allowable working stress for reinforcement steel) but assumed that the vapor pressure above the liquid was no greater than atmospheric. Vapor pressures greater than atmospheric require additional reductions in the allowable liquid depths. In addition, the effect of the through wall thermal gradient under steady-state storage (10 °F/ft) and transient fill (20 °F/ft) conditions was considered. The results are summarized below in Table A-6.

Table A-6. Allowable Liquid Levels (in.) as a Function of WasteSpecific Gravity and Through Wall Thermal Gradient.

Tank Farm	Storage Volume (Kgal)	SpG	Allowable Liquid Depth (in.) for Through Wall Gradient	
			Steady State (10 °F/ft)	Transient Fill (20 °F/ft)
A	1,000 (365 in.)	1.3	Full	Full
		1.6	Full	Full
		1.9	Full	348
		2.2	319	300
		2.5	280	264
AX	1,000 (365 in.)	1.3	Full	Full
		1.6	Full	Full
		1.9	Full	Full
		2.2	Full	Full
		2.5	350	333
SX	1,000 (365 in.)	1.3	Full	348
		1.6	313	285
		1.9	265	242
		2.2	230	210
		2.5	204	187
BY, S, TX, TY	758 (281 in.)	1.3	236	228
		1.6	212	204
		1.9	192	183
		2.2	174	165
		2.5	159	152
B, BX, C, T, U	530 (189 in.)	1.3	Full	Full
		1.6	192	186
		1.9	170	168
		2.2	158	156
		2.5	150	147

These results are clearly more restrictive than the limits recommended in SD-RE-TI-012 (1983), i.e., tank filled to capacity with bulk liquid specific gravity of 2.0.

ARH-78, 1967, *PUREX TK-105-A Waste Storage Tank Liner Instability and Implications on Waste Containment and Control*, S. J. Beard, P. Hatch, G. Jansen, and E. C. Watson, Jr, Atlantic Richfield Hanford Company, Richland, Washington.

HN-197, 1968, *Report of Study of Hanford Waste Tank Structures*, prepared by Holmes & Narver, Inc., Los Angeles, California for Atlantic Richfield Hanford Company, Richland, Washington.

This report documents the results of an independent third party review of all previous and then current studies pertaining to existing and proposed high-level waste storage tanks at Hanford (see HN-197 for list of documents reviewed, much of which consisted of letter reports of preliminary results). The scope of work included:

1. Review work performed by Illinois Institute of Technology (IIT) and consultants, Professor. Eben Vey covering "Investigation of Earth Pressures and Settlements of Waste Tank Structures" (see ISO-R-83. 1967) and Professor K. P. Milbradt covering "Strength and Stress Analysis of Waste Tank Structures" (results reported latter under ARH-R-45, 1969 and ARH-120, 1972).
2. Review effect of storage conditions – chemical composition, solids content, specific gravity, pressure, age, environmental effects, temperature control, filling, and agitation on the intergity of the tank structure.
3. Review of the application of heat transfer principles to the known and assumed waste storage conditions that have been used to establish tank temperature limits.
4. Analysis of the application of theory, assumptions, interpretations of data, and conclusions on the approach, method, state of stress, and expected tank condition for a typical tank in the existing 241-A and 241-AX Tank Farms.
5. Analysis of the application of theory, assumptions, interpretations of data, and conclusions on the approach to the design of the new double-shell 241-AY Tank Farm.
6. Perform an independent computer analysis of one set of load parameters and other associated boundary conditions as furnished by IIT, and conclusion on the validity of results obtained from the IIT program.
7. Perform a preliminary study of selected seismic actions on the AY-inner tank based on horizontal ground motions corresponding to the average spectra given in TID-7024 (1963), *Nuclear Reactors and Earthquakes*, scaled to a maximum ground acceleration of 0.25 g.

In general, the conclusion was that the soil and thermal study provided reasonable input values for the computer analysis but that both were subject to limitations. The computer analysis of the structures was performed in accordance with accepted practice but it is subject to limitations. The cylindrical wall-to-slab footing region is a critical region in the stress analysis but the predicted temperature and the soil loading in this region has the greatest uncertainty. However, although high stresses in the wall-to-slab region will lead to wider cracks in this region they are less important for maintaining the stability of the structure because they are thermal induced. Overstress in the haunch region of the dome, however, is of concern because these stresses are primarily due to gravity loads rather than thermal loads.

Accepted design practice for the design of concrete shell structures at that time was provided in "Concrete Shell Structures, Practice and Commentary" – Report by ACI Committee 334, Proceedings, American Concrete Institute, Vol. 61, No. 9, September 1964. This report includes the following:

1. For elastic analysis, concrete may be assumed uncracked, homogeneous, and isotropic. Poisson's ratio may be assumed equal to zero.
2. Elastic behavior is the commonly accepted basis for determining stresses, displacements, and stability of thin shell structures. This type of analysis is satisfactory for design purposes but not necessarily sufficient to predict the actual stress at a specific point in the structure.
3. An ultimate strength analysis may be used only as a check on the adequacy of the design. It is not to be used as a sole criterion for design except where it can be proven to be applicable.
4. The principle tensile stresses shall be resisted entirely by reinforcement.

On the basis of available information and design procedures available when the 241-A Tank Farm was designed in 1953 and when the 241-AX Tank Farm was designed in 1963, the approach appears reasonable and consistent, and there is no basis on which to criticize the theory and the design.

However, these tanks have been subjected to temperatures greater than the original design temperature of 250 °F (see Table 2). There is a nominal margin of safety of at least 60 percent above the design stress in steel or concrete construction and the thermal induced stresses are only a part of the total stress resisted by the structure. Hence, *some increase in temperature and resulting thermal gradients may be accommodated provided that the higher temperatures do not significantly degrade the strength properties of the concrete. The result will be an increase in cracking and crack width in the concrete, but the presence of cracks are expected in concrete and do not necessarily imply a loss of load carrying capacity. However, more detailed analysis and/or testing may be required to more accurately assess the residual load capacity of those tanks that have operated significantly outside their original design conditions.*

ARH-R-45, 1969, *Interim Summary Report, Stress and Strength Analysis for Waste Tank Structures at Hanford Washington*, K. P. Milbrandt, Illinois Institute of Technology, Atlantic Richfield Hanford Company, Richland, Washington.

This report documents the results of *elastic*, thin shell, isotropic analyses for the A, AX, SX, BY, and BX tank configurations. The initiation of this study was motivated by the desire to fill the tanks to greater depths by utilizing the soil pressure to counteract the hydrostatic head and by a desire to determine the reusability of the waste tanks. The original design of the reinforced concrete shells was based on a membrane solution, which is acceptable, except that temperature loading was not considered in the design process. Furthermore, the base slab and footing were designed without total knowledge of the soil or temperature loading. These load parameters were ignored in part due to the state of knowledge at the time of design.

The main objective of ARH-R-45 was to define the states of stress and potential for leakage in the reinforced concrete SSTs as created by the soil, liquid head, vapor pressures, and temperature. Operating data on tank heating rates were used in the analysis. The load parameters considered are summarized in Table A-7.

Table A-7. Load Parameters Considered in ARH-R-45 Single-Shell Tank Analysis.

Tank	Load Case	Soil Cover Depth (ft)	Waste		Temperature (°F)			Sludge Depth (ft)	Active Soil Pressure Coef. (K_a)	Reactive Soil Pressure from Bottom to Top of Wall (lbf/in²)	Comment
			Level (in)	Specific Gravity	Surface of Liquid	Vapor	Bottom of Sludge				
A	1		360	1.3	222		250				
	2	7	336	1.3	222	212	300	2.5	1/3	11 to 6	
	4		312	1.28	221		300				
AX	1		348	1.5	230		300			10 to 6	
	4		252	1.2	218		300			10 to 6	
	5		348	1.5	230		350		1/3	12 to 8	
	5.1		348	1.5	230		350			30 to 26	Probable max. reactive soil pressure
	5.2	6.5	348	1.5	230	212	350	0.5	0	0	Free standing tank
	5.3		348	1.5	230		350				Axial stiffness
	5.4		348	1.5	230		350		1/3	12 to 8	decreased form
	5.5		348	1.5	230		350				3×10^6 psi (uncracked) to 0.15×10^6 psi (5% steel)
SX	1		312	1.65	236						
	1.1	7	387	1.65	236	212	300	3	1/3	10	
	2		240	1.5	230						
BY	3	8	276	1.8	200	160	200	3	1/3	10 to 6	
	5.1		276	1.8	230	212	230				
BX	1		192	1.15	100	95	100				
	2		204	1.15	100	95	100				
	3	7	204	1.4	100	82	100	3	1/3	3	
	4		204	1.4	140	122	140				

Soil density = 110 lbf/ft³.

For the load conditions considered, all but the BX results indicated that a containment compromise is possible with through-thickness, vertical, tensile fractures in the concrete at the junction between the footing and cylindrical wall. The presence of such cracks was verified from core samples removed from the footing extension of tanks 241-A-101, 241-SX-107, and 241-SX-108. However, the structural integrity of the waste tank dome was not predicted to be impaired. The results indicate a safety factor of 4 based on the soil cover depth listed in Table A-7.

The reactive soil pressure distribution along the bottom of the slab and footing was investigated; as well as, the reactive soil pressure along the cylindrical wall as a result of radial thermal expansion of the wall into the soil. A 20,000-psi/in soil modulus for the 24-in. footing width appeared to be the best solution. It limited the highest values of the soil stress to 75 lbf/in^2 and contained the soil pressure to the region under the cylindrical wall.

ARH-R-120, 1972, *Final Report: Strength and Stress Analysis for Waste Tank Structures at Hanford, Washington*, prepared by K. P. Milbradt, Department of Civil Engineering, Illinois Institute of Technology for Atlantic Richfield Hanford Company, Richland, Washington.

Results are presented comparing the analyses and experimental results from ARH-R-47 (1969) at 1.25 times the working load and at the ultimate load of 241-A tanks. The working load is based on a 7-foot soil overburden above the dome apex with a soil unit weight of 110 lbf/ft^3. This results in an equivalent uniform load of 958 lbf/ft^2 when accounting for the non-uniform load from the soil between a horizontal plane at the dome apex and the dome outer surface. The analysis uses a modified version of ASOLID, a thick shell computer program written by E. Wilson (University of California, Berkeley). The analysis assumed a 30,000-lbf/in^2 yield strength for the model reinforcing steel. The ultimate load results for the experimental model was 5,400 lbf/ft^2 while the analysis predicted 4,100 lbf/ft^2 (4.3 times the soil working load). The discrepancy was attributed to a potentially unknown increase in the concrete compressive strength when the concrete structure was exposed to steam during a thermal test prior to the ultimate load test.

Stress results for the prototype 241-A waste tank were also presented for static working and ultimate loads. A load factor against ultimate collapse of 6.2 times the soil load over the tank dome (no change in lateral load on the cylindrical wall) was predicted. The analysis assumed a best-estimate yield strength of 50,000 lbf/in^2 for the ASTM A15 reinforcing steel which has a specified minimum yield strength of 40,000 lbf/in^2. In the analysis and the experimental model, the bottom slab of the tank was not included. It is possible that the footing may possess a lower ultimate load capacity than indicated by the model or the analysis. Generally, the ultimate load design criteria of AISC and ACI lead to load factors of 1.7 to 2.0 against collapse for normal loading.

Results for the service or working loads indicate a maximum reinforcing steel stress of 15,000 lbf/in^2 in the outside meridional steel at the junction of the dome and cylinder. The allowable reinforcing steel stress was taken as 20,000 lbf/in^2 in the meridional and circumferential directions of the reinforcement steel.

Results from a soil-structure interaction seismic analysis using the DYNAX computer program for an OBE (operations basis earthquake) and DBE (design basis earthquake) level earthquake indicated no additional risk of containment loss beyond risk associated with static loading only. The peak ground acceleration for the DBE was reported as 0.187 g in the horizontal direction and 0.25 g in the vertical direction. *This is counter to all previous seismic analyses for the Hanford Site, which usually assumed a peak ground acceleration of 0.25 g in the horizontal direction and 2/3 of the horizontal in the vertical direction. It is also not clear how or if the hydrodynamic loading and resulting sloshing of the waste from the seismic ground motion was accounted for in the seismic analysis.*

This report lacks sufficient detail as to the load conditions and combinations being evaluated beyond the soil loading. There does not appear to be any treatment of the thermal loading or creep response of the concrete. In addition, the seismic analysis does not appear to be adequate. Hence, the results are of limited value.

ARH-2883, 1973, *Creep and Cracking Analyses of the 241-BY-112 Reinforced Concrete, Underground Waste Storage Tank*, F. R. Vollert, Atlantic Richfield Hanford Company, Richland, Washington.

This report summarizes structural analyses to predict how the 241-BY tank structure responds to the 250 °F and propose 280 °F liquid temperatures that could occur during solidification activities. The analysis evaluated time-dependent creep, cracking and stresses using the SAFE-CRACK® computer program.

Loads considered were:
- 7 ft of soil cover (120 lbf/ft^3)

- 23 ft (276 in.) of liquid with specific gravity of 1.25
- Thermal distribution resulting from 250 °F liquid and 280 °F liquid
- Heat-up was 3.7 °F/day to 250 °F and 1.5 °F/day from 250 to 280 °F.

Temperature distributions were predicted by finite element heat transfer analysis using the AMGAB computer program for an axisymmetric model of the 241-BY-112 SST and surrounding soil with the liquid at a constant temperature. The maximum thermal gradient up the wall was 7.5 °F/ft.

The material properties for the axisymmetric structural model of the tank (steel liner and base slab not modeled) were:

Concrete modulus of elasticity	3.0×10^6 lbf/in^2
Steel modulus of elasticity	30.0×10^6 lbf/in^2
Concrete coefficient of thermal expansion	6.0×10^{-6} in/in-°F
Steel coefficient of thermal expansion	6.0×10^{-6} in/in-°F
Concrete Poisson's ratio	0.16
Steel Poisson's ratio	0.3
Concrete uniaxial tensile strength	300 lbf/in^2
Concrete compressive strength	3,000 lbf/in^2
Reinforcing steel yield strength	33,000 lbf/in^2

No degradation in concrete strength or modulus with time at temperature was considered. This was justified on the basis that concrete compressive strength increases beyond its 28-day strength with age. The expected decrease in strength at the temperatures being evaluated was estimated at 10 percent, which was apparently assumed to be within the expected increase in strength with age before the application of the elevated temperatures being considered.

The concrete-creep relation in the SAFE-CRACK® program was based on experimental data to a maximum temperature of 200 °F. As there was no basis for extrapolation to higher temperatures, the program used the 200 °F data for temperatures greater than 200 °F. This was justified as not invalidating the analysis because the expected increase in the creep with increasing temperature would bring the deflected shape of the structure closer to the original shape before thermal expansion. Concrete creep for the length of time associated with these analyses would nearly halt. In addition, the amount of and nature of the reinforcement steel in the 241-BY-112 structure would limit the creep response.

In the analysis, creep and cracking nearly ceased before the 1,900 days of total time for the analysis. Most of the concrete cracking occurs in the lower wall portion of the tank, which is sensitive to the assumed fixity of the wall-to-base slab connection. In the finite element model, the base of the wall was modeled as fixed. This conservatively neglected the thermal expansion of the base of the wall as the base slab radially expands against the frictional drag of the soil with increasing temperature. Hence, the extent of the predicted cracking in the wall was considered conservative.

Although cracks were predicted to develop along the lower region of the wall and in the dome haunch region, the conclusion was that the tank stresses were acceptable for a 280 °F liquid waste temperature. The predicted cracking was sufficient that the concrete structure could not be relied on to maintain containment of the liquid if the steel liner were to leak.

ARH-C-11, 1976, *Thermal-Creep and Ultimate Load Analysis of 241-AX Structure*, prepared by Y. R. Rashid, ANATECH Research Corporation, San Diego, California for Atlantic Richfield Hanford Company, Richland, Washington.

The purpose of this analysis was to evaluate the existing million gallon AX Tank structures originally constructed between 1963 and 1964 to a new temperature history that will involve heating up to a temperature of 350 °F. Previous operation included temperatures up to 250 °F with several heating and cooling cycles possible. The new analysis considered a maximum heating rate of 2.85 °F/day from 65 °F to a final temperature distribution that varied from 230 °F on some regions of the dome outer surface to 350 °F at the tank base. The previous operational history was ignored. In addition to the new thermal loading, the structure was assumed subject to an overburden soil cover of 8 feet at a unit weight of 115 lbf/ft^3 and a live load of 100 tons over a 30-foot diameter area at the crown of the tank. A lateral earth pressure coefficient of 0.4 was assumed. The specific gravity of the waste was taken as 2.0. Although the tank concrete structure was built with the cylindrical wall fixed to the basemat, the base slab was not explicitly modeled but its behavior was simulated. A roller-support condition was applied at the base of the tank cylindrical wall and the resulting free expansion of the base slab with simulated soil resistance was applied at the base of the cylindrical wall.

The stiffness properties of concrete had been shown to change at a constant sustained temperature of 350 °F based on initial 1975 PCA report, *Elastic and Strength Properties of Hanford Concrete Mixes at Room and Elevated Temperatures*, later released as RHO-C-28 in 1979. A conservative interpretation of this limited data was applied to extend this effect to the creep, ultimate strength, and cracking properties of the concrete. The SAFE-CRACK® built-in creep properties, modulus and ultimate strength were modified through a linear time shift factor. The details of the built-in creep properties were not provided in ARH-C-11.

The thermal-creep analysis covered an evaluation period of 2,000 days in the following sequence: initial mechanical loading at time zero, 100-day heat-up period, steady-state thermal condition to 2,000 days followed by an increase of the mechanical loading (overburden + live load) to up to 500% of its initial value. The end of the 2,000-day period established the stationary creep and cracking condition of the structure. The last loading phase determines the residual safety factor of the structure. The following material properties were used in the analysis using the SAFE-CRACK® computer program.

Table A-8. Material Properties Used in Thermal-Creep andUltimate Load Analysis of AX Tanks.

Component		Elastic Modulus (10^6 psi)	Yield Strength (ksi)	Poisson's Ratio	Coeff. of Thermal Expansion (10^{-6} in/in-°F)	Compressive Strength (ksi)	Cracking Strength (psi)
Liner		30	31	0.3	6.6		
Reinforcement		30	40		6.6		
Concrete	28 days	3.5		0.15	6.6	4	400
	10-years at 350 °F	2.15		0.15	6.6	2.45	245

Note that the 50% mismatch in thermal expansion between Hanford concrete and steel (see RHO-CD-1172, 1981) was not considered. A 39% reduction in the concrete modulus, compressive strength, and cracking strength was assumed for concrete at a sustained temperature of 350 °F. *A more recent re-assessment of the Hanford concrete test data from RHO-C-28 (1979) indicated that the reduction in the concrete strength at a sustained temperature of 350 °F was approximately 35%, but that the reduction in modulus was approximately 53% (WHC-SD-WM-DA-153 1994 and WHC-SD-WM-DA-207, 1995). The results from the more recent re-assessment are more in line with the lower bound predictions given in BNL-52384 (1993) which were based on a broader database. At elevated temperatures a direct result of the decrease in strength and modulus is a reduction in the load carrying capacity and the induced thermal loads, respectively. Hence, the ARH-C-11 analysis is conservative relative to the use of the lower bound modulus. However, it must be recognized that there is a large scatter band for the residual modulus and compressive strength of concrete with increasing temperature. More rationally the response to thermal gradients may be bracketed by comparing the response between the upper bound strength and modulus and the lower bound strength and modulus relations, such as given in Figures 1 and 2 of BNL-52384 (1993).*

The predicted peak stresses in the concrete and steel and the time at which they occurred are summarized in Table A-9.

Table A-9. SAFE-CRACK® Predicted Peak Stresses in AX Tank Structure for 2,000 Day Thermal-Creep Analysis with 350 °F Sustained Peak Temperature.

Structural Component	Location	Stress* (psi)	Stress Type	Time Occurred (days)
Concrete	Cylinder base	-925	hoop	2,000
Liner		-29,400		
Reinforcement		29,000		

*Tension is positive.

At 2,000 days the thermal-creep analysis predicted some cracking in the haunched portion of the dome and in the lower portion of the wall. In the haunch area of the tank, radial cracks were predicted due to high tensile hoop stresses. The cracks in the lower portion of the cylindrical wall were meridional cracks caused by high tensile stresses in the R-Z plane.

The 10-year ultimate strength was reached at a load factor of 3.5 on the 8-foot soil overburden plus 100-ton concentrated load. The effected region was confined to a small region of the tank haunch which was believed to not substantially influence the overall stability of the structure. The effect of any prior thermal cycling was not addressed in this analysis.

ARH-C-19, 1977, *Analysis and Evaluation of the 241-U Concrete Waste Tank with Added Holes in Roof*, V. B. Watwood, Atlantic Richfield Hanford Company, Richland, Washington.

RHO-R-6, 1978, *Analysis of Underground Waste Storage Tanks 241-AX at Hanford, Washington*, prepared by URS/John A. Blume & Associates, Engineers, San Francisco, California for Vitro Engineering, Richland, Washington.

This document reports the results of analyses to assess the ability of the AX tanks after 14 years of service to structurally withstand all credible loading conditions and to maintain leak integrity during their use. Results from a seismic analytical study, including hydrodynamic loads, were combined with a thermal-creep analysis (ARH-C-11, 1976) to assess the overall adequacy of the structure. The seismic evaluation was performed for a 0.25-g peak horizontal ground motion that was developed in 1971 for the Safe Shutdown Earthquake (SSE) evaluation of the Fast Flux Test Facility (FFTF). The seismic motion in the vertical direction was taken as two-thirds of the horizontal.

There are four AX tanks in a 2x2 square array within the AX Tank Farm. The tanks are 49 feet high with an overburden soil height of approximately 7 feet. The outside diameter of each tank is 80 feet and there is an approximate minimum of 22-ft clear seperation between the walls of adjacent tanks.

The soil-structure interaction for an idealized isolated single tank and the surrounding soil were modeled as an assemblage of axisymmetric, thin, conical shell and toroidal solid finite elements. Thus, any tank-to-tank interaction and non-axisymmetric effects were neglected. Coupling between the sloshing liquid motion and the tank motion was assumed to be negligible because of the large frequency difference of the two motions, thus allowing separate calculation of each. A time history, equivalent linear, dynamical analysis was used in the soil-tank seismic interaction analysis through application of the AXIDYN computer program. The FLUSH computer program was used in the deconvolution analysis to obtain the base motion for the tank that envelopes the free-field ground motion. A quasi-static approach similar to that suggested by Housner was used to determine the hydrodynamic induced pressures from the sloshing fluid.

The concrete elastic and stiffness properties used in the seismic analysis were determined by considering the pattern of cracks and the reduction in elastic properties predicted from the thermal-creep analysis (ARH-C-11). The results from the seismic analysis were compared to the reserve capacity of the structure as estimated from the nonlinear thermal-creep and ultimate load analysis reported in ARH-C-11 (1976).

Although the resulting stresses from the seismic analysis were shown to be within the reserve capacity of the structure, as established from the thermal-creep analysis, the report states that because of a number of simplifying assumptions it is difficult to evaluate fully the seismic adequacy of the AX tanks for the postulated SSE motion. Some simplifying assumptions were unavoidable because of limitation in the then state of the art. Other idealizations, made necessary by the scope of work in this preliminary study, were believed to be conservative, except for the case of neglecting the influence of the adjacent tanks, which might result in amplified responses. Questions of uncertainties in the hydrodynamic analysis also need to be resolved to substantiate the reasonableness of the SEE-induced hydrodynamic loads utilized in the combined analysis. It was also suggested that the actual thermal loading history of the tanks be considered in the thermal-creep analysis, that actual concrete properties at elevated temperature be used, and that possible consequences of uncertainties associated with the material properties be addressed.

Some of the above uncertainties were addressed in the seismic analysis of the C106 tank (WHC-SD-W320-ANAL-002, 1995). The effect of tank-to-tank interaction was found not to be significant. This is inline with BNL-52361 (1995) which indicated that tank-to-tank interaction effects are not expected to be significant unless the spacing between tanks is less than one-half the tank radius. For the 100-Series SSTs the tank-to-tank spacing is 23 feet which is greater than one-half the tank radius (38.5/2 = 19.25 feet).

RHO-CD-638, 1979, *Engineering Study on Tanks 105-C and 106-C for Long Term Structural Integrity*, S. S. Bath, G. D. Campbell, and D. W. Everly, Rockwell Hanford Operations, Richland, Washington.

This report provides background and data required for the justification of a program to develop a passive cooling method for the high-heat generating Tanks 241-C-105 and -106. The report includes a discussion of tank structural integrity, the technically feasible alternatives for the stabilization of Tanks 241-C-105 and -106, and the reliability of the data used. The report draws on existing data from the then ongoing test program conducted by the Portland Cement Association to characterize the effect of long-term elevated temperature on the strength of Hanford-mix concrete and the ARH-C-11 (1979) structural analysis in establishing a maximum tank temperature limit of 350 °F.

RHO-SA-108, 1979, *Structural Evaluation of Existing Underground Reinforced Concrete Tanks for Radioactive Waste Storage*, F.R. Vollert, Rockwell Hanford Operations, Richland, Washington.

Presents results of nonlinear, time-dependent creep and ultimate load analysis of 241-U Farm Tanks for a soil cover depth of 7 feet at the dome apex, a unit soil weight of 115 lbf/ft^3, and a maximum waste temperature of 350 °F. The temperature profile was based on a heat transfer analysis with a salt cake heat generation of 0.40 Btu/hr-ft^3 and a soil conductivity of 0.20 Btu/hr-ft-°F. The creep analysis was conducted for a time period of 10 years. The creep and cracking was stationary for the given conditions at the end of this period. At the end of this creep period the soil load above the dome was factored in the subsequent ultimate load analysis. The approximate compressive strength of the concrete cylindrical wall, assumed to be 1,900 lbf/in^2 (compared to 3,000 lbf/in^2 28-day specified compressive strength) for the age and temperature conditions based on experimental data, was reached at a factored soil height of approximately 20 feet.

Shippell, R. J., Jr., G. H. Beeman, and C. A. Williams, August 1980, *Continued Analysis of the Load-Displacement Behavior Study of the 104-SX Tank on the 241-SX Tank Farm, Hanford, Washington*, Westinghouse Hanford Company, Richland, Washington.

RHO-CD-1485, 1981, *Description of Potential Failure Modes for Single-Shell Waste Tanks*, J. V. Egger, Rockwell International, Richland, Washington.

This report describes various conditions that could lead to the failure of Hanford single-shell high-level waste storage tanks. The report also includes an extensive bibliography of studies or information related to single-shell tanks. The report distinguishes between two types of "failures," structural and functional. Structural failure is defined as the inability of the waste tank to carry loads. Functional failure is defined as the inability of the waste tank to contain or isolate the high-level waste in the tank from the surrounding environment.

Letter, 1982, *Vapor Pressure, Single-Shell Tanks* (internal letter 65460-82-263 C. DeFigh-Price to D. W. Nelson, June 24), Rockwell Hanford Operations, Richland, Washington (see SD-RE-TI-035, Rev. 1, 1985).

SD-RE-TI-037, 1982, *Tank 241-A-106 Steady-State Heat Transfer Analysis*, Rev. 0, by K. E. Bruce, Rockwell Hanford Operations, Richland, Washington.

This report documents the results of a steady-state heat transfer analysis of Tank 241-A-106 for a waste-sludge peak temperature of 600 °F. The results were used to characterize the temperature distribution in the tank concrete structure for input to structural analysis that is documented in SD-RE-TI-012. This waste sludge peak temperature bounds all recorded SST temperature data. There are five SSTs with recorded waste temperatures in excess of 350 °F (see Table 2) and they are listed as follows:

Table A-10. Single-Shell Tanks with Recorded Waste Temperatures in Excess of 350 °F.

Tank 241-	Waste Peak Temperature °C (°F)
A-101	204 (399)
A-102	216 (420)
A-104	221 (430)
A-106	312 (594)
SX-107	199 (390)

The maximum waste sludge temperature of 594 °F in Tank 241-A-106 was recorded for only one day (May 15, 1963). However, the maximum sludge temperature averaged 440 °F for over 30 months and greater than 530 °F for approximately one month during which an essentially steady-state condition would have developed. Thus, the 600 °F steady-state heat transfer simulation provides reasonable, slightly conservative, thermal condition for a worst case structural analysis.

The steady-state thermal condition of the tank structure was predicted through an axisymmetric HEATING5 computer model for the following modeling conditions and assumptions:

- The axisymmetric steady-state heat transfer model simulation includes the waste sludge, supernate, vapor space, concrete tank, and surrounding soil.
- The heat generating bottom sludge layer was estimated to be 3.6 ft deep (118 Kgal) covered by a 24.4-ft (806 Kgal) layer of liquid supernate.
- Effect of steel liner was neglected because it is thin, has a high thermal conductivity (23 Btu/h-ft-°F), and hence would not significantly affect the results of the steady-state heat transfer solution. The effect of the liner becomes more important under extreme thermal transient conditions.
- The vapor space and supernate was defined to be at a constant (isothermal) temperature of 250 °F because of constant temperature boiling in the supernate. Since a steady-state condition is assumed, the inside surface of the concrete in contact with the vapor space and supernate is taken to be at 250 °F. This defined isothermal condition assumes sufficient steam loss through the vent system.
- Although steam was being vented, the air ventilating exhauster system was assumed to be off.

- Heat transfer through the waste sludge, structural concrete, and soil was assumed to be by conduction only. The thermal conductivity of the soil and concrete were taken as 0.25 and 0.54 Btu/h-ft-°F, respectively. *This is a departure from earlier SST thermal analyses (HW-47087) which conservatively assumed that the thermal conductivity of the soil and concrete were equal in order to simplify the calculation. The thermal conductivity of soil depends on the soil type and moisture content. For sand, the thermal conductivity ranges from 0.19 (dry) to 0.49 Btu/h-ft-°F (10-wt. percent water). For 200 East Area soil with 2 to 5% water a best estimate value of 0.33 Btu/h-ft-°F was given in HW-47087. The 0.25 Btu/h-ft-°F assumed in SD-RE-TI-037 appears to be a reasonable estimate for the soil thermal conductivity.*

- The thickness of the vertical cylindrical wall of the tank was taken to be 1.25 ft over the full height of the wall in the heat transfer model. *The actual thickness is 2 ft for 17 ft above the tank bottom, slopes to 1.25 ft over a 6 ft height, and then remains constant at 1.25 ft for 9.5 ft where the wall reaches the dome haunch (see drawing H-2-55911).*

- An adiabatic boundary in the soil was set at a radial distance of 51 ft from the center of the tank to simulate a tank in the middle of a large array of tanks at the same thermal condition. However, because the 241-A-106 tank is on the corner of a small two-by-three array of 6 tanks at different thermal conditions, this violates the axisymmetric assumption. The axisymmetric idealization should result in an over prediction of the maximum temperature in the concrete wall but an under prediction of the maximum thermal gradient. However, the under prediction of the thermal gradient is somewhat offset by the higher assumed steady-state sludge peak temperature of 600 °F compared to the actual temperature history of Tank 241-A-106, as discussed above.
- Forced convection heat transfer at the earth's surface was specified with a convection heat transfer coefficient of 2.0 Btu/h-ft^2-°F and an ambient air temperature of 70 °F.

- An isothermal temperature boundary was set 200 ft below the surface at 55 °F.

The recommend input to the structural analysis corresponded to the steady-state heat transfer solution for an assumed wet sludge layer. Studies had indicated that for wet sludge the thermal conductivity ranges from 0.8 to 1.0 Btu/h-ft-°F. In the recommended case for structural analysis, a sludge thermal conductivity of 1.0 Btu/h-ft-°F was assumed. The heat generation rate was determined to be 868,000 Btu/h to achieve a steady-state peak temperature of 600 °F in the sludge at the center of the tank. The maximum concrete wall temperature was predicted as 511 °F on the inside surface just above the footing. Along the inside surface of the wall, the temperature decreases to 452 °F at mid-height of the 3.6-ft thick sludge layer and decreases to a 250 °F isothermal temperature at the sludge-to-liquid interface. Thus, the maximum change in temperature vertically along the inside surface of the wall is predicted as (452 °F - 250 °F)/1.8 ft = 112 °F/ft over the top 1.8 ft of sludge. At the sludge-to-liquid interface level, the inside surface of the wall goes from 250 °F to 280 °F at 0.62 ft into the wall and to 297 °F at the outer surface of the 1.25 ft wall. This corresponds to a linearized thermal gradient of (280 °F – 250 °F)/0.62 ft = 47 °F/ft and (297 °F - 250 °F)/1.25 ft = 37 °F/ft, respectively.

The predicted temperature distribution throughout the tank steady-state heat transfer model was then applied to the structural evaluation model of Tank 241-A-106 with stress results given in SD-RE-TI-012.

SD-RE-TI-012, 1983, *Single-Shell Waste Tank Load Sensitivity Study*, Rev. A-0, prepared by A. L. Ramble, Rockwell Hanford Operations, Richland, Washington.

This document is the basis document for SST structural related operating limits. The report presents the results of structural analyses of the four sizes (550, 758, and 1,000 Kgal 100-Series and 55 Kgal 200-Series) of SSTs to assess their sensitivity to various service loads and to estimate their reserve capacities. The service loads included soil loads, equipment loads, hydrostatic loads, and elevated temperatures. A worst case thermal load was considered for the A-tank thermal-creep analysis based on the thermal history data of the 241-A-106 tank in which the sludge layer approached 600 °F in the early 1960's.

A load sensitivity analysis was conducted for the 550- and 1,000-Kgal 100-Series and the 55-Kgal 200-Series SSTs. The tank concrete structure (excluding steel liner) and surrounding soil were modeled by *elastic*, axisymmetric, *isotropic* finite elements with the general purpose ANSYS® finite element computer program. A transformed modulus technique was applied to represent the composite mechanical properties of the reinforced concrete. As such the strength in tension and in compression are equal and the section is restrained to remain elastic thus preventing cracking and any redistribution of the load. Table A-11 summarizes the loads and load combinations that were considered in the load sensitivity analysis.

Table A-11. Load Sensitivity Elastic-Analysis Load Conditions.

	Load condition		Tank: 1,000 Kgal	Tank: 550 Kgal	Tank: 55 Kgal
Base load	Dead		Self weight of structure		
Base load	Hydrostatic load	Liquid level (in.)	360	192	300
Base load	Hydrostatic load	Specific gravity	1.7	2.0	1.7
Base load	Live load (LL)	Diameter (ft) of circular area	30	30	None
Base load	Live load (LL)	Concentrated load (tons)	50	50	None
Base load	Thermal load	RHO-LD-171 (Btu/h)	50	50	None
Base load	Soil load	Unit weight (lbf/ft^3)	115	115	115
Base load	Soil load	Depth (h) to dome apex (ft)	6	7.3	11
Load case	1		Base load		Base load
Load case	2		Base with LL=100 tons		Empty tank, h=11 ft
Load case	3		Base with LL=200 tons		Empty tank, h=30 ft
Load case	4		Empty tank, ambient temp.		
Load case	5		Base with h=15 ft		
Load case	6		Base with h=30 ft		
Load case	7		Empty tank, ambient temp., h=15 ft		
Load case	8		Empty tank, ambient temp., h=30 ft		

The results from the linear elastic sensitivity analysis indicated essentially identical results for the 1,000 and 550 Kgal tanks. Thus, it was concluded that these results bracket the 758 Kgal tanks. The results were fairly insensitive to the hydrostatic and live loads. The soil and thermal loading have the greatest effect.

In addition to the load sensitivity study, local and global analyses were conducted.

Analysis Method

- Local
  Footing analysis: hand calculation, dead loads (soil and concrete) only for 100-Series tanks, 100 lbf/ft^2 live load was included for the 200-Series (55 Kgal) tanks.

- Global
  Tank analysis: thermal creep and ultimate load analysis and seismic finite-element analysis for 100-Series tanks. The following material properties were used in these analyses.

Table A-12. Material Properties Used in Thermal Creep and Ultimate Load Analyses.

Reinforcing Steel	
Yield strength (lbf/in^2)	40,000
Ultimate strength (lbf/in^2)	70,000
Modulus of elasticity (10^6 lbf/in^2)	29
Poisson's ratio	0.3
Coefficient of linear expansion (10^{-6} in/in-°F)	6.5
Concrete	
Compressive strength (lbf/in^2)	3,200
Splitting tensile strength (lbf/in^2)	480
Modulus of elasticity (10^6 lbf/in^2)	2.3
Poisson's ratio	0.22

Coefficient of linear expansion (10^{-6} in/in-°F)	3.86
<u>Soil</u>	
Specific weight (lbf/ft3)	115
Lateral pressure coefficient	0.4
<u>Liquid Waste</u>	
Specific gravity	2.0

Acceptance Criteria

In the footing analysis, ACI 318-77 with assumed in-situ compressive strength and rebar yield strength of 3,000 and 40,000 lbf/in^2, respectively and with 1.5 dead load and 1.7 live load factors.

In the thermal creep and ultimate load analysis, ACI 359-80 with assumed in-situ compressive strength and rebar yield strength of 3,200 and 40,000 lbf/in^2, respectively. The in-situ compressive strength of 3,200 lbf/in^2 was based on a statistical analysis of the results from the comprehensive laboratory test program (RHO-C-22, 1978), as well as, from core sample data from the PURX building structure and tank core samples from 241-SX-115 (RHO-RE-CR-2, 1982). *However, only data for temperatures up to 250 °F were used in determining the 3,200 lbf/in^2 compressive strength value. This value is not justified for the evaluation of tanks exposed to temperatures from 300 to 600 °F such as the Mgal SX, A, and AX tanks (see Table 2). A more appropriate approach would have been to consider the range of strength and modulus reduction with temperature as given in Figures 1 and 2 of BNL-52384 (1993).*

Results for 200-Series Tanks

Maximum soil depth = 17 ft 5 inches on basis of footing analysis.
Roof slab can carry 11 ft of soil plus 100-lbf/ft^2 live load.

Results for 100-Series Tanks

All tanks evaluated were found to be fairly insensitive to changes in the equipment or hydrostatic loads. The critical section for all tanks was the footing with the tank empty.

However, the analysis of the footings was based on hand calculations which did not consider the effect of the lateral soil pressure (assumed tank wall was hinged at footing) and the extent of the effective footing width included in the analysis appeared to be arbitrary due to the changing thickness of the bottom slab which acts as the footing. Although in all cases the rebar continues through from the footing into the cylindrical wall, providing a degree of moment transfer between the wall and the footing, the analysis assumes no moment transfer. Clearly, an assumption of total fixity would be overly conservative but the assumption of no fixity is unconservative. This assumption is somewhat offset by the conservative scaling used to determine the maximum allowable soil cover height to just meet the ACI 318 criteria which includes a dead load factor of 1.4. That is, in determining the allowable soil height above the dome apex, the total dead load (including the weight of the concrete plus the weight of the soil between the dome outer surface and a horizontal plane at the apex plus 7 feet of soil cover) was scaled rather than just the contribution from the 7-foot soil cover. Note that, the footing evaluations did not include any live load contribution, although it is a small percentage of the total load acting on the footing. Another potential unconservative assumption was that the soil unit weight was assumed to be 115 lbf/ft^3. More recent field measurements of the backfill soil over double-shell tanks (DSTs) has resulted in in- situ soil unit weights ranging from 110 to 122 lbf/ft^3 assuming an average 4 percent moisture content by weight (WHC-SD-WM-SOIL-001, 1994). A bounding value of 125 lbf/ft^3 has been imposed in dome load control evaluations of the DSTs where in-situ data is not available (HNF-IP-1266, 1997). There is a lack of specific soil density data for the SSTs.

In addition, the footing and lower wall region of many of the SSTs may be extensively cracked due to high thermal loads experienced over time from the stored waste. This cracking would reduce the effectiveness of the concrete to act as a secondary leakage barrier in the event of leakage from the steel liner. Currently, no waste additions and no soil additions are allowed for SSTs (HNF-SD-WM-TSR-006, 1998). However, a potential failure of the footing *region does not necessarily lead to a global structural failure of the tank and many of the tanks have already leaked to the soil. A rigorous analysis of the footing would require a more detailed system model of the tank and its interaction with the surrounding soil. A proper modeling of the soil-structure interaction is difficult because of the complex behavior of soils and the lack of specific soil data, which would require realistic bounding soil properties to be considered.*

With the above uncertainties in mind, the reported maximum *allowable* soil cover for the SSTs are summarized in Table A-13.

Table A-13. 100-Series Single-Shell Tank Maximum Allowable Soil Cover Height Based on Footing Analysis with 3,000 lbf/in^2 Concrete Compressive Strength.

Tank	Storage Volume (Kgal)	ACI Code Based Allowable Soil Height (ft)
B, BX, C, T, U	550	10.58
BY, S, TX, TY	758	12.5
SX	1,000	10.25
A	1,000	10.16
AX	1,000	16.0

Results from the thermal-creep and ultimate load analyses are summarized in Table A-14. In this case, the maximum soil height is based on dome failure or the maximum value at the end of the analysis. The duration of the creep analysis, the maximum wall temperature, heat-up rate, concrete compressive strength at failure load, and finite element computer program used in the analysis are identified in Table A-14.

Table A-14. 100-Series (75-ft Diameter) SST Thermal-Creep and Ultimate Load Analyses Input Data and Results.

Tank Farm	Storage Volume (Kgal)	Soil depth at crown (ft)		Analysis Parameters				Reference	Finite Element Program
		As-Built	Calculated Maximum at Ultimate Capacity or at End of Analysis	Time Period (days)	Maximum Wall Temp. (°F)	Heat-Up Rate (°F/day)	Concrete Compressive Strength at Critical Section (ksi)		
BX	530	7	20	33	387	21.1	3.2	SD-RE-TI-012 (1983)	SAFE-CRACK*
U	530	7	20	3,650	315	4.9	1.9	RHO-SA-108 (1979)	SAFE-CRACK*
BY	758	7	N/A	900	250	3.7	3.0	ARH-2883 (1973)	SAFE-CRACK*
SX	1,000	6	27	3,752	387	10.4	3.0	SD-RE-TI-012 (1983)	SAFE-CRACK* NONSAP-C*
AX	1,000	8	29	2,000	350	2.9	2.45	ARH-C-11 (1976)	SAFE-CRACK*
A	1,000	6	20	15	511	48.4	-	SD-RE-TI-012 (1983)	SAFE-CRACK*

Maximum wall temperature occurred at bottom section of tank wall. Failure predicted at transition of wall to upper haunch.

From the above it was concluded that all 100-Series tanks have adequate structural capacity to resist the applied soil and thermal loads. *Note that the calculated maximum soil depths in Table A-14 are the soil depths at maximum capacity and as such do not include any safety factor. A safety factor of <u>at least</u> 1.56 [ACI dead load factor (1.4) divided by ACI ϕ-factor (0.9) on bending moment capacity] is recommended.*

Results from the seismic analysis of the AX tank (RHO-R-6, 1978) to 0.25 g when combined with results from the thermal-creep analysis did not exceed the reserve strength capacity of the tank. Hence, it was concluded that the 100-Series tanks are capable of supporting the specified soil, thermal, and seismic loads.

Based on the above results the following operating and design limits were proposed for all SSTs.

Table A-15. Recommend Operating and Design Limits for All Single-Shell Tanks.

<table>
<tr><th colspan="2"></th><th>100-Series (75-ft Diameter)</th><th>200-Series (20-ft Diameter)</th></tr>
<tr><td colspan="2">Maximum soil cover depth (ft)</td><td>10</td><td>12</td></tr>
<tr><td colspan="2">Live load</td><td>100 tons over 10-ft radius area</td><td>50 tons over 10-ft radius area</td></tr>
<tr><td colspan="2">Hydrostatic</td><td>Filled to full storage capacity at 2.0 specific gravity</td><td>Filled to full storage capacity at 2.0 specific gravity</td></tr>
<tr><td rowspan="2">Maximum concrete temperature (°F)</td><td>Wall at base</td><td>380</td><td rowspan="5">N/A (these tanks have never been subjected to elevated temperatures, see Table 3)</td></tr>
<tr><td>Dome</td><td>250</td></tr>
<tr><td colspan="2">Maximum heat-up/cool-down rate (°F/day)</td><td>20</td></tr>
<tr><td colspan="2">Thermal gradients</td><td>No change</td></tr>
<tr><td colspan="2">Vapor pressure</td><td>No change</td></tr>
<tr><td colspan="2">Seismic</td><td>0.25 g</td><td>0.25 g</td></tr>
</table>

WHC-SD-WM-DA-062, 1990, *Analytical Assessment of Single-Shell Tanks 241-B-110 and 241-U-110 for Addition of Two 12-inch Risers*, Rev. 0, J. A. Ryan, Westinghouse Hanford Company, Richland, Washington.

WHC-EP-0347, 1991, *Summary of Single-Shell Tank Waste Stability*, G. L. Borsheim and N. W. Kirch, Westinghouse Hanford Company, Richland, Washington.

Bandyopadhyay, K. K., 1993, *A Review of the Technical Bases of Temperature Limits for High-Level Waste Storage Tanks at Hanford*, Brookhaven National Laboratory, Upton, New York.

WHC-SD-W340-ES-001, 1993, *Project W-340 Manipulator Retrieval System Tank 241-C-106*, D. A. Wallace, Westinghouse Hanford Company, Richland, Washington.

WHC-EP-0772, 1994, *Characterization of the Corrosion Behavior of the Carbon Steel Liner in Hanford Site Single-Shell Tanks*, Rev. 0, by R. P. Anantatmula, E. B. Schwenk (WHC), and M. J. Danielson (PNNL), Westinghouse Hanford Company, Richland, Washington.

WHC-SD-WM-DA-150, 1994, *Structural Sensitivity Evaluation of Single- and Double-Shell Waste Storage Tanks for Accelerated Safety Analysis - Phase I*, Rev. 0, by W. W. Chen, W. S. Peterson, L. L. Hyde, C. J. Moore, and T. W. Fisher, Westinghouse Hanford Company, Richland, Washington.

This sensitivity analysis of the 100-Series SSTs and DSTs addressed the effect on tank stresses of varying individual load parameters from a reference load condition. Inelastic analysis methods were utilized to account for concrete cracking and the resulting load/stress redistribution. The ANSYS® general purpose finite-element computer program was used with an updated concrete constitutive model. Generic finite-element tank models were developed for the 550-, 758-, and 1,000-Kgal SSTs based on the design-detail comparison given in WHC-SD-WM-TI-598 (1994). The range of loads selected for the sensitivity analysis was based on WHC-SD-WM-ES-286 (1994). Seismic loads were not addressed in WHC-SD-WM-DA-150. Uncertainties in concrete properties with increasing temperature and uncertainties in soil properties were discussed also.

Because the sensitivity analysis conducted in WHC-SD-WM-DA-150 is based on nonlinear analyses, the results from one load condition cannot be scaled and combined with another load condition to determine combined load stress results. Although the results of WHC-SD-WM-DA-150 can not be used directly for structural qualification of the tanks, the results do provide valuable insights into the sensitivities of the analytical model assumptions and load parameters considered. A summary of the WHC-SD-WM-DA-150 results is provided in WHC-SD-WM-SARR-012 (1994).

WHC-SD-WM-ES-286, 1994, *Single- and Double-Shell Tanks Load Report for Accelerated Safety Analysis*, Rev. 0, by D. L. Becker, Westinghouse Hanford Company, Richland, Washington.

This document identifies the loading parameters used in the original analyses and operational documents for the SSTs and DSTs. This document provides the basis for the range of loads considered in the sensitivity analysis completed in WHC-SD-WM-DA-150 (1994) and summarized in WHC-SD-WM-SARR-012 (1994).

WHC-SD-WM-SARR-012, 1994, *Accelerated Safety Analyses - Structural Analyses Phase I - Structural Sensitivity Evaluations of Single- and Double-Shell Waste Storage Tanks*, Rev. 1, D. L. Becker and L. L. Hyde, Westinghouse Hanford Company, Richland, Washington.

This report summarizes the results of the sensitivity analysis of the SSTs and DSTs given in WHC-SD-WM-DA-150 (1994). The tank stresses were tabulated by tank region and by individual load application within a load set. However, Code-based load combinations were not made for comparison to allowable Code-based limits in this Phase-I effort. The maximum tabulated stresses reported were taken from a group of finite elements within various tank regions. The tabulated stresses lack specific finite-element identification, location, and stress orientation, which would preclude accurate combination of the tabulated results. In addition, because the sensitivity analysis results are based on nonlinear concrete constitutive relations, the results from one load case cannot be scaled and combined with the results of another load case for comparison to Code-based allowable stress resultants. The range of analysis variables considered in the Phase-I effort for the SSTs is summarized in the table below.

Sensitivity Study Variable	**Range**
Soil Depth (at dome apex)	5 to 10 ft
Soil Density	101 to 130 lbf/ft^3
Soil Lateral Pressure (Rankine) Coefficient	0.5
Soil Subgrade Modulus	400 $lbf/in/in^2$
Waste Specific Gravity	1 to 2
Waste Depth	Empty to Full
Waste Temperature	70 to 350 °F
Vapor Pressure	-15 to +60 in. w.g.
Uniform Live Load	0 to 100 lbf/ft^2
Concentrated Live Load	0 to 100 tons
Creep Time	0 years

The soil stiffness assumed in the tank models was shown to greatly affect the foundation stress results. Further investigation was recommended to establish the appropriate soil stiffness for the tank models.

In the Phase-I analyses the temperature of the tank wall below the waste was assumed to be at the maximum waste temperature. The tank dome was assumed to be at the temperature of the vapor space above the waste. The temperature of the tank wall between the top surface of the waste and the dome was linearly transitioned over a 6-ft height of the wall. Although separate heat transfer analyses were conducted to investigate through-wall temperature gradients, the results were not applied to the tank stress analysis models. That is, the temperature through the wall was assumed constant in the tank stress analysis models. Hence, the effect of heat-up/cool-down rates on the tank stresses was not investigated in this Phase-I effort. In addition, creep and cyclic load effects were not considered in the sensitivity analysis of the SSTs.

Despite these limitations in the Phase-I effort, no changes in the then current operating limits were recommended, except that a maximum heat-up/cool-down rate of 3 °F/day was recommended not to be exceeded. Additional analysis was recommended, including qualification Code-based evaluations for both the SSTs and DSTs, which include realistic thermal conditions. Some additional analysis was conducted for DSTs but not for SSTs.

WHC-SD-WM-TI-598, 1994, *Single- and Double-Shell Waste Tank Design Comparisons at Hanford, Washington*, Rev. 0, T. W. Fisher, and D. J. Shank, Westinghouse Hanford Company, Richland, Washington.

The purpose of this report was to determine a technical basis for grouping similar tank designs for "generic" tank analyses of the SSTs and DSTs in support of the accelerated safety analysis effort (WHC-SD-WM-DA-150, 1994).

WHC-SD-WM-TI-623, 1994, *Static Internal Pressure Capacity of Hanford Single-Shell Waste Tanks*, Rev. 0, prepared by ADVENT Engineering Services, Inc. for Westinghouse Hanford Company, Richland, Washington.

This report documents the results of a structural analysis to estimate the static internal pressure capacity for the onset to failure of two sizes (553 and 1,000 Kgal) of generic Hanford Site SSTs. The onset-to-failure pressure was estimated through a nonlinear axisymmetric finite-element analysis of each tank under in-situ loading plus an internal static pressure. The resulting static onset-to-failure pressure was estimated at 14 and 11.6 psig for the 553 and 1,000 Kgal SST, respectively. These internal static pressures represent structural instability failure pressures. Permanent structural damage will likely occur before the onset-to-failure pressures are reached.

Thermal loading history and resulting potential degradation of concrete modulus and compressive strength with increasing temperature were not considered. Transient internal pressure loading with potential blow down through opening cracks in the concrete dome structures and resulting dynamic response of the tank structure also were not considered.

The ABAQUS® general purpose finite element computer program was used with the ANACAP-U® concrete constitutive model. A generic model bounding the construction details of each size of tank was used. The finite element model included the concrete vault, steel liner plus stiffener rings, and the surrounding soil. The minimum specified 28-day compressive strength of 3,000 lbf/in^2 was used in the nonlinear concrete material constitutive model. True elastic-plastic stress-strain curves were used for the tank steel liner (ASTM 283) and rebar reinforcement (A15, Grade 40). The surrounding soil was modeled with a Drucker-Prager elastic, perfectly plastic material constitutive model. A soil overburden depth of 7 feet at the dome apex with a soil density of 110 lb/ft^3 and a Rankine lateral soil pressure coefficient of 0.26 were assumed. Compression only elements were used to interface between the steel liner and the concrete and between the concrete and the soil elements. Hydrostatic pressure from the waste was applied assuming a uniform specific gravity of 1.7 with a waste depth of 363 inches for the 1,000 Kgal generic tank. For the 553 Kgal generic tank, a waste specific gravity of 2.0 was assumed for the waste from 0 to 35 inches and a specific gravity of 1.0 was assumed for the waste from 35 to 204 inches.

WHC-SD-W320-ANAL-001, 1995, *Tank 241-C-106 Structural Integrity Evaluation for In situ Conditions*, L. J. Julyk et al., Rev. 0 and 0A, Westinghouse Hanford Company, Richland, Washington.

This study evaluates the structural integrity of the high-heat 241-C-106 tank for its loading history to ACI 349 acceptance criteria and its reserve capacity to ultimate load under increasing uniform and concentrated load. The evaluation included a review of the related design documents; a simulation of the thermal and fill-and-drain history; application of concrete degradation relations with time-at-elevated temperature based on extensive test program for Hanford-mix concrete, and a design-by-analysis evaluation methodology. An American Concrete Institute (ACI) factored-load code check, thermal-creep analysis, and ultimate load analysis were conducted.

In Rev. 0 of the analysis the soil overburden was taken as 7 ft with an assumed unit weight 110 lbf/ft^3. This is in comparison to the actual soil overburden of 5 ft 7 in. and a backfill soil density of 100 lbf/ft^3 indicated on the design drawings. A 100-ton concentrated live load. The concentrated live load was applied at ground level over a 10-ft circular area at the center of the tank.

In Rev. 0A the soil overburden was reduced to its actual value of 5 ft 7 in. with an assumed unit weight 125 lbf/ft^3 since recent measurements for backfill over double-shell tanks indicated higher values. The DST soil density data appeared to be bounded by 125 lbf/ft^3. A uniform load of 40 lbf/ft^2 for snow and volcanic ashfall and a vapor pressure load of -15 in. water gauge (w.g.) for active ventilation were also introduced.

The general purpose ABAQUS® finite-element computer program was used with a user defined nonlinear concrete material constitutive model (ANACAP-U®) supplied by ANATECH Research, Inc. The ultimate load analysis methodology was benchmarked through a simulation of the 1:10 scale model test of the 241-A-105 tank that was reported in ARH-R-47 (1969). In the test, the uniform dome load at failure was reported as 5,400 lbf/ft^2 as compared to a calculated failure load of 3,900 lbf/ft^2. This discrepancy may be the result of the exposure of the test model to steam during the thermal test, prior to the ultimate load test. This may have resulted in an increase in the concrete compressive strength as discussed in ARH-R-120 (1972). However, this cannot be verified because no strength tests were conducted after the thermal test.

Although the C106 tank was not designed for self-boiling waste, historical records indicated that the tank had experienced temperature excursions in excess of boiling requiring water additions to control the temperature. An upper-bound thermal history for the C106 tank was generated based on available data for the tank. The resulting calculated time dependent temperature distribution in the concrete from the heat transfer analysis was applied to the structural analysis along with the corresponding fill and drain load history. Peak temperatures up to 310 °F were predicted to occur in 1979 at the center of the bottom concrete slab with corresponding temperatures in the dome near 220 °F. Because the temperatures anywhere in the concrete did not exceed 150 °F for at least 15 years after construction, the concrete strength was assumed to be higher due to aging than the initial 28-day minimum specified value of 3,000 lbf/in^2. The lower-bound 95% confidence band relation for concrete and modulus degradation with time at temperature, developed from the analysis of the test data reported in PNL-7779 (1988), was used in the thermal-creep analysis of the C106 tank for the upper-bound thermal history. To address the observed mismatch in thermal expansion between Hanford-mix concrete and the reinforcing steel a lower bound value of 1.6 x 10^{-6} and 6.5 x 10^{-6} in/in-°F was selected for the concrete and the steel, respectively. At the end of the thermal-creep analysis the structure was evaluated to the ACI 349 criteria and found acceptable. Then the soil load was increased until failure of the concrete structure was predicted to estimate the reserve capacity of the structure. A minimum safety factor of 4.8 was predicted with the revised Rev. 0A loading.

Report No. 941101-001, 1994, *Review and Parametric Studies for Tank 241-C-106 Dome Structure*, Rev. 0, prepared by A. Ghose, ARES Corporation for Westinghouse Hanford Company, Richland, Washington.

This document reports the results of a study to evaluate the effect of potential cracks or groves observed in a 1994 remote video inspection of the under side of the C106 tank dome. The indicated "irregularities" were believed to be the result of minor amounts of shifting or sagging of the trapezoidal plyform sheets that were used in the construction of the tank, and do not represent any compromise in the as-designed structural strength of the tank. However, in view of possible concerns about the potential loss of concrete section, or crack initiation at the location of these groves, a series of parametric structural evaluations of the tank were performed. The ADINA® finite element computer program, which has been used to analyze a wide variety of reinforced concrete structures, was used in this evaluation of the C106 tank.

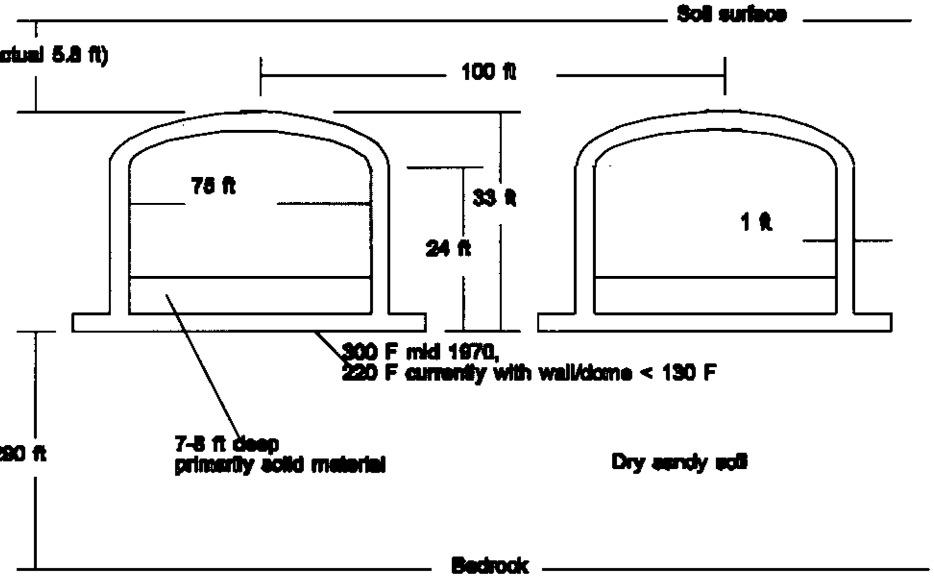
An ADINA® model of the 1:10-scale expermental model of Tank 241-A-105 (ARH-R-47, 1969) was used to benchmark the ADINA® program. The ultimate load results for the experimental model was 5,400 lbf/ft^2 while the ADINA® analysis of the scale-model test predicted an ultimate load of 4,644 lbf/ft^2. The load-deflection curve from the ANDIA® model showed good agreement with the test data under increasing load. Thus, it was concluded that the ADINA® constitutive model for concrete, its post-yield post-crushing capability, and the rebar post-yield model can accurately represent the structural behavior of a reinforced concrete tank structure such as C106.

The subsequent analysis of the C106 tank for normal and seismic loads with and without groves in the dome, applied the degraded concrete properties from temperature history as developed in WHC-SD-W320-ANAL-001 (1995). It was concluded that the dome surface irregularities observed during the 1994 video inspection have no significant impact on the as-designed structural integrity of the tank.

WHC-SD-W320-ANAL-002, 1995, *Seismic Evaluation of Tank 241C106 in Support of Retrieval Activities*, D. A. Wallace, et al., Rev. 0 and 0A, Westinghouse Hanford Company, Richland, Washington.

This report documents the seismic analysis of the high-heat 241-C-106 tank and the structural evaluation for the combination of seismic plus in situ loading history as determined in WHC-SD-W320-ANAL-001 (1995). A comprehensive analysis was performed using the state-of-art three-dimensional SASSI computer program to model the soil-structure interaction. The effect of adjacent tank-to-tank interaction was also evaluated. Figure A-1 shows some of the important parameters of the 241-C-106 tank. Rev. 0 and 0A were for an assumed soil overburden unit weight of 110 and 125 lbf/ft^3, respectively. ACI 349-90 requirements were satisfied in all cases. Highlights of the analysis and the results are summarized below.

Figure A-1. 530-Kgal Single-Shell Tank 241-C-106 Located in 200 East Area of Hanford Site.



REVISION 0 (ASSUMED SOIL OVERBURDEN UNIT WEIGHT OF 110 LBF/FT^3)

SEISMIC ANALYSIS

Seismic Excitation

- Non-reactor Safety Class 1 (high-hazard) structure
- Newmark-Hall response spectra corresponding to 7% damping anchored at 0.20 g horizontal peak ground acceleration (Hanford Plant Standard HPS-SDC 4.1, Rev. 12, 1993).
- Applied synthetic acceleration time histories (Weiner and Rohay 1992) corresponding to spectra with acceleration amplitudes scaled by a factor of 1.08 (ASCE 4-86 and NUREG-0800) to ensure enveloping of the design response spectra.
- Soil-structure Interaction (SSI) analysis assumes that horizontal ground motion is due entirely to vertically propagating shear waves.
- Vertical control motion set at 2/3 of horizontal (ASCE 4-86 and UCRL-15910).
- SSI analysis assumes that vertical ground motion is due entirely to vertically propagating compression waves.
- Peak responses from the three orthogonal excitation directions are combined via square root of sum of squares (SRSS).

- Inelastic demand-capacity ratio F_{μ} is taken as 1.0 (UCRL-15910 approach is used in current version of HPS-SDC-4.1).

3-D SASSI (Lysmer et al. 1991) Finite-Element Models

• Single tank horizontal excitation	1/4-model with vertical plane of symmetry parallel to excitation direction and vertical plane of anti-symmetry perpendicular to excitation direction (anti-symmetry verified against 1/2-model)
• Single tank vertical excitation	1/4-model symmetry
• Tank-to-tank interaction horizontal excitation	1/2-model symmetry with anti-symmetry plane between tanks (no significant difference between model with and without soil extended to anti-symmetry plane)

Tank Structural Model

- Isotropic elastic shell elements
- Transformed reinforced concrete section properties
  - Best-estimate properties based on predicted best-estimate in situ state at end of 55 years of service
  - Rebar (meridional)
  - Section cracking
  - Material degradation with time and temperature
  - Lower-bound properties (*soft tank* model) based on assumed wide-spread cracking with concrete modulus equal to 87% of best-estimate values (based on square root of ratio of lower-bound to best-estimate compressive strength)
- Damping for concrete specified as 7% of critical damping in accordance with BNL 52361 for response level 2 reinforced concrete structure (demand-to-code allowable ratio in range of 0.5 to 1.0).
- Hydrodynamic effect of waste from horizontal seismic excitation
  - For SST, effect of hydrodynamic wall pressures may counteract response from dynamic earth pressure (depending on phasing)
  - BNL 52361 recommends conservatively to consider the two opposing wall pressures separately
  - Waste modeled as lumped masses
  - Only impulsive effect is considered via cosine tributary mass distribution (lumped mass goes to zero as angle between excitation direction and tank node approaches 90°).
  - Convective sloshing mode neglected
  - Convective component for invisid liquid is generally small relative to impulsive component (BNL 52361)
  - Waste is primarily viscous sludge (expected to produce lower sloshing and greater mass participation in the impulsive mode [BNL 52361]).
  - Tank less than half full, eliminates possibility that slosh height would impact dome.
- Hydrodynamic effect of waste from vertical seismic excitation
  - Increased density of tank base material
  - Hydrodynamic pressure on tank wall neglected
- Steel liner not modeled (not attached to concrete).
- Near-field soil included in structural model with properties taken equal to corresponding free-field properties.

Pits and Risers

- Tank C106 has three reinforced concrete pits located above the tank dome separated by a layer of soil.
  - Steel pipe risers span vertically from the tank dome to the floor of each pit.
  - Top end of each riser is coupled with the pit floor in horizontal direction only.
- Risers and crude representations of the pits are added to the model to:
  - Assess seismically induced riser response for use in structural evaluation of risers.
  - Calculate pit-floor response spectra for use in evaluation of allowable loads for pit floors.

Soil Properties

- Grout Vault soil test data (Dames & Moore 1988)
  - Best-estimate properties
  - Upper-bound properties = $(1 + C_v)$ best-estimate properties
  - Lower-bound properties = best-estimate properties / $(1 + C_v)$

- If C_v cannot be determined in a probabilistic manner, ASCE 4-86 requires a C_v value not less than 0.5
- C_v taken as 1.0 because site specific properties were not available (BNL 52361 and NUREG-0800, Standard Review Plan). This may be overly conservative per EPRI NP-7395, *Guidelines for Soil-Structure Interaction Analysis*, pg 3-2.
  - Upper-bound properties = 200% best-estimate properties
  - Lower-bound properties = 50% best-estimate properties
- Strain dependent shear modulus and damping (Seed and Idriss 1970) determined in SASSI/SHAKE
  - Moduli are adjusted to respective bounding condition before SHAKE is run
  - Control motion is specified at outcrop of first competent soil layer (shear wave velocity greater than 750 ft/s)
  - Upper-bound shear modulus degradation curve
  - Lower-bound damping curve
- Poisson's ratio taken as constant equal to 0.44 based on wave speed data at depths greater than 9 ft (data at shallower depths appeared suspect).
- Soil damping calculated by SASSI/SHAKE range from 1 to 4%.
- Bounding conditions of soil properties and tank stiffness need not be considered in combination (BNL 52361)
  - Best-estimate soil properties are used with *soft tank* model (BNL 52361)
  - Upper-bound tank stiffness is not considered as a separate case.

SEISMIC RESPONSE RESULTS

Soil Properties Variation

- Lower-bound soil properties control in moment response.
- Lower-bound soil properties control in axial load response except for meridional axial load in wall and dome region where upper-bound soil properties control.

Controlling Soil Properties and
Percent Change in Maximum Response Relative to Response with Best-Estimate Soil Properties
for Horizontal Excitation without Tank-to-Tank Interaction

	Meridional		Circumferential	
	Axial	Moment	Axial	Moment
Base	L-B(+30%)	L-B(+21%)	L-B(+20%)	L-B(+39%)
Wall	U-B(+8%)	L-B(+45%)	L-B(+103%)	L-B(+48%)
Dome	U-B(+46%)	L-B(+5%)	L-B(+83%)	L-B(+1%)

U-B is upper-bound soil properties
B-E is best-estimate soil properties
L-B is lower-bound soil properties

Tank-to-Tank Interaction (TTI)

- The effect of TTI is relatively small but varies with location.
- Magnitude of peak response with TTI is significantly greater with lower-bound soil properties except for response of
  - meridional axial load in wall (+10%) and dome (+18%) and
  - circumferential moment in base (+7%) and dome (+12%) where response with best-estimate soil properties are slightly greater relative to response with lower-bound soil properties.

Percent Change in Maximum Response with TTI Relative to Response without TTI for Horizontal Excitation and Lower-Bound Soil Properties

	Meridional		Circumferential	
	Axial	Moment	Axial	Moment
Base	+7%	-4%	+4%	+22%
Wall	+15%	+3%	+8%	+6%
Dome	+20%	+21%	+45%	+1%

Tank Stiffness Variation

- Best-estimate tank stiffness is controlling.
- Meridional axial load response is
  - 3-times larger in the base for the best-estimate tank stiffness and
  - 50% and 129% larger in the wall and dome, respectively, for the lower-bound tank stiffness.
- Circumferential axial load response is 5- to 6.5-times larger for the best-estimate tank stiffness.
- Peak bending response in the base is 66% and 49% larger in the meridional and circumferential directions, respectively, for lower-bound tank stiffness.
- Peak bending response in wall and dome is 3- to 4-times larger for the best-estimate tank stiffness.

Percent Change in Maximum Response with Lower-Bound Tank Stiffness Relative to Response with Best-Estimate Tank Stiffness for Horizontal Excitation and Best-Estimate Soil Properties

	Meridional		Circumferential	
	Axial	Moment	Axial	Moment
Base	-69%	+66%	-85%	+49%
Wall	+50%	-74%	-83%	-76%
Dome	+129%	-64%	-81%	-73%

Horizontal versus Vertical Excitation

- Greatest effect of vertical excitation relative to horizontal is in dome
  - Meridional bending response increased by 70%
  - Circumferential bending response increased by 220%
  - Axial load response increased by factor from 7 to 20
- Response spectra amplification at dome apex from horizontal excitation is negligible.
- Response spectra amplification at dome apex from vertical excitation is considerable for
  - Frequencies greater than 4 Hz with peak of 1.0 g at ~12 Hz for best-estimate soil properties (without 100-ton mass at soil surface) and
  - Frequencies greater than 3 Hz with peak of 1.8 g at ~9 Hz for lower-bound soil properties (with 100-ton mass at soil surface).
  - All response spectra calculated for 7% damping.

Percent Change in Maximum Response for Vertical Seismic Excitation Relative to Response from Horizontal Excitation and Lower-Bound Soil Properties

	Meridional		Circumferential	
	Axial	Moment	Axial	Moment
Base	+15%	+50%	-56%	-10%
Wall	+267%	-57%	-77%	-69%
Dome	+2000%	+70%	+750%	+200%

Effect of Remediation 100-ton Live Load Mass

- 100-ton equipment mass applied at soil surface at dome apex.
- Vertical seismic excitation
- Lower-bound soil properties
- Effect is minor in tank base (+1%) and wall (+15%)
- Effect on dome:
  - Bending is minor (+15%) except for circumferential moment near dome apex where demand increases by a factor of 7.5
  - Axial load response increased by 65%

Effect of Waste for Horizontal Excitation

- Axial forces sometimes larger for empty tank
- Moments are generally larger when waste is considered
- Method used is approximate and conservative
  - Mass is distributed around full circumference
  - In reality, impulsive forces are applied to only one-half the tank wall at any given point in time.

Percent Change in Maximum Response with Waste Relative to Response without Waste

	Meridional		Circumferential		Comment
	Axial	Moment	Axial	Moment	
Base	+18%	+34%	-4%	+36%	Increases with distance from center
Wall	-15%	+50%	-11%	+80%	Varies along wall height
Dome	-14%	~0%	~0%	~0%	Small effect

Seismic versus Nonseismic Response

- SRSS of horizontal and vertical excitation seismic response loads includes effect of
  - Tank-to-tank interaction
  - Impulsive hydrodynamic waste effect
  - 100-ton mass load at soil surface directly over dome apex.
- Unfactored nonseismic response loads include
  - Deadweight of tank
  - Hydrostatic waste load
  - Lateral earth pressure
  - In situ temperature
  - Soil overburden
  - 40-lbf/in^2 distributed live load at soil surface
  - 100-ton concentrated live load at soil surface directly over dome apex.
- Seismic response loads are generally less than unfactored nonseismic response loads except for in-plane shear

Ratio of SRSS Seismic Response Loads to Total Nonseismic Response Loads for Lower-Bound Soil Properties

	Meridional		Circumferential		Shear	
	Axial	Moment	Axial	Moment	Transverse*	In-plane
Base	0.42	0.05	0.14	0.05	0.06	15.4
Wall	0.75	0.75	0.17	0.35	0.47	18.5
Dome	0.23	0.18	0.23	0.19	0.19	1.57

*Transverse shear stresses were calculated from moment derivatives based on shell equations.

STRUCTURAL EVALUATION RESULTS

Structural Acceptance Criteria

- Capacity of reinforced concrete tank structure
  - ACI 349-90
  - Material properties used in computing code-based capacities are based on 95% exceedance values estimated from tests of materials used in facility with consideration of degradation of concrete and reinforcement from long-term exposure to elevated temperature.
  - Lower-bound in situ concrete compressive strength range from approximately 3,400 lbf/in^2 in tank base to 4,500 lbf/in^2 in the haunch and dome.
  - Inelastic demand-capacity ratio F_μ (URCL-15910) taken conservatively as one.
- Capacity of steel risers
  - ASIC allowable stress design approach
  - Plastic design capacity factor (BNL 52361) conservatively neglected.

Evaluation

- Reinforced concrete tank structure
  - Worst-case seismic condition based on:
    - Lower-bound soil properties.
    - Tank-to-tank interaction.
    - Impulsive hydrodynamic waste.
  - SRSS of horizontal and vertical seismic excitation demands.
  - Positive and negative values of seismic demands considered.
  - Sign of nonseismic demands retained.
  - Seismic combined with nonseismic demands.
  - Demands compared to capacities at critical tank sections.
    - Moment/axial load interaction (amount of reinforcement for P-M capacity curves was discounted to account for minimum reinforcement required for in-plane shear demands where required by ACI Code procedures).
    - Transverse shear (minimum demand/capacity ratio = 0.82 at bottom of wall).
    - In-plane shear (sufficient reinforcement available).
    - Twisting moments (not a concern).
    - Construction joints (shear friction capacity).

Wall joint	Minimum demand/capacity ratio
Upper	0.6
Lower	0.85

  - ACI 349 requirements were satisfied.

  - Seismic load combinations (seismic loads + unfactored nonseismic load combinations) were less severe than nonseismic load combinations (with load factors).
  - Lower wall region critical region.
- Steel risers
  - Only seismic loads considered.
  - ASIC requirements were satisfied.

REVISION 0A (ASSUMED SOIL OVERBURDEN UNIT WEIGHT OF 125 LBF/FT^3)

The worst case seismic condition from Rev. 0 was re-evaluated for an assumed soil overburden unit weight of 125 lbf/ft^3. The analysis was based on lower-bound soil properties and included tank-to-tank interaction, impulsive hydrodynamic waste effects, and vertical live load. The seismic demand increase was in the range of 20 to 30 percent and as high as 160 percent at some locations of the structure. The resulting revised seismic response was then combined with the response to nonseismic loads with increased soil overburden and again compared to ACI 349-90 requirements. Although margins decreased at some locations as a result of the assumed increase in soil density, all tank locations evaluated maintained an acceptable margin for the worst case conditions.

The results from WHC-SD-W320-ANAL-002 are specific to the 241-C-106 tank but can be considered to envelop the 530 Kgal SSTs (B, C, T, U and BX) since these tanks are of the same design (see Table 1) and the thermal history of the 241-C-106 tank is bounding for these tanks (see Table 2). However, these results cannot be extended to bound all other SSTs because of design differences and because the thermal history of the 241-C-106 tank is not necessarily bounding to the remaining SSTs.

WHC-SD-W320-ANAL-003, 1995, *Tank 241C106 Structural Evaluation in Support of Project W320 Retrieval*, D. A.Wallace, et al., Rev. 0 and 0A, Westinghouse Hanford Company, Richland, Washington.

WHC-SD-TWR-RPT-002, 1996, *Structural Integrity and Potential Failure Modes of the Hanford High-Level Waste Tanks*, Rev. 0A, by F. C. Han, Westinghouse Hanford Company, Richland, Washington.

This report provided a review of the structural integrity analyses of the single- and double-shell tanks and their potential failure modes under various postulated accident scenarios as a basis for the consequence analyses in the BIO. The failure modes analysis relies on WHC-SD-TWR-RPT-003 (1996). The evaluation relies on the results from the existing design support documentation for the tanks. The review of the historical design analyses is cursory and the failure calculations are generic and have not addressed tank cyclic-thermal degradation.

The report points out the need to establish and maintain the thermal histories of the tanks because of their importance in evaluations of the current structural integrity of the tanks. Some currently outstanding thermal issues are identified relating to the fill/drain cycling (heat-up/cool-down rates) of the tanks. The resulting thermal gradients under high heat-up/cool-down rates can damage the single-shell tank concrete structure. Analysis models to date (WHC-SD-WM-DA-150, 1994) have not had the ability to accurately model these thermal transient conditions. Also there is a need to establish a consistent correlation to model the strength and modulus properties as a function of time-at-temperature and the creep behavior of the Hanford-mix concrete at elevated temperatures (WHC-SD-WM-DA-153, 1994).

The report concludes that these tanks are adequate for normal operating loads with current operating restrictions with considerable safety margin.

WHC-SD-TWR-RPT-003, 1996, *DELPHI Expert Panel Evaluation of Hanford High Level Waste Tank Failure Modes and Release Quantities*, Rev. 0, by F. C. Han, (complied and edited by L. Leach, independent consultant), Westinghouse Hanford Company, Richland, Washington.

This document describes a *qualitative* assessment of the failure modes of the tanks under accident conditions. This report was prepared to support the TWRS BIO. The experts panel concluded that the failure modes associated with the seismic event are minor in comparison to the off-site release accompanying the failure due to hydrogen deflagration. However, the seismic event could trigger a hydrogen deflagration.

The conclusions based on the overload/collapse thresholds identified during the proceedings lack the proper documentation to justify their use as operational limits or as justification to increase tank dome loading above that which is currently in place.

WHC-SD-WM-TI-775, 1996, *Structural Assessment of Accident Loads*, Rev. 0, by G. R. Wagenblast, ICF Kaiser Hanford Company for Westinghouse Hanford Company, Richland, Washington.

This report addresses specific potential accident load conditions for selected Hanford Site underground waste storage tanks based on failure load analysis. The evaluations are directed primarily to miscellaneous underground storage and process tank but does consider SSTs and DSTs for selected accidents. All structural assessments were performed using simplified bounding methods and did not necessarily include the effect of the load history for these existing tank structures. *Thus, the results should be considered as rough estimates of the failure loads for the accident scenarios considered.*

RLCA, 1996, *Evaluation of Hanford High Level Waste Tank Failure Modes for Seismic Loading*, prepared by Robert L. Cloud & Associates, Inc., Berkeley, California for U.S. Department of Energy - RL, Richland, Washington.

This report was an independent review of the seismic failure assumptions provided in the Delphi study (WHC-SD-TWR-RPT-003, 1996). Additional *simplified* seismic analyses were performed for both the SSTs and DSTs using the ANSYS® finite element program and hand calculations based on BNL 52361 (1995). The RLCA analysis conservatively neglected soil-structure interaction effects but followed the guidelines, methods, and criteria of DOE Standard DOE-STD-1020-94 (1994), *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities* and BNL 52361 (1995), *Seismic Design and Evaluation Guidelines for The Department of Energy High-Level Waste Storage Tanks and Appurtenances.* The ground spectral shapes applied were from Reg. Guide 1.60 rather than HNF-PRO-097 (1997).

The conclusions of the report agreed with the conclusions of the Delphi report. The results of the RLCA analyses confirm that the SSTs would not fail catastrophically until about 0.8 g. Tank failure would result from a lack of moment capacity close to the bottom of the concrete tank wall and the lack of hoop tension capacity close to the base of the liner.

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APPENDIX B

TANK 241-C-102, -104 AND -106 HISTORICAL WASTE LEVELS, TEMPERATURES, AND DOME ELEVATION SURVEY DATA

Waste level and temperature data obtained from WHC-SD-WM-ER-313 (1996) and dome elevation survey data obtained from N. J. Scott-Proctor of Technical Operations, Lockheed Martin Hanford Corporation.

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Figure B-1. Tank 241-C-102 Waste and Level History 1946-1995.

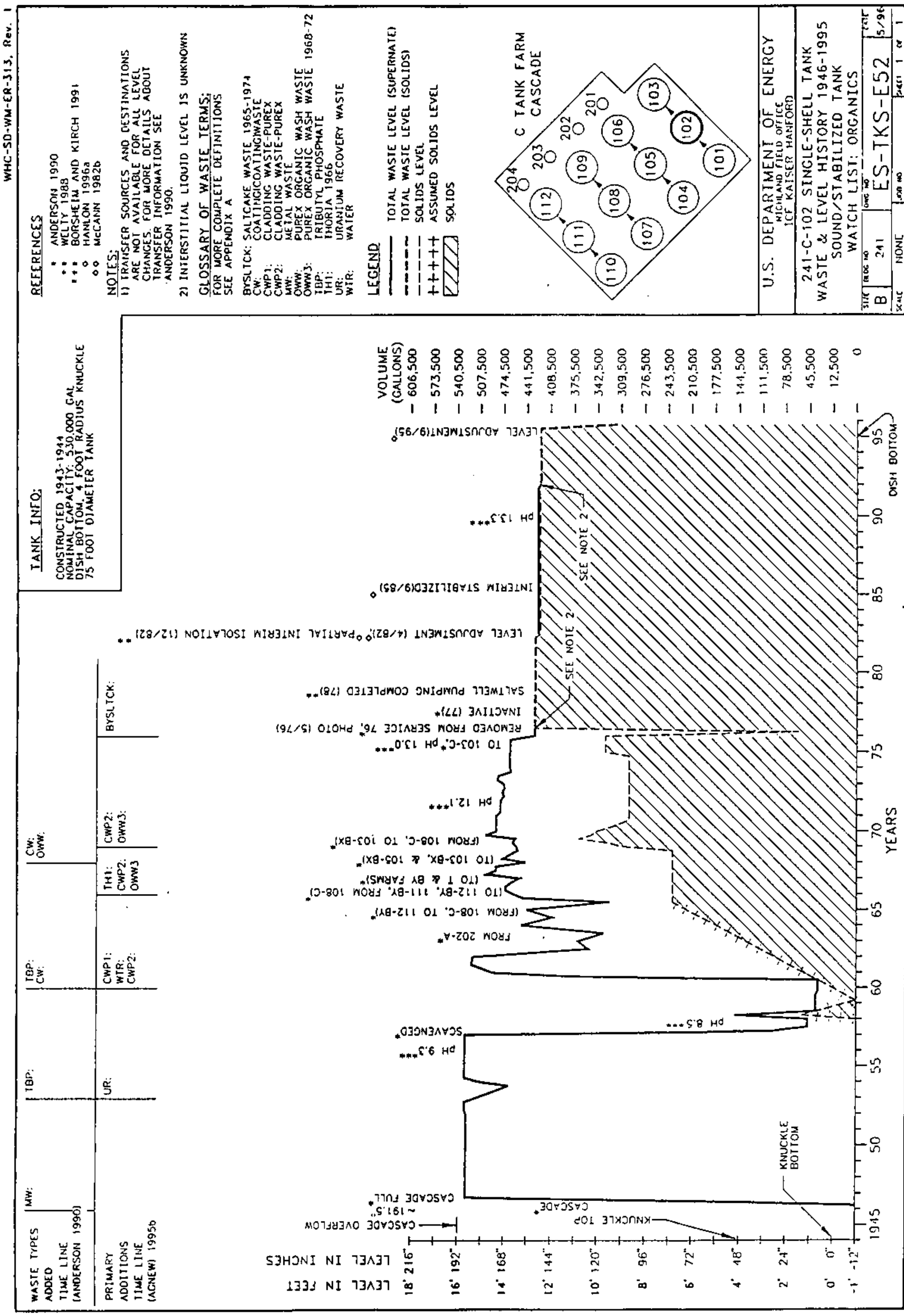


Figure B-2. Tank 241-C-104 Waste and Level History 1946-1995.

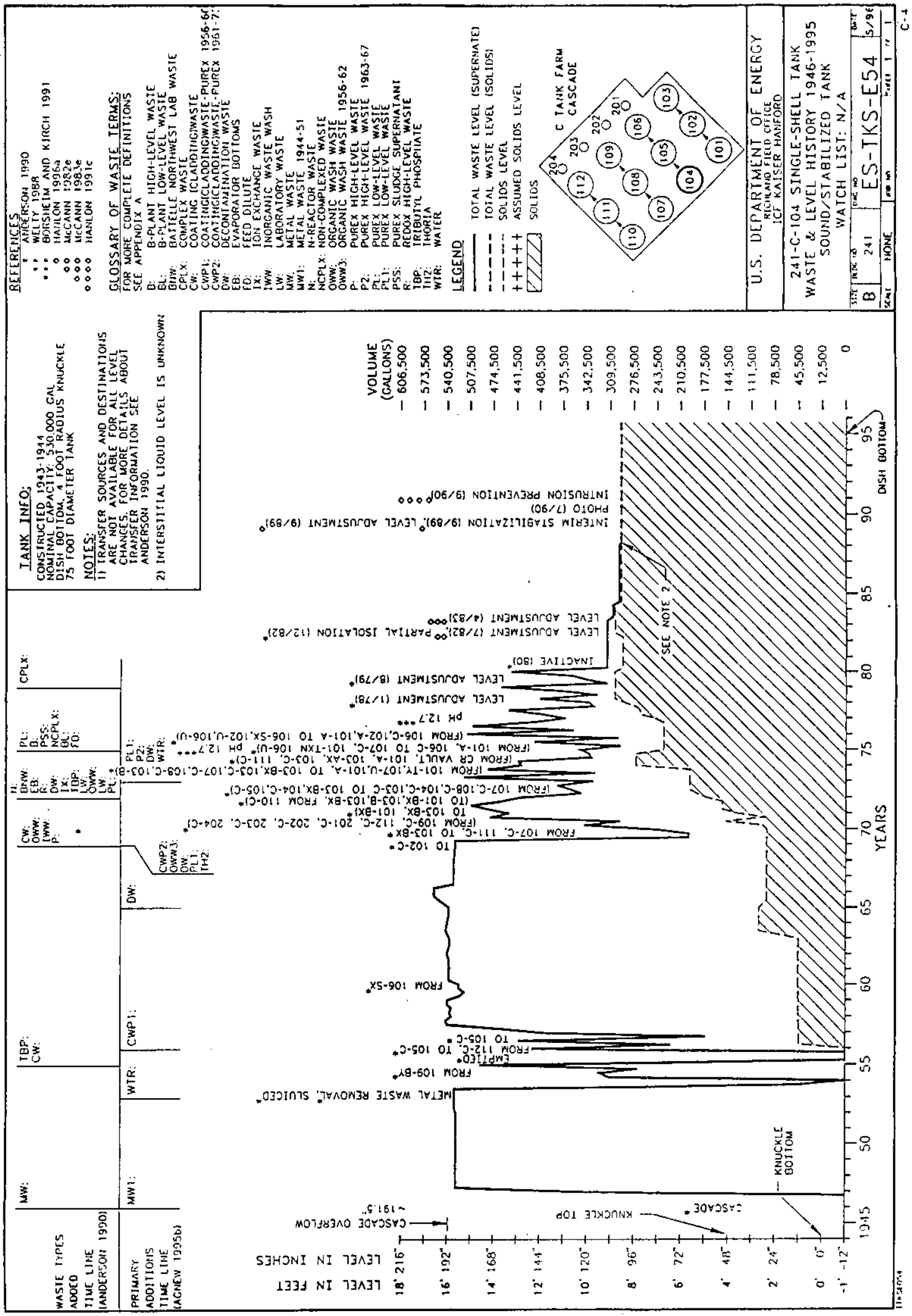


Figure B-3. Tank 241-C-106 Waste and Level History 1947-1995.

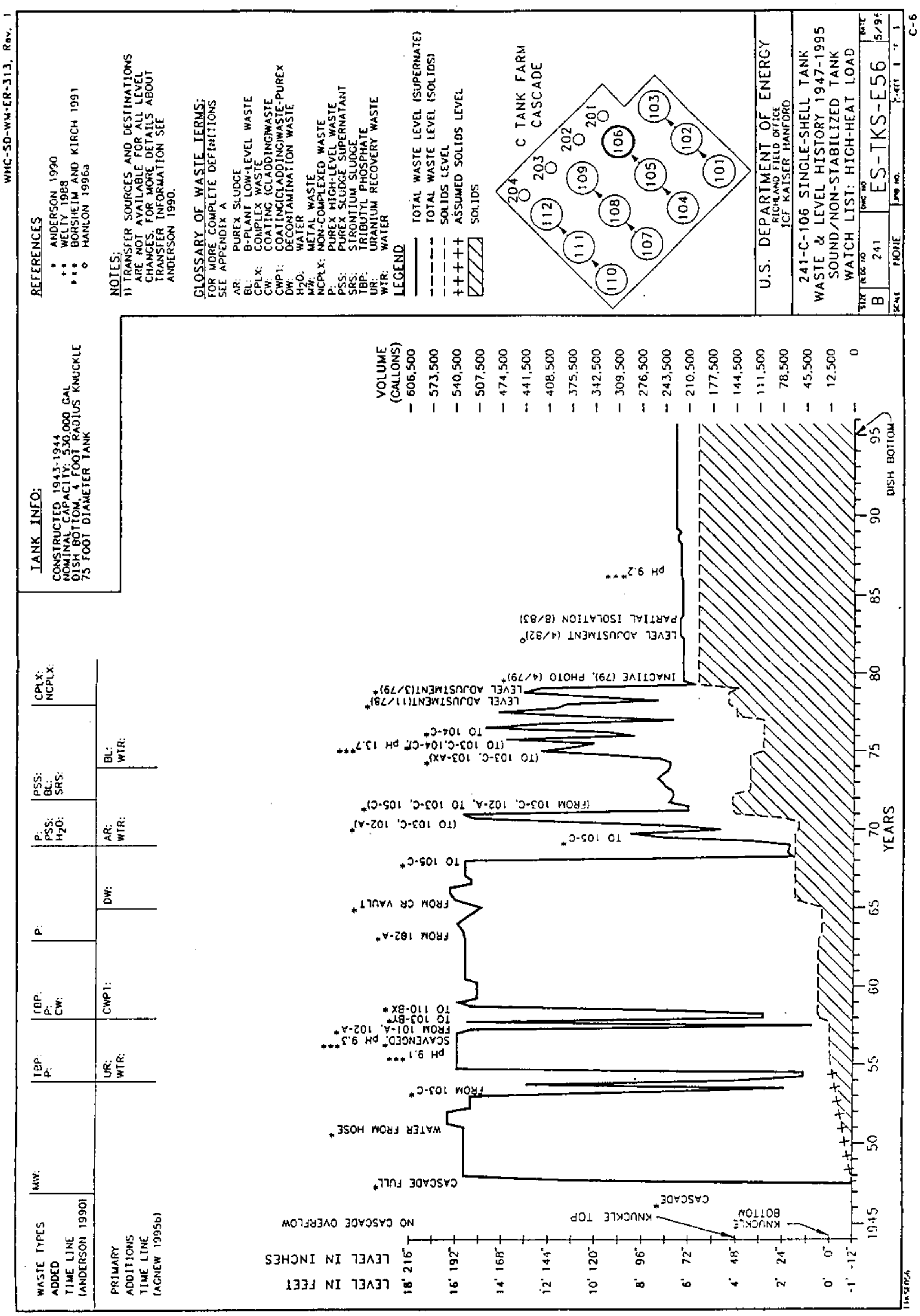
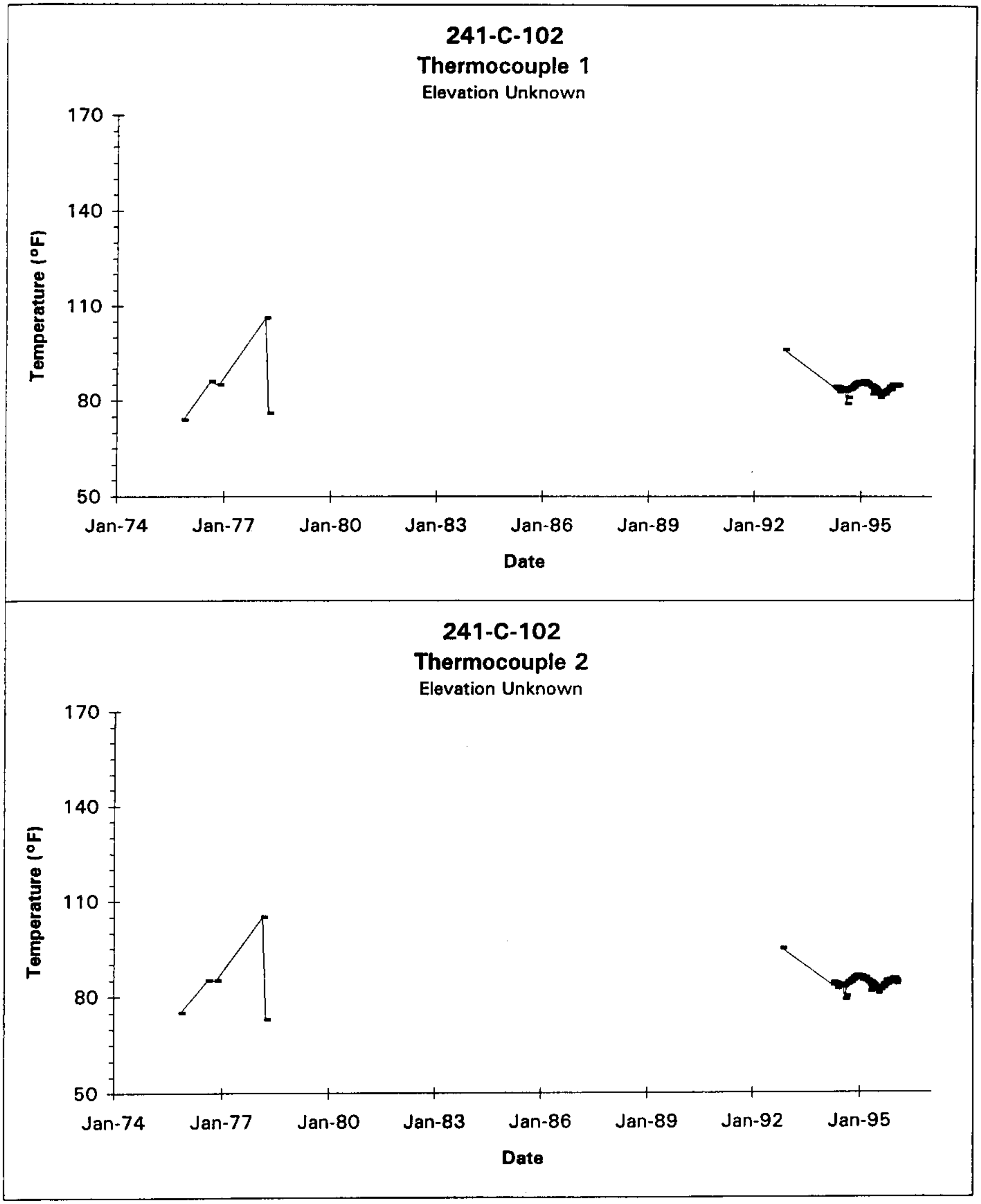
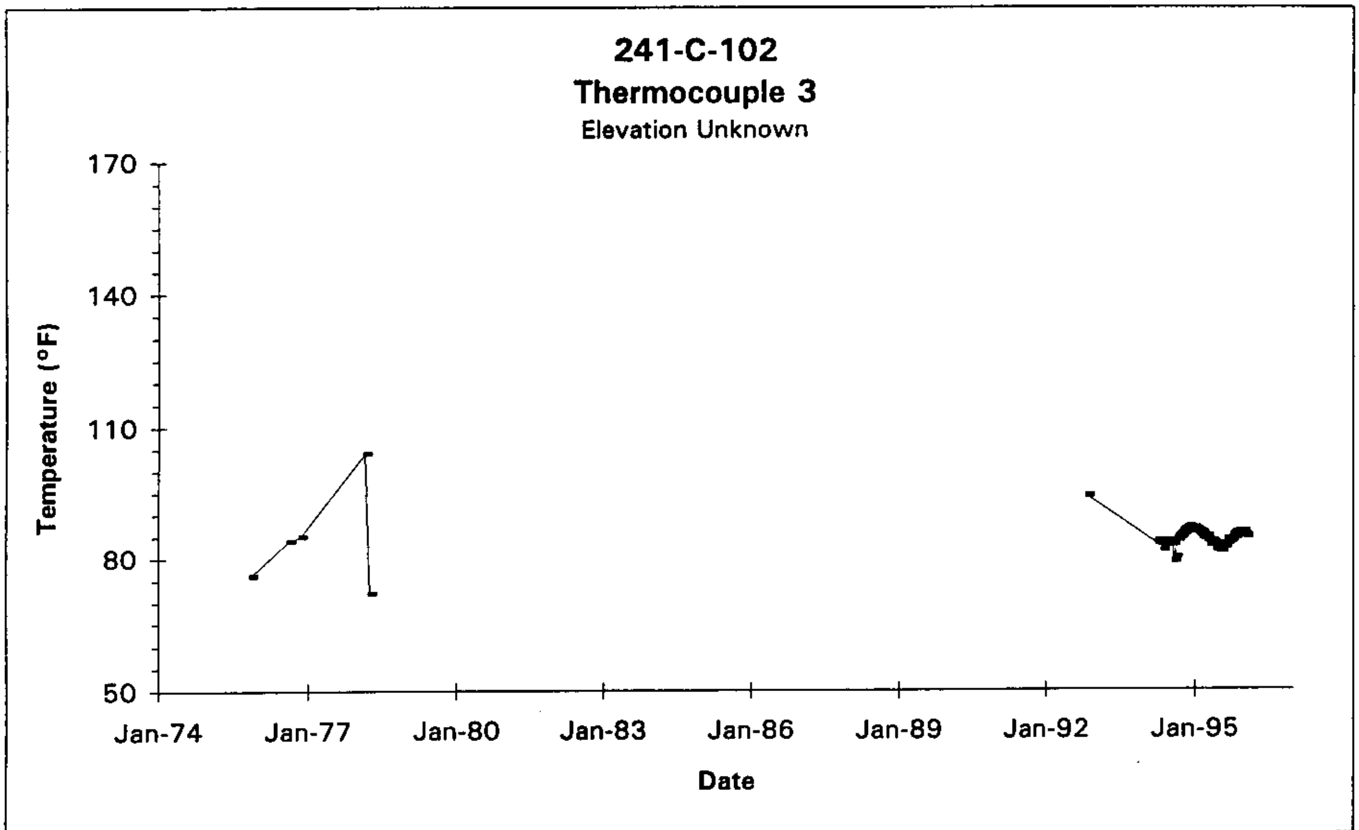


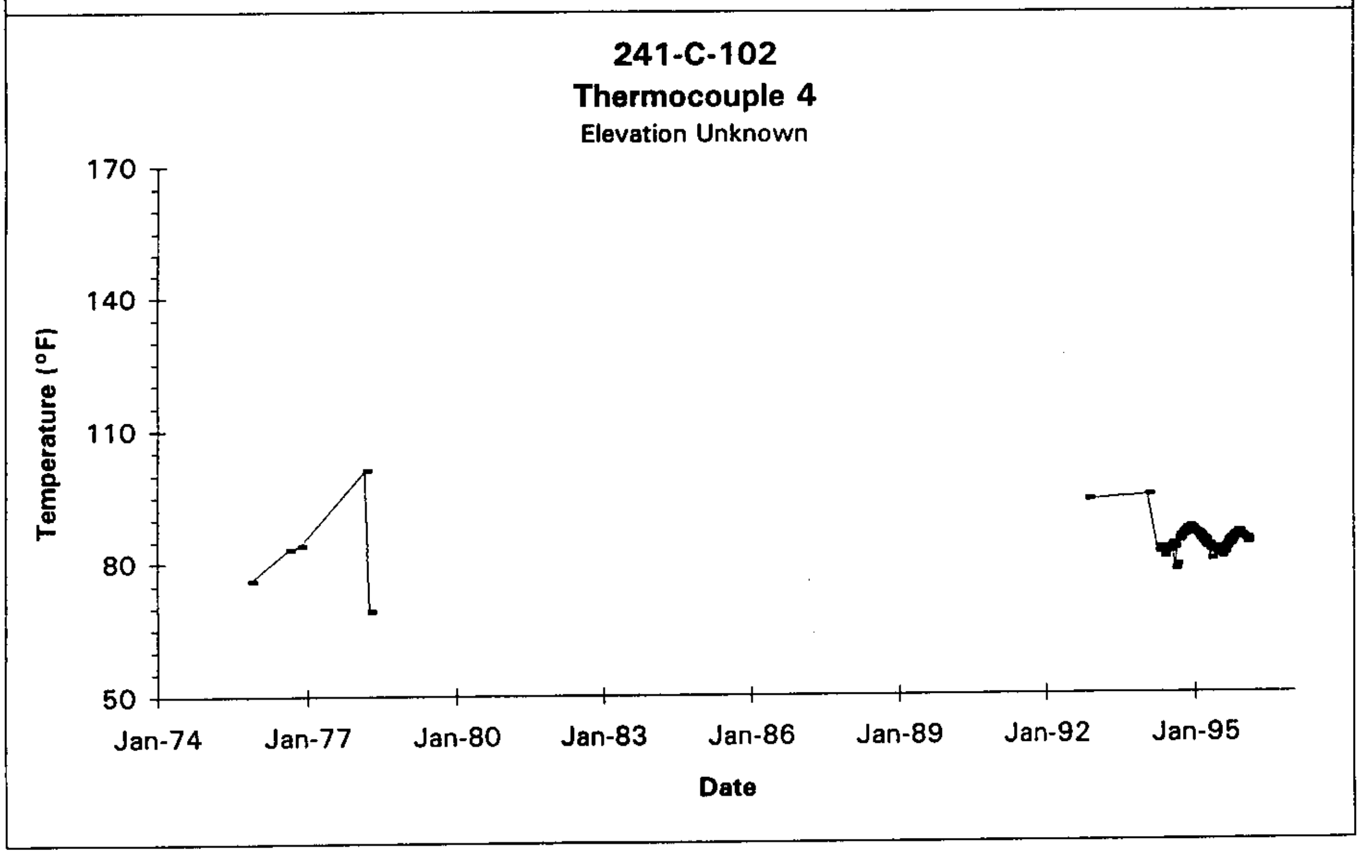
Figure B-4. Tank 241-C-102 Thermal History 1974-1995 Thermocouple 1 and 2.



Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

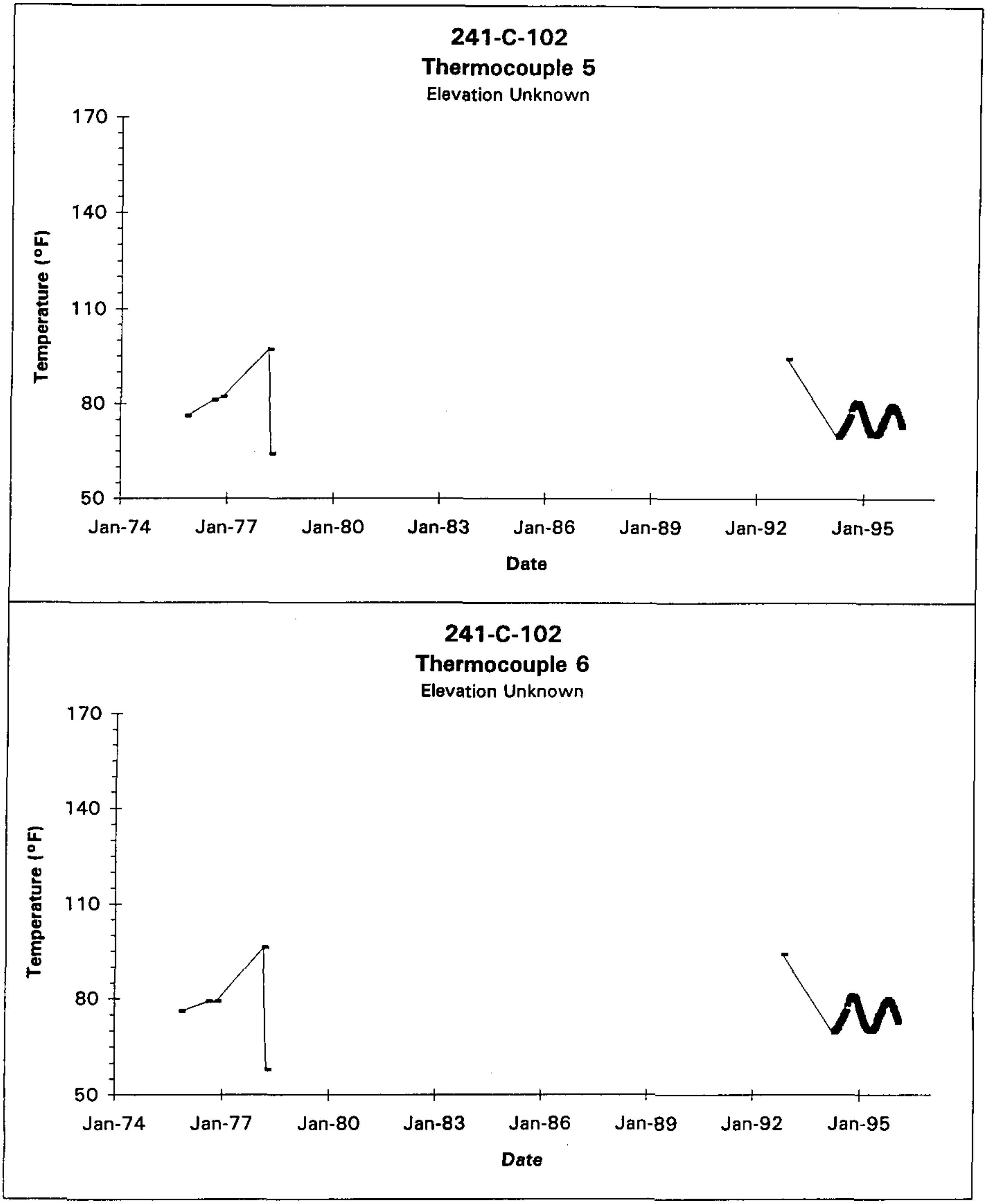
Figure B-5. Tank 241-C-102 Thermal History 1974-1995 Thermocouple 3 and 4.





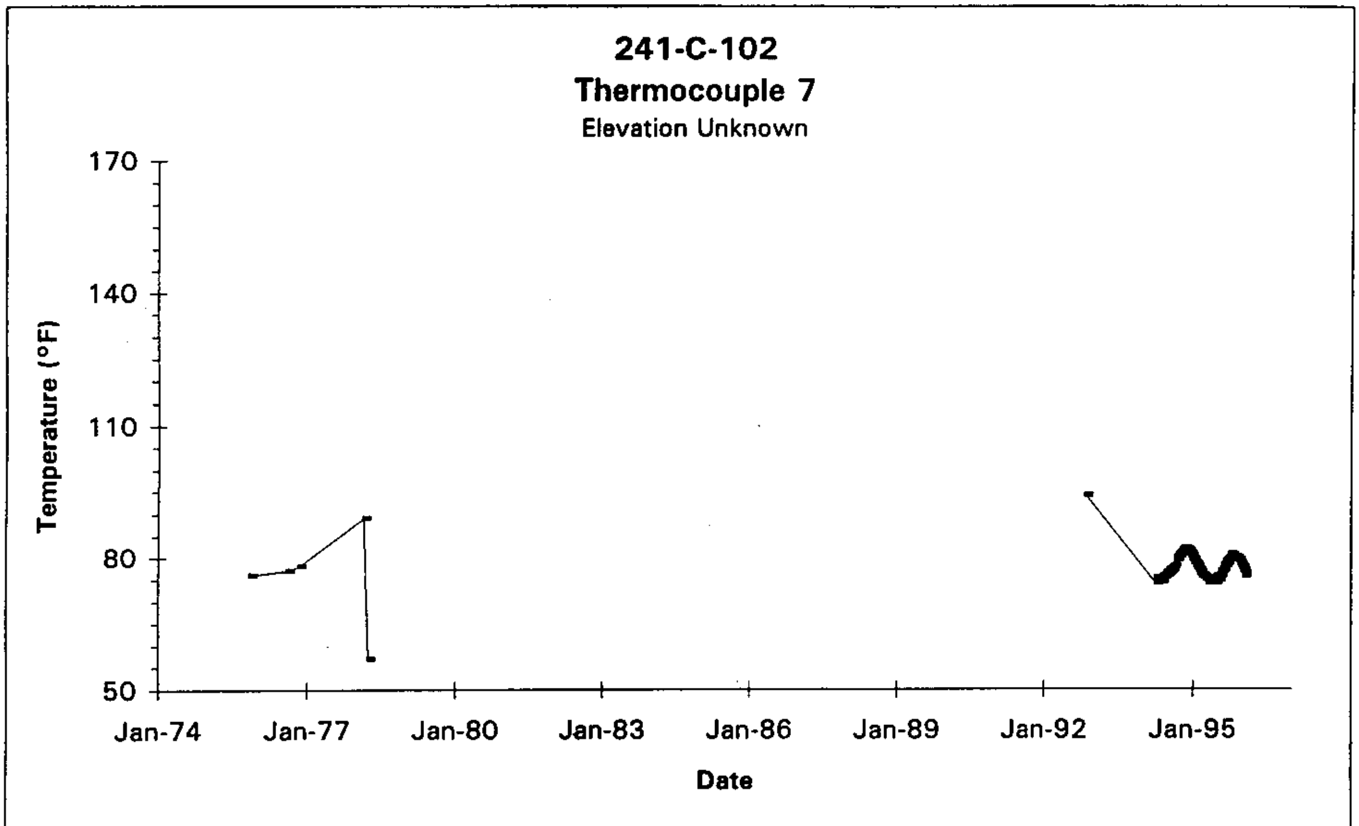
Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

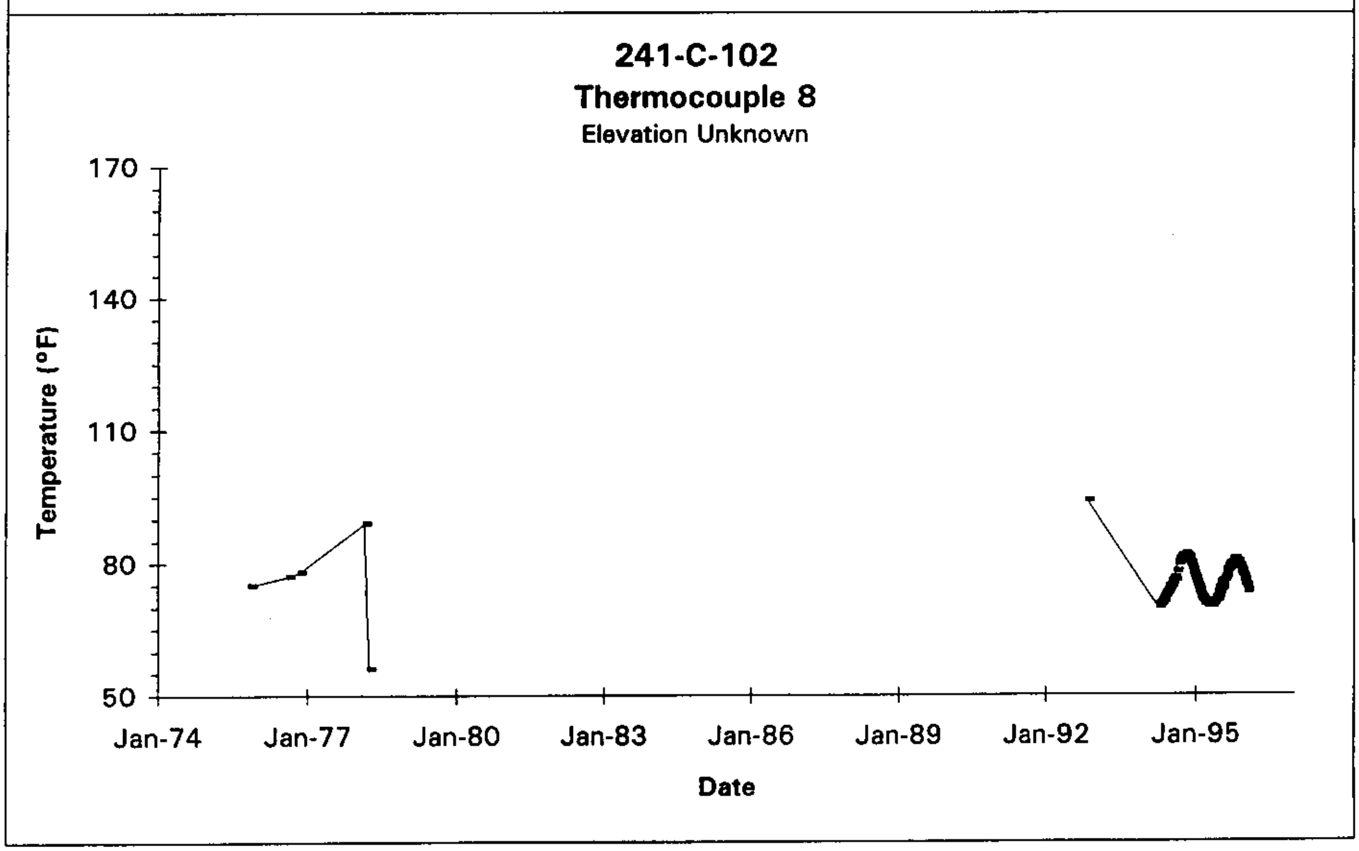
Figure B-6. Tank 241-C-102 Thermal History 1974-1995 Thermocouple 5 and 6.



Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

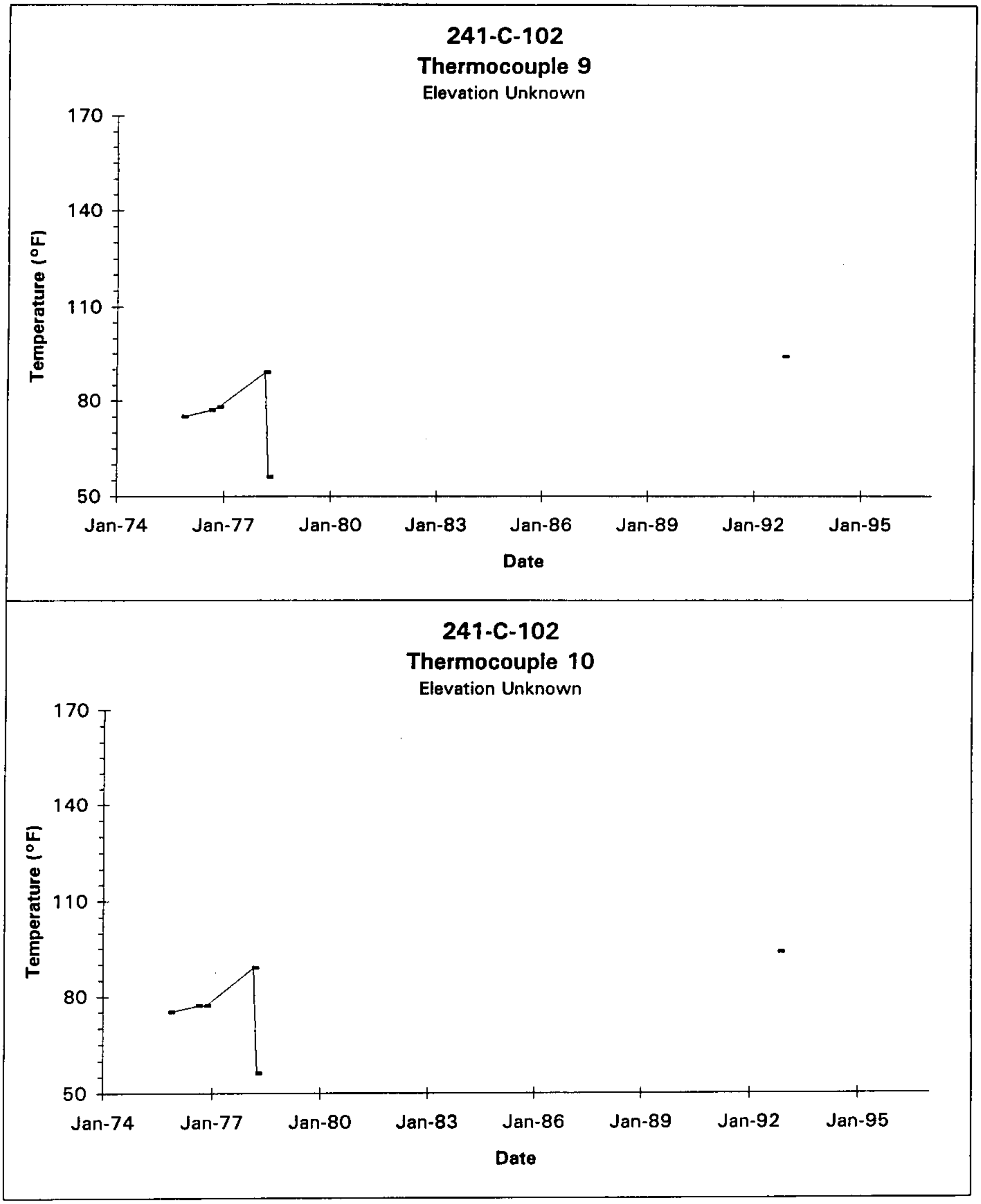
Figure B-7. Tank 241-C-102 Thermal History 1974-1995 Thermocouple 7 and 8.





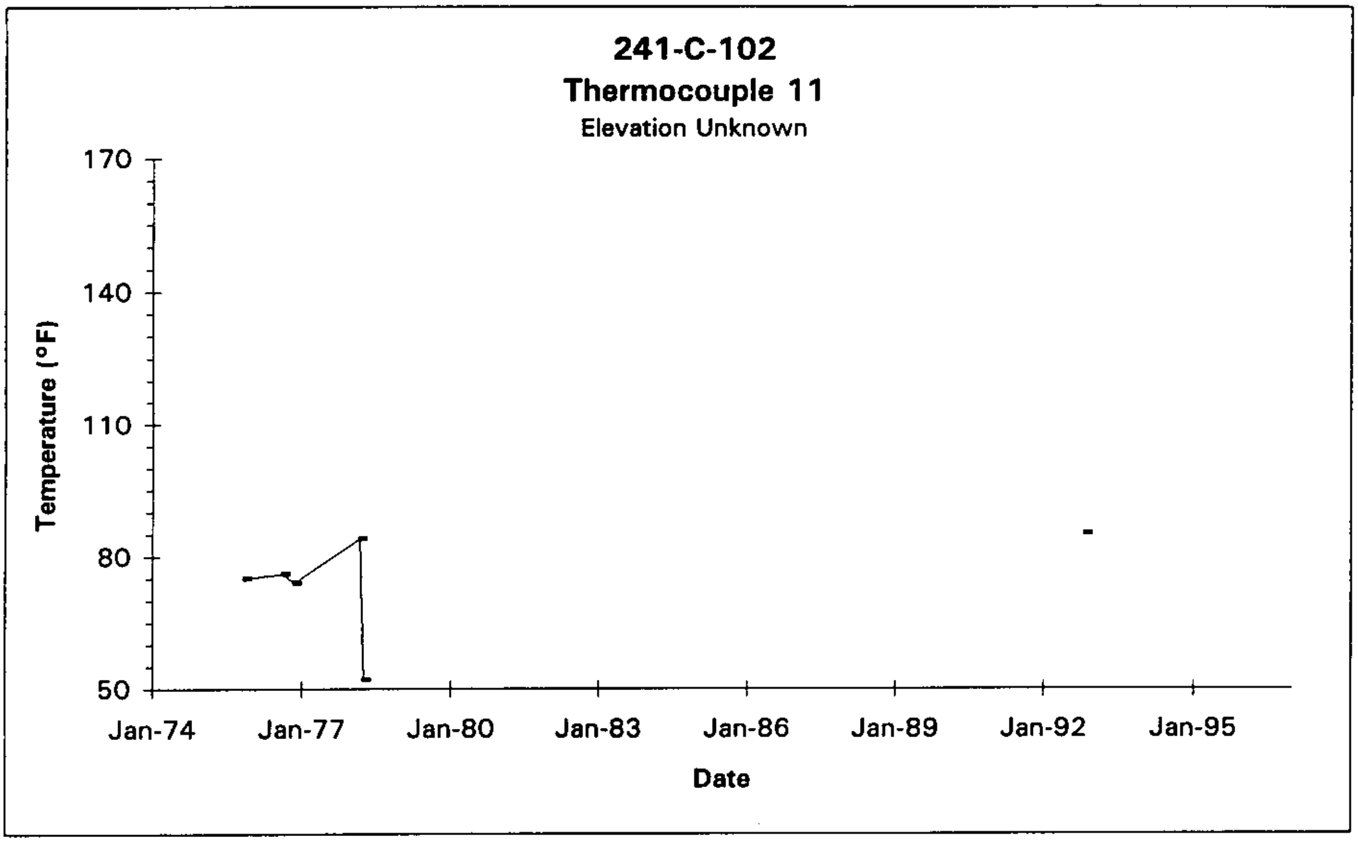
Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

Figure B-8. Tank 241-C-102 Thermal History 1974-1995 Thermocouple 9 and 10.



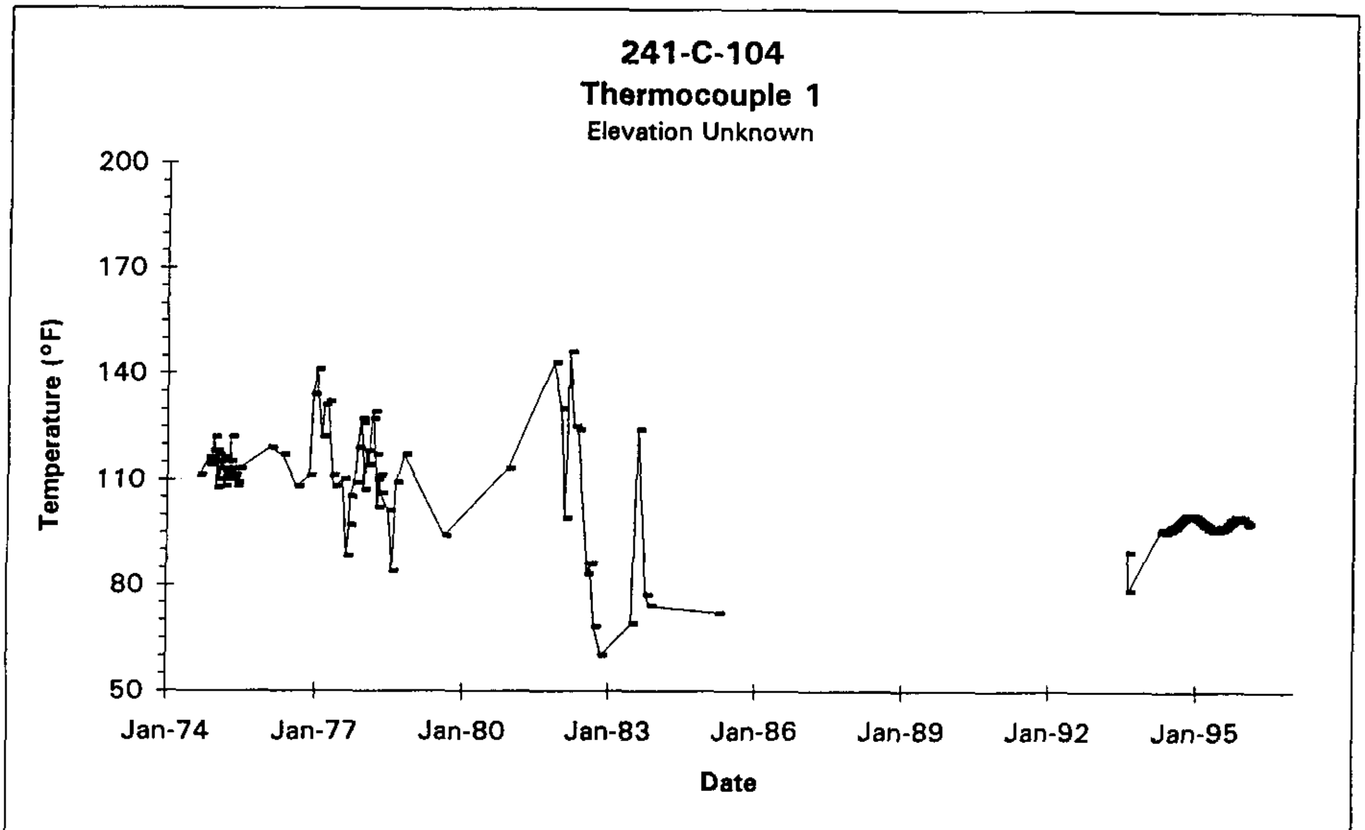
Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

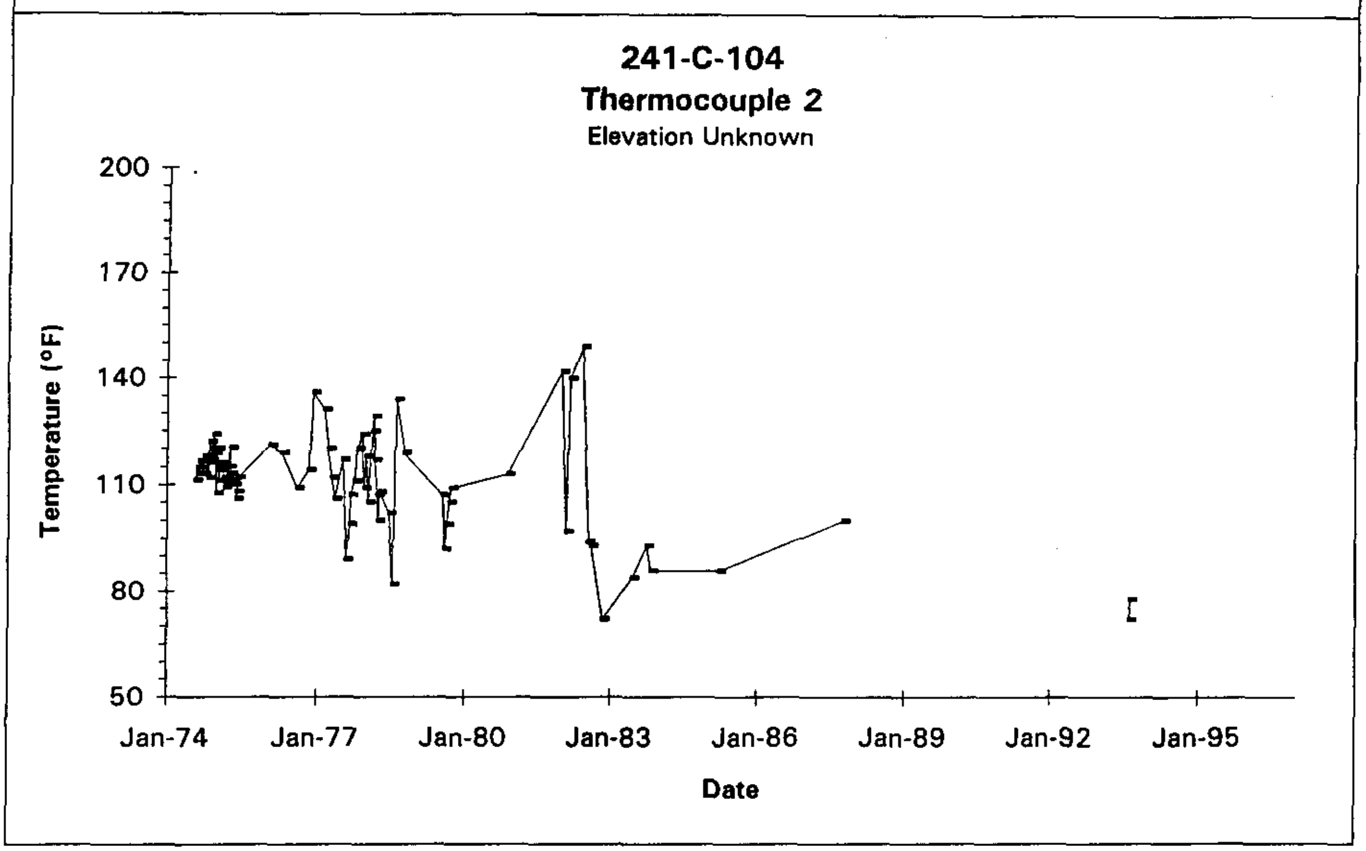
Figure B-9. Tank 241-C-102 Thermal History 1974-1995 Thermocouple 11.



Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

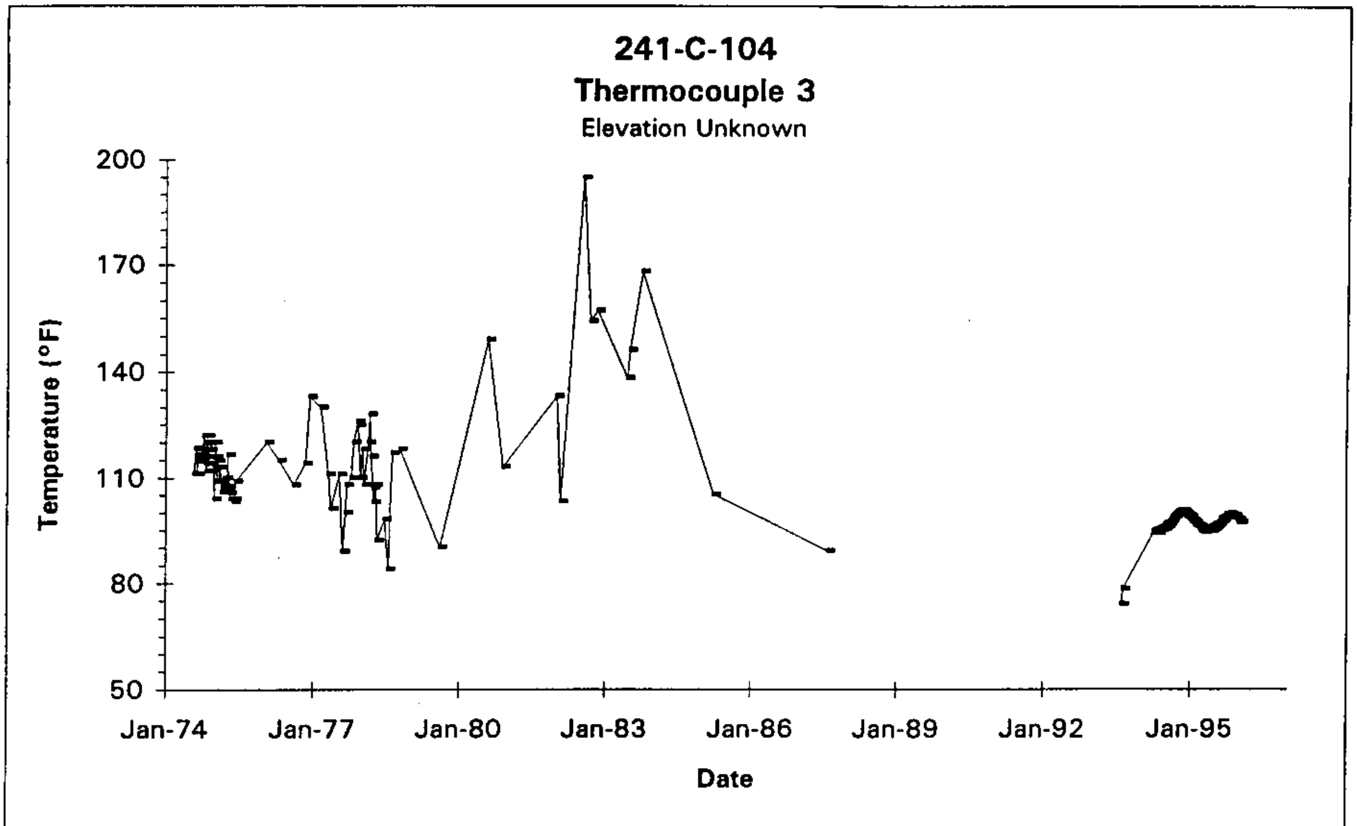
Figure B-10. Tank 241-C-104 Thermal History 1974-1995 Thermocouple 1 and 2.

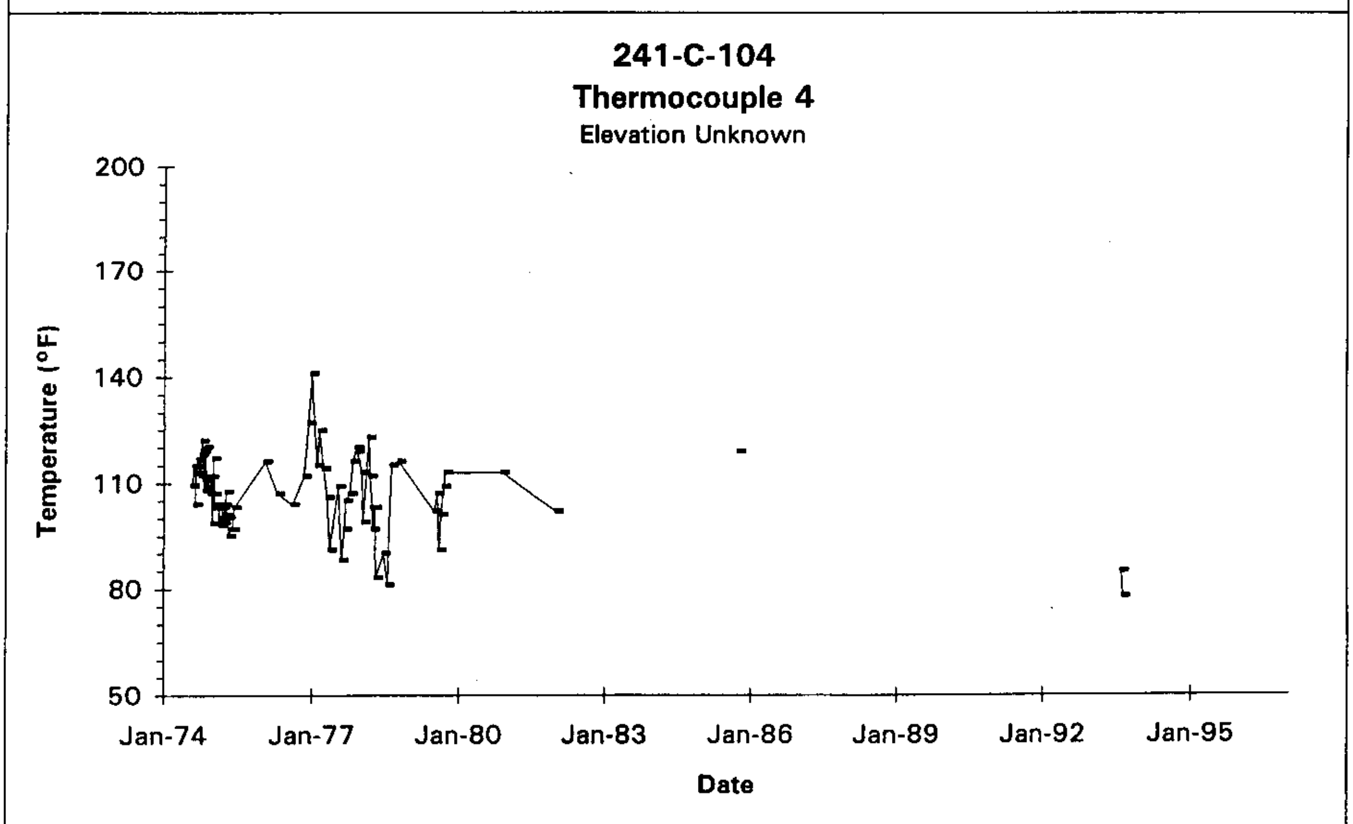




Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

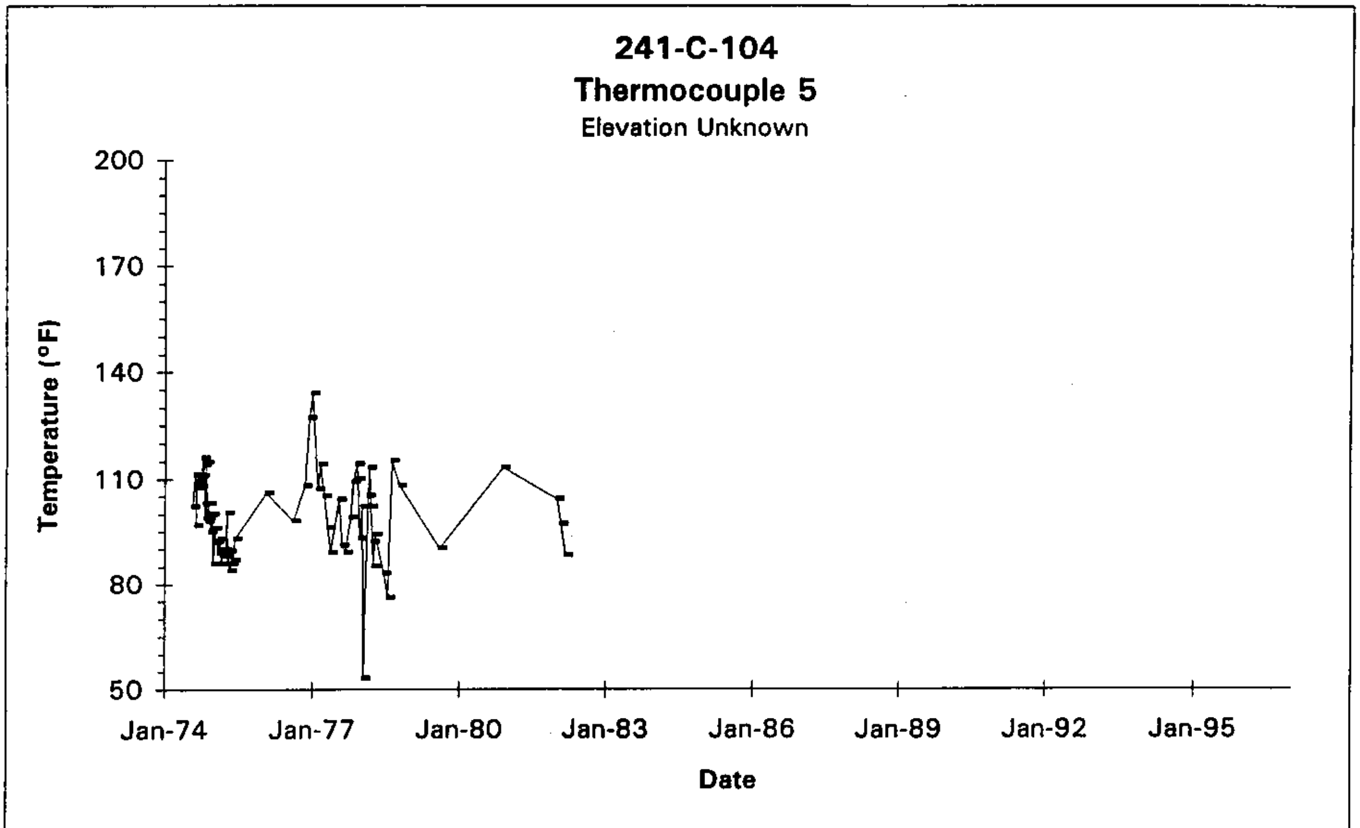
Figure B-11. Tank 241-C-104 Thermal History 1974-1995 Thermocouple 3 and 4.

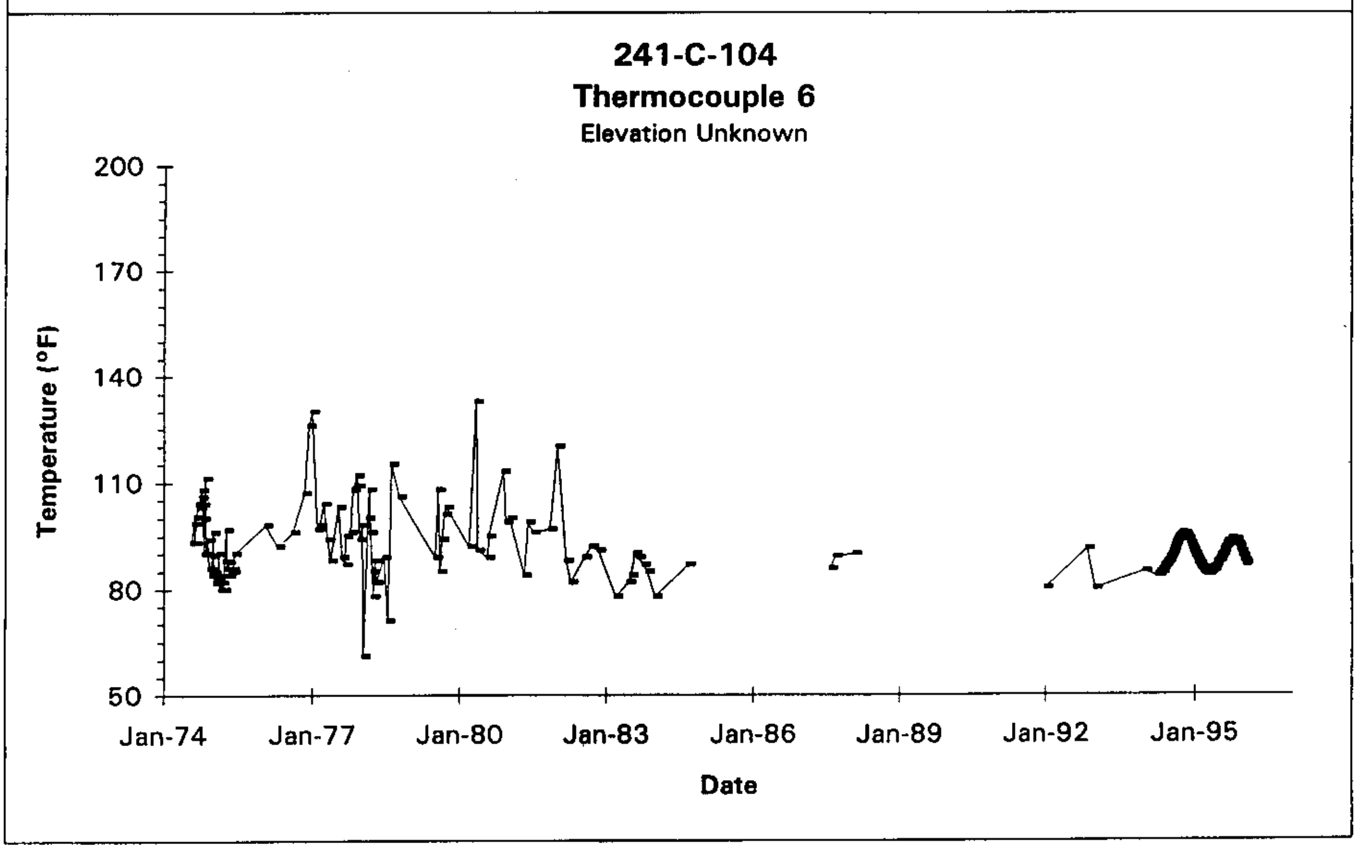




Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

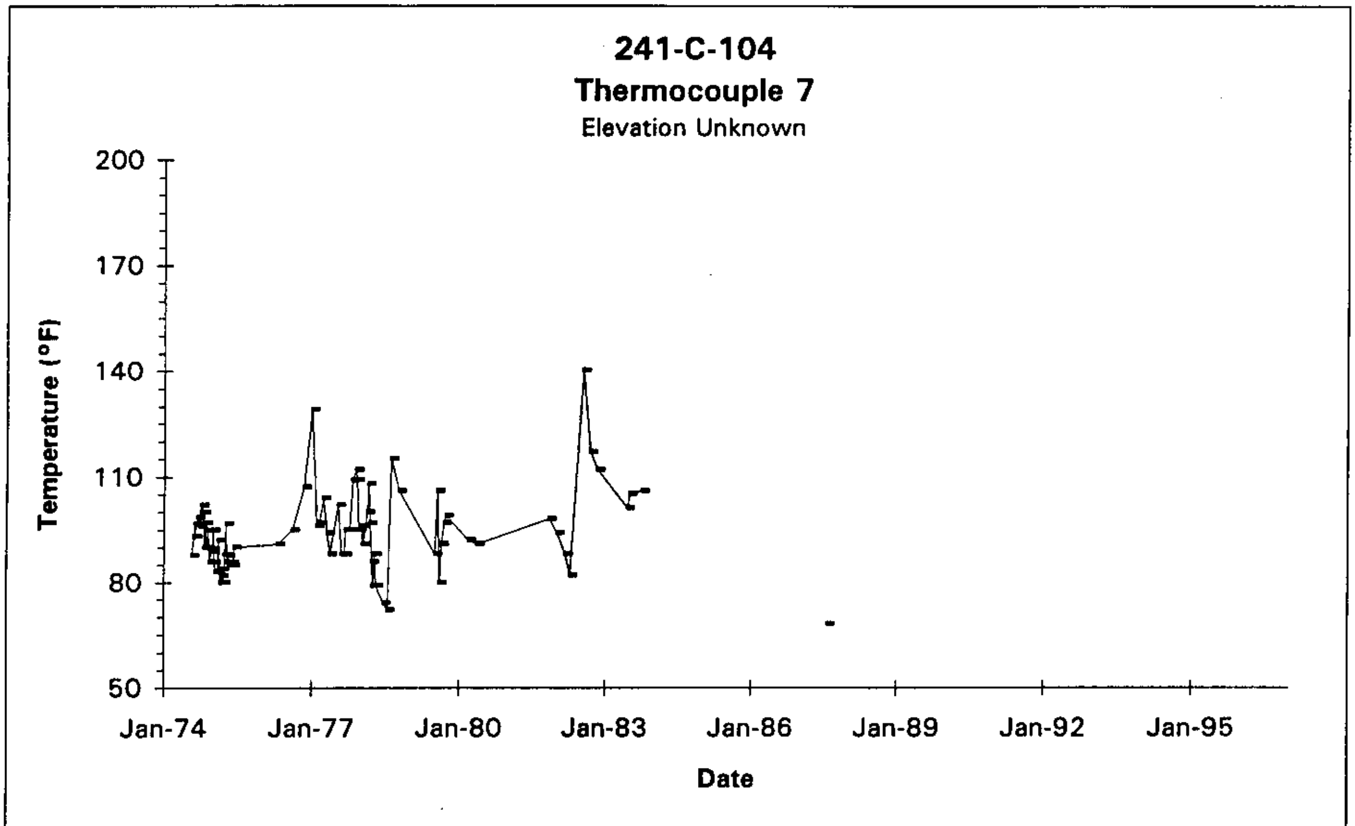
Figure B-12. Tank 241-C-104 Thermal History 1974-1995 Thermocouple 5 and 6.

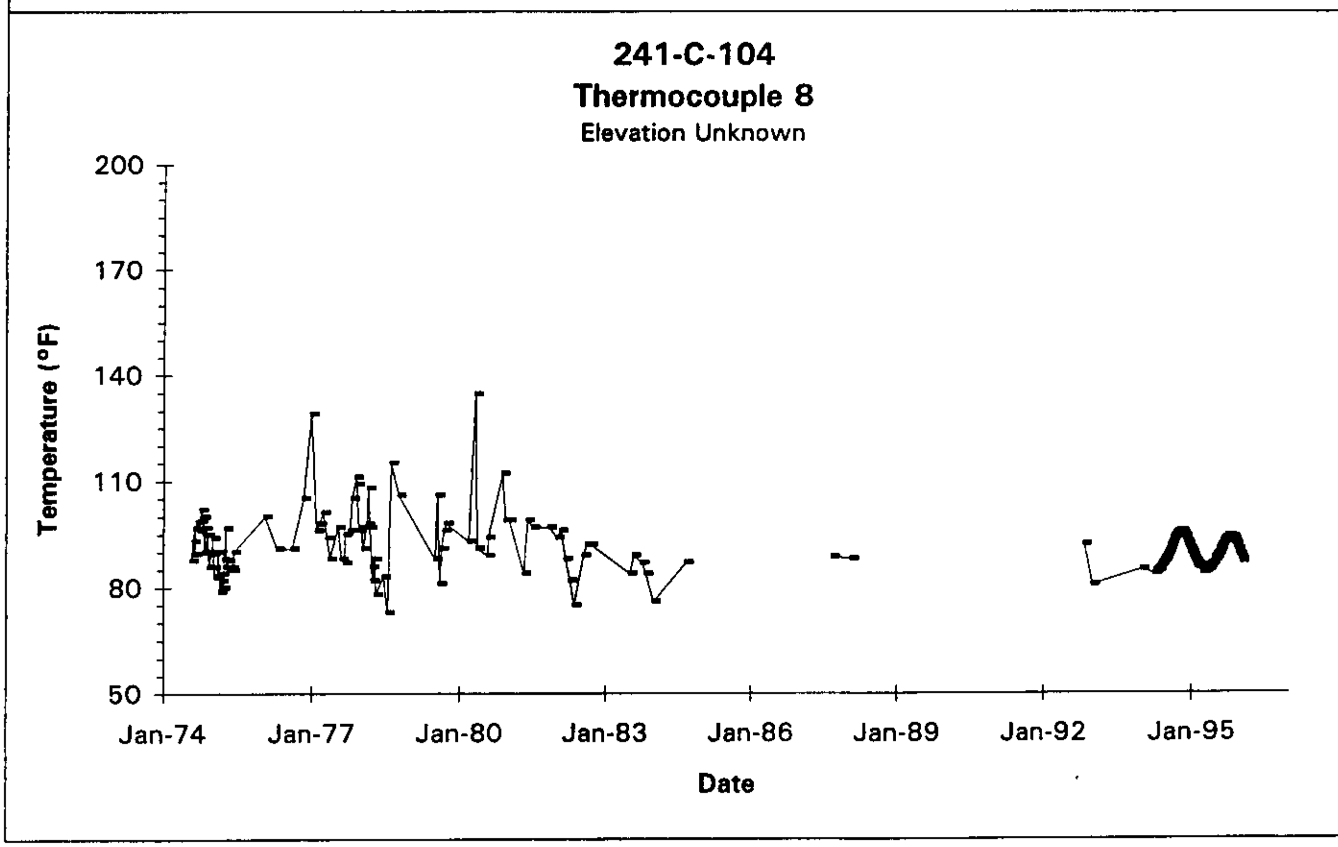




Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

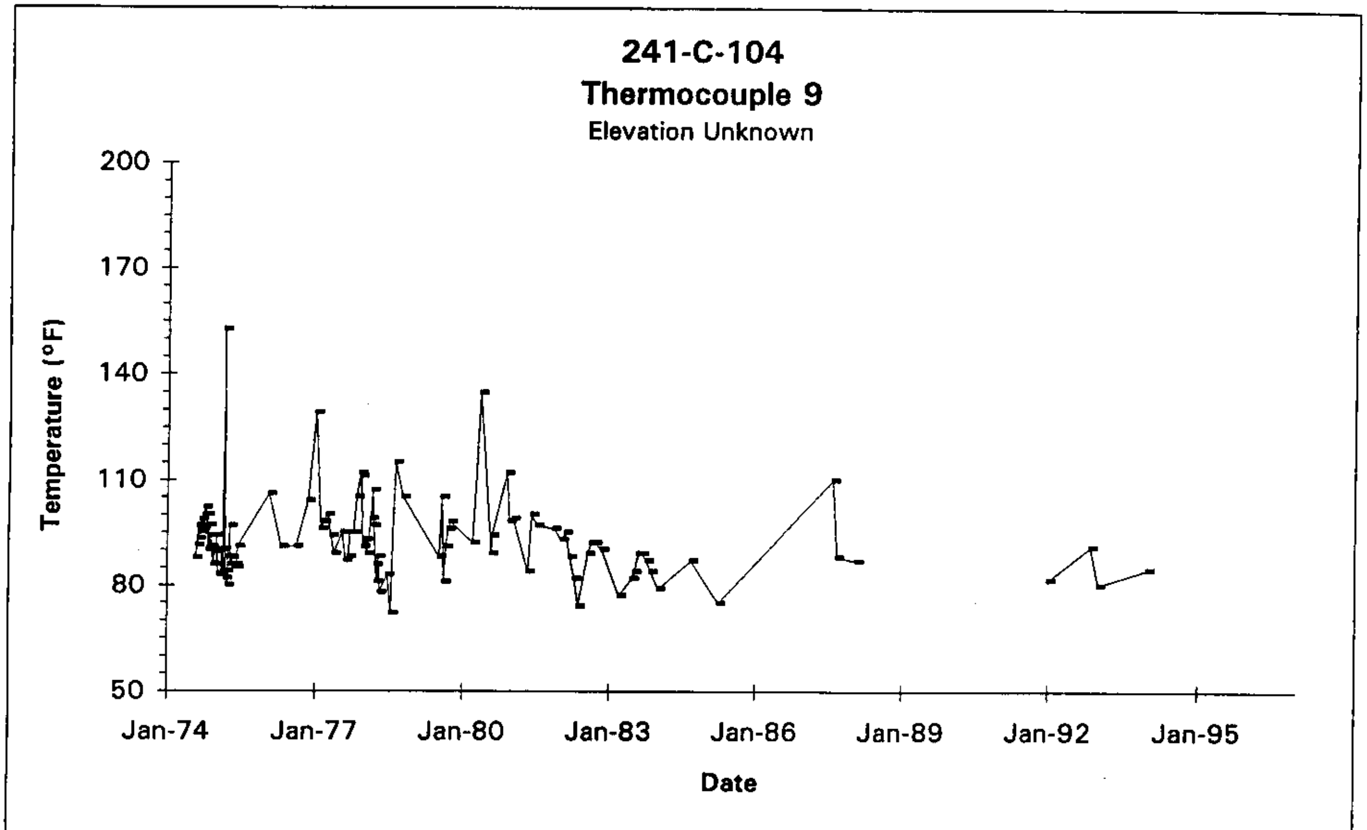
Figure B-13. Tank 241-C-104 Thermal History 1974-1995 Thermocouple 7 and 8.

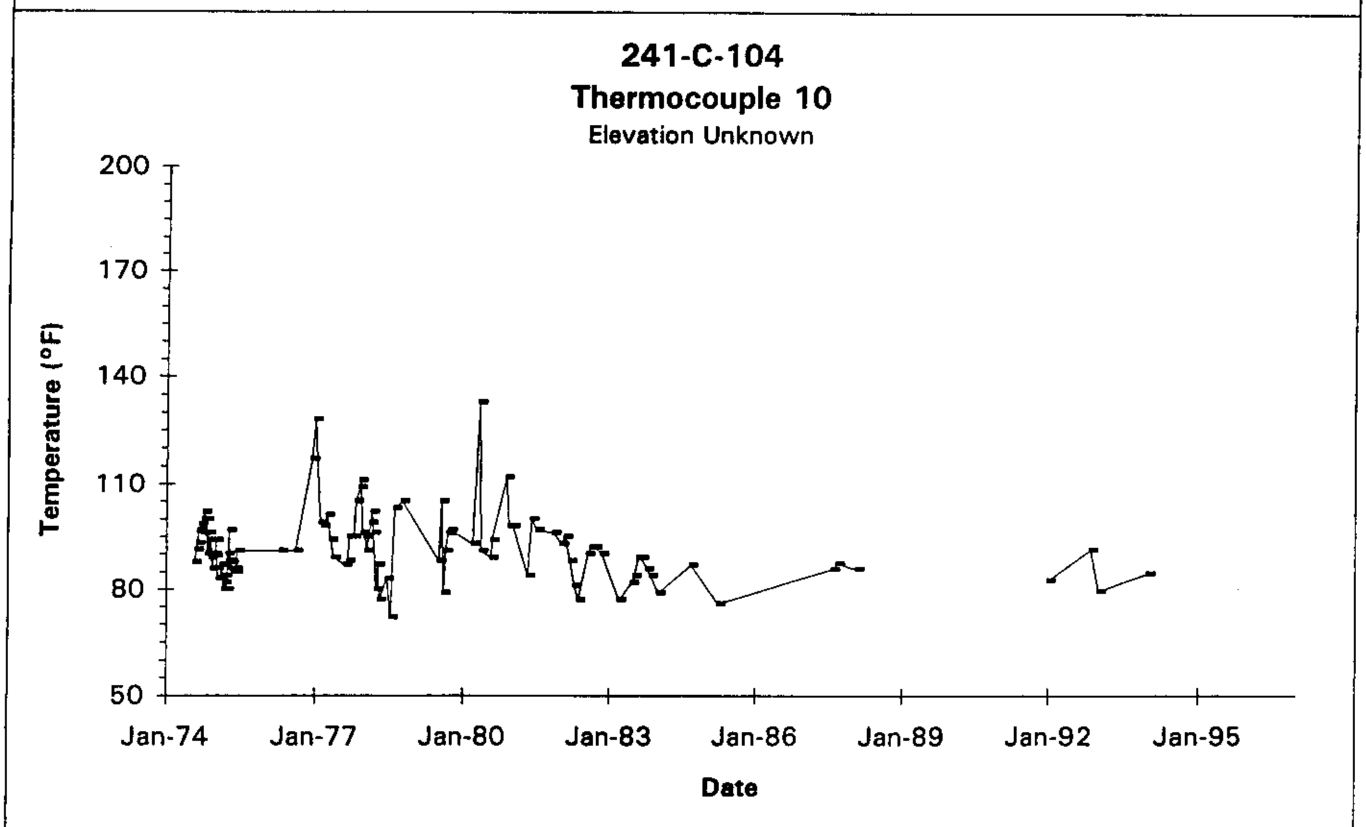




Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

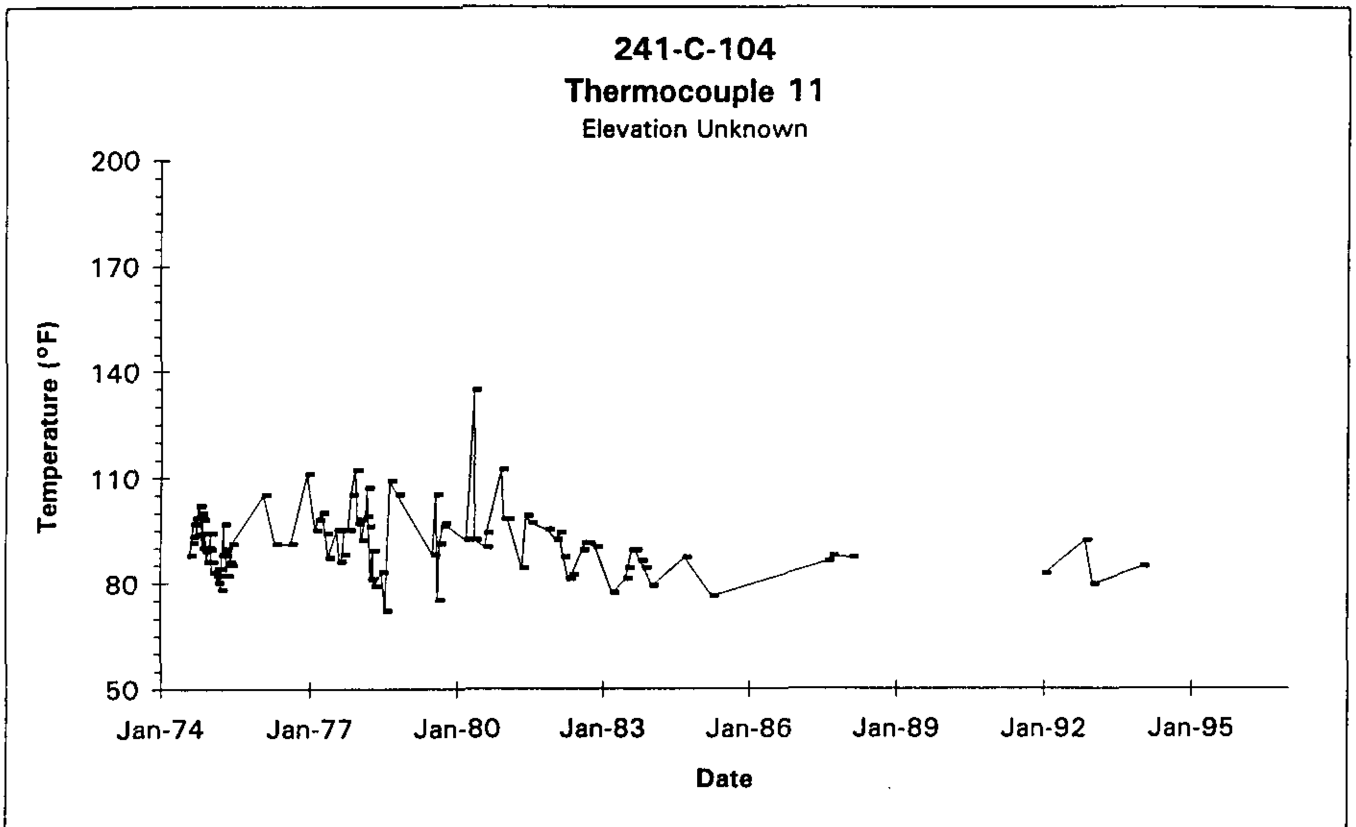
Figure B-14. Tank 241-C-104 Thermal History 1974-1995 Thermocouple 9 and 10.

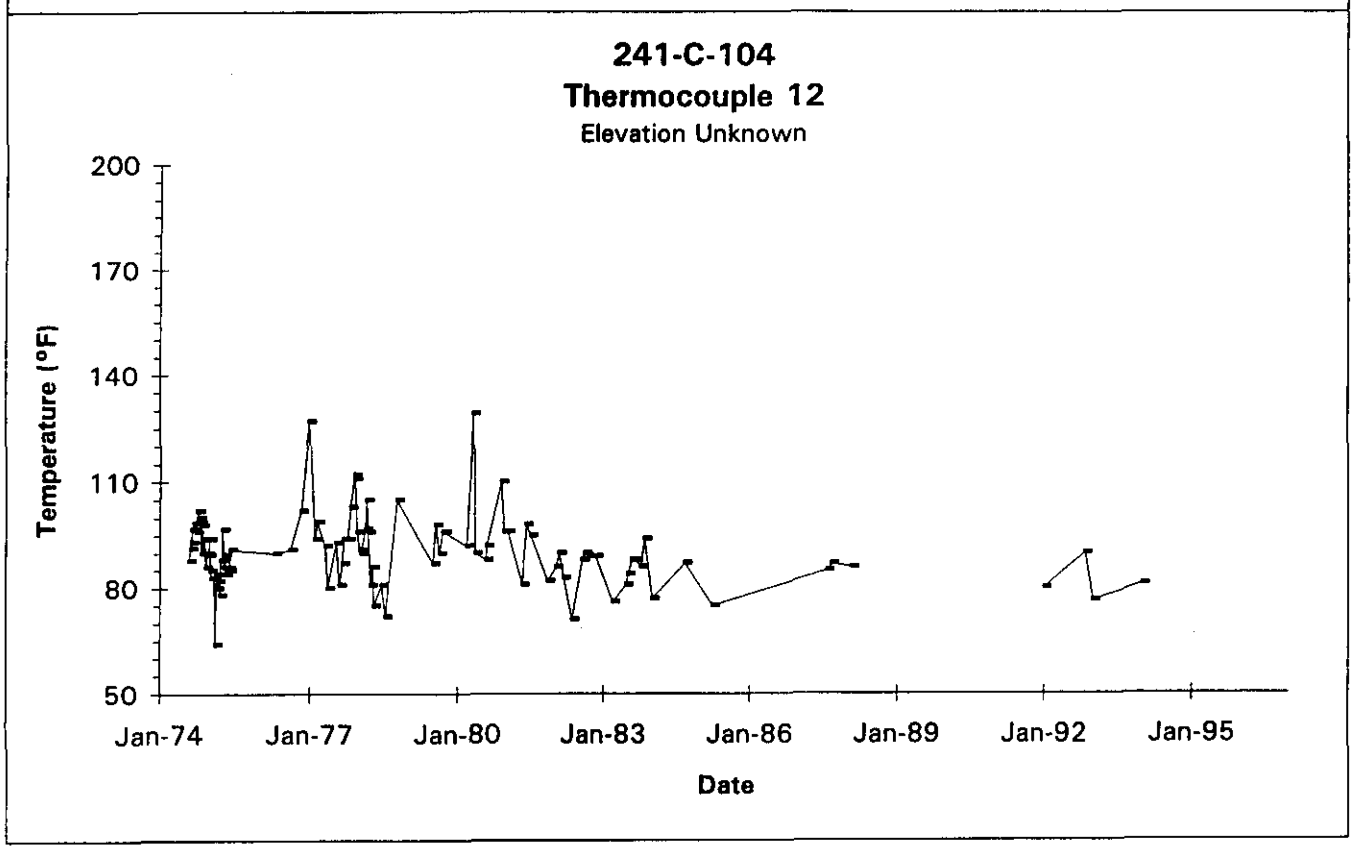




Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

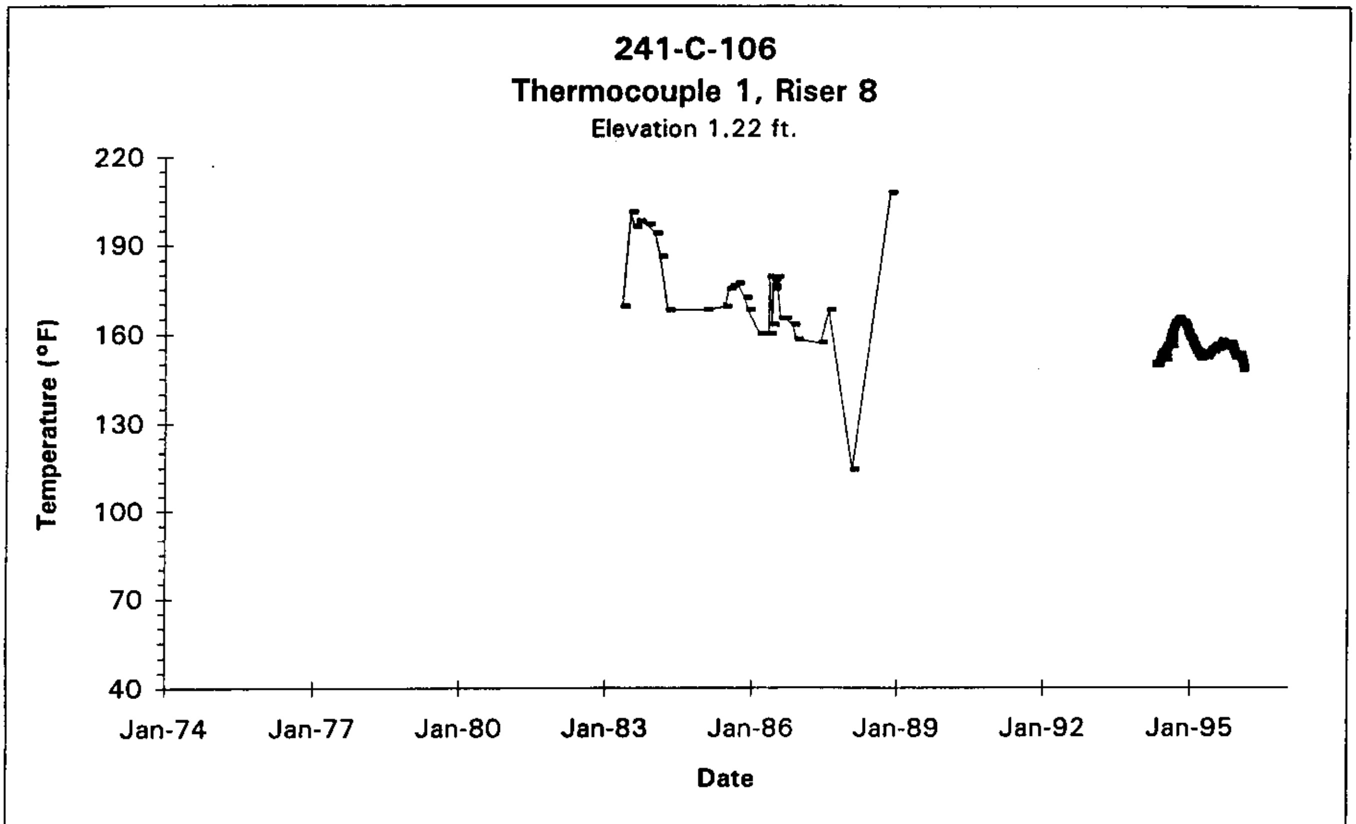
Figure B-15. Tank 241-C-104 Thermal History 1974-1995 Thermocouple 11 and 12.

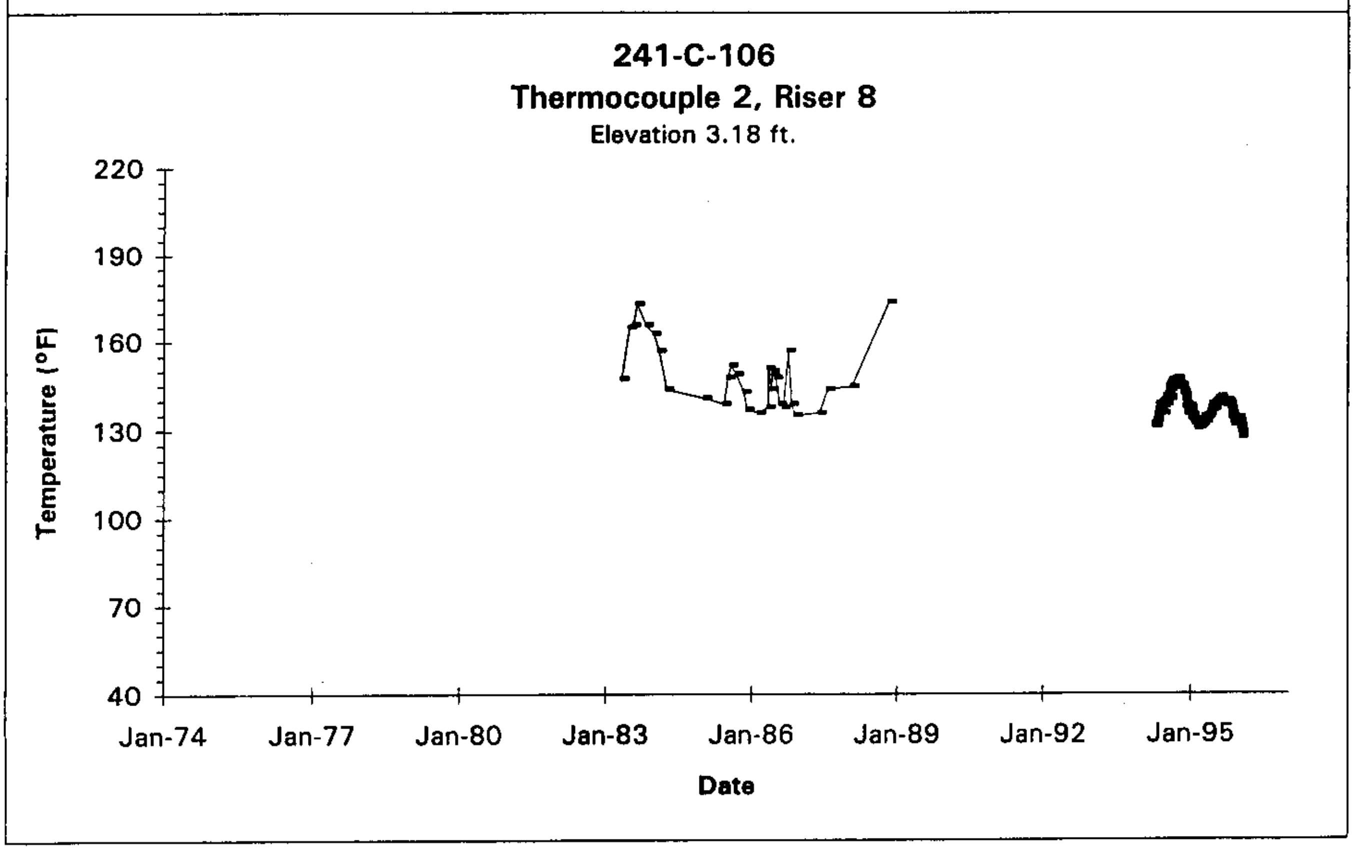




Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

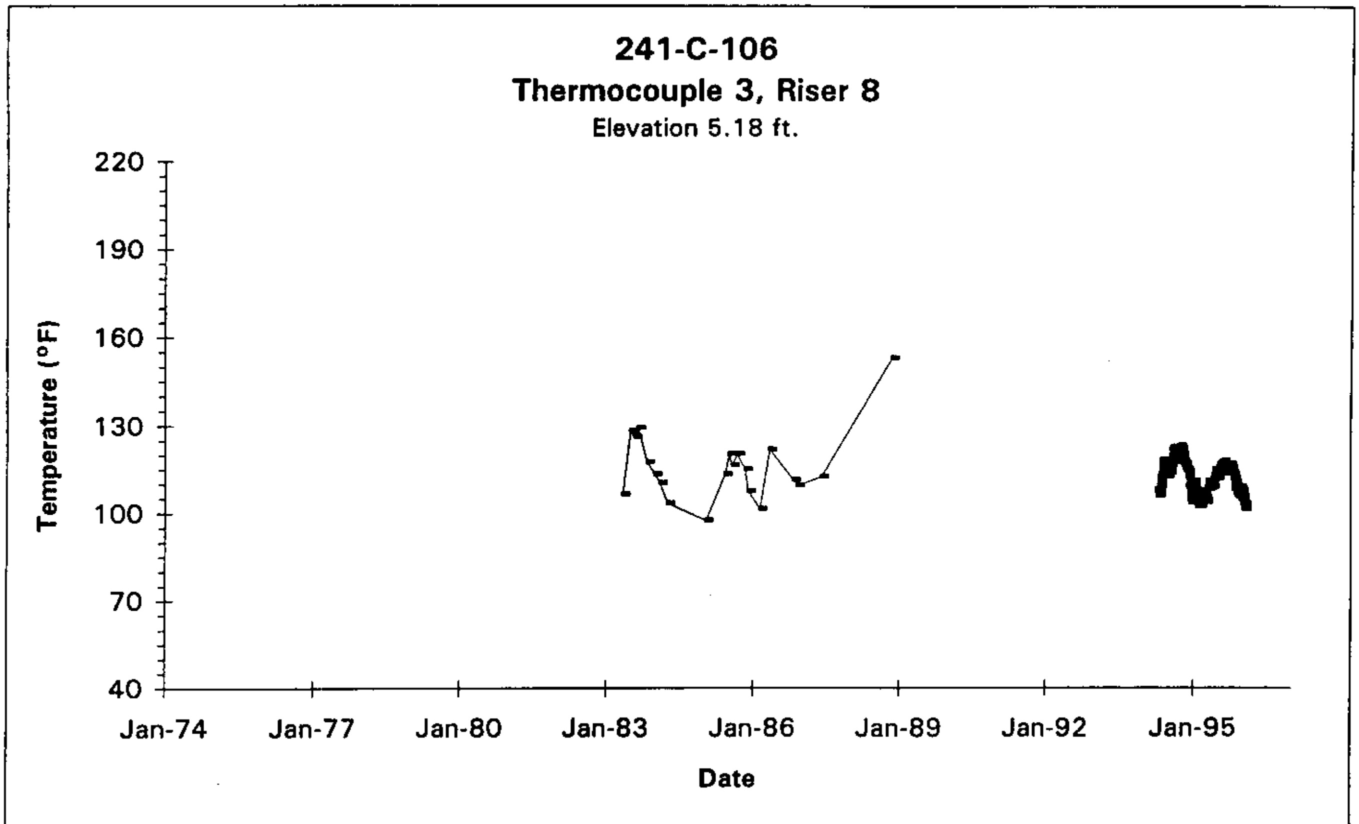
Figure B-16. Tank 241-C-106 Thermal History 1974-1995 Thermocouple 1 and 2, Riser 8.

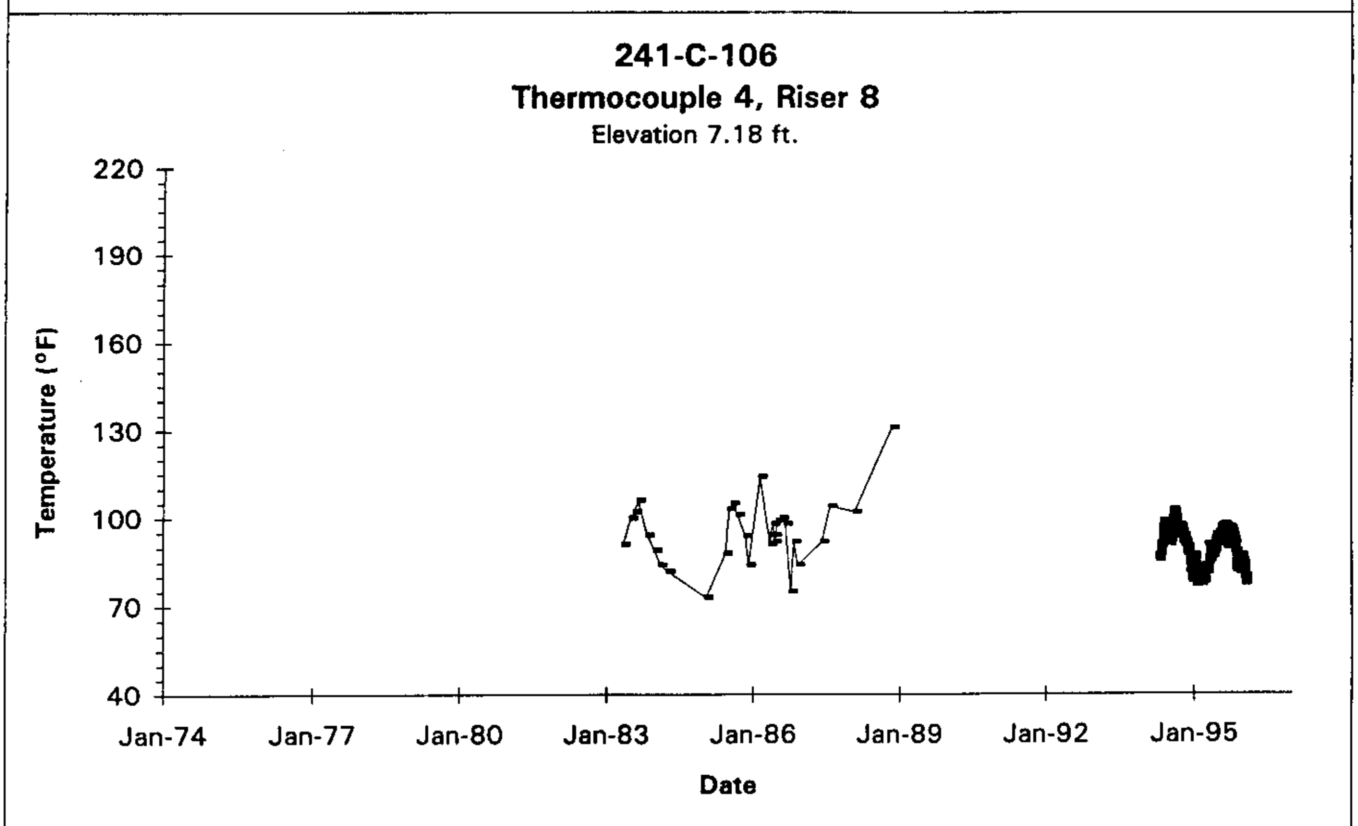




Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

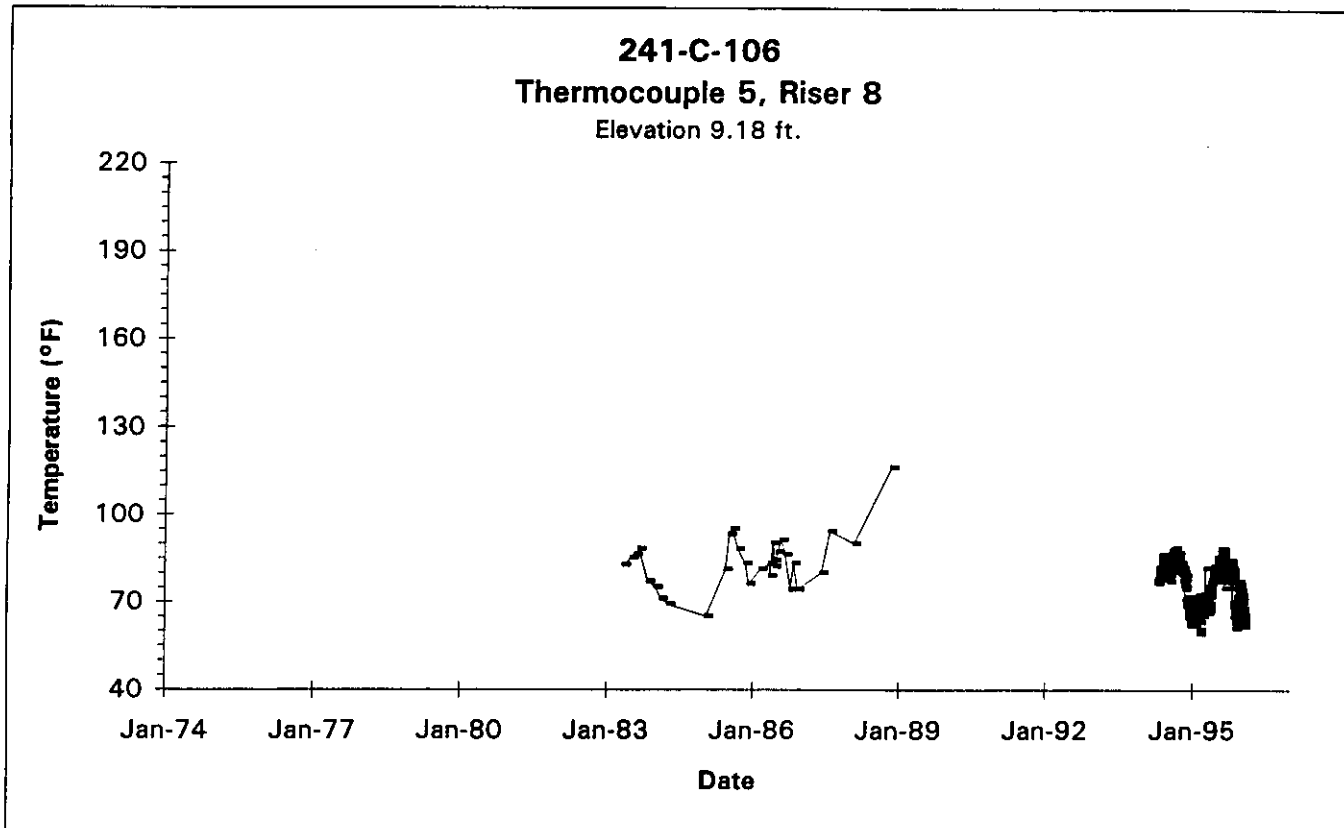
Figure B-17. Tank 241-C-106 Thermal History 1974-1995 Thermocouple 3 and 4, Riser 8.

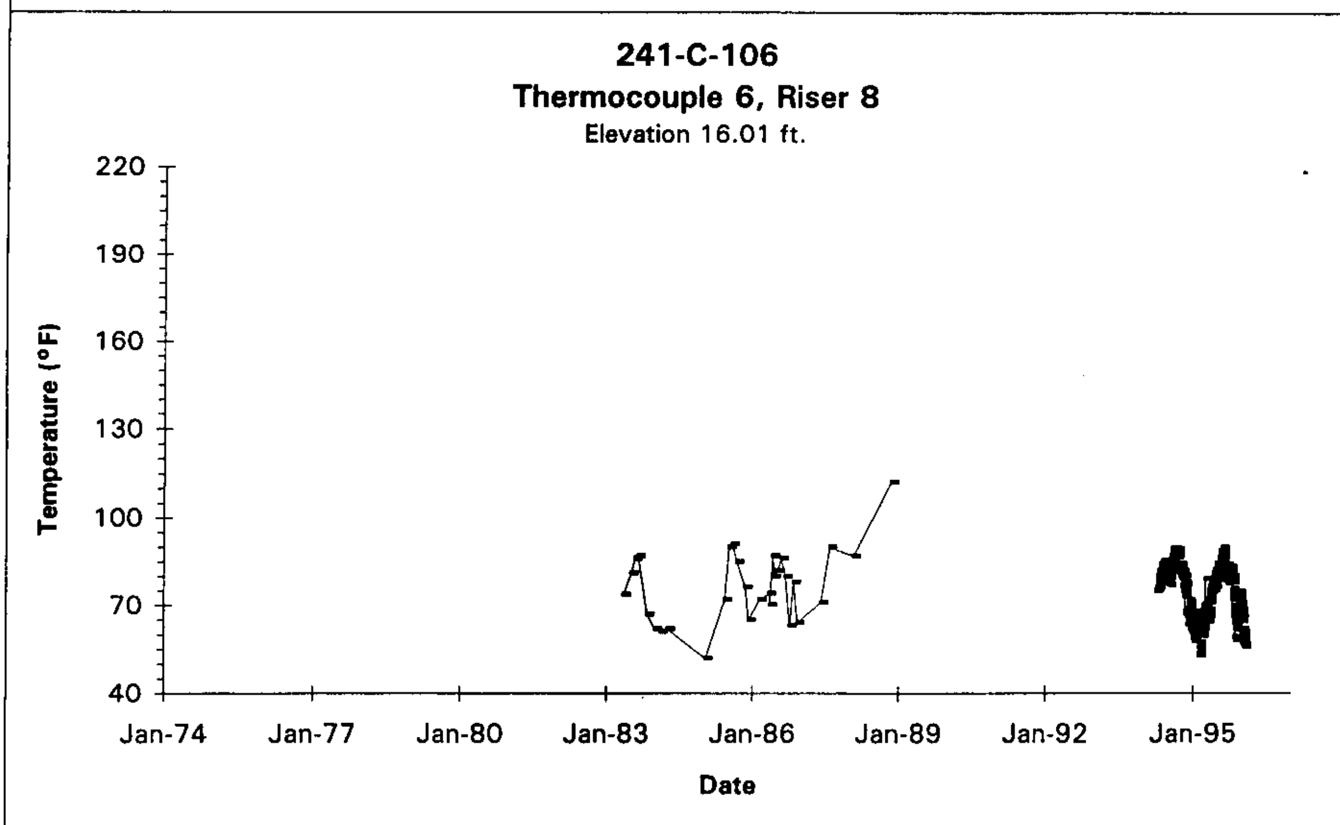




Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

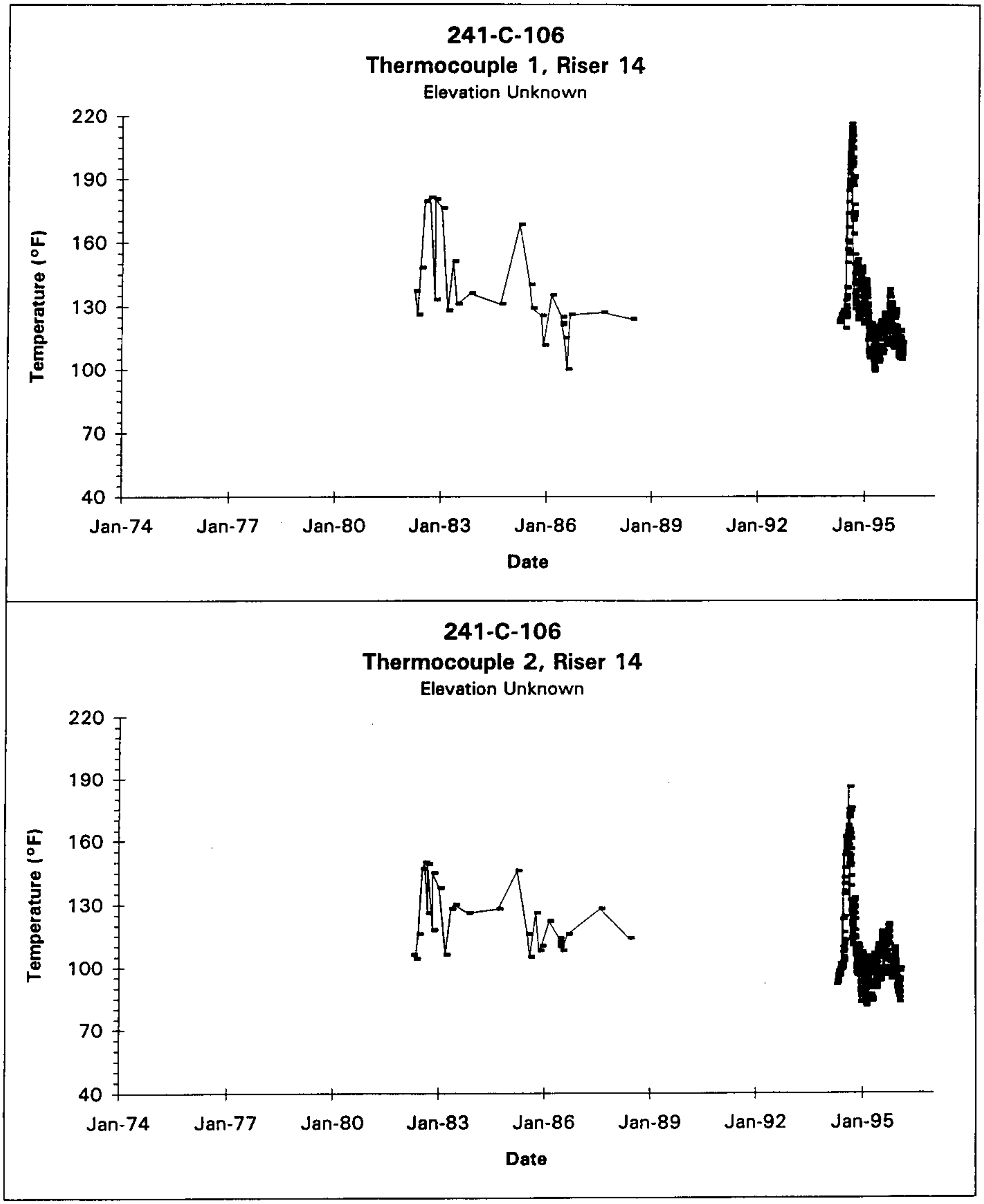
Figure B-18. Tank 241-C-106 Thermal History 1974-1995 Thermocouple 5 and 6, Riser 8.





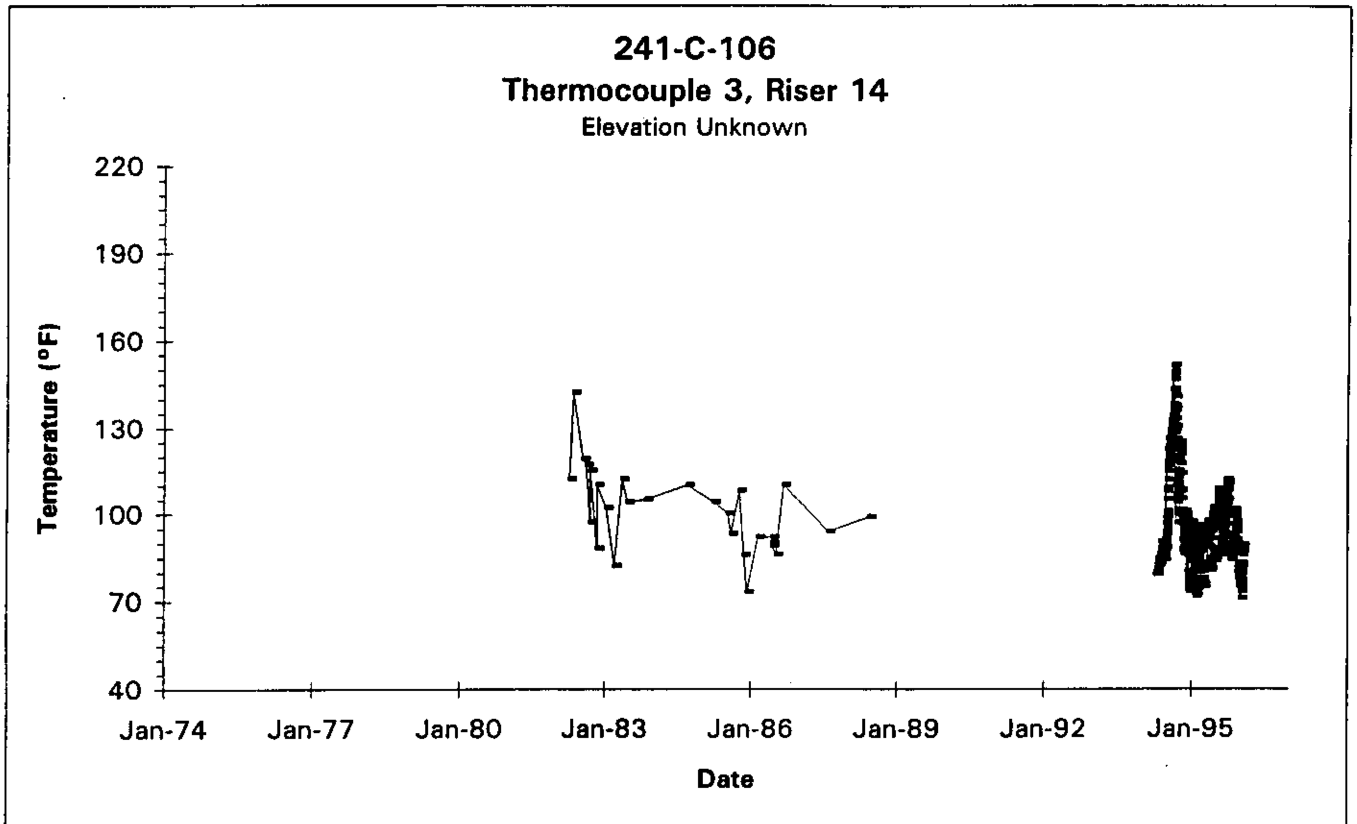
Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

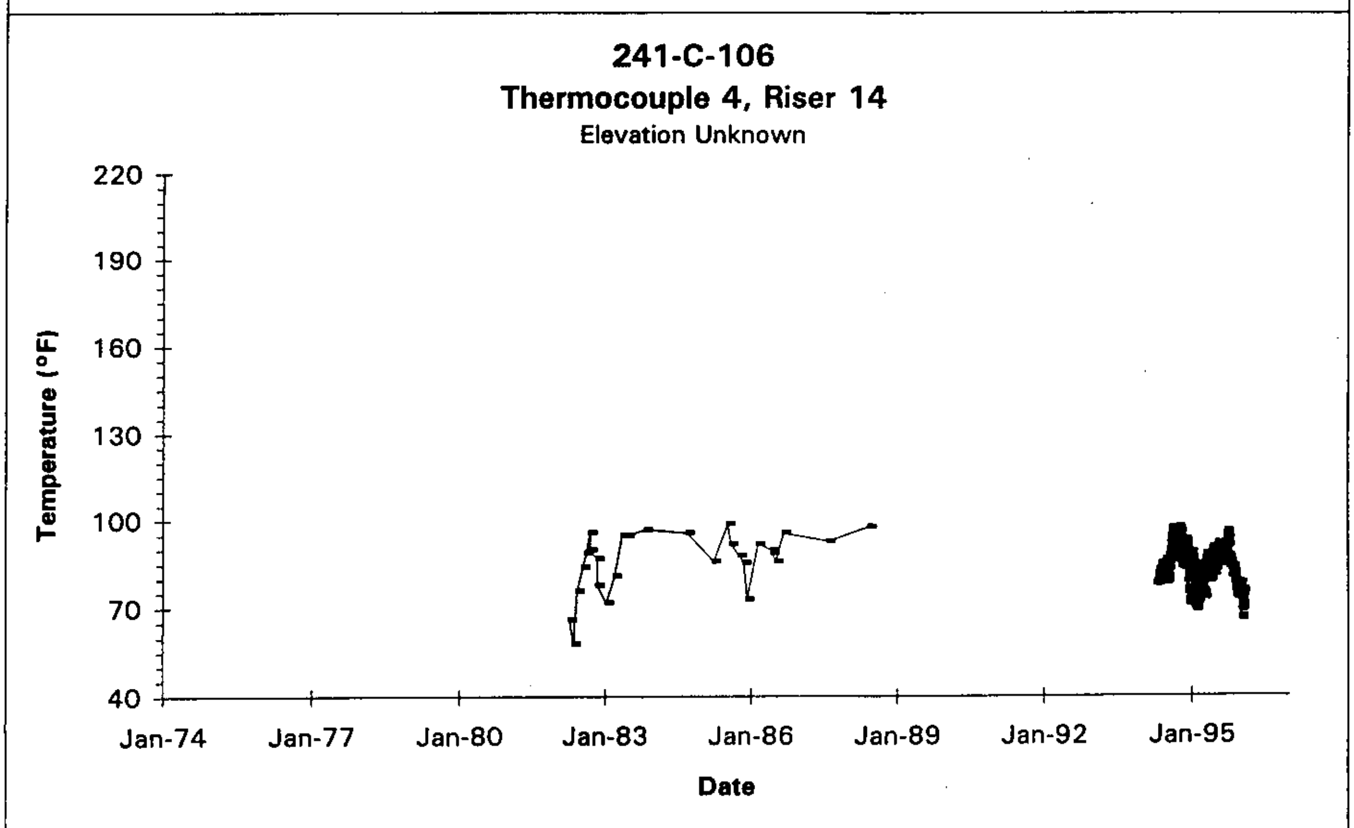
Figure B-19. Tank 241-C-106 Thermal History 1974-1995 Thermocouple 1 and 2, Riser 14.



Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

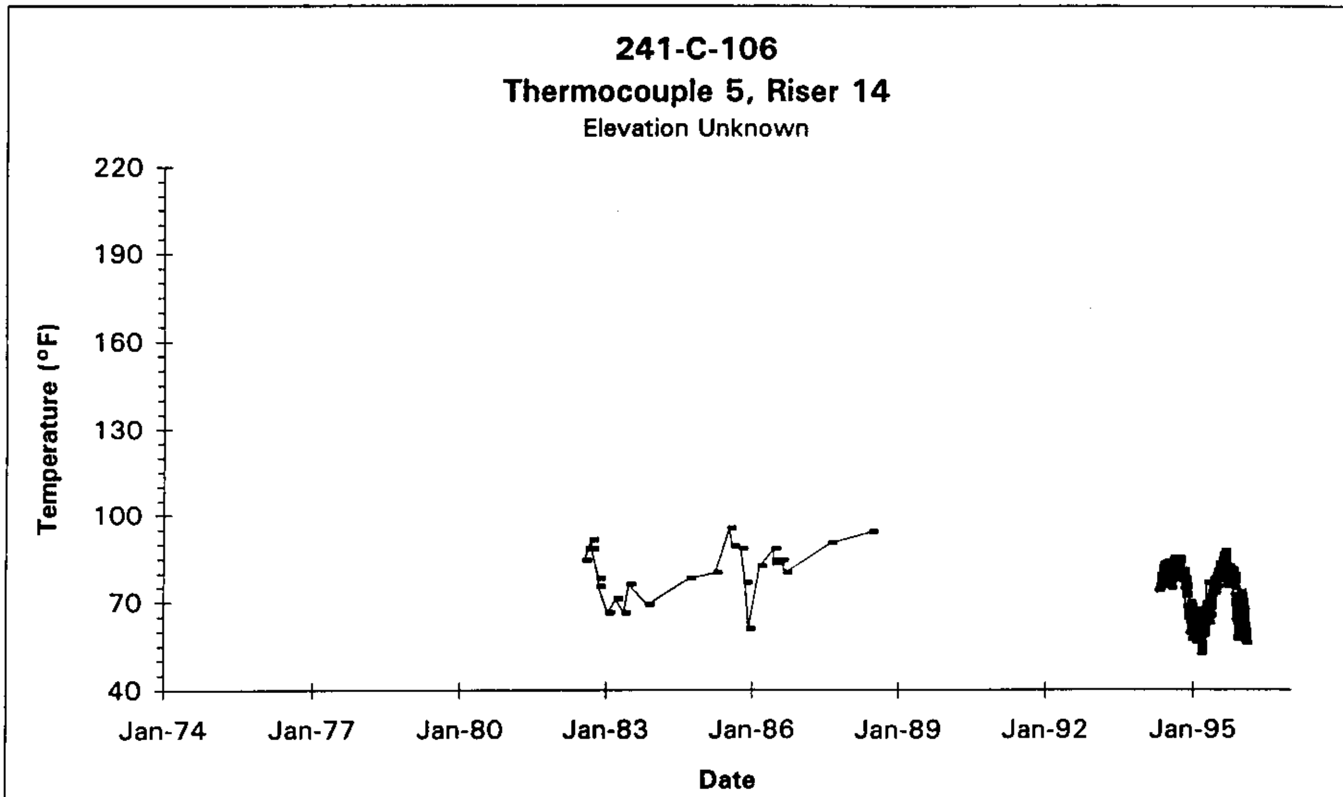
Figure B-20. Tank 241-C-106 Thermal History 1974-1995 Thermocouple 3 and 4, Riser 14.

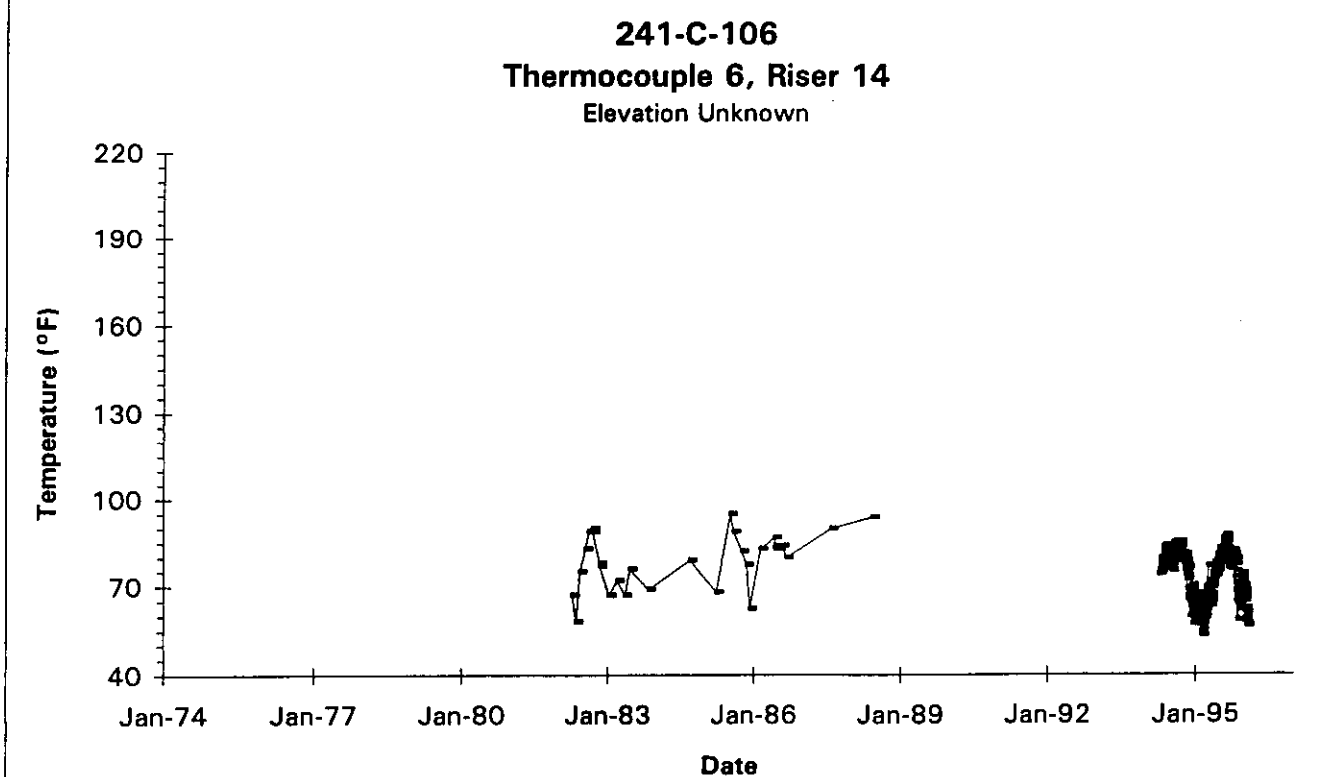




Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

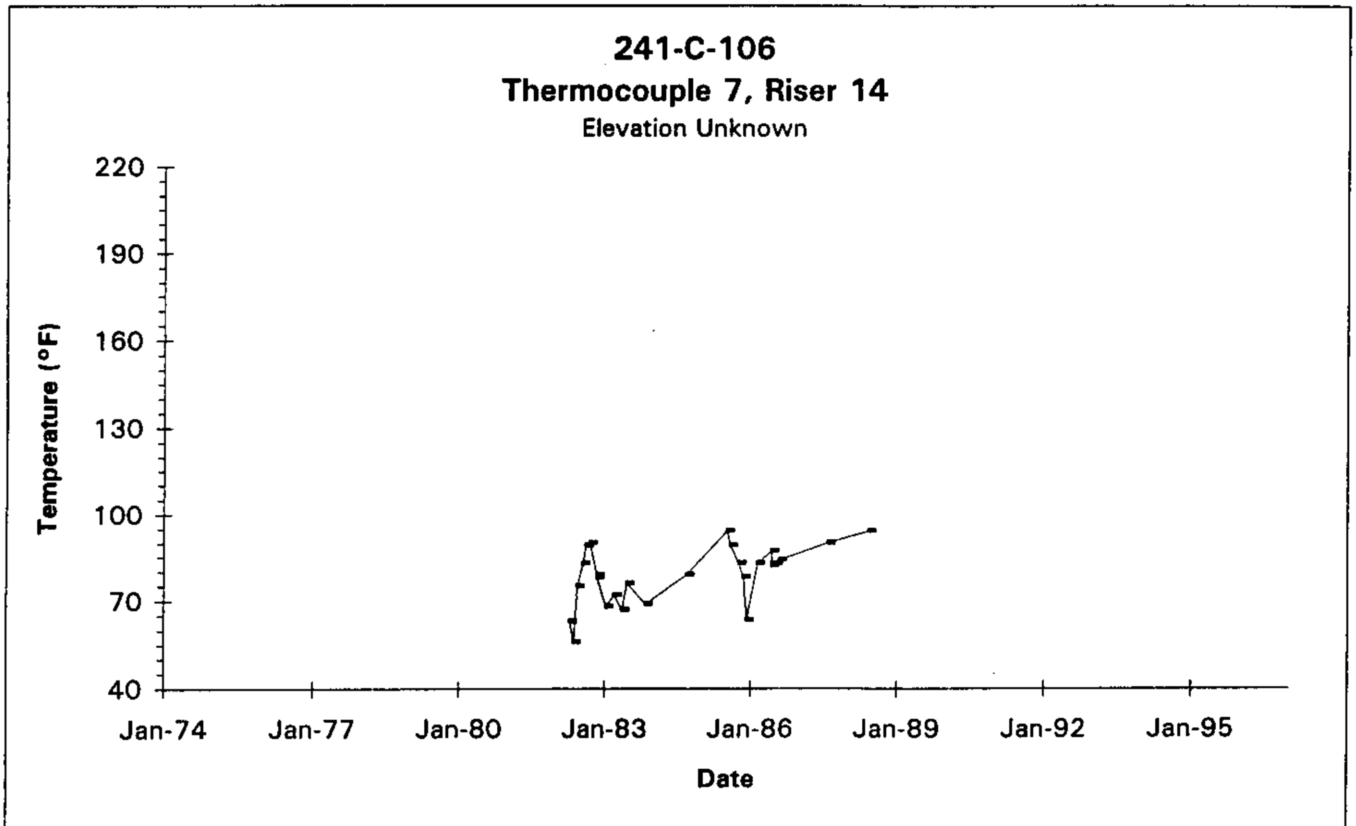
Figure B-21. Tank 241-C-106 Thermal History 1974-1995 Thermocouple 5 and 6, Riser 14.

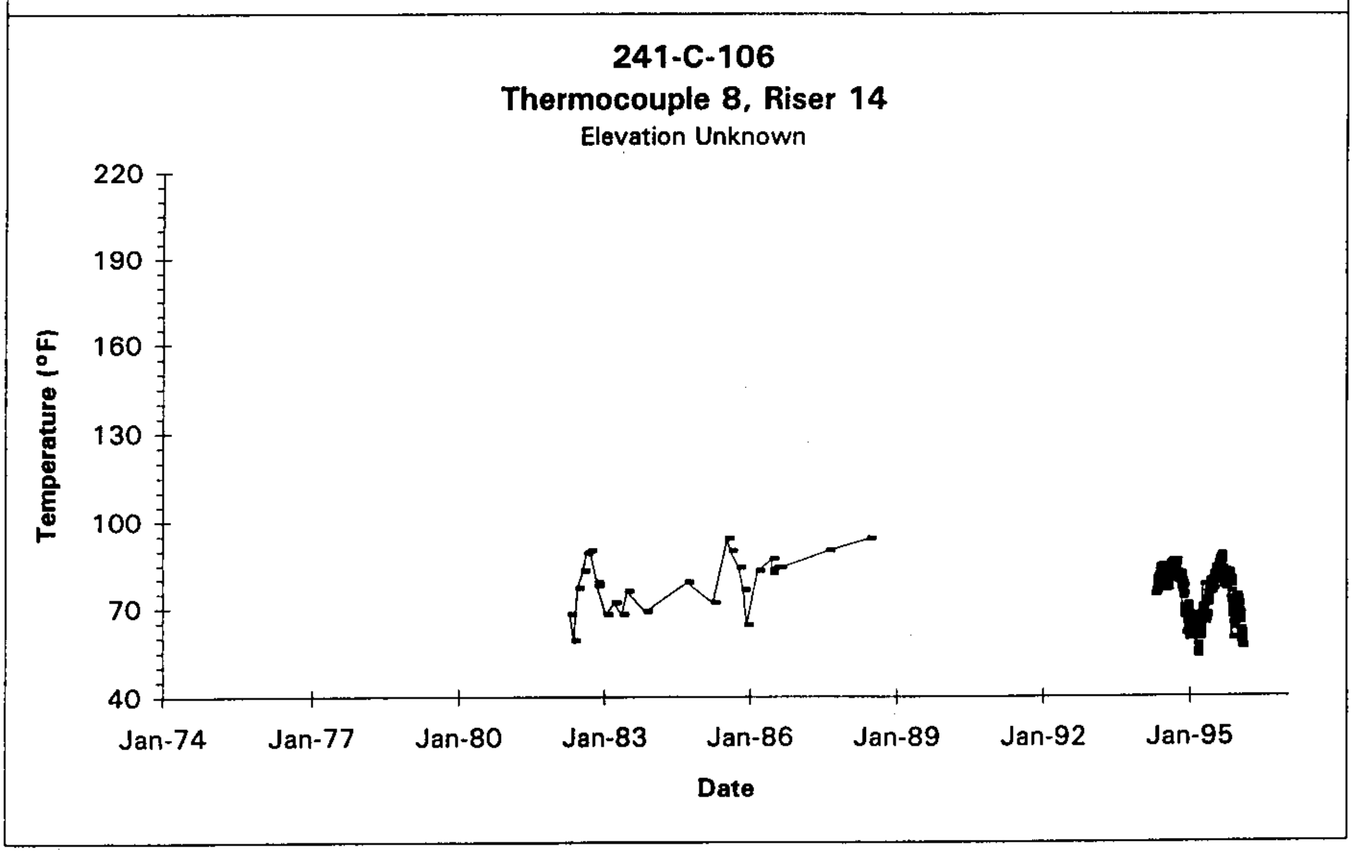




Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

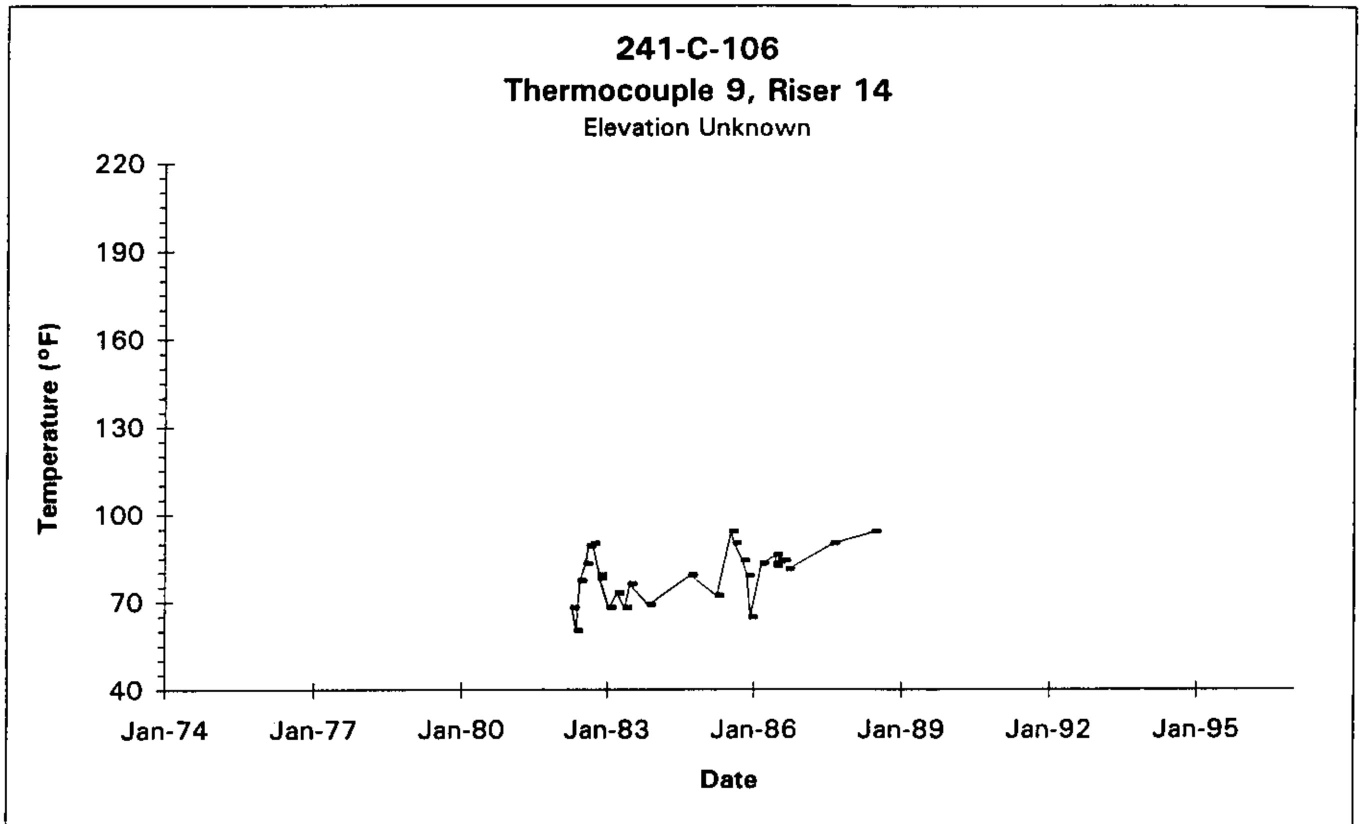
Figure B-22. Tank 241-C-106 Thermal History 1974-1995 Thermocouple 7 and 8, Riser 14.

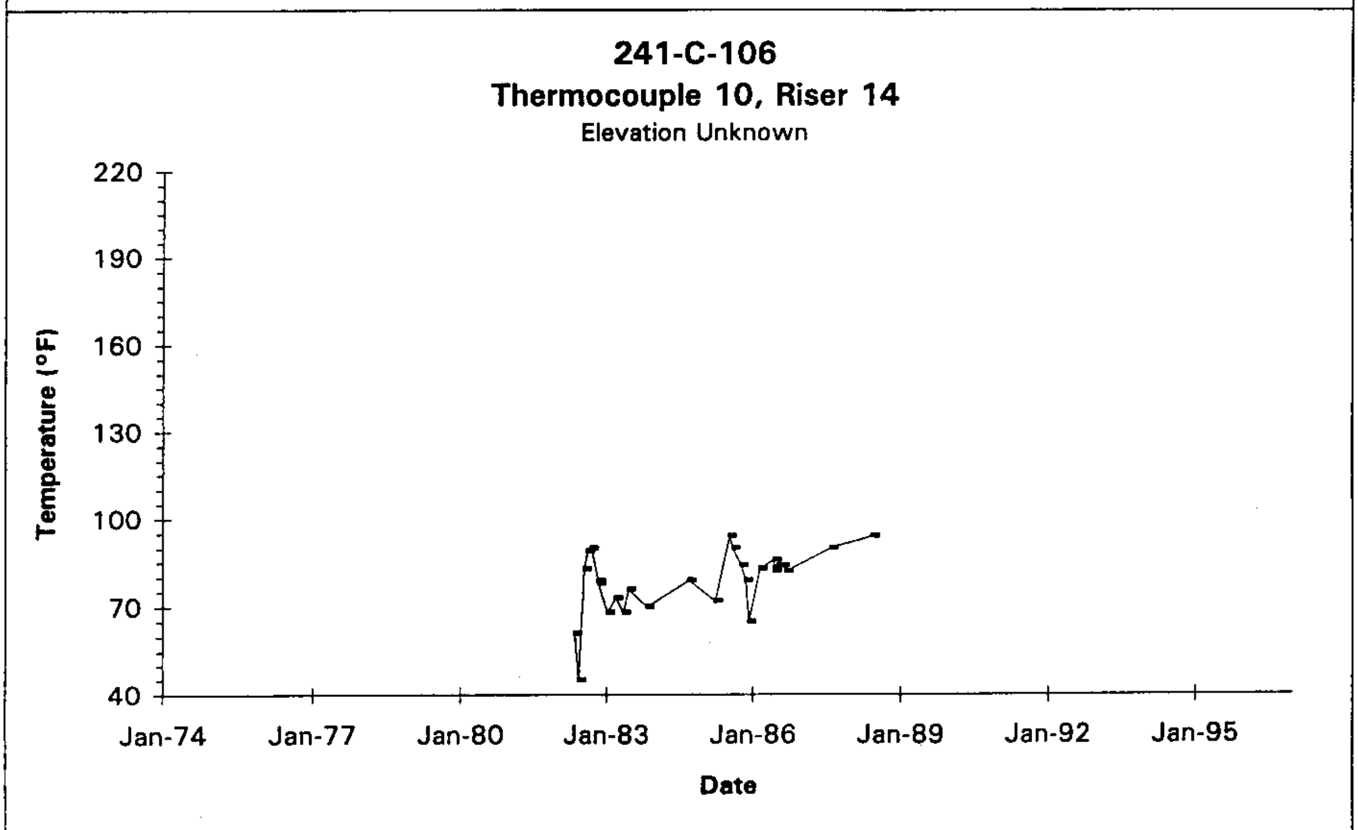




Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

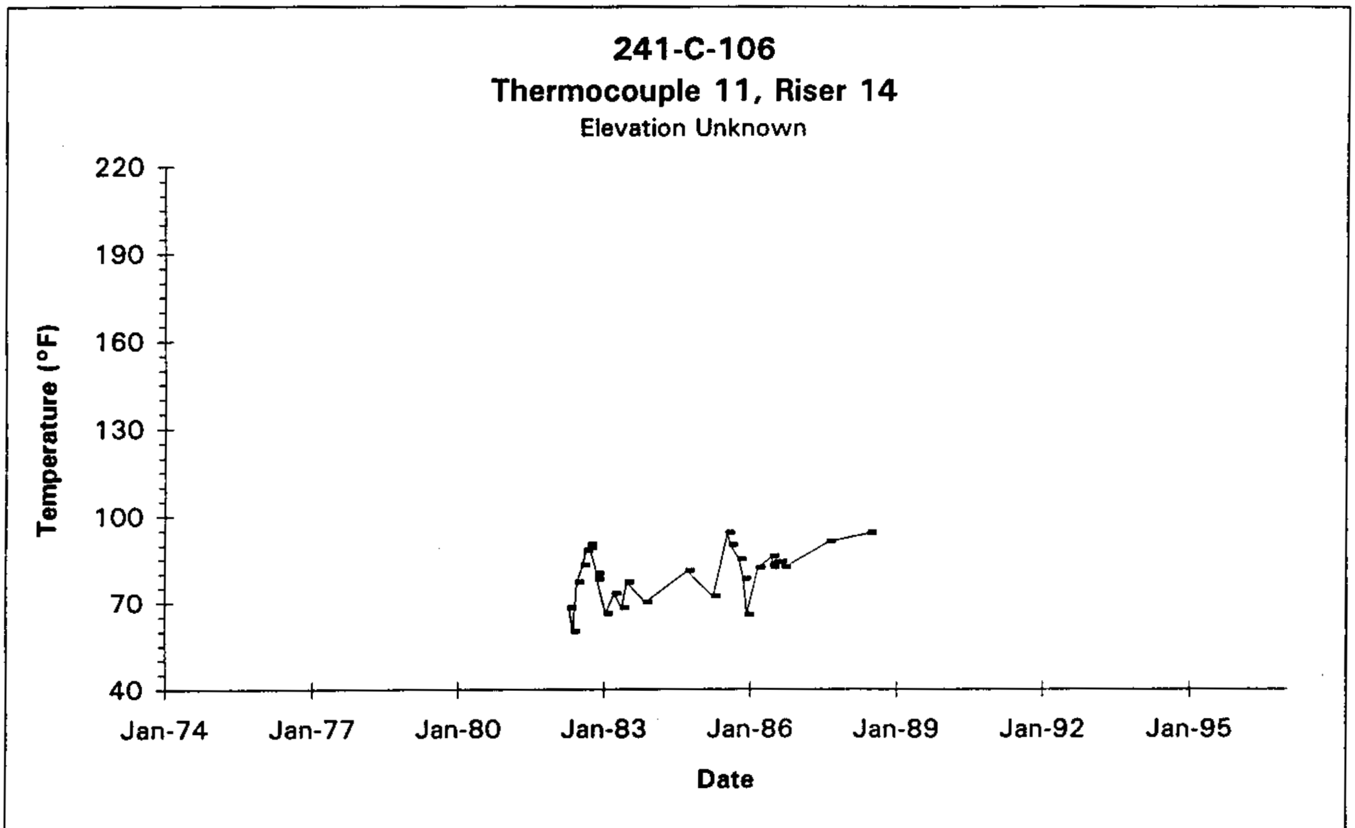
Figure B-23. Tank 241-C-106 Thermal History 1974-1995 Thermocouple 9 and 10, Riser 14.

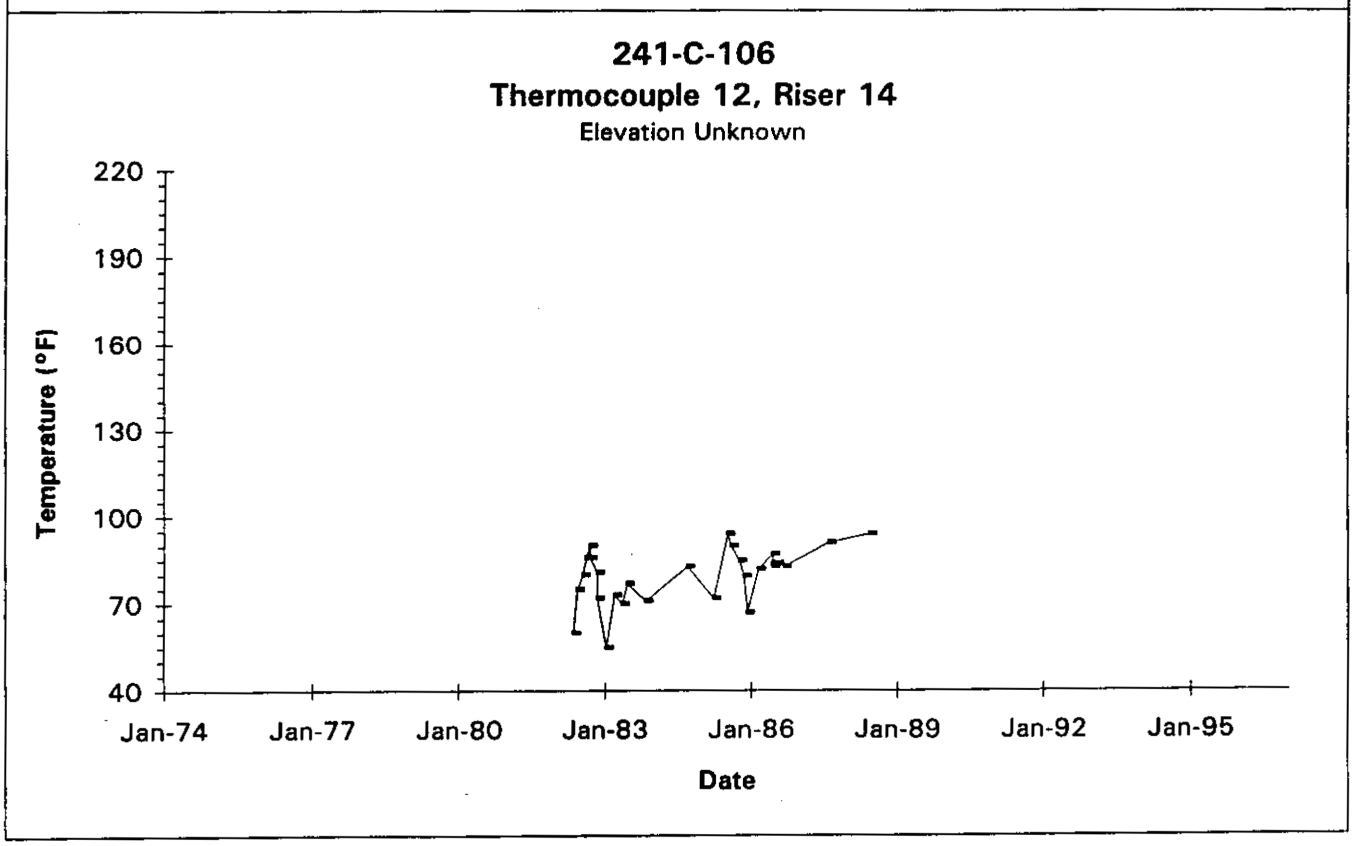




Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

Figure B-24. Tank 241-C-106 Thermal History 1974-1995 Thermocouple 11 and 12, Riser 14.

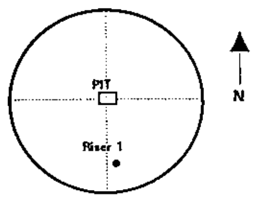




Data obtained from Surveillance Analysis Computer System (SACS), January 9, 1996.

Table B-1. Tank 241-C-102 Dome Elevation Survey Data Log 1984-1998.

Page 1

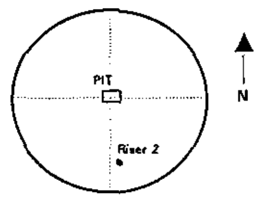


Tank Dome Elevation Survey Data Log

Tank Number 102-C

Date	File Number	Benchmark on Riser PIT — Baseline (feet) 8/15/84 648.229 WHC — Elevation in feet	Δ from Prev. Elev.	Cumulative Δ	Exceed Defl. Criteria?	Benchmark on Riser 1 — Baseline (feet) 8/15/84 647.204 WHC — Elevation in feet	Δ from Prev. Elev.	Cumulative Δ	Exceed Defl. Criteria?	Baseline (feet) 8/15/84 0.000 WHC — Elevation in feet	Δ from Prev. Elev.	Cumulative Δ	Exceed Defl. Criteria?	Baseline (feet) 9/5/84 Needed WHC — Elevation in feet	Δ from Prev. Elev.	Cumulative Δ	Exceed Defl. Criteria?
		Baseline (feet) *8/15/84*	*648.229*	*WHC*		*Baseline (feet)* *8/15/84*	*647.204*	*WHC*		*Baseline (feet)* *8/15/84*	*0.000*	*WHC*		*Baseline (feet)* *8/15/84*	*0.000*	*WHC*	
8/15/84		648.229	0.000	0.000	N	647.204	0.000	0.000	N								
9/4/86		648.229	0.000	0.000	N	647.203	-0.001	-0.001	N								
7/21/88		648.233	0.004	0.004	N	647.207	0.004	0.003	N								
1/25/90		648.238	0.005	0.009	N	647.213	0.006	0.009	N								
12/14/91		648.234	-0.004	0.005	N	647.208	-0.005	0.004	N								
10/1/93		648.230	-0.004	0.001	N	647.205	-0.003	0.001	N								
11/28/95	2ECF-005	648.236	0.006	0.007	N	647.212	0.007	0.008	N								
12/23/97	2ECF-005	No reading-supplied air required				No reading-supplied air required											
1/13/98	2ECF-005	648.236	0.000	0.007	N	647.211	-0.001	0.007	N								

Table B-2. Tank 241-C-104 Dome Elevation Survey Data Log 1984-1997.



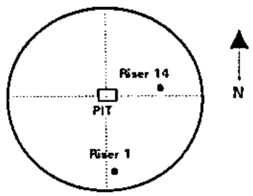
Tank Dome Elevation Survey Data Log

Tank Number 104-C

		Baseline (feet) 8/15/84 649.245 WHC				Baseline (feet) 8/15/84 648.129 WHC				Baseline (feet) 8/15/84 0.000 WHC				Baseline (feet) 9/5/84 Needed WHC			
		Benchmark on Riser PIT			Exceed	Benchmark on Riser 2			Exceed				Exceed				Exceed
Date	File Number	Elevation in feet	Δ from Prev. Elev.	Cumulative Δ	Defl. Criteria?	Elevation in feet	Δ from Prev. Elev.	Cumulative Δ	Defl. Criteria?	Elevation in feet	Δ from Prev. Elev.	Cumulative Δ	Defl. Criteria?	Elevation in feet	Δ from Prev. Elev.	Cumulative Δ	Defl. Criteria?
		Baseline (feet) 8/15/84 649.245 WHC				*Baseline (feet) 8/15/84 648.129 WHC*				*Baseline (feet) 8/15/84 0.000 WHC*				*Baseline (feet) 8/15/84 0.000 WHC*			
8/15/84		649.245	0.000	0.000	N	648.129	0.000	0.000	N								
9/4/86		649.237	-0.008	-0.008	N	648.119	-0.010	-0.010	N								
7/21/88		649.236	-0.001	-0.009	N	648.120	0.001	-0.009	N								
1/25/90		649.236	0.000	-0.009	N	648.120	0.000	-0.009	N								
12/14/91		649.233	-0.003	-0.012	N	648.119	-0.001	-0.010	N								
10/1/93		649.234	0.001	-0.011	N	648.119	0.000	-0.010	N								
11/13/95	2ECF-005	649.234	0.000	-0.011	N	648.120	0.001	-0.009	N								
12/23/97	2ECF-005	649.232	-0.002	-0.013	N	648.120	0.000	-0.009	N								

Table B-3. Tank 241-C-106 Dome Elevation Survey Data Log 1984-1997.

Page 1



Tank Dome Elevation Survey Data Log

Tank Number 106-C

		Baseline (feet) 8/15/84 647.210 WHC				Baseline (feet) 8/15/84 646.146 WHC				Baseline (feet) 8/15/84 645.528 WHC				Baseline (feet) 9/5/84 Needed WHC			
		Benchmark on Riser PIT				Benchmark on Riser 1				Benchmark on Riser 14							
Date	File Number	Elevation in feet	Δ from Prev. Elev.	Cumulative Δ	Exceed Defl. Criteria?	Elevation in feet	Δ from Prev. Elev.	Cumulative Δ	Exceed Defl. Criteria?	Elevation in feet	Δ from Prev. Elev.	Cumulative Δ	Exceed Defl. Criteria?	Elevation in feet	Δ from Prev. Elev.	Cumulative Δ	Exceed Defl. Criteria?
		Baseline (feet) *8/15/84* *647.210* *WHC*				*Baseline (feet)* *8/15/84* *646.146* *WHC*				*Baseline (feet)* *8/15/84* *645.528* *WHC*				*Baseline (feet)* *8/15/84* *0.000* *WHC*			
8/15/84		647.210	0.000	0.000	N	646.146	0.000	0.000	N	645.528	0.000	0.000	N				
9/4/86		647.214	0.004	0.004	N	646.152	0.006	0.006	N	645.533	0.005	0.005	N				
7/21/88		647.220	0.006	0.010	N	646.157	0.005	0.011	N	645.540	0.007	0.012	N				
1/25/90		647.220	0.000	0.010	N	646.162	0.005	0.016	N	645.538	-0.002	0.010	N				
12/14/91		647.212	-0.008	0.002	N	646.154	-0.008	0.008	N	645.531	-0.007	0.003	N				
10/1/93		No	Readings	Available		646.153	-0.001	0.007	N	645.532	0.001	0.004	N				
11/18/95	2ECF-005	647.212	0.000	0.002	N	No	Readings	Available		645.532	0.000	0.004	N				
12/23/97	2ECF-005	647.212	0.000	0.002	N	No	Readings	Available		645.531	-0.001	0.003	N				

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