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WHC-SD-WM-ES-218 (Rev. 1), *Engineering Study of Tank Leaks Related to Hydraulic Retrieval of Sludge from Tank 241-C-106*

EDT No.:

ECN No.: 197822

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Page 2 of 2

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SDD/DD	☐	Seismic/Stress Analysis	☐	Tank Calibration Manual	☐
Functional Design Criteria	☐	Stress/Design Report	☐	Health Physics Procedure	☐
Operating Specification	☐	Interface Control Drawing	☐	Spares Multiple Unit Listing	☐
Criticality Specification	☐	Calibration Procedure	☐	Test Procedures/Specification	☐
Conceptual Design Report	☐	Installation Procedure	☐	Component Index	☐
Equipment Spec.	☐	Maintenance Procedure	☐	ASME Coded Item	☐
Const. Spec.	☐	Engineering Procedure	☐	Human Factor Consideration	☐
Procurement Spec.	☐	Operating Instruction	☐	Computer Software	☐
Vendor Information	☐	Operating Procedure	☐	Electric Circuit Schedule	☐
OM Manual	☐	Operational Safety Requirement	☐	ICRS Procedure	☐
FSAR/SAR	☐	IEFD Drawing	☐	Process Control Manual/Plan	☐
Safety Equipment List	☐	Cell Arrangement Drawing	☐	Process Flow Chart	☐
Radiation Work Permit	☐	Essential Material Specification	☐	Purchase Requisition	☐
Environmental Impact Statement	☐	Fac. Proc. Samp. Schedule	☐	None Applicable	☒ [X]
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**ENGINEERING STUDY OF TANK LEAKS
RELATED TO HYDRAULIC RETRIEVAL OF SLUDGE
FROM TANK 241-C-106**

Issued by

Westinghouse Hanford Company

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ABBREVIATIONS AND ACRONYMS

ACI	American Concrete Institute
ALARA	as low as reasonably achievable
ASTM	American Society for the Testing of Materials
CERCLA	<i>Comprehensive Environmental Response, Compensation, and Liability Act of 1980</i>
DOE	U.S. Department of Energy
DSS/DSSF	double-shell slurry/double-shell slurry feed
DST	double-shell tank
Ecology	Washington State Department of Ecology
EIS	environmental impact statement
EPA	U.S. Environmental Protection Agency
FSAR	final safety analysis report
HEPA	high efficiency particulate air (filter)
LLW	low-level waste
PSAR	preliminary safety analysis report
PSE	preliminary safety evaluation
RCRA	<i>Resource Conservation and Recovery Act of 1976</i>
SRP	Savannah River Plant
SST	single-shell tank
TSD	treatment, storage, and disposal
WAC	Washington Administrative Code
WHC	Westinghouse Hanford Company

1.0 SUMMARY

Single-shell tank (SST) 241-C-106 is the only SST that requires periodic water additions and active ventilation to keep the waste temperature below 148 °C (300 °F). The water enhances the thermal conductivity of the sludge and increases the heat transfer at the surface due to the evaporative cooling effect. The requirement for periodic water additions and the uncertainty in tank response should these water additions be stopped has raised a safety concern for tank 241-C-106. One remedy for the safety issue is retrieval of all or part of the sludge such that the heat generation rate is reduced to the level that water additions may be stopped.

Single-shell tank waste retrieval demonstration is required under the Hanford Federal Facility Agreement and Consent Order [known as the Tri-Party Agreement (TPA), Ecology et al. 1990] milestone M-07. Tank 241-C-106 has been recommended as the retrieval demonstration tank. Hydraulic retrieval (i.e., sluicing) has been proposed as the method to demonstrate retrieval (milestone M-07) and as a reference retrieval technique for SST waste. There is a concern that the addition of large quantities of water to SSTs over the total time to retrieve the wastes may induce tank leaks.

The purpose of this study is to evaluate hydraulic retrieval of the waste in tank 241-C-106 with respect to the likelihood of tank leaks, gross volumes of potential leaks, and their consequences. A description of sluicing was developed to establish a baseline for the study. Historical leak information for the Hanford Site and the Savannah River Plant (SRP) was evaluated to identify postulated leak mechanisms. Leak models were developed to estimate the amount of waste that could potentially leak while sluicing. Transport models were used to describe the movement of the waste constituents in the surrounding soil to the groundwater after a leak occurs. The environmental impact and risk associated with the tank leaks was then evaluated.

Tank 241-C-106 is currently listed as sound (Westinghouse 1992a). The available leak source term prior to sluicing is the free-standing supernate on top of the sludge, and the drainable interstitial liquid within the sludge. The supernate would be removed prior to initiation of sluicing operations. The process of sluicing retrieval would involve jetting a slurry pump into the waste, and mobilizing the sludge (i.e., washing the sludge towards the pump) with two high volume, low pressure sluicers. This process would slowly uncover the tank wall as the waste is removed from the tank. Because the tank is not now leaking, it can be assumed that either there are no cracks or holes in the liner, or that these defects are plugged with waste. The leak source term during sluicing would then be any free-standing liquid present in the tank during the sluicing operation, the drainable interstitial liquid, and the sluicing stream as it impacts the tank wall. The total amount of free liquid present in the tank at any time during the retrieval operation will be limited to 20-25 m³ (5000-7000 gal).

There is considerable uncertainty in the estimate of the amount that may leak during sluicing, as the mechanisms by which leaks may occur are not fully understood. The most likely leak path is through many small cracks caused by stress corrosion cracking around welds in the tank liner, and subsequently out through the construction joint in the concrete encasement. The extent of

stress corrosion cracking of the liner is unknown. Results of a model developed at the SRP which predicts whether stress corrosion cracking may occur based on the chemistry of the waste are inconclusive. The concentrations of hydroxide and nitrate in the waste lie between the values for which cracking is either predicted to occur or not occur. The structural analysis which estimates the extent of cracking in the concrete encasement is incomplete. The actual leaked volume is expected to be on the order of a few cubic meters (several thousand gallons), based on historical leak rates and the assumption that sluicing would be halted upon detection of a leak. However, the analyses showed that leakage up to 150 m^3 (40,000 gal) may occur during sluicing by the most likely leak mechanism. This analytical conclusion reflects the conservative assumptions used to establish an upper bound on the amount of leakage and its impact. The leak rate is determined by the hydraulic head and the resistances of the cracks in the liner and the cracks in the concrete encasement or the gap in its construction joint, and the concrete-to-soil interface saturation/permeation level. The hydraulic head, the driving force behind any leaks, will decrease as sluicing progresses. Liquid is held up in the sludge to an estimated height of 61 cm (24 in.) by capillary forces alone. The hydraulic resistance of the sludge would impede liquid flow to any leak sites. The size of any leaks would be further limited by (1) the ability to detect leaks, (2) administrative controls on liquid inventories, (3) the tendency of solids in the sludge to plug any leaks, and (4) that free liquid in the sluice tank is limited and could be pumped out in a short time.

Any postulated waste leak, upon reaching the soil will be driven downward by rainfall and runoff from the tank dome. This moisture recharge is concentrated around the tank perimeter due to the "umbrella effect" of the tank structure. Travel time to the aquifer is calculated to be about 60 yr, provided the amount leaked is small (up to tens of cubic meters) compared to the rate of recharge (see Section 6.3). Even small leaks are expected to find their way to the groundwater. The recharge rate could be reduced by providing an impermeable cover over the tank and surrounding area. The contaminated soil could be recovered or treated after sluicing if required as part of the overall site closure activities under TPA milestone M-09. Additional sensitivity analysis of a surface barrier's area versus the recharge rate is needed. The transport model used to simulate the leak is conservative in that the travel time estimates are minimal for representative conditions and the relative concentrations in the groundwater are maximal.

Local concentrations of key waste constituents in the groundwater resulting from the most likely leak scenario are predicted to be greater than allowed by drinking water standards. These doses reflect the inherent conservatism in the transport model. The actual dose received will depend on the water well location, the extent of mixing in the aquifer, and any lateral spreading that occurs. Significant dilution is possible between the source of the leak and the dose receptor, such that the resulting dose may approximate or even be less than allowed by drinking water standards. This study assumed a well located 25 m from the tank.

The current leak detection system's sensitivity to leakage volumes or locations should be determined. Its capabilities may be enhanced and combined with process controls to monitor the waste inventory and liquid volumes in order to better detect leaks during sluicing. More frequent monitoring would

shorten the time to detect a leak at the monitoring drywells. Providing lateral monitoring wells beneath the tank could allow detection of smaller leaks. These methods could offer greater sensitivity for detecting small leaks.

Corrective actions, should a leak be detected, can include stopping the addition of water to the tank. During the initial stages of retrieval, the sludge must be kept moist to prevent overheating. In the latter stages of retrieval when only a small amount of sludge remains and leaking is most likely to occur, the heat generated would not be excessive and water additions could be stopped altogether.

Installation of a sub-surface barrier prior to retrieval operations would also limit the migration of any contaminants to the vicinity of the tank, should a leak develop. Development and design work is required on sub-surface barriers before they are proven to be viable.

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2.0 INTRODUCTION

2.1 BACKGROUND

A program was begun in the late 1960s to reduce waste volumes and remove all single-shell tanks (SSTs) from liquid storage service. Problems with a sludge washing/solids settling step resulted in the inadvertent transfer of aging waste sludge to tank 241-C-106. The tank presently contains a total of 746 m³ (197,000 gal or 68 in. height) of sludge and 121 m³ (32,000 gal or 12 in. height) of supernate; there is approximately 182 m³ (48,000 gal) of drainable liquid (Westinghouse 1992a). Radionuclides in the sludge generate heat at the rate of approximately 44 kW (150,000 Btu/hr). Water is periodically added to the tank for evaporative cooling and to promote heat transfer, and the tank is on active ventilation. The tank is classified as not leaking because the leak detection surveillance data indicate there has been no loss of liquid attributed to a breach in integrity.

The Tri-Party Agreement (Ecology et al. 1990) requires interim stabilization of tank 241-C-106 by September 1996 (milestone M-05-08). Interim stabilization would require removal of as much liquid as possible to limit the liquid source in case of a tank leak. The minimum amount of liquid necessary to prevent the temperature of the waste and tank dome to increase to unacceptable levels is not known. Limits have been established on these temperatures to ensure the structural integrity of the SSTs (Westinghouse 1990a). If the water additions to tank 241-C-106 are stopped, there is a potential for uncontrolled temperature increase which could lead to potential structural failure (i.e., dome collapse), with possible release of high-level waste. This concern caused tank 241-C-106 to be listed as a Hanford Waste Tank Safety Issue in accordance with Public Law 101-510, Section 3137 (Wyden Amendment).

One remedy of the safety issue includes retrieval of all or part of the sludge such that the heat generation rate in the tank is reduced to less than 12 kW (40,000 Btu/hr). Calculations have shown this to be the heat generation rate at which the temperature limits for the waste and tank dome will not be exceeded if the sludge is allowed to dry. Options for retrieving the sludge from tank 241-C-106 have been studied (Squires et al. 1991). Hydraulic retrieval was not recommended because it did not meet the evaluation criteria which required no leakage. Sluicing would be less costly and require a shorter time to implement than the recommended method. However, sluicing would introduce large volumes of water into a tank and could increase the potential for the tank to leak.

2.2 PURPOSE

The purpose of this study is to evaluate hydraulic retrieval of SST waste with respect to the likelihood of tank leaks, gross volumes of potential leaks, and their consequences. The results will be used (by others) in comparing hydraulic retrieval to other retrieval methods. Application to tank 241-C-106 will be emphasized initially; a generic approach with application to other SSTs (later) is also a study objective.

2.3 SCOPE

A description of hydraulic retrieval was developed to establish a baseline for the study. The description was based on past-practice sluicing of SSTs. Historical leak information was evaluated to establish postulated leak mechanisms. Leak models were developed to estimate the amount of waste that could potentially leak while sluicing. Transport models were developed that describe the movement of the waste constituents in the surrounding soil and groundwater after a leak occurs. The models were based on operating scenarios which represent bounding limits of the retrieval operation. The environmental impact and risk associated with the tank leaks is evaluated. Criteria were developed to provide the basis for evaluating the use of barriers, determining acceptable leak amounts, and establishing requirements for performance of the retrieval system. Work in fiscal year 1992 focuses on tank 241-C-106 only.

3.0 TANK-TO-TANK SLUICING

In the late 1960s, a program was initiated to reduce waste volumes and remove all SSTs from storage service. The following is a description of sluicing used to retrieve sludge from the 241-A and 241-AX tank farms. The description of the sluicing equipment and its operation is from *Hanford Radioactive Tank Cleanout and Sludge Processing* (Rasmussen 1980) and *Hanford Waste Tank Sluicing History* (Rodenhizer 1987). Sludge was sluiced in batches to the 244-AR Vault, dissolved with nitric acid, and shipped via the 244-CR Vault to 221-B Plant for recovery of strontium-90. Sluicing proceeded in this manner until only a small amount of sludge remained in each tank. The sludge heels in several tanks were then sluiced together and consolidated into another tank. (This method was called tank-to-tank sluicing.)

For the purposes of this study, sluicing of the sludge in tank 241-C-106 is assumed to be similar to past-practice tank-to-tank sluicing described above. The past operating conditions and equipment descriptions will be used in developing the leak model. The availability of a double-shell tank for receiving the sludge retrieved from tank 241-C-106 is assumed. Suitable tank farm support facilities and equipment (e.g., transfer lines, diversion boxes, etc.) which meet current requirements are also assumed to be available.

3.1 SLUICING PROCESS DESCRIPTION

Tank-to-tank sluicing is shown in Figure 1. The sluicing system used a recirculating liquid to carry sludge from one underground storage tank to another. The sludge receiver tank was initially charged with a large volume of supernate or water to be used as the sluicing liquid. Liquid from the receiver tank was pumped by the sluice pump through the sluice nozzle to dislodge and suspend the sludge in the sluice tank. The resultant slurry was pumped by the slurry pump back to the receiver tank. A large liquid inventory in the receiver tank was needed so the residence time of the liquid in the tank was long and the solids would settle.

The sluice pump delivered the sluicing liquid to the nozzle at rates up to $0.022 \text{ m}^3/\text{s}$ (350 gal/min). The liquid exited the 1-in. diameter nozzle and impinged on the sludge surface at velocities of up to 46 m/s (150 ft/s). Maximum sluicing efficiency was obtained when the liquid level above the sludge was minimized. The free liquid inventory in the sluice tank was maintained between 7 m^3 and 10 m^3 (2,000 and 3,000 gallons) at all times. Solids concentrations in the circulating sluicing liquid would reach 10 to 20% solids.

During initial sluicing operations, the sluicing nozzle was directed towards the slurry pump. Sludge surrounding the pump was rapidly retrieved in this manner. The nozzle then scanned the tank in a predetermined pattern chosen to cover the entire bottom of the tank and move sludge toward the intake of the slurry pump. Sluicing continued until a drop-off in solids recovery was noted. At low sludge levels, the liquid jet tended to push the solids around the tank bottom, reducing the sluicing efficiency. Below sludge levels of 10 to 15 cm (4 to 6 in.) the continuity of B Plant operations could not be maintained. In the mid 1970s, three tanks in the 241-A tank farm were

Figure 1. Tank-to-Tank Sluicing.

sluiced to residual sludge levels of approximately 4 cm (1.5 in.). The total time required to sluice a tank was typically 9-12 months including downtime, based on operating records. Final cleanout could take anywhere from several days to several weeks (Rodenhizer 1987).

Sludge retrieval progress was monitored by measuring the radiation level of the slurry in the return line from the sluice tank to the receiver tank. The specific gravity of the sluicing liquid was also measured. Sluicing was periodically stopped and the liquid in the sluice tank was pumped down in order to photograph the sludge surface to determine what sections of the tank required further sluicing. Less than an hour was required to pump out the liquid in the sluice tank.

3.2 SLUICING EQUIPMENT DESCRIPTION

Sluicer

The sluicer configuration is shown in Figure 2. The sluicer rested on a base plate in the sluice pit or pump pit, and extended through a riser opening into the tank. Sluicing liquid was supplied by a short jumper in the pit to a vertical pipe connected to a flexible hose attached to a rotary joint and the sluicing nozzle. The nozzle was aimed precisely by means of a calibrated sluicer control unit (calibrated head) that provided for both horizontal and vertical adjustments. Two concentric control rods in a guide tube connected the nozzle to the calibrated head.

The pipe housing the two control rods extended through the concrete pit cover blocks into the calibrated head, where the control rods engaged the horizontal and vertical adjustment mechanisms. A turntable engaged the horizontal sweep control rod. A motorized drive unit connected to a rocker

Figure 2. Sluicer Configuration.

arm provided horizontal motion to the turntable. Various sweep widths and sluicer directions could be set by pinning the sweep arm into the proper slots. Depending upon the gear box connected to the arm, a sweep could be made once per hour or once per minute. Turning motion of the lower assembly was taken up by the flexible hose connecting the nozzle feed piping to the fixed main stem of the sluicer. The hose allowed horizontal turning up to 120 degrees in either direction.

Vertical adjustment of the nozzle was made by means of a hydraulic-assisted activator connected to the vertical nozzle angle control rod. The control rod turned the rotary pipe joint (Chiksan joint) of the sluicer. The joint allowed a 110 degree positive (forward) adjustment, and a 45 degree negative (backward) adjustment from vertical.

Slurry Pumps

Heavy-duty single-stage centrifugal slurry pumps were used to recover the bulk of the sludge. The pumps were manufactured by Barrett Haentjens and generally gave years of service under extreme operating conditions. The pumps were lubricated with water which was completely isolated from the process liquid. The intake of the slurry pump was funnelled to allow operation at the low liquid levels necessary for sluicing. The intake was designed to support the 5,000-kg (6-ton) weight of the pump. A ring of water jets around the intake was used to sluice the pump into the sludge if necessary. If for some reason the pump could not be fully inserted into the tank, a spool piece was used on the riser to raise the pump 15 cm (6 in.).

Turbine pumps were used for final cleanout of tanks where the solids concentration in the slurry was low (less than 4%). Both process liquid- and oil-lubricated pumps were successfully utilized. A sheet metal skirt around the intake was effective at minimizing vortexing and wave action which was expected at low liquid levels. A jackscrew baseplate was used to adjust the length of the pump up to 15 cm (6 in.). The screws were turned by a remotely-operated wrench, gradually lowering the pump as the sludge level decreased. The jackscrews also eliminated the need for spacers if the tank bottom was uneven.

Sluice Pumps

When sludge was being retrieved in batches to the 244-AR Vault, a single-stage water lubricated centrifugal pump manufactured by Barrett Haentjens was used to pump sluicing liquid from the receiver tank to the sluicing nozzle. The 150-hp pump was constructed of carbon steel and provided years of trouble-free service. The pump had a rated capacity of 0.04 m³/s (600 gal/min) at a total dynamic head of 120 m (390 feet). The pump intake was located approximately 1.3 m (53 in.) above the bottom of the receiver tank.

Adjustable-length turbine pumps were used to pump sluicing liquid to the nozzle during tank-to-tank sluicing. These were the same pumps used for final cleanout of the sluice tank.

Failed Equipment Removal

Steel capsules were generally used as transport and burial containers for failed equipment. A variety of steel capsules were available for failed pumps, agitators, etc. The capsules were considered disposable and were buried after loading with failed equipment. Failed or obsolete equipment was generally washed down, sprayed with a plastic film to fix remaining contamination, loaded into a capsule and hauled to the burial ground. Removal and replacement of failed equipment had to be performed when weather conditions would permit and winds were calm. Some items such as pumps were transported to 221-T Plant for decontamination and repair, and then returned to service.

4.0 LEAK MODEL BASES

4.1 CONSTRUCTION OF TANK 241-C-106

Tank 241-C-106 is constructed of reinforced concrete encasement with a welded carbon steel liner on the bottom and sides. The tank has a 0.3-m (12-in.) dished bottom and a usable waste depth of approximately 4.9 m (16 ft) at the sidewall. The bottom of the tank consists of a 0.15-m (6-in.) layer of reinforced concrete covered with a 3 ply asphaltic membrane waterproofing, a 5-cm (2-in.) layer of grout and a 0.95-cm (3/8-in.) steel plate liner. The knuckle (transition from bottom to sidewall) is 0.95-cm (3/8-in.) steel plate. The tank sides are 0.33-m (13-in.) thick reinforced concrete. The steel liner extends 5.5 m (18 ft) up the straight side of the tank, varying in thickness from 0.64 to 0.95 cm (1/4 to 3/8 in.). The top of the steel liner is covered with lead flashing to prevent moisture from entering between the liner and concrete wall. The dome is constructed of 38-cm (15-in.) thick reinforced concrete. The tank vapor space is held at a slight negative pressure by the use of a mechanical ventilation system.

The tank liner was fabricated from American Society for the Testing of Materials (ASTM) A7-39 carbon steel. The outer concrete encasement was fabricated from Portland cement type II, and reinforced with ASTM A15-39 carbon steel. The steel liner provides a liquid-tight container while the concrete encasement provides all the necessary structural resistance to internal and external pressures and loads. The steel liner was designed to the American Water Works Association specification for elevated steel water tanks which was current at the time of construction. All the welds were X-rayed. The tank liner was not annealed after welding and residual internal stresses remain. General design data included service for liquids with a pH greater than 10 and at temperatures less than 104 °C (220 °F) (General Electric 1949).

4.2 STRESS CORROSION CRACKING OF CARBON STEEL LINER

Stress corrosion cracking can occur when the carbon steel liner is exposed to aqueous solutions containing sodium hydroxide and sodium nitrate. The cracks form in and near the welds in the tank sidewall where there are residual stresses. The period of time for cracking to occur cannot be predicted and may be anywhere from as little as a few months to several years.

Stress corrosion cracking led to failure of waste Tank 16 at the SRP in 1959 after only 5 years of service (Poe 1974). Tank 16 is constructed with a steel tank, or primary containment vessel, resting in a shallow steel pan, or saucer. A massive concrete encasement surrounds the tank and pan. The pan is a liner for the concrete encasement. Any leakage through the steel tank wall is collected in the pan in the annular space between the steel tank and concrete encasement. Approximately 700 m³ (185,000 gal) of waste leaked from the primary tank to the catch pan within the concrete encasement. Leak rates peaked at more than 300 mL/s (4 gal/min) when the tank contained up to 8 m (300 in.) of waste. Later examination revealed an estimated 350 leak sites. The leaks occurred from cracks up to 15 cm (6 in.) long and 0.05 mm (0.002 in.) wide which penetrated the tank wall. The cracks were located

perpendicular to welds in the heat-affected zone where large residual stresses would be expected. The leaks tended to be self-sealing in that the cracks would plug with solid deposits when low humidity was maintained in the annulus between the primary tank and the concrete encasement. The primary tank rests on a layer of sand in the catch pan and could not be inspected for leaks. A small amount of waste (tens of gallons) overflowed the catch pan and escaped through a construction joint in the concrete encasement. The opening in the construction joint in Tank 16 was estimated to be 0.8 mm (1/32 in.) from the observed leak rates. No leakage from the tank bottom due to stress corrosion cracking was detected.

A model was later developed to predict and prevent stress corrosion cracking, based on metallurgical analysis of the failed Tank 16 material and corrosion tests at SRP with simulated wastes (Ondrejcin 1978). The model was developed for PUREX-type wastes and allows one to graphically determine if a waste will cause nitrate-assisted stress corrosion cracking. The model predicts that, based on the chemistry of the waste in Tank 16 (i.e., nitrate concentration 4-5M and pH > 11), stress corrosion cracking is very likely to occur.

The waste in tank 241-C-106 is dilute in nitrate (0.02M) and the pH is approximately 9.8. The SRP model predicts these compositions are only borderline in causing stress corrosion cracking. The thickness of the liner in tank 241-C-106 is only 0.95 cm (3/8-in.) thick versus 1.6 cm (5/8-in.) thick in tank 16. Thicker materials tend to have higher residual stresses. Tank 241-C-106 contains less waste than tank 16 did when it leaked, so the number of cracks below the liquid level which could leak is less and the hydraulic head (the driving force behind any leaks) is also less. Sluicing could be expected to dissolve or displace deposits in these cracks, which may initiate leaking. However, the number of cracks in tank 241-C-106 is likely fewer than found in tank 16 due to differences in tank construction and waste chemistry, all other factors being equal.

4.3 THERMAL CRACKING OF CONCRETE ENCASEMENT

The concrete encasement for tank 241-C-106 was analyzed for cracking. The analysis was based on a thermal history of the tank developed separately. Cracking in the concrete wall and floor of the encasement is predicted as a result of earlier loading of high-heat waste material. The predicted crack distribution is sensitive to the thermal conditions, and the waste loading sequence and waste temperatures should be reviewed for correctness. Recommended crack sizes for use in leak-rate studies are provided in Appendix A. The preliminary structural analysis is contained in Appendix B.

Through-floor cracking is predicted with an integrated total crack opening of 0.03 cm (0.01 in.) at an average radius of 3.45 m (136 in.). The *American Concrete Institute (ACI) Manual of Concrete Practice* (ACI 1987) was applied to these results to predict an individual crack size of 0.005 cm (0.002 in.), which corresponds to 5 cracks in the floor of the concrete encasement. Based on the time-temperature history of the tank, these cracks are now partly closed. It is conservative to estimate a current crack opening of 0.003 cm (0.001 in.).

Through-wall cracking is predicted in the encasement sidewall between a vertical location of 1.72 m and 5.99 m (68 in. and 236 in.), measured from the bottom surface of the tank, with an integrated crack opening of 1.5 cm (0.6 in.). Guidance from the ACI manual predicts a crack size of 0.0061 cm (0.0024 in.), which corresponds to 250 cracks over the tank circumference based on the rebar size and spacing in the wall. These cracks also are partly closed now, based on the time-temperature history. A conservative crack opening of 0.003 cm (0.001 in.) may be assumed for leak-rate estimation.

4.4 CHARACTERIZATION OF WASTE IN TANK 241-C-106

Tank 241-C-106 presently contains 746 m³ (197,000 gal or 68 in. height) of sludge and 121 m³ (32,000 gal or 12 in. height) of supernate. The sludge contains 61 m³ (16,000 gal) of drainable interstitial liquid, based on an estimated sludge porosity of 12.5% and a capillary height (the height to which liquid is held up in the sludge by capillary forces) of 61 cm (24 in.) (Westinghouse 1992a, Kirk 1980).

A core sample taken from tank 241-C-106 in 1986 was analyzed to determine the composition of the sludge solids and drainable liquid. The results are provided in Tables 1 and 2, respectively (Weiss and Schull 1988). The portion of the sludge which is water soluble is also shown. A layer of hard sludge approximately 30 cm (12 in.) thick was encountered at the bottom of tank 241-C-106 during core sampling. Rotary drilling was used with approximately 15,000 kPa (2200 lb_f/in.²) bit pressure to obtain the rest of the sample. Radiation measurements taken in the field during core sampling indicate the activity in the sludge is roughly the same at the top and bottom of the sludge, with a slight increase (25%) in the center.

4.5 ASSUMPTIONS USED IN LEAK-RATE CALCULATIONS

Following is a list of assumptions used to determine the potential leak rate from the tank.

- Cracks in the tank liner are initially plugged by solidified waste. The tank is not leaking.
- Double-shell tank (DST) 241-AY-102 is designated as the sludge receiver tank. Tank 241-AY-102 contains 121 m³ (32,000 gal) of sludge and 2050 m³ (542,000 gal) of supernate (Westinghouse 1992b).
- Supernate in tank 241-C-106 is pumped out to the sludge receiver tank.
- The supernate already in tank 241-AY-102, plus the supernate from tank 241-C-106 and additional dilute low-level waste (LLW), is used as the sluicing liquid.
- The slurry pump is installed in the pump pit of tank 241-C-106. One sluicer is installed on the opposite side of the tank in the sluice pit. Another sluicer is installed in the pump pit alongside the slurry pump to promote mixing. The slurry pump is jetted into the

Table 1. Composition of Tank 241-C-106 Sludge Solids.

Component	Water Soluble Fraction	Total
Ag $\mu\text{g/g}$	1.36	529
Al	21.5	40900
B	4.77	19.5
Ba	2.89	4890
Bi	15.0	501
Ca	45.6	11900
Cd	12.0	370
Co	0	4.81
Cr	1.42	984
Cu	1.52	128
Fe	1.11	52100
K	156	1470
Mg	11.4	6560
Mn	4.94	1840
Na	45500	117000
Ni	32.2	973
P	440	2910
Pb	44.9	2410
Si	24.7	71000
Sr	0	103
U	7.48	406
Zn	0	46.3
Zr	113	2170
F $\mu\text{g/g}$	unk	720
NO ₃	928	928
SO ₄	unk	4850
TOC $\mu\text{g/g}$	4620	4620
C-14 $\mu\text{Ci/g}$	2.32E-04	2.32E-04
Co-60	0.0279	0.888
Sr-90	0.204	1980
Tc-99	0.0353	0.219
Ru/Rh-106	unk	900
Sb/Te-125	unk	9.49
I-129	unk	8.10E-05
Cs-137	62.2	330
Pu-239/240	0.245	3.05
Am-241	0.0150	1.05
Total Gamma	63.7	363
pH		10.5
SpG g/mL		1.43

Radionuclide activities as measured in ~1986.

Table 2. Composition of Tank 241-C-106 Drainable Liquid.

Component	Amount	Component	Amount
Ag mg/L	10.1	C-14 $\mu\text{Ci/L}$	0.351
Al	34.3	Co-60	12.7
B	10.4	Sr-90	1650
Ba	5.00	Tc-99	141
Bi	111	Ru/Rh-106	300
Ca	11.4	Cs-137	27800
Cd	25.2	Pu-239/240	978
Cr	6.17	Am-241	13.9
Cu	3.32	Total Gamma	27800
Fe	12.4		
K	422		
Mg	12.8		
Mn	202		
Na	94900		
Ni	71.5		
P	344		
Pb	81.8		
Si	105		
U	958		
Zr	293		
F mg/L	320	pH	9.81
NO ₃	1430	SpG g/mL	1.22
SO ₄	6470	Viscosity cP	3.66
TOC mg/L	2520		

Radionuclide activities as measured in ~1986.

sludge only to the depth necessary to pump out the added sluicing liquid, keeping the layer of sludge between the pump and the tank walls and bottom as thick as possible.

- The level of free liquid around the slurry pump is maintained at 61 cm (24 in.) throughout the sluicing operation.
- Free liquid is pumped out during periods of downtime. The sludge is kept moist by sprinkling with water to prevent overheating.
- During the initial stages of sluicing, the tank walls are protected by a layer of sludge. The sluicing liquid does not impinge directly on the tank liner and no leakage occurs.
- During the last 500 hr of actual sluicing, the sludge on all or a portion of the tank walls is removed exposing the liner. The sluicing liquid dissolves the deposits filling the cracks in the tank liner, and leaking occurs.

- Free liquid is maintained below the elevation at which cracks may exist in the sidewall of the concrete encasement.
- The layer of hard sludge in the bottom of the tank is impermeable to the sluicing liquid and is undisturbed by sluicing.
- No credit is taken for the asphalt membrane between the tank liner and concrete encasement. (The condition of the asphalt membrane cannot be estimated after having been exposed to high temperatures and radiation for almost 50 years.)
- The vent system normally operates at a gage pressure of -0.5 kPa (-2 in. water); the Limiting Condition for Operation, based on maintaining the structural integrity of the tank, is a gage pressure of -2.5 kPa (-10 in. water).

4.6 LEAK MECHANISMS

Several leak mechanisms were postulated for this study. The most likely leak path is flow through stress corrosion cracks in the tank liner, with subsequent seepage out through the construction joint. This is based on the tank operating history and results of the structural analysis, and assumptions made as to the planned sluicing operation. Leakage would occur as the layer of sludge protecting the tank liner is removed by sluicing. Approximately 20 stress corrosion cracks are postulated to currently exist around the tank circumference in the vertical 61 cm (24 in.) portion of the tank liner immersed in the free liquid. The cracks in the 0.95-cm (3/8-in.) thick liner are assumed to be 10 cm (4 in.) long by 0.005 cm (0.002 in.) wide, and have an average surface roughness of 0.003 cm (0.001 in.) (Abdollahian and Chexal 1983). This is based on the length of welds used to join the plates in constructing tank 241-C-106, and assumes a cracking frequency and crack sizes similar to that experienced by tank 16 at the SRP. The construction joint is under a large compressive load due to the weight of the tank wall, dome, and soil overburden. It is highly improbable that there is any opening at all in the construction joint, even during any cooldown period during sluicing. A crack opening of 0.003 cm (0.001 in.) with an average surface roughness of 0.0005 cm (0.0002 in.) is conservative to assume for leak-rate calculations. The length of the flow path through the construction joint is approximately 78.0 cm (30.7 in.).

Other leak mechanisms were considered. These leak mechanisms and operating conditions are further described in Appendix C. Figure 3 shows a cross-section of the tank and the potential leak locations. Changes in some of the assumptions were investigated to determine the effect on the predicted leak rates. The operating conditions were also varied to determine the sensitivity of the leak rates to changes in hydraulic head and tank pressure.

4.7 SOURCE TERM USED FOR DOSE ASSESSMENT

The sluicing liquid consists of the supernate already in tank 241-AY-102, plus the supernate added from tank 241-C-106 and additional dilute LLW. The

Figure 3. Cross Section of Tank 241-C-106 Showing Potential Leak Paths.

volume of dilute LLW added is such that the volume of the supernate and sludge in tank 241-AY-102, and the supernate and sludge from tank 241-C-106, plus the dilute LLW will fill a DST [i.e., 3785 m³ (1 million gal)]. The supernate in tank 241-AY-102 and the dilute LLW are assumed to have the composition of typical double-shell slurry/double-shell slurry feed (DSS/DSSF) diluted to 0.1M Na. This assumption is necessary because tank 241-AY-102 is a receiver tank for noncomplexed dilute LLW; the composition of the LLW varies considerably based on the source.

The action of the sluicing liquid will tend to dissolve the soluble components of the sludge in tank 241-C-106. During the last 500 hr of actual sluicing, all of the soluble fraction of the sludge is assumed to have dissolved and mixed with the sluicing liquid in the sludge receiver tank. The free liquid in the sluice tank then has the same composition as the sluicing liquid. This is conservative in that some sludge will remain in tank 241-C-106 that has not been contacted by the sluicing liquid; the extent of dissolution achieved by the sluicing liquid will also likely be less than measured in the laboratory with the core sample. Volume changes on sludge dissolution are assumed negligible. Any liquid that leaks will have the composition of the sluicing liquid during the last 500 hr of actual sluicing. The composition of the leaked liquid (and the source term for assessing the impact of any leaks) is shown in Table 3.

Table 3. Composition of Leaked Liquid.

Component	Amount	Component	Amount
Ag mg/L	0.92	C-14 $\mu\text{Ci/L}$	0.32
Al	310	Co-60	11
B	2.2	Sr-90	160
Ba	1.3	Tc-99	21
Bi	10	Ru/Rh-106	62000
Ca	18	Sb/Te-125	170
Cd	5.5	I-129	4.78E-03
Co	2.2E-07	Cs-137	30000
Cr	5.1	Pu-239/240	130
Cu	0.74	Am-241	6.3
Fe	1.1		
Hg	0.056		
K	2200		
Mg	4.8		
Mn	10		
Mo	0.55		
Na	23000		
Ni	15		
P	180		
Pb	20		
Ru	2.5E-05		
Si	15		
U	44		
Zn	0.17		
Zr	53		
Cl mg/L	61		
CO ₃	52		
F	150		
NO ₂	820		
NO ₃	2100		
OH	1500		
PO ₄	6.9		
SO ₄	1200		
TOC mg/L	1800		

Radionuclide activities as measured in ~1986.

5.0 LEAK RATE DETERMINATION

A table of nomenclature used in calculating the leak rates is provided at the end of this section.

5.1 LAMINAR TUBE FLOW

The general parameters that define the viscous flow of a fluid through a straight leak passage of circular section are expressed in familiar form by the Hagen-Poiseuille equation (Bird et al. 1960):

$$P_o - P_b = \frac{8\mu V_s L}{r^2} \quad (1)$$

The validity of the Poiseuille equation is predicated on the following assumptions used in the development of the laminar flow law:

- The fluid is incompressible (constant density, ρ).
- The flow is laminar (no turbulent motion of the fluid), Reynolds number less than 2,000.
- The flow velocity profile is constant throughout the length, L , of the leak passage.
- The flow velocity at the leak walls is zero. (Wall friction forces are predominant.)

For fluid moving through leaks at low velocities and with a low differential pressure across the leak, the flow may be considered as incompressible. However, with increased velocities, longer leak paths and significant pressure differentials, the fluid density is no longer constant between entrance and the exit of the leak passage. The Hagen-Poiseuille equation must therefore be modified to treat the flow of a compressible fluid with its accompanying density changes. The following relationships can be substituted into Equation 1:

$$V_s = \frac{Q_v}{A} \quad (2)$$

$$A = \frac{\pi D^2}{4} \quad (3)$$

$$Q_v = \frac{w}{\rho_{av}} \quad (4)$$

where ρ_{av} represents the average density of the fluid flowing through the leak as determined by the upstream and downstream pressure P_o and P_b . The laminar flow equation for an incompressible fluid then becomes:

$$w = \frac{\pi r^4}{8\mu L} \rho_{av} (P_o - P_b) \quad (5)$$

Poiseuille's equation (Bird et al. 1960) for laminar liquid leakage flow in terms of flow diameter is:

$$w = \frac{\rho_{av} \pi}{8} \left(\frac{D}{2}\right)^4 \frac{P_o - P_b}{\mu L} \quad (6)$$

Although Equations 5 or 6 define the parameters that govern under laminar flow conditions, it should be recognized that these equations are applicable only to simple leak geometries. In practice, the leak geometries are likely to deviate from the circular hole or the rectangular cross section. The laminar flow rate derived from the equations can therefore only be interpreted as a reasonable prediction of the leakage, which will be modified by the effect of the indeterminate leak geometries.

Laminar flow occurs under conditions determined by the average velocity of flow across a plane transverse to the leakage path, and by the density of the fluid and its viscosity. A dimensionless ratio called the Reynolds number (R_e) may be used to express the conditions for laminar flow as well as other types of flow. The Reynolds number is the ratio of the inertial to the viscous forces in a flowing fluid. It is expressed as:

$$R_e = \frac{\rho_{av} DV_s}{\mu \mathcal{G}_c} \quad (7)$$

The critical value of the Reynolds number defining the region between laminar and turbulent flow has been shown to be dependent upon the entrance conditions, roughness of the walls, and the shape of the flow path. In general, for smooth tubes with well-rounded entrances, the critical value is about 1200.

5.2 TURBULENT TUBE FLOW

Although the majority of significant leaks in a fluid system may well be in the laminar flow regime at a low pressure, it is conceivable that with higher velocities, turbulent flow conditions will be attained in irregular leak passages. The region of transition between viscous flow and turbulent flow may be conservatively estimated to begin with a Reynolds number, R_e , greater than 2,000. Flow instabilities are usually present at the transition region between viscous and turbulent flow that make prediction of the type of prevailing flow uncertain. Transition effects as the fluids enter and leave the irregularly shaped leak passage further complicate the flow character in the region of turbulent flow.

If the leak path in a fluid system is of a nature to permit the development of turbulent flow, the mass flow rate, w , may be derived from the Darcy-Weisbach equation (Streeter 1971), assuming that the leak cross section approaches the geometry of a small-diameter tube. The equation commonly appears as:

$$P_o - P_b = f \frac{L\rho}{D} \frac{V_s^2}{2} \quad (8)$$

where f is the friction factor, which is a function of the leak passage surface roughness and the Reynolds number for the flow conditions through the leak.

Since Equation 8 is for circular leaks of diameter D , another geometric parameter must be introduced for the cases of non-circular cross sections. In fluid systems, leaks of non-circular cross section are more likely to exist than uniformly round leaks. By introducing the geometric parameter R_h , the hydraulic radius, the Darcy-Weisbach equation may be modified for non-circular cross-sections:

$$P_o - P_b = f \frac{L}{4R_h} \frac{\rho V_s^2}{2} \quad (9)$$

The validity of Equations 8 and 9 is limited, however, to turbulent flow in leak passages of constant cross section for the entire length of the leak. Such conditions are not necessarily encountered with leaks in fluid systems. The equations should, therefore, be interpreted as merely indicative of the parameters involved in the turbulent flow regime for leak geometries not greatly departing from the generalized geometric parameters.

A fluid flowing through a leak passage where the length to diameter ratio, L/D , and pressure differential, ΔP , may be appreciable would not only encounter wall friction losses but would also expand and increase its kinetic energy. Although under such conditions the behavior may change from incompressible to compressible flow, an approximation may be made by treating the problem as incompressible and assigning average values for the variables in Equation 8. Thus:

$$P_o - P_b = f \frac{L}{D} \frac{\rho_{av} V_s^2}{2} \quad (10)$$

which yields for volumetric flow rate:

$$Q_v = A \left[\frac{2D}{fL\rho} (P_o - P_b) \right]^{1/2} \quad (11)$$

and for mass flow rate:

$$w = \rho_{av} A \left[\frac{2D}{fL\rho} (P_o - P_b) \right]^{1/2} \quad (12)$$

Because of the indeterminate nature of the leak surface roughness, any attempt to assign a value to the friction factor in Equation 12 can only be interpreted as an approximate estimate. Instead, for flows with Reynolds numbers of 4,000 to 100,000 it may be assumed that the profile of flow through the leak approaches that of a smooth tube, for which the empirical expression

of the Blasius law may be applied in the determination of a conservative friction factor. For circular leaks:

$$f = \frac{0.316}{R_e^{1/4}} = \frac{0.316}{\left(\frac{V_s D \rho}{\mu g_c} \right)^{1/4}} \quad (13)$$

and for non-circular leaks:

$$f = \frac{0.316}{\left(\frac{4 R_h V_s \rho}{\mu g_c} \right)^{1/4}} \quad (14)$$

Substituting Equation 14 into the Darcy-Weisbach Equation (Equation 12) and rearranging terms yields:

$$w = \rho_{av} A \frac{D^{5/7}}{\mu^{1/7}} \left(\frac{g_c}{\rho} \right)^{3/7} \left(\frac{2(P_o - P_b)}{0.316 L} \right)^{4/7} \quad (15)$$

Although this approach tends to generalize the actual conditions of flow through leaks that may be better categorized as "rough" tubes, the results obtained in mass flow from Equation 15 will yield values on the high side, except possibly for unstable flow in the transition region between laminar and turbulent flow.

The relationships defined by Equations 12 and 15 should be recognized as indicative of the influence of pressure, temperature and viscosity on turbulent flow through an idealized leak of circular cross section. It is inconceivable that many leaks existing in fluid systems would have smooth, straight surfaces and a uniform circular cross section for the entire length of the flow path. Instead, it would be more likely that the leak would have an irregular surface and a variable flow area. Notwithstanding, the above equation serves to define, in a general way, the more important parameters affecting flow in the turbulent domain for the purpose of a comparative study of leak behavior.

5.3 FLOW THROUGH AN ORIFICE

The general parameters that define the viscous flow of a non-compressible fluid through a orifice leak passage of circular section are expressed by the following equation from Idel'Chik (1966):

$$Q_v = C_e A \sqrt{\frac{2 g_c (P_o - P_b)}{\rho}} \quad (16)$$

At Reynolds numbers greater than 10,000, the entrance coefficient, C_e , for a circular orifice is given as:

$$C_e = 0.59 + \frac{5.5}{\sqrt{R_e}} \quad (17)$$

for a rectangular orifice:

$$C_e = 0.59 + \frac{8.9}{\sqrt{R_e}} \quad (18)$$

and for a square orifice:

$$C_e = 0.58 + \frac{8.9}{\sqrt{R_e}} \quad (19)$$

For the above equations, the mass flow rate is:

$$w = C_e \rho A \sqrt{\frac{2g_c(P_o - P_b)}{\rho}} \quad (20)$$

5.4 FLOW THROUGH A CRACK

The general parameters that define the viscous flow of a single phase fluid through an intergranular stress corrosion crack, of arbitrary cross section, is expressed in equation form as follows (Abdollahian and Chexal 1983):

$$w = A \sqrt{\frac{2g_c(P_o - P_b)}{v_d \left(1 + \frac{fL}{D_h} + \frac{1}{C_e^2}\right)}} \quad (21)$$

The liquid specific volume v_d , is taken to be at the upstream pressure, P_o . The friction factor, f , is given by:

$$f = \frac{1}{\left(2 \log \frac{D_h}{2K} + 1.74\right)^2} \quad (22)$$

where the surface roughness, K , is taken to be 0.003 cm (0.001 in.) which is typical for intergranular stress corrosion cracks (Abdollahian and Chexal 1983). Equation 21 assumes that the flow is not choked, and accounts for entrance and single phase friction losses (C_e and f terms, respectively). A depiction of an idealized crack is shown in Figure 4. For the current analysis it is assumed that the inlet area has the same cross section as the exit area.

Figure 4. Geometry of an Idealized Crack.

5.5 FLOW THROUGH POROUS MEDIA

The general parameters that define the viscous flow of a single phase fluid through a porous passage, of arbitrary cross section, is expressed in equation form from Bird et al. (1960), as follows:

$$Q_v = A \left[\frac{D_p^2}{150\mu L} (P_o - P_b) \right] \frac{\epsilon^3}{(1-\epsilon)^2} \quad (23)$$

and for mass flow rate:

$$w = \rho_{av} A \left[\frac{D_p^2}{150\mu L} (P_o - P_b) \right] \frac{\epsilon^3}{(1-\epsilon)^2} \quad (24)$$

The area, A, used in the above equation is the cross-sectional area of the leak passage containing the porous media. The void fraction, ϵ , is the ratio of the volume of the voids over the volume of the leak flow passage. The diameter, D_p , is the average particle size of the porous media within the flow passage. The above equations for porous media flow are valid for Reynolds numbers less than 10 where:

$$R_o = \frac{\rho_{av} D_p V_s}{\mu g_c} \left(\frac{1}{1-\epsilon} \right) \quad (25)$$

5.6 STATIC HYDRAULIC HEAD DETERMINATION

To determine the driving force for the leakage calculation the static hydraulic head is required. The following equation from Streeter (1971) is used:

$$P = \gamma h_t \quad (26)$$

The specific weight of the fluid column, γ , can be expressed in terms relative to the specific weight of water:

$$\gamma = S_{gr} \gamma_w \quad (27)$$

This determines the static hydraulic head. For the absolute pressure, the local vent space pressure, P_v , must be added to the static hydraulic head. The driving force for fluid flow through a leak passage, as used in this report, is therefore:

$$P_o - P_b = S_{gr} \gamma_w h_t + P_v \quad (28)$$

where P_b is the local atmospheric pressure, assumed to be 101 kPa (2116.2 lb_f/ft²; 14.696 lb_f/in.²).

5.7 LEAK RATE CALCULATIONS

Leak rates were calculated for the postulated leak mechanisms using the equations in Sections 5.1 through 5.6. Results for the most likely leak mechanism described in Section 4.6 (i.e., stress corrosion cracking of the steel tank liner with leakage out through the construction joint in the concrete encasement) are shown in Tables 4 and 5. Three cases were considered in which the layer of protective sludge is removed from the liner around one-fourth, one-half, and the entire tank circumference. Flows through the cracks in the liner are smaller than the flows through the construction joint so that the liner cracks limit the overall leak rate. Calculated leak rates for the other postulated leak mechanisms and for various operating conditions are provided in Appendix C.

Table 4. Leakage Through Stress Corrosion Cracks in Tank Liner.

Fraction of Wall Exposed	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
1/4	0.258	19	35
1/2	0.516	38	69
1	1.03	77	140

Table 5. Leakage Through Construction Joint from Stress Corrosion Cracks.

Fraction of Wall Exposed	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
1/4	4.36	38	68
1/2	8.72	76	140
1	17.4	150	270

5.8 HISTORICAL LEAK RATES

Leak rate data were statistically analyzed to identify the maximum probable leak rate from a single shell tank (Isaacson and Gasper 1981). Five confirmed leaking tanks had leak rates ranging from 0.3 mL/s (0.005 gal/min) to 1.5 mL/s (0.023 gal/min). Six tanks of questionable integrity were also included in the analysis. These tanks had leak rates ranging from 0.095 mL/s (0.0015 gal/min) to 1.3 mL/s (0.020 gal/min). This analysis concluded that with 95% confidence, 95% of the single shell tanks would leak at a rate of less than 2 mL/s (0.03 gal/min).

Some notable exceptions to the above estimated leak rates have occurred in the past and were not included in the analysis. Tank 241-T-106 lost 435 m³ (115,000 gal) over a period of approximately 50 days (Routson, et al. 1979). This leak started approximately three weeks after 1,500 m³ (400,000 gal) of ion exchange waste was added on top of 280 m³ (74,000 gal) of waste that had been in the tank for some time. This leak amounts to an average leak rate of 100 mL/s (1.7 gal/min). Tank 241-SX-115 lost 190 m³ (51,000 gal) in March 1965 over a period of one week (Westinghouse 1992b). This leak occurred after 522 m³ (138,000 gal) of condensate was added to the tank and corresponds to a leak rate of 320 mL/s (5.1 gal/min). The leak was attributed to stress corrosion cracking of the steel liner; the condensate is thought to have dissolved deposits which were sealing the cracks. Tanks 241-T-106 and 241-SX-110 were omitted from the previous analysis because they were considered to have been subjected to anomalous operational conditions (sudden stresses and prolonged high temperatures, corrosion conditions, and structural and construction inadequacies). These conditions are not expected to be representative of tank 241-C-106. At least ten other SSTs (leakers and assumed leakers) on the Hanford Site have experienced leaks in the range of 40 m³ to 260 m³ (10,000 to 70,000 gal) for various reasons (Westinghouse 1992a). At the SRP, tank 16 leaked waste from the primary tank into the annulus at a rate up to 300 mL/s (4 gal/min). Leaking in this case was also attributed to stress corrosion cracking (see Section 4.1). Leaking occurred during a period of high humidity when the deposits filling the cracks dissolved. However, no leakage was observed when the tank was subsequently cleaned out and decontaminated.

These historical leak rates provide the only current means to estimate the bounds and validity of the leak rate models and their assumptions. The

| major leaks that have occurred in the past are many times the 20-25 m³
| (5000-7000 gal) of liquid that will be in the sluice tank at any given time.
Leak detection capability coupled with administrative controls, and the
ability to pump out the liquid in the sluice tank in less than an hour should
a leak be detected, make a major leak highly unlikely.

NOMENCLATURE FOR LEAK RATE CALCULATIONS

Symbol	Definition	Units
A	Cross-sectional area of a leak path	ft ²
c	Half-length of a rectangular leak path	ft
C _e	Entrance loss coefficient	-
D	Diameter of a leak of circular cross-section	ft
D _h	Hydraulic diameter, 4R _h	ft
D _p	Particle diameter	ft
f	Friction factor	-
g _c	Gravitational acceleration conversion constant (32.174)	lb _m -ft/ lb _f -s ²
h _t	Height of a fluid column	ft
H	Periphery of flow channel, perimeter of non-circular leak	ft
K	Crack average surface roughness	ft
L	Length of leak path (between pressure zones P _o and P _b)	ft
m	Mass	lb _m
R _e	Reynolds number	-
P	Pressure	lb _f /ft ²
P _o	Absolute pressure upstream of leak path	lb _f /ft ²
P _b	Absolute pressure downstream of leak path	lb _f /ft ²
P _{av}	Average of pressure upstream and downstream of leak path	lb _f /ft ²
P _v	Pressure of tank vent space	lb _f /ft ²
ΔP	Pressure differential	lb _f /ft ²
Q _v	Volumetric flow rate	ft ³ /s
r	Radius of a leak of circular cross-section	ft
R _h	Hydraulic radius, A/H	ft
S _{gr}	Specific gravity of a fluid relative to water	-
t	Time	s
T	Temperature	°F
v _o	Specific volume of a fluid at upstream pressure	ft ³ /lb _m

Symbol	Definition	Units
V_s	Average fluid velocity	ft/s
w	Mass flow rate	lb _m /s
γ	Specific weight of a fluid	lb _f /ft ³
γ_w	Specific weight of water	lb _f /ft ³
δ	Width of rectangular passage (crack) transverse to flow	ft
ϵ	Void fraction	-
μ	Viscosity of fluid	lb _f -s/ft ²
ρ	Density of fluid	lb _m /ft ³
ρ_{av}	Average of the fluid density at the pressures upstream and downstream of the leak passage	lb _m /ft ³

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6.0 LEAK TRANSPORT MODELING

Hydraulic retrieval of sludge from a SST can potentially result in leakage of sluicing liquid and dissolved contaminants from the tank liner and concrete encasement into the surrounding soil. The purpose of leak transport modeling is neither to estimate the probability of leak occurrence nor to elaborate methodology for leak detection, but to make some assessment of the consequences if leakage does occur. In this section quantitative consequences of leakage to the soil are given in terms of contaminant concentration in the underlying unconfined aquifer and travel time thereto. These results are used in Section 7.0 to estimate health hazard through the drinking water pathway. Qualitative results are also provided on the extent of soil contamination. These latter results are intended to support future work on subsurface barriers and/or recharge mitigation.

6.1 CONCEPTUAL LEAK TRANSPORT MODEL

The results to be provided in this section are based on the premise that leakage of fluid (assumably water) containing dissolved contaminants occurs during the sluicing operation. Various combinations of leakage through the liner and through the concrete encasement are analyzed in Section 5. The cases considered here are based on leakage which ultimately penetrates the construction joint between the wall and side of the concrete encasement of a SST and propagates into the surrounding soil. The leakage rates of fluid to the soil are representative of the most likely leak mechanism discussed in Section 4.

The sluicing operation is expected to take a year to complete. Leakage is only expected to occur at most during the last 500 hours of sluicing at the end of the year long activity. If a leak occurs, it is assumed that liquid is only introduced to the soil during the time (up to 500 hours) that sluicing is actively performed; otherwise it is assumed that only minimal residual fluid is standing in the tank and that leakage to the soil is negligible. Leakage to the soil will actually be a series of pulses with opportunity for dissipation of fluid and drying of the soil between pulses. This alternating sequence of wetting and drying slows the rate of eventual migration downward to the unconfined aquifer. It also enhances lateral spreading of contamination. For conservatism it is assumed that leakage to the soil persists at a constant rate for 500 hours, after which no more leakage occurs (i.e., it is assumed for modeling purposes that leakage due to the sluicing operation is continuous and constant).

After the 500 hour leakage period, the only driving force affecting the discharged plume is infiltration of precipitation. The net soil moisture recharge from infiltration of precipitation is the amount of equivalent rainfall minus the amount of evapotranspiration. For the present model it is assumed that the rate of soil moisture recharge is constant, and that the value is $q = 1 \text{ cm/yr}$ (0.4 in./yr) prior to construction of the 241-C tank farm. After completion of construction of the 241-C tank farm in 1943, the rate of recharge for that site is assumed to be 10 cm/yr (4 in./yr) since there is no significant vegetation and hence no root zone and no transpiration

for that time period. This is consistent with recharge rates simulated by Smoot et al., 1989, for the 241-T tank farm in the 200 West Area.

During the 49 year period from 1943 to 1992, not only is the recharge rate elevated in the tank farms but each SST is also assumed to act effectively as an umbrella. The volume of water due to recharge at 10 cm/yr over the area of a tank is assumed to runoff uniformly around the perimeter of the tank. Purely by assumption, the incremental radius (Δr) of the ring of enhanced recharge is taken to be one-eighth meter (4.9 in.). This ring of enhanced recharge due to an assumed umbrella effect is shown below in Figure 5.

Figure 5. Umbrella Effect of Single-Shell Tank.

The model domain is taken to include exactly one SST in the 241-C tank farm, particularly SST 241-C-106. The model includes precisely the driving forces (fluid sources) described above, but does not encompass possible interactions with discharges or umbrella effects of neighboring tanks. It is further assumed that the particular tank of interest (241-C-106) has not leaked prior to the start of sluicing. By virtue of the assumption of no prior leakage coupled with the assumed umbrella effect, the soil column directly under the floor of the tank is very dry at the beginning of sluicing and therefore inhibits radially inward lateral migration of contaminated fluid from leakage during sluicing. Moreover, the enhanced recharge around the perimeter of the tank creates a preferential pathway downward to the unconfined aquifer. This preferential pathway is well established by 1992, and hence 1992 is taken as the reference time for start of leakage due to sluicing. The simulated moisture profile (relative saturation) at the time of start of leakage is shown in Figure 6.

Vertically the domain extends from the bottom of the tank floor (concrete encasement) through the underlying soil column to the unconfined aquifer. The unconfined aquifer is also included and is assumed to be uniform and 10 meters (30 ft) in vertical extent. The intervening soil column between the tank

Leak Transport Model for SST 241-C-106

Simulated Relative Saturation Prior to Sluicing (1992)

Recharge = 10 cm/yr from 1943 to 1992

Figure 6. Relative Saturation Contours
Prior to Tank Leak

floor and the unconfined aquifer is an alternating sequence of layers of variably saturated medium sand and gravelly sand, and is represented graphically in Figure 7. This rendering of the soil column is based on previous modeling work performed by Stewart et al., 1987. The stratigraphy shown in Figure 7 is generally representative of the 241-AP tank farm. Hence, although the modeling reported in this section is focused on SST 241-C-106, the results are generally applicable to any 23-m (75-ft) diameter SST, in the vicinity of the 241-AP tank farm, which has not previously leaked.

Values of characteristic hydraulic parameters for the soil layers in the model domain are taken from Smoot et al., 1989. The soil type assignments ("AP-1" and "AP-3") shown in Figure 7 are indicative of textural descriptions of soil samples collected from 241-AP tank farm excavation. The particular sequence of soil type assignments in Figure 7 is representative of the observed stratigraphy, and also produces the minimum simulated travel time to the water table for realistic soil moisture recharge rates (Stewart et al., 1987). Values of the hydraulic parameters for the selected material types are shown below in Table 6. Values of the corresponding moisture content parameters are shown in Table 7. In the unsaturated zone between the tank floor and the water table, relative saturation is assumed to follow the van Genuchten theory and relative hydraulic conductivity is obtained by the Mualem type prediction. These formulations are summarized in the users guide for Version 1.2 of the PORFLO-3* computer code (Runchal et al., 1992), which is the vehicle used for implementing the conceptual model in numerical simulations. The van Genuchten parameters (α and n) in Table 6 characterize a relationship between soil moisture tension and soil moisture content during drying (as opposed to wetting). This relationship is referred to as a "drainage curve" and generally produces relative hydraulic conductivities that are no less than those obtained from a counterpart wetting curve for the same soil (i.e., wet soil drains faster than dry soil wets). Use of drainage curves for hydraulic conductivity in the unsaturated zone tends to reduce simulated travel time to the aquifer. Moreover, hydraulic conductivity is assumed to be isotropic.

Table 6. Hydraulic Parameters for Leak Transport Model.

Soil Type	Saturated Conductivity K_{sat} (m/yr)	Van Genuchten Parameters	
		α (m)	n (unitless)
AP-1	820.5	0.1	2.9271
AP-3	255.6	0.2	3.5189

The conceptual model used for this work is in some part validated by previous work. This is not to mean simply that similar modelling approaches have been used elsewhere. For instance, a stratigraphic cross-section nearly equivalent to that shown in Figure 7 was used for a two-dimensional model of discharges from the 216-A-37-1 crib in the 200 East Area (see Figure D.3-2 in

*PORFLO-3 is a copyright of Analytic & Computational Research, Inc.

Figure 7. Conceptual Leak Transport Model.

Table 7. Soil Moisture Content for Leak Transport Model.

Soil Type	Saturation θ_{sat}	Residual Saturation θ_r
AP-1	0.42	0.04
AP-3	0.43	0.06

Westinghouse 1990b). The hydraulic properties used in that model were taken from the same source as the hydraulic properties used for the work reported by Smoot et. al., 1989. The A-37-1 crib model produced tritium travel times to groundwater that agreed well with observed tritium travel times (Appendix D of Westinghouse 1990b). Those results give credence to the stratigraphic cross-section used for modelling flow and transport in the vadose zone in the vicinity of the southeastern 200 East Area, and also to the source of hydraulic properties. The source of hydraulic properties is analysis of soil samples taken from the vicinity of the southeastern 200 East Area (appendices in Smoot et. al., 1989). Although the stratigraphy used here is similar to that used for the A-37-1 crib model, the particular values of hydraulic properties used here are similar but not identical to those used for the A-37-1 crib model. The particular sequence of hydraulic property assignments used here is that which produces minimum travel time to the aquifer as reported by Stewart et. al., 1987. Secondly, the rate of soil moisture recharge due to infiltration of precipitation (assumed to be 10 cm/yr for tank farms) is consistent with that calibrated by Smoot et. al., 1989 to laboratory estimates of residual moisture content. Finally, Smoot et. al., 1989, noted the umbrella effect in the results of their PORFLO-3 model of flow and transport around the SST 241-T-106. That model extended vertically from the unconfined aquifer up to the ground surface with recharge imposed at ground surface. Tank 241-T-106 was modelled as an impervious block, and simulation results showed shedding of water around the tank. Unfortunately Smoot et. al., 1989, quantified neither the rate of enhanced recharge nor the incremental radius of the ring of enhanced recharge.

6.2 NUMERICAL TRANSPORT MODEL

Although the natural geometry of the conceptual model is cylindrical and three-dimensional, a two-dimensional "equivalent" rectangular formulation is used for numerical simulations. This is not only in accordance with the original work plan, but is prudent in terms of both turnaround of computer simulations on the Hanford CRAY (2-dimensional versus 3-dimensional) and conservatism of approximated concentrations of transported solute in the aquifer (rectangular versus cylindrical). The 2-dimensional rectangular domain is the xz-plane shown in Figure 7, and is assigned a unit thickness ($\Delta y = 1$ m) through which leakage from the construction joint and influx of natural recharge (including the umbrella effect) enters the domain. The xz-plane is taken to be coincident with a diameter of the tank, and the positive x-direction is assumed to be the general direction of flow in the aquifer. The hydraulic gradient in the aquifer is assigned a value of 10^{-4} m/m (approximately 6 inches per mile), with flow being from left to right in Figure 7. The general direction of flow varies in the 200 Areas with the

seasons of the year. The assumed value of the hydraulic gradient (10^{-4}) is representative of typical large scale gradients at the Hanford Site.

Intersection of the 1-meter (3-ft) thick rectangular domain with the circular profile of the tank is shown from a top view in Figure 8a. The thickness of the domain ($\Delta y = 1$ m) is small compared to the outer circumference of the tank ($L = 75$ m). Flow of fluid due to tank leakage (Q) enters the simulation domain through an approximately rectangular interface shown in Figure 8b. It is assumed that all of the flux across the shaded interface in Figure 8b is transformed into a vertical flux (w) over a lateral distance Δx . This Δx is taken for convenience to be the same as Δr in the ring of enhanced recharge from Figure 5; i.e., $\Delta x = \Delta r = 0.125$ m (4.9 in.). The vertical flux, w , is used as a flux boundary value in the PORFLO-3 based model.

The value of Q is the volumetric leak rate in units of gallons per minute. The corresponding leak area A is the area available for flow in units of square inches. The quantity Q is a leak rate, and the corresponding equivalent flux is $Q' = Q/A$. The area of the shaded interface in Figure 8b through which flux enters the simulation domain is a small proportion of the total leak area, and is expressed as $A' = A\Delta y/L$, where again L is the circumference of the outside surface of the concrete tank encasement. The volumetric rate of fluid flow into the small computational cell shown in Figure 8b is then $Q'A'$ and the vertical flux is then:

$$w = \frac{Q'A'}{\Delta x \Delta y} = \frac{Q}{\pi D \Delta x}. \quad (29)$$

Four different values of Q were used for four simulation runs. The four cases are based on previously assumed leak geometries and leak durations that yielded leak volumes of 150 m^3 (40,000 gal) or less. These cases are summarized in Table 8.

Table 8. Identification of Simulation Runs.

Run Number	Leak Rate Q (mL/s)	Effective Flux w (m/yr)
I	21.3	72.1
II	5.67	19.2
III	2.66	72.1
IV	0.707	19.2

Cases I and II are based on leakage around the entire tank perimeter. Under the equivalent rectangular formulation, flux w due to leakage is applied where the perimeter intercepts the negative x -axis and the positive x -axis for these first two cases (Figure 8a). For the latter two cases (III and IV),

Figure 8. Equivalent Rectangular Formulation.

leakage is assumed to be limited to 45 degrees of an arc on the perimeter. In these two cases that leak is assumed to intercept the positive x-axis of the numerical model domain (Figure 8a). In all four cases the flux, w , is less than the saturated conductivity of the top layer of soil (see Figure 7 and Table 6). Flux in excess of saturated conductivity could flood the soil column. Assuming a unit hydraulic gradient, the maximum flux which will not cause flooding under the present modeling assumptions is equivalent to the saturated conductivity:

$$255.6 \text{ m/yr} = K \frac{\delta P}{\delta Z} = |w_{\max}| = \frac{Q_{\max}}{\pi D \Delta x} \quad (30)$$

and hence the maximum leak rate is:

$$Q_{\max} = 255.6 \pi D \Delta x \text{ m}^3/\text{yr},$$

which is about 76 mL/s (1.2 gal/min). Note that this is a zeroth order approximation and does not account for possible anisotropy of the hydraulic conductivity.

The boundary conditions for the numerical model are taken from Figure 7. Simulation case I started with steady state flow based on assumed Pre-Hanford recharge of 1 cm/yr. From that a transient flow field was simulated for the period 1943 to 1992 with the assumed umbrella effect of the tank and elevated recharge rate of 10 cm/yr. The net influx for the ring of enhanced recharge (Figure 5) is 4.83 m/yr from natural recharge. The tank floor is assumed to provide zero flux of fluid (no leaks) for the duration of the simulation. During the 500 hour period of leakage due to sluicing the flux w from Figure 8a is set according to Table 8 and then reset to the value of 4.83 m/yr after sluicing stops. Cases II, III, and IV are restarted from Case I at the time of start of leakage with the flux w set according to Table 8. In each case advective, diffusive and dispersive transport of conservative species (no decay, no retardation) of dissolved contaminant is simulated in terms of relative concentration. To accomplish this the leakage nodes (Figure 8b) are assigned a unit concentration for the duration of the leakage period. All nodes on the top boundary are otherwise set to zero concentration. The coefficient of molecular diffusion of contaminant is set to 0.03 m²/yr (0.3 ft²/yr), longitudinal dispersivity is set to 1 m (3 ft), and transverse dispersivity is set to 0.1 m (4 in.), everywhere in the spatial domain. These values of transport parameters are consistent with values used previously (e.g. Smoot et al. 1989 and Westinghouse 1990b). During the leakage period (500 hours), contaminant mass enters the domain predominantly by advection due to tank leakage. In the domain contaminant mass is transported by advection, diffusion, and dispersion. Advection is typically the dominant mode of contaminant transport. The rate of advection (groundwater speed) determines the travel time for a path between two points. Advection is simply translation of a front in the direction of flow, but does not affect the value of the concentration at the front. Dispersion is mixing of the fluid along the path of travel due to transverse velocity components and nonuniformity of the longitudinal velocity component. Diffusion refers to mixing due to random molecular motion and occurs independently of the velocity field. The processes of diffusional and dispersional mixing along a path of travel

gradually diminish the contaminant concentration over the front. Contaminant mass concentrations in the domain are fractional parts of the boundary value. Simulated values of concentration for a particular species are obtained by multiplying actual concentration of the species in the leaked fluid by the simulated relative concentration. The relative concentration at a point in the domain can be treated as a dilution factor for the path of travel to that point.

6.3 RESULTS OF TRANSPORT MODEL (Cases I - IV)

The results of simulation runs I through IV are portrayed graphically, and are also summarized for use in follow-on dose calculations in terms of peak concentration in the aquifer and travel time thereto. The time history of contaminant mass concentration at the source node(s) during the leakage period is essentially a rectangular pulse. As time passes, the pulse is drawn downward due to advection and gradually spreads out into a plume due to diffusion and dispersion. As the plume enters the aquifer, lateral advection due to a gradient is assumed to dominate. Figures 9 through 12 show the evolution of the contaminant plume for Case I.

As the plume migrates through the soil column and the aquifer, the original rectangular pulse of contaminant is smoothed by the effects of diffusion and dispersion. At a point in the aquifer 25 meters (82 ft) radially "downstream" from the center of the tank, the contaminant pulse time history takes on the characteristic shape resembling a Gaussian density. The four simulation cases I through IV were carried out until the pulse passed the above described reference point in the aquifer. Time history of relative contaminant mass concentration in the aquifer is shown for cases I, II, III, and IV in Figures 13, 14, 15, and 16, respectively. Peak concentration at the reference point in the aquifer and contaminant travel time to that point for each case are summarized in Table 9 below. The results of these four cases (I-IV) are not appreciably dissimilar from case to case. This is due to the similarities of the simulation set up for the four runs; i.e., only the source term changes from case to case, and only for the duration of the leakage period (500 hours). Beyond that each simulation is dominated by the driving force from the ring of enhanced recharge due to the assumed umbrella effect. For instance, the total volume of leakage for case I is about 150 m^3 (40,000 gal), while recharge at 10 cm/yr (4 in/yr) over a disk of radius 25 m (82 ft) for a period of one year provides almost 200 m^3 (50,000 gal) of water, every year. This is what dominates the results of cases I through IV. Essentially under the numerical model setup used here, simulated travel time of contaminant from the tank floor at the perimeter to the reference point in the unconfined aquifer is about sixty years, and the peak relative contaminant mass concentration at the same point is less than two percent of the initial concentration (i.e., two percent of the contaminant mass concentration in fluid at the point of leakage to the soil column).

6.4 DISCUSSION OF TRANSPORT MODEL RESULTS (Cases V - VII)

The contaminant transport results reported in Section 6.3 are applicable to each species of dissolved contaminant in the liquid which can leak from the tank during sluicing recovery of sludge. Typical contaminants of interest

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Relative Concentration: End of Leak (2000 Hours)

Figure 9. Case I: Concentration Plume at End of Sluicing

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement
Relative Concentration: 5 Years After Start of Leakage

Figure 10. Case I: Concentration Plume 5 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement
Relative Concentration: 50 Years After Start of Leakage

Figure 11. Case I: Concentration Plume 50 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement
Relative Concentration: 100 Years After Start of Sluicing

Figure 12. Case I: Concentration Plume 100 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement
Time History of Relative Concentration in the Unconfined Aquifer
Twenty-Five Meters From Center of the Tank

Figure 13. Case I: Time History of Concentration

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement
Time History of Relative Concentration in the Unconfined Aquifer
Twenty-Five Meters From Center of the Tank

Figure 14. Case II: Time History of Concentration

Sluicing Recovery of Sludge From SST 241-C-106

45 Degree Leak From Joint In Concrete Encasement
Time History of Relative Concentration in the Unconfined Aquifer
Twenty-Five Meters From Center of the Tank

Figure 15. Case III: Time History of Concentration

Sluicing Recovery of Sludge From SST 241-C-106

45 Degree Leak From Joint In Concrete Encasement
Time History of Relative Concentration in the Unconfined Aquifer
Twenty-Five Meters From Center of the Tank

Figure 16. Case IV: Time History of Concentration

Table 9. Summary of Numerical Simulation Results.

Run Number	Travel Time (Years)	Maximum Relative Concentration	Description
Section 6.3			
I	57	1.66×10^{-2}	Base Case. Leakage along entire tank perimeter.
II	58	4.22×10^{-3}	Base Case. Leakage along entire tank perimeter.
III	57	1.63×10^{-2}	Base Case. Leakage along 45° of tank perimeter.
IV	58	4.22×10^{-3}	Base Case. Leakage along 45° of tank perimeter.
Section 6.4			
V	2700	10^{-30}	Case I with Sr-90 Transport.
VI	106	6.6×10^{-3}	Case III with anisotropy.
VII	45000	2.4×10^{-3}	Case I with zero recharge.
Section 6.5			
VIII	>500	-	Case I with low recharge.
IX	>500	-	Case I with recharge barrier.
X	>150	-	Case I with concrete subsurface barrier.

include soluble radionuclides such as Tc-99 and U-235 as well as organic contaminants and nitrates. The simulation results are directly applicable to conservative species such as these (i.e., species which do not decay appreciably and which are not retarded due to adsorption). However, the results are also applicable to species which do decay substantially in the time period of interest and/or which are adsorbed by the host soil. Retardation due to adsorption directly amplifies the contaminant travel time. Increased travel time enhances the effects of decay. Retention of

contaminants in the soil due to adsorption along the path of travel decreases the contaminant concentration in the transporting fluid. These effects combine to not only prolong the arrival of the peak concentration in the aquifer, but also to diminish the value of the peak concentration.

An archetypical example of a sorbing, decaying species which is sufficiently abundant in the tank inventory to warrant consideration is Sr-90. Assuming an effective retardation coefficient with value $R_d = 45$ for Sr-90, the retarded contaminant travel time from the tank floor to the reference point in the aquifer based on results of run I is $T = 60R_d = 2700$ years. With a half-life of 29 years for Sr-90, the decay rate is $\lambda = 0.024$ and hence the maximum relative concentration of Sr-90 in the aquifer would be less than $C = 0.02e^{-\lambda T} = 10^{-30}$. Carrying out a simulation for a long enough time period to estimate the same quantities directly from the numerical model could be impractical due to the prolonged computer run time. Purely as an example to support this, simulation run number V was set up from run number I but with effective retardation $R_d = 45$ and a twenty-nine year half-life to approximate development of a Sr-90 plume in terms of relative concentration. Results from run V showing development of the Sr-90 plume in the vadose zone over a one hundred year period are contained in Figures 17 through 20. Note from Figure 20 that a relative concentration of 10^{-6} kg/m³ (equivalent to one part per billion) has only penetrated to a depth of about 45 m above the water table and that the center of mass of the plume still resides within about 10 meters of the source after 100 years. In fact, the retardation factor for Sr-90 might be as much as 100 in which case the results are even more exaggerated, and more impractical to approximate numerically. The results for Case V shown in Table 9 are in fact estimated as above since the simulation for Case V was indeed only carried out for a period of 250 years.

The basic numerical model used for simulations I through V is a conservative model in that the travel time estimates are thought to be minimal for representative conditions and relative concentrations in the aquifer are likely maximal. Some of the important conservatisms built into the model are elaborated previously in the text, and are summarized below.

- Soil type assignments to the stratigraphy of the soil column yield minimum travel times for a range of recharge rates.
- The recharge rate in the 241-C tank farm is elevated due to lack of vegetation.
- Leakage during the last 500 hours of the sluicing operation is assumed to be continuous and constant.
- Leakage is assumed to occur at a maximal rate during the entire leakage period, and is introduced to the soil column in an area coincident with a ring of enhanced recharge due to assumed umbrella effect.
- Effective hydraulic conductivity is based on a drainage curve and is assumed to be isotropic.
- Relative concentrations are simulated for conservative species with no radioactive decay or retardation.

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement
Relative Sr-90 Concentration: 3 Years After Beginning of Leakage

Figure 17. Case V: Sr-90 Plume 3 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement
Relative Sr-90 Concentration: 10 Years After Beginning of Leakage

Figure 18. Case V: Sr-90 Plume 10 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Relative Sr-90 Concentration: 30 Years After Beginning of Leakage

Figure 19. Case V: Sr-90 Plume 30 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Relative Sr-90 Concentration: 100 Years After Beginning of Leakage

Figure 20. Case V: Sr-90 Plume 100 Years After Start of Leakage

The presence of adjacent tanks or other structures was not addressed in the model. Travel times to the aquifer and concentrations of contaminants in the groundwater could potentially be reduced if runoff from these structures contributes to the recharge.

Use of this model provides bounding results that are appropriate for downstream dose calculations; however, the results are based on minimal lateral spreading of contaminant. Aside from dose to man via the drinking water pathway, it might also be useful to be able to bound the extent of soil contamination due to lateral spreading under less conservative conditions. To show how this can be addressed, simulation run VI was set up from run III with the following changes:

- Hydraulic conductivity is anisotropic in a ratio of 10 to 1; i.e., vertical conductivity is only one-tenth the lateral conductivity. Macroscopic anisotropy has generally not been shown to exceed a value of 10 to 1 at the Hanford site.
- The source term due to the 500 hour leakage period is uniformly distributed over a full year.

Results of run VI are displayed graphically in Figures 21 through 24. From Figure 24 it appears that after 100 years the center of mass of the plume is still in the vadose zone, and from all of the Figures 21 through 24 enhanced lateral spreading of contaminant is easily seen. From those same plots (Figures 21 through 24) it might be possible to make a zeroth order approximation to the maximum volume of soil which is contaminated by a threshold concentration of some species. Superposition of dry well locations on the plots might benefit analysis of leak detection schemes. Peak relative concentration at the reference point in the aquifer is less than 0.007 for case VI, and the corresponding travel time is 106 years. As shown from case V, travel time to the aquifer can be lengthened dramatically by retardation due to adsorption; coupled with radioactive decay, the prolonged travel times can mitigate problematic concentrations in the aquifer. But this is only applicable to species which decay and adsorb significantly. Alternately for conservative species results from run VI indicate that, while travel time to the aquifer is at least sixty years based on the most conservative model, it is also probably not much more than a hundred years. As has already been pointed out, the non-retarded travel time to the aquifer is effectively controlled by natural recharge due to precipitation. The exact value of the mean travel time should be inversely proportional to the true mean value of Δr , the incremental radius of the ring of enhanced recharge. Given the "correct" value of Δr , however, the travel time is still controlled by the natural soil moisture recharge. To illustrate this conclusion, run VII was setup to duplicate run I again, but this time with no recharge after the beginning of sluicing. The assumption underlying case VII is that the SST site is protected by a recharge barrier (a large "tent" with runoff collection capability) for as long a period of time as needed. The simulation results from case VII indicate that in the absence of recharge after leakage has occurred during sluicing, travel time to the reference point in the aquifer is almost 45,000 years and peak relative concentration in the aquifer is less than 0.0024. These results are of course predicated on assumptions that drainage is not impacted by any disturbance, geologic or otherwise, and the recharge barrier is intact and functional for the duration. A sequence of

Sluicing Recovery of Sludge From SST 241-C-106

45 Degree Leak From Joint In Concrete Encasement

One Year Leak with Anisotropy

Relative Concentration: 3 Years After Beginning of Leakage

Figure 21. Case VI: Concentration Plume 3 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

45 Degree Leak From Joint In Concrete Encasement

One Year Leak with Anisotropy

Relative Concentration: 10 Years After Beginning of Leakage

Figure 22. Case VI: Concentration Plume 10 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

45 Degree Leak From Joint In Concrete Encasement

One Year Leak with Anisotropy

Relative Concentration: 30 Years After Beginning of Leakage

Figure 23. Case VI: Concentration Plume 30 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

45 Degree Leak From Joint In Concrete Encasement

One Year Leak with Anisotropy

Relative Concentration: 100 Years After Beginning of Leakage

Figure 24. Case VI: Concentration Plume 100 Years After Start of Leakage

plots in Figures 25 through 29 shows the evolution of the plume under assumptions of case VII. Although the time of travel of the center of mass of the plume to the reference point in the aquifer is dramatically prolonged for this case, Figures 25 through 29 show that it is accompanied by pronounced lateral spreading of the contaminant plume. The benefit is that a recharge barrier controls arrival of highest concentration in the aquifer. As can be anticipated from the results in Figures 25 through 29, a smaller peak reaches the aquifer early, in spite of the recharge barrier, as an artifact of the ring of enhanced recharge. This can be seen in the time history plot for case VII shown in Figure 30. Therefore, while a recharge barrier might be easier and cheaper to install than subsurface barriers, it must be done early to mitigate the effect of the ring of enhanced recharge before starting sluicing.

6.5 RECHARGE MITIGATION (Cases VIII - X)

All of the preceding leakage simulation cases (I-VII) are initiated from reference time 1992 under the assumptions that soil moisture recharge was 1 cm/yr prior to construction of the 241-C Tank Farm and 10 cm/yr there after due to nearly complete elimination of vegetation and root zone in the vicinity of the tanks. These assumptions regarding recharge are consistent with previous modelling work (e.g. Smoot et al, 1989). Alternately it has been argued informally that after construction of tank farms and elimination of vegetation and the root zone, the loss of transpiration enhances recharge in wetter months but the gain in recharge is lost due to enhanced evaporation during the warm dry months. An attempt to prove or disprove either theory is beyond the present scope of this work. To do so would first require soil moisture data at varying depths over a period of time so that simulations using varying recharge rates can be performed in an effort to find the best matching simulated soil moisture distribution and thereby infer the recharge rate. However, one of the most important results of the previous simulations (Cases I-VII) is that travel time to the aquifer is dependent primarily on rate of recharge, and secondarily on rate of leakage (especially for large leaks).

To further illustrate the effect of the recharge rate, simulation case VIII was setup from Case I, but with a reduced recharge rate of 0.05 cm/yr from 1943 through the duration of the simulation and with the 500 hour leakage period being in year 1993 rather than 1992. Relative concentration contours at 100 and 500 years after beginning of leakage are shown in Figures 31 and 32, respectively, for case VIII. Figure 31 shows that with the reduced recharge rate the center of mass of the contaminant plume has moved about the same distance as in Case VII in the same period of time (Figure 28). Figure 32 shows that the center of mass of the plume is still about 50 meters above the aquifer after 500 years of evolution with the reduced recharge rate of 0.05 cm/yr for case VIII. A striking difference between the contours for case VII (Figures 25 through 29) and the contours for case VIII (Figures 31 and 32) is that case VII shows substantially greater lateral migration, assumably due to higher soil moisture content although a barrier was emplaced at the start of leakage. Case VIII suggests that a lower recharge rate not only prolongs travel time to the aquifer, but also reduces lateral spreading. This would be desirable if the soil contaminated by any leakage must be recovered.

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Zero Recharge After Start of Leakage

Relative Concentration: End of Leak

Figure 25. Case VII: Concentration Plume at End of Sluicing

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Zero Recharge After Start of Leakage

Relative Concentration: 3 Years After Beginning of Leakage

Figure 26. Case VII: Concentration Plume 3 Years After Start of Leakage .

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Zero Recharge After Start of Leakage

Relative Concentration: 30 Years After Beginning of Leakage

Figure 27. Case VII: Concentration Plume 30 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Zero Recharge After Start of Leakage

Relative Concentration: 100 Years After Beginning of Leakage

Figure 28. Case VII: Concentration Plume 100 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Zero Recharge After Start of Leakage

Relative Concentration: 1000 Years After Beginning of Leakage

Figure 29. Case VII: Concentration Plume 1000 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Zero Recharge After Start of Leakage

Time History of Relative Concentration in the Unconfined Aquifer
Twenty-Five Meters From Center of the Tank

Figure 30. Case VII: Time History of Concentration

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement
Reduced Recharge After Construction of Tank Farm
Relative Concentration at 100 Years After Start of Leakage

Figure 31. Case VIII: Concentration Plume 100 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement
Reduced Recharge After Construction of Tank Farm
Relative Concentration at 500 Years After Start of Leakage

Figure 32. Case VIII: Concentration Plume 500 Years After Start of Leakage

It is clear that a low soil moisture recharge rate is beneficial in the event of a moderate size leak during sluicing retrieval of sludge, both in the respect of travel time and lateral spreading. If however the recharge rate is as much as 10 cm/yr after construction of the tank farms, the benefits of the lower recharge rate can be obtained by emplacing a recharge barrier prior to start of sluicing. Simulation case IX was setup from case I again, but with recharge of 10 cm/yr from 1943 to 1993 (instead of 1992) and then emplacement for an assumed surface recharge barrier of 99.5% efficiency for five years (1993-1998) prior to the 500 hour leakage period. A timeline for case IX showing the sequence of events and recharge history is shown below in Figure 33. The five year period during which the assumed barrier is in place prior to leakage from the sluicing operation allows the soil to drain and establishes soil moisture conditions not unlike the conditions at the start of leakage in case VIII. This is evidenced by Figures 34 and 35 which show evolution of a relative concentration plume for case IX at 100 and 500 years respectively after beginning of leakage. Comparing Figures 34 and 35 to Figures 31 and 32 respectively suggests that movement of the center of mass of the plume and lateral spreading for the plume is similar in both cases. The conclusion to be drawn from case IX is that a surface barrier can effectively mitigate effects of a high recharge rate on leakage during the sluicing operation. The premise is however that the barrier is emplaced five years before the leakage occurs. For purposes of the simulation, it is also assumed that the barrier remains intact and effective for an indefinite period of time.

Figure 33. Sequence of Events and Recharge History for Case IX.

The final flow and transport case considered here is again derived from case I, but incorporates a subsurface barrier. The barrier is conceptually a one foot thick concrete containment structure, the walls of which are assumed to be 10 feet outside the concrete encasement walls of the tank, and

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement
Recharge Barrier Employed 5 Years Before Leak
Relative Concentration at 100 Years After Start of Leakage

Figure 34. Case IX: Concentration Plume 100 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Recharge Barrier Employed 5 Years Before Leak

Relative Concentration at 500 Years After Start of Leakage

Figure 35. Case IX: Concentration Plume 500 Years After Start of Leakage

correspondingly, the bottom is 10 ft beneath the tank floor. The difficulty of emplacing such a structure is not addressed here, but for purposes of modeling, the structure is assumed to be a continuous and uniform concrete container. Characteristic van Genuchten parameters and saturated hydraulic conductivity used in the model are taken from Table 2-1 of the report by Walton and Seitz, 1991. The idealized structure is effective for containing leakage and recharge as is shown by the sequence of plots in Figures 36 through 43. Figures 37 and 38 suggest that somewhere between 10 and 30 years after start of leakage the soil in the enclosure is fully saturated below the elevation of the tank floor. After 150 years of simulation time following the 500 hour leakage period, the enclosure is flooded to the point that pressure buildup in the enclosure and the steep pressure gradient across the concrete to the relatively dry soil outside caused numerical difficulty and the simulation was terminated at that point. Use of a surface barrier is needed to prevent flooding of the subsurface barrier due to the recharge. The important conclusion from case X is that the idealized concrete enclosure is effective in mitigating advection of contaminants.

6.6 COMPARISON OF RESULTS WITH LEAK FROM 241-T-106

On June 8, 1973, the 241-T-106 single shell tank was confirmed as having leaked 435 m³ (115,000 gal) of liquid. (This leak is approximately 3 times greater than that postulated for 241-C-106). Table 10 summarizes the chemical and radionuclide composition of the supernate in tank 241-T-106.

On June 12, 1973, a study was initiated to delineate the contaminated zone and to confirm the predicted extent of movement (ARHCO 1973). Since the objective of this study was to confirm as quickly as possible the concentration and the depth to which the waste liquid had percolated, only the most rapid method of analysis was used. Consequently, a gamma energy analysis spectrometer was employed, a method of analysis which measures quantities of radionuclides which decay by gamma emission. Of the radionuclides detected, Cs-137 was distributed in the smallest volume of sediments. In all instances where Cs-137 was found to be present, there was characteristic bremsstrahlung radiation indicating the presence of Sr-90.** In this liquid waste, which contains a high concentration of carbonate ion and has a high pH, strontium carbonate can be expected to precipitate and thus the retardation of Sr-90 may actually be as high or higher than that of Cs-137. This study concluded that about 70 percent of the activity which leaked into the sediments remained within a volume of about 314 m³ [an ellipsoid with radii 5 m, 6 m and 2.5 m (16 ft, 20 ft, and 8 ft)]. The maximum leak volume (as defined by the 1 µCi/liter Ru-106 concentration isopleth) was estimated to enclose a soil volume of 25,000 m³ (an ellipsoid with radii 21 m, 28 m and 10 m). The maximum vertical penetration was 27 m below ground surface or 35 m above the water table.

**Strontium-90 does not emit a gamma ray directly in its decay. However, Sr-90 has a very high energy beta particle which interacts with surrounding atoms to produce a continuum of X rays (known as bremsstrahlung radiation). These secondary rays can be detected by this analytical system.

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint in Concrete Encasement

Concrete Subsurface Barrier

Relative Saturation at 5 Years After Start of Leakage

Figure 36. Case X: Relative Saturation at 5 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint in Concrete Encasement

Concrete Subsurface Barrier

Relative Saturation at 10 Years After Start of Leakage

Figure 37. Case X: Relative Saturation at 10 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint in Concrete Encasement

Concrete Subsurface Barrier

Relative Saturation at 30 Years After Start of Leakage

Figure 38. Case X: Relative Saturation at 30 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Concrete Subsurface Barrier

Relative Concentration at 10 Years After Start of Leakage

Figure 39. Case X: Concentration Plume 10 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Concrete Subsurface Barrier

Relative Concentration at 30 Years After Start of Leakage

Figure 40. Case X: Concentration Plume 30 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Concrete Subsurface Barrier

Relative Concentration at 50 Years After Start of Leakage

Figure 41. Case X: Concentration Plume 50 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Concrete Subsurface Barrier

Relative Concentration at 100 Years After Start of Leakage

Figure 42. Case X: Concentration Plume 100 Years After Start of Leakage

Sluicing Recovery of Sludge From SST 241-C-106

360 Degree Leak From Joint In Concrete Encasement

Concrete Subsurface Barrier

Relative Concentration at 150 Years After Start of Leakage

Figure 43. Case X: Concentration Plume 150 Years After Start of Leakage

Table 10. Chemical and Radionuclide Composition of the Tank 241-T-106 Supernate.

Component	Value	Units
Na	4.0	<u>M</u>
NH ₄ ⁺	0.065	<u>M</u>
K	0.016	<u>M</u>
Ca	0.0022	<u>M</u>
NO ₂ ⁻	1.18	<u>M</u>
CO ₃ ⁻	0.79	<u>M</u>
NO ₃ ⁻	0.77	<u>M</u>
SO ₄ ⁻²	0.25	<u>M</u>
OH ⁻	0.24	<u>M</u>
Al	0.064	<u>M</u>
pH	11.9	None
Ce-144/Pr-144	1.18 x 10 ⁴	μCi/L
Cs-137	8.85 x 10 ⁴	μCi/L
Eu-155	1.69 x 10 ³	μCi/L
Cs-134	1.32 x 10 ³	μCi/L
Sb-125	1.12 x 10 ³	μCi/L
Sr-89/Sr-90	2.98 x 10 ⁴	μCi/L
Ru-106/Rh-106	6.12 x 10 ⁵	μCi/L
Pu-239	9	μCi/L
Pu-240	2	μCi/L
Am-241	2	μCi/L

Reference: Routson, et al. 1979.

An investigation conducted in 1978 (Brown, et al. 1979; Routson, et al. 1979) indicated that the contamination had moved very little from the 1973 position. The maximum vertical penetration was located 33 m below ground surface. This investigation also concluded that most of the movement of Cs-137 occurred between 1973 and 1974.

The Ru-106 1- $\mu\text{Ci/L}$ isopleth appeared to have migrated about 5 to 7 m downward at two drywells but showed little movement elsewhere. Because of its relatively short (1-year) half-life, Ru-106 would decay from the estimated 2.7×10^{11} μCi leaked from the tank in 1973 to about 8.4×10^9 μCi in 1978. However, the decay process alone was not considered to be sufficient to explain the stationary nature of the 1 $\mu\text{Ci/L}$ isopleth of Ru-106 observed in 1978 (Smoot, et al. 1989).

The impacts of the 241-T-106 leak compare well with the calculated impacts of a leak extending along the joint in the concrete encasement in 241-C-106 through an arc of 45 degrees. As shown in Figure 17, three years after developing a leak in 241-C-106, the contamination plume would have moved to 20 m (66 ft) below the tank floor or about 32 m (100 ft) below ground surface.

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NOMENCLATURE FOR FLOW AND TRANSPORT CALCULATIONS

Symbol	Definition	Units
A	Cross-sectional area of a leak path	m ²
A'	Area of leak path intersected by equivalent rectangular model ($A\Delta y/L$)	m ²
C	Relative contaminant concentration (i.e., percentage of contaminant concentration at the leak)	-
D	Outside diameter of SST concrete encasement walls	m
K	Effective hydraulic conductivity of soil	m/yr
K _{sat}	Saturated hydraulic conductivity	m/yr
K _{rel}	Relative hydraulic conductivity (K/K_{sat})	-
L	Outside perimeter of SST concrete encasement walls (πD)	m
n	van Genuchten pore size parameter	-
p	Total pressure head	m
q	Rate of soil moisture recharge	m/yr
Q	Leak rate from tank	gal/min, m ³ /yr
Q'	Effective lateral flux from tank leak (Q/A)	m/yr
R	Radius to outside of SST concrete encasement wall ($0.5D$)	m
Δr	Incremental radius of ring of enhanced recharge	m
R _d	Retardation factor	-
t	Time	yr
T	Travel time	yr
w	Equivalent vertical flux due to tank leak ($Q/\pi D\Delta x$)	m/yr
x,y,z	Global space coordinates	-
Δx	Flux transition distance	m
Δy	Thickness of equivalent rectangular domain	m
α	van Genuchten bubbling pressure parameter	m
λ	Radioactive decay rate	1/yr
θ	Volumetric moisture content of soil	-
θ_{sat}	Saturated moisture content (total porosity)	-

Symbol	Definition	Units
θ_r	Residual moisture content	-
θ^*	Relative saturation [approximately $(\theta - \theta_r)/(\theta_{sat} - \theta_r)$]	-

7.0 ENVIRONMENTAL RISK AND IMPACT EVALUATION

The concentration in the groundwater is calculated from the initial concentration (C_0) which leaks from the tank, and the final concentration ratio (C/C_0) computed in the transport model. Retardation due to adsorption onto the soil particles is included. Groundwater concentrations are corrected for radioactive decay.

The results from the PORFLO model were used in the calculations below. Two cases were examined which correspond to Runs I and II in Table 8. The first assumes a leak volume of 150 m³ (40,000 gal) produces a peak relative concentration of 0.0166. The second assumes a leak of 41 m³ (10,800 gal) produces a peak relative concentration of 0.00422. In both cases, the travel time to the groundwater is 57 years. The cases considered here are based on leakage which ultimately penetrates the construction joint between the wall and side of the concrete encasement of a SST and propagates into the surrounding soil. The leakage rates of fluid to the soil are representative of the most likely leak mechanism discussed in Section 4.

7.1 CONCENTRATIONS IN LEAKED WASTE

The initial concentration of chemical species and radionuclides in the leaked waste is provided in Table 3. The U-238 concentration is calculated from the uranium concentration using the specific activity for U-238, 3.36E-04 $\mu\text{Ci}/\text{mg}$. The U-234 concentration is calculated from the U-238 concentration assuming they are in equilibrium, so that the activity concentrations are equal.

7.2 RETARDATION AND DECAY FOR RADIONUCLIDES

Groundwater concentrations were computed as the product of the dilution factor (C/C_0) and corrected for radioactive decay. The decay time is the travel time for water times the retardation factor shown in the equation below.

$$R_d = 1 + K_d \rho_s / \theta_s \quad (31)$$

where,

- R_d = retardation factor, i.e., the ratio of the transport speed of the water to the transport speed of the contaminant.
- K_d = distribution coefficient, in milliliters per gram. Values shown in Tables 11 and 12 were taken from Ames and Serne, 1991.
- θ_s = soil porosity, i.e., the fraction of the void space in a unit volume. The value 0.4 is assumed for soil porosity.
- ρ_s = bulk density of the soil, in grams per milliliter. The value 1.6 g/ml is used in the tables below.

Having defined C_0 and R_d , the groundwater concentration is calculated using the equation below.

$$C_w = C_0 (C/C_0) \text{Exp}(-\lambda R_d T_w) \quad (32)$$

where,

- C_w = concentration of a component in the groundwater, in micro-curies (or milligrams) per liter of water.
- C/C_o = ratio of final concentration in groundwater to the initial concentration without consideration of retardation or radioactive decay.
- λ = radioactive decay constant for a radionuclide, namely, the natural logarithm of 2 divided by the radioactive decay half-life in years. Values shown in Table 11 are from the *Radioactive Decay Data Tables* (DOE/TIC-11026). Chemicals have an infinite half-life, so their decay constants are zero.
- T_w = travel time from the bottom of the tank to the groundwater, in years.

The distribution coefficient, K_d , varies with the soil properties and the general chemical environment (i.e., alkaline, acid or organic). Values determined under a set of chemical conditions are only applicable to systems with similar chemical conditions. The rate of transport to the groundwater can change significantly if the chemical conditions are altered (e.g., by the addition of acid). Some Hanford Site-specific sorption data were used for this analysis. The sorption characteristics of the soils in the vicinity of the 241-C tank farm should be determined in order to better predict the mobility of key constituents of the waste in tank 241-C-106.

7.3 INGROWTH OF RADIOACTIVE DAUGHTERS

Two radionuclides, Pu-239 and Am-241, decay to other radionuclides with long half-lives. In particular, Pu-239 decays to U-235, and Am-241 decays to Np-237. The "parent" radionuclides (Pu-239 and Am-241) have much larger retardation factors than the "daughter" radionuclides (U-235 and Np-237). Therefore the groundwater concentration of the daughter radionuclides was estimated by using the following argument. The amount of the parent radionuclide decreases exponentially with time as it migrates through the soil. The daughter atoms are more mobile in the soil, and are transported ahead of the parent. The groundwater concentration of the daughter radionuclide as a function of which year of parent decay (T) is shown in the equation below.

$$C_{bw} = C_{ao} (C/C_o) \lambda_b T_r \text{Exp}[-\lambda_b R_b T_w - (\lambda_a R_a - \lambda_b R_b)T] \quad (33)$$

where,

- C_{bw} = concentration of daughter radionuclide in groundwater, in micro-curies per liter of water.
- C_{ao} = concentration of parent radionuclide in the liquid which leaks from the tank, in micro-curies per liter.
- λ_a = radioactive decay constant for the parent radionuclide, per year.
- λ_b = radioactive decay constant for the daughter radionuclide, per year.
- R_a = retardation factor for the parent radionuclide
- R_b = retardation factor for the daughter radionuclide
- T_r = time period over which the daughter activity accumulates prior to being separated from the parent by its faster migration speed. This time is assumed to be one year. It is also assumed that $(\lambda_b T_r) \ll 1$.

T = number of years since the leak started to the present age of the parent radionuclide as it migrates through the soil. This variable is tested for values which will maximize the groundwater concentration of the daughter radionuclide.

For both Pu-239 and Np-237, the coefficient of T is positive, therefore the largest concentration of the daughter in the groundwater is for T=0 years. This value was used in calculating the concentration in the groundwater. The choice of zero for T simply means that the amount of daughter activity produced during the first year of transport reaches the groundwater with a higher concentration than any other year of daughter production.

7.4 GROUNDWATER CONCENTRATION RESULTS

Results from the transport model were used to calculate the contaminant concentrations in Tables 11 and 12. Note that due to the differing retardation factors, the various contaminants arrive at differing times after sluicing. The estimated travel times are shown in the columns headed "Time, yr."

Table 11. Groundwater Concentrations of Radionuclides.

Nuclide	Half-life, yr	K_d , mL/g	Time, yr ($R_d \cdot T_w$)	Groundwater Conc, $\mu\text{Ci/L}$	
				Case I	Case II
C-14	5,730	0	57	5.3E-03	1.3E-03
Co-60	5.271	50	11,000	0	0
Sr-90	28.6	25	5800	0	0
Tc-99	213,000	0	57	0.35	0.089
Ru-106	1.0088	20	4600	0	0
Sb-125	2.77	0	57	1.8E-06	4.6E-07
I-129	1.57E+07	0	57	7.9E-05	2.0E-05
Cs-137	30.17	50	11,000	0	0
U-234	244,500	2	510	2.5E-04	6.2E-05
U-235	7.04E+08	2	510	0	0
U-238	4.47E+09	2	510	2.5E-04	6.2E-05
Np-237	2.14E+06	2	510	0	0
Pu-239	24,131	250	57,000	0.42	0.11
Am-241	432.2	200	47,000	0	0

Note: The travel time to the groundwater without retardation is 57 years for both cases.

7.5 COMPARISON WITH RADIATION STANDARDS

The Environmental Protection Agency (EPA) has set a drinking water dose limit at 4 mrem per year (EPA 1991). This dose is based on an assumed water consumption rate of 2 liters each day, or 730 liters per year. Radiation doses are computed using the dose conversion factors shown in Table 13 obtained from Napier 1988, DOE 1988, and EPA 1988. The dose from contaminated

Table 12. Groundwater Concentrations of Chemicals.

Chemical	K_d , mL/g	Time, yr ($R_d \cdot T_w$)	Groundwater Conc, mg/L	
			Case I	Case II
Ag	30	6900	0.015	0.0039
Al	20	4600	5.1	1.3
B	0	57	0.037	0.0093
Ba	25	5800	0.022	0.0055
Bi	1000	230,000	0.17	0.042
Ca	20	4600	0.30	0.076
Cd	30	6900	0.091	0.023
Co	50	11,000	0	0
Cr	0	57	0.085	0.022
Cu	30	6900	0.012	0.0031
Fe	50	11,000	0.018	0.0046
Hg	30	6900	9.3E-04	2.4E-04
K	4	970	37	9.3
Mg	20	4600	0.080	0.020
Mn	50	11,000	0.17	0.042
Mo	0	57	0.0091	0.0023
Na	4	970	380	97
Ni	30	6900	0.25	0.063
P	0	57	3.0	0.76
Pb	30	6900	0.33	0.084
Ru	20	4600	0	0
Si	20	4600	0.25	0.063
U	2	510	0.73	0.19
Zn	30	6900	0.028	7.2E-04
Zr	40	9200	8.9	2.3
Cl	0	57	1.0	0.26
CO ₃	0	57	0.86	0.22
F	0	57	2.5	0.63
NO ₂	0	57	14	3.5
NO ₃	0	57	35	8.9
OH	0	57	25	6.3
PO ₄	50	11,000	0.11	0.029
SO ₄	0	57	20	5.1
TOC	0	57	30	7.6

Note: The travel time to the groundwater without retardation is 57 years for both cases.

Table 13. Dose Conversion Factors for Radionuclides.

Nuclide	Dose Conversion Factors, mrem/ μ Ci		
	GENII	DOE	EPA
C-14	2.06	2.1	2.09
Co-60	26.5	26	26.9
Sr-90	132	140	153
Tc-99	2.23	1.3	1.46
Ru-106	27	21	27.4
Sb-125	3.69	3.38	3.65
I-129	245	280	276
Cs-137	47	50	50.0
U-234	283	260	283
U-235	270	250	267
U-238	272	243	268
Np-237	5240	3900	4440
Pu-239	3570	4300	3540
Am-241	3630	4500	3640

drinking water is the product of the water concentration, the volume of water consumed, and the ingestion dose conversion factor. Table 14 summarizes doses for the above water concentrations. Very small doses (less than 0.01 mrem) are not shown.

Note that the doses for Pu-239 in Table 14 are well above the drinking water standard of 4 mrem/yr. This is not a logical result from the analysis since plutonium is strongly adsorbed onto the soil particles, which retards its transport to the groundwater. These doses may not occur for more than 57,000 years. The travel time is so great that if the period of interest is restricted to 10,000 years, then the dose from Pu-239 is negligible.

Radiation doses for U-235 and Np-237 were computed from the amounts of the parent radionuclides (Pu-239 and Am-241) initially present. From the doses shown in Table 14, it is evident that these progeny radionuclides are very small contributors to the final dose.

The doses listed in Table 14 reflect the inherent conservatism of the two-dimensional transport model. The doses shown correspond to that received from drinking the groundwater at a point 25 m from the center of the tank in the direction of flow in the aquifer. The actual dose received will depend on the drinking water well location and the extent of mixing in the groundwater. Lateral spreading will occur and any well is unlikely to be aligned in the same general direction as the flow in the aquifer. Conversely, there may be other sources of contamination (i.e., new leaking tanks or existing contamination from prior leaks) which could contribute to the dose. Scenarios postulated for other work in which mixing occurs as a result of pumping out the groundwater for irrigation and drinking water purposes combined with replenishment showed that significant dilution may occur.

Table 14. Ingestion Dose from Drinking Water.

Nuclide	Case I Ingestion Dose, mrem/yr			Case II Ingestion Dose, mrem/yr		
	GENII	DOE	EPA	GENII	DOE	EPA
C-14	7.9	8.1	8.0	2.0	2.1	2.0
Co-60						
Sr-90						
Tc-99	570	330	370	140	84	94
Ru-106						
Sb-125						
I-129						
Cs-137						
U-234	51	47	51	13	12	13
U-235						
U-238	49	44	48	12	11	12
Np-237	0.13	0.096	0.11	0.033	0.024	0.028
Pu-239*	1.1E+06	1.3E+06	1.1E+06	2.8E+05	3.3E+05	2.8E+05
Am-241						

*The travel time of 57,000 yr postpones any adverse effect.

Some of the doses in Table 14 are well above the standard, so that tank leakage cannot be ignored. As has been previously discussed, all that needs to be done in practice is to reduce the flow of water past the outside of the tanks. This could be done by constructing a waterproof barrier over the tanks, and maintaining it until suitable remediation can be accomplished.

7.6 COMPARISON WITH CHEMICAL STANDARDS

The EPA and the Washington State Department of Ecology (Ecology) have established standards for the allowable concentrations of certain chemical species in drinking water (Ecology 1982). The standards are based on a lifetime risk of $1.0\text{E-}06$ to an individual drinking 730 liters per year for 70 years. These standards are shown in Table 15 below, along with the ratio of the predicted concentration in the groundwater to the standard. Ratios less than one percent are not shown. As with radionuclides, some of the concentrations are well above the standard, so that tank leakage cannot be ignored. However, significant dilution will occur between the source of the leak and any users of the groundwater.

7.7 TANK CLOSURE

Under Chapter 173-303 of the Washington Administrative Code (WAC) (Ecology 1987) the State of Washington administers portions of the Resource Conservation and Recovery Act of 1976 (RCRA) (RCRA 1976). The Tri-Party Agreement requires that the SSTs be operated and closed in accordance with

Table 15. Comparison of Chemical Concentrations in Groundwater with Standards for Drinking Water.

Chemical	Standard mg/L	Ratio to Standard	
		Case I	Case II
Ag	0.05	0.31	0.08
Ba	1	0.02	
Cd	0.01	9.1	2.3
Cr	0.05	1.7	0.43
Cu	1	0.01	
Fe	0.3	0.06	0.02
Hg	0.002	0.46	0.12
Mn	0.05	3.3	0.84
Pb	0.05	6.6	1.7
Zn	5		
Cl	250		
F	2	1.25	0.32
NO ₂	45	0.30	0.08
NO ₃	45	0.77	0.20
SO ₄	250	0.08	0.02

these state and federal regulations governing treatment, storage, and disposal (TSD) facilities.

"Clean Closure" is the baseline requirement for closure of RCRA regulated hazardous waste storage tanks in Washington State. All RCRA interim status TSD closures are subject to the general closure requirements specified in 40 CFR 265 Subpart G, which are incorporated into Washington's dangerous waste program by reference. The requirements for closure of a tank system require the owner or operator to remove or decontaminate all waste residues, contaminated system components, and contaminated soils. Under the state standard, removal or decontamination is complete when residues of federal designated hazardous wastes and waste constituents are at background environmental levels, and residues from state-only designated dangerous wastes are below specified limits.

These regulations also include provisions for closure of the tank system as land disposal units if the owner or operator demonstrates that not all contaminated soils can be practicably removed or decontaminated. This risk-based approach requires the owner or operator of a landfill to cover the landfill with a final cover designed and constructed to: (1) provide long-term minimization of migration of liquids through the closed landfill; (2) function with minimum maintenance; (3) promote drainage and minimize erosion or abrasion of the cover; (4) accommodate settling and subsidence so that the cover's integrity is maintained; and (5) have a permeability less than or equal to the permeability of any bottom liner system or natural subsoils present. After final closure, the owner or operator must comply with

post-closure requirements including maintenance and monitoring throughout the post-closure care period. The owner or operator must: (1) maintain the integrity and effectiveness of the final cover, including making repairs to the cover as necessary to correct the effects of settling, subsidence, erosion, or other events; (2) maintain and monitor the leak detection system, and comply with all other applicable leak detection system requirements; (3) maintain and monitor the ground-water monitoring system and comply with all other applicable requirements; (4) prevent run-on and run-off from eroding or otherwise damaging the final cover; and (5) protect and maintain surveyed benchmarks used in complying with RCRA.

Any closure and corrective actions conducted for SST system remediation will also have to satisfy NEPA requirements. The DOE-RL has previously committed to provide NEPA documentation in the form of an environmental impact statement (EIS) to address the issue of SST waste disposal. Preparation of an EIS involves analyzing several reasonable alternatives, including no action, that will implement the final disposal of SSTs. A discussion of each alternative presents, in comparative form, information on environmental consequences and a description of the affected environment for each alternative.

Closure of tank 241-C-106 may require some form of soil remediation because of previous leaks and spills in the 241-C tank farm and in general throughout the tank farms. It is likely that removal or decontamination of all of the contaminated soil will not be practicable and at least some parts of the tank farms will be closed as land disposal units. Specific details will be spelled out in the closure plan for the SSTs required under RCRA. The closure plan for the full-scale tank farm closure demonstration project is due to Ecology for approval in December 2003 (TPA milestone M-08-03).

Further contamination of the soil beneath 241-C tank farm as a result of a leak in tank 241-C-106 is a possibility. The bulk of the contaminants would remain in the upper layers of the soil column for some time. A surface barrier would slow the migration of the contaminants. Use of a subsurface barrier, while it would contain the waste, would still entail some local contamination and possibly require recovery of the contaminated barrier itself. Development of subsurface barriers is still in its initial stages. A subsurface barrier must first be demonstrated and its performance verified before it can be considered practicable.

NOMENCLATURE FOR RISK AND IMPACT EVALUATION

Symbol	Definition	Units
C_{ao}	Concentration of parent radionuclide in the liquid which leaks from the tank	$\mu\text{Ci/L}$
C_{bw}	Concentration of daughter radionuclide in the groundwater	$\mu\text{Ci/L}$
C_o	Concentration of the liquid which leaks from the tank	$\mu\text{Ci/L}$ or mg/L
C_w	Concentration of a component in the groundwater	$\mu\text{Ci/L}$ or mg/L
C/C_o	Ratio of final concentration in groundwater to the initial concentration without consideration of retardation or radioactive decay	-
K_d	Distribution coefficient	mL/g
R_a, R_b	Retardation factors for the parent and daughter radionuclides in a decay chain	-
R_d	Retardation factor (i.e., the ratio of the transport speed of the contaminant to the transport speed of the water)	-
T	Number of years since the leak started to the present age of the parent nuclide as it migrates through the soil	yr
T_r	Time period over which the daughter activity accumulates prior to being separated from the parent by its faster migration speed	yr
T_w	Travel time from the bottom of the tank to the groundwater	yr
λ	Radioactive decay constant for a radionuclide (i.e., the natural logarithm of 2 divided by the radioactive decay half-life)	yr^{-1}
λ_a, λ_b	Radioactive decay constants for the parent and daughter radionuclides in a decay chain	yr^{-1}
ρ_s	Bulk density of the soil	g/mL
θ_s	Soil porosity (i.e., the fraction of the void space in a unit volume)	-

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8.0 CONCLUSIONS

There is considerable uncertainty in the estimate of the amount that may leak during sluicing, as the mechanisms by which leaks may occur are not fully understood. The most likely leak path is through many small cracks caused by stress corrosion cracking around welds in the tank liner, and subsequently out through the construction joint in the concrete encasement. The extent of stress corrosion cracking of the liner is unknown. Results of a model developed at the SRP which predicts whether stress corrosion cracking may occur based on the chemistry of the waste are inconclusive. The concentrations of hydroxide and nitrate in the waste lie between the values for which cracking is either predicted to occur or not occur. The structural analysis which estimates the extent of cracking in the concrete encasement is incomplete. The actual leaked volume is expected to be on the order of a few cubic meters (several thousand gallons), based on historical leak rates and the assumption that sluicing would be halted upon detection of a leak.

However, the analyses showed that leakage up to 150 m³ (40,000 gal) may occur during sluicing by the most likely leak mechanism. This analytical conclusion reflects the conservative assumptions used to establish an upper bound on the amount of leakage and its impact. The leak rate is determined by the hydraulic head and the resistances of the cracks in the liner and the cracks in the concrete encasement or the gap in its construction joint, and the concrete-to-soil interface saturation/permeation level. The hydraulic head, the driving force behind any leaks, will decrease as sluicing progresses. Liquid is held up in the sludge to an estimated height of 61 cm (24 in.) by capillary forces alone. The hydraulic resistance of the sludge would impede liquid flow to any leak sites. The size of any leaks would be further limited by (1) the ability to detect leaks, (2) administrative controls on liquid inventories, (3) the tendency of solids in the sludge to plug any leaks, and (4) that free liquid in the sluice tank is limited and could be pumped out in a short time.

Any postulated waste leak, upon reaching the soil will be driven downward by rainfall and runoff from the tank dome. This moisture recharge is concentrated around the tank perimeter due to the "umbrella effect" of the tank structure. Travel time to the aquifer is calculated to be about 60 yr, provided the amount leaked is small (up to tens of cubic meters) compared to the rate of recharge (see Section 6.3). Even small leaks are expected to find their way to the groundwater. The recharge rate could be reduced by providing an impermeable cover over the tank and surrounding area. The contaminated soil could be recovered or treated after sluicing if required as part of the overall site closure activities under TPA milestone M-09. Additional sensitivity analysis of a surface barrier's area versus the recharge rate is needed. The transport model used to simulate the leak is conservative in that the travel time estimates are minimal for representative conditions and the relative concentrations in the groundwater are maximal.

Local concentrations of key waste constituents in the groundwater resulting from the most likely leak scenario are predicted to be greater than allowed by drinking water standards. These doses reflect the inherent conservatism in the transport model. The actual dose received will depend on the water well location, the extent of mixing in the aquifer, and any lateral spreading that occurs. Significant dilution is possible between the source of

the leak and the dose receptor, such that the resulting dose may approximate or even be less than allowed by drinking water standards. This study assumed a well located 25 m from the tank.

Closure of 241-C-106 may require some form of soil remediation because of previous leaks and spills in the 241-C tank farm and in general throughout the tank farms. It is likely that removal or decontamination of all of the contaminated soil will not be practicable and at least some parts of the tank farms will be closed as land disposal units. Specific details will be spelled out in the closure plan for the SSTs required under RCRA.

Further contamination of the soil beneath 241-C tank farm as a result of a leak in 241-C-106 is a possibility. The bulk of the contaminants would remain in the upper layers of the soil column for some time. A surface barrier would slow the migration of the contaminants. Use of a subsurface barrier, while it would contain the waste, would still entail some local contamination and possibly require recovery of the contaminated barrier itself. Development of subsurface barriers is still in its initial stages. A subsurface barrier must first be demonstrated and its performance verified before it can be considered practicable.

Tank 241-C-106 is currently not leaking. Should tank 241-C-106 start to leak during a retrieval operation, steps can be taken to ensure that any waste that leaks stays in the vicinity of the leak site. A cover over the tank would greatly reduce recharge driven migration.

The current leak detection system's sensitivity to leakage volumes or locations should be determined. Its capabilities may be enhanced and combined with process controls to monitor the waste inventory and liquid volumes in order to better detect leaks during sluicing. More frequent monitoring would shorten the time to detect a leak that has reached the drywells. Providing lateral monitoring wells beneath the tank could allow detection of smaller leaks.

Installation of a sub-surface barrier prior to retrieval operations would also limit the migration of any contaminants to the vicinity of the tank. Development and design work is required on sub-surface barriers before they are proven to be viable. In the event that removal of contaminated soil from around and beneath the tanks is required as part of the SST operable unit closure, sub-surface barriers may prove to be unworkable. Analyses show that surface barriers are effective in limiting the migration of any tank leaks. Ultimately, any decision made on sluicing or retrieval with or without some form of barrier must address tank farm closure requirements and impacts.

9.0 RECOMMENDATIONS FOR FURTHER WORK

The current tank 241-C-106 leak and transport model could be improved with additional work. Recommendations for further work include the following, which are not prioritized.

1. Further define the sluicing operation and properties of the waste to be retrieved.

This would include the equipment and facilities to be used; duration of sluicing; results of thermal modeling of tank 241-C-106 that show whether the sludge must be kept covered with water for cooling purposes; and results of analyses on new core samples (i.e., waste chemistry, physical properties, sludge solubility, layering, particle size, porosity, and permeability). Solubility could be important in determining under what conditions the solid deposits which plug any cracks would dissolve.

2. Re-evaluate the leak mechanisms.

Verify the results of the thermal analysis of the tank based on its time-temperature history. Incorporate the results in the structural analysis of the concrete encasement to determine whether cracking has occurred. Ascertain the physical condition of the tank liner either by direct inspection or parametric studies.

3. Model the tank contents.

Consider the time-dependent sludge removal, variable hydrostatic head, forces imparted by sluicing on the tank wall and the possibility of initiating tank leaks, liquid holdup in the sludge by capillary forces, flow through the sludge to potential leak sites, variations in the source-loading history, and plugging of leaks by the sludge itself.

4. Determine actual values for those parameters which affect the rate of leak transport.

Determine the soil types beneath the 241-C tank farm and the hydraulic parameters for leak transport. Measure the soil moisture, its sorption characteristics, and the recharge rate due to rainfall in and around the tank farm.

5. Use a 3-dimensional transport model.

A 3-dimensional model would introduce less conservatism and could account for recharge and leakage from adjacent tanks. Initial work focused on transport to the aquifer, which is dominated by the recharge rate and width of the ring of enhanced recharge. Sensitivity analyses of the recharge rate and width of the ring of enhanced recharge would identify any interdependencies. Transport in the groundwater to a hypothetical user could be modeled to determine the extent of dilution and the effective dose received.

6. Calibrate the transport model using information on past leaks.

Review field measurements used to track migration of past leaks. Analyze data for applicability. Compare leak movement with that predicted. Adjust model as needed based on results.

Additional work in support of the overall effort to retrieve waste from the underground storage tanks on the Hanford Site should include the following:

7. Generalize the tank 241-C-106 leak and transport model for other tanks.

Consider tank location (proximity to other structures), condition (leaking or non-leaking), status (partially or interim stabilized), operating history, waste types and properties (sludge and salt cake), and soil properties and transport characteristics.

8. Investigate the use of surface barriers to eliminate recharge.

Additional sensitivity analysis is needed to determine the covered area needed to affect the recharge rate. Identify different cover types for single tanks and entire tank farms. Develop these concepts in sufficient detail to determine feasibility, assess performance, and estimate cost.

9. Evaluate current leak detection capability and alternate technologies.

Capabilities of the current leak detection system and its operation should be investigated. Identify any limitations or improvements needed. Determine the sensitivity of the current system to leaked volumes or locations. Propose and evaluate alternate technologies (e.g., soil resistivity).

10. Investigate the use of subsurface barriers to contain leaks.

Identify potential subsurface barrier types. Describe any previous applications or stage of development. Develop these concepts in sufficient detail to determine feasibility, assess performance, and estimate cost.

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APPENDIX A

**RECOMMENDATIONS ON TANK 241-C-106 CRACK SIZES
TO BE USED FOR LEAKAGE RATE STUDIES**

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**Westinghouse
Hanford Company****Internal
Memo**

From: Component Stress Analysis
Phone: 6-1063 H5-56
Date: November 4, 1992
Subject: RECOMMENDATIONS ON 241-C-106 CRACK SIZES TO BE USED FOR LEAKAGE RATE STUDIES

CSA:CJM:ggb:92/16

To: J. P. Harris S4-55

cc: D. J. Green H5-53
L. J. Julyk *LJ* H5-56
S. S. Lowe H5-49
M. A. Scott H5-56
K. G. Squires S4-55
J. B. Truitt *JB* H5-56
M. P. Weis *MPW* H5-56
CJM File/LB

References:

1. Moore, C. J. and M. P. Weis, "241-C-106 Preliminary Concrete Cracking Characterization," Internal Memo to J. P. Harris, October 22, 1992, Westinghouse Hanford Company, Richland, Washington.
2. Bander, T. J., "Thermal History of Tank 241-C-106," WHC-SD-WM-ER-161, Rev. 0, 1992, Westinghouse Hanford Company, Richland, Washington.
3. *ACI Manual of Concrete Practice*, 1987, American Concrete Institute, Detroit, Michigan.

Per your request on October 30, 1992 the 241-C-106 single-shell waste tank cracking characterization provided in Reference 1 and associated construction joints for C-106 were reviewed. The following recommendations on crack sizes for use in leak-rate studies are provided.

241-C-106 CONSTRUCTION JOINT

The only C-106 tank construction joint that exists below any current or past waste level is located between the concrete side wall and the footing region of the tank floor. The 1944 engineering drawings of this construction joint show a firm anchorage with 3/4-in. diameter rebar looped around the footing rebar. The weight of the wall, dome, and soil over burden maintains a compressive load on this construction joint at all times. Additionally, the construction joint and the floor of the concrete tank is sealed with an additional 2-in. thick layer of grout.

The large compressive load makes it highly improbable that there will be any opening of the construction joint. This situation would also apply to any

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cooldown period induced during sluicing. It would be conservative to assume a compression closed crack with a 0.001-in. opening for maximum leak-rate studies.

THERMAL INDUCED CRACKING

The preliminary concrete cracking characterization from ABAQUS analysis results documented in Reference 1, showed through-wall cracking in the concrete wall and floor during the initial days of loading of high heat waste material. The predicted temperature distribution of the tank structural components was obtained from Reference 2. The predicted crack distribution is sensitive to the thermal conditions. Hence, the exact waste loading sequence and the waste temperatures should be reviewed for correctness in the Reference 2 characterization.

Data on the waste loading sequence and the waste temperature is needed to refine the thermal and structural predictions. If the current thermal characterization is consistent with the waste loading procedure used, then Component Stress Analysis is able to recommend modifications to these procedures to minimize through cracking in future waste storage tanks.

Additionally, Reference 1 showed preliminary predicted laminar in-plane cracking in the concrete floor slab that was observed to reach a maximum as the peak temperature of C-106 thermal history is reached. This in-plane cracking is not expected to promote through-wall leakage. Recommended through-wall crack sizes in tank floor and wall to be used in leak-rate studies are given below.

THROUGH FLOOR CRACKING

The Reference 1 preliminary analysis results predicted an initial radial tensile strain of 1.5×10^{-4} in/in over a finite region in the concrete floor at an average radius of 136 in. This is interpreted to result in a region of through-floor cracking with an integrated total crack opening of 0.01 in. The ACI manual of concrete practice has been applied to these results to obtain recommendations for leak-rate calculations. As shown below the ACI equation for tension induced cracking of the floor predicts an individual crack size of 0.002 in. This 0.002-in. crack size corresponds to 5 cracks within the 1.5×10^{-4} in/in strain region in the concrete floor of the tank.

$$w = 0.10 f_s \sqrt[3]{d_c A} \times 10^{-3}$$

Ref. 3 pg. 224 R-20, eq (4.10)

w = Crack width in inches

f_s = rebar stress in kip/in²

d_c = concrete cover over rebar in inches

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$$\begin{aligned}
 A &= \text{area of concrete symmetric with reinforcing steel divided by number of bars in in}^2 \\
 f_s &= E\epsilon = (29 \times 10^6 \text{ kip/in}^2) (1.5 \times 10^{-4} \text{ in/in}) \\
 &= 4.35 \text{ kip/in}^2 \\
 d_c &= 1.5 \text{ in.} \\
 A_c &= 12 \times 6 = 72 \text{ in}^2 \quad (6\text{-in.-thick concrete slab with 12-in. bar spacing) \\
 w &= (0.10) (4.35) \sqrt[3]{1.5(72)} \times 10^{-3} = 0.002 \text{ in.} \\
 \text{Number of cracks} &= \frac{0.01 \text{ in.}}{0.002 \text{ in.}} = 5
 \end{aligned}$$

The time history analysis results from Reference 1 showed that the through-floor crack region in the floor is in radial compression in 1992. Hence, the cracks are forced closed. Considering the initial opening of the 5 cracks it would be conservative to assume for the leak-rate study that they are now open with a crack size opening of 0.001 in. The cracks should be idealized as rough surfaces with a through path length including grout of at least 8 in.

THROUGH WALL CRACKING

The Reference 1 preliminary analysis results showed that the hoop strain of 2.0×10^{-4} in/in (between a vertical location of 68 in. and 236 in. measured from the bottom surface of the tank) results in a wall region of through-wall cracking with a integrated crack opening of 0.6 in. The ACI manual of concrete practice has been applied to these results to obtain recommendations for leakage rate calculations. As shown below the ACI equation for tension induced cracking of the wall predicts a crack size of 0.0024 in. The 0.0024-in. crack size corresponds to 250 cracks over the tank circumference based on the rebar in the wall.

$$\begin{aligned}
 f_s &= (29 \times 10^6 \text{ kip/in}^2) (2 \times 10^{-4} \frac{\text{in}}{\text{in}}) = 5.8 \text{ kip/in}^2 \\
 d_c &= 2 \text{ in.} \\
 A &= \frac{6 \times 12}{2} = 36 \text{ in}^2 \quad (12\text{-in. thick concrete wall with} \\
 &\quad 6 \text{ in. bar spacing and 2 hoop bars through thickness) \\
 w &= (0.10) (5.8) \sqrt[3]{2(36)} \times 10^{-3} = 0.0024 \text{ in.} \\
 \text{Number of cracks} &= \frac{0.6 \text{ in.}}{0.0024 \text{ in.}} = 250
 \end{aligned}$$

J. P. Harris
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The time history analysis results from Reference 1 showed that the through crack region in the wall is in hoop compression in 1992. The cracks are forced closed. Considering the initial opening of the 250 estimated cracks it would be conservative to assume for the leak-rate study that they are open 0.001 in. The cracks should be idealized as rough surfaces with a through path length of at least 12 in. The applicable crack length will depend on the waste height assumed for the leak-rate study. In addition, the study should address the variation in hydrostatic pressure with the waste depth.

ADDITIONAL COMMENTS

The leak-rate calculation should incorporate the effects of individual cracks into the study. The crack surface roughness combined with fluid boundary layer effects will result in significantly less leakage than an analysis assuming a single large orifice.

C. J. Moore, Principal Engineer
Component Stress Analysis

ggb

APPENDIX B

TANK 241-C-106 PRELIMINARY CONCRETE CRACKING CHARACTERIZATION

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**Westinghouse
Hanford Company**

WHC-SD-WM-ES-218 Rev. 1

**Internal
Memo**

From: Component Stress Analysis
Phone: 6-1063 H5-56
Date: October 22, 1992
Subject: 241-C-106 PRELIMINARY CONCRETE CRACKING CHARACTERIZATION

CSA:CJM:ggb:92/15

To: J. P. Harris S4-55

cc: D. J. Green H5-53
L. J. Julyk H5-56
S. S. Lowe H5-49
K. G. Squires S4-55
J. B. Truitt *[Signature]* H5-56
CJM File/LB
MPW File/LB

Reference: Bander, T. J., "Thermal History of Tank 241-C-106," WHC-SD-WM-ER-161, Rev. 0, 1992, Westinghouse Hanford Company, Richland, Washington.

INTRODUCTION

The objective of this effort was to estimate possible concrete crack sizes in the 241-C-106 waste storage tank to refine estimates of potential leak rates. Archived preliminary results from the terminated C-106 structural integrity assessment initiated in FY 92 were retrieved and used for this effort. The FY 92 structural integrity assessment addressed in situ conditions (steady-state and transient temperature distributions and drain and fill cycles) to simulate aging effects on the structure, including concrete property degradation and creep due to high temperature exposure. The thermal analysis by Bander was used to simulate the 241-C-106 temperature history.

The axisymmetric finite-element model, used in the structural integrity assessment, employs a nonlinear reinforced-concrete constitutive model. The prediction of cracks by the concrete constitutive model uses a smeared-crack approach, where the onset of cracking is predicted at element integration points as a function of the mechanical tensile-strain. While discrete cracks can not be predicted, the smeared-crack theory allows for the prediction of cracked regions. Cracked regions in the finite-element model are depicted by symbols at concrete element integration points as illustrated in Figure 1. The cracks can either be open or closed, but are not allowed to heal.

CRACK OPENING SIZE DETERMINATION AND RESULTS

To provide a quantitative measure of the crack opening size and subsequently the crack flow area, the mechanical tensile-strain is integrated over the cracked region. This provides the total crack opening size in the direction

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of tensile strain. The explicit distribution of crack sizes in the tank concrete cannot be determined. If the wall exhibits hoop tensile-strain, the integrated crack opening is calculated as follows:

$$\text{Crack opening size} = \int_0^{2\pi} \epsilon_{\text{hoop}} R d\theta = 2\pi R \epsilon_{\text{hoop}}$$

This method does not provide a discrete distribution of crack sizes but represents the total sum over the characteristic length ($2\pi R$) of the tank circumference.

The minimum mechanical-tensile strain required for crack initiation is about 10^{-4} in/in. Mechanical-strain contour plots for the radial, vertical, and hoop directions are provided in Figures 2 through 4, respectively for 1992 in situ conditions. The only strain contour plot that shows a mechanical tensile-strain in excess of 10^{-4} in/in is Figure 3 for vertical strain in the slab region of the tank. This vertical strain can be attributed to a Poisson's effect induced by the radial and hoop compressive loads applied by the confining soil as the slab thermally expands from the waste heat generation loading. Because these strains are in the vertical direction they lead to laminar cracks that are not conducive to through-wall leakage. Other cracks shown in Figure 1 that are predicted to penetrate through the tank wall and slab are predicted to be closed for 1992. These cracks were created at an earlier point in time, and now lie in compressive regions.

To estimate the maximum mechanical-tensile-strain, history plots of the mechanical strain were generated for various points around the tank. The results are illustrated in Figures 5 through 13. Mechanical-strain contour plots in Figures 14 through 16 confirm that the cracks were initiated early in the thermal history. Table 1 provides an estimate of the historical maximum mechanical-tensile strain as obtained from the strain history plots.

Table 1. Maximum Mechanical-Tensile Strain.

Location	Maximum Tensile-Strain (10^{-4} in/in)	Direction
Wall	0.6	vertical
Wall	2.0	hoop
Slab	1.5	radial
Slab	0.2	hoop

Table 1 lists strains for the crack directions that promote leakage; it does not include strains that promote laminar in-plane cracking (i.e. radial cracks in the wall and vertical cracks in the slab). The crack pattern (Figure 1) shows that open in-plane cracks currently exist in the slab. The vertical strain history of Figure 13 confirms this, as the mechanical tensile-strains are currently on the order of $2 \cdot 10^{-4}$ in/in.

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A conservative estimate of the 1992 crack opening size can be determined by assuming that the historical maximum mechanical-tensile strain is constant over all predicted crack regions in the same direction. Applying this criteria, the crack opening sizes and flow areas were calculated as summarized in Table 2.

Table 2. 241-C-106 Concrete Crack Opening Size and Total Leakage Flow Area.

Location	Crack Opening Size (in)	Total Crack Flow Area (in ²)	Direction
Wall	0	0	vertical
Wall	0.6	97	hoop
Slab	0.01	8	radial
Slab	0	0	hoop

The calculations are presented in Attachment 1. A basic assumption of the calculations is that the maximum tensile strain (Table 1) existed over the entire cracked region. The crack area in the wall region, according to Figure 1, is distributed evenly over the cylindrical section of the tank above the bottom knuckle region. This area can be reduced proportionally according to the height of the waste in the tank above the lowest crack in the wall. The waste height history in the tank up to the present time is given in Bander.

The approach used to calculate the crack sizes in Table 1 is conservative. However, the finite-element model used to generate this data has not been rigorously reviewed. In fact, the structural integrity assessment work scheduled for FY 93 includes refinements of the current model. The results should be viewed as preliminary, although they are expected to be conservative.

SUMMARY AND CONCLUSIONS

A preliminary evaluation of the concrete cracking with calculations of the maximum crack opening areas has been completed. The cracking plots and strain contours indicate early through-thickness cracking in the concrete slab and the concrete wall due to the concentrated waste heat in the lower region. Additional laminar in-plane cracking in the concrete slab is observed to be a maximum as the peak temperature of the C-106 thermal history is reached.

The high heat conditions of the C-106 tank contribute to the cracking of the concrete wall and the concrete slab. Although, the thermal induced stresses contribute directly to concrete cracking, the ABAQUS computer model predicts closed through-thickness cracks for 1992. This result should be tempered by the understanding that debris from the concrete cracking and soil may lodge

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in the cracks thus maintaining a leakage path. The maximum crack opening sizes obtained from the preliminary analysis are maximums observed during the full time history of the tank.

The ABAQUS model used for this analysis does not yet have a final soils model. Soil input parameters for Hanford specific soil must be developed and applied. Also, parameter studies are needed to investigate the interaction of the soil induced loads with the tank. It is the opinion of the CSA engineering staff that the soil modeling can significantly change the predicted cracking in the foot region of the tank.

Data on the tank construction sequence including the soil profiles and fill and compaction procedures are needed to refine the nonlinear ABAQUS model. Typical conservative design assumptions are not necessarily conservative when employed for prediction of cracking conditions.

Any comments or questions on this material may be directed to Carleton Moore (6-1063), Mike Weis (6-5446), or Larry Julyk (6-4608).

C. J. Moore, Principal Engineer
Component Stress Analysis

M. P. Weis, Engineer
Component Stress Analysis

ggb

Attachment

Figure 1. Crack Pattern for 1992 In Situ Conditions.

Figure 2. Radial Mechanical-Strain Contours for 1992 In Situ Conditions.

Figure 3. Vertical Mechanical-Strain Contours for 1992 In Situ Conditions.

Figure 4. Hoop (Tangential) Mechanical-Strain Contours for 1992 In Situ Conditions.

Figure 5. Complete Hoop Strain History in the Wall.

NOTES: VERSION 4-8-1

Figure 6. Initial Hoop Strain History in the Wall.

REVISED 4-9-1

Figure 7. Complete Vertical Strain History in the Wall.

APR 1983 VERSION 4-3-1

Figure 8. Initial Vertical Strain History in the Wall.

ANALYSIS VERSION 4-8-1

Figure 9. Complete Radial Strain History in the Slab.

ADDITIONAL VERSION 4-3-1

Figure 10. Initial Radial Strain History in the Slab.

REVISION 4-9-1

Figure 11. Complete Hoop Strain History in the Slab.

NOTES: SECTION 4-5-1

Figure 12. Initial Hoop Strain History in the Slab.

REVISED VERSION 4-9-1

Figure 13. Complete Vertical Strain History in the Slab.

APPROX VERSION 4-9-1

Figure 14. Radial Mechanical-Strain Contours for Early Tank History.

Figure 15. Vertical Mechanical-Strain Contours for Early Tank History.

Figure 16. Hoop (Tangential) Mechanical-Strain Contours for Early Tank History.

ATTACHMENT

DESIGN CALCULATION

(1) Drawing H-2-xxxxx (2) Doc. No. _____ (3) Page 1 of 3
 (4) Building _____ (5) Rev. _____ (6) Job No. _____
 (7) Subject Crack opening size and total crack flow area - Tank 241-C-106
 (8) Originator Michael P. Weis Date 10/22/92
 (9) Checker [Signature] Date 10/22/92
 (10)

WALL

Calculation of crack opening size and total crack flow area because of hoop cracks

$$R_i = 451 \cdot \text{in} \quad R_o = R_i + 12 \cdot \text{in}$$

$$R = \frac{R_o + R_i}{2}$$

average radius of the tank wall

$$R = 38.083 \cdot \text{ft}$$

$$L = 2 \cdot \pi \cdot R$$

characteristic length over which strain occurs

$$L = 239.285 \cdot \text{ft}$$

$$\epsilon = 2 \cdot 10^{-4}$$

maximum hoop strain

$$w = L \cdot \epsilon$$

total width of crack opening

$$w = 0.574 \cdot \text{in}$$

$$h = (236 - 68) \cdot \text{in}$$

total wall height

$$A = w \cdot h$$

total area of crack opening

$$A = 96.48 \cdot \text{in}^2 \quad A = 0.67 \cdot \text{ft}^2$$

Distributing the crack area along the wall, as hydrostatic pressure varies with depth of waste

$$N = 10$$

number of sections

$$\Delta h = \frac{h}{N}$$

length of section

$$\Delta h = 16.8 \cdot \text{in}$$

$$\Delta A = w \cdot \Delta h$$

area of crack per section

$$\Delta A = 9.648 \cdot \text{in}^2 \quad \Delta A = 0.067 \cdot \text{ft}^2$$

FLOOR

Calculation of crack opening size and total crack flow area because of hoop cracks

$$\Delta L = (165 - 107) \cdot \text{in}$$

characteristic length over which strain occurs

$$\epsilon = 1.5 \cdot 10^{-4}$$

maximum radial strain (strain history plot)

$$w = \Delta L \cdot \epsilon$$

total width of crack opening

$$w = 0.009 \cdot \text{in}$$

$$r = \left(\frac{165 + 107}{2} \right) \cdot \text{in}$$

radius of location at center of cracked region

$$r = 136 \cdot \text{in}$$

$$A = w \cdot (2 \cdot \pi \cdot r)$$

total area of crack opening

$$A = 7.434 \cdot \text{in}^2$$

DESIGN CALCULATION

(1) Drawing H-2-xxxxx (2) Doc. No. _____ (3) Page 2 of 3
 (4) Building _____ (5) Rev. _____ (6) Job No. _____
 (7) Subject Crack opening size and total crack flow area - Tank 241-C-106
 (8) Originator Michael P. Weis *MPW* Date 10/22/92
 (9) Checker *J. Gulye* Date 10/22/92
 (10)

Wall height over which cracked area is distributed

241-C-106 CREEP ANALYSIS

DESIGN CALCULATION

(1) Drawing H-2-xxxxx (2) Doc. No. _____ (3) Page 3 of 3
 (4) Building _____ (5) Rev. _____ (6) Job No. _____
 (7) Subject Crack opening size and total crack flow area - Tank 241-C-106
 (8) Originator Michael P. Weis M. P. Weis Date 6/22/92
 (9) Checker J. Galyk Date 6/22/92
 (10)

Characteristic length of radial strain (crack - inducing) in the slab

241-C-106 CREEP ANALYSIS

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APPENDIX C
POSTULATED LEAK MECHANISMS AND CALCULATED LEAK RATES

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APPENDIX C

POSTULATED LEAK MECHANISMS AND CALCULATED LEAK RATES

Several leak mechanisms were postulated for this study. Changes in some of the assumptions were investigated to determine the effect on the predicted leak rates. The operating conditions were also varied to determine the sensitivity of the leak rates to changes in hydraulic head and tank pressure. The leak descriptions and operating conditions are described below.

Leak Descriptions:

- Leak Description A: Leakage through stress corrosion cracks in the steel liner on the tank wall with flow out through the construction joint. A total of 20 cracks occur in a 61 cm (24 in.) vertical section of the liner around the tank circumference. The cracks in the 0.95-cm (3/8-in.) thick liner measure 10 cm (4 in.) long by 0.005 cm (0.002 in.) wide with an average surface roughness of 0.003 cm (0.001 in.). Sludge is removed from the liner around one-fourth, one-half, and the entire tank circumference. The opening in the construction joint is 0.003 cm (0.001 in.) wide with an average surface roughness of 0.0005 cm (0.0002 in.) and a flow path 78.0 cm (30.7 in.) long. Ignore the asphalt membrane between the steel liner and concrete encasement.
- Leak Description B: Leakage through stress corrosion cracks in the steel liner on the tank bottom with flow out through thermal-induced cracks in the concrete encasement. A total of 100 cracks occur in the steel liner covering the tank bottom. The cracks in the liner measure 10 cm (4 in.) long by 0.005 cm (0.002 in.) wide with an average surface roughness of 0.003 cm (0.001 in.). Five circular cracks occur in the bottom of the concrete encasement. The cracks in the 20-cm (8-in.) thick concrete have an average radius of 3.45 m (136 in.) and are 0.003 cm (0.001 in.) wide with an average surface roughness of 0.0005 cm (0.0002 in.). Assume the sludge in the tank bottom offers negligible resistance to flow. Ignore the asphalt membrane.
- Leak Descriptions C-D-E: Leakage through single holes in the tank liner with flow out through the construction joint. The holes in the liner are 0.079 cm, 0.64 cm, and 2.5 cm (1/32 in., 1/4 in., and 1 in.) in diameter. Assume flow from a point source through the construction joint occurs along 1/8 the tank circumference.
- Leak Description F: Leakage out the tank bottom through a large crack in the concrete encasement. The crack measures 91 cm (36 in.) long by 0.32 cm (1/8 in.) wide with an average surface roughness of 0.031 cm (0.012 in.) (Perry and Green 1984). Assume the steel liner is similarly failed and offers negligible resistance to flow. Assume the sludge in the tank bottom offers negligible resistance to flow.

- Leak Description G: Leakage out the tank bottom through a large hole in the concrete encasement. The hole measures 23 cm (9 in.) in diameter and is filled with pulverized concrete with 1.9-cm (3/4-in.) pebble size and 1% void space. Assume the steel liner is similarly failed and offers negligible resistance to flow. Assume the sludge in the tank bottom offers negligible resistance to flow.

Operating Conditions:

- Operating Condition A: Tank ventilation operating at a gage pressure of -0.5 kPa (-2 in. water) and constant 61 cm (24 in.) hydraulic head.
- Operating Condition B: Tank ventilation operating at a gage pressure of -0.5 kPa (-2 in. water) and constant 91 cm (36 in.) hydraulic head.
- Operating Condition C: Tank ventilation operating at a gage pressure of -2.5 kPa (-10 in. water) and constant 91 cm (36 in.) hydraulic head.

Leak rates were calculated for the postulated leak mechanisms and different operating conditions. The results are as follows:

Table C-1. Leakage Through Stress Corrosion Cracks in Liner on Tank Wall (Leak Description A, 1/4 Wall Exposed).

Hydraulic Head (cm)	Vent Pressure (kPa)	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
61	-0.5	0.258	19	35
91	-0.5	0.258	24	43
91	-2.5	0.258	21	39

Table C-2. Leakage Through Stress Corrosion Cracks in Liner on Tank Wall (Leak Description A, 1/2 Wall Exposed).

Hydraulic Head (cm)	Vent Pressure (kPa)	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
61	-0.5	0.516	38	69
91	-0.5	0.516	48	86
91	-2.5	0.516	43	77

Table C-3. Leakage Through Stress Corrosion Cracks in Liner on Tank Wall (Leak Description A, Entire Wall Exposed).

Hydraulic Head (cm)	Vent Pressure (kPa)	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
61	-0.5	1.03	77	140
91	-0.5	1.03	95	170
91	-2.5	1.03	86	150

Table C-4. Leakage Through Construction Joint from Stress Corrosion Cracks in Liner on Tank Wall (Leak Description A, 1/4 Wall Exposed).

Hydraulic Head (cm)	Vent Pressure (kPa)	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
61	-0.5	4.36	38	68
91	-0.5	4.36	47	84
91	-2.5	4.36	42	76

Table C-5. Leakage Through Construction Joint from Stress Corrosion Cracks in Liner on Tank Wall (Leak Description A, 1/2 Wall Exposed).

Hydraulic Head (cm)	Vent Pressure (kPa)	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
61	-0.5	8.72	76	140
91	-0.5	8.72	94	170
91	-2.5	8.72	84	150

Table C-6. Leakage Through Construction Joint from Stress Corrosion Cracks in Liner on Tank Wall (Leak Description A, Entire Wall Exposed).

Hydraulic Head (cm)	Vent Pressure (kPa)	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
61	-0.5	17.4	150	270
91	-0.5	17.4	190	340
91	-2.5	17.4	170	300

Flows through the cracks in the liner on the tank wall (Tables C-1 to C-3) are less than the flows through the construction joint (Tables C-4 to C-6) so that the liner cracks limit the overall leak rate for Leak Description A.

Table C-7. Leakage Through Stress Corrosion Cracks in Liner on Tank Bottom (Leak Description B).

Hydraulic Head (cm)	Vent Pressure (kPa)	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
61	-0.5	5.16	390	690
91	-0.5	5.16	480	860
91	-2.5	5.16	430	770

Table C-8. Leakage Through Concrete Encasement from Stress Corrosion Cracks in Liner on Tank Bottom (Leak Description B).

Hydraulic Head (cm)	Vent Pressure (kPa)	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
61	-0.5	27.6	460	820
91	-0.5	27.6	570	1020
91	-2.5	27.6	520	920

Flows through the cracks in the liner on the tank bottom (Table C-7) are less than the flows through the thermal cracks in the concrete encasement (Table C-8) so that the liner cracks limit the overall leak rate for Leak Description B.

Table C-9. Leakage Through 0.079-cm (1/32-in.) Diameter Hole in Liner (Leak Description C).

Hydraulic Head (cm)	Vent Pressure (kPa)	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
61	-0.5	4.95E-03	1.2	2.0
91	-0.5	4.95E-03	1.4	2.5
91	-2.5	4.95E-03	1.3	2.3

Table C-10. Leakage Through 0.64-cm (1/4-in.) Diameter Hole in Liner (Leak Description D).

Hydraulic Head (cm)	Vent Pressure (kPa)	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
61	-0.5	0.317	67	120
91	-0.5	0.317	83	150
91	-2.5	0.317	74	130

Table C-11. Leakage Through 2.5-cm (1-in.) Diameter Hole in Liner (Leak Description E).

Hydraulic Head (cm)	Vent Pressure (kPa)	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
61	-0.5	5.07	1000	1900
91	-0.5	5.07	1300	2300
91	-2.5	5.07	1200	2100

Table C-12. Leakage Through Construction Joint from Hole in Liner (Leak Descriptions C-D-E).

Hydraulic Head (cm)	Vent Pressure (kPa)	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
61	-0.5	2.18	19	34
91	-0.5	2.18	23	42
91	-2.5	2.18	21	37

Flows through the construction joint (Table C-12) are less than the flows through the 0.64-cm (1/4-in.) and 2.5-cm (1-in.) diameter holes (Tables C-10 and C-11, respectively) so that the construction joint limits the overall leak for Leak Descriptions D and E. Flows through the construction joint are greater than the flows through the 0.079-cm (1/32-in.) diameter hole (Table C-9) so that the hole limits the overall leak rate for Leak Description C.

Table C-13. Leakage Through 91-cm (36-in.) Long Crack in Concrete Encasement (Leak Description F).

Hydraulic Head (cm)	Vent Pressure (kPa)	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
61	-0.5	29.0	7500	13,000
91	-0.5	29.0	9300	17,000
91	-2.5	29.0	8400	15,000

Table C-14. Leakage Through 23-cm (9-in.) Diameter Hole in Concrete Encasement (Leak Description G).

Hydraulic Head (cm)	Vent Pressure (kPa)	Flow Area (cm ²)	Leak Rate (mL/s)	Amt Leaked in 500 hr (m ³)
61	-0.5	410	6.7	12
91	-0.5	410	10.3	18
91	-2.5	410	8.3	15

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