In-Tank Precipitation (ITP) Structures Summary Report

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DOE Contract No. DE-AC09-89SR18035

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WSRC-TR-94-341

Rev. 1

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September 20, 1994

Structural Mechanics
Westinghouse Savannah River Company
Savannah River Site
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Executive Summary

The H-Area In-Tank Precipitation (ITP) Safety Issues Resolution Program Plan requires that a comprehensive structural evaluation be conducted for the H-area tank farm ITP facility. The facility includes three 1.3 million gallon, Type III-A high level waste storage tanks 48-50, a filtration building, and four above ground transfer lines. Analysis is performed for the structures to demonstrate compliance to DOE Order 6430.1A, "General Design Criteria", DOE Order 5480.28, "Natural Phenomena Hazards Mitigation" (NPH), and UCRL 15910, "Design and Evaluation Guideline for DOE Facilities subjected to NPH". The criteria design motion for evaluation of the structures is the Blume response spectra anchored to 0.2g. The evaluations show that demands resulting from normal operating and natural phenomena hazard loads do not exceed the structural capacities when evaluated to modern criteria and the facility usage classifications.

Table of Contents

1. INTRODUCTION	5
1.1 REFERENCES	6
2. CONCLUSIONS.....	7
2.1 WASTE TANKS	7
2.2 FILTER BUILDING	7
2.3 TRANSFER LINES	8
2.4 REFERENCES	8
3. FACILITY DESCRIPTIONS	9
3.1 HIGH LEVEL WASTE TANKS	9
3.2 FILTER BUILDING	9
3.3 TRANSFER PIPING	10
4. ANALYSIS AND EVALUATION OF HIGH LEVEL WASTE TANKS.....	10
4.1 NORMAL OPERATING LOADS.....	10
4.1.1 Concrete Vaults.....	10
4.1.1.1. Inputs	10
4.1.1.2. Methodology.....	12
4.1.1.3. Results	12
4.1.1.4. Conclusions For The Vault Subject to Normal Operating Loads	12
4.1.2 Steel Tanks.....	13
4.2 DYNAMIC SEISMIC LOADING	14
4.2.1 Concrete Tank Vaults.....	14
4.2.1.1. Input.....	15
4.2.1.2. Methodology.....	15
4.2.1.3. Analyses	16
4.2.1.4. Results	16
4.2.1.5. Extrapolation of Results to Beyond EBE Events	17
4.2.1.6. Conclusions For The Vault Subject to Seismic Dynamic Loading	17
4.2.2 Steel Tanks.....	17
4.2.2.1 Input.....	18
4.3 SEISMIC LOADING - DIFFERENTIAL SETTLEMENT	19
4.3.1 Concrete Vaults.....	20
4.3.1.1. Inputs	20
4.3.1.2. Methodology and Results.....	21
4.3.1.3. Extrapolation of Results to Beyond EBE Events	24
4.3.1.4. Conclusion - Concrete Vault Settlements	24
4.3.2 Steel Tanks.....	24
4.3.2.1 Input.....	24
4.3.2.2 Methodology.....	24
4.3.2.3 Results and Conclusions	25
4.3.2.4 Extrapolation of Results to Beyond EBE - Events.....	25
4.3.3 Comparison of Allowable Settlements to Predicted Soil Settlement.....	25
4.4 TORNADO LOADS ON STRUCTURES AND EQUIPMENT	26
4.4.1 Methodology	26
4.4.2 Conclusions For The Vault Subject to Tornado Loading.....	27
4.5 REFERENCES	27

5. ANALYSIS AND EVALUATION OF FILTER BUILDING	29
5.1 REFERENCES	35
6. ANALYSIS AND EVALUATION OF TRANSFER LINES	36
6.1 REFERENCES	37
APPENDIX A - ITP STRUCTURES SUPPORTING DOCUMENTATION	38

1. Introduction

The H-Area In-Tank Precipitation (ITP) Safety Issues Resolution Program Plan [1.1] requires that a comprehensive structural evaluation be conducted for the H-area tank farm ITP facility. The facility includes three 1.3 million gallon, Type IIIA High Level Waste Storage Tanks (Numbers 48 through 50), a filtration building, and four above ground transfer lines. The ITP facility treats and separates radioactive waste salt solutions from H- and F-Area waste tanks into two waste streams for further processing.

The Safety Issues Resolution plan is in compliance with DOE Order 6430.1A, "General Design Criteria" [1.2], DOE Order 5480.28, "Natural Phenomena Hazards (NPH) Mitigation" [1.3], and UCRL 15910, "Design and Evaluation Guideline for DOE Facilities subjected to NPH" [1.4].

Tanks 48, 49, and 50 are existing high level waste storage tanks that are embedded in an above ground embankment constructed of controlled backfill. These tanks were evaluated in 1973 [1.5], and 1978 [1.6]. The In-Tank Precipitation Facility Hazards Assessment [1.7] classified Tanks 48 and 49 as High Hazard and Tank 50 as Low Hazard. However, at this time the classification for all tanks is High Hazard until further direction from DOE. Further evaluation was required to determine if the tanks meet current design criteria and to incorporate possible additional seismic response effects of site specific soil backfill and Subgrade property profiles. In addition, potential seismic induced soil settlements require evaluation.

The filter building and above ground transfer lines are new designs and are required to comply with the requirements of DOE Order 6430.1A.

The original design for the ITP buildings used NPH loads specified by the Site Specification 7096, Revision 2 [1.7] for a standard classification. A DOE Order 6430.1A compliance review was conducted for the ITP. This review identified several design concerns including the need for a Hazards Assessment Document (HAD). The HAD [1.8] specified new usage categories for the ITP buildings based on an unmitigated release of the hazardous inventory contained in various facility segments. The HAD classified the 241-96H Filter Building into two segments, the Filter Cell and the Enclosure Building. The Filter Cells were classified as Moderate Hazard and the steel Enclosure Building as Low Hazard. These revised classifications require a higher level of NPH loads than the original design loads.

Accordingly, new structural evaluations were required in order for the 241-96H Filter Building to be in compliance with the Moderate Hazard design basis. The new evaluations are based on DOE Order 6430.1A and WSRC Engineering Requirement 01060-02-R [1.9].

This report summarizes the results of structural evaluations conducted for the three high level waste storage tanks, filtration building, and four above ground transfer lines associated with the ITP facility. The evaluations are performed to assure that demands resulting from normal operating and natural phenomena hazard loads do not exceed the structural capacities when evaluated to current criteria and the revised usage classifications.

1.1 References

- 1.1 The H-area (ITP) Seismic Safety Issues Resolution Program Plan (U) HLW-ENG-9300017 Rev. 1 January 31, 1994
- 1.2 DOE Order 6430.1A, "General Design Criteria", 4-6-89
- 1.3 DOE Order 5480.28, "Natural Phenomena Hazards Mitigation", 1-15-93
- 1.4 UCRL - 15910, "Design and Evaluation Guideline for Department of Energy Facilities Subjected to Natural Phenomena Hazard", June 1990.
- 1.5 URS/John A. Blume & Associates, Engineers, "Phase II Earthquake Analyses of Underground Waste Storage Tanks 1-24 and 29-34 at the Savannah River Plant", Aiken, South Carolina, Report to E. I duPont de Nemours & Company, June 1973 (Revised June 1975).
- 1.6 URS/John A. Blume & Associates, Engineers, "Comparative Structural Analysis of Type III Underground Waste Storage Tanks", 1978.
- 1.7 "Structural Specification for Building code Requirements", Savannah River Site Specification 7096, Rev. 2, 11-30-79
- 1.8 ITP facility Hazard Assessment SRL-NPS-910052, October 7, 1991
- 1.9 Engineering Requirement # 01060-02-R, "Structural Design Criteria", 12-31-91

2. Conclusions

A summary of the analysis and results is provided in Sections 4.0, 5.0, and 6.0 for the structures associated with the high level waste storage tanks 48 - 50, Filter Building (241-96H), and the four above ground waste transfer lines. The discussion summarizes the technical bases and demonstrates that the ITP structures covered by the analyses listed in Appendix A are adequately designed to withstand the Evaluation Basis Earthquake (EBE) and other NPH loads.

2.1 Waste Tanks

The analysis of the Type IIIA waste tanks evaluated for the effects of gravity, normal operating loads, and loads due to natural phenomena hazards. The evaluation of the Type IIIA waste tanks considers effects of dynamic loads from seismic ground motions and post-seismic settlement of foundation soils under the tanks. The seismic ground motion is developed using the Blume site specific earthquake spectrum [2.1] anchored to 0.2g peak ground acceleration (PGA) shown in Figure 4.8. The allowable structural differential settlements are compared with soil differential settlements predicted using the site controlling earthquake motions. The allowable structural differential settlement is 4.7 inches and is greater than the 3-1/8 inch predicted for static and dynamic loads.

The effects of seismic events beyond the Evaluation Basis Earthquake (EBE) were estimated. These estimates were based on extrapolating the results of the EBE analysis using mean value of aged material properties. The reinforced concrete vault structural response for seismic events beyond the EBE is judged to be acceptable up to a 0.7g PGA and for differential settlements up to 23 inches. Steel tank response beyond the EBE is judged to be acceptable up to 0.4g PGA and for a differential settlement up to 12 inches.

2.2 Filter Building

The Enclosure Building steel structure and crane control room are classified as Low Hazard. This does not constitute a significant increase in the criteria above that used in the original design. However, due to its proximity over the Filter Cells, the Enclosure Building was strengthened to maintain its structural integrity during Moderate Hazard events by installing additional bracing in the longitudinal and transverse directions. In particular, the steel frame was modified to withstand the effects of the Moderate Hazard Design Basis Tornado and the EBE [2.2, 2.3, 2.4, 2.7]. The building is judged to withstand ground accelerations up to 0.6 g. This judgment is based on the margin that exists between the actual design demands and the ultimate capacity of the Enclosure Building and Filter Cells. The margin is based on the cumulative effects of the code based demand to capacity ratios, the factor of safety between the code allowable capacity and the ultimate strength of the building materials, and conservatism in the design methodologies applied.

The filtrate storage tanks located in the Filter Building, are classified as Low Hazard by the HAD. An evaluation of tank support members [2.5] for lateral seismic loads found the supports to be inadequate. The supports were strengthened for the Low Hazard seismic loads [2.6].

The Filter Cells are classified as Moderate Hazard due to the inventory contained in the ITP filters. The Moderate Hazard classification requires the application of higher loads and more rigorous analytical methods than were used in the original design in order to achieve the goals of DOE Order 6430.1A and UCRL 15910. The concrete filter cells are found to have sufficient margin to withstand the increased loads of the EBE [2.8].

The removable concrete covers of the Filter Cells are predicted to collapse if Enclosure Building steel collapses on the cell covers. As stated above the enclosure building is judged to withstand free field seismic loads up to 0.6g. For seismic ground motion greater than 0.6g, the resulting impact from debris would fall on the filters, heat exchangers and their associated piping causing potential loss of the liquid content of those components into the concrete cell. It is judged that the concrete Filter Cells would maintain confinement capability for this scenario.

2.3 Transfer Lines

Four above-ground transfer lines were evaluated for dynamic effects of seismic loads and found adequate to withstand free field ground motion PGAs up to the 0.2g. Loads from postulated differential settlements were not evaluated.

2.4 References

- 2.1 DPE-3699, "Update of Seismic Design Criteria for the Savannah River Plant", URS/Blume, May 1982.
- 2.2 Calculation #C-CLC-H-00290, Design of Filter Enclosure Building 241-96H Seismic Loads.
- 2.3 Calculation #C-CLC-H-00309, Design of the 241-96H Laboratory.
- 2.4 Calculation #C-CLC-H-00331, ITP Facility Frame Bracing Building 241-96H.
- 2.5 Calculation #C-CLC-H-00255, Evaluation of Misc. Component of Building 241-96H.
- 2.6 Calculation #C-CLC-H-00294, ITP Facility Modification of Filter Hold Tank Support System (241-96H).
- 2.7 Calculation #C-CLC-H-00249, Evaluation of Filter Enclosure Building 241-96H Seismic Loads.
- 2.8 Calculation #C-CLC-H-00292, Structural Design of Filter Cells 241-96H.

3. Facility Descriptions

As neutralized high-level waste ages in the waste tanks, it settles by gravity into a salt solution (supernate) over a layer containing solids (sludge). Decontamination of the salt solution in H-Area is accomplished by using existing waste tanks specifically designated for that purpose, by a combination of precipitation, adsorption (Tank 48) and filtration. The filtrate from this process is treated to remove volatile organics and pumped to Tank 50 until it is ready for disposal at the Z-Area Saltstone Facility. The precipitate from this process is washed, then transferred to Tank 49 to be fed to the S-Area DWPF for further processing. Transfers between Tank 48 and the Filter Building are made with four above ground transfer lines. The general layout of the facilities is shown in Figures 3.1 and 3.2.

3.1 High Level Waste Tanks

Each primary steel tank is made of two concentric steel cylinders joined to washer-shaped top and bottom plates by curved steel knuckle plates having 4 feet and 1 foot top and bottom radii, respectively (Figure 3.3). The tank steel is normalized heat treated, ASTM A-537 Class I(N) plate ranging in thickness from one half inch to five-eighths inches thick.

The primary steel tank is set on a 8-inch refractory cement insulating pad between the primary tank and the secondary liner. The pad is grooved radially so that ventilating air can flow from the inner annulus to the outer annulus.

The primary steel tank is contained in a concrete vault that has an internal steel liner. The concrete vault is 5 feet larger in diameter than the primary tank providing a 2 1/2 feet wide outer annulus. The liner is 3/8-inch thick ASTM A-516, Grade 70 steel plate. Its sidewalls rise to the full height of the primary steel tank. The nested tank and liner assembly is attached within the cylindrical reinforced concrete vault enclosure that has a 30-inch-thick wall. The vault has a reinforced concrete roof that varies in thickness from 48 inches at the 30 inch thick exterior wall to 60 inches at a 5 foot diameter center column. The vault basemat slab is 43 inches thick and has a 76 inch stepped center thickness. The vault is constructed of 3000 psi concrete reinforced with Grade 60 reinforcing steel.

3.2 Filter Building

The Filter Building consists of the two Filter Cells and the Enclosure Structure and is part of Building 241-96H located on the western side of the H-area tank farm hill (Figure 3.4).

The two filter cells in 241-96H are rectangular, reinforced concrete boxes. The filter cells have 3-1/2 feet thick cell walls and mat and are lined with a stainless steel liner. The cells are covered with 3 feet thick, unlined, removable concrete covers. The cells house two cross-flow filters that separate the radioactive precipitate from the decontaminated supernate.

The Enclosure Building is a steel braced frame structure that covers the filter cells, filtrate hold tanks, crane control room and adjacent truck access bay. The two story crane control room is a three sided reinforced concrete box with the fourth (south) side formed by the steel enclosure building wall. The foundations of 241-96H consist of concrete spread footings under column lines

1 and 2 with concrete strip footings between column lines 5 and 6. The maintenance crane is 40 feet above the floor of the building and runs between column lines 1 and 5.

The two filtrate hold tanks are vertical tanks with a total capacity of 24,000 gallons. Located inside the Enclosure Building, the tanks temporarily store the filtered solutions for analysis before eventual transfer to tank 50 or 22. The hold tank legs are supported from spread footings which are separated from the Enclosure Building foundation.

3.3 Transfer Piping

Waste is transferred between the building and tanks through lead shielded piping lines. Specific line numbers include, 705A, 1105A, 1151A and 1152A. These piping lines consist of a 6-inch diameter stainless steel core pipe and 10-inch diameter carbon steel jacket pipe. The steel pipes are enclosed in 4.25-inches of lead shielding which is covered with a 20-gauge carbon steel rain cover. The lines are supported by bearing on the tank top, and by several supports founded on the embankment independent of the tank. The piping is then supported from a structural steel frame by spring hangers where it enters the filter building.

4. Analysis and Evaluation of High Level Waste Tanks

The high level waste tanks consist of a primary steel tank and an enclosing reinforced concrete vault. The structural analysis of the high level waste tanks was performed for normal operating, seismic, and post-seismic soil settlement conditions consistent with the analysis criteria in Appendix B of the H-Area/In Tank Processing (ITP) Seismic Safety Issues Resolution Plan [4.1].

4.1 Normal Operating Loads

4.1.1 Concrete Vaults

4.1.1.1. Inputs

The reinforced concrete vaults are evaluated for normal operating loads shown in Table 4.1. The ITP process operates at a temperature less than 140°F [4.7]. A maximum operating temperature of 136°F was obtained from temperature records [4.8]. Therefore 150°F used for the thermal analysis[4.9]. This temperature is less than the 400°F that is specified in the waste farm technical standards [4.5] and construction drawings [4.6].

Table 4.1 Waste Tanks 48 to 50, Vault Normal Operating Loads

Tank Top Live Load Allowance (Including tank top equipment)	300 psf
Maximum fill limits	372" (31')
Fluid density	93.6 pcf
Negative tank pressure	0.2 psig
Negative annulus pressure	2.0 psig
Maximum temperature of the concrete vault	150°F
Thermal gradients for the concrete vault [4.6]	
Basemat	100°F
Cyl. wall - lower 2'	100°F
Cyl. wall - remainder	30°F
Top slab	50°F

The normal operating load combinations for the vault are summarized in Table 4.2. The combinations, defined in the Structural Mechanics Acceptance Criteria [4.10] are the same as the ACI 349-85 [4.2] load combinations.

Table 4.2 Vault Normal Operation Load Combinations

Load Case	Load Mult. Factor	Dead	Live	Fluid	Lateral Earth	Temp	Eqpt React.	Neg. Vent. Pressure
		D	L	F	H	To	Ro	Po
1	1.0	1.4	1.7				1.7	1.4
2	1.0	1.4	1.7	1.4			1.7	1.4
3	1.0	1.4	1.7	1.4	1.7		1.7	1.4
4	0.75	1.4	1.7			1.4	1.7	1.4
5	0.75	1.4	1.7	1.4		1.4	1.7	1.4
6	0.75	1.4	1.7	1.4	1.7	1.4	1.7	1.4

- where:
- D Dead load consisting of the dead weights of the vault, tank, secondary liner and refractory insulation.
 - F Hydrostatic fluid loading of the tank contents.
 - H At rest lateral soil pressure.
 - To Thermal loading.
 - L Live load acting on the tank top.
 - Ro Equipment reactions acting on the tank top (included with the live load).
 - Po Steel tank vacuum and annulus pressure loads (included with the dead load).

Where the structural effects of differential settlement, creep, or shrinkage may be significant, they were included with the dead load in load combinations 4, 5, and 6. Where D, H, or L reduce the effects of other loads, the corresponding coefficients were taken as 0.9 for D and H and zero for L.

4.1.1.2. Methodology

An elastic axisymmetric finite element model of the reinforced concrete vault is used to determine the structural demands of the basemat, wall, vault top and center column for the normal operation load cases. The vault was represented by second order axisymmetric shell elements and the column capital was modeled with second order axisymmetric solid elements (Figure 4.1). Reduction in bending stiffness due to cracking is incorporated by reducing the elastic modulus for selected elements where cracking moments are exceeded. The supporting soil was represented by Winkler springs. Subgrade moduli of 15 pci and 30 pci were used to evaluate the effects of soil stiffness on the structure. The general purpose finite element program, ABAQUS, Version 5.2N [4.11] was used to conduct the analyses.

4.1.1.3. Results

The structural demands in the basemat, wall, tank top and center column are generally well within ACI 349-85 allowable capacities for the normal operating loads and load combinations. Selected demand-capacity curves for critical areas having the lowest demand to capacity are discussed below.

Figure 4.2 shows that the maximum demand to capacity ratio for the vault top subject to dead, live, fluid and lateral soil loads occurs at 18 feet from the tank center. The demand to capacity ratio at this location is 0.77.

Figure 4.3 shows the demand and capacity for the basemat shear subject to dead, live, fluid and lateral soil loads. At $d/2$ from the face of the column, the demand to capacity ratio is 0.94.

Figure 4.4a shows the demand and capacity for radial basemat bending moments subject to dead, live, fluid, lateral soil and thermal loads. These moments represent load redistribution due to the cracking of concrete. The demand to capacity ratio is 0.90.

Figure 4.4b shows that the circumferential moment demand in the basemat is slightly larger than the capacity at a radius of 23 feet with a demand to capacity ratio of 1.15. The basemat thermal bending moment is primarily due to the 100°F gradient, assumed in the original type III waste tank design and implemented in subsequent tank designs [4.6]. Thermal gradients were considered as self relieving secondary stresses in the original design.

Figure 4.5 shows the demand and capacity for the tank top radial bending moments subject to dead, live, fluid, lateral soil and thermal loads. At an 18' foot radius, the demand and capacity are approximately equal. Allowing additional cracking in the tank top would cause relaxation of thermal bending moments.

4.1.1.4. Conclusions For The Vault Subject to Normal Operating Loads

The concrete vault structural demands are within ACI 349-85 capacities for normal operating loads and load combinations consisting of dead and live load, fluid, lateral earth pressures, equipment vibrations and negative ventilation pressure. When thermal loads are included without

considering moment redistribution from cracked section the demand to capacity ratio is 1.15. However, secondary stresses such as thermal cause cracking and load redistribution in the basemat and tank top without degrading the ultimate capacity of the vault.

4.1.2 Steel Tanks

4.1.2.1 Inputs

The steel tanks are evaluated for normal operating conditions that include loading from gravity, hydrostatic pressure, tank annulus pressure, and thermal. Table 4.3 summarizes these loads.

Table 4.3 Waste Tanks 48 to 50
Steel Tank Normal Operating Loads

Tank dead load	0.286 lb./in ³
Tank annulus pressure	2 psi
Fluid density	1.52 sg (94.8 pcf)
Tank fill height	372 in (31 ft)
Thermal (temperature difference, tank to vault)	70°F
Equipment (cooling coil) loads	Not significant

The following normal load condition is checked for conformance to the ASME Code, Section III, Division 1, Subsection NC-3200, criteria for normal conditions:

$$D + F + T_o + R_o$$

where:

D	=	Dead load, including annulus pressure
F	=	Hydrostatic load
T _o	=	Thermal load
R _o	=	Equipment load

4.1.2.2 Methodology

The steel tank is analyzed with a three-dimensional finite element model as illustrated in Figure 4.a. A full 360-degree model is developed and is used for both the axisymmetric normal loads and the non-uniform hydrodynamic (seismic) loads. The model represents the tank top, knuckles, side, bottom, center column, and column capital. Knuckle regions, being regions of high stress gradient, are adequately refined. The tank bottom is restrained to represent frictional resistance of the base support pad.

Supplementary calculations are performed to address shell local buckling, tank sliding, anchor studs, and cooling coil brackets.

4.1.2.3 Results and Conclusions

The results of the analysis are summarized in Table 4.4.

Table 4.4 Steel Tank Margin to Code

Location	Stress Classification	Margin to Code*
Bottom Section of Outside Wall (7/8")	Primary Membrane	62%
Bottom Outside Knuckle	Primary Membrane & Bending	117%
Bottom Inside Knuckle	Primary & Secondary	7%
Bottom Inside Knuckle	Buckling	28%

*Margin to Code is defined as $100 \left(\frac{\text{Allowable}}{\text{Applied}} - 1 \right) \%$

The results of this analysis show that the tank meets the criteria for normal operation. The secondary thermal stresses are within allowable capacities and shake down to the elastic range and, therefore, do not combine with the seismic stresses of the EBE.

4.2 Dynamic Seismic Loading

4.2.1 Concrete Tank Vaults

The concrete tank vaults were evaluated for horizontal and vertical seismic dynamic effects corresponding to the site EBE. Soil structure interaction (SSI) effects are considered.

The horizontal analyses use strain dependent soil stiffness and damping properties for the embankment fill and subgrade derived from site test data. The SASSI [4.15] computer program was used to conduct the SSI analyses using three-dimensional models of the tank and soil. SSI evaluations consider:

- Soil stiffness uncertainties by using upper bound (UB), best estimate (BE) and lower bound (LB) properties consistent with the provisions of ASCE 4-86.
- Effects of embedment using full, partial and no embedment conditions.
- Effects of fluid contents. Full and empty conditions are considered. The fluid is modeled with dynamic properties to allow prediction of the fluid response.
- Tank-soil-tank interaction.

The vertical seismic evaluation is a linear response spectrum analysis using the Blume spectra.

Some tank supported structures are not designed to withstand seismic dynamic effects. The potential for damage from postulated failure of these structures is discussed in Section 4.4.

4.2.1.1. Input

The seismic criteria motion is the Blume response spectra [4.3] anchored to 0.2g pga. The motion is defined at the surface of the free field at Elevation 298 feet. The comparison of the Blume spectra enveloping the site controlling spectra [4.14] at the frequencies important to the concrete vault and the liquid filled steel tank is shown in Figure 4.7. The SSI analyses were done in accordance with "Structural Mechanics Acceptance Criteria for High Level Waste Tanks" [4.10].

The reinforced concrete vault is analyzed considering the seismic load combination shown in Table 4.5.

Table 4.5 Vault Seismic Load Combination

Load Mult. Factor	Dead	Live	Fluid	Lateral Earth Press.	Seismic	Temp	Equip. React.	Tank Pressure
	D	L	F	H	E	To	Ro	Po
1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

where: D Dead load consisting of the dead weights of the vault, tank, secondary liner and refractory insulation.
F Hydrostatic fluid loading of the tank contents.
H At rest lateral soil pressure.
E Seismic loading
To Thermal loading.
L Live load acting on the tank top.
Ro Equipment reactions acting on the tank top (included with the live load).
Po Steel tank vacuum and annulus pressure loads (included with the dead load).

4.2.1.2. Methodology

The soil structure interaction (SSI) evaluations use the computer code SASSI [4.15]. SASSI requires equivalent linear elastic soil properties representative of the soil strain levels expected for the DBE. The computer codes SHAKE [4.16] and FLUSH [4.17] are used to develop the best estimate, upper bound and lower bound equivalent linear elastic strain dependent shear moduli and damping.

The SSI considerations include the effects of embedment, embankment, base fixity, fluid contents, and tank-soil-tank interaction. Results from the analysis consist of maximum accelerations of the concrete vault, maximum acceleration of the fluid mass, and horizontal shear at the vault base.

The concrete vault is modeled as a three dimensional finite element representation composed of shell elements for the base mat, vault top, vault walls, and beam elements for the column. The fluid response is modeled using a spring-mass system with characteristics consistent with fluid responses calculated using the Seismic Design and Evaluation Guidelines for the Department of Energy High Level Waste Tanks [4.18]. The SSI finite element model is shown in Figure 4.8.

The axisymmetric finite element ABAQUS vault model used for the normal operating load analyses is also used for the vertical response spectrum analysis. The ASCE 4-86 lateral soil load shown in Figure 4.9 is included with the asymmetric loads and displacements [4.25].

4.2.1.3. Analyses

The SSI analyses are conducted to determine the tank response to vertically propagating, horizontal shear waves.

These analysis include:

- a) A fixed base analysis of the tank structure neglecting the embankment, fluid, and soil structure interaction effects.
- b) Five analysis that include the effects of soil-structure interaction effects and tank-soil-tank interaction (Figure 4.10)
- b) Five analyses that consider SSI, fluid structure interaction, and tank-soil-tank interaction (Figure 4.11)
- c) Four analyses that consider variability of soil properties and embankment size (Figure-4.12).

The resultant maximum vault and fluid accelerations are shown on Table 4.6.

Vertical response of the concrete vault is evaluated using a response spectrum analysis of the vault with the vertical criteria motion at the base elevation of the tank. Analysis using the lateral loading based on the pressure profile from ASCE 4-86 develops conservative wall shears and moments using an equivalent static approach that are conservative when compared to SASSI results.

4.2.1.4. Results

The results of the concrete vault SSI analyses described in Section 4.2.1.1 to 4.2.1.3 are summarized in Table 4.6.

Base shears were typically in the range of 3500 to 4500 kips for the SSI evaluations. These base shears result in shear stresses less than 100 psi and are less than ACI 349-85 allowable for in plane shear.

The maximum wall bending stresses were obtained from an analysis with the ASCE 4-86 lateral soil loads. The maximum wall bending moment is 51 ft-kip/ft. These forces, when combined with the operating loads are less than those allowed by ACI 349-85, as shown in Figure 4.13.

Table 4.6 Summary of Vault SSI Responses

Tank Condition	Response	Minimum (g)	Maximum (g)
Full Tank	Top Slab Acceleration	0.23	0.26
"	Base Slab Acceleration	0.20	0.25
	Fluid Acceleration	0.26	0.48
Empty Tank	Top Slab Acceleration	0.22	0.24
	Base Slab Acceleration	0.20	0.23
Tank to Tank Interaction	Top Slab Acceleration	0.22	0.24
	Base Slab Acceleration	0.20	0.23
	Fluid Acceleration	0.22	0.29
Fixed Base (No SSI)	Top Slab Acceleration		0.30
	Base Slab Acceleration		0.20

The vault top stresses were obtained from the vertical response spectrum analyses. The natural frequency of the cracked tank top is approximately 8 Hz. Uncertainty in structural stiffness is accounted for by reducing the natural frequency by 15%. A response spectra analysis of the softened structure is performed with 10% critical damping. The maximum tank top bending moment is 76 ft-kip/ft. The combined seismic and operating loads are less than the ACI 349-85 allowable capacities, as shown in Figure 4.14.

4.2.1.5. Extrapolation of Results to Beyond EBE Events

Vault demands from the seismic load case are well within the ACI 349-85 allowable capacities. The highest demand to capacity ratios in the vault obtained from the SASSI analyses are less than one quarter of the vault wall capacity.

The reinforced concrete vault structural capacity is judged to be able to withstand the beyond EBE pga of 0.7g. For this evaluation, increased in-situ concrete strength capacity is used when extrapolating the EBE seismic demands. The critical failure modes at the 0.7 g pga level would be extensive yield deformations or collapse of the top slab and extensive shear cracking of the vault cylinder wall.

4.2.1.6. Conclusions For The Vault Subject to Seismic Dynamic Loading

The seismic loads on the concrete vault are less than the ACI 349-85 allowable capacities. The maximum demand is less than that computed for load combination 3, (1.4D + 1.7L + 1.7H + 1.4F), discussed in Section 4.1.1. Extrapolation of available capacity indicates that the concrete vault can withstand beyond EBE events up to 0.7g pga.

4.2.2 Steel Tanks

4.2.2.1 Input

The steel tanks are evaluated for the horizontal and vertical seismic dynamic effects corresponding to the site EBE. The dominant seismic loading on the tank is produced by the hydrodynamic response of the tank contents. The hydrodynamic pressures on the tank wall are calculated from the methodology developed by the DOE TSEP [4.18].

The response frequencies of the horizontal impulsive mass, horizontal convective mass and the vertical fluid mass are calculated. The seismic accelerations corresponding to these frequencies are obtained from the Blume response spectra [4.3]. These accelerations are used to calculate hydrodynamic pressures.

Using the acceleration values from the response spectrum and the TSEP methodology, the hydrodynamic pressure distributions are calculated for the following conditions:

- Horizontal Convective Mode
- Horizontal Impulsive Mode
- Tank Top Contact - Constrained Fluid Response
- Tank Top Contact - Fluid Impact
- Vertical Response Mode

Figure 4.15 illustrates the hydrodynamic pressures on the tank wall. The hydrodynamic pressure distributions and the dead load, negative pressure and hydrostatic pressure are combined and converted into finite element input values.

4.2.2.2 Methodology

The calculated hydrodynamic fluid pressures are applied to the same three-dimensional finite element model as is used for the normal operating conditions.

The following load condition is checked for conformance to the ASME, NC-3200, criteria for faulted conditions:

$$D + F + R_o + E_{ss}$$

where: E_{ss} = Seismic loading

The acceptance criteria is the level D stress limits of ASME, Section III, Division 1, Appendix F-1331.

4.2.2.3 Tank Top Contact Analysis

Since the tank does not have adequate freeboard height to allow unconstrained sloshing (convective mode), additional forces are produced when the convective fluid contacts the tank top. The additional forces are produced by two different conditions, fluid impact and the inertial effects of the constrained convective fluid.

An additional pressure (Δ -impulsive pressure) is generated during the time when the wave is in contact with the tank top. Since the convective fluid is constrained by the tank top and wall, the fluid responds with the tank as an impulsive mass. Thus Δ -impulsive pressure is produced by the constrained convective fluid mass as it responds at the frequency and acceleration of the impulsive mode.

When the sloshing fluid impacts the tank top, the fluid is not totally constrained due to the free surface in the direction of the tank center. The analysis uses the free surface as a flow path; loads are developed and dynamic amplification is applied.

4.2.2.4 Results and Conclusions

The results of the seismic analysis show that the limiting condition is produced by the combined hydrodynamic loads. The maximum stress component is hoop stress in the top 17 feet of the wall, over a 100 degree sector. Table 4.7 summarizes the analysis results. Allowable stresses are based on ASME Code minimum material properties.

Table 4.7 Steel Tank Margin to Code for Dynamic Load

Location	Stress Classification	Margin to Code*
Upper Section of Outside Wall (1/2")	Primary Membrane	73%
Overall Tank	Buckling	110%

*Margin to Code is defined as $100 \left(\frac{\text{Allowable}}{\text{Applied}} - 1 \right) \%$

The results of this analysis show that the tank meets the criteria for the EBE condition.

4.2.2.5 Extrapolation of Results to Beyond - EBE Events

Extrapolated results using median material properties show that the steel tank can withstand acceleration up to 0.4g. At some point above this PGA level, the steel tank wall will likely rupture with a vertical crack in the 1/2 inch thick section at the top 17 feet of the tank propagating into the thicker material in the lower section. These high stress levels occur over a 100 degree sector of the steel tank and would release the contents to the vault annulus.

4.3 Seismic Loading - Differential Settlement

The predicted magnitude of the soil differential settlement and the shape of the settlement were developed independent of this analysis [4.1]. The objective of this analysis is to determine the allowable structural settlement for the postulated settlement profiles. These allowable structural differential settlements are compared with the predicted soil settlement profiles and show that the tank vault system will maintain confinement for this event.

For the purpose of calculating the allowable differential settlement, two soil settlement profiles are assumed. The first is the center settlement profile as shown in Figure 4.16. This profile has a diameter, D_s , that can vary up to the 95 feet diameter of the vault. For settled region diameters larger than vault, the differential displacement, D , is the maximum differential displacement across a 95 foot diameter circle.

The second is the edge settlement profile is shown in Figure 4.17. In this mechanism, the support is removed from the bottom of the vault over a distance, l . The differential settlement is measured under the edge of the vault, as shown in Figure 4.15. The soil under the vault has an eccentric loading that causes the vault to undergo rigid body rotation. For small widths of the settled region, the vault will cantilever over the settled region. For widths of the settled region approaching $1/2$ of the vault diameter, and greater, the vault will rotate as a rigid body and come into contact with the soil in the settled region.

4.3.1 Concrete Vaults

4.3.1.1. Inputs

The operating loading combinations for the vault are defined in the Structural Mechanics Acceptance Criteria [4.10] which uses the ACI 349-85 load combinations, as summarized in Table 4.8.

Table 4.8 Vault Normal Operation Load Combinations

Load Case	Load Mult. Factor	Dead	Live	Fluid	Lateral Earth	Temp	Eqpt React.	Neg. Vent. Pressure	Seismic Differential Settlement
		D	L	F	H	To	Ro	ΔP	Ess
8	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

where

- D Dead load, which consist of the dead weight of the vault, tank, secondary liner and refractory.
- F Hydrostatic fluid loading of the tank contents.
- H At rest (ko) lateral soil acting on the tank.
- To Thermal loading.
- L Live load acting on the tank top.
- Ro Equipment reactions acting on the tank top.
- ΔP Vacuum acting in the steel tank, and in the annulus due to ventilation.
- Ess Seismically induced settlement loading.

The magnitude of these applied loads is defined in Section 4.1.1.

Seismically induced differential settlement loads are expected to result in plastic deformation of the vault that relieve the secondary thermal stresses. Thus, thermal loads are not considered in the seismically induced settlement analysis. At the ITP maximum operating temperature of 150°F, effects of thermal degradation are negligible and virgin material properties are used.

The lateral earth pressure is based on the at-rest condition using $K_o = 1 - \sin f$, with a minimum internal friction angle, f , for the soil. Lateral soil pressures are resisted primarily by hoop compression in the cylindrical vault wall, and are not considered in this differential settlement analysis.

The acceptance criteria for the settlement analysis is defined in the Structural Mechanics Acceptance Criteria (SMAC) for the High Level Waste Tanks [4.10]. Key provisions from the acceptance criteria are summarized below:

	Primary Criteria	Alternates / Exceptions
Vault Structural Capacity	ACI 349-85	A high confidence low probability of failure (HCLPF) shear strength may be used in lieu of the ACI 349 provisions for low rise walls.
Vault Allowable Deformation	ACI 349-85 Appendix C Rotation Limits	The allowable deformation of a flexural-shear hinge may be determined by comparison to test data, provided the resulting rotation is less than the ACI 349-85 Appendix C rotation limits which corresponds to the onset of very severe cracking but not collapse.

4.3.1.2. Methodology and Results

The methodology used in this analysis and the results are presented below for the center and edge settlement profiles.

Center Settlement Profile

The center settlement profile is shown schematically in Figure 4.16. In this mechanism, the support is removed from the bottom of the vault over a diameter, D_s , and the depth of the settled region is denoted Δ .

For small diameters of center settlement, a partial plastic hinge may form in the basemat and the basemat will be capable of spanning the settled area, as shown in Figure 4.18. The allowable differential settlement for this profile is not limited. Yield line theory is used to determine the spanning capacity of the basemat.

As the diameter of the settled region gets larger, a pair of radial hinges form in the basemat and the center of the basemat settles as shown in Figure 4.17. Circumferential hinges also form in the basemat as shown in Figure 4.18. The allowable settlement for this mechanism is the product of the ACI 349-85 Appendix C hinge rotation and the distance between hinges. Basemat hinge rotations limit the allowable differential settlement for the center settlement profile, and are shown in Table 4.9. As the diameter of the settled region increases, the distance between hinges increases and the allowable settlement increases.

Table 4.9 Differential Settlement Corresponding to the
Onset of Very Severe Cracking Using Lower Bound Material Capacities
Center Settlement Profile

Diameter of Settled Region	1.5 Specific Gravity	
	Full Tank (31' @ 1.5 sg)	Half Full Tank (15.5' @ 1.5 sg)
< 50'	unlimited	unlimited
50'	unlimited	unlimited
60'	5.1"	6.5"
70'	7.3"	7.3"
80'	9.5"	9.5"
90'	11.2"	11.2"

The ACI 349-85 hinge rotation limits are 68% of the average rotation corresponding to the ultimate capacity for a series of tests performed by Mattock [4.19]. In Mattock's test, the ultimate capacity was limited by the cracking, surface spalling and eventually crushing of the concrete on the compression side of the beam. For a displacement controlled loading, such as settlement of the basemat, rotation beyond the ultimate load does not cause collapse. Thus, the ACI rotation limits, at 68% of the ultimate load capacity, corresponds to the onset of very severe cracking, not collapse of the waste tank.

The settlements shown in the table neglect the additional displacement that would occur from the increased unit bearing pressure on the soil remaining in contact with the basemat.

When the capacity of the basemat mechanism is reached, the center of the tank settles, and a second set of radial and circumferential hinge mechanisms form on the vault top. The bending capacity at an 18 foot radius is less than at the edge of the column capital, due to the reinforcing layout, and a yield line forms at this radius. The vault top relies on the column capital for support and the interior column remains in compression. The allowable settlement for the vault top mechanism is the product of the ACI 349-85 Appendix C hinge rotation and the distance between hinges. Unlike the basemat, structural collapse could occur when the ACI 349 rotation limits for the vault top are exceeded. This corresponds to a vault top differential settlement of 23 inches.

Edge Settlement Profile

The edge settlement profile is shown schematically in Figure 4.21. In this mechanism, the support is removed from the bottom of the vault over a distance, 'l'. The distance from the center of the tank to the edge of the settled region, is denoted 'b' in Figure 4.21. The projection of the edge of the settled region on the bottom of the vault is assumed to be a straight line as shown in the figure.

The soil under a vault with an edge settlement profile has an eccentric loading due the portion of the vault cantilevered over the settled region. This eccentric loading, combined with a compressive load over a reduced bearing area will cause the vault to rotate into the settled region, as shown in Figure 4.22. Use of the lower bound subgrade modulus (15 pci) would double the rigid body rotation resulting in a larger allowable rotation. The allowable displacement for the edge

settlement profile, is the summation of the displacements due to this rigid body rotation and the structural displacement of the vault. Rigid body rotation is inversely proportional to the subgrade modulus. Thus, the rigid body rotation was conservatively calculated with an upper bound soil subgrade modulus of 30 pci (51 kcf).

The structural capacity of the vault with an edge settlement profile is limited by either (1) the capacity of the basemat; or (2) the flexural - shear capacity of the entire vault cross section. The allowable displacement for each of these two mechanisms is calculated separately. The minimum structural displacement is added to the rigid body rotation displacement and is the total allowable displacement for the edge settlement profile.

The basemat will act as an edge supported, flat circular plate which spans over the settled region between the tank wall and the remaining soil support. Loads acting on the basemat include the dead weight of the basemat and the fluid loading from the tank contents. The basemat has the potential for both bending and shear failure modes. For the bending mode, shown in Figure 4.21, a series of plastic hinges form in the basemat and if the basemat's load carrying capacity is exceeded, the mat deforms to meet the soil settlement profile in a displacement controlled loading. The load capacity of the unsupported span was determined by yield line analysis. The allowable settlement that the mat can tolerate is limited by the ductility of the reinforced concrete. ACI 349 rotation limits are used for the reinforced concrete ductility acceptance criteria. Allowable settlements for the basemat bending mechanism, Δ in Figure 4.23, are greater than 16 inches.

In the preceding discussion, it is assumed that the basemat is supported by the cylinder wall. The shear loads from the basemat are applied to the wall, along with the dead weight of the wall and vault top, in addition to live loads. The unsupported vault walls then act as a cantilever beam, transferring these loads back to the portion of the vault that remains supported directly on the soil.

Shear forces are resisted by the vault wall which acts as a horizontal shear wall. Bending forces are resisted by the wall's in-plane strength along with portions of the basemat and vault top, which act as flange elements. If the vault cantilever capacity is exceeded, then shear-bending cracks form in the wall. The vault then deforms plastically and follows the ground settlement, as shown in Figure 4.24.

The allowable flexural-shear displacement of the vault wall limits the total differential settlement of the vault for the edge settlement profile. If the tank cannot cantilever over the depression, then diagonal shear cracks form in the tank wall and the cantilevered portion of the tank settles an amount equal to the depth of the depression, less the displacements due to rigid body rotation. Summing the structural displacements with the displacements due to rigid body rotation, yield the allowable differential settlement for the edge settlement profile as shown in Table 4.10. This data limits the wall rotation to less than 0.006 radians for the edge settlement profile.

The wall flexural shear deformation limits are based on the lower bound of test data corresponding to the ultimate capacity of shear walls. For a displacement controlled loading, such as the vault edge settlement profile, deformation beyond the ultimate load does not cause collapse. Thus, the displacement limits in the Table 4.10 correspond to the onset of very severe cracking, not collapse of the waste tank.

**Table 4.10 Differential Settlement Corresponding to the
Onset of Severe Cracking Using Lower Bound Material Capacities
Edge Settlement Profile**

Diameter of Settled Region	1.5 Specific Gravity	
	Full Tank (31' @ 1.5 sg)	Half Full Tank (15.5' @ 1.5 sg)
< 30'	unlimited	unlimited
30' - 40'	18"	> 24"
> 40'	> 24"	> 24"

4.3.1.3. Extrapolation of Results to Beyond EBE Events

Exceeding the ACI 349-85 rotation limits in the basemat will cause cracking, spalling and possibly crushing of the reinforced concrete, but do not correspond to collapse of the vault. Exceeding of the wall's flexural-shear ductility limit will cause severe cracking in the wall, spalling of concrete from the face of the wall, and possibly crushing of concrete in the wall itself, but does not correspond to collapse of the vault.

Structural collapse could occur when the ACI 349 rotation limits for the tank top are exceeded, which corresponds to a tank top differential settlement of 23 inches. Using median rotation limit data instead of the ACI 349 rotation limit would indicate collapse of the tank top at differential settlements beyond 34 inches.

4.3.1.4. Conclusion - Concrete Vault Settlements

For the ITP project, with a maximum tank specific gravity of 1.5, the allowable differential settlements are 5.1 inches for the center settlement. Differential settlement beyond this limit will cause cracking and spalling of the reinforced concrete. Collapse of the tank top does not occur for settlements less than 23 inches.

4.3.2 Steel Tanks

4.3.2.1 Input

The steel tanks are evaluated for postulated differential settlement profiles [4.13]. The loading conditions included in the soil differential settlement analysis are gravity, hydrostatic pressure, tank annulus pressure and soil settlement profiles. The steel tanks acceptance criteria is the ASME Code, Section III, Division 1, Appendix F-1331 for stress values and ASME Code Case N-284 for compressive (buckling) stress.

4.3.2.2 Methodology

The analysis of the steel tank for the soil settlement profiles is performed using finite element analysis (FEA) techniques and hand calculations.

A three-dimensional finite element model is used to evaluate a deflected tank shape conforming to the non-symmetrical deflected concrete profiles resulting from the soil settlement. This model is a modification of the tank model used to analyze normal conditions, Figure 4.a. The model is modified by refining the element size in the tank base to address geometric discontinuities of the base slab profile. The analysis is conducted using linear elastic large displacement theory. Hydrostatic pressure of a full tank with contents at the maximum specific gravity is applied to force the tank base to conform with the deflected vault basemat profiles. The center settlement and edge settlements using vault wall failure are analyzed with FEA methods. The edge settlement without wall failure is analyzed using hand calculations.

4.2.2.3 Results and Conclusions

The results of the soil differential settlement analysis show that the limiting condition is produced by the edge soil differential settlement with vault wall failure. The governing stress components are meridional compression and shear and occur in the 7/8" thick lower section of the tank wall. The allowable differential settlement for this condition, based on ASME code material minimum properties and Appendix F criteria, is 4.7".

4.2.2.4 Extrapolation of Results to Beyond EBE - Events

The analysis results are extrapolated to differential settlement values at or above those expected for an EBE. The 4.7" differential settlement for the EBE is based on the buckling stability limits of ASME code case N-284. Overall buckling stability analysis shows that local buckling stability is significantly under predicted by Code Case N-284, and buckling was not limiting. On this basis, the differential settlement limit for beyond design basis event is evaluated using ASME Code stress limits. The limiting condition for soil differential settlement is edge settlement with vault wall failure. The governing stress is hoop stress in the 7/8" thick lower section of the steel tank wall. The extrapolated results show that the steel tank can withstand 12" of differential soil settlement. At settlements above this value, the steel tank wall may likely rupture with a vertical crack, releasing the contents to the concrete vault annulus.

4.3.3 Comparison of Allowable Settlements to Predicted Soil Settlement

The maximum allowable differential settlement as limited by the structural capacity of the Tank Vault system is 4.7 inches. The differential settlements predicted by site Geotechnical Services are reported in Reference 4.20. These differential settlements include components from collapse of subsurface materials, dynamic settlement, settlement from load variation, and long term effects.

The differential settlement from collapse of subsurface materials is two inches. The total settlement from the remaining three components is 2 1/4 inches. Reference 4.20 suggests that the differential settlement is half of the total settlement or 1 1/8 inches for these three components. Therefore the total differential soil settlement is 3 1/8 inches and includes the sum of the settlement from the collapse of the subsurface material, the differential settlement from dynamic settlement, load variations and long term effects. Comparing this with the allowable structural differential settlement shows that the vault-tank system will maintain confinement for differential settlement from the EBE and long term settlement effects.

4.4 Tornado Loads on Structures and Equipment

Tornado loads affect the exposed vault structure, the equipment and supporting structures (SSC's) located on and adjacent to the tank tops. Analysis was done for the effects of collapse of structures and equipment on the vault, the effects of tornado missiles and the effects of atmospheric pressure drops. The concrete vault meets design criteria specified in UCRL 15910 for tornado generated missiles and atmospheric pressure drop.

Tank tops systems, structures and components installed are designed for normal wind loads which are less than the tornado wind and missile loads required for the waste tanks. Therefore, the tank analysis assumes that the tank top structures and equipment can fall onto the tank top during the design basis tornado. Local and global effects of the resulting impacts are evaluated for potential damage to the tank top. Results of this analysis show that impact of the structures and equipment located on tanks 48 to 50 would not result in local or global failure of the tank top.

Since failure of the structures and equipment supported on the tank top is presumed and the consequences are acceptable, the conclusions from the impact evaluation are applicable to structure and equipment impacts initiated by seismic or operating loading conditions.

Tornado generated loads as specified by DOE Order 6430.A and UCRL 15910 consist of (1) a 15 lb., 2x4 timber plank with a horizontal velocity of 150 mph or a vertical velocity of 100 mph; (2) a 75 lb., 3 inch diameter standard steel pipe with a horizontal velocity of 75 mph or a vertical velocity of 50 mph; and (3) a 3000 lb. automobile which is rolling and tumbling at 25 mph. The tornado pressure drop is 125 psf at the rate of 50 psf/sec.

4.4.1 Methodology

UCRL 15910 criteria requires a concrete thickness greater than 12 in. to resist the tornado missiles. The exposed tank tops vary in thickness between 4 ft. and 5 ft. which exceeds that required to resist tornado generated missiles and are not analyzed.

The 125 psf tornado pressure drop is less than the weight of the roof of 600 psf and therefore does not result in net uplift forces. Thus, the structure can withstand the pressure demands placed on it due to tornado conditions.

The roof supported structures and equipment are presumed to fail under tornado loads. The resulting impacts to the tank top are evaluated. Both local and global effects of impact are considered. Global effects consider the overall response of the structure that may lead to collapse. Provisions for global effects of impact on the tank structure are in accordance with Appendix C of ACI 349-85. Local effects consist of scabbing, penetration and perforation, and are evaluated with the modified NDRC formulas [4.21]. Tank top equipment is conservatively represented as a solid steel cylinder using a length equal to twice the diameter.

4.4.2 Conclusions For The Vault Subject to Tornado Loading

Postulated failure of tank top-supported equipment and structures was evaluated for Tanks 48 through 51 and shows that, for the weight and heights of the equipment, no penetration or collapse of the top occurs.

Access to risers was evaluated based on SQUG experience and walkdown of buildings for beyond EBE events up to 0.25g PGA for Tanks 48 and 49 [4.22]. Equipment located on the tanks and adjacent structures and structures located on the tank tops were evaluated. No interactions are expected that would preclude access to tank top risers for alignment and startup of the Emergency Purge Ventilation Exhausters (EPVEs). No evaluation has been performed for access to risers on Tanks 50 and 51; however, access would be similar to that for Tanks 48 and 49 based on similarity of equipment and structures located on Tanks 50 and 51.

4.5 References

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5. Analysis and Evaluation of Filter Building

The Filter Building design methods are consistent with the requirements of DOE Order 6430.1A, UCRL 15910 and SRS Engineering Requirement 01060-02-R. The design basis documents and load combinations used for the earthquake and wind design are summarized in the attached Tables 5.1, 5.2, 5.3 and 5.4. The NPH loads used are summarized in the attached Table 5.5 for the appropriate usage categories of the structure segments.

The concrete cell portion is classified as Moderate Hazard which requires a 0.11g PGA using a dynamic analysis methodology. A pseudo-dynamic methodology was used to evaluate the concrete cells as follows:

- 1) The peak from the USNRC Regulatory Guide (RG) 1.60, 5% damped curve was scaled to a PGA of 0.15g to judgmentally account for amplification of the hill and
- 2) The peak of the RG 1.60 spectrum was multiplied by 1.5 and this acceleration used as input for the pseudo-dynamic analysis of the concrete cells to account for multi-mode participation.

The resulting calculations [5.1, 5.2] show the concrete cells to be adequate with low demand to capacity ratios based on the ACI 349 code criteria. Thus, the Filter Cells were found to be adequate for the Moderate Hazard loads.

In lieu of a formal crane drop analysis for impact on the cell covers, experience with the New Waste Transfer Facility was used, which is of similar design and construction. This comparison showed that the ITP cell covers would not be adequate to withstand a crane drop. It was therefore determined that the crane be parked over the truck bay away from the concrete cells during tornado warnings and when not in use, thus minimizing the risk to the filter cells and their contents.

Since the steel superstructure has the potential of impacting and damaging the Moderate Hazard cells, the existing structure was evaluated for Moderate Hazard tornado and earthquake loads [5.3 through 5.6]. For this evaluation, the tornado wind loads used were increased to account for the added exposure of the 30 foot high tank farm hill. The building seismic analysis was by static equivalent using 1.5 times the maximum acceleration of a 5% damped USNRC RG 1.60 spectrum scaled to a 0.11g PGA and multiplied by 1.5 for multi-mode participation. The tornado wind loads governed the design in the East-West direction and the earthquake loads governed in the North-South direction. Using the above loads it was found that the existing steel braced frame in the North-South direction and the moment frame in the East-West direction were stressed to the point of failure when subject to the lateral loads. This resulted in the addition of bracing in both directions to strengthen the steel frame to accept the Moderate Hazard loads within the ASCE/ANSI N690 code criteria.

The filtrate storage tank supports were evaluated for seismic loads using the UBC equivalent static method for the Low Hazard PGA of 0.11g. The existing tank support legs were found to have insufficient capacity for the lateral loads and were modified to include bracing [5.7]. The bracing design for the lateral loads satisfy the UBC code criteria for steel.

Table 5.1 Comparison of Criteria for 241-96H
Original Design Basis Vs. DOE Order 6430.1A

Original Design Basis	DOE Order 6430.1A
Dupont Site Specification 7096, R2	Engineering 01060-02-R, R0
Classification: Standard	Classification: Enclosure Building = Low Hazard Filter Cells = Moderate Hazard
Wind Design: Design Methodology per ANSI A58.1 Standard wind speed = 85 mph	Wind Design: Methodology per ASCE 7-88 (Replaces ANSI A58.1) Low Hazard wind speed = 84 mph Moderate Hazard wind speed controlled by tomado = 138 mph.

**Table 5.2 Comparison of Criteria for 241-96H
Original Design Basis Vs. DOE Order 6430.1A**

Original Design Basis	DOE Order.6430.1A
<p>Seismic Loads:</p> <p>Seismic design procedure similar to UBC 85 with some modifications:</p> <p>$E = 0.10(D + 0.25L)$</p>	<p>Seismic Loads:</p> <p>Per DOE 6430.1A, seismic design to follow procedures in UBC 88 for Low Hazard with a $PGA=0.11g$</p> <p>$V = [ZICW]/R_w$</p> <p>Where, for the Low Hazard Superstructure: $Z = 0.11g$ $I = 1.25$ $C = 2.75$ (Max) $R_w = 6$ (OMRSF) $W = D + 0.25L$ Therefore $V = 0.063D + 0.016L$</p> <p>Dynamic analysis methods are used for Moderate Hazard Concrete Cells with a minimum $PGA=0.11g$</p>
<p>Concrete Design Code:</p> <p>ACI 318 - 77 for Standard</p>	<p>Concrete Design Code:</p> <p>UBC 88 for Low Hazard is required and is based on ACI 318 - 83 with added requirements for earthquake resistance.</p> <p>ACI 349 is required for Moderate Hazard design</p>
<p>Steel Design Code:</p> <p>AISC 7th Edition for Standard</p>	<p>Steel Design Code:</p> <p>UBC 88 for Low Hazard is required and is based on AISC with added requirements for earthquake resistance.</p> <p>AISC/ANSI N690 is required for Moderate Hazard</p>

**Table 5.3 Comparison of Criteria for 241-96H
Original Design Basis Vs. DOE Order 6430.1A**

Original Design Basis	DOE Order 6430.1A
<p>Load Combinations for Standard:</p> <p>Concrete: Wind: (D + L + W)</p> <p>$U = 0.75(1.4D + 1.7L + 1.7W)$ $U = 0.9D + 1.3W$</p> <p>Earthquake: (D + L + E)</p> <p>$U = 0.75(1.4D + 1.7L + 1.87E)$ $U = 0.9 + 1.43E$</p> <p>Steel (ASD):</p> <p>D + L @ F D + L + W or E @ 1.33F D + W or E @ 1.33F</p>	<p>Load Combinations for Low Hazard:</p> <p>Concrete: Wind: (D + L + W)</p> <p>No Change</p> <p>Earthquake: (D + L + E)</p> <p>$U = 1.4(D + L + E)$ $U = 0.9D + \text{or} - 1.4E$</p> <p>Steel (ASD):</p> <p>Essentially the same, however UBC 88 has the following requirements for steel used as lateral force resisting frames where section 2722 requires full strength of the members to be developed for earthquake loads.</p> <p>Flexure $M_s = ZF_y$ Shear $V_s = 0.55 F_y dt$ Axial Compression $P_{sc} = 1.7F_a A$ Axial Tension $P_{st} = F_y A$</p> <p>Connections: Full Penetration Welds $F_y A$ Partial Penetration Welds 1.7F Bolts and Fillet Welds 1.7F</p>

Table 5.4 Comparison of Criteria for 241-96H
Original Design Basis Vs. DOE Order 6430.1A

Original Design Basis	DOE Order 6430.1A
None	Load Combinations for Moderate Hazard Concrete: Wind: $U = D+F+L+H+T_O+R_O+W_t$ Earthquake: $U = D+F+L+H+T_O+R_O+E_{ss}$ Steel (ASD): Wind: $U = D+F+L+H+T_O+R_O+W_t @ 1.6F$ Earthquake: $U = D+F+L+H+T_O+R_O+E_{ss} @ 1.6F$

Table 5.5 Natural Phenomena Hazards Criteria

Building Category	General Use	Important or Low Hazard	Moderate Hazard	High Hazard
Performance Goal Annual Probability of Exceedance	1×10^{-3}	5×10^{-4}	1×10^{-4}	1×10^{-5}
Seismic	Annual Hazard Exceedance Probability	2×10^{-3}	1×10^{-3}	2×10^{-4}
	Peak Ground Acceleration* (PGA)	0.08g	0.11g	0.20g
Wind	Annual Hazard Exceedance Probability	2×10^{-2}	2×10^{-2}	1×10^{-3}
	Wind Speed (Including Importance Factor)	78 mph	84 mph	109 mph
	Missile Criteria	-	-	2x4 timber plank 15 lb @ 50 mph (horiz); max height 30 ft.
Tornado	Annual Hazard Exceedance Probability	-	-	2×10^{-5}
	Wind Speed (Including Importance Factor)	-	-	137 mph
	Atmospheric Pressure Change (APC)	-	-	40 psf @ 20 psf/sec
	Missile Criteria	-	-	2x4 timber plank 15 lb @ 100 mph (horiz); max height 150 ft; 70 mph (vert) 3 in dia std steel pipe, 75 lb @ 50 mph (horiz); max height 75 ft; 35 mph (vert)
Flood	Most of SRS is excluded from flood consideration. However, river flooding from natural phenomena and dam, levee or dike failure must be considered on a site location specific basis. Loads and load combinations shall satisfy Chapter 6 of UCRL 15910.			
	Site specific meteorological data of local precipitation must be considered to design for site drainage and roof ponding. Loads and load combinations shall satisfy Chapter 6 of UCRL 15910.			

* Hazardous Waste Facilities which fall within SCDHEC regulation shall be designed for a minimum of 0.15g PGA.

5.1 References

- 5.1 Calculation #C-CLC-H-00292, Structural Design of Filter Cells 241-96H
- 5.2 Calculation #C-CLC-H-00255, Evaluation of Misc. Component of Building 241-96H
- 5.3 Calculation #C-CLC-H-00239, Fire Building and Crane Access Platform Building
- 5.4 Calculation #C-CLC-H-00249, Evaluation of Filter Enclosure Building 241-96H Seismic Loads
- 5.5 Calculation #C-CLC-H-00290, Design of Filter Enclosure Building 241-96H Seismic Loads
- 5.6 Calculation #C-CLC-H-00309, Design of the 241-96H Laboratory
- 5.7 Calculation #C-CLC-H-00294, ITP Facility Modification of Filter Hold tank Support System (241-96H)

6. Analysis and Evaluation of Transfer Lines

The above ground transfer lines make transfers between Tank 48 and the filter building. The lines are fabricated as double wall piping with a 10" carbon steel jacket pipe and a 6 inch stainless steel core pipe. The piping is independently supported inside 4-1/4" thick lead shielding. The piping and shielding is supported on structural steel beams. The loads used in the structural evaluation and a summary of the results are provided in this section.

Four above-ground transfer lines (705A, 1105A, 1151A, 1152A) were evaluated. The lines were evaluated for thermal expansion, dynamic effects of seismic ground motion, soil settlement, and wind. A summary of the highest demand to capacity ratios are provided in Table 6.1 for each loading.

Table 6.1 Demand to Capacity Ratios

Loading	D/C Ratio	Reference
Thermal Expansion	.94	6.2
Seismic Piping	.56	6.2
Seismic Supports		
Connections	.97	6.2
I-Beams	.24	
1-1/4" Static Settlement	.33	6.2
6" Dynamic Settlement	.15	6.2, 6.3
Wind	.62	6.1

Thermal expansion loads have been evaluated [6.2] for the transfer lines. The lines were evaluated for a 140°F differential temperature. The lines were evaluated to ASME B31.3 criteria.

For seismic considerations, frequency analysis was performed to obtain the appropriate seismic acceleration levels. The seismic ground motion input to the transfer lines was a .20G PGA earthquake. The Blume spectral shape was used for the evaluation [6.2]. Since the piping is supported inside the lead shielding two independent structural evaluations were performed. The first analysis included only the piping. Stress levels in seismic analysis were compared to a $2.4S_H$ allowable per ASME Code. The second analysis was to evaluate the structural steel supports and included the lead shielding. The damping in the second analysis was increased to 10% to account for the many gaps in the lead shielding. The structural steel was evaluated to AISC code criteria.

Two cases of soil settlement were evaluated for the transfer lines. A 1-1/4" static settlement was evaluated. The static settlement addressed the effect of filling the waste tank. The static settlement was evaluated as an anchor movement to a $2.4S_H$ allowable stress. A 6" dynamic settlement due to earthquake loading was evaluated. The two settlement cases were not combined because the end of the piping near the filter building governed the 6" dynamic settlement loading. The filter building was not evaluated for a static settlement. An increased allowable [6.3] was developed for the 6" dynamic settlement loading based on ASME fatigue criteria. Seismic interaction effects were also evaluated for the transfer lines.

The transfer lines have been qualified for tornado wind levels [6.1] of 185 mph as specified in UCRL-15910 of High Hazard Natural Phenomena Hazards events [1.4]. The wind loads were generated using methodology given in ANSI A58.1-1982. For tornado effects, the elevation and geometry of the transfer lines were used to obtain the appropriate wind induced loads. The load path from the transfer lines to the supporting steel was evaluated in response to the calculated wind loads. The stress in the supporting steel was evaluated to AISC-N690.

In addition, the transfer lines were evaluated for damage due to High Hazard tornado generated missiles. Per UCRL-15910 the lines required evaluation for effects due to a steel pipe, wooden plank, and a rolling and tumbling automobile. Due to the physical location of the transfer lines and shielding by surrounding structures, the automobile missile was judged to be incredible. The pipe and plant missile were considered for possible perforation of the transfer lines. The perforation evaluation was performed neglecting the impact absorption capability of the surrounding lead shielding. Even neglecting the lead, the transfer lines were shown to have adequate capacity to resist perforation due to missile impact.

6.1 References

- 6.1 Calculation #T-CLC-H-00071, Rev. 2, 7-21-94, 241-H Piping Line Natural Phenomena Hazard Qualification (U)
- 6.2 Calculation No. T-CLC-H-00135, Rev. 0, Analysis for Tank 48 Above Ground Transfer Lines.
- 6.3 Calculation No. T-CLC-H-00137, Rev. 0, ITP Piping Allowable Stress for Soil Settlement, Beyond Design Basic Allowable Stress Value.

Appendix A - ITP Structures Supporting Documentation

Structural Mechanics Acceptance Criteria for High Level Waste Tanks (WSRC-TR-94-335)

Structural Evaluation of the Reinforced Concrete Vault for Operating Loads, High Level Waste Tanks 48 - 51

T-CLC-G-00013	Type III high level waste storage tanks reinforced concrete capacity
T-CLC-G-00016	Waste Storage Tanks 48 -51 Thermal Analysis
T-CLC-G-00018	ITP Waste Storage Tank 48 - 51 Static Analysis

Structural Evaluation of the Reinforced Concrete Vault for Seismic Loads, High Level Waste Tanks 48 - 51

T-CLC-H-00142	H-area/ITP Seismic Safety Issue - ASCE 4-86 Lateral Soil Load
T-CLC-H-00085	H-area/ITP Seismic Safety Issue - Waste Tank Free-Field Analysis
T-CLC-H-00086	ITP - Blume Earthquake Record
T-CLC-H-00087	H-area/ITP Seismic Safety Issue - Waste Tank SSI 2 Tanks Analysis
T-CLC-H-00088	H-area/ITP Seismic Safety Issue - Waste Tank SSS w/ Embankment
T-CLC-H-00015	Impact Assessment for Top Slabs of IIIA Tank # 48 - 51
T-CLC-H-00119	H-area/ITP Seismic Safety Issue- Waste Tank Vertical Analysis

Structural Evaluation of the Reinforced Concrete Vault for Seismically Induced Settlements, High Level Waste Tanks 48 - 51

T-CLC-G-00011	Differential settlement ITP- waste tank type IIIA tank # 48 - 51
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Structural Evaluations of the Steel Tanks 48-51

T-CLC-H-00100	Effective Single D.O.F. Fluid Model
T-CLC-H-00102	ITP Type IIIA Waste Tank Seismic Analysis (Steel Tank)
T-CLC-H-00103	ITP Type IIIA Waste Tank Soil Settlement Analysis (Steel Tank)

Structural Evaluation of the ITP Buildings & Equipment

T-CLC-H-00071	241-H Piping Line NPH Qualification
T-CLC-H-00135	Analysis for Tank 48 Above Ground Transfer Lines
T-CLC-H-00137	ITP Piping Allowable Stress for Soil Settlement, Beyond EBE Stress...
C-CLC-H-00239	Filter Building and Crane Access Platform 241-96H (Wind)
C-CLC-H-00249	Evaluation of Filter Enclosure Building 241-96H Seismic Loads
C-CLC-H-00250	Evaluation of Benzene Stripper Building 241-96H for Seismic Loads
C-CLC-H-00255	Evaluation of Misc. Components of Building 241-96H
C-CLC-H-00290	Design of Filter Enclosure Building 241-96H Foundation
C-CLC-H-00292	Structural Design of Filter Cells 241-96H
C-CLC-H-00294	ITP Facility Modification of Filter Hold Tank Supp. System 241-96H
C-CLC-H-00309	Design of the 241-96H (Wind and Seismic Modifications)
C-CLC-H-00331	ITP Facility Building Frame Bracing Building 241-96H

FIGURES

FIGURE 3.1 H- area ITP Facility (W- 704285)

PARTIAL PLAN
NOT TO SCALE

FIGURE 3.2 --H- area ITP Facility section A-A

M91dec013.06

Figure 3.3 Cross Sectional View of Type IIIA Tank 48-50

Figure 3.4 Plan View of Filter/Stripper Building 241-96H

Figure 4.1 Axisymmetric Model for Operating Load Analysis

Figure 4.2 Vault Top - Demand vs. Capacity for Radial Bending Moment
Ksoil = 15 pci

Figure 4.3 Basemat - Demand vs. Capacity for Shear
Ksoil = 15 pci

(a)

(b)

Figure 4.4 Basemat - Demand vs. Capacity for Radial and Circumferential Bending Moments, $K_{soil} = 15 \text{ pci}$

Figure 4.5 Tank Top - Demand vs. Capacity for Radial Bending Moment
Ksoil = 15 pci

Figure 4.6 Waste Tank Finite Element Model

Figure 4.7 Blume - Site Controlling Response Spectra

SECTION A-A

Figure 4.8 Three Dimensional SASSI Finite Element Model

Figure 4.9 ASCE 4-86 Lateral Soil Load (ksf)

Figure 4.10 SSI 3-D Tank Models (Empty) Maximum Acceleration

Figure 4.11 SSI 3-D Tank Models (Full) Maximum Acceleration

Figure 4.12 SSI 3-D Tank Models (Full) Maximum Acceleration

Figure 4.13 Vault Longitudinal Bending Moment for Combined Operating and ASCE 4-86 Seismic Loads

Figure 4.14 Tank Top Radial Bending Moment for Combined Operating and Vertical Seismic Loads

Figure 4.15 Hydrodynamic Pressure Profiles

Figure 4.16 Center Settlement Profile

Figure 4.17 Edge Settlement Profiles

Figure 4.18 Basemat with Small Diameter Center Settlement Profile

Figure 4.19 Vault Hinge Mechanisms for the Center Settlement Profile

Figure 4.20 Basemat Yield Lines

Figure 4.21 Edge Settlement Profile

Figure 4.22 Rigid Body Rotation of the Vault due to
Compression of the Soil

Figure 4.23 Cross Section Through Vault with Edge Settlement Profile and Possible Basemat Collapse Mechanism

**Figure 4.24 Vault Wall Shear and Bending Mechanism
for the Edge Settlement Profile**