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ENGINEERING DATA TRANSMITTAL

Page 1 of 1

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No 6 2807

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1	WHC-SD-WM-TI-775	all	0	STRUCTURAL ASSESSMENT OF ACCIDENT LOADS	NA	1	1	1

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(G) Reason	(H) Disp.	(J) Name	(K) Signature	(L) Date	(M) MSIN	(G) Reason	(H) Disp.	(J) Name	(K) Signature	(L) Date	(M) MSIN
		Design Authority									
		Design Agent									
1	1	Cog.Eng. GR WAGENBLAST	<i>GR Wagenblast</i>	9/20/96							
1	1	Cog. Mgr. MS RUBEN	<i>MS Ruben</i>	9/20/96							
		QA									
		Safety									
		Env.									

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BD-7400-172-1

STRUCTURAL ASSESSMENT of ACCIDENT LOADS

GR Wagenblast

ICF KAISER, Richland, WA 99352

U.S. Department of Energy Contract DE-AC06-87RL10930

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Key Words: tanks, IMUST, MISF, DCRT, accident loads

Abstract: Structural assessments were made for specific accident loads for specific catch, receiver, and storage tanks. The evaluation herein represents level-of-effort order-of-magnitude estimates of limiting loads that would lead to collapse or rupture of the tank and unmitigated loss of confinement for the waste. Structural capacities were established using failure criteria. Compliance with codes such as ACI, ASCE, ASME, RCRA, UBC, WAC, and DOE Orders was NOT maintained. Normal code practice is to prevent failure with margins consistent with expected variations in loads and strengths and confidence in analysis techniques. The evaluation herein represent estimates of code limits without code load factors or code strength reduction factors, and loading beyond such a limit is considered as an onset of some failure mode. The exact nature of the failure mode and its relation to a safe condition is a judgement of the analyst. Consequently, these RESULTS SHALL NOT BE USED TO ESTABLISH OPERATING OR SAFETY LOAD LIMITS FOR THESE TANKS.

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Karen H. Noland
Release Approval

9/24/96
Date

Approved for Public Release

Structural Assessment of Accident Loads for FSAR

May 28, 1996

Objective

The objective is to perform structural assessments of specific tanks subjected to accident conditions. Results from the assessment establish either the limiting load that will not result in structural failure or the potential for defined load or condition to cause structural failure. In either case, the affect of the structural failure which is to be evaluated in the assessment is the loss of confinement for the waste, i.e., when an unobstructed opening or path is created between the waste and atmosphere. Therefore, the structural failures that will be evaluated in this assessment are those that would cause the collapse or rupture of the tank structure. Openings in the joints between cover blocks, temporary lifting of cover blocks, and damage to HVAC systems will not be evaluated.

The results from these assessments will be based on failure limit analyses. Compliance with codes such as ACI, ASCE, ASME, UBC, RCRA, WAC is not maintained. Therefore, these results can not be applied to establish operations load limits for these tanks. It should be recognized that normal code practice is to prevent failure with margins consistent with expected variations in loads and strengths, economics of design, and confidence in analysis techniques. The efforts herein represent level-of-effort estimates of code limits without strength reduction factors, and loading beyond such a limit is considered as an onset of some failure mode. The exact nature of the failure mode and its relation to openings and lack of confinement is a judgement on the part of the analyst. Detailed physical calculations to determine sizes of openings and sequences of failures is beyond the scope of this effort. In some instances, results indicate that the structure is not satisfactory with normal loads. These indications are not conclusive, and they might change with refined analysis. Consequently, failure under current conditions is not conclusive.

Results are summarized in the section on task descriptions and results.

Details of the analyses are given in the appendices as indicated in the task descriptions and results. Numbering of these appendices generally follows the task numbering except for tasks have been combined for efficiency.

Scope

This work plan will cover structural assessments for the following tanks in each defined group. Equipment components such as riser, pumps, instruments and HVAC systems are not included in this work scope.

Catch Tanks (CT)

241-A-302-A
241-A-417
241-AX-152
241-AZ-151
241-AZ-154
241-ER-311
241-EW-151
241-S-304
241-TX-302-C
241-U-301-B
241-UX-302-A

Double-Contained Receiver Tanks (DCRT)

244-A Lift Station
241-A-350 Lift Station
244-BX Catch Station
244-CR-003 Vault Tank
244-S Lift Station
244-TX Catch Station
244-U Catch Station

200-Series Single-Shell Tanks (SSTs)

241-B
241-C
241-T
241-U

100-Series Single-Shell Tanks

All Tanks

Double-Shell Tanks (DSTs)

All Tanks

Units

Efficient units, from the viewpoint of drawings and support material, are found throughout the document without conversion. Some may need explanation as follows.

psi	lbf/in ²
pci	lbf/in ³
K	1000 lbf or 1000 lbf·ft/ft for moment/length
kip	1000 lbf
ksf	1000 lbf/ft ²
kcf	1000 lbf/ft ³
lb	lbf

Additional notations in the appendices are as follows

#	lbf
"	in.
'	ft

Task Descriptions and Results

For DCRTs, Catch Tanks, and 200-Series SSTs

Determine the maximum limit for the loads described in tasks 1 through 4.

1. Concentrated Static Surface Load

Determine the maximum concentrated static load that can be applied at the soil surface, either directly over a buried structure or adjacent to a buried structure, and not result in a loss of confinement. The assumed contact area shall be based the maximum possible pressure loading for the soil.

Results. See Table 1.1. Two directly buried steel tanks cannot be shown satisfactory under existing soil loads, but no opening to the atmosphere is expected if the tanks collapse. All other cases have various concentrated load capacities ranging from 6 kips upward.

Details. See Appendix 1.

2. Impact Energy from Dropped Equipment.

Determine the maximum potential energy for equipment that would not result in a loss of confinement if the equipment were dropped. This energy limit shall be the minimum for any position over the tank. Provide the results as a function of contact area; ranging from a cross section area of 0.1 ft² to 2.0 ft² (three data points minimum). The shape assumed for the contact area shall produce the lowest allowed energy value.

Results. An equipment of 20,000 pounds with 2 feet diameter base was considered. The results of the evaluations indicate that the perforation of the cover block from the falling equipment is not realistic. The energy as high as 29,000,000 inch-pounds would be needed to perforate a 20 inch thick cover block if a 20,000 pound equipment is dropped from a height of 120 feet. See Table 2.1. On the other hand, the results of the evaluations indicate that the cover blocks do not have adequate bending strength to resist impact. A failure of the cover block leading to an opening is most likely to occur due to the energy from the falling of a 20,000 lb equipment from any height. See Table 2.2. The impact with the soft ground was considered for three drop heights from 10 ft to 20 ft. The results show that the vault wall and roof for the DCRTs, catch tanks and the 200 series SSTs are adequate to withstand the impact. See Table 2.3.

Details. See Appendix 2.

3. Internal Transient Pressure Caused by Hydrogen Deflagration.

Determine the maximum peak pressure for a Hydrogen deflagration inside the primary waste tank that would not result in a loss of confinement. Peak pressures above 120 psig do not need to be evaluated. For the DCRTs, the peak pressure is applied inside the steel tank. Assume the pressure rise time is essentially instantaneous.

Results. See Table 3.1. Tanks marked OK in the status column will not result in a loss of confinement given the data and assumptions used in the detailed calculations. The remaining cases are considered to yield an opening, because (1) the steel tank was not evaluated for pressurization (lack of data), or (2) there is no steel tank, or (3) the steel tank pressure exceeded capacity.

Details. See Appendix 3.

4. Seismic Event

Determine the maximum ZPA for a free-field ground motion that would not result in a loss of confinement. Ground motions in both the horizontal and vertical directions shall be evaluated to determine which one controls. Combination of ground motions in more than one orthogonal direction is not required, unless a significant increase in the result is anticipated. If multiple directions are combined, the magnitude of the ground motion shall be the same for each direction. The ground motion applied in this evaluation shall be scaled from the response spectra defined in SDC 4.1 for the Hanford-Site. For this evaluation assume the steel tanks in the DCRTs fail during the seismic event.

Results. Results are summarized in Table 4.1 along with the maximum ZPA and failure mode. Some directly buried tanks such as 241-ER-311 are shown to collapse without seismic loading, i.e., they were not shown good for static loading alone. As such, the simple analysis procedure could well overestimate the static loading, and refined analysis would be required. Given a collapse in a seismic event, the tank volume is small compared to a characteristic soil volume above it, and in the analyst's judgement a significant opening is unlikely. Pit wall and cover block collapse is assumed to lead to an opening corresponding to the plan form. No further failure sequence evaluated. In the case of the lower wall of 244-CR, a refined analysis might show integrity with static soil loads by considering plate edge restraints and distribution of moments in the wall. Consequently, failure under current conditions is not implied.

Evaluate the specific loads and conditions described in tasks 6 through 8.

5. Internal Vacuum Caused by Organic Pool Fire

Assess the potential for a quasi-static vacuum of -8 psig to cause a loss of confinement. This vacuum is caused by an organic pool fire on the waste surface, followed by cooling of the high temperature gasses inside the tank. Assume the concrete tanks remain at normal operating temperatures. Assume the steel tanks can reach 200 F.

Results. Some directly buried steel tanks are not satisfactory against buckling, but an opening to the atmosphere is not expected unless liquid is squeezed out through a normal opening by tank collapse under soil

load. Aside from these directly buried tanks, roof and floor structures are not expected to collapse, and openings to the atmosphere are not expected.

Details. See Appendix 5.

6. Internal Hydrostatic Load

Assess the potential for the hydrostatic load on the tank wall to cause a loss of confinement. Assume the tank is full of fluid and the specific gravity for the fluid is 2.0.

Result. No opening to the atmosphere is expected with the internal hydrostatic loads.

Details. See Appendix 1.

7. High Temperature Waste Sludge

Assess the potential for high temperature waste sludge to cause a loss of confinement. Assume the sludge temperature is 600 F and the sludge is stored in the tank for 1 year. Apply the sludge temperature as the inside surface temperature for the bottom half of the tank wall and apply an assumed inside surface temperature of 250 F for the upper half of the tank wall and dome. For DCRTs, where the steel tank is not in direct contact with the vault of caisson structure, assume a temperature of 250 F for entire inside surface of the vault structure, including the cover. Assume a through wall temperature gradient for concrete foundations, walls, and covers that is $\frac{1}{2}$ of the difference between the inside surface temperature and an assumed ambient temperature of 50 F, unless a more appropriate temperature gradient can be established and justified.

Result. No roof, cover block or dome collapse is expected from the thermal loading. Consequently, no significant opening to the atmosphere is expected. See Tables 7.1 and 7.2.

Details. See Appendix 7.

8. Aging and Degradation

Assess the potential for aging and degradation mechanism to cause a loss

of containment. Specifically address how each of the following mechanisms could individually affect confinement of the waste.

- Corrosion
- High temperature waste
- Thermal cycling

Results.

Corrosion. Pitting corrosion is local and is not expected to cause tank collapse; leakage is possible. Uniform corrosion is not a problem for about 60 years of tank life. Stress corrosion is not a concern for tank collapse. Reinforcement corrosion is of concern only as a result of tank leakage. Waste leaking past a steel liner can migrate directly through uncracked concrete, but obviously at a much lower rate than along a crack.

High temperature waste. This is discussed in Item 7.

Thermal cycling. The temperature of the vault at each cycle is below moderately high temperature, say 250 F, and thermal cycling does not affect the dome integrity.

Details. See Appendix 8.

For DCRTs only

9. Pressure Transient Caused by Gasoline Fire.

Assess the potential for a transient pressure, external to the primary waste tank, to cause a loss of confinement. The pressure transient for this accident is caused the 20 gallons of gasoline being spilled into the vault that contains the primary waste tank. The gasoline is then vaporized and ignited. Assume a peak pressure of 140 psig and a pressure rise time that is essentially instantaneous.

Results. None of the structures survive this scenario, and all roofs and cover blocks are expected to be blown off with a full opening to the atmosphere. Tanks in vaults with the gasoline ignition are expected to collapse if evaluated in Table 3.1, i.e., if information was available. See Table 9.1.

Details. See Appendix 5.

For Catch Tanks Only

10. Uniform Static Surface Load

Determine the maximum uniform static surface load that can be applied over the catch tank and not result in a loss of confinement.

Results. See Table 1.1. Two directly buried steel tanks cannot be shown satisfactory under existing soil loads. No opening to the atmosphere is expected were the tanks to collapse. All other cases have various uniform load capacities ranging from 0.26 ksf upward.

For DSTs and 100-Series-SSTs

11. Internal Vacuum Caused by Organic Pool Fire

Assess the potential for a quasi-static vacuum of -8 psig to cause a loss of confinement. This vacuum is caused by an organic pool fire on the waste surface, followed by cooling of the high temperature gasses inside the tank. Assume the concrete tanks remain at normal operating temperatures. Assume the steel tanks can reach 200 F.

Results. Same as Item 5.

Details. See Appendix 5.

For DSTs Only

12. Pressure Transient Caused by Gasoline Fire.

Assess the potential for a transient pressure, external to the primary waste tank, to cause a loss of confinement. The pressure transient for this accident is caused the 20 gallons of gasoline being spilled into the annulus between the primary and secondary tank liners. The gasoline is then vaporized and ignited. Assume a peak pressure of 140 psig and a pressure rise time that is essentially instantaneous.

Results. Included with item 9. An opening of about 30 feet in diameter at the crown is expected along with collapse of the primary tank.

Details. See Appendix 5.

For 100-Series SSTs Only

13. High Concrete Temperatures and Pressure Caused by Organic Nitrate Reaction

Review the attached memo that describes a failure mechanism for 100-Series SSTs cause by high temperatures and internal pressure form an in-tank organic pool fire. Assess whether or not this failure mechanism is expected to occur as described. Provided discussion of the reasoning applied in this assessment.

Results. Dome collapse is considered possible with substantial parts of the dome and soil cover falling into the waste. Two effects pointing to collapse are possible that are not brought up in the memo. First, dome cracking under pressure may be biased toward the center of the dome with less leakage area and higher pressure development. Second, concrete degradation through the thickness may occur with hot gases passing through cracks. Even if the dome did not collapse, it would have severe strength reductions and should be considered unstable.

Details. See Appendix 13.

Additional Notes:

The following apply to all structural assessments.

- * For each accident load or event described herein, the structural failure modes expected to occur first are described in the details along with explanations how these failure modes could affect confinement of the waste.
- * All structural assessments were performed using simplified methods and bounding analyses. In particular, detailing code requirements were not checked.
- * All structural assessments were be based on present existing live and dead loads. In LRFD and ACI code terminology, load factors were taken as unity. Material properties were taken as minimum and strength reduction factors were not assigned.
- * The affect of operational loads such as pressure and temperature and the

affect of the allowed variations in these loads were included.

- * Best estimate values for soil properties were used. Soil density is 125 pcf.
- * Corrosion was neglected if its affect on the analysis results was considered negligible. Otherwise reasonable values for corrosion were selected, and technical justifications for the value were provided.
- * Property data for concrete at 600 F were be extrapolated from property data at 450 F.
- * The thermal history for tank C-106 were used to determine initial degraded concrete properties for 100-series SSTs. For all other tanks, the design properties for the concrete were used.
- * Openings between the joints in cover blocks are acceptable with respect to the confinement issue.
- * Soil density over all tanks is 125 lbf/ft³.
- * The structure in not serviceable after the accident load.

Table 1.1. Uniform and Concentrated Load Summary

TANK OR PIT	STRUCTURAL ITEM	ULTIMATE LOAD
241-A-302-A 241-ER-311	Pump Pit concrete cover blocks at grade level	119 kips, or 3.7 ksf
	Steel Tank 18 ft below grade	Fails under soil dead loads
241-A-417	Pump Pit conc. cover blocks at grade level	49 kips, or 1.0 ksf
	Concrete tank roof slab 14 ft below grade	0.96 ksf
241-AX-152	Pump/Diverter Pit conc. cover blocks at grade level	224 kips, or 14.9 ksf
241-AZ-151	Pump Pit conc. cover blocks at grade level	170 kips, or 3.9 ksf
	Concrete tank roof slab 16 ft below grade	62 kips, or 1.7 ksf
241-AZ-154	Pump Pit conc. cover block at grade level	191 kips, or 6.0 ksf
241-EW-151	Diversion Box conc. cover blocks at grade level	97 kips, or 3.0 ksf
241-S-304	Pump Pit conc. cover blocks at grade level	51 kips, or 1.0 ksf
241-TX-302C 241-UX-302A	Pump Pit steel cover plate at grade level	7 kips, or 3.4 ksf
	Steel Tank 26 ft below grade	Fails under soil dead loads
241-U-301B	Pump Pit steel cover plate at grade level	7 kips, or 3.4 ksf
	Concrete domed tank 11 ft below grade	2,689 kips, or 7.7 ksf
244-A	Pump Pit conc. cover blocks at grade level	61 kips, or 0.9 ksf

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241-A-350	Pump Pit conc. cover blocks at grade level	53 kips, or 1.2 ksf
244-BX, TX, U	Pump Pit conc. cover blocks at grade level	70 kips, or 1.3 ksf
244-CR-003	Pump Pit conc. cover blocks at grade level	53 kips, or 2.0 ksf
244-S	Pump Pit conc. cover blocks at grade level	31 kips, or 0.38 ksf
200-Series SST	Concrete Tank roof slab 11 ft below grade	327 kips, or 1.04 ksf

Client WHC

WO/Job No.

Subject	Evaluation of Impact Energy	Date	5/15/96	By	M.A. ISLAM
	SERTS, Catch Tank & 200 Series SST	Checked	5-20	By	R.A. Giller
Location	200 Area	Revised		By	

SUMMARY OF EVALUATIONSA. Impact Energy Required to perforate Cover Block
(Table 2.1)

Tank Data		Missile Data		Energy Req'd. for perforation	
Tank	Cover Depth (in)	Weight (lbs)	Diameter (in)	Height of Fall (h) (in)	Energy, E = Wh (in-lbs)
241-A-350 (CERT)	20	20,000	24	1440 (120 FT)	28,800,000

B. Impact with Yielding Surface (Ground)
(Table 2.2)

Tank Data		Analysis Data (Assumed)				Result	Remarks
Tank	Soil Cover Depth (in)	Missile Weight, W (lbs)	Height of Fall, h (in)	Soil Density, γ (#/cu ft)	Soil Friction Angle, ϕ (Deg.)	Force @ Impact, F _i (lbs)	
200 Series SST 241-B,C,T,U	144 (12 Ft)	20,000	120	125	30	55,897	74.5 FI dissipates significantly as it approaches Tank roof. Therefore, Tank roof will maintain its integrity.

Client WHC

WO/Job No. _____

Subject Evaluation of Impact EnergyDate _____ By M.A. ISLAMChecked S-20- By R.A. GillerLocation 200 Area

Revised _____ By _____

SUMMARY OF EVALUATIONS CONTD.C. Impact with non-yielding surface e.g. coverblock, pit, riser etc.
(Table 2.J)DCRTs* using an equipment
weight, $W = 20,000$ lbs

Group	Tank Data				* Energy Req'd. to Cause Bending Failure in Cover Block. (in-lb)
	Listed Tanks	Cover Block Depth(in)	Span (in)	Rebar Area(in ²)	
1	241-A-350	20	108	#5 @ 12 $A_s = .31/F$	None
2	244-TX, 244-U 244-BX	24	216	#8 @ 6 $A_s = 1.58/F$	None
3	244-CR-003	24	53	#6 @ 12 $A_s = .44/F$	None
4	244-A, 244-S	24	252	#6 @ 12 $A_s = .44/F$	None

CATCH TANKS

1	241-TX-302C, 241-UX-302A 241-A-302A, 241-ER-311 241-EW-151	20	72	#5 @ 12 $A_s = .31/F$	None
2	241-Ax-152	68	84	#6 @ 6, $A_s = .85$	None
3	241-S-304	30	130	#6 @ 12, $A_s = .44$	None
4	241-A-417	2 @ 15 = 30	96	#6 @ 12, $A_s = .72$ (3 layers)	None
200 Series SST	241-B, C, T, U	12	240	#6 @ 12 $A_s = .44/F$	None

Table 3.1 Internal Transient Pressure Summary

	VESSEL MATERIAL	VESSEL DIA	WALL THICKNESS	WALL STRESS (1)	YIELD STRESS	STATUS
Catch Tanks						
241-A-302-A	carbon steel direct buried	9'-0"	9/16"	23,040 psi	30,000 psi	OK
241-A-417	lined concrete vault, no vessel					Vault Fails
241-AX-152	lined concrete vault, no vessel					Vault Fails
241-AZ-151	lined concrete vault, no vessel					Vault Fails
241-AZ-154	lined concrete vault, no vessel					Vault Fails
241-ER-311	carbon steel direct buried	9'-0"	9/16"	23,040 psi	30,000 psi	OK
241-EW-151	stainless steel	4'-5"	3/8"	17,280 psi	25,000 psi	OK
241-S-304	carbon steel	9'-0"	5/8"	20,736 psi	30,000 psi	OK
241-TX-302C	carbon steel direct buried	9'-0"	1/2"	25,920 psi	30,000 psi	OK
241-U-301B	precast concrete vault, no vessel					Vault Fails
241-UX-302A	carbon steel direct buried	9'-0"	1/2"	25,920 psi	30,000 psi	OK
Double-Contained Receiver Tanks						
244-A				not evaluated		Vault Fails See Appendix 5
241-A-350	stainless steel	4'-5"	3/8"	17,280 psi	25,000 psi	OK

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244-BX/TX/U	carbon steel	12'-0"	1/4"	69,120 psi	30,000 psi	Tank Ruptures Vault Fails See Appendix 5
244-CR-003	stainless steel		3/8"	not evaluated		Vault Fails See Appendix 5
244-S	carbon steel	15'-0"	1/4"	86,000 psi	30,000 psi	Tank Ruptures Vault Fails See Appendix 5
200-Series Single Shell Tanks						
241-B/C/T/U	lined concrete vault, no vessel					Vault Fails

1. at 120 PSIG using dynamic factor of 2: $STRESS = 2 \times (120 \text{ psi} \times DIA/2) / (Thickness)$

Table 4.1 Seismic Summary

Item	ZPA, g	Failure Mode	Opening
241-A-201A	> 1.0		No
241-A-417	0.60	Roof/soil collapse Pit wall collapse	Yes
241-AX-152	0.35	Pit wall collapse	Yes
241-AZ-151	0.38	Pit wall collapse	Yes
241-ER-311	0.00	Collapse	No - buried deep
241-EW-151	> 1.0		No
241-TX-302C	0.32	Collapse	No - buried deep
241-UX-302A	> 1.0		No
241-U-301-B	> 1.0		No
241-A-350	> 1.0		No
244-A	0.82	Pit wall collapse	Yes
244-BX/TX/U	0.39	Pit wall collapse	Yes
244-CR	0.00	Lower wall collapse, refined static analysis required	Yes
244-S	0.63	Cover blocks collapse	Yes
241-S-304	0.60	Caisson roof collapse	Yes
200-S	0.48	Roof collapse	Yes

Table 5.1 Internal Vacuum Summary

Tank	Structural Item	OK/NG by Item	Overall
241-A-302-A	Steel tank	OK	OK
241-A-417	Tank roof	OK	OK
241-AX-152	Tank roof	OK	OK
241-AZ-151	Tank roof Tank side wall	OK Marginal	OK, no opening
241-ER-311	Steel tank	NG	NG, no opening
241-EW-151	Steel tank Vault roof	NG OK	OK
241-TX-302-C	Steel tank	OK	OK
241-UX-302-A	Steel tank	NG	NG, no opening
241-U-301-B	R/C tank	OK	OK
241-A-350	Steel tank Caisson roof	Not evaluated OK	OK
244-BX/TX/U	Steel tank Vault roof	NG OK	OK
244-CR	Steel tank Vault roof	Not evaluated OK	OK
244-S	Steel tank Caisson roof	NG OK	OK
241-S-304	Steel tank	OK	OK
200 Series	R/C tank	OK	OK
DST/SST	R/C tank	OK	OK

Notes: NG = no good (fails structural criteria)
OK = satisfies structural criteria

Table 7.1 material properties at high temperatures

DEGRADATION OF MATERIAL PROPERTIES AT HIGH TEMPERATURE							
material	designation	*	normal	250 F		600 F	
			(psi)	(psi)	degradation	(psi)	degradation
carbon steel	SA-516	fy	32000	28750	10%	23600	26%
		E	3122000	2012000	36%	439000	86%
concrete	f c=3000 psi	f c	3000	2509	16%	1592	47%
		f st	217	195	10%	123	43%

*material properties:

fy = yield strength,

E = modulus of elasticity,

f c = compressive strength,

f st = splitting tensile strength.

NOTE: The duration for concrete at high temperature is one year. Duration is not a factor for steel.

DESIGN ANALYSIS

Client WESTINGHOUSE HANFORD COMPANY WO/Job No. E21425 / F6N1A1
Subject Tank Farm Accident Load Evaluation Date May 1996 By K.C. Tu
High Temperature Waste Sludge Checked _____ By _____
Location 200 East Area Revised _____

Table 7.2 High Temperature Waste Sludge Summary

Tank	Structural Item	by Item	Overall	Remark
Catch Tanks				
241-A-302-A	Steel tank	OK	OK	Buried steel tank
241-A-417	Tank roof	Shear NG	OK to Atm	conc. w/liner
241-AX-152	Cover blocks	OK	OK	concrete tank
241-AZ-151	Cover blocks	OK	OK	concrete tank
241-ER-311	Steel tank	NG	OK to Atm	Steel tank buried deep in soil
241-EW-151	Steel tank Cover blocks	OK OK	OK	Steel tank inside concrete vault
241-S-304	Steel tank R/C roof	OK Shear NG	OK	
241-TX-302-C	Steel tank	OK	OK	Buried steel tank
241-U-301-B	R/C tank	NG	OK to Atm	By observation
241-UX-302-A	Steel tank	NG	OK to Atm	Steel tank buried deep in soil
Double-Contained Receiver Tanks (DCRT)				
241-A-350	Steel tank Cover blocks	OK OK	OK	
244-A	Steel tank Cover blocks	OK OK	OK	
244-BX/TX/U	Steel tank Cover blocks & roof	OK OK	OK	
244-CR-003	Steel tank Cover blocks	OK OK	OK	
244-S	Steel tank Cover blocks	OK OK	OK	
200-Series Single-Shell Tanks				
241-B/C/T/U	R/C tank w/ liner	Shear NG	OK to A	

Notes: NG = no good (fails structural criteria)

OK = satisfies structural criteria

OK to Atm = no significant opening to atmosphere

Shear NG = no good for structural shear criteria but not collapse

Table 9.1. Gasoline Ignition Summary

Tank	Structural Item	OK/NG by item	Overall
241-A-350	Caisson roof	NG	Fully open
244-BX/TX/U	Vault roof	NG	Fully open
244-CR	Vault roof	NG	Fully open
244-S	Caisson roof	NG	Fully open
241-S-304	Caisson roof	NG	Fully open
DST	R/C dome	NG	Open, ~25' dia.

Notes: NG = no good (fails structural criteria)
OK = satisfies structural criteria

Appendix 1

Calculations for Uniform and Concentrated Loads and Hydrostatic Pressure

Concentrated and distributed failure loads;

The tank farm catch tanks, double containment receiver tanks (DCRT), and 200 series single-shell tanks (SST) were evaluated for external concentrated and distributed failure loads. For this evaluation, failure is defined as the ultimate strength of the concrete cover blocks, concrete slab, or concrete dome, and the yield strength of the steel cover plates. See Table 1.1 for a tank by tank list of the calculated failure loads for each tank.

Three tanks were calculated to fail under soil loads. These horizontal steel tanks are buried under 18 to 26 ft of soil, and when they fail they will buckle inward filling up with soil. This type of tank failure may contaminate the soil 18 to 26 ft below grade but will not be a release to the atmosphere.

The specific design property assumptions used are stated in the calculations. In addition, assumptions on failure mode and critical member have also been made. Based on engineering judgement, we have concluded that concrete cover blocks, slabs and tank domes will fail before the tank walls, pit walls or foundations.

Hydrostatic Pressure;

The evaluation of hydrostatic loads applied to the catch tanks, DCRTs, and SSTs is based on engineering judgement. We have concluded that hydrostatic pressure may cause cracks in the walls and leak liquids to the surrounding soil but liquids would not reach the soil surface. Cracks and leaks near the bottom of buried tank rarely cause structural collapse of these type tanks.

Table 1.1. Uniform and Concentrated Load Summary

TANK OR PIT	STRUCTURAL ITEM	ULTIMATE LOAD
241-A-302-A 241-ER-311	Pump Pit concrete cover blocks at grade level	119 kips, or 3.7 ksf
	Steel Tank 18 ft below grade	Fails under soil dead loads
241-A-417	Pump Pit conc. cover blocks at grade level	49 kips, or 1.0 ksf
	Concrete tank roof slab 14 ft below grade	0.96 ksf
241-AX-152	Pump/Diverter Pit conc. cover blocks at grade level	224 kips, or 14.9 ksf
241-AZ-151	Pump Pit conc. cover blocks at grade level	170 kips, or 3.9 ksf
	Concrete tank roof slab 16 ft below grade	62 kips, or 1.7 ksf
241-AZ-154	Pump Pit conc. cover block at grade level	191 kips, or 6.0 ksf
241-EW-151	Diversion Box conc. cover blocks at grade level	97 kips, or 3.0 ksf
241-S-304	Pump Pit conc. cover blocks at grade level	51 kips, or 1.0 ksf
241-TX-302C 241-UX-302A	Pump Pit steel cover plate at grade level	7 kips, or 3.4 ksf
	Steel Tank 26 ft below grade	Fails under soil dead loads
241-U-301B	Pump Pit steel cover plate at grade level	7 kips, or 3.4 ksf
	Concrete domed tank 11 ft below grade	2,689 kips, or 7.7 ksf
244-A	Pump Pit conc. cover blocks at grade level	61 kips, or 0.9 ksf

Structural Assessments

Page 12

241-A-350	Pump Pit conc. cover blocks at grade level	53 kips, or 1.2 ksf
244-BX, TX, U	Pump Pit conc. cover blocks at grade level	70 kips, or 1.3 ksf
244-CR-003	Pump Pit conc. cover blocks at grade level	53 kips, or 2.0 ksf
244-S	Pump Pit conc. cover blocks at grade level	31 kips, or 0.38 ksf
200-Series SST	Concrete Tank roof slab 11 ft below grade	327 kips, or 1.04 ksf

DESIGN ANALYSIS

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Revision _____

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Client WHC

WO/Job No. _____

Subject _____

Date 5-3-96 By RA Giller

Checked 5/20/96 By M.A. ISLAM

Location _____

Revised _____

By _____

Z41-A-302A Catch Tank

Z41-ER-311

H-2-71601
71643
71644

CVI - 71531

26

DESIGN ANALYSIS

Client WHC

WO/Job No. _____

Subject _____

Date 5-3-96 By RA GillerChecked 5/20/96 By M.A. ISLAM

Location _____

Revised _____

By _____

241-A-302A Pump Pit Catch Tank # 241-ER-311

H-2-71601, 71643, 71644

Capacity :

$$a = \frac{A_s F_y}{.85 f'_c b} = .751''$$

$$M_n = A_s F_y (d - a/2)$$
$$= 2034 \text{ in-k}$$

#5 @ 12 EW T#B

7- #5 bars EW T#B

 $A_s = 2.17 \text{ si}$ $f'_c = 3000 \quad f_y = 60,000$ $b = 68'' \quad d = 16''$

Allowable Conc. Load :

$$M_n > M_u = \frac{1.0 w l^2}{8} + \frac{1.0 P l}{4}$$

 $w = 250 \text{ psf} \quad l = 5.67'$

$$2034 = \left[\frac{250(5.67)^2}{8} + \frac{P(5.67)}{4} \right] \frac{12}{1000}$$

Check Shear

$$169500 = 1005 + 1.4 P$$

$$2 \sqrt{f'_c} b d = 119 \text{ kips}$$

$$P = 120.3 \text{ kips}$$

O.K.

Reduce $P = 119 \text{ kips}$ $w = 3.7 \text{ ksf}$

DESIGN ANALYSIS

Client WHC

WO/Job No. _____

Subject _____

Date 5-10-96 By RA GillerChecked 5/20/96 By M.A. ISLAM

Location _____

Revised _____ By _____

241-A-302A & 241-ER-311 Catch Tank

Capacity : (Roark 6th Ed pg 263)

$$M = \frac{WRk_z}{\pi}$$

$$M = \frac{W(54")(1.0)}{\pi} = 17.2W$$

$$f_b = \frac{M}{S}$$

$$30000 \text{ psi} = 17.2W / .5$$

$$W = 872 \text{ lb}$$

Soil Dead Load :

$$18' \times 125 \text{ pcf} = 2250 \text{ lb} > 872 \text{ lb}$$

soil dead loads exceed

tank allowable load ∴

No. Good

$$A = (.5")(12") = 6$$

$$I = (12)(.5)^3 / 12 = .125$$

$$S = 12(.5)^2 / 6 = .5$$

$$\alpha = I / AR^2$$

$$= .125 / 6(.54)^2$$

$$= 0.0$$

$$K_2 = 1 - \alpha = 1.0$$

Client WHC
Subject _____

WO/Job No. _____
Date 5-3-96 By KA Giller
Checked 5/20/96 By MA ISLAM
Revised _____ By _____

Location _____

241-A-417 Surface Condenser Facility

H-2-56800
-71671

Client WHC

WO/Job No.

Subject

Date 5-3-96 By RA Giller

Checked 5/20/96 By MA ISLAM

Location

Revised By

241-A-417 Surface Condenser Facility

H-2-56800, -71671

Capacity :

$$a = \frac{A_s f_y}{.85 f'_c b} = .82''$$

$$M_n = A_s f_y (d - \frac{a}{2})$$
$$= 1398 \text{ in-k}$$

Allowable Conc. Load :

$$M_n > M_u = \frac{w l^2}{8} + \frac{P l}{4}$$

$$1398 = \left[\frac{188 (9.25)^2}{8} + \frac{P (9.25)}{4} \right] \frac{12}{1000}$$

$$116500 = 2011 + 2.3P$$

$$P = 49 \text{ kips}$$

$$w = 1.0 \text{ ksf}$$

cover block

5- #6 bars T & B

$$A_s = 2.2 \text{ si}$$

$$f'_c = 3000 \quad f_y = 60000$$

$$b = 63'' \quad d = 11''$$

$$w = 188 \text{ psf} \quad l = 9.25'$$

Check Shear :

$$27 f'_c b d = 76 \text{ kip}$$

O.K.

Client WHC

WO/Job No.

Subject

Date 5-3-96 By RA Griller

Checked 5/20/96 By MA ISLAM

Location

Revised By

241-A-417 Continued

H-2-S6800

Capacity:

$$a = \frac{A_s f_y}{.85 f'_c b} = 1.15''$$

$$M_n = A_s f_y (d - \frac{a}{2})$$
$$= 243 \text{ in-kip}$$

Allowable Uniform Load:

$$M_n > M_u = \frac{w l^2}{8}$$

$$243 = \frac{w (6.8')^2}{8} \cdot 12$$

$$w = 3.5 \text{ ksf}$$

$$w_{LL} = 3.5 - .125 - 1.8$$

$$= 1.57 \text{ ksf}$$

Reduce w by $\frac{9.8}{11.9}$

$$w_{LL} = .96 \text{ ksf}$$

$$A_s = .88 \text{ si/\# width}$$
$$f'_c = 3000 \quad f_y = 40000$$
$$b = 12'' \quad d = 7.5''$$

$$\text{Conc. wt} = .125 \text{ ksf}$$

$$\text{Soil wt.} =$$

$$\frac{125 \text{ psf} \times 14.5'}{1000} = 1.8 \text{ ksf}$$

check shear:

$$2 \sqrt{f'_c} b d = 9.8 \text{ kips}$$

$$\frac{3.5 (6.8')}{2 \text{ ends}} = 11.9 \text{ kip} > 9.8 \text{ k}$$

Client WHC

WO/Job No. _____

Subject _____

Date 5-3-96 By RA Giller

Checked 5/20/96 By MA ISLAM

Location _____

Revised _____

By _____

241-AX-152 Pump Pit, Diverter Pit, & Catch Tank

H-2-44580

81

82

83

Cover Blocks

DESIGN ANALYSIS

Client WHC

WO/Job No. _____

Subject _____

Date 5-6-96 By KA GillerChecked 5/20/96 By MA ISLAM

Location _____

Revised _____ By _____

241-AX-152

Capacity:

$$a = \frac{A_s F_y}{0.85 f'_c b} = .863$$

$$M_n = A_s F_y (d - \frac{a}{2})$$

$$I = 4475 \text{ in-k}$$

4- #6 T & B

$$A_s = 1.76$$

$$f'_c = 3000 \quad F_y = 40000$$

$$b = 32" \quad d = 64"$$

Allowable Conc. Load:

$$M_n > M_u = \frac{w l^2}{8} + \frac{P l}{4}$$

$$4475 = \left[\frac{850(6)^2}{8} + \frac{P(6)}{4} \right] \frac{12}{1000}$$

$$372917 = 3875 + 1.5P$$

$$P = 246 \text{ K}$$

$$w = 850 \text{ psf}$$

$$l = 6'$$

Check Shear:

$$21 f'_c b d = 224 \text{ K}$$

O.K.

Reduce $P = 224 \text{ K}$

$$w = 14.9$$

DESIGN ANALYSIS

Client WHC

WO/Job No. _____

Subject _____

Date 5-16-96 By RA Giller

Checked 5/20/96 By MA ISLAM

Location _____

Revised _____ By _____

241-A2-KS1 Pump Pit, Catch Tank

H-2-68352

H-2-68368

H-2-68316

Client WHC

WO/Job No. _____

Subject _____

Date 5-16-96 By RA GillerChecked 5/20/96 By MA ISLAM

Location _____

Revised _____ By _____

241-AZ-1S1 Cover Block

Capacity :

$$a = \frac{A_s F_y}{0.85 f'_c b} = .646$$

$$M_n = A_s F_y (d - \frac{a}{2})$$

$$I = 3343 \text{ in-k}$$

7- #5 bars

$$A_s = 2.17$$

$$f'_c = 3000 \quad F_y = 6000$$

$$d = 26" \quad l = 6.5'$$

Check Shear

$$2 \sqrt{f'_c} b d = 225 \text{ k}$$

O.K.

$$M_n > M_u = \frac{w l^2}{8} + \frac{P l}{4}$$

$$3343 = \left[\frac{375(6.5)^2}{8} + \frac{P(6.5)}{4} \right] \frac{12}{1000}$$

$$278583 = 1980 + 1.625 P$$

$$\boxed{170 \text{ k} = P}$$

$$\boxed{3.9 \text{ ksf} = w}$$

DESIGN ANALYSIS

Client WHC

WO/Job No.

Subject

Date 5-16-96 By RA Giller
Checked 5/22/96 By MA ISLAM

Location

Revised By

241-AZ-151 Catch Tank

Capacity:

$$a_{top} = \frac{(0.31)(60)}{.85(3)(12)} = .608''$$

$$M_{n_{Top}} = A_s F_y (d - \frac{a}{2}) = 171 \text{ in-k}$$

$$a_{bot} = \frac{(0.2)(60)}{.85(3)(12)} = .392''$$

$$M_{n_{bot}} = A_s F_y (d - \frac{a}{2}) = 111 \text{ in-k}$$

Allowable Dist. Load:

$$M_n > M_u = \frac{w l^2}{12} + \frac{P l}{8} \text{ (Fixed Ends)}$$

$$171 = \left[\frac{2150(6)^2}{12} + \frac{P(6)}{8} \right] \frac{12}{1000}$$

$$14250 = 6450 + .75 P$$

$$10400 \text{ \#/unit width} = P$$

$$10.4 \times 6' = 62 \text{ kip} = P$$

$$1.7 \text{ ksf} = W$$

$$A_{s_{Top}} = .31$$

$$A_{s_{bot}} = .2$$

$$f'_c = 3000 \quad f_y = 60000$$

$$d = 9.5'' \quad l = 6'$$

$$W = 150 + 16' \times 125 \\ = 2150$$

check shear

$$2 \sqrt{f'_c} b d = 74928 \text{ \#}$$

DESIGN ANALYSIS

Client WHL

WO/Job No. _____

Subject _____

Date 5-16-96 By RA Giller

Checked 5/20/96 By MA ISLAM

Location _____

Revised _____ By _____

241-AZ-154 Pump Pit, Catch Tank

H-2-68352

H-2-68367

H-2-68315

DESIGN ANALYSIS

Client WHC

WO/Job No.

Subject

Date 5-16-96 By RA Giller

Checked 5/20/96 By MA ISLAM

Location

Revised

By

241-AZ-154

Capacity :

$$a = \frac{AF_y}{.85f_c' b} = .762''$$

$$M_n = A_s F_y (d - a/2)$$

$$= 3335 \text{ in-k}$$

#5 @ 12"

7- #5 bars

 $A_s = 2.17$ $f_c' = 3000 \quad F_y = 60000$

$$M_n > M_u = \frac{w l^2}{8} + \frac{P l}{4}$$

$$3335 = \left[\frac{375(5.5)^2}{8} + \frac{P(5.5)}{4} \right] \frac{12}{1000}$$

$$w = 375 \text{ psf}$$

$$d = 26'' \quad l = 5.5'$$

$$277917 = 1418 + 1.375P$$

$$201 \text{ kips} = P$$

Check Shear

$$27f_c' b d = 191 \text{ kips}$$

O.K.

$$\text{Reduce } P = 191 \text{ kips}$$

$$w = 6 \text{ ksf}$$

DESIGN ANALYSIS

Client _____

WO/Job No. _____

Subject _____

Date 5-7-96 By RA Giller

Checked 5/20/96 By MA ISLAM

Location _____

Revised _____ By _____

241-EW-151 Vent Station Diversion Box

H-Z-43148
S3
S4

Client WHC

WO/Job No.

Subject

Date 5-7-96 By RA Giller

Checked 5/20/96 By MA ISLAM

Location

Revised By

241-EW-1S1

Capacity:

$$a = \frac{A_s F_y}{.85 f_c' b} = .812$$

$$M_n = A_s F_y (d - \frac{a}{2})$$

$$= 2069 \text{ in-k}$$

Cover Block

6-#6

$$A_s = 2.64$$

$$f_c' = 3000 \quad F_y = 40000$$

$$b = 51" \quad d = 20"$$

Concentrated Load:

$$M_n > M_u = \frac{w l^2}{8} + \frac{P l}{4}$$

$$2069 = \left[\frac{300(7)^2}{8} + \frac{P(7)}{4} \right] \frac{12}{1000}$$

$$172417 = 1838 + 1.75 P$$

$$\boxed{97 \text{ kips} = P}$$
$$\boxed{3 \text{ ksf} = w}$$

$$w = .300 \text{ psf}$$

$$l = 7'$$

Clear Shear:

$$\frac{2 \sqrt{f_c'} b d}{1000} = 111.7 \text{ kips}$$

Client WHC
Subject _____

WO/Job No. _____
Date 5-7-96 By RA Giller
Checked 6/20/96 By MA ISLAM
Revised _____ By _____

Location _____

241-S-304 Catch Tank

H-2-85019-36

Client WHC
Subject _____WO/Job No. _____
Date 5-6-96 By RA Griller
Checked 5/20/96 By MA ISLAM
Revised _____ By _____

Location _____

241-S-304 Catch Tank

Capacity :

$$a = \frac{A_s F_y}{.85 f_c' b} = .862$$

$$M_n = A_s F_y (d - a/2)$$
$$= 2067 \text{ in} \cdot \text{lb}$$

4- #6 bars

$$A_s = 1.76$$

$$f_c' = 3000 \quad F_y = 60000$$

$$b = 48 \quad d = 20$$

Concentrated Loads :

$$M_n > M_u = \frac{w l^2}{8} + \frac{P l}{4}$$

$$2067 = \left[\frac{300(13)^2}{8} + \frac{P(13)}{4} \right] \frac{12}{1000}$$

$$172250 = 6337 + 3.25 P$$

$$51 \text{ kips} = P$$

$$1.0 \text{ ksf} = w$$

$$w = 300 \text{ psf}$$

$$l = 13'$$

Check Shear :

$$\frac{2 \sqrt{f_c'} b d}{1000} = 105 \text{ kips}$$

O.K

Client WHC
Subject _____

WO/Job No. _____
Date 5-6-96 By RA Giller
Checked 5/20/96 By MA ISLAM
Revised _____ By _____

Location _____

241-TX-302C, Pump Pit, Catch Tank

241-UX-302A, Pump Pit, Catch Tank

H-2-71643
660
665

H-2-833
835
836

9'φ, 36' long, 1/2" thick
SST - 18-8C

Client WHC

WO/Job No.

Subject

Date 5-6-96 By RA Giller

Checked 5/20/96 By MA ISLAM

Location

Revised By

241-TX-302C, 241-UX-302A

Capacity :

$$F_y = 36,000 \text{ psi}$$

Uniform Load :

Roark 6th Ed pg 429

$$M = \frac{qa^2(3+\nu)}{16}$$

$$\sigma = \frac{6M}{t^2}$$

$$36000 = \frac{6qa^2(3+\nu)}{16t^2}$$

$$23.7 \text{ psi} = q$$

$$3420 \text{ psi} = q$$

Concentrated Load: 6" ϕ circle

$$M = \frac{W}{4\pi} \left[(1+\nu) \ln \frac{a}{r} + 1 \right]$$

$$M = .861 W$$

$$\sigma = \frac{6M}{t^2} = \frac{.861 W (6)}{t^2}$$

$$36000 = 5.166 W$$

$$6968 \text{ lb} = W$$

ASTM A36 steel

$$\nu = .3 \quad F_y = 36000 \text{ psi}$$

$$t = 1" \quad a = 35"$$

$$r = 3" \\ a = 35"$$

DESIGN ANALYSIS

Client WHC

WO/Job No. _____

Subject _____

Date 5-7-96 By RA Giller

Checked 5/20/96 By MA ISLAM

Location _____

Revised _____ By _____

241-U-301-B Settling Tank, Pump Pit

H-2-71653

Client WHC

WO/Job No.

Subject

Date 5-7-96 By RA GillerChecked 5/20/96 By NA ISLAM

Location

Revised By

241-u-301-B Settling Tank(Roark, 6th Ed, pg 547, Fig 3b) #4 @ 6" O.CConcentrated Load P .Edge Moment M_0

$$M_0 = -B_2 \frac{P}{4\pi}$$

$$\alpha = 2 \left[\frac{3(1-\nu^2)h^2}{t^2} \right]^{1/4} = 7.7$$

$$B_2 = -.07 \text{ @ } \alpha = 7.7$$

$$M_0 = -.0056 P$$

$$F_y = 60000 \quad f_c' = 2500$$

$$t = 4" \quad h = 3' = 36"$$

$$\nu = .3 \quad d = 8" \text{ @ edge}$$

$$a = \frac{A_s F_y}{.85 f_c' b} = \frac{.4(60)}{.85(2.5)(12)} = .941$$

$$M_n = A_s F_y \left(d - \frac{a}{2} \right) = \frac{(.4)(60000) \left(8 - \frac{.941}{2} \right)}{12} = 15059 \text{ in-lb/ft}$$

$$M_n > M_0$$

$$15059 = .0056 P$$

$$2,689,107 \text{ lb} = P$$

$$2,689 \text{ kip} = P$$

$$\text{uniform Load} = \frac{P}{\pi R^2} = 7763 \text{ psf}$$

Client _____

WO/Job No. _____

Subject _____

Date 5-7-96 By RA Giller

Checked 5/20/96 By MA ISLAM

Location _____

Revised _____ By _____

244 A Lift Station (DCRT)

H-2-38203

38206

DESIGN ANALYSIS

Revision _____

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 Client WHC

WO/Job No. _____

Subject _____

 Date 5-2-96 By RA Giller

 Checked 5/20/96 By MA ISLAM

Location _____

Revised _____

By _____

244-A Lift Station (DCRT)
H-2-38203-6

Capacity :

$$a = \frac{A_s F_y}{.85 f_c' b} = .986''$$

$$M_n = A_s F_y (d - \frac{a}{2})$$

$$= 2456 \text{ in-k}$$

6- #6 T & B

 $A_s = 2.64$
 $f_c' = 3000 \quad F_y = 60000$
 $b = 63'' \quad d = 16''$

Find allowable conc. load :

$$M_n > M_u = \frac{1.0 w l^2}{8} + \frac{1.0 P l}{4}$$

$$2456 = \left[\frac{250(13)^2}{8} + \frac{P(13)}{4} \right] \frac{12}{1000}$$

$$204667 = 5281 + 3.25 P$$

$$P = 61 \text{ kips}$$

$$w = .9 \text{ ksf}$$

48

$$w = 250 \text{ psf}$$

$$l = 13'$$

Check Shear :

$$\frac{2 \sqrt{f_c'} b d}{1000} = 110 \text{ kips}$$

O.K.

Client WHC
Subject _____

WO/Job No. _____
Date 5-3-96 By RA Giller
Checked 5/20/96 By MA ISLAM
Revised _____ By _____

Location _____

241-A-350 Lift Station (DCRT)

H-2-70318
70538

DESIGN ANALYSIS

Calc. No. _____
Revision _____
Page No. _____ of _____

Client WHC

WO/Job No.

Subject

Date 5-2-96 By RA Giller

Checked 5/20/96 By MA ISLAM

Location

Revised By

Z41-A-350 Lift Station (DCRT)

H-2-70318, 70538

Capacity:

$$a = \frac{A_s F_y}{.85 f_c' b} = .629''$$

$$M_n = A_s F_y (d - \frac{a}{2})$$
$$= 1459 \text{ in-k}$$

5- #5 bar T & B

 $A_s = 1.55$ $f_c' = 3000 \quad f_y = 60,000$ $b = 58'' \quad d = 16''$

Allowable conc. load:

$$M_n > M_u = \frac{1.0 w l^2}{8} + \frac{1.0 P l}{4}$$

$$1459 = \left[\frac{250(9)^2}{8} + \frac{P(9)}{4} \right] \frac{12}{1000}$$

$$121583 = 2531 + 2.25 P$$

$$P = 53 \text{ kips}$$

$$w = 1.2 \text{ ksf}$$

$$w = 250 \text{ psf}$$

$$l = 9'$$

Check Shear:

$$\frac{27 f_c' b d}{1000} = 101.6 \text{ kip}$$

O.K.

DESIGN ANALYSIS

Client	WHC		WO/Job No.	
Subject			Date	5-3-96 By RA Giller
			Checked	5/20/96 By MA ISLAM
Location			Revised	By

244 BX, TX, U (DCRV)

H-2-73785 thru 73797

DESIGN ANALYSIS

Client WHC

WO/Job No.

Subject

Date 5-2-96 By RA Giller

Checked 5/20/96 By MA ISLAM

Location

Revised

By

244 BX, TX, U Salt Well Receiver Vessel (DCRT)

H-2-73785-97

Capacity :

$$a = \frac{A_s F_y}{.85 f'_c b} = 3.10''$$

$$M_n = A_s F_y (d - a/2)$$

$$= 5247 \text{ in-k}$$

Find allowable conc. load :

$$M_n > M_u = 1.0 \frac{w l^2}{8} + 1.0 \frac{P l}{4}$$

$$5247 = \left[\frac{300(18)^2}{8} + \frac{P(18)}{4} \right] \frac{12}{1000}$$

$$(437250 = 12150 + 4.5 P)$$

$$P = 94.5 \text{ kips}$$

Reduce P to 70 kips

$$w = 1.3 \text{ ksf}$$

52

6- #8 T 4B

$A_s = 4.74 \text{ si}$

$f'_c = 3000 \quad f_y = 60000$

$b = 36'' \quad d = 20''$

$w = 300 \text{ psf}$

$l = 18'$

Check Shear :

$$\frac{27 f'_c b d}{1000} = 79 \text{ kips}$$

DESIGN ANALYSIS

Client	WHC	WO/Job No.	
Subject		Date	5-2-96 By KA Giller
		Checked	5/20/96 By MA ISLAM
Location		Revised	By

244 CR Process Tank Vault (DCRT)

H-2-41530
41982
41983
41889

DESIGN ANALYSIS

Client WHCWO/Job No. Subject Date 5-2-96 By RA GillerChecked 5/28/96 By MA ISLAMLocation Revised By 244-CR Process Tank Vault (DCRT)

H-Z-41530

41982

41983

41889

Capacity :

$$a = \frac{A_s F_y}{.85 f'_c b} = .746''$$

$$M_n = A_s F_y (d - \frac{a}{2})$$

$$= 1407 \text{ in-k}$$

4 #6 bars T & B

$$A_s = 1.76 \text{ in}^2$$

$$f'_c = 3000 \quad f_y = 40,000 \text{ psi}$$

$$b = 37'' \quad d = 20''$$

Allowable Conc. Load :

$$M_n > M_u = \frac{10 w l^2}{8} + \frac{1.0 P l}{4}$$

$$1407 = \left[\frac{300 (8.6)^2}{8} + \frac{P (8.6)}{4} \right] \frac{12}{1000}$$

$$(117250 = 2774 + 2.15 P)$$

$$P = 53 \text{ kips}$$

$$w = 2 \text{ ksf}$$

$$w = 300 \text{ psf}$$

$$l = 8.6'$$

Check Shear:

$$\frac{2 \sqrt{f'_c} b d}{1000} = 81 \text{ kips}$$

O.K.

DESIGN ANALYSIS

Client WHL

WO/Job No.

Subject

Date 5-2-96 By KA 6-11-96

Location

Checked 5/20/96 By NA ISLAM
Revised _____ By _____

244 S Catch station (DLRT)
H-2-71043 thru 71046

DESIGN ANALYSIS

Client WHC

WO/Job No. _____

Subject _____

Date 5-2-96 By RA Giller

Checked 5/20/96 By MA ISLAM

Location _____

Revised _____

By _____

244 S Catch Station (DCRT)

H-2-71043-46

Capacity:

$$a = \frac{A_s F_y}{.85 f_c' b} = .863''$$

$$M_n = A_s F_y (d - \frac{a}{2})$$

$$= 2067 \text{ in-k}$$

4 - #6 bar Top & Bottom

$$A_{s \text{ TOP}} = A_{s \text{ BOT}} = 1.76 \text{ si}$$

$$f_c' = 3000 \text{ psi} \quad f_y = 60000 \text{ psi}$$

$$b = 48'' \quad d = 20''$$

Find allowable concentrated Load:

$$M_n > M_u = 1.0 \frac{w l^2}{8} + 1.0 \frac{P l}{4}$$

$$w = 300 \text{ psf}$$

$$l = 20'$$

$$2067 = \left[\frac{300(20)^2}{8} + \frac{P(20)}{4} \right] \frac{12''}{1000}$$

Check Shear:

$$(172250 = 15000 + 5P)$$

$$\frac{2 \sqrt{f_c' b d}}{1000} = 105 \text{ kips}$$

$$P = 31 \text{ kips}$$

$$w = .38 \text{ ksf}$$

O.K.

DESIGN ANALYSIS

Client _____

WO/Job No. _____

Subject _____

Date 5-7-96 By RA Giller

Checked 5/20/96 By MA ISLAM

Location _____

Revised _____ By _____

260 Series SST

HW-72742

(Ramble Report)

Existing calculations
(SD-RE-TI-012) shows
this tank roof is good
for a load of 422 kips

Max dead (soil) load
for this tank is 95 kips

Allowable live load is

$$422 - 95 = 327 \text{ kips}$$

$$w = 1 \text{ ksf}$$

Appendix 2

Calculations for Impact Energy from Dropped Equipment

The effect of the impact load on the integrity of the containment for the tanks in the DCRTs, Catch Stations, and 200 Series SST was evaluated. The impact load may occur from the falling equipment anywhere on the tanks during the postulated accident scenarios. The following impact load conditions were evaluated:

- A. Impact energy required to perforate the cover block for the tank in DCRTs and the Catch Stations.
- B. Impact energy required to cause a bending failure in the cover block or in the roof when impact is associated with a non-rigid surface (cover block, concrete pit, riser etc.).
- C. Same as in B. except when impact is associated with a yielding surface (ground).

The perforation or the bending failure in the cover block or in the roof will lead to develop an opening in the containment for the tanks in DCRTs, Catch Stations, and 200 Series SSTs.

The impact response of structures is characterized by intense dynamic stresses that produce crushing, cratering, shear failure, and tensile fractures in highly nonhomogeneous materials. Because of the complex nature of this response, detailed analyses by means of computational mechanics have not been developed sufficiently to apply in the design of the structures to sustain impact.

This evaluation determined the capacity of the cover block and the roof of the tank vault to resist perforation and fracture. The weight and shape of the falling equipment were assumed to be 20,000 pounds and 2 feet diameter respectively. These data along with other assumed values for the concrete, reinforcement, and soil were used throughout the evaluations. For the evaluation of the perforation, 20-in thick cover block was used. This thickness was smallest of all the cover blocks in DCRTs and Catch Stations. Because 200 Series SSTs do not have cover blocks, perforation was not considered a reality. For the impact with the non-rigid surface, the capacities of the cover blocks to resist fracture was determined in groups depending on depth, reinforcement, and span. The stiffness of the cover block also played a key role in the determination of the impact capacity.

For the impact with the soft ground (yielding), the analysis is different. The idea underlying the scenario with the soft ground is to determine the impact force by solving two simultaneous equations. These are equations of

motion and soil reaction. The solutions of these simultaneous equations determine the depth of penetration of the falling equipment in ground and the impact force. This impact force was used to evaluate the integrity of the tank roof and the vault wall.

The results of the evaluations indicate that the perforation of the cover block from the falling equipment is not realistic. The energy as high as 29,000,000 inch-pounds would be needed to perforate a 20 inch thick cover block if a 20,000 pound equipment is dropped from a height of 120 feet.

On the other hand, the results of the evaluations indicate that the cover blocks do not have adequate bending strength to resist impact. A failure of the cover block leading to an opening is most likely to occur due to the energy from the falling of a 20,000 lb equipment from any height. The results are tabulated in page 2 of this evaluations.

The impact with the soft ground was considered for three drop heights from 10 ft to 20 ft. The results show that the vault wall and roof for the DCRTs, catch tanks and the 200 series SSTs are adequate to withstand the impact. These results are tabulated along with the result of the perforation in page 1 of this evaluation.

Calc. No. _____
 Revision 0 _____
 Page No. i of ii

DESIGN ANALYSIS

Client WHC	WO/Job No.	F6N1A1
Subject Evaluation for Impact Energy	Date 5/7/96	By M.A. Islam
for DCRTs, Catch Tanks, and	Checked S-20	By RA Giller
200 Series SST		
Location 200 Area Gen.	Revised	By

Scope

Evaluate the impact energy required to cause an opening in the cover block for DCRTs and Catch Tanks and roof of 200 Series SSTs. The evaluation will specifically address the following:

- Energy required to perforate the cover block.
- Energy required to cause a bending failure in the cover block or in the roof when impact is associated with a non-yielding surface (cover block, concrete pit, riser etc.).
- Energy required to cause a bending failure in the roof of 200 Series SST when impact is associated with a yielding surface (ground).

The perforation or the bending failure in the cover block or in the roof will lead to develop an opening in the containment for the tanks in DCRTs, Catch Stations, and 200 Series SST.

Material Properties

Concrete compressive strength, $f'_c = 3,000$ psi;

Reinforcement, $f_y = 60,000$ psi

Calc No. _____
 Revision 0
 Page No. ii of ii

DESIGN ANALYSIS

Client WHC	WO/Job No. F6N1A1
Subject Evaluation for Impact Energy	Date 5/7/96 By M.A. Islam
for DCRTs, Catch Tanks, and	Checked 5/20/96 By R.A. Giller
200 Series SST	
Location 200 Area Gen.	Revised By

References

1. Design of Structures for Missile Impact, BC-TOP-9-A, Revision 2, Bechtel Power Corporation, November 1974.
2. ACI Building Code, ACI 318-89.
3. Design of Welded Structures, Chapter 2, O.W. Blodgett, June 1966.
4. Supporting Document, WHC-SD-WM-SAR-067, Revision A.
5. Single-shell Waste Tank Load Sensitivity Study, SD-RE-TI-012, March 1983.
6. Design of Machine Elements, Chapter 12, Second Edition, M.F. Spotts.
7. Structural Engineering Handbook, Chapters 5, Gaylord & Gaylord 1968 Edition.
8. Introductory Soil Mechanics and Foundations, 3rd Edition, Sowers & Sowers.
9. Hanford Plant Standards, GC-LOAD-01, (Formerly SDC-4.1, Rev 12).
10. Roark's Stress and Strain, 4th Edition

Conclusion

See pages 1 and 2 for the summary of the evaluations.

Client WHC

WO/Job No.

Subject Evaluation of Impact Energy Date 5/15/96 By M.A. ISLAM
DCRTs, Catch Tank & 200 Series SSR Checked 5-20 By R.A. Giller
Location 200 Area Revised By

SUMMARY OF EVALUATIONSA. Impact Energy Required to perforate Cover Block

Tank Data		Missile Data		Energy Req'd. for perforation	
Tank	Cover Depth (in)	Weight (lbs)	Diameter (in)	Height of Fall (h) (in)	Energy, E $E = Wh$ (in-lbs)
241-A-350 (DCRT)	20	20,000	24	1440 (120 FT)	28,800,000

B. Impact with Yielding Surface (Ground)

Tank Data		Analysis Data (Assumed)				Result	Remarks
Tank	Soil/Cover Depth (in)	Missile Weight W (lbs)	Height of Fall h (in)	Soil Dusty γ (% ϕ)	Soil Friction Angle, ϕ (deg.)	Force @ Impact F_i (lbs)	
200 Series SSR 241-B,C,T,U	144 (12 FT)	20,000	120	125	30	55,897	74.5 F _i dissipates significantly as it approaches Tank roof. Therefore, Tank roof will maintain its integrity...

Client	WHC	WO/Job No.	
Subject	Evaluation of Impact Energy	Date	By M.A. ISLAM
		Checked S-20-	By R.A. Giller
Location	200 Area	Revised	By

SUMMARY OF EVALUATIONS CONTD.

C. Impact with non-yielding surface e.g. cover block, pit, riser etc

DCRTs

* using an equipment weight, $W = 20,000$ lbs

Group	Tank Data				* Energy Req'd. to Cause Bending Failure in Cover Block. (in-lb)
	Listed Tanks	Cover Block Depth (in)	Span (in)	Rebar Area (in ²)	
1	241-A-350	20	108	#5 @ 12 $A_s = .31/H$	None
2	244-TX, 244-U 244-BX	24	216	#8 @ 6 $A_s = 1.58/H$	None
3	244-CR-003	24	53	#6 @ 12 $A_s = .44/FH$	None
4	244-A, 244-S	24	252	#6 @ 12 $A_s = .44/H$	None

CATCH TANKS

1	241-TX-302C, 241-UX-302A 241-A-302A, 241-ER-311 241-EW-151	20	72	#5 @ 12 $A_s = .31/FH$	None
2	241-AX-152	68	84	#6 @ 6, $A_s = .88$	None
3	241-S-304	30	130	#6 @ 12, $A_s = .44$	None
4	241-A-417	2 @ 15=30	96	#6 @ 12, $A_s = .132$ (3 #4 @ 6)	None

200 Series SST	241-B, C, T, U	12	240	#6 @ 12 $A_s = .44/F$	None
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Client	WHC	WO/Job No.	
Subject	Evaluation of Impact Energy DCRTs	Date	5/7/96 By M.A. ISLAM
Location	200 Area	Checked	5-20 By R.A. Giller
		Revised	By

Double Contained Receiver Tanks (DCRTs)

The following is a list of DCRTs :

**** 244-S Lift station
** 244-TX Catch Station
** 244-U " " } 200 W Area

Ref.
WHC-SD-WM-SAR-
067, Rev A

**** 244-A Lift station
** 244-BX Catch Station
*** 244-CR-003 Vault Tank
* 241-A-350 Lift station } 200 E Area

Client WHC

WO/Job No.

Subject Evaluation of Impact Energy
DCRTs

Date 5/7/96 By M. A. ISLAM

Checked S-20 By RA Giller

Location ZDD Area

Revised

By

Determine Energy Required to perforate Cover block h = height of fall W = weight of falling equipment,
assume 20,000 lbsAssumed diameter of perforation, $D = 24$ inRefer to "Design of structures for Missile Impact
BC-TOP-9-A, Rev 2" by Bechtel Power Corp.
Nov. 1974The thickness of a concrete element that will be perforated
by a missile (falling object)

$$T = \frac{427}{\sqrt{f'_c}} \cdot \frac{W}{D^{1.8}} \cdot \left(\frac{V_s}{1000} \right)^{1.33} \quad \text{where } f'_c = 3000 \text{ Psi}$$

V_s = striking Velocity,
ft/sec

$$\text{Velocity, } V_s = \sqrt{2gh} = \sqrt{2 \times 32.2 h} = 8.025 h^{1/2}$$

$$V_s^{4/3} = (8.025)^{4/3} \cdot h^{1/2 \cdot 4/3} = 16.07 (h)^{2/3}$$

$$\therefore T = \frac{427}{\sqrt{3000}} \cdot \frac{W}{(24)^{1.8}} \cdot \frac{16.07 (h)^{2/3}}{(1000)^{1.33}} = 0.00041067 W (h)^{2/3}$$

$$W h^{2/3} = \frac{20}{0.00041067} = 486,998$$

$$\text{if } W = 20,000 \text{ lbs, } h = 120 \text{ Ft}$$

Therefore, Energy reqd, $E = Wh = 20000 \times 120 \times 12 = 28,800,000$

to cause perforation of the cover block

DESIGN ANALYSIS

Client WAC

WO/Job No.

Subject Evaluation of Impact Energy Date 5/8/96 By M. A. ISLAM
DCRTS VJ Checked 5-20 By R A Giller
Location 200 Area DCRTs, Catch Tanks Revised By

The results of the evaluation for the perforation of the cover block indicate that a 20,000 lb equipment, if dropped from a height of 120 Feet, the cover block will be perforated with a hole diameter of 2 Feet.

20,000 lb equipment, 2 Ft. dia. perforation are realistic, however height of 120 Ft. is unrealistic. Therefore, the results conclude that the cover block, under normal impact scenario, will not be perforated.

Determine Energy Req'd. to cause a crack (opening) in cover block

Moment Capacity of the cover block :

Locate Concrete compression block,

$$a = \frac{A_s f_y}{.85 \times b \times f_c} = \frac{.31 \times 60000}{.85 \times 12 \times 3000} = .6078"$$

Ultimate Moment Capacity, M_u

$$M_u = \phi A_s f_y (d - a/2) = 1 \times .31 \times 60000 (17.5 - .6078/2) = 319,848 \text{ in-lb/Ft}$$

Find Bending moment to be developed by the fall of W,

$$W_{DL} = \frac{20}{12} \times 150 \times 1 = 250 \text{ \#/ft}; M_{DL} = \frac{W_{DL}^2}{8} = \frac{250 \times 9^2}{8} \times 12 = 30,375 \text{ in-lb}$$

Max. bending Moment, M_u due to P & DL

$$M_u = \frac{PL}{4} + M_{DL} \text{ or } P = \frac{4(M_u - M_{DL})}{L}$$

$$P = \frac{4(319848 - 30375)}{9 \times 12} = 10,721 \text{ \#/Ft of width}$$

where P is the impact load caused by the fall of W from a height of h

Client	WTC	WO/Job No.	
Subject	Evaluation of Impact Energy	Date	5/8/96
	DCRTS	By	M. A. ISLAM
		Checked	5-20
		By	R A Giller
Location	200 Area DCRTS, Catch Tanks	Revised	
		By	

$$\therefore P = 10,721 \text{ \# (Impact load)}$$

Refer to Blodgett's "Design of Welded Structures", chapter 2

$$\text{Impact Load, } P = W + \sqrt{W^2 + 2KW(h)} \text{ where } K \text{ is bending stiffness of the cover block}$$

Because cover block is simply supported and loaded by P ,

$$K = \frac{48EI}{L^3} \text{ where } E = 3 \times 10^6 \text{ Psi for concrete}$$

$$\begin{aligned} \therefore K &= \frac{48 \times 3 \times 10^6 \times 8,000}{108^3} & I &= \frac{bd^3}{12} = \frac{12 \times 20^3}{12} \\ &= 914,495 \text{ \#/in} & &= 8,000 \text{ in}^4 \end{aligned}$$

$$\therefore P = 10,721 = W + \sqrt{W^2 + 2KW(h)}, \text{ using } W = 20,000 \text{ \#}$$

$$20,000 + \sqrt{20,000^2 + 2 \times 914,495 \times 20,000(h)} = 10,721$$

$$\sqrt{20,000^2 + 2 \times 914,495 \times 20,000(h)} = -9,279$$

$$h = -0.009 \text{ in}$$

$\therefore$ Energy, E needed for failure is insignificant

conclusion: Because bending strength ($M_u @ A_g = 31 \text{ in}^2$) is small, stiffness of cover block (K) is high, very small energy is enough to cause a crack in the cover block. This crack will eventually develop an opening in cover block.

Client WHC WO/Job No. _____
 Subject Evaluation of Impact Energy Date 5/9/96 By M.A. ISLAM
DCRTS Checked 5-20 By R.A. Giller
 Location 200 Area Revised _____ By _____

By comparison to analysis for Tank 241-A-350 perforation of 24 in thick is not practical scenario. Therefore analysis is only shown to determine energy at which cover block fails.

Determine Energy Req'd. to cause a crack in Cover block #8 @ 6"

Similarly, $a = \frac{A_s f_y}{.85 b f_c'} = \frac{1.58 \times 60}{.85 \times 12 \times 3} = 3.098'$ $A_s = .79 \times 2 = 1.58 \text{ in}^2$

$d = 24 - 1\frac{1}{2} - 1 = 21.5'$ $w_{DL} = \frac{24}{12} \times 150 = 300 \text{ #/ft}$

$M_u = 1. \times 1.58 \times 60000 (21.5 - 3.098/2) = 1,891,355 \text{ #ft}$

$M_{DL} = w_{DL} L^2 / 8 = 300 \times 18^2 / 8 \times 12 = 145,800 \text{ in-lb}$

Again, $M_u = \frac{PL}{4} + M_{DL}$

$\therefore P = \frac{4(M_u - M_{DL})}{L} = \frac{4(1,891,355 - 145,800)}{18 \times 12} = 32,325 \text{ #/ft of width}$

$I = \frac{12 \times 24^3}{12} = 13,824 \text{ in}^4$

$K = \frac{48 \times 3 \times 10^6 \times 13,824}{(18 \times 12)^3} = 197,531 \text{ #/in}$

Client WHC

WO/Job No.

 Subject Evaluation of Impact Energy Date 5/9/96 By M.A. ISLAM
DCRT6 Checked 5-20 By R.A. Giller
 Location 200 Area Revised _____ By _____

$$\therefore P = 32,325 = W + \sqrt{W^2 + 2KWh}, \text{ Again using } W = 20000^\#$$

$$20000 + \sqrt{20000^2 + 2 \times 197531 \times 20000 (h)} = 32,325$$

 $h = \text{Negative}$
 $\therefore$ Energy needed is insignificant to cause failure

Conclusion: Insignificant energy will cause failure in cover block and eventually develop an opening in cover block.

DCRT 244-CR SEC. A-A

* conservative

PLAN
Determine Energy Req'd. to cause a crack in Cover block

$$a = \frac{.44 \times 60}{.85 \times 12 \times 3} = .863 \text{ in}; d = 24 - 2.5 = 21.5$$

$$M_u = 1.0 \times .44 \times 60000 \left(21.5 - \frac{.863}{2} \right) = 556,209 \text{ in-lb}$$

$$W_{DL} = \frac{24}{12} \times 150 = 300 \text{ #/F}$$

$$M_{DL} = \frac{W_{DL} L^2}{8} = \frac{300 \times 4.42^2}{8} \times 12 = 8791 \text{ #-in}$$

Client	WHC	WO/Job No.	
Subject	Evaluation of Impact Energy	Date	5/9/96
	DCRTs	By	M. A. ISLAM
Location	200 Area	Checked	5-20
		By	R. A. Giller
		Revised	
		By	

$$\therefore P = \frac{4(M_L - M_{BL})}{L} = \frac{4(556209 - 87912)}{4.42 \times 12} = 41,283 \text{ \#}$$

$$I = \frac{12 \times 24^3}{12} = 13,824 \text{ in}^4$$

$$K = \frac{48 \times 3 \times 10^6 \times 13,824}{(4.42 \times 12)^3} = 13.37 \times 10^6 \text{ \#/in}$$

$$\therefore P = 41,283 = W + \sqrt{W^2 + 2KWh}, \text{ Again using } W = 20000 \text{ \#}$$

$$20000 + \sqrt{20000^2 + 2 \times 13.37 \times 10^6 \times 20000(h)} = 41,283$$

$$h = 0.000099 \text{ negligible}$$

$$\therefore \text{Energy, } E = 20000 \times 0.0001 \approx 0 \text{ in-lb}$$

Conclusion: Same conclusion as in Tank 241-A-350.

Client	WHC	WO/Job No.	
Subject	Evaluation of Impact Energy	Date	5/10/96
	DCRTS	By	M. A. ISLAM
		Checked	S. Z
Location	ZOO Area	Revised	By

cover block
(TYP)

PLAN

$$a = \frac{44 \times 60}{185 \times 12 \times 3} = 1.863' ; d = 21.5'$$

$$M_u = 1 \times 44 \times 60000 (21.5 - \frac{1.863}{2}) = 556,209 \text{ in.}$$

$$M_{BL} = 300 \times \frac{21^3}{8} \times 12 = 198,450 \text{ in.}$$

$$P = \frac{4M_u}{L} = \frac{4(556,209 - 198,450)}{21 \times 12} = 5,679 \text{ \#}$$

Similar to Tank 244-CR,

$$I = \frac{12 \times 24^3}{12} = 13,824 \text{ in}^4$$

$$K = \frac{48 \times 3 \times 10^6 \times 13,824}{(21 \times 12)^3} = 124,393 \text{ \#/in}$$

$$P = 42,430 = W + \sqrt{W^2 + 2KW(h)}, \text{ Again using } W = 20000 \text{ \#}$$

$$20000 + \sqrt{20000^2 + 2 \times 124393 \times 20000(h)} = 5,679$$

h = Negative

$\therefore$ Energy, E = Negligible

Conclusion: Insignificant energy will cause failure in cover block and develop an opening eventually.

Client	WHC	WO/Job No.	
Subject	Evaluation of Impact Energy Catch Tanks	Date	5/10/96 By M.A. ISLAM
Location	200 Area DCRTs, Catch Tank	Checked	5-20 By R.A. Giller
		Revised	By

Catch Tanks

The following is a list of Catch Tanks :

*** 241-S-304
* 241-TX 302C
241-U-301B
* 241-UX-302-A } 200W Area

* 241-A-302A
**** 241-A-417
** 241-AX-152
Void ~~241-AZ-151~~
*** 241-AZ-154
* 241-ER-311 } 200E Area

* 241-EW-151 600 Area

Ref.

WHC-SD-WM-SAR-067, Rev A

Catch Tanks 241-TX-302C
241-UX-302A
241-A-302A
241-ER-311
241-EW-151

SECTION A-A
(Section corresponds to 241-A-302)

$$\text{Similarly, } a = \frac{.31 \times 60}{.85 \times 12 \times 3} = .6078$$

$$d = 20 - 2.5 = 17.5$$

$$w_{DL} = 1.4 \times \frac{20}{12} \times 150 = 350 \text{ #/ft}$$

$$M_u = 1. \times .31 \times 60000 \left(\text{ft} \cdot \text{s} - \frac{.6078}{2} \right)^2$$

$$= 319,847 \text{ in} \cdot \text{lb/ft}$$

$$M_{DL} = \frac{250 \times 6^2}{8} \times 12 = 13,500 \text{ in} \cdot \text{lb}$$

73

Client	WHC	WO/Job No.	
Subject	Evaluation of Impact Energy	Date	5/10/96
	Catch Tank	By	M.A. ISLAM
Location	200 Area	Checked	5-20
		By	RA Giller
		Revised	
		By	

$$\text{Impact force, } P = \frac{4(M_u - M_{DL})}{L} = \frac{4(319847 - 13,500)}{6 \times 12} = 17,019 \#$$

$$I = \frac{12 \times 20^3}{12} = 8000 \text{ in}^4$$

$$K = \frac{48 \times 3 \times 10^6 \times 8000}{(6 \times 12)^3} = 3.086 \times 10^6 \#/\text{in}$$

$$\therefore P = 17,019 = W + \sqrt{W^2 + 2KW(h)}, \text{ using } W = 20000 \#$$

$$20000 + \sqrt{20000^2 + 2 \times 3.086 \times 10^6 \times 20000(h)} = 17,019$$

$$h = \text{negative}$$

Insignificant energy needed to cause a failure.

Conclusion: The result is identical to DERT 241-A-350

Catch Tank 241-Ax-152

$$a = \frac{A_s f_y}{0.85 f_c'} = \frac{.88 \times 60}{.85 \times 12 \times 3} = 1.73 \text{ "}; \quad d = 68 - 3 = 65 \text{ "}$$

$$M_u = 1.0 \times .88 \times 60000 (65 - 1.73/2) = 3,386,328 \text{ in-lb}$$

$$W_{DL} = \frac{68}{12} \times 150 = 850 \#/\text{ft}; \quad M_{DL} = 850 \times 7^2/8 \times 12 = 62,475 \text{ in-lb}$$

$$\text{Impact load, } P = \frac{4(M_u - M_{DL})}{L} = \frac{4(3,386,328 - 62,475)}{7 \times 12} = 158,279 \#$$

Client WHC

WO/Job No.

 Subject Evaluation of Impact Energy Date 5/10/96 By M. A. ISLAM
Catch Tank Checked 5-20 By RA Giller
 Location 200 Area Revised _____ By _____

$$I = \frac{12 \times 65^3}{12} = 274,625 \text{ in}^4$$

$$K = \frac{48 \times 3 \times 10^6 \times 274,625}{(7 \times 12)^3} = 66.7 \times 10^6 \text{ #/in}$$

$$P = 158,279 = W + \sqrt{W^2 + 2KW(h)} \text{ using } W = 20000 \text{ #}$$

$$20000 + \sqrt{20000^2 + 2 \times 66.7 \times 10^6 \times 20000(h)} = 158,279$$

$$h = .007 \text{ negligible}$$

Insignificant energy is enough to fail the cover block and develop an opening.

Catch Tank 241-S-304 & 241-AZ-154

SEC. A-A

$$a = \frac{.44 \times 60}{.85 \times 12 \times 3} = 863'' ; d = 30 - 2.5 = 27.5''$$

$$M_u = 1.0 \times .44 \times 60000(27.5 - .863/2) = 714,608 \text{ in-lb / Ft}$$

$$W_{DL} = \frac{30}{12} \times 150 = 375 \text{ #/ft} ; M_{DL} = \frac{375 \times 10.83^2}{8 \times 12} = 65,975 \text{ in-lb / Ft}$$

$$P = \frac{4(M_u - M_{DL})}{L} = \frac{4(714,608 - 65,975)}{10.83 \times 12} = 19,964 \text{ #}$$

Client WHC

WO/Job No.

Subject Evaluation of Impact Energy

Date 5/13/96 By M.A. ISLAM

Catch Tank

Checked 5-20 By RA Giller

Location ZEO Area

Revised By

$$I = \frac{12 \times 27.5^3}{12} = 20,797 \text{ in}^4$$

$$K = \frac{48 \times 3 \times 10^6 \times 20,797}{(10.83 \times 12)^3} = 1.36 \times 10^6 \text{ #/in}$$

$$P = 19,964 = W + \sqrt{W^2 + 2KW(h)} \quad \text{using } W = 20000 \text{ #}$$

$$20000 + \sqrt{20000^2 + 2 \times 1.36 \times 10^6 \times 20000(h)} = 19,964$$

$h = \text{Negative}$

Insignificant energy is enough to fail the cover block and develop an opening in the cover block.

Catch Tank 241-A-417

Because compression block (a) is usually small, consider 3 layers of re-bar will be available in tension.

Determine d'

$$.44 \times 2.5 = 1.1$$

$$.44 \times 2 \times 15 = 13.2$$

$$\frac{1.1}{1.32} = \frac{14.3}{14.3}$$

$$\bar{y} = \frac{14.3}{1.32} = 10.83$$

$$d = 30 - 10.83 = 19.17$$

Client	NHC	WO/Job No.	
Subject	Evaluation of Impact Energy	Date	5/13/96
	Catch Tank	By	M.A. ISLAM
Location	200 Area	Checked	5-20
		By	RA Giller
		Revised	
		By	

$$\alpha = \frac{(3 \times .44) \times 60}{185 \times 12 \times 3} = 2.59''; \quad WDL = \frac{30}{12} \times 150 = 375 \#/\text{ft}$$
$$M_{DL} = 375 \times \frac{8^2}{8} \times 12 = 36,000 \text{ in-lb}$$

$$M_u = 1.0 \times 3 \times .44 \times 60000 \left(19.17 - 2.59/2\right) = 1,415,700 \text{ in-lb/ft}$$

$$I = \frac{12 \times 30^3}{12} = 27,000 \text{ in}^4$$

$$K = \frac{48 \times 3 \times 10^6 \times 27000}{(8 \times 12)^3} = 4.39 \times 10^6 \#/\text{in}$$

$$P = \frac{4(M_u - M_{DL})}{L} = \frac{4(1,415,700 - 36,000)}{8 \times 12} = 57,488 \#$$

$$\therefore P = 57,488 = W + \sqrt{W^2 + 2KW(h)}, \quad \text{using } W = 20000 \#$$

$$20000 + \sqrt{20000^2 + 2 \times 4.39 \times 10^6 \times 20000(h)} = 57488$$

$$\therefore h = .006 \text{ in Negligible}$$

$\therefore$ Insignificant energy is enough to cause failure in cover block and develop an opening eventually.

Client	WHC	WO/Job No.	
Subject	Evaluation of Impact Energy	Date	5/13/96
	200 Series Tanks	Checked	5-20
Location	200 Area	Revised	
		By	RA Giller

200 Series Tanks (241-B,C,T,U)

* From Ramble Report,
(SD-RE-TI-012)

PLAN

SEC. A-A

SEC. B-B

Client	WHC	WO/Job No.	
Subject	Evaluation of Impact Energy	Date	5/14/96 By M.A. ISLAM
	200 Series Tanks	Checked	5-20 By RA Giller
Location	200 Area	Revised	By

ACI-318-89, Sec. 10.7

89, Sec. 10.7
Ratio of depth to clear span = $\frac{13}{20} = .65 < \frac{4}{5} = .8$ $\therefore$ it is not a deep beam. Consider 'T' beam action

Eff. width of 'T' beam (Sec. 8.10.3)

control $\Rightarrow$ i) $b_w + \frac{1}{12} \text{span} + 19.5 = 12 + \frac{1}{12} \times 20 \times 12 + 19.5 = 51.5$

ii) $6t = 6 \times 12 = 72''$

iii) space of beam, (use half span) = 120"

Total re-bar in width (#6@12")

$$A_3 = \underset{\text{NCS.}}{5 \times .44} \times \underset{\text{TEB}}{2} = 4.4 \text{ in}^2$$

$$d = 13 \times 12 - 6 = 150''; \text{ span, } L = 20'$$

$$a = \frac{A_s f_y}{0.85 b f_c} = \frac{4.4 \times 60}{0.85 \times 12 \times 2.5} = 10.35"$$

$f_c' = 2500 \text{ Psi per Dwg. HW-72417}$ $w_{DL} = \frac{12 \times 120 + 12 \times 13 \times 2}{144} \times 150 = 3450 \text{ #/}$

$$\therefore M_u = \phi A_s f_y (d - a/2) = 1.0 \times 4.4 \times 60000 (150 - \frac{10.35}{2})$$

$$= 38,233,800 \text{ in-lb} \quad \therefore M_{DL} = 3450 \times \frac{20^2}{8} \times 12 = 2,070,000$$

$$\text{Max. Impact load, } P = \frac{4(M_u - M_{\text{bal}})}{L} = \frac{4(387.33800 - 2.67000)}{20 \times 12}$$

$$= 602.730 \text{ \#}$$

C.G. of beam, $\frac{12 \times 120 \times 6}{12 \times 12 \times 12 \times (12 + 72)} = \frac{8640}{145,152}$ $\bar{y} = \frac{153792}{3168} = 48.5''$

$$I = \frac{12 \times 144^3}{12} + 12 \times 144 (12 + 72 - 48.5)^2 + 12 \times 120 (48.5 - 6)^2 = 7.76 \times 10^6 \text{ in}^4$$

$$K = \frac{48 \times 3 \times 10^6 \times 7.76 \times 10^6}{(20 \times 12)^3} = 80.9 \times 10^6 \text{ #/in}$$

$$P = 602,730 = W + \sqrt{W^2 + 2KW(h)} \quad \text{using } W = 20000 \#$$

∴ Energy, $E = 20000 \times 105 = 2096 \text{ mJb} \Rightarrow \text{Negligible}$
Insignificant energy is enough to cause failure in roof covering tank

Client MHC

WO/Job No. _____

Subject Evaluation of Impact Energy
Impacting with Ground

Date 5/16/96

By M.A. ISLAM

Checked 5-20

By RA Giller

Location 200 Area

Revised _____

By _____

Equipment to drop onto soft Ground (Yielding Surface)

DCRTs / catch Tanks

200 Series Tanks

Equipment weight, W , dropping onto the soft ground produces impact load, F_I which will act as surcharge load for the DCRTs and catch tanks. However, F_I in the case of 200 Series Tanks will cause bending in roof of tank.

First, the impact load F_I will be determined and then it will be reviewed if it may cause failure in the wall or roof.

Assumptions:

1. Unit wt. of soil, $\gamma = 125 \text{ }^{\circ}/\text{CF}$
2. Angle of internal friction, $\phi = 30^{\circ}$
3. Foot print of impact, 2-Ft. dia.
4. Equipment Weight, $W = 20000 \text{ }^{\circ}$ and height of fall, $h = 120 \text{ }^{\circ}$

Client	WHC	WO/Job No.	
Subject	Evaluation of Impact Energy	Date	5/14/96
		By	M.A. ISLAM
		Checked	5-20
		By	R.A. Griller
Location	200 Area	Revised	
		By

At the completion of the impact, the equipment penetrates 'Δ' into the ground.

$$\text{Area of impact} = \frac{\pi}{4} (24)^2 = 452 \text{ in}^2$$

$$\text{Impact Force, } F_I = W \left[1 + \sqrt{1 + \frac{2h}{\Delta}} \right] \quad \text{Ref. "Design of Machine Elements" M.F. Spotts}$$

$$F_I = 20000 \left[1 + \sqrt{1 + \frac{2 \times 120}{\Delta}} \right] \text{ ----- Eqn ①}$$

The subgrade reaction force due to the compressive strength of the soil, F_s

$$F_s = q_u A \quad \text{where } q_u = \text{ult. compressive strength of soil}$$

Find q_u

$$q_u = 1.3 c N_c + q N_q + 0.4 \gamma B N_\gamma \quad \text{where } c = \text{cohesion for Sand } c = 0$$

$$\left. \begin{array}{l} N_c = 37 \\ N_q = 22.5 \\ N_\gamma = 20 \end{array} \right\} @ \phi = 30^\circ \quad \text{Ref. Gaylord \& Gaylord, "Struc. Engineering Handbook"}$$

$$\gamma = \text{soil unit wt.} = 125 \text{ #/ft}^3 = 0.0723 \text{ #/in}^3$$

$$B = \text{width of equipment in contact with soil} = 24 \text{ "}$$

$$q = \text{Compressive strength of soil} = \gamma \Delta = 0.0723 (\Delta)$$

$$\therefore q_u = 0 + 0.0723 (\Delta) 22.5 + 0.4 \times 0.0723 \times 24 \times 20$$

$$\text{or } q_u = 1.627 (\Delta) + 13.88$$

$$\therefore F_s = q_u A = [1.627 (\Delta) + 13.88] 452 = 6274 + 735.4 (\Delta) \text{ ----- Eqn ②}$$

Client NHC

WO/Job No.

Subject Evaluation of Impact Energy

Date 5/14/96

By M.A. ISLAM

Checked 5-20

By RA Giller

Location 200 Area

Revised

By

In order to maintain equilibrium, F_I must equal F_s

$$\therefore 20000 \left[1 + \sqrt{1 + \frac{240}{\Delta}} \right] = 6274 + 735.4 (\Delta)$$

$$1 + \sqrt{1 + \frac{240}{\Delta}} = .3137 + .03677 (\Delta)$$

$$.03677 (\Delta) - \sqrt{1 + \frac{240}{\Delta}} = .6863$$

$$\text{Try } \Delta = 70'' \quad \text{L.H.S.} = .469$$

$$75'' \quad \quad \quad = .708$$

$$74.5'' \quad \quad \quad = .685 \Rightarrow \therefore \Delta = 74.5''$$

$$\text{Impact Load, } F_I = 20000 \left[1 + \sqrt{1 + \frac{2 \times 120}{\Delta}} \right] = 20000 \left[1 + \sqrt{\frac{240}{74.5} + 1} \right]$$

$$= 61,092 \#$$

Determine Impact load @ roof top, $F_I(R)$

Refer Fig. 9.12, 'Introductory Soil Mechanics and Foundations, 3rd Ed.
- Sowers & Sowers

Load coefficient @ 5.792' = .175

$$\therefore \text{Net } F_I = .175 \times 61,092 = 10,691 \#$$

By comparison to previous analysis, we can conclude that 10,691# load will not cause failure in the integrity of the roof.

Similarly for DCRTs and Catch Tanks, F_I will develop a surcharge load (radial to tank vault wall), p

$$p = \frac{F_I (K_0)}{A} \quad \text{where } K_0 = \text{rest pressure coeff.} = .5$$

$$= \frac{61092 \times .5}{\pi/4 \times 24^2} = 67.5 \#/\text{in}^2 \quad \times \text{ per GG-SAFED}$$

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Client WHC

WO/Job No.

Subject Evaluation of Impact Energy

Date 5/23/96

By M.A. ISLAM

Checked 5-28-96

By R.A. Giller

Location 200 Area

Revised

By

From Roark's (4th Ed.), Table XIII, Case 12

$$\text{Max } M (\text{due to } p) = \frac{p}{2\gamma^2} e^{-b\gamma} \sin b\gamma$$

use Vault $R=54''$
" $t=18''$ where $b = 12 \text{ in}$ (half of $2\frac{1}{4}$ base)

$$\gamma = \left[\frac{3(1-\nu^2)}{R^2 t^2} \right]^{\frac{1}{4}} = \left[\frac{3(1-.15^2)}{54^2 \times 18^2} \right]^{\frac{1}{4}} = .042$$

$$\therefore M = \frac{67.5}{2 \times .042^2} \times e^{-12 \times .042} \times \sin(12 \times .042) = 101.7 \text{ \#-in} \Rightarrow \text{Small}$$

Vault walls are OK.

Consider other drop heights

$$\text{Try } h = 15', \text{ from page 20, } .03677(\Delta) - \sqrt{1 + \frac{2h}{\Delta}} = .6863$$

$$h = 120''$$

$$\therefore .03677(\Delta) - \sqrt{1 + \frac{360}{\Delta}} = .6863$$

$$\Delta = 81.8'' \Rightarrow F_I = 20000 \left[1 + \sqrt{1 + \frac{2 \times 180}{81.8}} \right] = 66,480 \text{ \#}$$

$$\text{Try } h = 20', \text{ Similarly, } .03677(\Delta) - \sqrt{1 + \frac{2 \times 240}{\Delta}} = .6863$$

$$\Delta = 87.8'' \Rightarrow F_I = 70,860 \text{ \#}$$

Change in impact load for increased drop heights will not cause failure either to vault wall or roof.

Appendix 3

Internal Transient Pressure Caused by Hydrogen Deflagration

Appendix 3

Internal Transient Pressure Caused by Hydrogen Deflagration.

The following catch tanks, Double-Contained Receiver Tanks (DCRT), and 200-Series Single Shell Tanks (SST) were evaluated for a peak pressure of 120 psig for a Hydrogen deflagration inside the primary waste tank. For catch tanks and DCRT's having a primary steel tank within a reinforced concrete vault, the peak pressure was applied inside the primary steel tank. The peak pressure was assumed to be applied instantaneous. A static pressure evaluation was performed using a dynamic load factor of two in order to account for the instantaneous application of pressure.

Catch Tanks (CT)

241-A-302-A
241-A-417
241-AX-152
241-AZ-151
241-AZ-154
241-ER-311
241-EW-151
241-S-304
241-TX-302-C
241-U-301-B
241-UX-302-A

Double-Contained Receiver Tanks (DCRT)

244-A Lift Station
241-A-350 Lift Station
244-BX Catch Station
244-CR-003 Vault Tank
244-S Lift Station
244-TX Catch Station
244-U Catch Station

200-Series Single-Shell Tanks (SST)

241-B 200-Series SST
241-C 200-Series SST
241-T 200-Series SST
241-U 200-Series SST

The subject catch tanks, DCRT's, and 200-Series SST's were evaluated for loss of confinement for the waste. Loss of confinement occurs when an unobstructed opening or path is created between the waste and atmosphere. Therefore, the structural failures that were evaluated in this assessment were those that would cause the rupture of the steel tank structure and subsequent failure of the reinforced concrete vault. The concrete vaults of the DCRT's were evaluated in Appendix 5 for a gasoline fire and the resulting pressure of 140 psi and were concluded to fail and result in an unobstructed opening to the atmosphere. As such, the concrete vaults of catch tanks, DCRT's, and 200-Series SST's not having primary steel tanks are concluded to fail if exposed to a hydrogen deflagration pressure of 120 psi. The catch tanks and DCRT's having a primary steel tank within a reinforced concrete vault whose primary steel tank fails due to the 120 psig internal pressure are assumed to have loss of confinement and unobstructed opening or path between the waste and atmosphere due to subsequent failure of the concrete vault structure surrounding the tanks. The catch tanks that are buried directly in the soil will not have unobstructed openings between

the waste and atmosphere even if the tank were to rupture.

Following is a table summarizing the evaluation of the steel tanks for 120 psig internal pressure.

Table 3.1 Internal Transient Pressure Summary

	VESSEL MATERIAL	VESSEL DIA	WALL THICKNESS	WALL STRESS (1)	YIELD STRESS	STATUS
Catch Tanks						
241-A-302-A	carbon steel direct buried	9'-0"	9/16"	23,040 psi	30,000 psi	OK
241-A-417	lined concrete vault, no vessel					Vault Fails
241-AX-152	lined concrete vault, no vessel					Vault Fails
241-AZ-151	lined concrete vault, no vessel					Vault Fails
241-AZ-154	lined concrete vault, no vessel					Vault Fails
241-ER-311	carbon steel direct buried	9'-0"	9/16"	23,040 psi	30,000 psi	OK
241-EW-151	stainless steel	4'-5"	3/8"	17,280 psi	25,000 psi	OK
241-S-304	carbon steel	9'-0"	5/8"	20,736 psi	30,000 psi	OK
241-TX-302C	carbon steel direct buried	9'-0"	1/2"	25,920 psi	30,000 psi	OK
241-U-301B	precast concrete vault, no vessel					Vault Fails
241-UX-302A	carbon steel direct buried	9'-0"	1/2"	25,920 psi	30,000 psi	OK
Double-Contained Receiver Tanks						
244-A				not evaluated		Vault Fails See Appendix 5
241-A-350	stainless steel	4'-5"	3/8"	17,280 psi	25,000 psi	OK

244-BX/TX/U	carbon steel	12'-0"	1/4"	69,120 psi	30,000 psi	Tank Ruptures Vault Fails See Appendix 5
244-CR-003	stainless steel		3/8"	not evaluated		Vault Fails See Appendix 5
244-S	carbon steel	15'-0"	1/4"	86,000 psi	30,000 psi	Tank Ruptures Vault Fails See Appendix 5
200-Series Single Shell Tanks						
241-B/C/T/U	lined concrete vault, no vessel					Vault Fails

1. at 120 PSIG using dynamic factor of 2: $STRESS = 2 \times (120 \text{ psi} \times DIA/2) / (Thickness)$

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Appendix 4
Seismic Event

Appendix 4

Seismic Event

The task is to determine the maximum free field ZPA that would not result in a loss of confinement for each of the catch tanks, DCRTs and the 200-series tanks. These are buried structures. Simple methods based on code practice are coupled with engineering judgement to make the required determinations. Considerable uncertainty and most likely conservatism are expected with the soil effects. Further refinement requires soil-structure interaction analysis with finite element methods.

In general, code capacities are conservative when compared with expected loads in seismic events, especially with ductile structures. Given a design basis earthquake, DOE practice typically reduces the seismic demand before comparison with code capacity. Determining the ZPA herein that would not result in a loss of confinement is guided by DOE Standard 1020-94. Consider PC 3 structures. Appendix C of 1020, Section C.2.1, has a Scaled Design/Evaluation Basis Earthquake SDBE defined by

$$\text{SDBE} = 1.5 \times \text{SF} \times \text{DBE}$$

where

SF = scale factor, = 1 for PC 3

DBE = design basis earthquake, g.

The code evaluation procedure to the DBE load level is estimated to correspond to a 10% probability of unacceptable performance for the SDBE. Consequently, ZPAs estimated by code criteria should be scaled up by a factor $S_s = 1.5$ to reflect the inherent conservatism on code practice with seismic loading. Since the simple procedure herein omits strength reduction factors, S_s is modified to 1.35. Any use of the resulting maximum acceptable accelerations for comparison with an acceptable DBE should be reduced by a factor of 1.5 before the comparison.

The vertical acceleration is taken equal to the horizontal acceleration. Modal and directional combinations are considered only if necessary in the analyst's judgement. Typically, a fundamental mode excited by only one direction of loading will dominate the structural response.

Part of the 1020-94 procedure includes inelastic capacity factors which reduce the demand for the more ductile failure modes. In the case of reinforced concrete structures, they range from 1.0 for out-of-plane shear to 1.75 for flexure. The use of these factors is predicated on adequate detailing, i.e., proper design of the reinforcing steel components per current code requirements.

Detailing checks and evaluations of shortfalls are out of the work scope, so the capacity factors are taken as unity in accordance with 1020-94. This indicates, however, that substantial gains could be obtained with additional work.

Seismic pressures on plane walls are estimated with the elastic method characterized by Figure 3500-1 of ASCE 4-86, attached. This approach is considered conservative, and refinements would require soil-structure interaction (SSI) analyses which can be involved. Most calculations herein simplify the figure to $\sigma_2^o = 1.15\gamma H$. The depth H is limited to the pit floor in situations where the pit is situated over a caisson. The depth H extends down to the foundation in cases where the plane wall extends down to the foundation as well, e.g., 244-CR. Treatment of a caisson under a pit or a cylindrical underground structure is based on experience with the MWTf analysis which was conducted with a ZPA of 0.38g. A check of the MWTf report shows that the wall's shear stress from SSI analysis is half the stress obtained by multiplying the best estimate free field soil strain at maximum depth by the concrete shear modulus. The curve of free field shear strain vs depth is attached. In-plane shear stresses obtained in this manner are compared with normal code allowables without amplification to find the acceptable ZPA.

Cover blocks and similar structures not enclosed in soil are amplified for dynamic effects by a factor of two. This includes domed structures with a soil depth that is small compared to the tank diameter. Cover blocks pressing against a wall against soil with horizontal loading are not amplified.

Coincident static soil pressures are estimated with a coefficient $K_0 = 0.4$.

Results are summarized in Table 4.1 along with the maximum ZPA and failure mode.

Some directly buried tanks such as 241-ER-311 are shown to collapse without seismic loading, i.e., they were not shown good for static loading alone. As such, the simple analysis procedure could well overestimate the static loading, and refined analysis would be required. In the analyst's judgement, however, the tank volume is small compared to a characteristic soil volume above it, and a significant opening is unlikely.

Pit wall and cover block collapse is assumed to lead to an opening. No further failure sequence evaluated. In the case of the lower wall of 244-CR, a refined analysis might show integrity with static soil loads by considering plate edge restraints and distribution of moments in the wall.

WHC-SD-WM-TI-775, Rev. 0

Table 4.1 Seismic Summary

Item	ZPA, g	Failure Mode	Opening
241-A-201A	> 1.0		No
241-A-417	0.60	Roof/soil collapse Pit wall collapse	Yes
241-AX-152	0.35	Pit wall collapse	Yes
241-AZ-151	0.38	Pit wall collapse	Yes
241-ER-311	0.00	Collapse	No - buried deep
241-EW-151	> 1.0		No
241-TX-302C	0.32	Collapse	No - buried deep
241-UX-302A	> 1.0		No
241-U-301-B	> 1.0		No
241-A-350	> 1.0		No
244-A	0.82	Pit wall collapse	Yes
244-BX/TX/U	0.39	Pit wall collapse	Yes
244-CR	0.00	Lower wall collapse, refined static analysis required	Yes
244-S	0.63	Cover blocks collapse	Yes
241-S-304	0.60	Caisson roof collapse	Yes
200-S	0.48	Roof collapse	Yes

imum relative dynamic movement between anchor points (such as a building attachment point) and the adjacent soil, which occurs as a result of the dynamic response of the anchor point, shall be calculated. In calculating maximum forces and strains in the buried structure, the motion of adjacent anchor points shall be considered to be out-of-phase, so as to result in an absolute sum determination of the relative displacements.

(b) Forces and strains associated with the relative movement of anchor points shall be combined with the corresponding forces and strains from wave propagation effects using the SRSS method.

3.5.3 Earth-Retaining Walls

3.5.3.1 General Requirements--

(a) Earth-retaining walls shall be analyzed for seismic-induced soil pressures using an elastic solution, unless wall displacements can be tolerated without loss of wall function in order to justify the use of an active earth pressure solution.

(b) For cantilever retaining walls, the active solution may be used.

(c) The summation of the calculated dynamic seismic soil pressures and the static earth pressure shall not exceed the soil static passive earth pressure.

3.5.3.2 Elastic Solution-- Acceptable procedures to determine seismic loads on earth-retaining walls are as follows:

1. Dynamic finite-element analyses that conform to the requirements in 3.3.3 may be used. However, soil-structure connectivity shall be assumed over the entire embedment height, and the effect of structure-structure interaction on dynamic soil pressures shall also be considered.
2. When there is no significant structure-structure interaction, a conservative estimate of dynamic soil pressures may be obtained from Fig. 3500-1, and the corresponding resultant force and overturning moment may be obtained from Fig. 3500-2.

3.5.3.3 Active Solution-- The Mononobe-Okabe approach may be used to establish dynamic soil pressures, provided that wall displacements required to develop the active earth pressure can be

The calculated resultant dynamic force shall be applied at two-thirds of the soil embedment height as measured from the bottom.

3.5.4 Aboveground Vertical Tanks

3.5.4.1 General Requirements--

(a) Except as otherwise noted, the requirements set forth in this section are applicable to aboveground vertical, fluid-containing tanks in which the upper fluid surface is essentially unconstrained (free).

(b) At least two horizontal modes of combined fluid-tank vibration and at least one vertical mode of fluid vibration shall be included in the analysis.

(c) The horizontal response analysis shall include at least one impulsive mode, in which the response of the tank shell and roof are coupled together with the portion of the fluid contents that moves in unison with the shell, and the fundamental sloshing (convective) mode.

3.5.4.2 Horizontal Impulsive Mode

3.5.4.2.1 Effective Weight of Fluid - Impulsive Mode-- In the fundamental horizontal impulsive mode, the effective fluid weight (only a portion of the total weight) shall be determined and used for analyses in lieu of the total fluid weight. When determining the effective fluid weight, the tank may be assumed to be rigid.

3.5.4.2.2 Spectral Acceleration - Impulsive Mode-- Damping values to be used to determine the spectral acceleration in the impulsive mode shall be equal to the values for the tank shell material specified in 3.1.2.2.

It is necessary to estimate the fundamental frequency of vibration of the tank including the impulsive fluid weight. Flexibility of the tank shall be included unless the assumption of a rigid tank can be justified. The horizontal impulsive mode spectral acceleration shall be determined using this impulsive mode frequency and tank shell damping. In lieu of determining the impulsive mode fundamental frequency, it is permissible to use the peak horizontal spectral acceleration for the tank shell damping value.

3.5.4.2.3 Overturning Moment at Base of Tank - Impulsive Mode-- The overturning movement at the base of the tank due to the fundamental impulsive mode shall

H =	embedment height
Y =	distance from base of retaining structure
γ =	soil unit weight
ν =	Poisson's ratio
σ_0^2 =	lateral dynamic soil pressure against the retaining structure for 1.0g horizontal earthquake acceleration

FIGURE 3500-1 VARIATION OF NORMAL DYNAMIC SOIL PRESSURES FOR THE ELASTIC SOLUTION

effective fluid weight and the effects of the weight of the tank shell acting in phase.

3.5.4.2.4 Hydrodynamic Pressure on Tank Shell - Impulsive Mode-- The effect of tank shell flexibility shall be included when determining the hydrodynamic pressure on the tank shell for the impulsive mode.

3.5.4.3 Horizontal Sloshing (Convective Mode)

3.5.4.3.1 Effective Weight of Fluid - Sloshing Mode-- The effective fluid weight acting in the horizontal sloshing mode may be determined on the basis of an assumed rigid tank.

3.5.4.3.2 Spectral Acceleration - Sloshing Mode-- In determining the spectral acceleration in the horizontal sloshing mode, the fluid damping shall be taken as 0.5% of critical damping unless a higher value can be substantiated. The fundamental frequency of the sloshing

mode may be determined on the basis of an assumed rigid tank shell. The horizontal sloshing mode spectral acceleration shall be determined using the sloshing mode fundamental frequency and damping ratio.

3.5.4.3.3 Overturning Moment at Base of Tank - Sloshing Mode-- The overturning moment at the base of the tank due to fundamental sloshing mode shall be determined.

3.5.4.3.4 Hydrodynamic Pressure on Tank Shell - Sloshing Mode-- The hydrodynamic pressure on the tank shell resulting from the horizontal sloshing fluid mode may be determined on the basis of an assumed rigid tank shell.

3.5.4.3.5 Fluid Slosh Height - Fundamental Sloshing Mode-- The fluid slosh height may be determined based upon an assumption of a rigid tank shell.

DESIGN ANALYSIS

Revision

0

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Client Westinghouse Hanford Co.
 Subject SHAKE Analysis 200E and 200W Areas
 Location 200E/200W

WO/Job No. W236A
 Date 1-11-95 By GR Wagenblast
 Checked 2-27-95 By C.K. Wong
 Revised By

241-A-417

MWTF SEISMIC ANALYSES
 SEISMIC STRAIN IN SOIL LAYERS

(0.38g 2PA)

Figure 7.2

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DESIGN ANALYSIS

Client	WO/Job No.
Subject 241-A-302A Seismic	Date 5/96 By EDWerner
	Checked 5/28/96 By ARW
Location	Revised By

241-A-302A Seismic

Vertical: See internal vacuum calcs

$$P_{soil} = 19 \text{ psi (static, 1g)}$$

$$P_u = 38 \text{ psi}$$

$$a_{max} = \frac{1.35}{1} \left(\frac{P_u - P_{soil}}{P_{soil}} \right) = 1.35 \times \frac{38 - 19}{19} = \underline{\underline{1.35}}$$

Hr 21: $a_{max} > 1g$ by judgement

Client	WO/Job No.		
Subject	Seismic	Date	5/96 By Cow
		Checked	5/28/96 By ARW
Location		Revised	By

Seismic
241-A-417

Vert: Cover blks. See uniform load calcs (App 1)

$$P_{live} = 1.0 \text{ ksf} \quad (\text{acceptable live load})$$

$$h = 2.5'$$

$$I_g: P_{pres} = \gamma_{con} h = .15 \text{ kcf} \times 2.5' = .375 \text{ ksf}$$

$$a_{max} = \frac{S_s}{A} \frac{P_{live}}{P_{pres}} = \frac{1.35}{2} \cdot \frac{1.0}{.375} = \underline{\underline{1.8g}}$$

Carsson roof under soil:

App 5

$$\text{See int. vac. calc: } P_{soil} = 12 \text{ psi} \quad M_n = 23 \text{ K} \quad L = 7.5'$$

$$M = \frac{PL^2}{8} = \frac{12 \text{ psi} \times (7.5 \times 12)^2}{8} = 12.2 \text{ K}$$

$$a_{max} = \frac{S_s}{A} \frac{M_n - M}{M} = \frac{1.35}{2} \frac{23 - 12.2}{12.2} = \underline{\underline{0.60g}}$$

Huzl:

Cover blks & wall, inertial load of blocks puts wall in bending & shear.

$$b = \text{blk depth} + \frac{1}{3} (\text{unblocked wall})$$

$$= \frac{2 \times 15''}{12} + \frac{1}{3} (14.5' - 2.5') = 6.5'$$

$$I_{ave} = \left(10.5'' \times 1.25 + 14.5'' \times 1.25 - 12'' \times 1.25 + 18'' \times 4.0 \right) / 6.5 = 16''$$

$$c_{lv} = 7.5' \quad d = 16 - 1.5 = 14.5''$$

$$\# 5 @ 14' \text{ EFW} \quad j = .95$$

$$M_n = j d \frac{A_b}{s} f_y = .95 \times 14.5'' \times \frac{.31''}{14} \times 40 \text{ ksi} = 12.2 \text{ K}$$

96

Client

WO/Job No.

Subject

Seismic

Date

5/96

By

EOW

Checked

5/20/96

By

NRW

Location

Revised

By

241-A-417 (cont'd)

$$I_g: W = \gamma h_s = .15 \frac{\text{kcf}}{\text{ft}} \times 2.5' \times 2.7' = 3.3 \frac{\text{K}}{\text{ft}}$$

$$M = \frac{W L^2}{8 h} = \frac{3.3 \times 16^2}{8 \times 6.5} = 16.2 \text{ K}$$

Soil resistance per UBC Table 29-B

$$P_{soil} = 150 \text{ psf} \left(1 + \frac{1}{2}\right) = 230 \text{ psf (ave at blks)}$$

$$W_{soil} = P_{soil} h = .23 \frac{\text{kcf}}{\text{ft}} \times 2.5' = 0.6 \frac{\text{K}}{\text{ft}}$$

$$M_{soil} = \frac{W_{soil}}{W} M = \frac{0.6}{3.3} 16.2 \text{ K} = 3.0 \text{ K}$$

$$a_{max} = \frac{S_s}{A} \frac{M_n + M_{soil}}{M} = \frac{1.35}{1} \frac{12.2 + 3.0}{16.2} = 1.3 g$$

$$V_n = 2 \sqrt{f_c'} d b = 110 \text{ psi} \times 14.5'' \times 6.5 \times 12'' = 124 \text{ K}$$

$$V \approx W L / 2 = 3.3 \frac{\text{K}}{\text{ft}} \times 16' / 2 = 26 \text{ K (does not control)}$$

Pit wall:

$$H = 14.5' \quad P_{seis} = 1.158 H = 1.158 \times 14' = 16.2 \text{ kcf} \times 14' = 2.0 \text{ ksf}$$

$$P_{soil} = K_o \gamma H_{ave} = 0.4 \times 1.25 \times 7 = .35 \text{ ksf}$$

Wall is simply supported all around

$$\text{Roarke 4th Table X, 36} \quad a = 16' \quad b = 13.3'$$

$$a/b = 16/13.3 = 1.2 \quad \beta = .376$$

$$M = \frac{t^2}{6} s = \frac{\beta}{6} p b^2 = \frac{.376}{6} \times 2.0 \text{ ksf} \times 13.3^2 = 22.4 \text{ K}$$

$$d = 16.5'' \quad M_n = \frac{16.5}{14.5} 12.2 \text{ K} = 13.9 \text{ K}$$

See Mn above

$$M_{soil} = \frac{P_{soil}}{P_{seis}} M = \frac{.35}{2.0} \times 22.4 = 3.9 \text{ K}$$

Client		WO/Job No.	
Subject	<u>SEISMIC</u>	Date	<u>5/16</u> By <u>EDW</u>
		Checked	<u>5/28/96</u> By <u>ARW</u>
Location		Revised	By _____

241-A-417 (con'd)

$$a_{max} = \frac{S_s}{A} \frac{M_n - M_o}{M} = \frac{1.35}{1} \frac{13.9 - 3.9}{22} = \underline{\underline{.61g}}$$

CAISSON wall.

Depth $H_{max} = 30'$ MWTF $\gamma = 0.021g$

$$U = G\gamma/2 = 1.526 \times \frac{220}{2} \text{ psi} = 140 \text{ psi}$$

$$U_c = 2\sqrt{f_c} = 110 \text{ psi}$$

HR21 steel = #5 @ 11" EF $d = 0.8h = 0.8 \times 10' = 8'$

$$U_s = \frac{A_s f_s}{s_2 d} = \frac{0.31' \times 40 \text{ ksi}}{11 \times 8} = 140 \text{ psi}$$

$$U_n = U_c + U_s = 110 + 140 = 250 \text{ psi}$$

$$a_{max} = \frac{S_s}{A} q \frac{U_n}{U} = \frac{1.35}{1} \frac{0.38}{0.38} \frac{250}{140} = \underline{\underline{0.92g}}$$

Client		WO/Job No.	
Subject	<u>Seismic</u>	Date	<u>5/96</u> By <u>EDW</u>
		Checked	<u>5/28/96</u> By <u>MHW</u>
Location		Revised	By _____

241-AX-152 Seismic

Vertical:

 Cover blocks: $P_{live} = 11 \text{ ksf}$ (App I) $h = 5'8''$ stout, OK
 Hnz1:

Cover blocks (H-2-44SP1 Sect E)

$$\text{Wall ht} = 10'6'' + 1'5'' + 4'1'' = 16'$$

 Use $1/3$ wall below blocks

$$b = 5.7 + \frac{1}{3}(16 - 5.7) = 9.1'$$

$$h_{ave} = \frac{14'' \times 5.7 + 18'' \times (9.1 - 5.7)}{7.1} = 15.5''$$

$$cbr = 2'' \quad d = 13.5''$$

$$\#6 @ 12'' \quad j = .95$$

$$M_n = .95 \times 13.5 \times \frac{144}{12} \times 40 = 18.8 \text{ K}$$

$$I_g: w = 8hs = .15^{144} \times 5.7' \times 6.8' = 5.8 \text{ K/1}$$

$$M = \frac{wL^2}{8b} = \frac{5.8 \times 22.2^2}{8 \times 9.1} = 39 \text{ K}$$

$$\text{Soil resistance UBC Table 29-B} \quad w_{soil} = 150 \text{ psf} \left(1 + \frac{4.7}{2}\right)$$

$$w_{soil} = 560 \text{ psf}$$

$$M_{soil} = \frac{.15}{5.6} 39 \text{ K} = 3.4 \text{ K}$$

$$a_{max} = \frac{S_s}{A} \frac{M_n + M_{soil}}{M} = \frac{1.35}{1} \frac{18.8 + 3.4}{39} = 0.77 g$$

Lower wall:

$$H = 681 - 652 = 29' \quad \text{Deep wall}$$

$$P = 0.75 \times .125 \times 29 = 2.7 \text{ ksf}$$

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Client		WO/Job No.	
Subject	Seismic	Date	5/96 By EOW
		Checked	5/25/96 By MRW
Location		Revised	By

241-AX-152 (cont)

$$P_{soil} = K_o \gamma H_{ave} = 0.4 \times .125 \times (29 - 1.5 - 11.5/2) = 1.1 \text{ ksf}$$

Roarks 4th Table X.3C: $b = 11.5'$ $a = 22.2'$ $a/b = 2$

$$\beta = .610$$

$$M = \frac{\beta}{6} P b^2 = \frac{.61}{6} \times 2.7 \times 11.5^2 = 36 \text{ K}$$

$$M_{soil} = \frac{1.1}{2.7} 36 = 14.7 \text{ K}$$

$$\#6 @ 10" \quad d = 18 - 2 = 16"$$

$$M_n = .95 \times 16 \times \frac{.44}{10} \times 40 = 26.7 \text{ K}$$

$$a_{max} = \frac{1.35}{1} \frac{26.7 - 14.7}{36} = \underline{\underline{0.45 g}}$$

Upper wall

$$P_{seis} = 1.15 \times H_{max} = 1.15 \times .125 \times 29 = 4.2 \text{ ksf}$$

$$P_{soil} = 0.4 \times .125 \times 11' = .55 \text{ ksf}$$

$$a = 14.7' \quad b = 10.5 + 1.5 + 4.1 - 5.7 = 10.4'$$

$$a/b = 14.7/10.4 = 1.4 \quad \beta = 0.45$$

$$M = \frac{\beta}{6} P_{seis} b^2 = \frac{.45}{6} \times 4.2 \times 10.4^2 = 34 \text{ K}$$

$$M_{soil} = \frac{.55}{4.2} 34 = 4.5 \text{ K}$$

$$\#6 @ 20" \quad d = 18 - 2 = 16" \quad j = .95$$

$$M_n = .95 \times 16 \times \frac{.44}{20} \times 40 = 13.4 \text{ K}$$

$$a_{max} = \frac{1.35}{1} \frac{13.4 - 4.5}{34} = \underline{\underline{0.35 g}}$$

-100-

Client	WO/Job No.		
Subject	S.E.S.M.I.R.	Date 5/96	By EOW
Location		Checked 5/29/96	By ARW
		Revised	By

241-A2-151

Wall:	Lower	Upper
A2-151	#6 @ 12"	#6 @ 12"
Ax-152	#6 @ 10"	#6 @ 14"

Scale Ax-152 results by rebar spacing

Lower Wall: $a_{max} = 0.45 \times \frac{10}{12} = \underline{\underline{0.38 g}}$

Upper Wall: $a_{max} = 0.35 \times \frac{20}{12} = \underline{\underline{0.58 g}}$

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Client	WO/Job No.		
Subject	<u>Seismic</u>	Date	<u>5/96</u> By <u>EDW</u>
		Checked	<u>5/20/96</u> By <u>ARW</u>
Location		Revised	By

241-ER-311 Seismic

N.G. w/o seismic. See App 5

Client	WO/Job No.
Subject <u>Sersm.c</u>	Date <u>5/16</u> By <u>EDW</u>
	Checked <u>5/20/16</u> By <u>BRW</u>
Location	Revised _____ By _____

2t1-GW-151 Sersm.c

 Vert: Cover bts $p_{live} = 2.1 \text{ ksf (App 1)} \quad h = 2'$

 Ig: $P_{seis} = h \gamma = 2 \times .15 = .3 \text{ ksf} < 2.1 \text{ ksf} = 7 \text{ stout}$

Horiz:

Cover bts

$$b = 2' + \frac{1}{3}(5.7 - 2') = 3.2'$$

$$h_{ave} = (2 \times 6.5" + 1.2 \times 12") / 3.2 = 8.6"$$

$$clv = 2' \quad d = 6.6" \quad \#6 @ 12 \quad j = .95$$

$$M_n = .95 \times 6.6 \times \frac{.44}{12} \times 40 = 9.2 \text{ K}$$

$$Ig: w_{seis} = \gamma h_s = .15 \times 2.0 \times 7 = 2.1 \text{ K/1}$$

$$M_{seis} = \frac{wL^2}{8b} = \frac{2.1 \times 8^2}{8 \times 3.2} = 5.3 \text{ K}$$

$$p_{soil} = 150(1 + 1/2) = 230 \text{ psf} = .23 \text{ ksf}$$

$$w_{soil} = p b = .23 \times 3.2 = .74 \text{ K/1} \quad M_{soil} = \frac{.74}{2.1} 5.3 = 1.9 \text{ K}$$

$$a_{max} = \frac{S_s}{A} \frac{M_n + M_{soil}}{M_{seis}} = \frac{1.35}{1} \frac{9.2 + 1.9}{5.3} = 2.8 g$$

$$V_n = 2\sqrt{f'_c} d b = 110 \text{ psi} \times 6.6" \times 3.2 \times 12" = 28 \text{ K}$$

$$V = (w_{seis} - w_{soil}) L / 2 = (2.1 - .7) 8 / 2 = 5.6 \text{ K} \quad \underline{\underline{OK}}$$

 Lower wall: $a = b = 8.3' \quad \beta = .287$

$$h_{ave} = 16.5 - 1.2 - 4.2 = 11.1'$$

$$p_{soil} = K_o \gamma h = .4 \times .125 \times 11.1 = .56 \text{ ksf}$$

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DESIGN ANALYSIS

Client

WO/Job No.

Subject

Seismic

Date 5/96

By EOW

Checked 5/28/96

By JAW

Location

Revised

By

241-EW-151 (con'd)

$$H_{max} = 16.5'$$

$$p_{seis} = 0.8 \text{ sft} = 0.8 \times 12.5 \times 16.5 = 1.65 \text{ ksf}$$

$$\#6 @ 12 \quad h = 12" \quad d = 10" \quad j = .95$$

$$M_n = .95 \times 10 \times \frac{1.44}{12} \times 40 = 13.9 \text{ K}$$

$$M_{seis} = \frac{e}{8} p b^2 = \frac{.287}{8} \times 1.65 \times 8.32 = 5.4 \text{ K}$$

$$M_{soil} = \frac{.56}{1.65} \times 5.4 = 1.8 \text{ K}$$

$$a_{max} = \frac{S_s}{A} \cdot \frac{M_n - M_{soil}}{M_{seis}} = \frac{1.35}{1} \cdot \frac{13.9 - 1.8}{5.4} = \underline{\underline{3.0 g}}$$

DESIGN ANALYSIS

Client _____

WO/Job No. _____

Subject _____

Seismic

Date

5/96

By

EOW

Checked

5/20/96

By

ARW

Location _____

Revised _____

By _____

241-TX-302C SeismicSee App 5: $P_{soil} = 33 - 8 = 25 \text{ psi}$ $P_u = 31 \text{ psi}$

$$\text{Vert: } a_{max} = \frac{S_s}{A} \cdot \frac{P_u - P_{soil}}{P_{soil}} = \frac{1.35}{1} \cdot \frac{31 - 25}{25} = \underline{\underline{0.32g}}$$

Client	WO/Job No.		
Subject	<u>Seismic</u>	Date	<u>5/96</u> By <u>EDW</u>
		Checked	<u>5/20/96</u> By <u>EDW</u>
Location		Revised	By

241-UX-302A Seismic

See App 5: $P_{soil} = 25 - 8 = 17 \text{ psi}$ $p_u = 31 \text{ psi}$

$$a_{max} = \frac{S_s}{f} \cdot \frac{p_u - P_{soil}}{P_{soil}} = \frac{1.35}{1} \cdot \frac{31 - 17}{17} = \underline{\underline{1.1 g}}$$

DESIGN ANALYSIS

Client _____

WO/Job No. _____

Subject _____

Sersmuc

Date 5/96

By EOW

Checked 5/96

By MRS

Location _____

Revised _____

By _____

241-U-301-B Sersmuc

Vert: See App 5 Internal vacuum for dome

$$P_{cap} = 27 \text{ psi}$$

$$P_{soil} = 6.3 \text{ psi}$$

$$a_{max} = \frac{S_s}{A} \frac{P_{cap} - P_{soil}}{P_{soil}} = \frac{1.35}{2} \frac{27 - 6.3}{6.3} = \underline{\underline{2.29}}$$

Hrztl:

Wall: steel: Vert #4 @ 12"
Hrztl #4 @ 9" (~ 1/2 from bot)

$$v_s = \frac{f_y A_b}{S_x d} \quad h = 6" \quad d \approx 0.8h = 4.8"$$

$$\text{With Hrztl steel: } v_s = \frac{40 \text{ ksi} \times 0.2 \text{ in}^2}{9" \times 4.8"} = 185 \text{ psi}$$

$$v_c = 2\sqrt{f'_c} = 110 \text{ psi}$$

$$v_n = v_c + v_s = 295 \text{ psi}$$

$$l_{tmax} = 18 + 6 + 2.5 = 26.5'$$

$$M/WTF \quad s = 0.01790$$

$$v = G^{3/2} = 1.386 \text{ psi} \times \frac{0.017}{100 \times 2} = 110 \text{ psi}$$

$$a_{max} = 1.35 \times 0.38 \times \frac{295}{110} = \underline{\underline{1.49}}$$

Client	WO/Job No.
Subject	Date <u>5/96</u> By <u>LOW</u>
Location	Checked <u>5/20/96</u> By <u>RAW</u>
	Revised _____ By _____

241-A-350 Seismic

Vert: See App 1: $p_{live} = 2.1 \text{ ksf}$ $h = 1.7'$

$$1g: p_{seis} = 8h = .15 \times 1.7 = .26 \text{ ksf}$$

$$a_{max} = \frac{1.25}{2} \frac{2.1}{.26} = \underline{\underline{5.5 g}} \quad (\text{cover blocks})$$

Hr=1:

Cover blocks & wall

$$b = 1.7' + \frac{1}{3}(11 - 1.7') = 4.8'$$

$$h_{ave} = (1.7 \times 7.5'' + 3.1 \times 12'') / 4.8$$

$$= 10.4''$$

$$\#5 @ 12'' \quad j = .95 \quad 1.5' \text{ ckr } d = 8.9''$$

$$1g: W_{seis} = .15 \times 1.7 \times 10 = 2.6 \text{ K/l}$$

$$M_{seis} = \frac{WL^2}{8b} = \frac{2.6 \times 10^2}{8 \times 4.8} = 6.8 \text{ K}$$

$$P_{soil} = 150 \text{ pcf} \quad W_{soil} = .15 \times 1.7 = .26 \text{ K/l}$$

$$M_{soil} = \frac{.26}{2.6} \times 6.8 = 0.7 \text{ K}$$

$$M_u = jd \frac{A_b}{s} f_y = .95 \times 8.9 \times \frac{.31}{12} \times 40 = 8.9 \text{ K}$$

$$a_{max} = \frac{S_s}{A} \frac{M_u + M_{soil}}{M_{seis}} = \frac{1.35}{1} \frac{8.9 + .7}{6.8} = \underline{\underline{1.9 g}}$$

Upper wall:

$$H_{ave} = \frac{10.5}{2} = 5.2' \quad P_{soil} = K_o \gamma H_{ave} = .4 \times .125 \times 5.2 = .26 \text{ ksf}$$

$$P_{seis} = 1.15 \times H_{max} = 1.15 \times .125 \times 10.5 = 1.51 \text{ ksf}$$

Square wall $a = b = 10'$

Client

WO/Job No.

Subject

Seismic

Date

5/96

By

EOW

Checked

5/28/96

By

MEW

Location

Revised

By

241-A-350 Seismic

$$M_{seis} = \frac{.287}{6} \times 1.51 \times 10^2 = 7.2^k \quad M_{soil} = \frac{.26}{1.51} \times 7.2 = 1.2^k$$

$$\#5 @ 12" \quad h = 12" \quad 1.5" \text{ clr} \quad d = 10.5" \quad j = .95$$

$$M_u = .95 \times 10.5 \times \frac{.31}{12} \times 40 = 10.3^k$$

$$a_{max} = \frac{1.35}{1} \times \frac{10.3 - 1.2}{7.2} = \underline{\underline{1.7g}}$$

Caisson: 12 ga corrugated steel is ductile, judged OK

Note: Expect 2" corrugations, $r = 0.69"$ (rad. gyr.)

Per Struct Engr Hdbk Chap 29

$$\sigma_{allow} = \frac{30000 \text{ psi}}{2k} \quad k = 90\% \text{ T-99 std soil dens.} = .75$$

$$\sigma_{allow} = \frac{30000 \text{ psi}}{2 \times .75} = 20 \text{ ksi}$$

$$t = .109" \quad (12 \text{ ga}) \quad P_{allow} = \sigma_{allow} \frac{2t}{D} = \frac{20 \text{ ksi} \times 2 \times .107}{8 \times 12}$$

$$P_{allow} = 45 \text{ psi}$$

Lateral soil pressures (static)

$$\text{Top } p_{soil} = 0.4 \times .125 \times 12' = .6 \text{ ksf} = 4 \text{ psi}$$

$$\text{Bottom } = 0.4 \times .125 \times 29' = 1.5 \text{ ksf} = 16 \text{ psi}$$

are small compared to $P_{allow} \Rightarrow$ Caisson OK
in role to provide confinement for soil
supporting pit during shaking.

Client	WO/Job No.		
Subject	Seismic	Date	5/19/96 By EOW
		Checked	5/20/96 By MHW
Location		Revised	By

244-A Seismic

Vertical

Cover blocks: Appl: $p_{live} = 0.67 \text{ ksf}$ $h = 20" = 1.7'$

$$1g: p_{seis} = .15 \times 1.7 = .25 \text{ ksf}$$

$$a_{max} = \frac{1.35}{2} \cdot \frac{.67}{.25} = \underline{10K}$$

Horizontal:

Cover blks & wall

#5 @ 10"

$$b = 1.7' + \frac{1}{3}(12 - 1.7) = 1.7 + 3.4$$

$$= 5.1$$

$$h_{we} = (1.7 \times 7.5" + 3.4 \times 12") / 5.1 = 10.5"$$

$$1.5" \text{ c/t } d = 9" \quad j = .95$$

$$M_n = .95 \times 9 \times \frac{.21}{10} \times 40 = 10.6 \text{ K}$$

$$1g: w_{seis} = .15 \times 1.7 \times 13.7 = 3.5 \text{ k/ft}$$

$$M_{seis} = \frac{wL^2}{8b} = \frac{3.5 \times 13^2}{8 \times 5.1} = 14.5 \text{ K}$$

$$P_{soil} = 150 \text{ psf} \quad w_{soil} = .15 \times 1.7 = .26 \text{ k/ft}$$

$$M_{soil} = \frac{.26}{3.5} \times 14.5 = 1.1 \text{ K}$$

$$a_{max} = \frac{1.35}{1} \cdot \frac{10.6 + 1.1}{14.5} = \underline{1.1g}$$

Upper wall: $a = 13$ $b = 17$ $a/b = 1.2$ $\beta = .376$

#5 @ 12 $d = 10.5$ $M_n = .95 \times 14.5 \times \frac{.21}{12} \times 40 = 10.3 \text{ K}$

Have = 6' $P_{soil} = .4 \times 125 \times 6 = 0.3 \text{ ksf}$

110

Client

WO/Job No.

Subject

SEISMIC

Date

5/96

By

EOW

Checked

5/10/96

By

JRW

Location

Revised

By

244-A (con'd)

$$P_{seis} = 1.15 \times H_{max} = 1.15 \times .125 \times 12 = 1.73 \text{ Ksf}$$

$$M_{seis} = \frac{\beta}{6} P L^2 = \frac{.376}{6} \times 1.73 \times 11^2 = 13.1 \text{ K}$$

$$M_{soil} = \frac{.3}{1.73} \times 13.1 = 2.3 \text{ K}$$

$$q_{max} = \frac{1.35}{1} \frac{10.3 - 2.3}{13.1} = \underline{\underline{0.82 \text{ g}}}$$

Caisson: R/C + caisson liner + grout $\Rightarrow$ stout

Client	WO/Job No.		
Subject	Seismic	Date	5/96 BY EOW
		Checked	5/28/96 BY MOW
Location		Revised	By

244-BX/TX/V

$$\#6 @ 12 \quad d = 18 - 3 = 15" \quad M_n = .95 \times 15 \times \frac{.44}{12} \times 60 = 31K$$

$$a_{max} = \frac{1.35}{1} \frac{31 - 6.4}{85} = \underline{\underline{0.39 g}}$$

Bottom wall is thicker with #11 @ 6 OK

Client

WO/Job No.

Subject

SEISMIC

Date

5/19/86

By

EDW

Checked

5/20/86

By

ARW

Location

Revised

By

244-CR

Vert: Appl: $P_{live} = 1.5/2 \text{ ft}$ $h = 2'$ $P_{seis} = .15 \times 2 = .3 \text{ ksf}$
OK

Hrzt: Cover blocks & walls: OK

Lower wall #8 @ 12" EFEW ~ 2/3 from bottom

$$h = 2' \quad 2" \text{ clv} \quad d = 22" \quad M_u = .95 \times 22 \times \frac{.79}{12} \times 40 = 55 \text{ K}$$

$$b = 26' \quad a = 29' \quad a/b = 29/26 = 1.11 \quad \beta = 1.333$$

$$H_{max} = 21.0 + 2 + 29.5 + 2.7 = 55.2'$$

$$P_{seis} = 0.8 \times .125 \times 55.2 = 5.5 \text{ K}$$

$$H_{ave} = 21 + 2 + 15 = 38' \quad P_{soil} = .4 \times .125 \times 38 = 1.9 \text{ ksf}$$

$$M_{seis} = \frac{P}{8} p b^2 = \frac{1.333}{6} \times 5.5 \times 26^2 = 206 \text{ K}$$

$$M_{soil} = \frac{1.9}{5.5} \times 206 = 71 \text{ K} > 55 \text{ K} \quad \text{N.G. (static)}$$

Refined calculations with edge restraints might show a positive margin; not in scope.

Upper wall #6 @ 9" INS

$$h = 2' \quad 2" \text{ clv} \quad d = 22" \quad M_u = .95 \times 22 \times \frac{.44}{9} \times 40 = 41 \text{ K}$$

$$b = 19' \quad a = 26' \quad a/b = 26/19 = 1.37 \quad \beta = .43$$

$$H_{max} = 55.2' \quad P_{seis} = 1.15 \times .125 \times 55.2' = 7.9 \text{ Ksf}$$

$$H_{ave} = 19/2 = 9.5' \quad P_{soil} = 0.4 \times .125 \times 9.5 = .48 \text{ Ksf}$$

$$M_{seis} = \frac{P}{8} p b^2 = \frac{.43}{6} \times 7.9 \times 19^2 = 204 \text{ K}$$

$$M_{soil} = \frac{.48}{7.9} \times 204 = 12.4 \text{ K}$$

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Client	WO/Job No.		
Subject	<u>Seismic</u>	Date	<u>5/96</u> By <u>EDW</u>
		Checked	<u>5/20/96</u> By <u>SRW</u>
Location		Revised	By

$$q_{max} = \frac{1.35}{1} \cdot \frac{41 - 12.4}{204} = 0.19 g$$

Client

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Subject

Seismic

Date

5/96

By

EOW

Checked

5/28/96

By

KRW

Location

Revised

By

244-S SeismicVert: Cover blks: Appl: $p_{live} = 0.28 \text{ ksf}$ $h = 2'$

$$a_{max} = \frac{1.35}{2} \cdot \frac{0.28}{0.15 \times 2} = \underline{\underline{0.63g}}$$

Hrztl: Cover blks & wall

$$b = 2 + \frac{1}{3}(11-2) = 5'$$

$$h_{ave} = (2 \times 11.5'' + 3 \times 16'') / 5$$

$$= 14.2''$$

$$2'' \text{ clv } d = 12.2'' \quad \#8 @ 8'' \quad j = .95$$

$$M_n = .95 \times 12.2 \times \frac{.79}{8} \times 60 = 69 \text{ K}$$

$$W_{seis} = .15 \times 2 \times 20 = 6 \text{ K/11}$$

$$M_{seis} = \frac{W L^2}{8 b} = \frac{6 \times 20^2}{8 \times 5} = 60 \text{ K}$$

$$W_{soil} = 2' \times 150 (1 + \frac{1}{2}) = 146 \text{ K/11}$$

$$M_{soil} = \frac{.46}{6} 60 = 4.6 \text{ K/11}$$

$$a_{max} = \frac{1.35}{1} \cdot \frac{69 + 4.6}{60} = \underline{\underline{1.6g}}$$

Upper well

$$H_{max} = 11' \quad p_{seis} = 1.15 \times H = 1.15 \times .125 \times 11 = 1.58 \text{ ksf}$$

$$b = 9' \quad a = 20' \quad a/b = 2.2 \quad \beta = .63$$

$$\#8 @ 12 \quad d = 16-2 = 14'' \quad M_n = .95 \times 14 \times \frac{.79}{12} \times 60 = 53 \text{ K}$$

$$M_{seis} = \frac{\beta}{6} p b^2 = \frac{.63}{6} \times 1.58 \times 9^2 = 13.4 \text{ K}$$

$$H_{ave} = 5.5' \quad p_{soil} = 0.4 \times .125 \times 5.5 = .28 \text{ ksf}$$

$$M_{soil} = \frac{.28}{1.58} \times 13.4 = 2.3 \text{ K}$$

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	Checked <u>SLW/gb</u> By <u>SLW</u>
Location	Revised _____ By _____

$$a_{max} = \frac{1.35}{1} \frac{53 - 2.3}{13.4} = \underline{\underline{5g}}$$

CAISSON: $H_{max} = 66' - 61.9' = 47' \quad \gamma_{MWTP} = 0.033 \rho_o$

$$v = C \gamma / 2 = 1.326 \times \frac{0.033}{2 \times 100} = 215 \text{ psi}$$

$$v_c = 2\sqrt{f_c} = 110 \text{ psi}$$

$$v_s = \frac{A_v f_y}{s_2 d} \quad d = 0.8h \quad h = 12" \quad \#5 @ 12 \text{ hr} \geq 1$$

$$v_s = \frac{.71 \times 6003}{.8 \times 12 \times 12} = 161 \text{ psi}$$

$$v_n = v_c + v_s = 110 + 161 = 271 \text{ psi}$$

$$a_{max} = 1.35 \times .38 \times \frac{271}{215} = \underline{\underline{0.64g}}$$

Note: extra strength expected from
corrugated liner & grout

Client	WO/Job No.		
Subject	SEISMIC	Date	5/96 By EBG
Location		Checked	5/20/96 By ARW
		Revised	By

241-S-304 Seismic

 Vertical: Cover blks: App 1 $P_{live} = 0.73 \text{ ksf}$ $h = 2.5'$

$$a_{max} = \frac{1.35}{2} \frac{.73}{.15 \times 2.5} = 1.39$$

Horiz: Cover blks & wall

$$b = 1.7 + \frac{1}{3}(7.5 - 1.7) = 3.3'$$

$$h_{ave} = (7.7 \times 7.5' + 1.9 \times 12) / 33$$

$$= 10.8''$$

$$clr = 1.5'' \quad d = 9.3'' \quad \#5 @ 12$$

$$M_n = .95 \times 9.3 \times \frac{.31}{12} \times 60 = 13.6 \text{ K}$$

$$W_{seis} = 8 h s = .15 \times 2.5 \times 11.8 = 4.4 \text{ K}$$

 $W_{soil} \approx \text{neglect}$

$$M_{seis} = \frac{W L^2}{8 b} = \frac{4.4 \times 11.8^2}{8 \times 3.3} = 23.2 \text{ K}$$

$$a_{max} = \frac{1.35}{1} \frac{13.6}{23.2} = 0.79 g$$

Caisson roof: Pit is not concentric with caisson & seismic inertia (trial) induces roof loads

$$W = (5.8 \times 4 \times 11.8 + 2.5 \times 13.2) \times .15 \text{ kcf}$$

$$= 41 \text{ K} + 63 \text{ K} = 104 \text{ K}$$

$$F_i = W @ 1g$$

$$h_i = (41 \times 5.8/2 + 63 \times (5.8 + 2.5/2)) / 104$$

$$= 5.4'$$

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Client	WO/Job No.		
Subject	<u>241-6-304</u>	<u>Seismic</u>	Date <u>7/96</u> By <u>EDW</u>
		Checked <u>5/28/96</u>	By <u>MRW</u>
Location		Revised	By

$$\begin{aligned}
 F_w &= \text{wall soil load (1g)} \\
 &= 1.15 \times 1.25 \times 5.5 (5.5 \times 12.8 \sqrt{2}) \\
 &= 78^k
 \end{aligned}$$

$$h_w = 5.8/2 = 2.9'$$

$$\begin{aligned}
 F_R &= \text{reaction load on roof} \\
 &= (F_w h_w + F_i h_i) / h_R \\
 &= (78^k \times 2.9 + 104^k \times 5.4') / 8.5 \\
 &= 93^k \quad (1g)
 \end{aligned}$$

$$\text{From pit wt: } F_{Dw} = \frac{2.5' W}{8.5'} = \frac{2.5}{8.5} 104 = 31^k$$

W8 beams support roof wt

R/C root: #6 @ 9" $h=15"$ $clr=3/4"$ $L=14.25"$ $j=.95$

$$M_n = .95 \times 14.25 \times \frac{.44}{9} \times 60 = 40^k$$

Est F_R distributed over center 20% square plate

$$a=b=\sqrt{\frac{\pi}{4}} 2R = \sqrt{.785} \times 16 = 14.2'$$

Roarke 4th $M = \frac{\beta}{6} F$ $\beta = 1.82$ @ 20% $M = 0.3 F$

$$M_R = 0.3 F_R = 0.3 \times 93 = 28^k$$

$$M_{Dw} = 0.3 F_{Dw} = 0.3 \times 31^k = 9^k$$

$$a_{max} = \frac{1.35}{2} \cdot \frac{M_n - M_{Dw}}{M_R} = \frac{1.35}{2} \cdot \frac{40 - 9}{28} = \underline{\underline{0.75g}}$$

Reversing soil-wall and pit inertia loads will lift roof off caisson wall at F_L where dowel are

DESIGN ANALYSIS

Client

WO/Job No.

Subject 241-S-304

Seismic

Date

5/29

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EDW

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ARW

Location

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By

not long enough, Lift-off
 $\Rightarrow$ impact, damage & loss
 of shear capacity for down
 loads. Est soil over roof
 load

$$\begin{aligned}
 W_s &= (\overline{ht} \times \overline{en} \times \overline{width}) \gamma_s \\
 &= (5.5' \times 17' \times 4') \gamma_s \\
 &= 370 \text{ ft}^3 \times 125 \text{ pcf} = 47 \text{ K}
 \end{aligned}$$

F_L (dead wt):

$$F_{DW} = \frac{104 \text{ K} \times 2.5' + 47 \text{ K} \times 9.5'}{13'} = 59 \text{ K}$$

F_L (lg):

$$F_L = \frac{78 \text{ K} \times 2.9 + 104 \text{ K} \times 5.4}{13} = 61 \text{ K}$$

$$a_{max} = \frac{1.35}{2} \cdot \frac{F_{DW}}{F_L} = \frac{1.35}{2} \cdot \frac{59}{61} = \underline{\underline{0.60g}}$$

DESIGN ANALYSIS

Calc. No. P. 4-33

Revision

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Client

WO/Job No.

Subject

Seismic

Date

5/29

By

EGW

Checked

5/30/96

By

ARW

Location

Revised

By

200 Series SeismicVertical: Flat roof: see App 5: $M_n = 26^K$ $p_{soil} = 9.7 \text{ psi}$
 $= 1.4 \text{ ksf}$ Est. equiv plate as $8' \times 9'$, $\beta = 0.3$

$$M = \frac{\beta}{6} p b^2 = \frac{.3}{6} \times 1.4 \times 8^2 = 4.5^K$$

$$a_{max} = \frac{1.35}{2} \times \frac{26}{4.5} = \underline{\underline{4 g}}$$

Per App 1: $p_{live} = 1.0 \text{ ksf}$

$$a_{max} = \frac{1.35}{2} \times \frac{1.0}{1.4} = \underline{\underline{0.48 g}} \quad (\text{conservative})$$

Hr=1:

$$H_{max} = 37' \quad \gamma_{MWF} = .028\%$$

$$U = 1.326 \times \frac{.028}{2 \times 100} = 180 \text{ psi}$$

$$U_2 = 2\sqrt{U} = 110 \text{ psi}$$

$$U_1 = 182 \text{ psi} \quad (\text{Rumble, PE-2'}) \quad U_n = 110 + 172 = 282 \text{ psi}$$

$$a_{max} = \frac{1.35}{1} \times .38 \times \frac{282}{180} = \underline{\underline{0.80 g}}$$

Appendix 5

Internal Static Vacuum

DCRTs, catch tanks and 200-series SSTs are structurally evaluated for a vacuum loading of -8 psig to determine if a an opening could result to the atmosphere.

Most of the structures have steel tanks inside a vault or caisson under a pit capped with cover blocks at the surface. The first check in such cases is an evaluation for buckling of the steel tank. Although the procedure involves a capacity reduction factor of 0.5 which appears to provide a margin between design an minimum test data, it is considered prudent for this level of effort and lack any of tank condition assessments to retain the factor and interpret the result as a minimum strength. Most steel tanks are ok at the -8 psig load. For those that are not and those for which information was not located, the vacuum is extended to the enclosing vault or caisson on the basis of volume ratios, and the vault is then check. All these cases prove satisfactory, except one in which the side wall is marginal. Nevertheless, a significant opening is not expected.

Some steel tanks are buried directly in the soil, and the major loading appears to be soil load. Considerable variation can be found in procedures for assessing soil loads on buried pipe. Simple hydrostatic pressure based on soil density was used for this level of effort, and the vacuum loading was added directly. Some directly buried tanks fail under this combined loading, but the depth to diameter ratio is so large that a significant opening to the atmosphere is not expected.

This load case produces no expected openings to the atmosphere as a result of structural failure. Normal openings to the atmosphere have not been considered. In the case of directly buried tanks that collapse, tearing need not be significant, and a liquid spout caused by bladder pressurization by the soil and tank wall is not precluded. This possibility has not been evaluated.

Table 5.1 Internal Vacuum Summary

Tank	Structural Item	OK/NG by Item	Overall
241-A-302-A	Steel tank	OK	OK
241-A-417	Tank roof	OK	OK
241-AX-152	Tank roof	OK	OK
241-AZ-151	Tank roof Tank side wall	OK Marginal	OK, no opening
241-ER-311	Steel tank	NG	NG, no opening
241-EW-151	Steel tank Vault roof	NG OK	OK
241-TX-302-C	Steel tank	OK	OK
241-UX-302-A	Steel tank	NG	NG, no opening
241-U-301-B	R/C tank	OK	OK
241-A-350	Steel tank Caisson roof	Not evaluated OK	OK
244-BX/TX/U	Steel tank Vault roof	NG OK	OK
244-CR	Steel tank Vault roof	Not evaluated OK	OK
244-S	Steel tank Caisson roof	NG OK	OK
241-S-304	Steel tank	OK	OK
200 Series	R/C tank	OK	OK
DST/SST	R/C tank	OK	OK

Notes: NG = no good (fails structural criteria)
OK = satisfies structural criteria

Client _____

WO/Job No. _____

Subject Int VacuumDate 5/13/96 By EDGChecked 5/96 By KCTH

Location _____

Revised _____ By _____

241-A-302-A

2.1 Internal Vacuum

Mat'l SSA A-416-304LERSC: $L/r = 16.5/4.5 = 3.7$ (Cg1) $t/r = 9/16/12/4.5 = 0.01$

$$\beta = 2.6 \times 10^{-4}$$

$$P_{cr} = E \frac{t}{r} \beta = 28 \times 10^6 \times 0.01 \times 2.6 \times 10^{-4}$$

$$= 76 \text{ psi}$$

$$P_u = \frac{1}{2} P_{cr} = 38 \text{ psi}$$

$$P = P_{soil} + P_{vac} = 22 \times 125/144 + 8 = 27 \text{ psi} < 38 \text{ psi} \quad \text{OK}$$

(Head):

$$P_u = 0.18 \times 1.21 E (t/R)^2$$

$$= 0.18 \times 1.21 \times 28 \times 10^6 \left(\frac{7/16}{102} \right)^2 = 112 \text{ psi} > 27 \text{ psi} \quad \text{OK}$$

Tank does not collapse.

Client

WO/Job No.

Subject

Date 5/13/96 By EDW

Checked 5/19/96

By KOS/In

Location

Revised

By

241-A-417

2.1 Internal vacuum - 8 psig

Dwg H-2-56800

Tank roof acts as
double tee beam. Est.
worst situation is roof
acting as flat plate.

$$L = \frac{1}{2} (24.6 - 9.5) = 7.5'$$

$$W = P_{soil} + P_{vac}$$

$$= 14' \times 125 \text{ pcf} / 144 \text{ in}^2 / \text{ft}^2 + 8 \text{ psi}$$

$$= 20 \text{ psi}$$

$$M_u = \frac{WL^2}{8} = \frac{20 \text{ psi}}{8} (7.5 \times 12)^2$$

$$= 20000' = 20 \text{ K}$$

Tank roof $h = 10''$ $7.5''$ dia

$$d = 8.5'' \quad \#6 @ 6'' \text{ EF}$$

$$\rho_s = .44 / 6 / 8.5 = .008 \quad j = .94 \quad \text{Est } f_y = 40 \text{ ksi}$$

$$M_n = j d A_s f_y = 0.94 \times 8.5'' \times 0.44'' / 6 \times 40 \text{ ksi} = 23 \text{ K} > 20 \text{ K}$$

Dowels are $\#6 @ 9''$ EF from roof to pit well OK

$$R = \frac{5}{8} WL = \frac{5}{8} 20 \text{ psi} \times 9'' \times 7.5 \times 12'' = 10000' = 10 \text{ K}$$

$$A_s f_y = 0.44 \times 40 \text{ ksi} = 17.6 \text{ K} > 10 \text{ K} \quad \text{OK}$$

Roof remains intact.

Client	WO/Job No.		
Subject	Date <u>5/13/96</u>	By <u>EDW</u>	
	Checked <u>5/96</u>	By <u>KCT/In</u>	
Location	Revised	By	

241-AX-152, 241-AZ-151

2.1 Internal vacuum - 8 psi g

241-AX-152

Tank roof: $h = 8"$ lined S.S.

#4 @ 8 EW BOT

$$P_s = .2/8/8 = .008$$

$$j = 0.94$$

$$d = 8 - \frac{3}{4} = 7.25"$$

$$\text{Est } f_y = 40 \text{ ksi}$$

$$M_n = j d A_s f_y = .94 \times 7.25" \times .20"/8 \times 40 \text{ ksi} = 6.8 \text{ K}$$

Use $8/12 \times 1 \text{ psi} = .7 \text{ psi}$ for floor wt

$$P = P_{\text{vac}} + P_{\text{conc}} = 8 + 0.7 = 8.7 \text{ psi}$$

$$M = \frac{P L^2}{8} = \frac{8.7 \text{ psi}}{8} (72")^2 = 5600 \text{ #} = 5.6 \text{ K} < 6.8 \text{ K} \text{ OK}$$

$$V_n = 2 \sqrt{f_c} d = 2 \sqrt{3000} \times 7.25 = 750 \text{ #/in}$$

$$\check{V} = \frac{P L}{2} = 8.7 \times 72/2 = 310 \text{ #/in} < 750 \text{ #/in} \text{ OK}$$

Side wall: $h = 18"$ $d = 18 - 3/4 = 17.25"$ #6 @ 10" vert. inside

$$j = 0.95 \quad f_y = 40 \text{ ksi}$$

$$M_n = j d A_s f_y = 0.95 \times 17.25" \times .44"/8 \times 40 \text{ ksi} = 37 \text{ K}$$

Soil load: $K_o = 0.4$ $\gamma = 125 \text{ pcf}$ $H = 21.5'$

$$W: P_{\text{soil}} = K_o \gamma H = 0.4 \times 125 \text{ pcf} \times 21.5' = 1080 \text{ psf}$$

$$= 7.5 \text{ psi}$$

$$P_{\text{tot}} = P_{\text{soil}} + P_{\text{vac}} = 7.5 + 8 = 15.5 \text{ psi}$$

Client	WO/Job No.
Subject	Date <u>5/13/96</u> By <u>EDW</u>
	Checked <u>5/96</u> By <u>RC:Jm</u>
Location	Revised _____ By _____

241-AX-152 (con'd)

2.1 (con'd)

$$M = \frac{wL^2}{8} = 15.5 \times (11.5 \times 12") / 8 = 37^k \leq 37^k \quad \underline{OK}$$

Expect less load + more strength:

- Partial fixity at edges
- Soil load decrease with wall movement

241-AZ-151

Similar to 241-AX-152. Same conclusion for vault roof (OK). Vault side walls do not pass since rebar is #6@12" vs #6@10" ($m_s = \frac{16}{12} - 1 = -.17$).
Calcs are conservative (see above), and bending failure in side wall is not expected to give significant opening to atmosphere.

Client	WO/Job No.
Subject	Date 5/10/96 By EOLW
	Checked 5/96 By RCTJ
Location	Revised By

241-ER-311

2.1. Internal vacuum - 8 psig

SAR-067

$$t = 9/16" \quad H = 20' \quad D = 9.5' \quad L = 40'$$

ERSC for capacity:

$$L/r = 40/9.5 \times 2 = 8.4$$

$$t/r = 9/16/12/9.5 \times 2 = 0.01$$

$$\beta = 1e-4$$

$$P_{cr} = E \frac{t}{r} \beta = 28e6 \text{ psi} \times 0.01 \times 1e-4 = 28 \text{ psi}$$

$$P_y \approx f_y \frac{t}{r} \approx 30e3 \text{ psi} \times 0.01 = 300 \text{ psi} \gg P_{cr}$$

$$\alpha = 1/2 \quad P_u = \alpha P_{cr} = \frac{1}{2} \times 28 \text{ psi} = 14 \text{ psi}$$

$$\text{Demand: } P_{soil} = \gamma(H+r) = 125 \text{ pcf} \times 25' / 144 \text{ in}^2/\text{ft}^2 \\ = 22 \text{ psi}$$

$$P_{vac} = 8 \text{ psi}$$

$$P_{tot} = P_{soil} + P_{vac} = 22 + 8 = 30 \text{ psi}$$

$$> 14 \text{ psi} \quad \underline{\text{No good}}$$

Tank could collapse, tearing expected, no significant opening to atmosphere with $H/D = 2$

Client _____

WO/Job No. _____

Subject _____

Date 5/9/96 By EDW

Checked 5/19/96 By [Signature]

Location _____

Revised _____ By _____

241-EW-151

2.1 Internal vacuum - 8 psig

SAR-029 lists tank as 309Cb SS

$$L = 6.8' \quad t = 3/8" \quad D = 4.5'$$

$$t_{bot} = 3/8" \quad t_{top} = 1/2"$$

$$ERSC: L/r = 6.8/2.25 = 3.0$$

$$t/r = 3/8/12/2.25 = 0.014$$

$$B = 5e-4$$

$$p_{cr} = 28061 \text{ psi} \times 0.014 \times 5e-4 \\ = 196 \text{ psi}$$

$$p_u = \alpha p_{cr} = \frac{1}{2} \times 196 = 98 \text{ psi} \gg 8 \text{ psi} \quad \text{OK (Cyl)}$$

Heads: use ASME Sect XIII AD-700

$$\text{Req'd head thk: } T = d \sqrt{\frac{CP}{KS}}$$

$$d = D = 54" \quad p = 8 \text{ psi}$$

Use $k = 1.2$ with S , est basic $S.S. S = 20 \text{ ksi}$

$$KS = 1.2 \times 20 = 24 \text{ ksi}$$

$$C = 0.44 \text{ (max in Fig AD-701.2)}$$

$$T = 54" \sqrt{\frac{0.44 \times 8}{24 \times 2}} = 0.65 > t_{top} \text{ or } t_{bot} \quad \text{N.G. (heads)}$$

 $\therefore$ heads may collapse. (Check)

Client	WO/Job No.
Subject	Date 5/96 By EOW
	Checked 5/96 By G. J. L.
Location	Revised By

241-EW-151 (con'd)

Expansion:

$$V_{tk} = \frac{\pi}{4} \times 4.5^2 \times 6.8 = 110 \text{ ft}^3$$

$$V_{\text{vault}} = 8.3 \times 8.5 \times 8 = 560 \text{ ft}^3$$

$$P_{\text{vault}} = P_{tk} \frac{V_{tk}}{V_{\text{vault}}} = 8 \text{ psi} \frac{110}{560} = 1.6 \text{ psi}$$

Vault roof: $h = 18"$ 1.5" clear #8 @ 9" for $L = 8.5'$

Use 1.5 psi for concrete wt

$$P_{\text{tot}} = P_{\text{vault}} + P_{\text{concr}} = 1.6 + 1.5 = 3 \text{ psi}$$

$$M_n: \rho_j = 0.95 \quad d = 16.5" \quad \text{Use } f_y = 40 \text{ ksi}$$

$$M_n = \rho_j d A_s f_y = 0.95 \times 16.5 \times 0.79 \times 40 \text{ ksi} = 55 \text{ K}$$

$$M = \frac{w L^2}{8} = 3 \text{ psi} \times (8.5 \times 12)^2 / 8 = 3900 \text{ ft-lb} = 4 \text{ K} < 52 \text{ K}$$

OK

Vault ceiling holds easily against vacuum.

Client	WO/Job No.
Subject	Date 5/9/96 By EOW
	Checked 5/16 By [Signature]
Location	Revised By

21 241-TX-302C,
241-UX-302A Internal vacuum - Pprig

$$ERSC: t/r = 9/16 / 4.5/12 = .010$$

$$L/r = 36 / 4.5 = 8$$

$$\beta = 1.1E-4$$

$$P_{cr} = 28EG \times .010 \times 1.1E-4 = 31 \text{ psi}$$

$$P_{tot} = \gamma H + p_{vac}$$

$$= 125 \times 29/144 + 8$$

$$= 33 \text{ psi (TX)} > 31 \text{ psi N.G. (H=2-852)}$$

$$= 125 \times 20/144 + 8 = 25 \text{ psi (UX)} < 31 \text{ psi OK}$$

Check head:

$$P_{cr} = 1.21E (T/R)^2 = 1.21 \times 28EG \text{ psi} \left(\frac{.563}{90} \right)^2 = 1330 \text{ psi}$$

$$\gamma = 0.18 \quad P_u = 0.18 \times 1330 \text{ psi} = 240 \text{ psi} \gg 31 \text{ psi OK}$$

UX does not collapse (H=20')

TX may collapse (H=29'). Soil would probably cave in, but no significant opening expected with H=29' vs D=9'. Whale spout thru riser is not addressed, but note that tearing might not take place as a result of collapse.

Client	WO/Job No.		
Subject	Date <u>5/9/96</u>	By <u>EDW</u>	
	Checked <u>5/9/96</u>	By <u>RODIN</u>	
Location	Revised	By	

241-U-301-B

2.1 Internal vacuum - 8 psi

Ave. soil/cover pressure on dome

$$P_{soil} = \left(6 + \frac{2.5}{2}\right) 125 / 14.4$$

$$= 6.3 \text{ psi}$$

$$P_{tot} = P_{soil} + P_{vac} = 6.3 + 8 = 14.3 \text{ psi}$$

Check dome membrane

$$\frac{PR}{2T} = \frac{14.3 \text{ psi} \times 21.3}{2 \times 0.375} = 460 \text{ psi} < 900 \text{ psi work. } \sigma. \quad \underline{OK}$$

Check dome buckling (Zarghameh & Heger, ACI Title No 80-45)

$$\beta_c = 0.5 \quad \beta_m = 0.5 \quad \phi = 0.7$$

$$P_{cr} = \phi \beta_c \beta_m 0.6 E \left(\frac{t}{R}\right)^2$$

$$= 0.7 \times 0.5 \times 0.5 \times 0.6 \times 3e6 \text{ psi} \left(\frac{0.375}{21.3}\right)^2 = 85 \text{ psi}$$

$$> 15 \text{ psi} \quad \underline{OK}$$

Check capacity of tension ring via A-105 test (ARH-R-47).

Pressure & stress = const under linear dim/scale. A-105

1/10 model ($r = 45''$) gave $\sim 5 \times 6'$ soil load as capacity.241-A has $9+9+7+6+5+3 \times 4 = 48$ #11 hoop bars in haunchwith $r = 40'$. $A_s = 48 \times 1.56 = 75 \text{ in}^2$. 1/4 scale = 7

$$A_{sA} = 75/16 = 4.7 \text{ in}^2, \text{ Have } 4 \text{ } \#9 = 4 \times 1.00 = 4 \text{ in}^2 = A_{stk}$$

$$Cap = 5 \times 6.3 \text{ psi} \frac{A_{stk}}{A_{sA}} = 5 \times 6.3 \times \frac{4}{4.7} = 27 \text{ psi} > 15 \text{ psi} \quad \underline{OK}$$

Client

WO/Job No.

Subject

Date 5/9/96 By EOW

Checked 5/9/96 By BCT

Location

Revised

By

241-A-350 (BCT)

2.1 Internal vacuum - P psig

Details for tank were not found.

Assume tank collapsed. Scale from
H-2-70538 & related dwgs

$$\text{Tank: } V_{tk} = \frac{\pi}{4} \times 4.5^2 \times 6 = 95 \text{ ft}^3$$

$$\text{Caisson: } V_{cais} = \frac{\pi}{4} \times 8^2 \times 21 = 1060 \text{ ft}^3$$

$$P_{cais} = P_{tk} V_{tk} / V_{cais}$$
$$= 8 \times 95 / 1060 = 0.7 \text{ psi}$$

$$\text{Caisson roof: } h = 12" \quad d = 12 - 3 = 9"$$
$$\#5 @ 12" \quad j = .95$$

$$f_y = 40 \text{ ksi}$$

$$M_u = j d A_s f_y = .95 \times 9" \times .31 / 12 \times 40 \text{ ksi}$$
$$= 8.8 \text{ K}$$

$$\text{Use 1 psi for concrete } P_{tot} = P_{cais} + P_{conc} = 0.7 + 1 = 1.7 \text{ psi}$$

$$M_u = \frac{P}{S} r^2 = \frac{1.7 \text{ psi}}{5} \times (48")^2 = 920 \text{ K} = 0.9 \text{ K} < 8.8 \text{ K}$$

OK

Roof holds

2.5 Gasoline burn, 140 psig

Same as 244-S

Client	WO/Job No.		
Subject	Date <u>5/28/94</u>	By <u>EDW</u>	
	Checked <u>5/96</u>	By <u>KCTL</u>	
Location	Revised	By	

244-A

2.1 Internal vacuum -8 psig

CYI file not available. Assume tank collapses.

$$V_{\text{tank}} = \frac{\pi}{4} \times 14^2 \times 16.5 = 2500 \text{ ft}^3$$

$$V_{\text{caiss}} = \frac{\pi}{4} \times 17^2 \times 22.5 = 5100 \text{ ft}^3$$

$$P_{\text{caiss}} = P_{\text{tank}} \frac{V_{\text{tank}}}{V_{\text{caiss}}} = 8 \times \frac{2500}{5100} = 4 \text{ psi}$$

Caisson roof: Hoop #5 @ 12 j = .95 h = 12" c/r = 3/4"

$$d = 11.25" \quad M_n = .95 \times 11.25 \times \frac{.31}{12} \times 40 = 11 \text{ K}$$

WSB beams support concrete & equipment

$$M = \frac{1}{5} p r^2 = \frac{4 \text{ psi}}{5} (8.5 \times 12")^2 = 8 \text{ K} < 11 \text{ K} \quad \underline{\underline{\text{OK}}}$$

2.5 Gasoline burn at 140 psi - vacuum of -8 psi

uses most of capacity. $\therefore$ 140 psi with DLF > 1
obviously fails.

Client	WO/Job No.		
Subject	Date <u>5/9/96</u>	By <u>EDW</u>	
	Checked <u>5/9/96</u>	By <u>RC/TL</u>	
Location	Revised	By	

244-BX/TX/U (DCRT)

2.1 Internal vacuum - Spring

Tank (SAR) $D=12'$ $L=35'$ $t=\frac{1}{4}"$ G.S.

$$ERCS: L/r = 35/6 = 5.8 \quad t/r = .25/6/12 = 0.0035$$

$$\beta = 3.38 \times 10^{-5}$$

$$P_{cr} = E \frac{t}{r} \beta = 2866 \times 0.0035 \times 3.38 \times 10^{-5} = 3.2 \text{ psi}$$

$$P_u = \frac{1}{2} P_{cr} = \frac{1}{2} \times 3.2 = 1.6 \text{ psi}$$

Tank collapses

Heads are 3' deep (SAR)

$$V_{tk} = \frac{\pi}{4} 12^2 \times (35+3) = 4300 \text{ ft}^3 \quad \text{Length} = 48'$$

$$V_{vault} = 16 \times 18 \times (t8 - 2 \times 2) = 12700 \text{ ft}^3$$

$$P_{vault} = P_{tk} \frac{V_{tk}}{V_{vault}} = 8 \text{ psi} \times 4300 / 12700 = 2.7 \text{ psi}$$

Vault roof:

W18 & W14 assumed to hold concrete & pit equip

$$h = 2' \text{ clr} = 1.5" \quad 3" \text{ dk} \quad d = 24 - 3 - 1.5 = 19.5" \quad \#11 @ 6"$$

$$f_y = 60 \text{ ksi} \quad p = 1.56 / 6 / 19.5 = .013 \quad j = 0.90$$

$$M_u = j d A_s f_y = 0.90 \times 19.5 \times 1.56 / 6 \times 60 = 270 \text{ K}$$

$$M = w \frac{L^2}{8} = \frac{8 \text{ psi}}{8} (18 \times 12")^2 = 47000 \text{ K} = 47 \text{ K} < 270 \text{ K} \quad \text{OK}$$

Vault holds under vacuum loading

Client

WO/Job No.

Subject

Date 5/9/96 By EDWJChecked 5/9/0By WCT/In

Location

Revised

By

244-BX/TX/U (con'd)

2.5 Gasoline burn at 140 psi

Check construction joint at top of vault wall.

Bars are #11 @ 6 EF

$$\text{Bar Cap/lin} = 2 \times 1.56 \frac{\text{in}^2}{\text{ft}} \times 60 \text{ ksi} = 31 \text{ K/ft}$$

$$\begin{aligned} \text{Pressure load} &= p A / C & C &= \text{perimeter} \\ &= 1.2 \times 140 \times 4.4 \times 18' / (2 \times (44' + 18')) \times 12 \text{ in/ft} \\ &= 13 \text{ K/ft} < 31 \text{ K/ft} \quad \text{OK} \end{aligned}$$

Check shear around 19' x 18' pit floor

$$V = \frac{p A}{C} = \frac{140 \text{ psi} \times 1.2 \times 19' \times 18' \times 12 \text{ in/ft}}{2 (19' + 18')} = 19000 \text{ #/ft}$$

$$V_n = 2 \sqrt{f_c'} d = 2 \sqrt{4000 \text{ psi}} \times 19.5 \text{ in} = 2500 \text{ #/ft} < 19000 \text{ #/ft} \quad \text{N.G.}$$

∴ vault/roof is expected to blow out, cover blocks are lifted out, open to atmosphere

DESIGN ANALYSIS

Calc. No. P.5-16

Revision

Page No. 14 of

Client

WO/Job No.

Subject

Date 5/9/96 By EOW

Checked 5/9/96 By [Signature]

Location

Revised By

274CR (DCRT)

2.1 Internal vacuum - 8 psig

Pit walls act as tee beam (massive).

Vault ceiling is 2' thick, massive,
reinforced. Check side walls:

Ave depth:

$$H = 20.3 + 2 + 29.5/2 = 37'$$

$$P_{soil} = K_o \gamma H = 0.4 \times 125 \text{ pcf} \times 37' \\ \times 1 \frac{1}{2} / 144 \text{ in}^2$$

$$= 12.8 \text{ psi}$$

$$\frac{P_{soil} + P_{vac}}{P_{soil}} = \frac{12.8 + 8}{12.8} = 1.63$$

so P_{vac} is in normal side soil safety factor of 1.7, OK
Vault does not collapse (see gasoline burn calcs below)

2.5 Gasoline vapor burn at 140 psf

check 8' span (pit floor): $h = 2'$ $d = 22"$ $1" \square @ 6"$

$$M_n = 0.95 \times 22' \times 1 \frac{1}{2} / 6" \times 40 \text{ ksi} = 139 \text{ K}$$

Use DLF = 1.2

$$M = \frac{wL^2}{8} = \frac{1.2 \times 140 (96")^2}{8} = 190000 = 190 \text{ K} > 139 \text{ K} \text{ N.G.}$$

Client	WO/Job No.
Subject	Date 5/9/96 By EOLJ
	Checked 5/9/96 By KOTJ
Location	Revised By

244-CR

Check shear

$$V = \frac{wL}{2} = \frac{1.2 \times 140 \text{ psi}}{2} \times 96 = 8000 \text{ #/ft}$$

$$V_n = 2\sqrt{f'_c} d = 2\sqrt{3000} \text{ psi} \times 22" = 2400 \text{ #/ft} < 8000 \text{ #/ft N.G.}$$

∴ Vault does not hold; put cover blocks blow off,
open to atmosphere.

2.1 Note: Info on tanks in vault not found. Collapse
under vacuum assumed & vault sees full vacuum.

DESIGN ANALYSIS

Client

WO/Job No.

Subject

Date 5/9/96

By EOW

Checked 5/9/96

By K. J. T. M.

Location

Revised

By

244-S (DCRT)

2.1 Internal vacuum - 8 psig

Tank in caisson flat bottomed $t = 1/2"$. Will collapse
Volumes

Tank: $L = 17'$ $D = 15'$

H2 71049

$$V_{tt} = \frac{\pi}{4} 17^2 \times 15 = 3300 \text{ ft}^3$$

Caisson: $L = 21'$ $D = 20'$

H2 71043

$$V_{cais} = \frac{\pi}{4} 20^2 \times 21 = 6600 \text{ ft}^3$$

$$P_{cais} = P_{tt} V_{tt} / V_{cais} = 8 \times 3300 / 6600 = 4 \text{ psi}$$

Caisson roof: $h = 12"$ incl 3" metal dte, 3/4" dr

$$d = 12 - 3 - .25 = 8.75"$$

$$f_y = 60 \text{ ksi} \quad \#5 @ 12 \text{ EW} \quad j = .95$$

$$M_n = j d A_s f_y = .95 \times 8.75" \times .31" / 12 \times 60 \text{ ksi} = 12.9 \text{ K}$$

W12 x 22 beams under roof are taken
to support roof & equipment.

$$M = \frac{P_{cais}}{5} r^2 = \frac{4 \text{ psi}}{5} (120")^2 = 11500 \text{ ft-lb} = 11.5 \text{ K}$$

$$< 12.9 \text{ K} \quad \text{OK}$$

2.5 Gasoline burn, 140 psig

Caisson roof capacity clearly exceed

Pressure on cover blocker = many times dead weight

Tank tearing expected, contents fully open to atmos.

Client	WO/Job No.	
Subject	Date 5/9/96	By EOW
	Checked 5/9/96	By KOT
Location	Revised	By

241-S-304 (DCRT)

2.1 Internal vacuum - 8 psig

Tank

ERSC $L/r = 15/4.5 = 3.3$

$t/r = .625/12/4.5 = .0116$

$\beta = 3.5e-4$

$$p_{cr} = Et/r \beta = 28e6 \times .0116 \times 3.5e-4$$
$$= 114$$

$$p_u = \frac{1}{2} p_{cr} = 57 \text{ psi} > 8 \text{ psi} \quad \text{OK}$$

Tank holds under vacuum

2.5 Gasoline burn 140 psi

Hook at eisson

wall-roof junction

is considered ineffective

- too close to inside

surface - for resistance

to uplift. $\therefore$ roof will not

hold.

Client

WO/Job No.

Subject

Date 5/8/96 By EDCW

Location

Checked 5/96 By

Revised

By

200 Series Tank (55,000 gal)

2.1 Internal vacuum - 8 psig

(Ramble) pp E10-11 analyzes top structure as tee beam and shows a large margin against soil loading.

Weakest section is flat plate section ABC which has to be checked for bending & dowel support.

Bending: Use 8' x large plate for ABC:

$$M = \frac{wL^2}{8}$$

$$p_{E-10} \text{ (Ramble)} \quad p_{soil} = 1265 \text{ psf}$$

$$p_{conc} = 150 \text{ psf}$$

$$p_{stc} \quad 1415 \text{ psf} = 9.7 \text{ psi}$$

$$p_{tot} = p_{vac} + p_{stc} = 8 + 9.7 = 17.7 \text{ psi}$$

$$M = \frac{17.7 \times (8 \times 12)^2}{8} = 20 \text{ K}$$

(Ramble) pE-11 shows #8 @ 12" EFEW. $h = 12"$ $cl = 1\frac{1}{2}"$ $d = 10.5"$

$$p = .78 / 12 / 10.5 = .006 \quad j = .95 \quad f_y = 40 \text{ ksi}$$

$$M_u = j d A_s f_y = 0.95 \times 10.5" \times 0.79 / 12 \times 40 \text{ ksi} = 26 \text{ K} > 20 \text{ K OK}$$

Dowels: (Ramble) pE-10,11 says there are 18 #8 bars supporting each segment such as ABC. Dowel capacity:

$$P_n = 18 \times A_s f_y = 18 \times .31 \text{ in}^2 \times 40 \text{ ksi} = 220 \text{ K}$$

141

Client	WO/Job No.
Subject	Date 5/8/96 By FGLS
	Checked 5/9/96 By RCT/In
Location	Revised By

200 Series Tanks (con'd)

$$\text{Segment area: } A = \frac{2}{3} \times 11 \times 7.5 = 55 \text{ ft}^2$$

$$P = \frac{5}{8} W = \frac{5}{8} P_{\text{tot}} A = \frac{5}{8} 17.7 \text{ psi} \times 55 \text{ ft}^2 \times 144 \frac{\text{in}^2}{\text{ft}^2} = 88^{\text{K}} < 220^{\text{K}}$$

OK

check side wall:

$$P_{\text{soil}} = K_o \rho H \quad K_o = 0.4 \quad \rho = 125 \text{ pcf} \quad H = 12' + \frac{25'}{2} = 25'$$

$$R = 10' \quad t = 12" \text{ (Ramble pp E10-11)}$$

$$P_{\text{soil}} = 0.4 \times 125 \frac{\text{pcf}}{\text{ft}^2} \times 25' / 144 \frac{\text{in}^2}{\text{ft}^2} = 8.7 \text{ psi}$$

$$P_{\text{tot}} = P_{\text{vac}} + P_{\text{soil}} = 8 + 8.7 = 16.7 \text{ psi}$$

$$\sigma = P R / t = 16.7 \text{ psi} \times 10 / 1 = 167 \text{ psi} < 0.8$$

$$f_c' = 3000 \quad \text{Use } \sigma_n = 0.85 f_c' = 0.85 \times 3000 = 2550 \text{ psi}$$

>> σ_n OK

$$\text{ERSC buckling: } t/R = 1/10 = 0.1 \quad L/R = 25/10 = 2.5$$

$$\beta = 0.0125$$

$$P_{\text{cr}} = E t / r \beta = 3e6 \text{ psi} \times 0.1 \times 0.0125 = 2750 \text{ psi}$$

$$P_y = P_{\text{tot}} \frac{\sigma_n}{\sigma} = 16.7 \frac{2600}{167} = 260 \text{ psi} < P_{\text{cr}}$$

 P_y governs, OK

check bottom: Rebar not determined. Bottom is 7" thick, could easily be overloaded by ~8 psf at bottom (no contents)

Summary: Top & sides hold, but bottom strength not known & could fail, but not expected to result in an opening loc.

Client	WO/Job No.
Subject	Date 5/8/96 By EOW
	Checked 5/96 By KOT
Location	Revised By

2.4 DST/SST Internal vacuum - 8 psi

Per 2.5 gasoline burn in DST annulus, primary tank sidewall, torus & dome collapse is expected at 1-2 psi. Only studs can hold dome up.

Check studs: $\frac{1}{2}$ " A307 @ 24" EW $f_u = 60 \text{ ksi}$ J-bolt

Use Criteria, Tables 4.1.4-1, 2

$$F_y = \frac{1}{2} A_s f_u \quad F_u = \frac{2}{3} A_s f_u$$

$$\begin{aligned} \text{Abnormal } F_a &= \min \{ 0.9 F_y, 0.5 F_u \} \\ &= \frac{1}{3} A_s f_u = \frac{1}{3} \times .785 \times 52 \times 60 \text{ ksi} \\ &= 3.9 \text{ K} \end{aligned}$$

$$P_a = F_a / A = 3.9 \text{ K} / 24^2 = 6.8 \text{ psi} < 8 \text{ psi N.G.}$$

studs are expected to give out.

Check v/c dome: current

analysis effort (Peterson) 1:

shows 40% of shear capacity used near haunch with 1.4 load factor @ 6' soil at crown +

concrete ^(15") load. Estimate equivalent pressure with 6' 10/3 soil:

$$\begin{aligned} P_{\text{nom}} &= \left(6 + \frac{10}{3} \right) \times 125 \text{ pcf} / 144 \text{ in}^2/\text{ft}^2 + 150 \times 1.3 / 144 \\ &= 9.4 \text{ psi} \end{aligned}$$

Client	WO/Job No.		
Subject	Date 5/8/96	By EOG	
	Checked 5/96	By [Signature]	
Location	Revised	By	

2.4 DST/SST Internal vacuum (cont'd)

Allow p_v per

$$P_{nom} + p_v = \frac{1.4 P_{nom}}{0.40}$$

$$p_v = P_{nom} \left(\frac{1.4}{.4} - 1 \right) = 9.4 \text{ psi} \left(\frac{1.4}{.4} - 1 \right) = 23.5 \text{ psi}$$

> 8 psi. OK

R/C dome will not collapse, but primary tank will
with possible tearing.

Client	WO/Job No.
Subject	Date <u>5/8/96</u> By <u>EGW</u>
	Checked <u>5/96</u> By <u> </u>
Location	Revised By

2.5 DST gasoline burn in annulus (140 psi)

Check primary tank cyl. buckling (ERSC)

t varies from 3/8" to 3/4"

$$L \approx 32' \quad r = 37.5' \quad L/r = 32/37.5 = .85$$

$$\text{try } 3/8" t : t/r = .375/12/37.5 = .00083 \\ \beta \approx 3e-5$$

$$P_{cr} = E t / r \beta = 2866 \times .00083 \times 3e-5 = 0.7 \text{ psi}$$

$$P_u = 1/2 P_{cr} = 0.4 \text{ psi}$$

$$\text{for } t = 3/4" : t/r = .00163 \quad \beta = 7e-5$$

$$P_{cr} = 2866 \times .00166 \times 7e-5 = 3.2 \text{ psi}$$

$$P_u = 1/2 P_{cr} = 1.6 \text{ psi}$$

Cyl collapse occurs easily on primary tank

Check primary tank dome buckling (ERSC)

$$t = 3/8" \quad R = 100'$$

$$P_u = 0.18 \times 1.21 E (t/R)^2 = 0.18 \times 1.21 \times 2866 (.375/1200)^2 \\ = 0.6 \text{ psi} \quad (\text{dome collapses w/o anchor supp't})$$

Check torus : $t = 3/8" \quad r = 4' \quad L = \text{long} \quad \text{use cyl } L/r = 40$

$$t/r = 3/8/12/4 = .0078 \quad \beta = 1.9e-5$$

$$P_{cr} = E t / r \beta = 2866 \times .0078 \times 1.9e-5 = 4 \text{ psi}$$

$$P_u = 1/2 P_{cr} = 2 \text{ psi} \quad (\text{collapse expected})$$

Secondary liner is not connected to primary tank.

Collapse conditions above show gases pass in

between primary tank dome and concrete

Client	WO/Job No.
Subject	Date 5/1/96 By EOW
	Checked G/G B By [Signature]
Location	Revised By

2.5 DST gasoline burn (con'd)

i. treat as SST with $\sim 1/2$ of 140 psi loading.

Note

$$\text{Plenum: } V \approx \frac{1}{2} \pi 37.5^2 (15-6) = 20000 \text{ ft}^3$$

$$\text{Annulus } V \approx 2\pi \times 38.8 \times 2.5 \times 33 = 20000 \text{ ft}^3$$

i.e., gases could expand 2:1 into primary tank plenum (reason for $1/2$ of 140 psi loading). Recent Delphi effort had doubts about SST dome integrity at 11 psi, so conclude dome opens up. Thinnest soil cover at center suggests opening not full, estimate $\sim 20'-30'$

Client	WO/Job No.		
Subject	Int Vac / Seismic / Gasoline	Date	5/96 By EOW
		Checked	6/96 By KCTIL
Location		Revised	By

References

WHC-SD-WM-067 12x A

ERSC, 1988, "Buckling of Steel Shells, European Recommendations," ECCS Technical Committee 8, 1988, No 56.

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ASCE 4-86, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary...", ASCE Standard 4-86, Sep 1986.

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Appendix 7

High Temperature Waste Sludge

DESIGN ANALYSIS

Client	WESTINGHOUSE HANFORD COMPANY	WO/Job No.	E21425 / F6N1A1
Subject	Tank Farm Accident Load Evaluation	Date	May 1996
	High Temperature Waste Sludge	By	K.C. Tu
		Checked	5/96
		By	EO Werner
Location	200 East Area	Revised	

OBJECTIVE

Assess the potential for high temperature waste sludge to cause a loss of confinement. Assume the sludge temperature is 600°F and the sludge is stored in the tank for one year. Apply the sludge temperature as the inside surface temperature for the bottom half of the tank wall and apply an assumed inside surface temperature of 250°F for the upper half or the tank wall and dome. For Double-Contained Receiver Tanks (DCRTs), where the steel tank is not in direct contact with the vault of caisson structure, assume a temperature of 250°F for entire inside surface of the vault structure, including the cover. Assume a through wall temperature gradient for concrete foundations, walls, and covers that is half of the difference between the inside surface temperature and an assumed ambient temperature of 50°F, unless a more appropriate temperature gradient can be established and justified.

FINDINGS:

Per Reference ASME, the yield strength of a typical carbon steel (SA-516) reduces from 32ksi at normal temperature to 23.6ksi at 600°F, it is about 26% reduction. In general, the steel tanks may survive at 600°F without combining with any other load such as seismic load.

The degradation of Hanford concrete properties at elevated temperature for one year are shown in the table on page 3. After one year at 600°F, the modulus of elasticity of concrete reduce to 14% of original value, and the splitting tensile strength is 57% of the original value. The thermal expansion coefficient for Hanford concrete is 4×10^{-6} , and the coefficient for steel is 6.5×10^{-6} . The concrete vault likely cracks through the total thickness due to the difference of thermal expansion between the steel rebar (and steel liner if it exist at that high temperature) and concrete. The rebar and/or steel liner are the only supporting mechanism after the concrete cracks completely. In this high temperature case, the compressive force in the reinforced concrete increases the bending capacity of the concrete member. The soil pressure and massive concrete weight increase the safety factor of the concrete vault. A106 tank has been recorded at a temperature closed to 600°F, and it has not been collapsed.

Now the question is whether the dome or top cover of the confinement will be broken, and open up to the atmosphere. It is assumed that the temperature at inside surface of concrete dome is about 250°F and the outside surface is about 150°F. There is no significant degradation on concrete properties if the concrete temperature keeps below 200°F. The compressive strength of concrete only reduces 16% at 250°F for one

DESIGN ANALYSIS

Client	WESTINGHOUSE HANFORD COMPANY	WO/Job No.	E21425 / F6N1A1
Subject	Tank Farm Accident Load Evaluation	Date	May 1996
	High Temperature Waste Sludge	By	K.C. Tu
		Checked	5/96
		By	ECW
Location	200 East Area	Revised	

year. The cover blocks are laid on the top of the pit on top of some vaults. The edges of the blocks are free to rotate. At 100°F gradient through the thickness, the blocks free to bend and no moment is introduced onto the block. The concrete blocks may survive this high temperature sludge phenomena with no breach of the confinement.

The thermal moment due to through-thickness gradient will impose to existing moment at fix-end moment domes or cover slabs of the vaults. The 150°F to 250°F temperature does not effect the concrete properties significantly. But the very low Hanford concrete thermal expansion coefficient causes the concrete cracks through the dome at 130°F temperature rise from 70°F normal temperature. Because of that, the concrete cracks significantly reduce the bending capacities of the concrete. The dome or top slab is able to support its weight and the soil on top of it at high temperature has been studied. The shear capacity of the reinforced concrete is close to zero. There is no significant stress on reinforced steel, the steel should be able to hold the concrete in place without collapsing.

Some of the vaults have deep beams or mass concrete through the top slab such as 200-series tanks and 241-A-417 tank. A corner of the pump pit of 241-S-304 catch tank is located almost at the center of the cover slab. The roof slabs have been studied do not collapse due to the dead load. See Table 7.2 for failure summary due to high temperature waste sludge.

DESIGN ANALYSIS

Client	WESTINGHOUSE HANFORD COMPANY	WO/Job No.	E21425 / F6N1A1
Subject	Tank Farm Accident Load Evaluation	Date	May 1996
	High Temperature Waste Sludge	By	K.C. Tu
		Checked	5/96
		By	EOG
Location	200 East Area	Revised	

REFERENCES/SOURCES

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- PNL 1988, *Modeling of Time-Variant Concrete Properties at Elevated Temperatures*, by C. H. Henager, G. F. Piepel, W. E. Anderson, P. L. Koehmstedt, and F. A. Simonen, Pacific Northwest Laboratory, Richland, Washington, April 1988.
- RHO 1983, *Single-Shell Waste Tank Load Sensitivity Study*, Prepared by A. L. Ramble, Document No. SD-RE-TI-002, Rockwell Hanford Operations, Richland, Washington, March 1983.
- WHC 1994, *Concrete Material Characterization Reinforced Concrete Tank Structure Multi-Function Waste Tank Facility*, By B. V. Winkel, Document No. WHC-SD-W236A-ER-013, Westinghouse Hanford Co., Richland, Washington, October 1994.

Table 7.1 material properties at high temperatures

DEGRADATION OF MATERIAL PROPERTIES AT HIGH TEMPERATURE							
material	designation		normal	250 F		600 F	
			(psi)	(psi)	degradation	(psi)	degradation
carbon steel	SA-516	fy	32000	28750	10%	23600	26%
concrete	f c=3000 psi	E	3122000	2012000	36%	439000	86%
		f c	3000	2509	16%	1592	47%
		f st	217	195	10%	123	43%

*material properties:

fy = yield strength,

E = modulus of elasticity,

f c = compressive strength,

f st = splitting tensile strength.

NOTE: The duration for concrete at high temperature is one year. Duration is not a factor for steel.

DESIGN ANALYSIS

Client	WESTINGHOUSE HANFORD COMPANY	WO/Job No.	E21425 / F6N1A1
Subject	Tank Farm Accident Load Evaluation	Date	May 1996
	High Temperature Waste Sludge	By	K.C. Tu
		Checked	5/96
		By	EDW
Location	200 East Area	Revised	

Table 7.2 High Temperature Waste Sludge Summary

Tank	Structural Item	by Item	Overall	Remark
Catch Tanks				
241-A-302-A	Steel tank	OK	OK	Buried steel tank
241-A-417	Tank roof	Shear NG	OK to Atm	conc. w/liner
241-AX-152	Cover blocks	OK	OK	concrete tank
241-AZ-151	Cover blocks	OK	OK	concrete tank
241-ER-311	Steel tank	NG	OK to Atm	Steel tank buried deep in soil
241-EW-151	Steel tank Cover blocks	OK OK	OK	Steel tank inside concrete vault
241-S-304	Steel tank R/C roof	OK Shear NG	OK	
241-TX-302-C	Steel tank	OK	OK	Buried steel tank
241-U-301-B	R/C tank	NG	OK to Atm	By observation
241-UX-302-A	Steel tank	NG	OK to Atm	Steel tank buried deep in soil
Double-Contained Receiver Tanks (DCRT)				
241-A-350	Steel tank Cover blocks	OK OK	OK	
244-A	Steel tank Cover blocks	OK OK	OK	
244-BX/TX/U	Steel tank Cover blocks & roof	OK OK	OK	
244-CR-003	Steel tank Cover blocks	OK OK	OK	
244-S	Steel tank Cover blocks	OK OK	OK	
200-Series Single-Shell Tanks				
241-B/C/T/U	R/C tank w/ liner	Shear NG	OK to A	

Notes: NG = no good (fails structural criteria)

OK = satisfies structural criteria

OK to Atm = no significant opening to atmosphere

Shear NG = no good for structural shear criteria but not collapse

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	High Temperature Waste Sludge	By	K.-C.Tu
Location	200 East	Checked	5/96
		Revised	By EOW

Concrete Properties at Elevated Temperatures

The equations for long-term concrete properties at elevated temperature are documented in reference PNL.

The properties are plotted versus time as follows to compare with the reference.

nominal initial compressive strength $S = 3$ ksi

constant value of temperature $T = 450$ F

set time

$$i = 0..9 \quad t_i = i^2 \quad \text{days} \quad \text{at } i = 9 \quad 9^{4.5} = 19683 \quad \text{days} \quad \frac{9^{4.5}}{365.25} = 54 \quad \text{years}$$

predicted modulus of elasticity (ksi)

$$E_i = (5.3947 + 0.1233 \cdot S - 0.006751 \cdot T - 0.1786 \cdot \ln(t_i + 1)) \cdot 1000$$

predicted compressive strength (psi)

$$r_{c_i} = 4416.338 + 490.919 \cdot S - 4.714 \cdot \text{if}(T > 350, T - 350, 0) - (230.241 - 1.273 \cdot \text{if}(T > 350, 0, 350 - T)) \cdot \ln(t_i + 1)$$

predicted splitting tensile strength (psi)

$$r_{st_i} = 448.1758 + 23.7436 \cdot S - 0.6077 \cdot \text{if}(T > 350, T - 350, 0) - 18.4341 \cdot \ln(t_i + 1)$$

3000 psi Hanford concrete properties versus time at 450 F

Note: The above curves match the curves on the Figures 6, 9, and 12 of the reference.

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		By	EOL

The equations are derived for testing data in which the concrete strength values are higher than nominal values which are used in design. So that, the values should be proportional down to the nominal values.

find 28 days property values

$$\text{set } i = 0.50 \quad t_0 = 28 \quad \text{days} \quad i = 0$$

$$T = 70 \quad F$$

$$E = (5.3947 + 0.1233 \cdot s - 0.006751 \cdot T - 0.1786 \cdot \ln(t_i + 1)) \cdot 10^6 \text{ psi}$$

$$E = 4691 \cdot \text{ksi}$$

$$r_c = [4416.338 + 490.919 \cdot s - 4.714 \cdot \text{if}(T > 350, T - 350, 0) - (230.241 - 1.273 \cdot \text{if}(T > 350, 0, 350 - T)) \cdot \ln(t_i + 1)] \text{ psi}$$

$$r_c = 6314 \cdot \text{psi}$$

$$r_{st} = (448.1758 + 23.7436 \cdot s - 0.6077 \cdot \text{if}(T > 350, T - 350, 0) - 18.4341 \cdot \ln(t_i + 1)) \text{ psi}$$

$$r_{st} = 457 \cdot \text{psi}$$

for nominal compressive strength 3000 psi concrete

$$E_{3000} = 57 \cdot \sqrt{s \cdot 1000} \text{ ksi}$$

$$E_{3000} = 3122 \cdot \text{ksi}$$

$$r_{c3000} = 3000 \cdot \text{psi}$$

the ratio for the property equations

$$r_E = \frac{E_{3000}}{E}$$

$$r_E = 67 \cdot \%$$

$$r_f = \frac{r_{c3000}}{r_c}$$

$$r_f = 48 \cdot \%$$

The concrete properties of 3000 psi concrete after one year at 600 F:

$$T = 600 \quad F$$

$$i = 1 \quad t_i = 365 \quad \text{days}$$

$$E = (5.3947 + 0.1233 \cdot s - 0.006751 \cdot T - 0.1786 \cdot \ln(t_i + 1)) \cdot 10^6 \text{ psi}$$

$$E = 660 \cdot \text{ksi}$$

$$E_{600} = E \cdot r_E$$

$$E_{600} = 439 \cdot \text{ksi}$$

$$r_c = [4416.338 + 490.919 \cdot s - 4.714 \cdot \text{if}(T > 350, T - 350, 0) - (230.241 - 1.273 \cdot \text{if}(T > 350, 0, 350 - T)) \cdot \ln(t_i + 1)] \text{ psi}$$

$$r_c = 3352 \cdot \text{psi}$$

$$r_{c600} = r_c \cdot r_f$$

$$r_{c600} = 1592 \cdot \text{psi}$$

$$r_{st} = (448.1758 + 23.7436 \cdot s - 0.6077 \cdot \text{if}(T > 350, T - 350, 0) - 18.4341 \cdot \ln(t_i + 1)) \text{ psi}$$

$$r_{st} = 259 \cdot \text{psi}$$

$$r_{st600} = r_{st} \cdot r_f$$

$$r_{st600} = 123 \cdot \text{psi}$$

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		Revised	By EOW

TANK 241-A-417 DOME

The concrete properties of 3000 psi concrete after one year at 250 F:

$$\begin{aligned}
 T &:= 250 \quad F \\
 t_1 &:= 1 \quad t_1 = 365 \quad \text{days} \\
 E &:= (5.3947 + 0.1233 \cdot s - 0.006751 \cdot T - 0.1786 \cdot \ln(t_1 + 1)) \cdot 10^6 \text{ psi} \\
 E &= 3023 \cdot \text{ksi} \\
 E_{250} &:= E \cdot r_E \quad E_{250} = 2012 \cdot \text{ksi} \\
 r_c &:= [4416.338 + 490.919 \cdot s - 4.714 \cdot \text{if}(T > 350, T - 350, 0) - (230.241 - 1.273 \cdot \text{if}(T > 350, 0, 350 - T)) \cdot \ln(t_1 + 1)] \cdot \text{psi} \\
 r_c &= 5281 \cdot \text{psi} \\
 r_{c_{250}} &:= r_c \cdot r_f \quad r_{c_{250}} = 2509 \cdot \text{psi} \\
 r_{st} &:= (448.1758 + 23.7436 \cdot s - 0.6077 \cdot \text{if}(T > 350, T - 350, 0) - 18.4341 \cdot \ln(t_1 + 1)) \cdot \text{psi} \\
 r_{st} &= 411 \cdot \text{psi} \\
 r_{st_{250}} &:= r_{st} \cdot r_f \quad r_{st_{250}} = 195 \cdot \text{psi}
 \end{aligned}$$

The dome is 10" thick concrete slab with #6@6" top & bottom and #4@12" top and bottom the other way.
Top rebar 2' clear and bottom 3/4" clear (see Dwg. H-2-56800)

Set: The temperature at bottom surface is 250 F, and 150 F at top.

thermal expansion coefficient
steel

$$\alpha_s = 6.5 \cdot 10^{-6}$$

Hanford concrete

$$\alpha_c = 4.0 \cdot 10^{-6}$$

See Ref. WHC 1994

modulus of elasticity of steel

$$E_s = 29 \cdot 10^6 \text{ psi}$$

slab depth

$$h = 10 \cdot \text{in}$$

rebar area per inch width

$$A_1 = \frac{.44}{6} \cdot \text{in}^2$$

$$A_1 = 0.0733 \cdot \text{in}^2$$

$$A_2 = \frac{0.20}{12} \cdot \text{in}^2$$

$$A_2 = 0.0167 \cdot \text{in}^2$$

distance

$$d_1 = \left(.75 + \frac{6}{16} \right) \cdot \text{in}$$

$$d_1 = 1.125 \cdot \text{in}$$

$$d_2 = \left(.75 + \frac{6}{8} + \frac{4}{16} \right) \cdot \text{in}$$

$$d_2 = 1.75 \cdot \text{in}$$

$$d_3 = \left(10 - 2 - \frac{6}{8} - \frac{4}{16} \right) \cdot \text{in}$$

$$d_3 = 7 \cdot \text{in}$$

$$d_4 = \left(10 - 2 - \frac{6}{16} \right) \cdot \text{in}$$

$$d_4 = 7.625 \cdot \text{in}$$

See next page for program "CRACKPC" output. (Note: The program "CRACKPC" is not been an ICH Kaiser vrfified program, but the final results can be checked by hand as follows.)

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		Revised	By

TANK 241-A-417 DOME (cont'd)

$$\text{bottom rebar temperature } T_1 = 250 - 100 \cdot \frac{d_1}{h} \quad T_1 = 239$$

$$\text{top top temperature } T_4 = 250 - 100 \cdot \frac{d_4}{h} \quad T_4 = 174$$

The difference of the strain between bottom steel and concrete is

$$\epsilon_d = T_1 (\alpha_s - \alpha_c) \quad \epsilon_d = 0.000597$$

The concrete cracks at strain

$$\epsilon_c = \frac{r_{st}}{E_{250}} \quad \epsilon_c = 0.000204 < \epsilon_d$$

The concrete slab cracks through the thickness at main reinforcement direction. Then, assume no concrete at all.

$$\sigma_c = \frac{E_s \alpha_s (T_1 - T_4)}{2} \quad \sigma_c = 6126 \cdot \text{psi}$$

O. K. checked with computer output

tensile force required to force the section to stretch the same amount as the thermal expansion

$$F_r = (\alpha_s - \alpha_c) \cdot [(T_1 - 70) \cdot A_1 + (T_4 - 70) \cdot A_s] \cdot E_s \quad F_r = 1.449 \cdot \text{kip}$$

deman moment due to dead load (soil & slab weight)

$$Dm = \frac{1}{8} (125 \cdot \text{pcf} \cdot (685 - 671) \cdot \text{ft} + 150 \cdot \text{pcf} \cdot 1.4 \cdot \text{ft}) \cdot \left[\left(\frac{26.33}{2} - 5.5 \right) \cdot \text{ft} \right]^2$$

$$Dm = 1.2 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{in}}$$

check concrete slab

$$\text{Compressive strength of concrete } f_c = f_{c250} \quad f_c = 2509 \cdot \text{psi}$$

$$\text{Yield stress for rebar at 250 F } f_{y250} = 40 \cdot \text{ksi} \cdot \frac{29.2 + 28.3}{32.02} \quad f_{y250} = 36 \cdot \text{ksi}$$

See ASME Section II, Code Case, Specification no. SA-516 Gr.60 from table 3 on p. 217.

$$\text{Yield stress of steel } f_y = f_{y250} \quad f_y = 36 \cdot \text{ksi}$$

$$\text{Modulus of elasticity of steel } E_s = 29000 \cdot \text{ksi}$$

$$\text{dimensions: Effective width } b = 1 \cdot \text{in}$$

$$\text{Thickness } h = 10 \cdot \text{in}$$

$$\text{Total area } A_g = b \cdot h \quad A_g = 10 \cdot \text{in}^2$$

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TANK 241-A-417 DOME (cont'd)**Rebar informations:**

$$\text{Tensile side rebar area} \quad A_s := A_1 \quad A_s = 0.0733 \cdot \text{in}^2$$

$$\text{Distance from extreme compression fiber to tension rebar} \quad d := h - d_1 \quad d = 9 \cdot \text{in}$$

$$\text{Compressive side rebar area} \quad A'_s := A_1 \quad A'_s = 0.0733 \cdot \text{in}^2$$

$$\text{Distance from extreme compression fiber to compression rebar} \quad d' := h - d_4 \quad d' = 2.375 \cdot \text{in}$$

$$\text{Ratio of tension reinforcement} \quad \rho := \frac{A_s}{b \cdot d} \quad \rho = 0.8263 \cdot \%$$

ACI 318-89 (10.5.1) minimum reinforcement required

$$\rho_{\min} := \frac{200 \cdot \text{psi}}{f_y} \quad \rho_{\min} = 0.5565 \cdot \%$$

ACI 318-89 (10.5.2)

$$A_s := \text{if} \left(\rho > \rho_{\min}, A_s, \frac{A_s}{1.33} \right)$$

ACI 318-89 (12.2.7.3) Factor β_1

$$\beta_1 := \text{if} \left[f_c < 4000 \cdot \text{psi}, 0.85, \text{if} \left[f_c \leq 8000 \cdot \text{psi}, \left(0.85 - \frac{f_c - 4000 \cdot \text{psi}}{1000 \cdot \text{psi}} \cdot 0.05 \right), 0.65 \right] \right] \quad \beta_1 = 0.85$$

Set the maximum strain at the extreme compression fiber

$$\epsilon_c := .003$$

Assume the depth of equivalent rectangular stress block (neglect A's)

$$a_n := \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b} \quad a_n = 1.236 \cdot \text{in}$$

Guess value:

$$x := \frac{a_n}{\beta_1}$$

Given

$$.85 \cdot f_c \cdot b \cdot \beta_1 \cdot x + A'_s \cdot \left(E_s \cdot \epsilon_c \cdot \frac{x - d'}{x} - 0.85 \cdot f_c \right) - A_s \cdot f_y = 0 \cdot \text{lb}$$

$$c_n := \text{find}(x)$$

$$c_n = 2.066 \cdot \text{in} \quad < d' \text{ change the above formula} \quad d' = 2.375 \cdot \text{in}$$

Given

$$.85 \cdot f_c \cdot b \cdot \beta_1 \cdot x + A'_s \cdot \left(E_s \cdot \epsilon_c \cdot \frac{x - d'}{x} \right) - A_s \cdot f_y = 0 \cdot \text{lb}$$

$$c_n := \text{find}(x)$$

$$c_n = 2 \cdot \text{in}$$

$$a_n := \beta_1 \cdot c_n$$

$$a_n = 1.7316 \cdot \text{in}$$

$$f_s := E_s \cdot \epsilon_c \cdot \frac{c_n - d'}{c_n}$$

$$f_s = -14428 \cdot \text{psi}$$

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		Revised	
		By	

TANK 241-A-417 DOME (cont'd)

For moment equilibrium: (Po=0)

(-) sign is in tension

$$M_n := A_s \cdot f_y \cdot \left(d - \frac{h}{2} \right) - A'_s \cdot f_s \cdot \left(\frac{h}{2} - d' \right) + .85 \cdot f'_c \cdot b \cdot a_n \cdot \left(\frac{h}{2} - \frac{a_n}{2} \right)$$

$$M_n = 28259 \cdot \text{in} \cdot \text{lb}$$

$$M_n = 2.355 \cdot \text{ft} \cdot \text{kip}$$

$$\phi := 0.90 \quad \text{for tension region}$$

$$\phi M_n := \phi \cdot M_n$$

$$\phi M_n = 2.119 \cdot \text{ft} \cdot \text{kip}$$

maximum allowable tensile force

$$T_f := 0.9 \cdot f_y \cdot (A_s + A'_s)$$

$$T_f = 4.744 \cdot \text{kip}$$

allowable mement for tensile force

$$F_r = 1.449 \cdot \text{kip}$$

$$M_a := \phi M_n \cdot \left(\frac{T_f - F_r}{T_f} \right)$$

$$M_a = 1.472 \cdot \text{kip} \cdot \text{ft} >$$

$$D_m = 1.2 \cdot \text{kip} \cdot \frac{\text{ft}}{\text{in}}$$

O. K.

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Subject	Tank Farm Accident Load Evaluation	Date	May 1996
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Location	200 East	Checked	S/96
		Revised	By E.O.W.

200 SERIES TANK DOME

concrete strength = 2500 psi at 28 days (see Dwg. HW-72417)

The dome is one foot thick concrete slab with 1" dia. steel each way at top and bottom. (See Ref. RHO, p. E-11)

The concrete properties of 2500 psi concrete after one year at 250 F:

$$S = 2.500 \text{ ksi} \quad T = 250$$

$$E = (5.3947 + 0.1233 \cdot S - 0.006751 \cdot T - 0.1786 \cdot \ln(t_i + 1)) \cdot 10^6 \text{ psi}$$

$$E = 3 \cdot 10^6 \text{ psi}$$

$$E_{c2500} = E \cdot r_E$$

$$E_{c2500} = 1.971 \cdot 10^6 \text{ psi}$$

$$r_c = [4416.338 + 490.919 \cdot S - 4.714 \cdot \text{if}(T > 350, T - 350, 0) - (230.241 - 1.273 \cdot \text{if}(T > 350, 0, 350 - T)) \cdot \ln(t_i + 1)] \text{ psi}$$

$$r_c = 5036 \text{ psi}$$

$$r_{c2500} = r_c \cdot r_f$$

$$r_{c2500} = 2393 \text{ psi}$$

$$r_{st} = (448.1758 + 23.7436 \cdot S - 0.6077 \cdot \text{if}(T > 350, T - 350, 0) - 18.4341 \cdot \ln(t_i + 1)) \text{ psi}$$

$$r_{st} = 399 \cdot \text{mass} \cdot \text{length}^{-1} \cdot \text{time}^{-2}$$

$$r_{st2500} = r_{st} \cdot r_f$$

$$r_{st2500} = 189 \text{ psi}$$

rebar area

$$A_r = \frac{0.79}{12} \text{ in}^2$$

$$A_r = 0.0658 \text{ in}^2$$

slab thickness

$$h = 12 \text{ in}$$

distance

$$d_1 = (.75 + \frac{8}{16}) \cdot \text{in}$$

$$d_1 = 1.25 \text{ in}$$

$$d_2 = (.75 + \frac{8}{8} + \frac{8}{16}) \cdot \text{in}$$

$$d_2 = 2.25 \text{ in}$$

$$d_3 = (12 - .75 - \frac{8}{8} - \frac{8}{16}) \cdot \text{in}$$

$$d_3 = 9.75 \text{ in}$$

$$d_4 = (12 - .75 - \frac{8}{16}) \cdot \text{in}$$

$$d_4 = 10.75 \text{ in}$$

See next page for program "CRACKPC" output.

The concrete slab cracks through the thickness in X-direction. assume no concrete at all.

$$T_1 = 250 - 100 \cdot \frac{d_1}{h}$$

$$T_3 = 250 - 100 \cdot \frac{d_3}{h}$$

$$T_1 = 240$$

$$T_3 = 169$$

$$\sigma_c = \frac{E_s \cdot a_s \cdot (T_1 - T_3)}{2}$$

$$\sigma_c = 6676 \text{ psi}$$

O. K. checked with computer output

tensile force required to force the section to stretch the same amount as the thermal expansion

$$F_r = (a_s - a_c) \cdot [(T_1 - 70) \cdot A_r + (T_3 - 70) \cdot A_r] \cdot E_s$$

$$F_r = 1.281 \text{ kip}$$

deman moment due to dead load (soil & slab weight)

$$Dm = \frac{1}{8} \cdot (125 \text{ pcf} \cdot 11 \cdot \text{ft} + 150 \text{ pcf} \cdot 1 \cdot \text{ft}) \cdot ((10 - 2 - 1) \cdot \text{ft})^2$$

$$Dm = 0.778 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{in}}$$

assume 4' width duct support & it will not fail

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	High Temperature Waste Sludge	By	K.-C.Tu
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		Revised	
		By	EDW

200 SERIES TANK DOME (cont'd)**check concrete slab**

Compressive strength of concrete $f_c := f_c'_{2500}$ $f_c = 2393 \cdot \text{psi}$

Yield stress for rebar at 250 F $f_y := 40 \cdot \text{ksi} \cdot \frac{29.2 + 28.3}{32.0 \cdot 2}$ $f_y = 36 \cdot \text{ksi}$

Modulus of elasticity of steel $E_s := 29000 \cdot \text{ksi}$

dimensions: Effective width $b := 1 \cdot \text{in}$

Thickness $h := 12 \cdot \text{in}$

Total area $A_g := b \cdot h$ $A_g = 12 \cdot \text{in}^2$

Rebar informations:

Tensile side rebar area $A_s := A_r$ $A_s = 0.0658 \cdot \text{in}^2$

Distance from extreme compression fiber to tension rebar $d := h - d_1$ $d = 11 \cdot \text{in}$

Compressive side rebar area $A_s' := A_s$ $A_s' = 0.0658 \cdot \text{in}^2$

Distance from extreme compression fiber to compression rebar $d' := h - d_3$ $d' = 2.25 \cdot \text{in}$

Ratio of tension reinforcement $\rho := \frac{A_s}{b \cdot d}$ $\rho = 0.6124 \cdot \%$

ACI 318-89 (10.5.1) minimum reinforcement required $\rho_{\min} := \frac{200 \cdot \text{psi}}{f_y}$ $\rho_{\min} = 0.5565 \cdot \%$

ACI 318-89 (10.5.2) $A_s := \text{if} \left(\rho > \rho_{\min}, A_s, \frac{A_s}{1.33} \right)$

ACI 318-89 (12.2.7.3) Factor β_1

$$\beta_1 := \text{if} \left[f_c < 4000 \cdot \text{psi}, 0.85, \text{if} \left[f_c \leq 8000 \cdot \text{psi}, \left(0.85 - \frac{f_c - 4000 \cdot \text{psi}}{1000 \cdot \text{psi}} \cdot 0.05 \right), 0.65 \right] \right] \quad \beta_1 = 0.85$$

Set the maximum strain at the extreme compression fiber

$$\epsilon_c := .003$$

Assume the depth of equivalent rectangular stress block (neglect A_s')

$$a_n := \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b} \quad a_n = 1.163 \cdot \text{in}$$

Guess value: $x := \frac{a_n}{\beta_1}$

Given $.85 \cdot f_c \cdot b \cdot \beta_1 \cdot x + A_s' \cdot \left(E_s \cdot \epsilon_c \cdot \frac{x - d'}{x} - 0.85 \cdot f_c \right) - A_s \cdot f_y = 0 \cdot \text{lb}$

$$c_n := \text{find}(x)$$

$$c_n = 1.952 \cdot \text{in} \quad < d' \text{ change the above formula} \quad d' = 2.25 \cdot \text{in}$$

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		Revised	By EOW

200 TANK DOME (cont'd)

Given $.85 \cdot f_c \cdot b \cdot \beta_1 \cdot x + A's \cdot \left(E_s \cdot \epsilon_c \cdot \frac{x - d'}{x} \right) - A_s \cdot f_y = 0 \cdot lb$

$$c_n := \text{find}(x)$$

$$c_n = 2 \cdot in$$

$$a_n := \beta_1 \cdot c_n$$

$$a_n = 1.6371 \cdot in$$

$$f_s := E_s \cdot \epsilon_c \cdot \frac{c_n - d'}{c_n}$$

$$f_s = -14638 \cdot psi$$

For moment equilibrium: (Po=0)

$$M_n := A_s \cdot f_y \cdot \left(d - \frac{h}{2} \right) - A's \cdot f_s \cdot \left(\frac{h}{2} - d' \right) + .85 \cdot f_c \cdot b \cdot a_n \cdot \left(\frac{h}{2} - \frac{a_n}{2} \right)$$

$$M_n = 32104 \cdot in \cdot lb$$

$$M_n = 2.675 \cdot ft \cdot kip$$

$$\phi = 0.9 \quad \text{for tension region}$$

$$\phi M_n = \phi M_n$$

$$\phi M_n = 2.408 \cdot ft \cdot kip$$

maximum allowable tensile force

$$T_f := 0.9 \cdot f_y \cdot (A_s + A's)$$

$$T_f = 4.259 \cdot kip$$

allowable mement for tensile force

$$F_r = 1.281 \cdot kip$$

$$M_a := \phi M_n \cdot \left(\frac{T_f - F_r}{T_f} \right)$$

$$M_a = 1.684 \cdot kip \cdot ft >$$

$$D_m = 0.778 \cdot kip \cdot \frac{ft}{in}$$

O. K.

DESIGN ANALYSIS

Client	Westinghouse Hanford Company	WO/Job No.	E21425 / F6N1A1
Subject	Tank Farm Accident Load Evaluation	Date	May 1996
	High Temperature Waste Sludge	Checked	5/96
Location	200 East	By	EDW
		Revised	
		By	

241-S-304 CATCH DOME TANK

Catch tank is located inside of the caisson. Assume concrete dome temperature is 250 F inside and 100 F outside.

$$\begin{aligned} \text{rebar area} \quad A_1 &:= 0.44 \cdot \frac{12}{9} \cdot \text{in}^2 & A_1 &= 0.587 \cdot \text{in}^2 \\ A_2 &:= 0.31 \cdot \text{in}^2 \end{aligned}$$

The maximum allowable tensile capacity

$$F_t := f_y \cdot (A_1 + A_2) \quad F_t = 32.22 \cdot \text{kips}$$

rebar temperature

$$\begin{aligned} T_1 &:= 250 - 100 \cdot \frac{3}{15} & T_1 &= 230 \\ T_4 &:= 250 - 100 \cdot \frac{12}{15} & T_4 &= 170 \end{aligned}$$

tensile force required to force the section to stretch the same amount as the thermal expansion

$$\begin{aligned} F_r &:= (a_s - a_c) \cdot [(T_1 - 70) \cdot A_1 + (T_4 - 70) \cdot A_2] \cdot E_s & F_r &= 9.05 \cdot \text{kip} \\ E_s &= 29000 \cdot \text{ksi} \end{aligned}$$

deman moment from disc shape due to dead load (soil & slab weight)

$$\begin{aligned} D_m &:= \frac{1}{5} \cdot (125 \cdot \text{pcf} \cdot 5.42 \cdot \text{ft} + 150 \cdot \text{pcf} \cdot 1.4 \cdot \text{ft}) \cdot (8 \cdot \text{ft})^2 & D_m &= 11.36 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}} \end{aligned}$$

check concrete slab

Compressive strength of concrete $f_c := f'_{c250}$

$$f_c = 2509 \cdot \text{psi}$$

dimensions:

$$\text{Effective width} \quad b := 12 \cdot \text{in}$$

$$\text{Thickness} \quad h := 12 \cdot \text{in}$$

$$\text{Total area} \quad A_g := b \cdot h$$

$$A_g = 144 \cdot \text{in}^2$$

Rebar informations:

Tensile side rebar area

$$A_s := A_1$$

$$A_s = 0.5867 \cdot \text{in}^2$$

Distance from extreme compression fiber to tension rebar

$$d := \left(15 - \frac{3}{4} - \frac{6}{8} - \frac{6}{16} \right) \cdot \text{in}$$

$$d = 13 \cdot \text{in}$$

Compressive side rebar area

$$A'_s := A_2$$

$$A'_s = 0.31 \cdot \text{in}^2$$

Distance from extreme compression fiber to compression rebar

$$d' := \left(2 + \frac{5}{8} + \frac{5}{16} \right) \cdot \text{in}$$

$$d' = 2.9375 \cdot \text{in}$$

Ratio of tension reinforcement

$$\rho := \frac{A_s}{b \cdot d}$$

$$\rho = 0.3725 \cdot \%$$

reber yield stress

$$f_y := 60 \cdot \text{ksi} \cdot \frac{29.2 + 28.3}{32.0 \cdot 2}$$

$$f_y = 54 \cdot \text{ksi}$$

ACI 318-89 (10.5.1) minimum reinforcement required

$$\rho_{\min} := \frac{200 \cdot \text{psi}}{f_y}$$

$$\rho_{\min} = 0.371 \cdot \%$$

ACI 318-89 (10.5.2)

$$A_s := \text{if} \left(\rho > \rho_{\min}, A_s, \frac{A_s}{1.33} \right)$$

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DESIGN ANALYSIS

Client Westinghouse Hanford Company
 Subject Tank Farm Accident Load Evaluation
High Temperature Waste Sludge
 Location 200 East

WO/Job No. E21425 / F6N1A1
 Date May 1996 By K.-C.Tu
 Checked EDW By S/96
 Revised _____ By _____

241-S-304 CATCH TANK DOME (cont'd)ACI 318-89 (12.2.7.3) Factor β_1

$$\beta_1 := \text{if} \left[f_c < 4000 \text{ psi}, 0.85, \text{if} \left[f_c \leq 8000 \text{ psi}, \left(0.85 - \frac{f_c - 4000 \text{ psi}}{1000 \text{ psi}} \cdot 0.05 \right), 0.65 \right] \right] \quad \beta_1 = 0.85$$

Set the maximum strain at the extreme compression fiber

$$\epsilon_c := .003$$

Assume the depth of equivalent rectangular stress block (neglect A's)

$$a_n := \frac{A_s f_y}{0.85 f_c b} \quad a_n = 1.236 \text{ in}$$

Guess value: $x := \frac{a_n}{\beta_1}$

Given $.85 f_c b \beta_1 x + A_s' \left(\epsilon_s \epsilon_c \frac{x - d'}{x} - 0.85 f_c \right) - A_s f_y = 0 \text{ lb}$
 $c_n := \text{find}(x)$
 $c_n = 2.034 \text{ in} \quad < d' \text{ change the above formula} \quad d' = 2.938 \text{ in}$

Given $.85 f_c b \beta_1 x + A_s' \left(\epsilon_s \epsilon_c \frac{x - d'}{x} \right) - A_s f_y = 0 \text{ lb}$
 $c_n := \text{find}(x)$
 $c_n = 2 \text{ in} \quad a_n := \beta_1 c_n \quad a_n = 1.7155 \text{ in}$
 $f_s := \epsilon_s \epsilon_c \frac{c_n - d'}{c_n} \quad f_s = -39627 \text{ psi}$

For moment equilibrium: ($P_o = 0$)

$$M_n := A_s f_y \left(d - \frac{h}{2} \right) - A_s' f_s \left(\frac{h}{2} - d' \right) + .85 f_c b a_n \left(\frac{h}{2} - \frac{a_n}{2} \right)$$

$$M_n = 5 \cdot 10^5 \text{ in-lb} \quad M_n = 40.729 \text{ ft-kip}$$

$$\phi := 0.9 \quad \text{for tension}$$

$$\phi M_n := \phi M_n \quad \phi M_n = 36.656 \text{ ft-kip}$$

maximum allowable tensile force

$$T_f := 0.9 f_y (A_s + A_s') \quad T_f = 43.502 \text{ kip}$$

allowable moment for tensile force

$$F_r = 9.053 \text{ kip}$$

$$M_a := \phi M_n \left(\frac{T_f - F_r}{T_f} \right)$$

$$M_a = 29.028 \text{ kip-ft} > \quad D_m = 11.36 \text{ kip} \cdot \frac{\text{ft}}{\text{ft}} \quad \text{O. K.}$$

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Appendix 8

Aging and Degradation

DESIGN ANALYSIS

Client	WESTINGHOUSE HANFORD COMPANY	WO/Job No.	E21425 / F6N1A1
Subject	Tank Farm Accident Load Evaluation	Date	May 1996
	Aging and Degradation	By	K.C. Tu
		Checked	5/96
		By	coweiner
Location	200 East Area	Revised	

OBJECTIVE

Assess the potential for aging and degradation mechanism to cause a loss of containment. Specifically address how each of the following mechanisms could individually affect confinement of the waste:

- corrosion
- high temperature waste
- thermal cycling

CORROSION:

There are three time-dependent corrosion mechanisms (Reference Abatt) on the steel tanks.

Pitting Corrosion:

The highest pitting corrosion rates from corrosion testing and laboratory simulated wastes are as shown on following table:

HIGHEST PITTING CORROSION RATES		
PHASE	MILS PER YEAR	INCH IN 10 YEARS
vapor	37	0.37
vapor/liquid	35	0.35
liquid	6.4	0.064

The rates are extremely conservative. The corrosion rate is depend on the type of liquid and inside surface condition of the tank. The pitting corrosion breaches at local area, and may not cause the tank collapse.

Uniform Corrosion:

The uniform corrosion are depended on the type of liquid, pH value, temperature, and duration. In general, the uniform corrosion rates are as follows:

UNIFORM CORROSION RATES			
PHASE	RANGE	MEDIAN VALUE	inch per ten years
	mils per year	mils per year	
vapor	0.6 - 4.1	0.8	0.008
vapor/liquid	0.01 - 1.6	0.2	0.002
liquid	0.01 - 3.1	0.2	0.002

DESIGN ANALYSIS

Client	WESTINGHOUSE HANFORD COMPANY	WO/Job No.	E21425 / F6N1A1
Subject	Tank Farm Accident Load Evaluation	Date	May 1996
	Aging and Degradation	By	K.C. Tu
		Checked	5/96
		By	EO Werner
Location	200 East Area	Revised	

Uniform corrosion should have been considered in the normal tank design. At normal condition, the uniform corrosion rate is very low, and is not a problem for about 60 years of tanks life.

Stress-Corrosion Cracking:

Stress-Corrosion Cracking occurs when tensile stress approaching or exceeding yield strength and an environment corrosive to the material are present simultaneously. The objective tanks are normally empty. Stress-Corrosion Cracking is not a concern.

Reinforcement:

The soil condition in Hanford area is dry and the reinforcement in underground concrete is in good condition. Some core drilled samples show no corrosion on the reinforced steel. If the primary tank leaks, the liquid seeps into concrete and corrodes the rebar. The concrete vault is not a water tied confinement, especially at crack section.

HIGH TEMPERATURE WASTE:**Steel Tank:**

The maximum temperature has been experienced in Hanford tanks is below 600°F at the sludge near bottom of A106 tank. The yield strength of carbon steel may reduce 26% of its normal strength at 600°F, that means the factor of safety of the tank reduces up to 26% if the stress at the high temperature area is near to the yield stress.

High temperature accelerates the corrosion process, and decreases the life of the tank.

Concrete Vault:

Concrete property values (modulus of elasticity, compressive strength, and splitting tensile strength) increase moderately years after years at concrete temperature at normal condition. But the values degrades at elevated temperature with time. The concrete properties do not degrade at temperature below 200°F. Beyond the concrete properties degradation, the difference of thermal expansions between steel and concrete at elevated temperature. Especially, the thermal expansion coefficient of Hanford concrete (4×10^{-6} Reference Winkel) is less than value of the regular concrete (5.6×10^{-6}). (Note, the thermal expansion coefficient for steel is 6.5×10^{-6} .) At high temperature, the steel rebar and/or steel liner stretches and causes the concrete cracks, and the capacities of the concrete section decreases.

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Subject	Tank Farm Accident Load Evaluation	Date	May 1996
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		By	EO Weiner
Location	200 East Area	Revised	

There is more detail discussion in the High Temperature Waste Sludge section of this report.

THERMAL CYCLING:

Steel Tank:

The thermal cycling do not effect on the steel tanks for the waste tanks at the temperature below 600°F except as it has been mentioned on the High Temperature Section.

Because of the difference of thermal expansion between steel and concrete, the thermal cycling may create the friction force between the bottom or its supporting members and the concrete foundation. The friction force may buckle the tank bottom or/and cracks the concrete.

Concrete Vault:

As mentioned previously, the concrete modulus of elasticity degrades with time at elevated temperature. The loss in modulus is not recoverable at returning lower temperature. In addition to the degradation of modulus, the thermal creep increases the instability of the concrete structure. The concrete vault behaviors highly in inelastic mode at high temperature. The deformation of the structure increases at each thermal cycle. Per Reference Anatech, there is no significant affect due to cycling after four cycles. But the concrete properties (modulus of elasticity, compressive strength, and splitting tensile strength) degrades continuously with the duration when the temperature reaches above 200°F.

The affect of thermal cycling is direct proportional to the temperature it reaches. In general, the temperature of the objective vault at each cycle is below moderate high temperature (say 250°F), the thermal cycling do not effect on the concrete vault integrity.

DESIGN ANALYSIS

Client	WESTINGHOUSE HANFORD COMPANY	WO/Job No.	E21425 / F6N1A1
Subject	Tank Farm Accident Load Evaluation	Date	May 1996
	Aging and Degradation	By	K.C. Tu
		Checked	5/96
		By	EO Werner
Location	200 East Area	Revised	

REFERENCES/SOURCES

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Appendix 9

Gasoline Ignition

DCRTs and DSTs are evaluated structurally for a gasoline ignition event occurring in the vault or annulus and producing 140 psig pressure. The statement of work gives an instantaneous rise time and a sufficient duration which calls for a dynamic load factor (DLF) of two. Prior to the finalized statement of work, information was received with a rise time sufficiently long to suggest a lower DLF, and the attached calculations were carried out with $DLF = 2$. The conclusions are no different. Little benefit is expected from large displacement effects. Consequently, the vault or annulus space was given a 1.2×140 psi static pressure loading and evaluated against the appropriate structural criteria. None of the structures pass, and an open configuration with any cover blocks blown off is concluded. With the exception of the DST, the opening is essentially the roof area. A somewhat smaller opening is expected with the DST as soil cover is thinner at the crown and the haunch concrete is at least twice as thick as the crown.

Detailed calculations are combined with those in Appendix 5, item by item.

Calculations for the gasoline ignition event are combined with the internal vacuum event under each configuration. (App. 5)

Table 9.1. Gasoline Ignition Summary

Tank	Structural Item	OK/NG by item	Overall
241-A-350	Caisson roof	NG	Fully open
244-BX/TX/U	Vault roof	NG	Fully open
244-CR	Vault roof	NG	Fully open
244-S	Caisson roof	NG	Fully open
241-S-304	Caisson roof	NG	Fully open
DST	R/C dome	NG	Open, "25' dia.

Notes: NG = no good (fails structural criteria)
OK = satisfies structural criteria

Appendix 13

High Concrete Temperatures and Pressure Caused by Organic Nitrate Reaction

Item 13 of the work plan requested a review of a proposed failure mechanism for 100-Series SSTs caused by high temperatures and internal pressure from an in-tank organic pool fire. The failure mechanism was described in a memo written by Al Webb and Robert Marusich and is attached to the work plan. The memo predicts internal pressures of 11 psig and temperatures up to 1020 degrees Kelvin. The failure mechanism predicted is: the rebar cover on the lower face of rebar spalls, the tank pressurizes cracking the concrete dome on a 2 foot by 2 foot grid, the internal gas vents through the concrete cracks, the venting gas heats the concrete and rebar to nearly 1000 degrees Kelvin damaging and reducing the strength of the concrete and rebar, the pressure then decreases allowing the dome to lower and sag at which point the dome lacks the strength to support the soil cover and the dome falls into the tank in one large cracked piece approximately 50 to 60 feet in diameter. The memo states that a crack area of 3 square meters is needed to maintain 11 psig.

We have reviewed the proposed failure mechanism and agree with the conclusion that a portion of the dome will possibly fall into the tank. The actual failure mechanism is difficult to predict and may be different from that which was proposed by Webb and Marusich.

- The soil overburden load on the dome is approximately 6 psi near the center of the dome increasing to as much as 15 psi near the haunch. The concrete dome thickness is 15 inches at the center of the dome increasing to 30 inches or more at the haunch. As such, cracking is expected to more severe near the center of the dome. Severe cracking of the dome may be limited to an area of 20 to 30 feet in diameter.
- Concrete cracking was predicted to be at 24 inch centers to coincide with the circumferential rebar placement. It is expected that the rebar cracking will be more closely spaced, more on the order of 12 inch spacing or closer and not associated with the placement of the circumferential rebar. As such, the crack width will be smaller and over a smaller area and the total crack area is estimated to be on the order of 1 square meter which is smaller than the 3 square meters needed to maintain the 11 psig internal pressure. As such the internal pressure would be greater than the 11 psig predicted.
- The 1000 degree Kelvin gas venting through the concrete cracks for up to 1500 seconds (25 minutes) duration will cause severe degradation and loss

of strength to the concrete in the immediate vicinity of the cracks through the thickness. As such, when the pressure reduces and the dome relaxes, the concrete may not be able to sustain the compressive loads associated with the dome arching action and result in partial dome collapse. The loss of rebar strength is expected to have minimal effects on dome collapse.

The proposed organic fire conditions of 11 psig and 1000 degrees Kelvin (1320 degrees F) are severe. Damage to the reinforced concrete tank will occur. Portions of the dome may collapse. Even if the dome does not collapse, it will be have severe strength reductions and should be considered unsafe and unstable.