SAVANNAH RIVER REMEDIATION LLC
Savannah River Site, Aiken, SC 29808

November 16, 2010

SRR-ESH-2010-00101

Mr. J.J. Schnabel
Division of Mining and Solid Waste Management
Bureau of Land and Waste Management
South Carolina Department of Health and
Environmental Control
2600 Bull Street
Columbia, South Carolina 29201-1208

SALTSTONE DISPOSAL FACILITY VAULT 2 CONSTRUCTION (U)

References:

1. Class 3 Landfill Permit for Savannah River Site (SRS) Z-Area Saltstone Disposal Facility, Facility ID No. 025500-1603, Aiken County, 9/9/08
2. CROM Corporation, Area Z Saltstone Storage Tanks, Aiken, South Carolina, Analysis and Design of Exterior Curb at Base of Wall, CROM Job Number 2008-M-084, 9/30/10
3. Salstone Disposal Unit 2 Performance Testing, C-TRT-Z-00001, Revision 0, 9/30/10

The purpose of this memorandum is to inform the South Carolina Department of Health and Environmental Control (SCDHEC) of a change in the Vault 2 disposal cell design described in General Condition B.1.b of Reference 1.

As was discussed during SCDHEC's visit on July 28, 2010, two external concrete curbs were proposed for installation on Vault 2, Cells 2A and 2B. The proposal has changed from the original concept to a one curb design, see Figure 1. The external concrete curb is a defense in depth measure being taken in an effort to aid the water tight design and as a design improvement for the Saltstone Performance Assessment (PA). The Saltstone PA assumes that over a long period of time (e.g., > 10,000 years) water from environmental sources will eventually infiltrate the disposal cell, will interact with the saltstone, and will migrate out of the cell through at least eight inches of sulfate resistant concrete at the wall and twelve inches of sulfate resistant concrete at the floor before it reaches the environment. The migration rate for the PA base case is assumed to be extremely slow.

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The interior curb installed in Vault 2, Cells 2A and 2B, is designed to provide the eight inch wall concrete requirement at the wall to floor interface. However, the detection of moisture on the exterior of the cells at this interface during testing was evidence of a higher than expected migration rate. Similarly, moisture detected at the floor to upper mud mat interface was evidence of a higher than expected migration rate through the floor. Although the minor amount of moisture observed is well within the PA sensitivity analyses, an exterior curb as conceptually depicted in Figure 1 is being installed on Vault 2, Cells 2A and 2B, in order to ensure the Saltstone PA Base Case assumptions are not challenged. The exterior curb will be cast in place using sulfate resistant concrete and steel reinforcement. Hydrophilic material will be incorporated into the design of the exterior curb as an extra layer of defense to aid water tightness.

The Vault 2, Cells 2A and 2B, were previously tested to their maximum capacity (i.e., 22 feet) without the installed exterior curb. The cell walls performed as expected (i.e. no moisture was observed) with the exception of moisture being observed at the wall to floor interface and the floor to upper mud mat interface as described above. The installation of the exterior curb will restrain cell wall movement. As the cell wall is loaded hydrostatically, during water tightness testing, a bending moment is imposed on the wall. The bending moment is acceptable from a structural perspective for a free standing (i.e., without backfill) cell as long as the test water level in the cell is maintained below ~12 feet. Therefore, in order to protect the wall during water tightness tests, a maximum test height of ~12 feet will be imposed. The 12 foot test height assures the structural integrity of the cell wall and the integrity of the modified wall-floor joint, and it verifies water tightness for all internal surface areas exposed to hydraulic head during Vault 2, Cells 2A and 2B, operation (see Appendix 2 engineering justification for test fill height). Higher hydrostatic loads can be accommodated once backfill is installed because the backfill will counter the bending moment provided by the hydrostatic loads, thereby eliminating the need for any long term hydrostatic limits (see Appendix 1). Regardless of these changes, in accordance with Reference 1, once Vault 2, Cells 2A and 2B are placed into operation, the drain water in the cells will be removed to the maximum extent practical at the end of each operating day.

In order to expedite installation of the exterior curb your prompt review and approval of the enclosed design is requested by November 29, 2010.

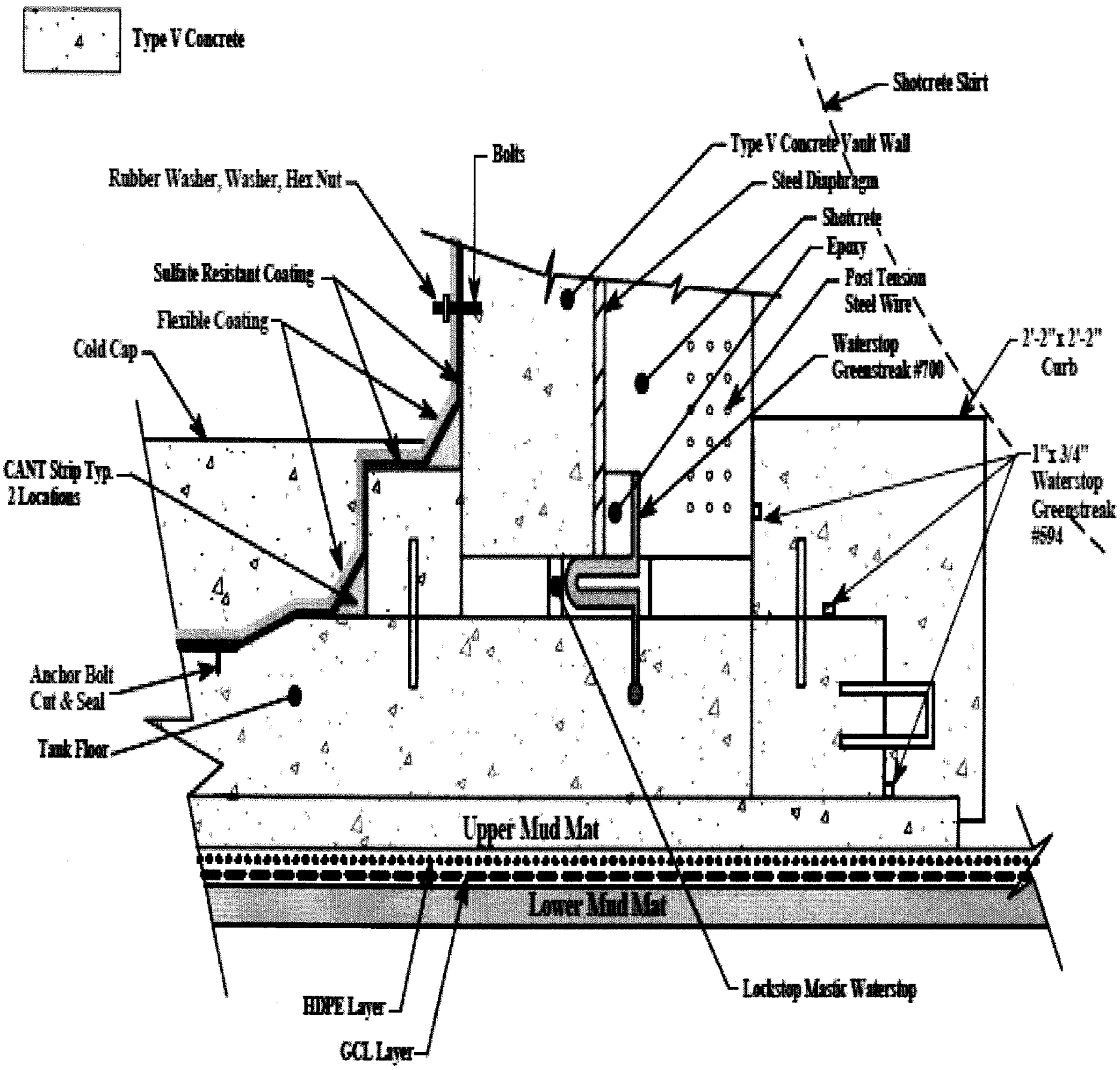
If you have any questions, please contact Keith Liner at (803) 208-6466.

Sincerely,

for Sam R Wilson
P.M. Allen
Manager,
Environmental, Safety, Health, Quality Assurance and Contractor Assurance
Savannah River Remediation, LLC

Figure 1

Note: Items shown in red are changes from original Vault 2, Cells 2A and 2B design.



Appendix 1

CROM Corporation
Area Z Saltstone Storage Tanks
Aiken, South Carolina
Analysis and Design of Exterior Curb at Base of Wall
CROM Job Number 2008-M-084
Summary of Results

PAGE 1
OF
1263

CROM

DESIGN CALCULATIONS

THE CROM CORPORATION

Prestressed Concrete Tanks

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Gainesville, FL 32607

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AREA Z SALTSTONE STORAGE TANKS
AIKEN, SOUTH CAROLINA
ANALYSIS AND DESIGN OF EXTERIOR CURB AT BASE OF WALL
CROM JOB NUMBER 2008-M-084

RP 10/4

SOUTH CAROLINA
THE CROM CORPORATION
No. C00656
CERTIFICATE OF AUTHORIZATION

SOUTH CAROLINA
PROFESSIONAL
KENNETH R. HARVEY

Digitally signed by Kenneth R. Harvey
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PRESTRESSED COMPOSITE TANKS

BUILDING TANKS SINCE 1953

ENGINEERING REPORT

Date:	September 30, 2010	Job No.:	2008-M-084
Job Name:	Area Z Saltstone Storage Tanks - Engineering Report #14 and #15		
Subject:	Addition of Exterior Curb at Base of Wall – IDC # 13		
Requested By:	Savannah River Remediation, LLC		
Performed By:	K. Ryan Harvey, PE		
Reviewed By:	Jeffrey S. Ward, PE		

Purpose of Report:

The purpose of this report is to address the request from Savannah River Remediation, LLC (SRR) to add an external curb to the base of the vaults on the above-referenced project. As provided in the attached email correspondence and sketches, the following items were to be addressed:

1. Monday, July 19, 2010 – SRR requests that Crom analyze the vault to determine if the curb shown on the attached sketch can be added to the exterior of the tank. It is desired to know if the curb can be added with the tank empty. If this condition fails, it is desired to know if the curb can be added with a full tank of water.
2. Tuesday, August 3, 2010 – After providing preliminary information to SRR regarding the addition of the curb on the morning of August 3, SRR requests that Crom analyze the addition of the curb with the vault empty in the following two conditions:
    a. Condition 1: "What is the maximum height of liquid with a specific gravity of 1.3 that the tank walls could safely see? This number would be with respect to the structure in a freestanding condition (no backfill)."
    b. Condition 2: "What would this same height be (specific gravity of 1.3) with full backfill (condition prior to hot operations)?"
3. Tuesday, August 31, 2010 - After providing a draft copy of the analysis to SRS regarding addition of the curb on August 3, 2010, SRR requests that Crom address questions by Ramesh Patel of SRR and include the backfill load with unit weights and earth pressure coefficients provided by SRR.
4. Tuesday, August 31, 2010 - Kevin Lancaster of SRR requests that Crom comment on the adequacy of the tank wall in the backfill only condition.

Design Requirements:

Project specifications require the vault design to be based on the minimum requirements of *ACI 318-05 Building Code Requirements for Structural Concrete, AWWA D110-04 Wire and Strand-Wound, Circular, Prestressed Concrete Water Tanks* and applicable portions of the project specifications. The dimensions and design loads of the structure used for analysis were as follows:

1. Diameter: 150' 0"
2. Sidewall Depth: 22' 0"
3. Unit Weight of Hydrostatic Test Liquid: 62.4 lbs/ft^3
4. Unit Weight of Design Liquid Contents: 82 lbs/ft^3

5. Concrete/Shotcrete Material Strength:
   a. Type V Concrete Design Strength for Wall Analysis, f'_c: 5000 psi
   b. Type V Concrete Field Strength for Foundation: 6000 psi
   c. Shotcrete Design Strength, f'_g : 4000 psi
6. Modulus of Elasticity Used in Analysis, E_c: 4.03 x 10^6 psi
7. Backfill Unit Weight: 112 pcf
8. Active Earth Pressure Coefficient, k_a = 0.33
9. At-Rest Earth Pressure Coefficient, k_o = 0.50

Wall sections used for analysis were as shown on the attached "as-built" drawing by The Crom Corporation. Backfill unit weight and the active earth pressure coefficient are shown on attached email from SRS.

Summary of Analysis:

A. Analysis Methods: We began our analysis by using the classical solution by Timoshenko for thin shell structures to determine moments and shears for a pinned base tank structure under full load of hydrostatic test liquid with a unit weight of 62.4 lbs/ft^3. An FEA model was constructed using the finite element analysis program STAAD.Pro V8i and compared to the classical solution for a pinned base tank to verify the acceptability of the model. Once the FEA model was verified for a pinned base tank structure, it was modified to include base restraint which would occur with the addition of the 10" tall concrete curb. This curb would in effect restrain the wall causing it to act like a partially fixed base structure. The moment and shear capacity of the "as-built" wall section was calculated using allowable stress methods and determined to be approximately 12.58 kip-ft/ft and 78 psi respectively. The model was compared to a classical analysis of a fixed based tank and the results compared favorably. Therefore, we found the model to be acceptable.

B. Analysis Results: The results of the FEA model were used to answer the following design questions:

1. **Analysis 1 - Curb Installed Prior to Hydrotest with No Backfill:**
   The exterior concrete curb was designed as a beam ledge using applicable methods provided in ACI 318-05. Concrete curb design calculations for this analysis are shown on pages 8 - 16 of the report. The allowable reaction force at the top of the concrete curb based on the design is 9.2 kips.

   In order to determine if the curb could be installed prior to testing the tank with a full depth of the hydrotest liquid, the model was run with a full tank at the hydrotest liquid unit weight of 62.4 lbs/ft^3. This liquid level produced a moment on the inside of the tank wall of 16.3 kip-ft/ft. This moment did not compare favorably to the moment capacity of the "as-built" wall section and therefore we conclude that this condition is not acceptable. Based on that result, a second model was generated with a liquid level of hydrotest liquid equal to 17' 10" or 18' 1" above the finish floor level. The liquid level in the second model produced a moment on the inside of the tank wall of 11.3 kip-ft/ft with a shear in the wall of 30.5 psi and a reaction in the . These values compare favorably to the allowable moment and shear calculated from the "as-built" section of the tank wall; however, the reaction force at the top of the curb for this condition was determined to be 20.63 from the model. This reaction value is unacceptable based on the available curb capacity. Therefore, we find that the reaction force at the top of the curb is the controlling factor in the depth of hydrostatic test liquid that can be placed in the tank.

Two additional models were run to determine the acceptable level of hydrostatic test liquid based on the available capacity of the concrete curb. These models were run using a hydrostatic test liquid depth of 8' 10" and 12' 10" of wall height. Results from those models are shown in pages 98 - 595 of this report. From these models, it was determined that the level of hydrostatic test liquid that would produce a force of 9.2 kips/ft would be 11' 10" of hydrostatic test liquid or 12' 0" above the finish floor level of the tank. Therefore, we conclude that the curb could be installed prior to the hydrotest if the hydrotest were carried out with a maximum hydrostatic test liquid depth of 12' 0" above the finished floor elevation of the tank.

2. **Analysis 2 – Curb Installed with Tank Empty – Determine Acceptable Level of Design Liquid After Backfill is in Place:**
The exterior concrete curb was again designed as a beam ledge using applicable methods provided in ACI 318-05 for this analysis condition taking into account the additional resisting force that is provided by the backfilled condition. Concrete curb design calculations for this analysis are shown on pages 17 - 25 of the report. The allowable reaction force at the top of the concrete curb based on the design is 16.1 kips.

   Using unit weights and active earth pressure coefficients provided by SRS, we ran an analysis to determine the maximum height of the design liquid that could be in the tank when the tank is in the backfilled condition. A model was generated with various liquid levels using the design liquid weight to determine the maximum allowable level for the design liquid in the backfilled condition. The final model utilized a liquid level of 19' 10" (or 20' 1" above finished floor) using the design liquid unit weight. This model produced a moment of 8.3 kip-ft/ft with a shear of 22.21 psi and a reaction force at the top of the curb of 15.10 kips. These values compare favorably to the allowable moment and shear calculated from the "as-built" section of the tank and with the design capacity of the concrete curb. A second model was generated to determine the available increase in the design liquid height for this condition based on the additional capacity of the curb. This model produced a moment of 9.8 kip-ft/ft with a shear of 25.37 psi and a reaction force at the top of the curb of 17.66 kips. Results of these models are shown in pages 596 - 1257 of this report. These values compare favorably to the allowable moment and shear calculated from the "as-built" section of the tank; however, the reaction force at the top of the curb was not acceptable based on the available capacity. Based on the two models and the reaction forces calculated at the top of the curb, we conclude that the curb could be installed with the tank empty if the tank were only filled to a level of 20' 4" of wall height or 20' 7" from finish floor with the design liquid after installation of the backfill.

3. **Analysis 3 – Curb Installed with Tank Full:**
In order to determine if the curb could be installed with the tank full of hydrostatic test liquid, the model was run with a load equal to the difference between the full height of design liquid of 82 lbs/ft^3 and a full height of the hydrostatic test liquid of 62.4 lbs/ft^3 or 19.60 lbs/ft^3. The moment in the wall using this model was determined to be 5.2 kip-ft/ft with the shear being 13.39 psi. The moments and shears in this model compare favorably with those calculated from the "as-built" wall section. There would be no reaction on the exterior curb in this condition and therefore, we conclude that the curb could be added after filling the tank full of hydrostatic test liquid.

4. **Analysis 4 – Curb Installed with Tank Empty – Determine Acceptable Level of Design Liquid without Backfill:**
In order to determine the maximum height of the design liquid that could be in the tank, we began by running a model with a liquid level of 4' 10" using the design liquid unit weight of 82 lbs/ft^3. This liquid level produced a moment in the tank wall of approximately .50 kip-ft/ft. A second model was generated with a liquid level of 8' 10" which produced a moment in the tank wall of approximately 3.0 kip-ft/ft. Based on these two moments, it was determined that the approximate acceptable

height of the liquid would most nearly be 12' 0". A third model was generated with a liquid level of 11' 10" using the design liquid unit weight. This model produced a moment of 6.28 kip-ft/ft with a shear of 21.62 psi; however, the model shows a reaction force at the curb of 15.0 kip. Without the resisting force of the backfill in place this reaction force exceeds the maximum allowable reaction force for the curb design and therefore we conclude that this condition is not acceptable at this level without backfill in place.

5. **Analysis 5 - Curb Installed with Tank Empty - Backfill Placed with No Internal Contents:** In addition to the analysis with design liquid, we were asked to perform analysis on the tank wall to determine if the curb could be installed with the tank empty prior to putting backfill in place. The backfill would then placed with the empty tank condition. The model from Analysis 4 was used to determine moments and shears for this condition based on a unit weight of soil as provided by SRS combined with an assumed At-Rest Earth Pressure Coefficient, k_o of 0.50 with the backfill in the drained condition. Based on model results, the moment in this condition would be of 4.3 kip-ft/ft on the inside of the tank wall and 14.5 kip-ft/ft on the outside of the tank wall. The shear in the tank wall from this condition would be 36.6 psi.

   The maximum allowable shear calculated from the "as-built" section of the tank wall compares favorably to this shear determined from this condition; however, we find that the available moment capacity of the "as-built" wall section is slightly less than the moment capacity of the tank wall for this condition using reinforcing steel alone; however, the moment is acceptable when calculating the allowable moment in the tank wall using the combined area of the reinforcing steel and the steel shell diaphragm. This is a standard calculation method in the industry. As shown on the attached calculation page 1261, this method of calculation results in a moment capacity of 17.75 kip-ft/ft and therefore we would conclude that the curb could be installed with the tank empty prior to backfill placement.

The concrete curb will be attached to the outside of the existing tank foundation using reinforcing steel dowels which are placed with an epoxy adhesive as depicted in the Exterior Curb Design drawing on page 1258 of the report. The attached product charts from Powers Fasteners shows the required embedment and ultimate tension and shear capacities for the epoxy adhesive. The embedment of the main #6 reinforcing bar in the curb which is anchored from the top of the footing is required to be 6 inches per the product data table; however, from a construction standpoint, this minimum embedment would encroach upon the bottom rebar mat in the footing. In order to alleviate this problem the rebar embedment will be reduced by ½ inch. As shown in the calculations titled Concrete Curb Design Analysis 1 and Concrete Curb Design Analysis 2, the curb rebar interaction is approximately 0.62 or 52% of the 1.2 allowed by Code. Based on stress levels calculated, the curb rebar will still have sufficient capacity, therefore 5 ½" embedment depth for the curb anchors is acceptable.

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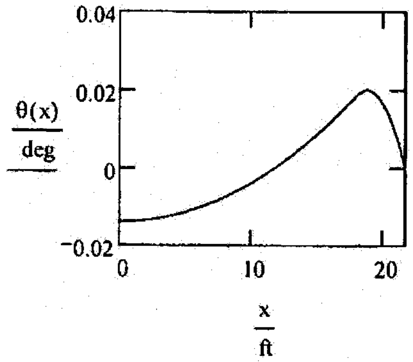
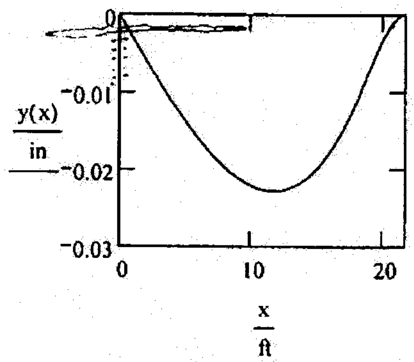


Deflection

$$y(x) := y_A + \theta_A \cdot x + \frac{M_A \cdot x^2}{2 \cdot E \cdot I} + \frac{R_A \cdot x^3}{6 \cdot E \cdot I} - (x > a) \cdot \left[\frac{W}{6 \cdot E \cdot I} \cdot (x - a)^3\right]$$

$y(x_1) = -0.02 \cdot in$



FINITE ELEMENT ANALYSIS SUMMARY OF RESULTS

Anaysis	Condition	Saltstone Drainwater $Height_5$ (ft)	Hydrostatic Test Liquid Height (ft)	Soil Height (ft)	Fx_1 (kip/ft)	Fy_1 (kip/ft)	Fr_2 (kip/ft)	Mx_3 (kip-ft/ft)	Sqx_4 (psi)
1A	Hydrostatic Test	0	8.8333	0	-6.156	-0.082	6.16	2.87	10.72
1B	Hydrostatic Test	0	12.833	0	-11.066	-0.147	11.07	5.6	17.95
2A	Saltstone Drain Water with Backfill	19.833	0	22	-15.1	-0.201	15.10	8.28	22.21
2B	Saltstone Drain Water with Backfill	20.833	0	22	-17.658	-0.235	17.66	9.8	25.37

Subscript Notes:

1. Fx and Fy are reaction values taken from node 243 at the top of the curb.
2. Fr is calculated by combining Fx and Fyusing square root sum of squares.
3. Moment is taken as maximum moment value at plate 6876
4. Shear is taken as maximum shear value at a distance "d" away from top of curb from plate 6877
5. Value is based on wall height. The 3" rise must be added to obtain the total liquid height as specified in the report.

CAPACITY SUMMARY OF RESULTS

Number	Calculation	Method	Value
1	Allowable Moment Capacity of Wall Section - Outside Steel (kip-ft/ft)	Hand	13.58
2	Allowable Moment Capacity of Wall Section - Inside Steel (kip-ft/ft)	Hand	12.58
3	Allowable Shear Capacity of Wall Section (psi)	Hand	77.78
4	Allowable Capacity of Concrete Curb for Fr (kip/ft) - Analysis 1	Mathcad	9.2
5	Allowable Capacity of Concrete Curb for Fr (kip/ft) - Analysis 2	Mathcad	16.1

Design of Concrete Curb

Reference: *ACI318-05 Building Code Requirements for Structural Concrete and Commentary.*
PCA Notes on ACI318-05
CROM Finite Element Analysis for Analysis Condition 1

Objective: *To design a perimeter concrete curb on the outside of a sliding base concrete tank wall*

A. INPUT:

I. Geometry Info:

i. Curb Info:

$b_w := 12 \cdot in$ Unit length of concrete curb

$W_{curb} := 26 \cdot in$ Width of curb (excluding tolerance)

$H_{curb} := 12 \cdot in$ Height of curb above foundation (excluding tolerance)

ii. Floor Info:

$t_{floor} := 11 \cdot in$ Thickness of existing Type V floor under curb

II. Loadings:

i. Load on Curb:

$V := 9.2 \cdot kip$ Reaction force per foot at top of curb

ii. Input Variables to Calculate Available Resistance to Ultimate Shear Force:

$h_{curb} := 25.75 \cdot in$ Height of curb at face (excluding tolerance)

$\gamma_{soil} := 112 \cdot \frac{lbf}{ft^3}$ Unit weight of soil to be used with active earth pressure coef. per SRS directior

$k_a := 0.33$ Active earth pressure coefficient per SRS direction

$EL_{tsoil} := 271.10 \cdot ft$ Upper elevation of soil at top of tank wall after backfill is placed

$EL_{tc} := 271.10 \cdot ft$ Elevation at top of curb

$EL_{bc} := 268.94 \cdot ft$ Elevation at base of curb

iii. Force on Face of Curb from Soil:

$v_{top} := \gamma_{soil} \cdot k_a \cdot (0)$	Pressure from soil at top of curb	$v_{top} = 0 \cdot psf$
$v_{base} := \gamma_{soil} \cdot k_a \cdot (0)$	Pressure from soil at base of curb	$v_{base} = 0 \cdot psf$
$V_1 := v_{top} \cdot h_{curb} \cdot b_w$	Force on curb face from rectangular portion of pressure dist.	$V_1 = 0$
$V_2 := \left(v_{base} - v_{top}\right) \cdot h_{curb} \cdot b_w \cdot \frac{1}{2}$	Force on curb face from triangular portion of pressure dist.	$V_2 = 0$
$V_T := V_1 + V_2$	Total force on face of curb from active soil pressure	$V_T = 0$

iv. Resisting Force Due to Soil on Top of Curb:

$v_{sw} := \left(EL_{tsoil} - EL_{tc}\right) \cdot \gamma_{soil}$	Calculate pressure from soil on top of curb	$v_{sw} = 0 \cdot psf$
$V_{sw} := v_{sw} \cdot b_w \cdot W_{curb}$	Calculate total force on top of curb from soil weight	$V_{sw} = 0\,lbf$
$V_{swtotal} := V_{sw} \cdot 2 \cdot 0.5$	Total force on top of curb with two surfaces in friction at 0.5	$V_{swtotal} = 0\,lbf$

v. Resisting Force Due to Wall Weight on Top of Foundation:

$w_{wall} := 5240 \cdot plf$	Weight of tank wall taken from tank design calculations, Rev B.	$w_{wall} = 5240 \cdot plf$
$V_{wallnorm} := w_{wall} \cdot b_w \cdot 0.5$	Resisting force from wall weight	$V_{wallnorm} = 2620\,lbf$

vi. Calculate total Force Taking Account of Shear Force and Resisting Forces:

$V_{TRS} := V_T + V_{swtotal}$	Total Resisting Force from soil above	$V_{TRS} = 0\,lbf$
$V_{TRW} := V_{wallnorm}$	Total Resisting Force from wall above	$V_{TRW} = 2620\,lbf$

$V := 9200 \cdot lbf$ Total unfactored shear force from FEA $V = 9200\,lbf$

$V_T := V - V_{TRS} - V_{TRW}$ Total shear force minus resistance of soil and wall weight $V_T = 6580\,lbf$

III. Material Properties:

i. Concrete:

$\mu := 0.6$ Coefficient of friction of concrete placed against hardened concrete with surface not intentionally roughened (ACI 11.7.4.3)

$f_c := 4000 \cdot psi$ Compressive strength of concrete for curb

$E_c := 57000\sqrt{f_c \cdot psi}$ Modulus of elasticity of concrete $E_c = 3605 \cdot ksi$

ii. Reinforcing Steel:

$f_y := 60000 \cdot psi$ Yield strength of reinforcing steel

$E_s := 29000 \cdot ksi$ Modulus of elasticity of steel

$rebar := 6$ Enter rebar size (4 for #4, 5 for #5, etc.)

$cover := 1.5 \cdot in$ Clear cover of concrete on outside of curb

$d_{bar}(rebar) = 0.75 \cdot in$ Diameter of bar $d_{bar}(rebar) = 0.75 \cdot in$

$A_{bar}(rebar) = 0.44 \cdot in^2$ Cross sectional area of bars $A_{bar}(rebar) = 0.44 \cdot in^2$

$sp_{bar} := 12 \cdot in$ Spacing of rebar

IV. ACI318 Provisions:

$\phi_f := 0.75$ Strength reduction factor for flexure for corbels (ACI 11.9.3.1)

$\phi_b := 0.65$ Strength reduction factor for bearing on concrete (ACI 9.3.2.4)

$\phi_v := 0.75$ Strength reduction factor for shear (ACI 9.3.2.3)

$\varepsilon_c := 0.003$ Maximum Compressive Strain of Concrete 0.003 (ACI 10.2.3)

B. CALCULATIONS:

I. Calculate Factored Load:

$V_u := 1.4 \cdot V_T$	Factored shear on concrete curb (ACI 9.2.1)	$V_u = 9.212 \cdot kip$

II. Check Bearing Capacity:

$A_1 := 1 \cdot in \cdot b_w$	Bearing area with 1" bearing height	$A_1 = 12 \cdot in^2$
$\phi P_n := \phi_b \cdot (0.85 \cdot f'_c \cdot A_1)$	Bearing capacity of concrete (ACI 10.17.1)	$\phi P_n = 26.52 \cdot kip$
$check_{bearing} := if(V_u \le \phi P_n, \text{"OK"}, \text{"INCREASE BEARING AREA"})$		$check_{bearing} = \text{"OK"}$

III. Check Shear Friction: (Use ACI 11.9.3)

$d := W_{curb} - cover - d_{bar}(rebar) \div 2$	Effective depth of concrete curb	$d = 24.125 \cdot in$
$A_{vf\_prov} := 2 \cdot \frac{b_w}{sp_{bar}} \cdot A_{bar}(rebar)$	Amount of steel provided for shear friction	$A_{vf\_prov} = 0.88 \cdot in^2$
$\phi V_{n\_vf\_1} := \phi_v \cdot A_{vf\_prov} \cdot f_y \cdot \mu$	Shear friction capacity (ACI 11.7.4.1)	$\phi V_{n\_vf\_1} = 23.76 \cdot kip$
$\phi V_{n\_vf\_2} := 0.2 \cdot f'_c \cdot b_w \cdot d$	Shear friction capacity (ACI 11.9.3.2.1)	$\phi V_{n\_vf\_2} = 231.6 \cdot kip$
$\phi V_{n\_vf\_3} := 800 \cdot psi \cdot b_w \cdot d$	Shear friction capacity (ACI 11.9.3.2.1)	$\phi V_{n\_vf\_3} = 231.6 \cdot kip$
$\phi V_{n\_vf} := \min(\phi V_{n\_vf\_1}, \phi V_{n\_vf\_2}, \phi V_{n\_vf\_3})$	Shear friction capacity of concrete curb (ACI 11.9.3.2)	$\phi V_{n\_vf} = 23.76 \cdot kip$
$check_{Vfriction} := if(V_u \le \phi V_{n\_vf}, \text{"OK"}, \text{"INCREASE REINFORCING STEEL"})$		$check_{Vfriction} = \text{"OK"}$

IV. Check Flexure Capacity for Corbels:

$l_f := 1$	Factor for location of force from top of curb	$l_f = 1$
$a_f := \frac{(H_{curb} + cover + d_{bar}(rebar) \div 2)}{l_f}$	Distance from load to floor reinforcement	$a_f = 13.875 \cdot in$
$M_u := V_u \cdot a_f$	Factored moment (ACI 11.9.3.3)	$M_u = 10.7 \cdot kip \cdot ft$
$A_{s\_prov} := \frac{b_w}{sp_{bar}} \cdot A_{bar}(rebar)$	Amount of steel provided for flexure	$A_{s\_prov} = 0.44 \cdot in^2$
$\alpha_1 := 0.85$	Factor for equivalent rectangular stress block (ACI 10.2.7.1)	$\alpha_1 = 0.85$

$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000 \cdot psi \\ 0.65 & \text{if } f'_c > 8000 \cdot psi \\ 1.05 - 0.05 \cdot \dfrac{f'_c}{1000 \cdot psi} & \text{if } 4000 \cdot psi < f'_c \le 8000 \cdot psi \end{cases}$ Ratio of depth of equivalent rectangular stress block, a, to depth to neutral axis,c (ACI 10.2.7.3) $\beta_1 = 0.85$

$a := \dfrac{A_{s\_prov} \cdot f_y}{\alpha_1 \cdot f'_c \cdot b_w}$ Depth of equivalent rectangular stress block $a = 0.647 \cdot in$

$c := \dfrac{a}{\beta_1}$ Depth of neutral axis $c = 0.761 \cdot in$

$\varepsilon_t := \dfrac{\varepsilon_c \cdot (d - c)}{c}$ Net tensile strain in extreme-tension steel at nominal strength $\varepsilon_t = 0.092$

$\varepsilon_s := \dfrac{\varepsilon_c \cdot (d - c)}{c}$ Tensile strain in centroid of steel $\varepsilon_s = 0.09207$

$\varepsilon_y := \dfrac{f_y}{E_s}$ Yield strain in tension $\varepsilon_y = 0.002$

$check_{strain1} := \begin{cases} \text{"STEEL HAS NOT YIELDED!!! Reduce Reinforcing Steel"} & \text{if } \varepsilon_t < \varepsilon_y \\ \text{"Assumption that Steel has yielded is valid."} & \text{otherwise} \end{cases}$

$check_{strain1} = \text{"Assumption that Steel has yielded is valid."}$

$check_{strain2} := \begin{cases} \text{"Tension Controlled Section"} & \text{if } \varepsilon_t \ge 0.005 \\ \text{"UPPER LIMIT NOT SATISFIED!!! Reduce Reinforcing Steel"} & \text{if } \varepsilon_t < 0.004 \end{cases}$

ACI - 10.3.5 sets a upper limit for steel at 0.004 for ε_t $check_{strain2} = \text{"Tension Controlled Section"}$

$M_n := A_{s\_prov} \cdot f_y \cdot \left(d - \dfrac{a}{2}\right)$ Moment capacity $M_n = 52.4 \cdot kip \cdot ft$ →

$\phi M_n := \phi_f \cdot M_n$ Moment capacity with strength reduction factor $\phi M_n = 39.3 \cdot kip \cdot ft$

$check_{M\_pos} := if(M_u \le \phi M_n, \text{"OK"}, \text{"CHECK SECTION CAPACITY"})$ Check Moment Capacity $check_{M\_pos} = \text{"OK"}$

V. Check Tension and Shear in Reinforcing Bars and Epoxy Embedment:

$dist_{bars} := W_{curb} - 2\cdot cover - d_{bar}(rebar)$	Distance between reinforcing bars resisting moment	$dist_{bars} = 22.25\cdot in$
$N_{u\_bar} := \frac{V_u\cdot H_{curb}}{dist_{bars}}\cdot\frac{sp_{bar}}{b_w}$	Tension in each bar	$N_{u\_bar} = 4.97\cdot kip$
$V_{u\_bar} := V_u\cdot\frac{sp_{bar}}{2\cdot b_w}$	Shear in each bar	$V_{u\_bar} = 4.61\cdot kip$

Use Powers PE1000+, fpr #6 bars with 6" min embedment

$h_{ef} := 6\cdot in$	Effective embedment depth	
$\phi V_{cb} := 14505\cdot lbf$	Shear capacity of each bar with epoxy (PE1000+)	
$\phi N_{cb} := 17760\cdot lbf$	Tension capacity of each bar with epoxy (PE1000+)	
$h_{min} := h_{ef} + 2\cdot d_{bar}(rebar)$	Minimum depth of concrete floor	$h_{min} = 7.5\cdot in$
$check_{depth} := if(H_{curb} \ge h_{min}, \text{"OK"}, \text{"INCREASE FLOOR THICKNESS"})$		$check_{depth} = \text{"OK"}$

VI. Check Tension and Shear in Reinforcing Bars Using Flexural Theory:

$a = \frac{A_{s\_req}\cdot f_y}{\alpha_1\cdot f'_c\cdot b_w}$	Depth of equivalent rectangular stress block	
$M_u = \phi_f\cdot A_{s\_req}\cdot f_y\cdot\left(d - \frac{a}{2}\right)$	Equilibrium equation	
$f(A_{s\_req}) := \phi_f\cdot A_{s\_req}\cdot f_y\cdot\left(d - \frac{A_{s\_req}\cdot f_y}{2\cdot\alpha_1\cdot f'_c\cdot b_w}\right) - M_u$		
$A_{s\_req} := root\left(f(A_{s\_req}), A_{s\_req}, 0\cdot in^2, 2\cdot in^2\right)$	Amount of reinforcing steel required for moment	$A_{s\_req} = 0.118\cdot in^2$
$a := \frac{A_{s\_req}\cdot f_y}{\alpha_1\cdot f'_c\cdot b_w}$	Depth of equivalent rectangular stress block	$a = 0.174\cdot in$
$c := \frac{a}{\beta_1}$	Depth of neutral axis	$c = 0.204\cdot in$

~~$\varepsilon_t := \frac{\varepsilon_c \cdot (d - c)}{c}$ Net tensile strain in extreme-tension steel at nominal strength $\varepsilon_t = 0.351$~~

$\varepsilon_s := \frac{\varepsilon_c \cdot (d - c)}{c}$ Tensile strain in centroid of steel $\varepsilon_s = 0.35103$

$check_{strain1} :=$ | "STEEL HAS NOT YIELDED!!! Reduce Reinforcing Steel" if $\varepsilon_t < \varepsilon_y$
| "Assumption that Steel has yielded is valid." otherwise

$check_{strain1} =$ "Assumption that Steel has yielded is valid."

$check_{strain2} :=$ | "Tension Controlled Section" if $\varepsilon_t \geq 0.005$
| "UPPER LIMIT NOT SATISFIED!!! Reduce Reinforcing Steel" if $\varepsilon_t < 0.004$

ACI - 10.3.5 sets a upper limit for steel at 0.004 for ε_t $check_{strain2} =$ "Tension Controlled Section"

$T := \phi_f \cdot A_{s\_req} \cdot f_y$ Tension in rebars $T = 5.32 \cdot kip$

$N_{u\_bar} := T \cdot \frac{sp_{bar}}{b_w}$ Tension in each bar $N_{u\_bar} = 5.32 \cdot kip$

$V_{u\_bar} := V_u \cdot \frac{sp_{bar}}{2 \cdot b_w}$ Shear in each bar $V_{u\_bar} = 4.61 \cdot kip$

$check_{shear} := if(V_{u\_bar} \leq \phi V_{cb}, "OK", "DECREASE BAR SPACING")$ $check_{shear} =$ "OK"

$check_{ten} := if(N_{u\_bar} \leq \phi N_{cb}, "OK", "DECREASE BAR SPACING")$ $check_{ten} =$ "OK"

$s_f := 1.0$ Factor to account for spacing of anchor $s_f = 1$

$et_f := 1.0$ Factor to account for edge distance of anchor in tension $et_f = 1$

$ev_f := 1.0$ Factor to account for edge distance of anchor in shear $ev_f = 1$

$OS := 1.0$ Overstress factor

$combine := \frac{V_{u\_bar}}{\phi V_{cb} \cdot s_f \cdot ev_f} + \frac{N_{u\_bar}}{\phi N_{cb} \cdot s_f \cdot ev_f}$ Combine effect of shear and tension, ACI App. D7.3 $combine = 0.617$

$check_{combine} := if(combine \leq 1.2 \cdot OS, "OK", "DECREASE BAR SPACING")$ $check_{combine} =$ "OK"

$$\frac{4.61}{\underset{12.952}{14.505}} + \frac{5.32 \xrightarrow{} 4.97}{\underset{15.79}{17.76}} = 0.617 \quad \boxed{= 0.63}$$

VII. Calculate Hanger Bar Reinforcement:

$rebar := 4$	Enter rebar size for hanger bar reinforcement	
$A_{bar}(rebar) = 0.2 \cdot in^2$	Reinforcing bar area	
$d_{bar}(rebar) = 0.5 \cdot in$	Reinforcing bar diameter	
$s := 9 \cdot in$	Spacing of hanger reinforcing bar	
$S := 12 \cdot in$	Distribution of force for hanger reinforcing bar	
$A_v := \frac{V_u \cdot s}{\phi_v \cdot f_y \cdot S}$	Calculate shear reinforcement for hanger bar	$A_v = 0.154 \cdot in^2$
$A_{prov} := A_{bar}(rebar)$	Calculate provided area of reinforcement for hanger ba	$A_{prov} = 0.2 \cdot in^2$
$CheckA_{vh} := if\left(A_v \le A_{prov}, \text{"OK"}, \text{"Decrease Spacing"}\right)$		$CheckA_{vh} = \text{"OK"}$

VIII. Check Embed Depth for Epoxy for Hanger Bar:

$V_u = 9212\, lbf$	Shear force that must be resisted by hanger bar	$V_u = 9212\, lbf$
$V_{hb} := V_u \cdot \frac{s}{S}$	Shear force that each hanger bar must carry	$V_{hb} = 6909\, lbf$
$h_{efhb} := 4 \cdot in$	Effective embedment depth	
$\phi V_{cbhb} := 6550 \cdot lbf$	Shear capacity of each bar with epoxy (PE1000+)	
$\phi N_{cbhb} := 8735 \cdot lbf$	Tension capacity of each bar with epoxy (PE1000+)	
$s_f := 1.0$	Factor to account for spacing of anchor	$s_f = 1$

$et_f := 1.0$	Factor to account for edge distance of anchor in tension	$et_f = 1$
$ev_f := 1.0$	Factor to account for edge distance of anchor in shea	$ev_f = 1$
$\phi N_{hb} := \phi N_{cbhb} \cdot s_f$	Total available tension force in anchor	$\phi N_{hb} = 8735\,lbf$
$Check\phi N_{hb} := if(\phi N_{hb} \geq V_{hb}, \text{"OK"}, \text{"Decrease Bar Spacing"})$	Check tension capacity	$Check\phi N_{hb} = \text{"OK"}$

General

CROM

The Crom Corporation
Project: Two - 2,900,000-Gallon Saltstone
Storage Tanks
Savannah River Site

Job #:2008-M-084

Design of Concrete Curb

Reference: *ACI318-05 Building Code Requirements for Structural Concrete and Commentary*
PCA Notes on ACI318-05
CROM Finite Element Analysis for Analysis Condition 2

Objective: *To design a perimeter concrete curb on the outside of a sliding base concrete tank wall for Analysis Condition 2 - Design Liquid in Tank with Backfill in Place*

A. INPUT:

I. Geometry Info:

i. Curb Info:

$b_w := 12 \cdot in$ — Unit length of concrete curb

$W_{curb} := 26 \cdot in$ — Width of curb (excluding tolerance)

$H_{curb} := 12 \cdot in$ — Height of curb above foundation

ii. Floor Info:

$t_{floor} := 11 \cdot in$ — Thickness of existing Type V floor under curb

II. Loadings:

i. Load on Curb:

$V := 16.1 \cdot kip$ — Maximum reaction force at top of curb.

ii. Input Variables to Calculate Available Resistance to Ultimate Shear Force:

$h_{curb} := 25.75 \cdot in$ — Height of curb at face (excluding tolerance)

$\gamma_{soil} := 112 \cdot \frac{lbf}{ft^3}$ — Unit weight of soil to be used with active earth pressure coef. per SRS direction

$k_a := 0.33$ — Active earth pressure coefficient per SRS direction

$EL_{tsoil} := 292.27 \cdot ft$ — Upper elevation of soil at top of tank wall after backfill is placed

$EL_{tc} := 270.86 \cdot ft$ — Elevation at top of curb

$EL_{bc} := 268.94 \cdot ft$ — Elevation at base of curb

iii. Force on Face of Curb from Soil:

$v_{top} := \gamma_{soil} \cdot k_a \cdot (EL_{tsoil} - EL_{tc})$	Pressure from soil at top of curb	$v_{top} = 791.314 \cdot \frac{lbf}{ft^2}$
$v_{base} := \gamma_{soil} \cdot k_a \cdot (EL_{tsoil} - EL_{bc})$	Pressure from soil at base of curb	$v_{base} = 862.277 \cdot \frac{lbf}{ft^2}$
$V_1 := v_{top} \cdot h_{curb} \cdot b_w$	Force on curb face from rectangular portion of pressure dist.	$V_1 = 1698.027\,lbf$
$V_2 := (v_{base} - v_{top}) \cdot h_{curb} \cdot b_w \cdot \frac{1}{2}$	Force on curb face from triangular portion of pressure dist.	$V_2 = 76.138\,lbf$
$V_T := V_1 + V_2$	Total force on face of curb from active soil pressure	$V_T = 1774.165\,lbf$

iv. Resisting Force Due to Soil on Top of Curb:

$v_{sw} := (EL_{tsoil} - EL_{tc}) \cdot \gamma_{soil}$	Calculate pressure from soil on top of curb	$v_{sw} = 2397.92 \cdot psf$
$V_{sw} := v_{sw} \cdot b_w \cdot W_{curb}$	Calculate total force on top of curb from soil weight	$V_{sw} = 5195.493\,lbf$
$V_{swtotal} := V_{sw} \cdot 2 \cdot 0.5$	Total force on top of curb with two surfaces in friction at 0.5	$V_{swtotal} = 5195.493\,lbf$

v. Resisting Force Due to Wall Weight on Top of Foundation:

$w_{wall} := 5240 \cdot \frac{lbf}{ft}$	Weight of tank wall taken from tank design calculations	$w_{wall} = 5240 \cdot \frac{lbf}{ft}$
$V_{wallnorm} := w_{wall} \cdot b_w \cdot 0.5$	Resisting force from wall weight	$V_{wallnorm} = 2620\,lbf$

vi. Calculate total Force Taking Account of Shear Force and Resisting Forces:

$V_{TRS} := V_T + V_{swtotal}$	Total Resisting Force from soil above	$V_{TRS} = 6969.658\,lbf$

$V_{TRW} := V_{wallnorm}$	Total Resisting Force from wall above	$V_{TRW} = 2620\,lbf$
$V = 16100\,lbf$	Total unfactored shear force from FEA	$V = 16100\,lbf$
$V_T := V - V_{TRS} - V_{TRW}$	Total shear force minus resistance of soil and wall weight	$V_T = 6510.342\,lbf$

III. Material Properties:

i. Concrete:

$\mu := 0.6$	Coefficient of friction of concrete placed against hardened concrete with surface not intentionally roughened (ACI 11.7.4.3)	
$f'_c := 4000 \cdot psi$	Compressive strength of concrete for concrete curb	
$E_c := 57000\sqrt{f'_c \cdot psi}$	Modulus of elasticity of concrete	$E_c = 3605 \cdot ksi$

ii. Reinforcing Steel:

$f_y := 60000 \cdot psi$	Yield strength of reinforcing steel	
$E_s := 29000 \cdot ksi$	Modulus of elasticity of steel	
$rebar := 6$	Reinforcing bar size (4 for #4, 5 for #5, etc.)	
$cover := 1.5 \cdot in$	Clear cover of concrete on outside of curb	
$d_{bar}(rebar) = 0.75 \cdot in$	Diameter of bar	$d_{bar}(rebar) = 0.75 \cdot in$
$A_{bar}(rebar) = 0.44 \cdot in^2$	Cross sectional area of bars	$A_{bar}(rebar) = 0.44 \cdot in^2$
$sp_{bar} := 12 \cdot in$	Spacing of rebar	

IV. ACI318 Provisions:

$\phi_f := 0.75$	Strength reduction factor for flexure for corbels (ACI 11.9.3.1)	
$\phi_b := 0.65$	Strength reduction factor for bearing on concrete (ACI 9.3.2.4)	
$\phi_v := 0.75$	Strength reduction factor for shear (ACI 9.3.2.3)	
$\varepsilon_c := 0.003$	Maximum Compressive Strain of Concrete 0.003 per (ACI 10.2.3)	

B. CALCULATIONS:

I. Calculate Factored Load:

$V_u := 1.4 \cdot V_T$	Factored shear on concrete curb	$V_u = 9.114 \cdot kip$

II. Check Bearing Capacity:

$A_1 := 1 \cdot in \cdot b_w$	Bearing area with 1" bearing height	$A_1 = 12 \cdot in^2$
$\phi P_n := \phi_b \cdot (0.85 \cdot f_c \cdot A_1)$	Bearing capacity of concrete	$\phi P_n = 26.52 \cdot kip$
$check_{bearing} := if(V_u \le \phi P_n, \text{"OK"}, \text{"INCREASE BEARING AREA"})$		$check_{bearing} = \text{"OK"}$

<u>NOTES:</u> Bending does not check according to ACI 11.8.7

III. Check Shear Friction: (Use ACI 11.8.3)

$d := W_{curb} - cover - d_{bar}(rebar) \div 2$	Effective depth of concrete curb	$d = 24.125 \cdot in$
$A_{vf\_prov} := 2 \cdot \frac{b_w}{sp_{bar}} \cdot A_{bar}(rebar)$	Amount of steel provided for shear friction	$A_{vf\_prov} = 0.88 \cdot in^2$
$\phi V_{n\_vf\_1} := \phi_v \cdot A_{vf\_prov} \cdot f_y \cdot \mu$	Shear friction capacity (ACI 11.7.4)	$\phi V_{n\_vf\_1} = 23.76 \cdot kip$
$\phi V_{n\_vf\_2} := 0.2 \cdot f_c \cdot b_w \cdot d$	Shear friction capacity (ACI 11.9.3.2.1)	$\phi V_{n\_vf\_2} = 231.6 \cdot kip$
$\phi V_{n\_vf\_3} := 800 \cdot psi \cdot b_w \cdot d$	Shear friction capacity (ACI 11.9.3.2.1)	$\phi V_{n\_vf\_3} = 231.6 \cdot kip$
$\phi V_{n\_vf} := \min(\phi V_{n\_vf\_1}, \phi V_{n\_vf\_2}, \phi V_{n\_vf\_3})$	Shear friction capacity of concrete curb (ACI 11.9.3.2)	$\phi V_{n\_vf} = 23.76 \cdot kip$
$check_{Vfriction} := if(V_u \le \phi V_{n\_vf}, \text{"OK"}, \text{"INCREASE REINFORCING STEEL"})$		$check_{Vfriction} = \text{"OK"}$

IV. Check Flexure Capacity for Corbels:

$l_f := 1$	Factor for location of force from top of curb	$l_f = 1$
$a_f := \frac{(H_{curb} + cover + d_{bar}(rebar) \div 2)}{l_f}$	Distance from load to floor reinforcement	$a_f = 13.875 \cdot in$
$M_u := V_u \cdot a_f$	Factored moment (ACI 11.9.3.3)	$M_u = 10.5 \cdot kip \cdot ft$
$A_{s\_prov} := \frac{b_w}{sp_{bar}} \cdot A_{bar}(rebar)$	Amount of steel provided for flexure	$A_{s\_prov} = 0.44 \cdot in^2$
$\alpha_1 := 0.85$	Factor for equivalent rectangular stress block (ACI 10.2.7.1)	$\alpha_1 = 0.85$

$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000 \cdot psi \\ 0.65 & \text{if } f'_c > 8000 \cdot psi \\ 1.05 - 0.05 \cdot \dfrac{f'_c}{1000 \cdot psi} & \text{if } 4000 \cdot psi < f'_c \le 8000 \cdot psi \end{cases}$ Ratio of depth of equivalent rectangular stress block, a, to depth to neutral axis, c (ACI 10.2.7.3) $\beta_1 = 0.85$

$a := \dfrac{A_{s\_prov} \cdot f_y}{\alpha_1 \cdot f'_c \cdot b_w}$ Depth of equivalent rectangular stress block $a = 0.647 \cdot in$

$c := \dfrac{a}{\beta_1}$ Depth of neutral axis $c = 0.761 \cdot in$

$\varepsilon_t := \dfrac{\varepsilon_c \cdot (d - c)}{c}$ Net tensile strain in extreme-tension steel at nominal strength $\varepsilon_t = 0.092$

$\varepsilon_s := \dfrac{\varepsilon_c \cdot (d - c)}{c}$ Tensile strain in centroid of steel $\varepsilon_s = 0.09207$

$\varepsilon_y := \dfrac{f_y}{E_s}$ Yield strain in tension $\varepsilon_y = 0.002$

$check_{strain1} := \begin{cases} \text{"STEEL HAS NOT YIELDED!!! Reduce Reinforcing Steel"} & \text{if } \varepsilon_t < \varepsilon_y \\ \text{"Assumption that Steel has yielded is valid."} & \text{otherwise} \end{cases}$

$check_{strain1} =$ "Assumption that Steel has yielded is valid."

$check_{strain2} := \begin{cases} \text{"Tension Controlled Section"} & \text{if } \varepsilon_t \ge 0.005 \\ \text{"UPPER LIMIT NOT SATISFIED!!! Reduce Reinforcing Steel"} & \text{if } \varepsilon_t < 0.004 \end{cases}$

ACI - 10.3.5 sets a upper limit for steel at 0.004 for ε_t $check_{strain2} =$ "Tension Controlled Section"

$M_n := A_{s\_prov} \cdot f_y \cdot \left(d - \dfrac{a}{2}\right)$ Moment capacity $M_n = 52.4 \cdot kip \cdot ft$

$\phi M_n := \phi_f \cdot M_n$ Moment capacity with strength reduction factor $\phi M_n = 39.3 \cdot kip \cdot ft$

$check_{M\_pos} := if(M_u \le \phi M_n, \text{"OK"}, \text{"CHECK SECTION CAPACITY"})$ Check Moment Capacity $check_{M\_pos} = \text{"OK"}$

V. Check Tension and Shear in Reinforcing Bars and Epoxy Embedment:

$dist_{bars} := W_{curb} - 2 \cdot cover - d_{bar}(rebar)$ Distance between reinforcing bars resisting moment $dist_{bars} = 22.25 \cdot in$

$N_{u\_bar} := \frac{V_u \cdot H_{curb}}{dist_{bars}} \cdot \frac{sp_{bar}}{b_w}$ Tension in each bar $N_{u\_bar} = 4.92 \cdot kip$

$V_{u\_bar} := V_u \cdot \frac{sp_{bar}}{2 \cdot b_w}$ Shear in each bar $V_{u\_bar} = 4.56 \cdot kip$

Use Powers PE1000+, fpr #5 bars with 5" min embedment

$h_{ef} := 6 \cdot in$ Effective embedment depth

$\phi V_{cb} := 14505 \cdot lbf$ Shear capacity of each bar with epoxy (PE1000+)

$\phi N_{cb} := 17760 \cdot lbf$ Tension capacity of each bar with epoxy (PE1000+)

$h_{min} := h_{ef} + 2 \cdot d_{bar}(rebar)$ Minimum depth of concrete floor $h_{min} = 7.5 \cdot in$

$check_{depth} := if(H_{curb} \ge h_{min}, \text{"OK"}, \text{"INCREASE FLOOR THICKNESS"})$ $check_{depth} = \text{"OK"}$

VI. Check Tension and Shear in Reinforcing Bars Using Flexural Theory:

$a = \frac{A_{s\_req} \cdot f_y}{\alpha_1 \cdot f'_c \cdot b_w}$ Depth of equivalent rectangular stress block

$M_u = \phi_f \cdot A_{s\_req} \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ Equilibrium equation

$f(A_{s\_req}) := \phi_f \cdot A_{s\_req} \cdot f_y \cdot \left(d - \frac{A_{s\_req} \cdot f_y}{2 \cdot \alpha_1 \cdot f'_c \cdot b_w}\right) - M_u$

$A_{s\_req} := root\left(f(A_{s\_req}), A_{s\_req}, 0 \cdot in^2, 2 \cdot in^2\right)$ Amount of reinforcing steel required for moment $A_{s\_req} = 0.117 \cdot in^2$

$a := \frac{A_{s\_req} \cdot f_y}{\alpha_1 \cdot f'_c \cdot b_w}$ Depth of equivalent rectangular stress block $a = 0.172 \cdot in$

$c := \frac{a}{\beta_1}$ Depth of neutral axis $c = 0.202 \cdot in$

$\varepsilon_t := \frac{\varepsilon_c \cdot (d - c)}{c}$ Net tensile strain in extreme-tension steel at nominal strength $\varepsilon_t = 0.355$

$\varepsilon_s := \frac{\varepsilon_c \cdot (d - c)}{c}$ Tensile strain in centroid of steel $\varepsilon_s = 0.35483$

$check_{strain1} :=$ | "STEEL HAS NOT YIELDED!!! Reduce Reinforcing Steel" if $\varepsilon_t < \varepsilon_y$
"Assumption that Steel has yielded is valid." otherwise

$check_{strain1} =$ "Assumption that Steel has yielded is valid."

$check_{strain2} :=$ | "Tension Controlled Section" if $\varepsilon_t \geq 0.005$
"UPPER LIMIT NOT SATISFIED!!! Reduce Reinforcing Steel" if $\varepsilon_t < 0.004$

ACI - 10.3.5 sets a upper limit for steel at 0.004 for ε_t $check_{strain2} =$ "Tension Controlled Section"

$T := \phi_f \cdot A_{s\_req} \cdot f_y$ Tension in rebars $T = 5.26 \cdot kip$

$N_{u\_bar} := T \cdot \frac{sp_{bar}}{b_w}$ Tension in each bar $N_{u\_bar} = 5.26 \cdot kip$

$V_{u\_bar} := V_u \cdot \frac{sp_{bar}}{2 \cdot b_w}$ Shear in each bar $V_{u\_bar} = 4.56 \cdot kip$

$check_{shear} :=$ if$\left(V_{u_bar} \leq \phi V_{cb}, \text{"OK"}, \text{"DECREASE BAR SPACING"}\right)$ $check_{shear} =$ "OK"

$check_{ten} :=$ if$\left(N_{u_bar} \leq \phi N_{cb}, \text{"OK"}, \text{"DECREASE BAR SPACING"}\right)$ $check_{ten} =$ "OK"

$s_f := 1.0$ Factor to account for spacing of anchor $s_f = 1$

$et_f := 1.0$ Factor to account for edge distance of anchor in tension $et_f = 1$

$ev_f := 1.0$ Factor to account for edge distance of anchor in shear $ev_f = 1$

$combine := \frac{V_{u\_bar}}{\phi V_{cb} \cdot s_f \cdot ev_f} + \frac{N_{u\_bar}}{\phi N_{cb} \cdot s_f \cdot ev_f}$ Combine effect of shear and tension, ACI App. D7.3 $combine = 0.61$

$check_{combine} :=$ if$(combine \leq 1.2, \text{"OK"}, \text{"DECREASE BAR SPACING"})$ $check_{combine} =$ "OK"

VII. Calculate Hanger Bar Reinforcement:

$rcbar := 4$ Enter rebar size for hanger bar reinforcement

$A_{bar}(rebar) = 0.2 \cdot in^2$	Reinforcing bar area	
$d_{bar}(rebar) = 0.5 \cdot in$	Reinforcing bar diameter	
$s := 9 \cdot in$	Spacing of hanger reinforcing bar	
$S := 12 \cdot in$	Distribution of force for hanger reinforcing bar	
$A_v := \frac{V_u \cdot s}{\phi_v \cdot f_y \cdot S}$	Calculate shear reinforcement for hanger bar	$A_v = 0.152 \cdot in^2$
$A_{prov} := A_{bar}(rebar)$	Calculate provided area of reinforcement for hanger ba	$A_{prov} = 0.2 \cdot in^2$
$CheckA_{vh} := if(A_v \le A_{prov}, \text{"OK"}, \text{"Decrease Spacing"})$		$CheckA_{vh} = \text{"OK"}$

VIII. Check Embed Depth for Epoxy for Hanger Bar:

$V_u = 9114.479 \, lbf$	Shear force that must be resisted by hanger bar	$V_u = 9114.479 \, lbf$
$V_{hb} := V_u \cdot \frac{s}{S}$	Shear force that each hanger bar must carry	$V_{hb} = 6835.859 \, lbf$
$h_{efhb} := 4 \cdot in$	Effective embedment depth	
$\phi V_{cbhb} := 6550 \cdot lbf$	Shear capacity of each bar with epoxy (PE1000+)	
$\phi N_{cbhb} := 8735 \cdot lbf$	Tension capacity of each bar with epoxy (PE1000+)	
$s_f := 1.0$	Factor to account for spacing of anchor	$s_f = 1$
$et_f := 1.0$	Factor to account for edge distance of anchor in tension	$et_f = 1$
$ev_f := 1.0$	Factor to account for edge distance of anchor in shea	$ev_f = 1$

$\phi N_{hb} := \phi N_{cbhb} \cdot s_f$	Total available tension force in anchor	$\phi N_{hb} = 8735\,\text{lbf}$
$\text{Check}\phi N_{hb} := \text{if}\left(\phi N_{hb} \ge V_{hb}, \text{"OK"}, \text{"Decrease Bar Spacing"}\right)$	Check tension capacity	$\text{Check}\phi N_{hb} = \text{"OK"}$

General

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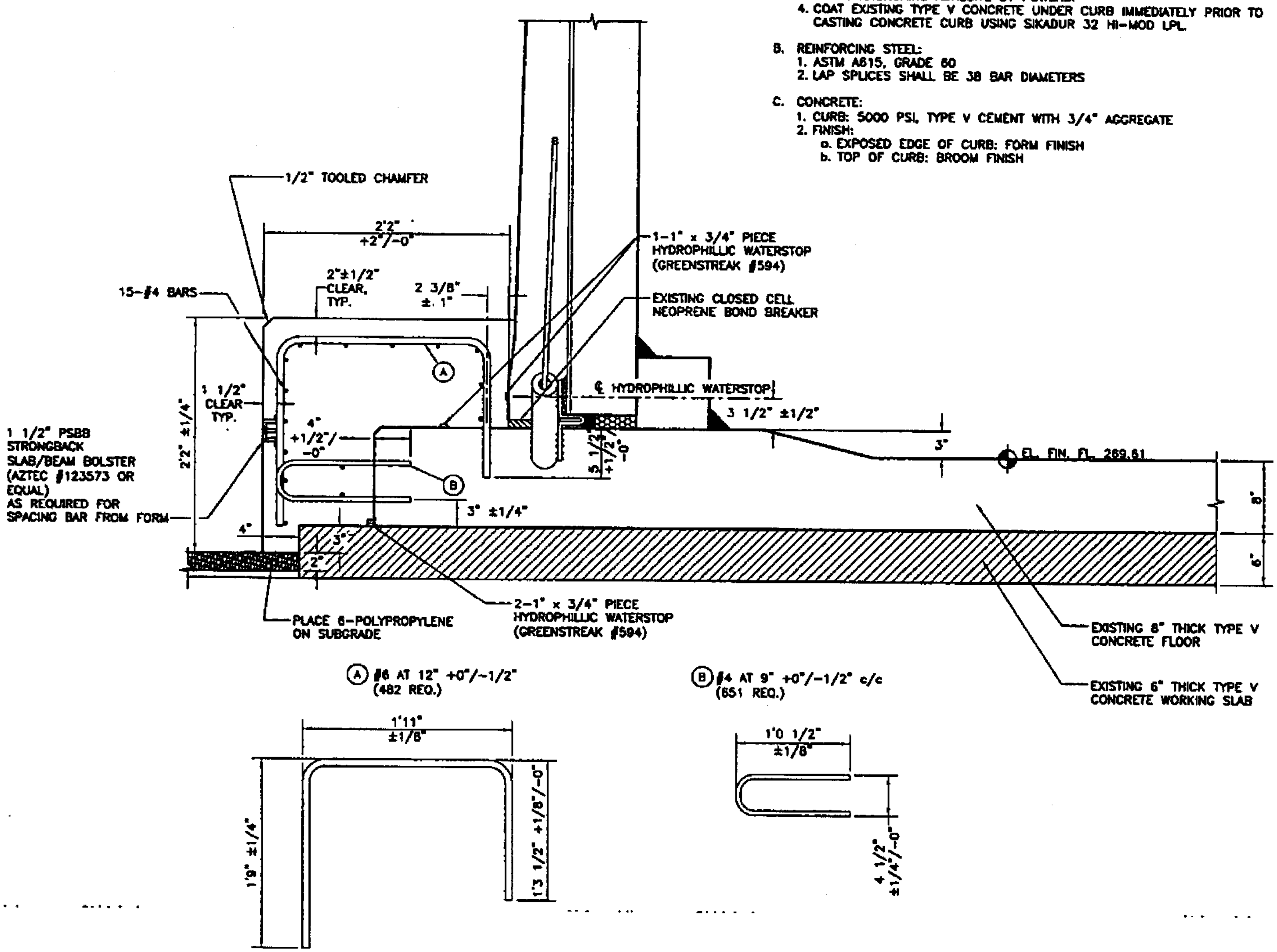
NOTES:

A. MISCELLENOUS:
1. TRIM EXISTING CLOSED CELL NEOPRENE PAD FLUSH WITH SHOTCRETE TANK WALL.
2. CLEAN CONCRETE SURFACES.
3. SET DOWELS INTO EXISTING TYPE V CONCRETE USING POWERS PE1000+ EPOXY ANCHORING ADHESIVE BY POWERS.
4. COAT EXISTING TYPE V CONCRETE UNDER CURB IMMEDIATELY PRIOR TO CASTING CONCRETE CURB USING SIKADUR 32 HI-MOD LPL.

B. REINFORCING STEEL:
1. ASTM A615, GRADE 60
2. LAP SPLICES SHALL BE 38 BAR DIAMETERS

C. CONCRETE:
1. CURB: 5000 PSI, TYPE V CEMENT WITH 3/4" AGGREGATE
2. FINISH:
a. EXPOSED EDGE OF CURB: FORM FINISH
b. TOP OF CURB: BROOM FINISH



CROM

K. Ryan Harvey, PE
South Carolina License Number: 24325
THE CROM CORPORATION
250 SW 36th Terrace
Gainesville, Florida 32607
Certificate of Authorization Number: C00656

OWNER: DEPT. OF ENERGY SAVANNAH RIVER SITE
AIKEN, SOUTH CAROLINA

CONSULTING ENGINEER: BECHTEL SAVANNAH RIVER, INC.
AIKEN, SOUTH CAROLINA

TANK DESCRIPTION: 2,900,000-GALLON SALTSTONE STORAGE TANKS

TANK DIMENSIONS: 150'0" ID x 22'0" SWD

DATE: 9/17/2010
DRAWN: JAT
CHECKED: KRH
APPROVED: LOB
DESIGNED: KRH

REV.	DESCRIPTION	DATE CK.BY

SOUTH CAROLINA THE CROM CORPORATION No. C00656 CERTIFICATE OF AUTHORIZATION

BAR IS ONE INCH ON ORIGINAL DRAWINGS 0 1"

SCALE: 0 1' 1 1/2"=1'0"

FILE NO. 2008-M-084B

SHEET 1 OF 1

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Appendix 2

Saltstone Disposal Unit 2 Performance Testing
C-TRT-Z-00001
Revision 0

Saltstone Disposal Unit 2 Performance Testing

C-TRT-Z-00001, Revision 0

November 10, 2010

Approver	Signature	Date
W. C. Miles, Jr. Originator		11/10/2010
M. E. Maryak Technical Reviewer		11/10/2010
L. K. Sonnenberg DWPF/Saltstone Facility Manager	L K Sonnenberg	11/10/10
C. J. Winkler SRR Chief Engineer		11/10/10

LIST OF REVISIONS

REVISION	DATE	DESCRIPTION OF CHANGE
0	11/10/2010	Initial Issue

TABLE OF CONTENTS

KEY WORDS

DSS	Decontaminated Salt Solution
FEA	Finite Element Analysis
MCU	Modular Caustic-Side Solvent Extraction Unit
MDM	Primary Contractor responsible for construction of SDU 2
PA	Performance Assessment
SCDHEC	South Carolina Department of Health and Environmental Control
SDC	Saltstone Disposal Cell – Cell refers to each single tank
SDU	Saltstone Disposal Unit – Unit refers to each set of 2 cells, two SDCs form a single SDU
SRR	Savannah River Remediation
TR&C	Task Requirements and Criteria

ABSTRACT

Saltstone Disposal Cells (SDCs) 2A and 2B underwent a water tightness test with each cell filled to 22 feet per the requirements of the procurement specification (Reference 4.1) and in accordance with the approved subcontractor procedures (Reference 4.3 and 4.4). Although the SDC roof and walls performed as expected (no moisture was observed), damp concrete was observed at the wall to floor joint and at the floor to upper mud mat interface. Post testing inspections of the SDCs concluded that the moisture pathways for dampness are attributed to coating failures along the interior curb and coating failures around drainwater collection system anchor bolts.

The strategy adopted to address potential liquid pathways to the exterior of the vaults and demonstrate SDC integrity and to accomplish the main testing objectives of a water tight SDC is to cut off anchor bolts flush with the tank floor, apply epoxy sealant patch at those locations, apply a second flexible coating to the interior surfaces, and install an exterior curb system.

The SDCs will be retested with all repairs and modifications completed thereby demonstrating the best-engineered solution for water tightness. Re-test verifies that repairs to the coatings and all other design enhancements result in a water tight SDC. Testing will be performed at a water fill height permitted by the Finite Element Analysis of the vault without earth backfill to ensure the tank concrete curb and wall interface is not overstressed. This technical report describes the basis for concluding the alternative test adequately demonstrates the structure's ability to meet the water tight design criteria.

1.0 Introduction

1.1 Purpose and Objective

The report describes the Saltstone Disposal Unit (SDU) Cells 2A and 2B repairs and modifications that have been made to meet the expectations of water tight performance and provides the technical justification for the subsequent water tightness re-test. Ultimately, the objective of the re-test is to demonstrate that the repairs and modifications adequately address the moisture observed during initial water tightness tests and to meet Department of Energy (DOE) expectations, the SCDHEC expectations (Reference 4.2) and Performance Assessment (Reference 4.7) base case assumptions. Test acceptance criteria are specified in the Technical Requirements and Criteria (TR&C) document (Reference 4.18) which is the upper tier technical basis document.

1.2 Background

SDU 2 consists of two (2) cylindrical 2.9 million gallon (nominal working volume) concrete cells that are 150 feet in diameter by 22 feet in height. Basic construction consists of a Class III sulfate resistant concrete floor slab, upper mud mat, and pre-stressed Class III sulfate resistant concrete walls. The main components of the pre-stressed walls are pre-cast panels of Class III sulfate resistant concrete mix, a steel shell diaphragm, reinforcing bars, pre-stressing wires, and Type II shotcrete fill. High strength prestressing wires are installed around the disposal unit and are designed to carry the hydraulic load and to maintain the walls in compression. Shotcrete is used to protect the reinforcing bars and prestressing wires. The reinforcing steel, pre-stressing wire, and steel shell diaphragm are all of carbon steel construction. An epoxy-novolac internal coating system is installed to ensure the Performance Assessment (PA) performance objective to prevent drain water contaminants from penetrating the cell concrete during operations and to mitigate sulfate attack from short-term saltstone bleed water penetration (through surface cracks and by capillary suction). As a secondary benefit, the coating system provides an additional degree of water tightness. The coating system coupled with the low permeability of the Type V concrete ensures base case PA models assumptions are not challenged.

These Saltstone Disposal Cells, SDCs, will be used for the disposition of Decontaminated Salt Solution (DSS) that has been processed by the Modular Caustic-Side Solvent Extraction Unit (MCU). The DSS is mixed with dry materials to form a flowable grout. Liquid in the SDCs originating from grout curing and process flush sequences, i.e., drainwater, is collected in a drainwater collection system that is installed around the interior circumference of the cell floor. The drainwater is pumped out of the cells back to the Salt Feed Tank based on requirements implemented in operating procedures.

There were two primary SDU 2 design objectives: 1) provide a disposal unit that meets SCDHEC regulatory requirements during radioactive operations and 2) provide a design that supports the PA analysis that characterizes cell performance over a 10,000 year cell

life. The TR&C design requirements for water tightness are as follows (excerpts from document):

DC.3.1.8.5 Vault cells shall be designed and constructed to be leak tight including roof/wall, wall/wall, and wall/floor joints. Hydrostatic testing of the tank is required. Water tightness shall be demonstrated using a dye or tracer having no evidence of dye or tracer external to the cell.

DC.3.2.1.12 Vault (tank) joint seals, walls, floors, and roof shall prevent liquid seepage to the exterior of the vault and prevent intrusion of rainwater. Vault structure shall be able to withstand hydrostatic head created by internal liquids. The specific gravity of the salt solution is 1.3.

Implementation of these requirements for the SDU design is based on American Waterworks Association (AWWA) D110 criteria and methodologies (Reference 4.9), but the expectation for water tight performance exceeds that prescribed in the AWWA standard. Whereas, the water tightness per the AWWA standard allows for the existence of damp spots on the cell footer and evidence of moisture that is not transferable, the TR&C requires successful completion of a dye test and no liquid seepage. The SDU water tightness test criteria and results of initial testing are provided below:

Test A/C from TR&C	Initial Test Results
DC.3.1.8.5 - Vault cells shall be designed and constructed to be leak tight including roof/wall, wall/wall, wall/floor joints. Hydrostatic testing of the tank is required. Water tightness shall be demonstrated using a dye or tracer having no evidence of dye or tracer external to the cell.	Neither damp spots nor traces of dye were observed in the roof/wall interface nor the wall/wall interfaces. Only damp spots and dye were at the wall/floor joints. *The roof/wall and wall/wall interfaces successfully passed the test with dye. The wall/floor joints require rework.*
DC.3.2.1.12 - Vault (tank) joint seals, walls, floors, and roof shall prevent liquid seepage to the exterior of the vault and prevent intrusion of rainwater. Vault structure shall be able to withstand hydrostatic head created by internal liquids. The specific gravity of the salt solution is 1.3.	*Liquid seepage, as defined by no measurable leakage of water, was not observed in the roof, walls or floor. The water test successfully passed the no liquid seepage test.*

Initial SDC 2B water tightness testing was performed without a dye tracer. The SDC Cells 2A and 2B water tightness testing was completed and the SDCs met DC.3.2.1.12 as no measurable liquid was detected seeping from the cells. The tests did successfully demonstrate DC.3.1.8.5 for the roof/wall and wall/wall interfaces as no damp spots were ever observed at these locations. Damp spots were observed in several locations at the wall to floor joint and at the floor to mud mat interface.

Inherent to the exterior wall design at the wall and floor interface are small cavities where different materials of construction meet (e.g., outer surface of waterstop, seismic cables, neoprene pad, and shotcrete, See Figure 1). These cavities can trap minor amounts of water (a.k.a., construction water) from concrete curing and rainwater. The tank designer neither previously identified nor communicated to SRR a potential of having water other than that originating from the interior of the tank during testing.

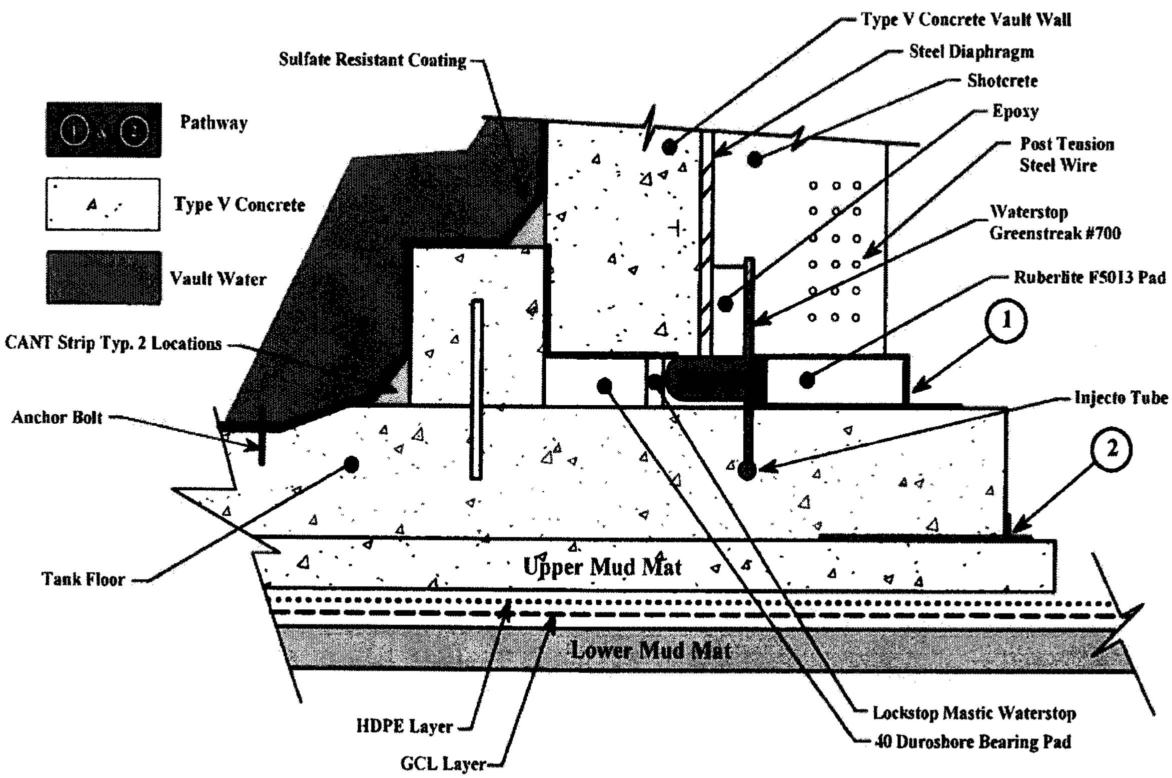

Figure 1 Observed Leak Sites and Postulated Pathways

Eventually, water from the environment or from construction activities would evaporate given a long enough waiting time. Test water would continue to recharge and remain evident. As an alternative to a long wait time, the requirement to use dye tracers was added to the TR&C and the water tightness test procedures were revised to include dye tracers to provide some discriminator as to the origin of any observed moisture exterior to the tank containment system.

Subsequent re-tests of SDCs 2A and 2B with a dye tracer confirmed moisture exhibited by damp spots and presence of dye in several locations at the wall to floor joint and at the floor to mud mat interface (See Figure 1 points 1 and 2). Neither damp spots nor traces of dye were ever observed in the roof/wall interface nor the wall/wall interfaces. Interior SDC inspections revealed coating failures, primarily in the floor, that provided a pathway for the moisture to interact with the cell concrete. Although coating and floor repairs were made to the SDCs, upon re-test, SDC 2B again failed to meet the dye tracer test

acceptance criteria. Upon re-inspection, the quality of the coating repairs was again determined to be inadequate in that additional coating failures were discovered during post test inspections and further analysis and reviews determined that epoxy delamination due to poor installation was a likely contributor.

Given the unsatisfactory results of the second SDC 2B water tightness test at the wall to floor joint and at the floor to mud mat interface, the project team placed the re-test of SDC 2A on hold and initiated an engineering path forward (Reference 4.11) to identify a repair strategy that would bring the SDCs into compliance with the TR&C requirements. Based on the exterior observations and the internal inspections, the locations of moisture pathways were determined to be through the anchor bolts in the floor and around the interior curb.

Anchor bolt installation requires a minimum engagement depth (nominally 3 inches). Installation procedures required the use of drill stops to ensure hole depths of three (3) inches were not exceeded. The design also requires a minimum of 3 inch cover for all rebar. Although no rebar was cut during the installation process, a leak path through these holes to a rebar layer could create a tortuous path to the exterior. This designed penetration provides a theoretical moisture/leak path to the exterior of the tank. Complete removal of the anchoring material was not practical and thus an alternate and more comprehensive solution, as is discussed below, addresses proper sealing around the anchor locations to prevent recurrence.

Likewise, the area around the interior curb could also provide a theoretical moisture/leak path. After wall assembly, a compressive loading is introduced at all wall closure joints through layers of tension wire that are successively spiral wound around the exterior of the tank in intermediate and alternating layers of shotcrete. Once complete the compressed wall panels reach their maximum radial inward position and remain under compression when the tank is filled. An interior cast in place curb having direct interface with the lower 8 inches of the interior wall panel limits inward movement of the wall. Any outward wall movement is addressed using resilient material (Figure 1 CANT strip) and bond breaker applied to the Cant strips at the upper curb to wall interface and again at the curb to floor interface. This resilient material is applied at the upper and lower interfaces and is designed to preclude leak pathways by bridging any small gaps formed as a result of wall panel movement with a water tight material. A 45° chamfer of this material provides a smooth transition from horizontal across the curb to vertical up the wall. This allows the original top coating flexibility to distort rather than fracture. The bond breaker strips ensure that the TL-45S Novolac epoxy adheres only where designed to allow free movement of the coating.

2.0 Test Resolution

The following repairs and modifications are being pursued to resolve the SDU 2 water tightness issues in preparation for the next water tightness test (Figure 2):

- Cut the floor drainwater collection system anchor bolts flush with the floor and repair/overlay the areas using the original TL-45S Novolac epoxy to form a continuous cell floor lining. (Reference 4.5). To eliminate the potential for damage to the concrete floor during the repair, anchors will be cut and ground flush with the floor. No rotary hammers or impact equipment will be used to remove the anchors to guard against crack development.
- Apply a second flexible topical coating (Blome EC-66) to the entire cell interior including floor (over the sealed anchor bolts), interior curb, and to a full height level of approximately 22 feet up the wall to form a secondary flexible water tight lining. (Reference 4.5)
- Install a washer, gasket and nut mechanical sealing system to each of the drainwater collection system anchor bolts that remain in the walls for sheet drain attachment (Reference 4.6). This mechanical system provides a positive and verifiable seal that addresses any potential sealing discontinuities at the interface between the threads of the anchor and the wall coating/lining system.
- Install an exterior curb system with 8 inch (minimum) Type V concrete that addresses the areas along the wall to floor interface and at the floor to upper mud mat interface. The objective of this curb is to restrain wall movement that might affect the interior coatings and to provide an additional moisture barrier of Type V concrete that supports the Performance Assessment base case assumptions. (Reference 4.7)

The integrated system provides multiple layers of defense between the waste form and the environment. The repairs and modifications provide a high level of confidence that the SDCs 2A and 2B will pass test acceptance criteria of no dye observed for wall/floor joints considering the results of the initial tests and how little moisture was observed over previous tests.

The exterior curb will prevent wall movement at the floor level during filling operations. This will help ensure that the interior curb to wall interface coatings remain intact by minimizing stresses on the coating system, thereby precluding a possible leak path in high stress areas. With regard to the PA base case assumptions, the exterior curbs provide an additional 8 inches of Type V concrete beyond the PA base case assumption requirement for the hydraulic diffusion path through cementitious material.

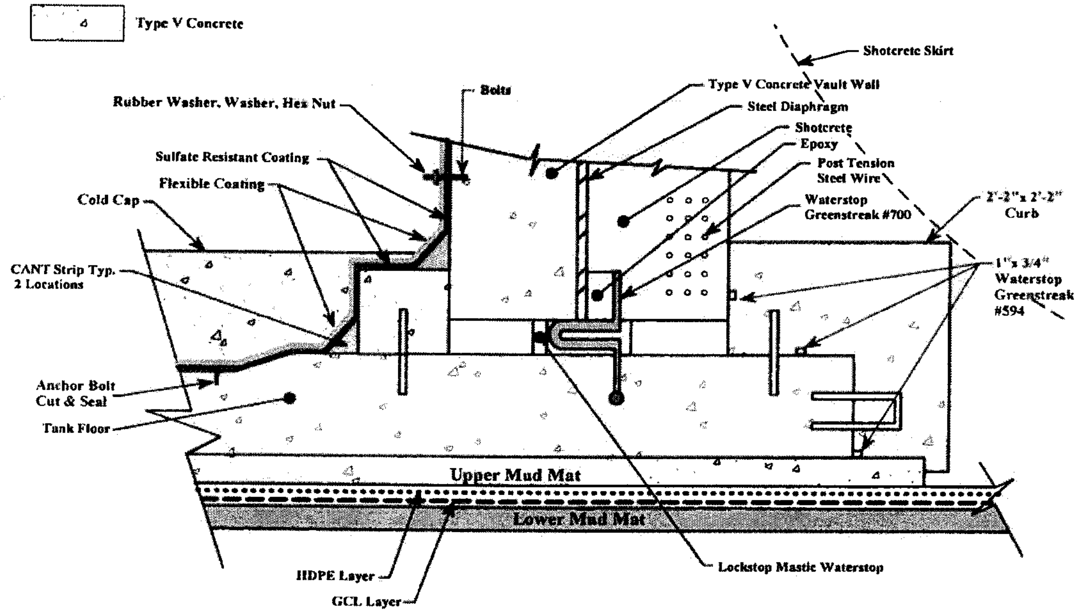

Figure 2 SDU 2 Design Enhancements (red font)

While the implementation of this integrated system provides confidence in meeting the goal of preventing any waste material from reaching the environment during filling operations, the installation of the curb at the wall to floor interface impacts re-test fill height. The installation of the exterior curb at the wall to floor interface restrains radial movement of the SDC wall. The stresses in the wall and the curb increase as the liquid height in the cell increases. Crom, the SDC designer, performed a Finite Element Analysis (FEA) (Reference 4.8) of the wall under filling conditions without backfill in place and has determined that a maximum fill height of 12 feet must be invoked due to potential to fail the top of the curb. Increasing fill height beyond the 12 foot level would potentially impact the wall stresses causing them to exceed code allowable stresses. The FEA further confirms that after backfill, wall and curb stresses are acceptable for a liquid with a SpG of 1.3 up to a fill height of 20 feet 7 inches.

The floor wall interface is the area of interest for the re-test. Flow through cracks simplistically follows the Bernoulli equation

$$Q = ac(2gh)^{1/2}$$

where the flow is proportional to the square root of the head (h).

Definitions:

Q = Flow
a = Area
c = Conductivity
g = Gravity

The head of 12 feet of water is sufficient to produce a driving head that will force any hydraulic communication from the inside to the outside of the structure. Testing at 12 feet instead of the full 22 feet reduces the driving head and reduces the potential flow by 36 percent for the given test duration. Therefore, the re-test duration will be lengthened to a minimum of 98 hours to provide sufficient time to equate to a driving head of 22 feet.

SRR has evaluated if the cell could be tested with 22 feet of water before adding the exterior curb with the following conclusions. Figure 3 compares the wall deflection estimated for the original full height leak tests with most recent FEA deflections values determined with external curb in place and lower wall restrained. Tank wall deflection during the original full height water tightness test was estimated to be 0.44 inches at the base. The re-test conditions, no backfill and a test height of 12 ft of water, result in a wall deflection at mid height calculated to be 0.016 inches (~1/64"). The exterior curb lowers the risk of damage to the interior coating at high stress areas around the interior curb by limiting wall movement. Limiting wall movement ensures the coating system remains intact by preventing excessive tensile elongation as discussed below. All modifications and repairs ensure water tightness and PA performance, thus, an integrated test best simulates the actual conditions under which the tanks will be operating. The most representative configuration is one after repairs to the SDCs are complete with the external curbs installed. Therefore, the test should reflect the design service configuration that provides the highest confidence of success without risk to the new repairs and modifications.

The additional 60 mil thick application of Blome International EC-66 high performance flexible epoxy coating placed over the entire interior surface of the tank further ensures water tightness, in concert with the original sulfate resistant coating. The manufacturer specifies a tensile elongation of 110% for the EC-66 (Reference 4.20) and 2% for the Novolac epoxy (Reference 4.21). The maximum wall deflection with active soil pressure and the cell filled to 22 feet with liquid of 1.3 SpG occurs approximately at mid height and is 0.073 inch. The maximum wall deflection of 0.073 inches results in a deflection at the curb-wall interface of 0.0075 inch which translates into a liner elongation of 0.75 %. This elongation is less than the rated elongation of the Novolac and EC-66 materials and will not challenge the integrity of the installed coatings (Reference 4.19). Resulting displacements calculated based on rigid body rotation mythology are conservative when compared to actual behavior of the tank. Actual wall behavior will result in shear and flexural rotation and not rigid body rotation.

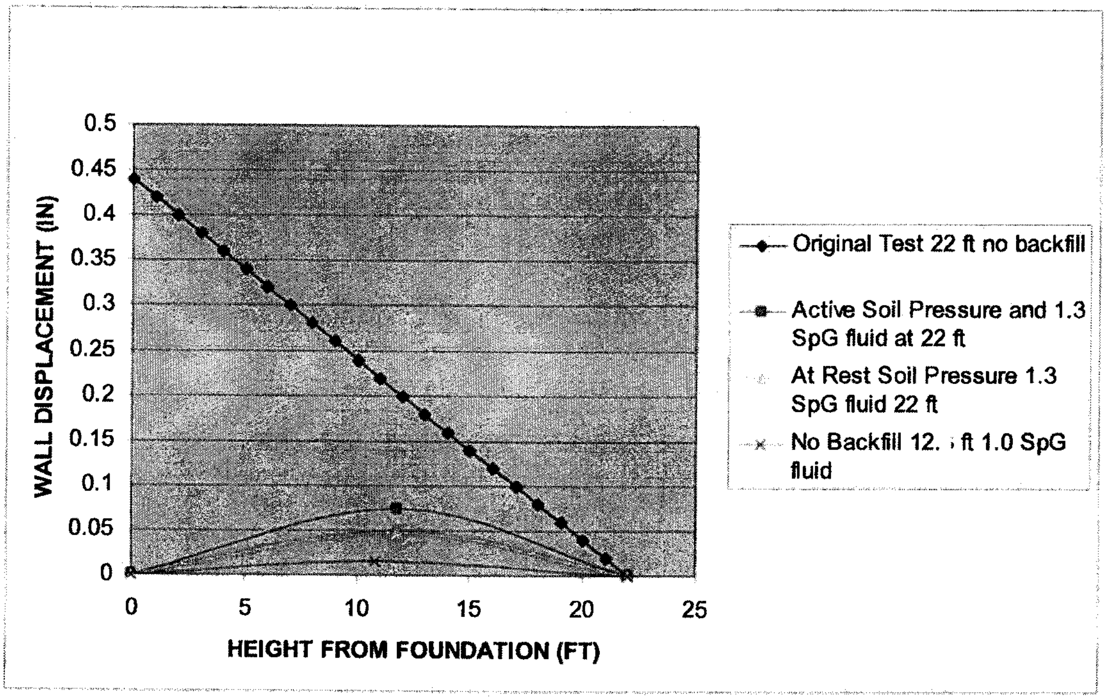

Figure 3 Tank Wall Displacement During Water Tightness Tests

Subsequent to repairs, modifications, and design enhancements, re-testing is required to demonstrate that the wall to floor joint meets acceptance criteria. Retest will be performed such that stresses placed on SDC components beyond design stresses are avoided. The technical justification that the 12 foot test height above the finished floor elevation is adequate to demonstrate SDU integrity is determined as follows:

1. Initial tests demonstrated that the SDCs met the TR&C water tightness criteria DC.3.2.1.12 for the wall panel, floor, and roof joint seals to prevent liquid seepage to the exterior of the vault. The modifications made to add the flexible membrane and exterior curb will not change the initial test results for the wall and roof joints and the original test remains valid.
2. Initial tests demonstrated that the SDCs met the TR&C water tightness criteria DC.3.1.8.5 for the roof/wall and wall/wall interfaces. Only the wall/floor interface failed the initial test.
3. Once backfill has been placed around the SDCs, the FEA shows that the SDCs can be filled with liquid of 1.3 SpG to a height of 20 feet 7 inches.

The SDU has been evaluated to several loading conditions that produce a range of deflections. All resulting deflections at the internal curb/wall interface do not challenge the integrity of the two liners. The maximum deflection determined was based on lateral soil pressure using an active soil condition which is K=0.33. A more realistic condition is a soil pressure at rest which is a K=0.5 which results in a smaller displacement.

Although the SDUs could be tested to full height after backfill, this approach is not recommended due to inability to visually confirm water tightness acceptance criterion at all locations. The wall to floor joint and floor to upper mud mat interface are inaccessible after backfill, therefore, visual inspections for water tightness would be unavailable and a non visual leak detection system would be required.

Finally, as successive layers of grout are poured, consideration was given to the possibility that a condition could exist where a "trapped" hydraulic head (uninterrupted 22 foot vertical slice of water) could develop away from the walls. This condition is not considered a concern due to the flow-ability of the grout. Each successive grout pour is poured upon the previous semi-cured layer. If the un-reinforced semi-cured grout (concrete) forms a crack, the new layer of flowing grout will hydraulically seek the lowest level thereby filling crack, voids, and inclusion created during the curing process. In addition, hydraulic head is not an issue away from the walls as the floor is designed to withstand the full load of the cured grout (SpG = 1.8). During SDC operation, flush and bleed water will be collected by the drainwater system and pumped back to the SFT as described earlier.

3.0 Conclusion

The integrated system of repairs and modifications to SDCs 2A and 2B provides confidence that the SDCs will meet the test acceptance criteria (i.e., no traces of dye on the exterior surfaces). The causes of leaks in past tests are understood and have been mitigated. The design provides additional moisture barriers to help meet water tightness requirements and the requirements of the PA base case assumptions. The exterior curb decreases the risk of interior coating failures at high stress areas around the interior curb. The initial 22 foot water tests for SDCs 2A and 2B previously demonstrated both structural integrity and water tightness at full height. The re-test will be performed with all modifications completed at a fill height of 12 feet which adequately tests the only areas that failed initial testing. Testing at a liquid height of approximately 12 feet without backfill prevents damage to the concrete curb. The proposed test criteria at this height (12 feet) with minimum test duration of 98 hours, adequately demonstrates water tightness in the backfill condition with 20 feet 7 inches of operating fluid.

4.0 References

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4.2 WSP-SSF-2005-00023, Revision 2, Z-Area Industrial Solid Waste Landfill Permit #025500-1603, Engineering Report for Vault 2 Construction
4.3 WB00001K-041-B-MDM, Vault Water Tightness Test Procedure
4.4 WB00001K-046-C-MDM, Red Dye Injection Procedure
4.5 C-DCF-Z-000152, Revision 0, Elimination of Potential Leak Paths for Vault 2A and 2B
4.6 C-DCF-Z-00154, Revision 0, Revision of Vault 2 Wall Anchors for Drainwater Collection System

4.7 C-DCF-Z-00153, Revision 0, The Addition of Two Concrete Curbs Around Vault 2 Perimeter

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4.13 J-JZ-Z-00112, Revision 0, Saltstone Scaling and Setpoint Document Z210-ZZZL-2200 Vault 2B Drainwater Level

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4.15 Tank 2A Final 72 Hour Blacklight Exam, 2010-IR15-WB00001K-0007, 07/08/10

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4.19 C-CLC-Z-00050, SDU 2A 2B Rigid Body Rotation Evaluation of Epoxy Coating at the Interior Curb

4.20 WB00001K-053-A-MDM, Tank Surface Prep, Coating, and Inspection Procedures

4.21 WSRC-TR-2008-00090, Rev. 0, SALTSTONE VAULT #2 INTERIOR LINING REVIEW (U), T. E. Skidmore, K. D. Billings

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K.D. Harp, SRR, 766-H, Rm. 2010
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V.F. Franklin, SRR, 705-1C, Rm. 7
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N.L. Bethurem, SRR, 705-1C, Rm. 6
J.E. Hyatt, SRNS, 705-3C, Rm. 134
E. Saldivar Jr, SRNS, 705-3C, Rm. 106

File Info: SCDHEC, Saltstone
10666, DOE/ADM
16-1.5(a) Permanent