ENCLOSURE 2

CALCULATION RSOFLHROGCDX00033320090003
"PMF TEMPORARY MODIFICATION ANALYSIS"

TVA Calculation Package

Title PMF Temporary Modification Analysis

Location Description: *(Optional)*	**Total Pages:** *(including appendices & attachments)* 192

Calculation ID *(All parts required to form a unique ID)*:

Org Code	Location/ Plant Code	Branch Code	Alphanumeric Part = Discipline Code (1) + Type Code (1) + "X" + Unit Field (3) + Sys Code (3) + Year (4) + Sequence No. (4)
RSO	FLH	ROG	CDX00033320090003

NOTE: When referencing the calculation ID, include all parts without spaces or dashes between them.

Unit(s), Spill gate(s), or Voltages (PSO): Unit 0	**Key Nouns (For CTS/CCRIS):** Stability Analysis, Temporary,Modification, PMF	
Applicable Design Document(s): None	**Rev**	**RIMS/EDMS Accession Number** *(Optional)*
	R0	
	R	
UNID System(s): 333	**R**	
	R	

	R0	R	R	R
DCN, PCN, NA	FLH-09-56			
Prepared: Sign→	[signature]			
Print Name	J. Justin Long			
Checked: Sign→	Coleman E. Hoskins [signature]			
Print Name	Coleman E. Hoskins			
	These calculations contain unverified assumption(s) that must be verified later? ☐ Yes ☐ No These calculations contain special requirements and/or limiting conditions? ☐ Yes ☐ No			
Approved: Sign→	[signature]			
Print Name	Kendal R. Lennon			
Approval Date	11/13/09			
	These calculations contain a design output attachment? ☐ Yes ☒ No			
Revision Applicability	☒ Entire calc	☐ Entire calc ☐ Selected pgs	☐ Entire calc ☐ Selected pgs	☐ Entire calc ☐ Selected pgs
	Computer output Microfiche generated? ☐ Yes ☒ No Number: ______			

Purpose of the Calculation: To evaluate the stability of Hesco Bastion Concertainers for use as a temporary flood wall to raise FLH to meet the revised PMF evaluation.

Abstract: Hesco Bastion Concertainers will be used to temporarily raise the elevation of the top of the dam while a permanent modification can be implemented. Several configurations of the Concertainers were evaluated including utilizing existing flood walls to support the temporary flood wall.This calculation also documents the applicability of the Concertainer flood wall system to CRH, TEH, and WBH.

☒ Electronically file and return calculation to Calculation Library.

☐ Electronically file and return calculation ______ Address: ______

TVA Calculation Coversheet
CTS Input Form

Preparer	Preparer Login ID	Date
J. Justin Long	jjlong	11/2/2009
Checker	**Checker Login ID**	**Date**
Coleman E. Hoskins	cehoskins	11/13/2009

Update Code: ☒ Add ☐ Change ☐ Delete ☐ Rename ☐ Supersede ☐ Duplicate ☐ Verify

The following section applies if a calculation is being renamed, superseded, or has a duplicate.

	Org Code	Plant	Branch	Number	Cur Rev	New Rev
Current Calc ID:						

The following section applies to all calculations.

Calc ID:	RSO	FLH	ROG	CDX00033320090003		0

Firm: *(TVA or Contractor)* ______________________________

Cross-References

A/C/D	Xref Code	Type	Org Code	Plant	Branch	Number	Rev
A	P	DW	RSO	FLH	ROG	10W222-1	0
A	P	DW	RSO	FLH	ROG	10W222-2	0
A	P	DW	RSO	FLH	ROG	10W222-3	0
A	P	DW	RSO	CRH	ROG	10W222-1	0
A	P	DW	RSO	CRH	ROG	10W222-2	0
A	P	DW	RSO	TEH	ROG	10W222-1	0
A	P	DW	RSO	TEH	ROG	10W222-2	0
A	P	DW	RSO	WBH	ROG	10W222-1	0
A	S	CN	RSO	WBH	ROG	CDX00033320090006	0

TVA Calculation Record of Revision

Calculation Identifier: RSOFLHROGCDX00033320090003

Title PMF Temporary Modification Analysis

Revision No.	Description of Revision
0	Initial Issue

TVA Computer File Storage Information Sheet

Calculation Identifier: RSOFLHROGCDX00033320090003 **Rev.** 0 **Plant:** FLH

Subject: PMF Temporary Modification Analysis

Software Name: NA **Revision Level:** NA

Vendor Name: NA

Address: NA

Executable Files

☒ No TVA developed **executable files** were used in this calculation.

Comments:

☐ TVA developed **executable files** used in this calculation have been stored electronically and sufficient identifying information is provided below for each executable file. *(Any retrieved file requires re-verification of its contents before use.)*

Input Files

☒ Electronic storage of the **input files** for this calculation is not required.

Comments:

☐ **Input files** for this calculation have been stored electronically and sufficient identifying information is provided below for each input file. *(Any retrieved file requires re-verification of its contents before use.)*

TVA Calculation Table of Contents

Calculation Identifier: RSOFLHROGCDX00033320090003 **Revision:** 0

Table of Contents

1.0 Introduction and Purpose

The HESCO Bastion Concertainers, hereinafter referred to as Concentainers, will be used as a flood wall to temporarily raise the elevation of Fort Loudoun, Tellico, Cherokee, and Watts Bar Dams to meet the impoundment requirements during the Probable Maximum Flood (PMF) event. Concertainers are a wire basket measuring 3-feet wide by 15-feet long by either 3 or 4-feet deep filled with sand or other fill material. Each 15-foot unit is divided into 5 equal compartments, each lined with a polypropylene nonwoven geotextile liner A review of product literature has been performed and shows that the Concertainers have been successfully deployed as flood walls at multiple locations in the United States. The body calculation will evaluate the sliding and overturning stability of the Concertainers under a hydrostatic loading for both typical Concertainer configurations and the atypical Concertainer configurations unique to Fort Loudoun Dam. Attachment 8 addresses the applicability of this calculation to the other three projects.

2.0 References

2.1 "Engineering Evaluation of Hesco Barriers Performance in Fargo, ND 2009." Wenck Associates, Inc., May 2009 (Wenck File #2283-01).

2.2 "Flood-Fighting Structures Demonstration and Evaluation Program: Laboratory and Field Testing in Vicksburg, Mississippi." US Army Corps of Engineers: Engineering Research and Development Center, ERDC TR-07-3, July 2007.

2.3 Ward, Don. "Amendment A: Re-Test of HESCO Bastion." US Army Corps of Engineers: Engineering Research and Development Center - Coastal & Hydraulics Laboratory, October 25, 2005.

2.4 "ACI 360.R-92: Design of Slabs on Grade." American Concrete Institute

2.5 TVA Calculation "Modify Dams - Dam Safety." Fort Loudoun & Tellico. RIMS Accession Number B66 88 0122 102.

2.6 "Engineering and Design: Stability Analysis of Concrete Structures." EM 1110-2-2100. United States Army Corps of Engineers. December 1, 2005.

2.7 Terzaghi, Karl et. al.,"Soil Mechanics in Engineering Practice," 3rd ed., John Wiley and Sons, New York, NY, 1996.

3.0 Assumptions

There are no unverified assumptions.

4.0 Literature Review

A review of product literature was performed to determine the applicability of the Concertainer system as a flood wall to raise the top elevation of the dams. The review included an engineering evaluation performed by Wenck Associates, Inc. at the request of Hesco Bastion and a USACE Engineering Research and Development Center study as directed by the US Congress in the 2004 Energy and Water Development Bill.

4.1 Wenck Associates, Inc.

As discussed in Ref. 2.1, the Concertainer system was deployed in Fargo, ND, as a flood wall along the Red River of the North. This engineerng evaluation was performed by Wenck Associates at the request Hesco Bastion in response to comments that surfaced following the flood fighting efforts regarding sliding/tipping stability and seepage rates. A copy of the Ref 2.1 is included in this calculation as Attachment 1.

As part of the evaluation Wenck Associates performed field tests to evaluate the coefficient of friction between the fill material and various test surfaces. The coefficients of friction computed from the test data were higher than published values. This deviation from published values was attributed to the deformation of the bottom of the basket that was observed as the basket began to move.

The Wenck Associates evaluation also assessed the overturning stability of the Concertainers. Based on their results the Concertainers have a factor-of-safety of 30 against overturning with up to 14-degrees of tilt. Therefore, the Wenck Associates evaluation concludes that overtuning is not a problem for the Concertainers unless there is an entire subgrade failure.

Finally, reported excessive seepage rates were also evaluated. Measurement of the seepage rates was not performed since the evaluation was not performed until after the conclusion of the flood fighting effort. Seepage was evaluated based on interviews with field personnel. The interview revealed varying reports of seepage rates where some thought it was excessive while others thought it was less than that of a traditional sandbag wall. United States Army Corps of Engineers (USACE) evaluated the Hesco Concertainer system as part of their flood-fighting structure demonstration which included measurement of seepage rates.

4.2 USACE Engineering Research and Development Center (ERDC) Study

Reference 2.2 (excerpts included in Attachment 2) is the ERDC Technical Release describing the Flood-Fighting Structures Demonstration and Evaluation Program. This program included both laboratory and field testing of three rapidly deployable flood-fighting systems including the Hesco Concertainer system compared to a sandbag levee as a baseline. The program evaluated deployment requirements, the effects of hydrostatic loading with and without wave action on seepage rates and displacement, effects of debris impacts, repair rates, and reuseability. Both the laboratory and field test levees were constructed by Government personnel at the direction of a Hesco representative.

The laboratory testing of the Concertainers was performed in May 2004. The laboratory testing revealed that the Concertainer system had higher seepage rates than the other systems tested including the sandbag levee baseline. The majority of the seepage was noted to be occuring at the vertical joints between the units and at the corners of the layout. The concertainer system performed well in the debris impact, wave action, and overtopping test scenarios. However, scour of the fill material was noted when the units were overtopped either by wave action or pool elevation. It was noted that the scour did not have an apparent negative impact on the stability of the Concertainers. One of the repairs performed on the concertainer system was to cover the units with a tarp to protect the fill material.

The field testing of the Concertainer system was performed between May and July 2004 on Government property in Vicksburg Harbor. Half of each rapidly deployable flood fighting system was constructed on bare earth and the other half was constructed directly on grass and weeds. As with the laboratory test, the field test revealed seepage rates much higher than those measured for the other rapidly deployable flood-fighting systems and the sandbag levee. Two repair attempts were made during the testing to reduce the seepage rate, both resulting in basically no reduction in the seepage rate. There was no discussion about any problems with piping under the Concertainers either on the bare soil or on the grass.

As discussed in Ref. 2.3, Hesco Bastion requested and funded a laboratory retest of the Concertainer system in 2005. The Concertainer system installation procedure was revised to include the installation of poly sheeting on the water side of the flood wall to reduce the seepage rates. The retest report indicates that the revised installation procedure reduced the seepage rates to comparable levels as the other rapidly deployable flood fighting systems evaluated.

4.3 Literature Review Conclusions

Based on the body of literature reviewed, the use of the Hesco Bastion Concertainer system to temporarily raise the dams is an appropriate application of this technology. However, following lessons from the testing should be considered when the developing the layouts and details for the installation.

1. Installation of the Concertainer system without the poly sheeting on the water side leads to seepage rates of up to 1.81gpm/ft. If this seepage rate is not acceptable, the poly sheeting will need to be installed on the upstream face of the flood wall.
2. If the flood wall elevation is such that it might be overtopped by wave action, either a geotextile or poly sheeting should be placed over the top of the Concertainers to protect the fill material. This same protection may be applicable in areas where the Concertainers are placed immediately adjacent to traffic to prevent the air flow from passing vehicles from scouring the fill material.
3. As with any temporary flood-fighting structure, the Concertainer system will require inspection and repair during the flood event. Reserve materials should be kept available on site to perform repairs to the flood wall as necessary.

5.0 Concertainer Weight

The Concertainer units bulge when they are filled and properly compacted. The deformed shape of the Concertainer unit allows the unit to hold more weight than in its square shape. Therefore, the additional weight per Concertainer is computed below based on a single 3-foot square unit. Since the Concertianers will be in a continuous line, only one or two sides of the unit will be able to bulge depending on the configuration. For the purposes of increasing the weight, the deformed shape is idealized as a circle with a circumference equivalent to the perimeter of the undeformed unit. The idealized shape will then be compared to the test data provide in Ref. 2.1.

Length of Unit Face, $L_{face} := 3ft$

Perimeter of Undeformed Unit, $P_{undeformed} := 4 \cdot L_{face}$ $\quad P_{undeformed} = 12\,ft$

Area of Undeformed Unit, $A_{undeformed} := L_{face}^2$ $\quad A_{undeformed} = 9\,ft^2$

Radius of Idealized Shape, $r_{idealized} := \dfrac{P_{undeformed}}{2 \cdot \pi}$ $\quad r_{idealized} = 1.91\,ft$

Area of idealized Shape, $A_{idealized} := \pi \cdot r_{idealized}^2$ $\quad A_{idealized} = 11.459\,ft^2$

Volume Increase Factor, $F_{volume} := \dfrac{A_{idealized}}{A_{undeformed}}$ $\quad F_{volume} = 1.273$

Therefore, the volume of the 3' x 3' x 4' unit tested in Ref. 2.1 would be 1.70 CY. Based on the data included in Table 4 of Ref. 2.1, the average volume of the unit was 1.78 CY. Therefore, the volume increase factor is slightly conservative for the purposes of this stability analysis. Since only one or two faces of the unit are able to deform in the flood wall, the volume increase will be evenly distributed to each face of the unit.

Volume Increase Factor per Face, $f_{volume} := \dfrac{F_{volume} - 1.0}{4}$ $\quad f_{volume} = 0.068$

6.0 Stability Analysis

The maximum water depth that maintains the minimum factor-of-safety for sliding stability will be calculated for several Concertainer configurations in accordance with USACE EM 1110-2-2100 and EM 1110-2-2200. The location of the resultant force under the Concertainer will be calculated for the water depth corresponding to the sliding stability analysis. The analyses will be performed for a unit length of the flood wall and the Concertainers will be assumed to act as a rigid bodies. The zero compression zone analysis included in USACE EM 1110-2-2200 will not be utilized in this calculation because, while the analysis in this calculation assumes rigid body behavior, the Concertainers will not uplift as rigid bodies but will deform under loading and not actually rotate about their downstream edge.

The fill material weight and friction coefficients used in the analyses are based on the data collected during Wenck Associates, Inc. field testing and published values.

Design Input

Unit Weight of Water, $\gamma_w := 62.4\text{pcf}$

Concertainer Width, $W := 3\text{ft}$

Concertainer Height, $H_4 := 4\text{ft}$ or $H_3 := 3\text{ft}$

Unit Weight of Lightly Compacted Fill Material, $\gamma_{fill\_L} := 102\text{pcf}$ (Ref. 2.1, Table 4, Compacted Sand)

Unit Weight of Dry Dense Uniform Sand, $\gamma_{Sand\_dry} := 109.43\text{pcf}$ (Ref. 2.7, Table 6.3)

Unit Weight of Saturated Dense Uniform Sand, $\gamma_{Sand\_sat} := 130.43\text{pcf}$ (Ref. 2.7, Table 6.3)

Unit Weight of Heavily Compacted Fill Material, $\gamma_{fill\_H} := \dfrac{\gamma_{Sand\_dry} + \gamma_{Sand\_sat}}{2}$

$\gamma_{fill\_H} = 119.93\,\text{pcf}$

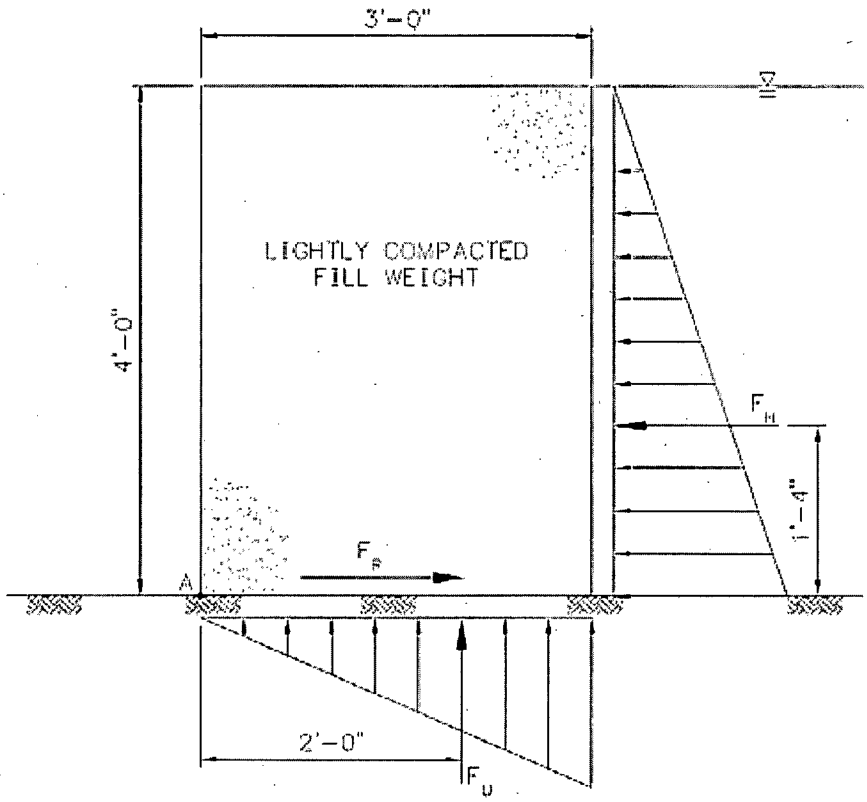
(See Moisture-Density Relationships for Sand Utilized in the Temporary Dams Modification Project in Attachment 7)

Coefficients of Friction,

- Concertainer on Grass-Muddy/Saturated, $\mu_{grass} := 0.65$ (Ref. 2.1, Table 4)
- Concertainer on PCC Street, $\mu_{PCC} := 0.57$ (Ref. 2.1, Table 4)
- Uniform Concrete Slab Thickness on Earth, $\mu_{slab} := 0.6$ (Ref. 2.4, Section 8.8)

Minimum Sliding Factor of Safety, $FS_{min} := 1.1$ (Ref. 2.6, Table 3-2, Extreme Condition)

6.1 Single 4-Foot Concertainer Flood Wall



6.1.1 Sliding Stability

Depth of Water, $D_w := 4ft$

Hydrostatic Resultant Force, $F_H := \frac{\gamma_w \cdot D_w^{2}}{2}$ $\qquad F_H = 499.2\,plf$

Concertainer Weight, $F_C := W \cdot H_4 \cdot \gamma_{fill\_L} \cdot \left(2 \cdot f_{volume} + 1.0\right)$ $\qquad F_C = 1391.223\,plf$

Uplift Force, $F_U := \frac{\gamma_w \cdot D_w \cdot W}{2}$ $\qquad F_U = 374.4\,plf$

Resisting Force on Grass, $F_{R\_grass} := \mu_{grass} \cdot \left(F_C - F_U\right)$ $\qquad F_{R\_grass} = 660.935\,plf$

Resisting Force on Pavement, $F_{R\_PCC} := \mu_{PCC} \cdot \left(F_C - F_U\right)$ $\qquad F_{R\_PCC} = 579.589\,plf$

Factor of Safety on Grass, $FS_{grass} := \frac{F_{R\_grass}}{F_H}$ $\qquad \boxed{FS_{grass} = 1.324}$

Factor of Safety on Pavement, $FS_{PCC} := \frac{F_{R\_PCC}}{F_H}$ $\qquad \boxed{FS_{PCC} = 1.161}$

6.1.2 Overturning Stability

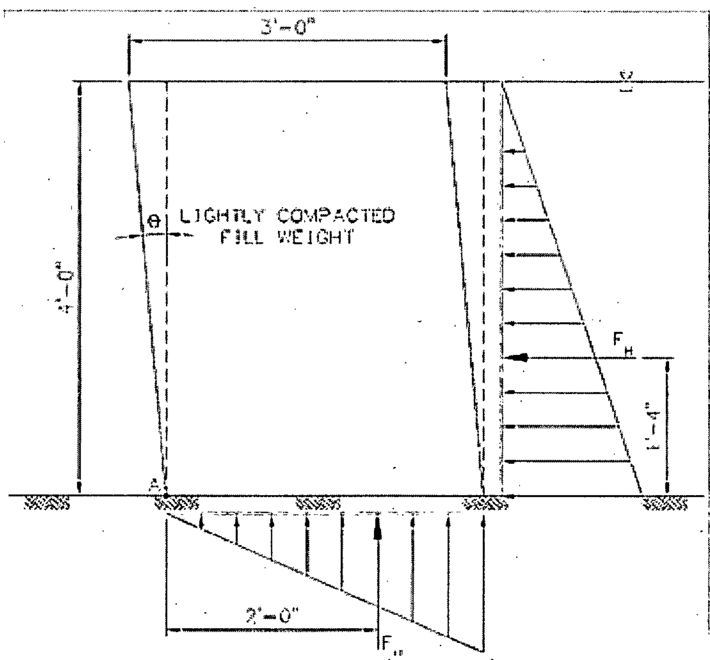
Compute the location of the resultant force under the Concertainer considering the Concertainer installed without any unit tilt loaded with the full hydrostatic and uplift pressures. This analysis assumes that the unit acts as a rigid body.

Overturning Moment About Point A, $M_o := F_H \cdot \frac{D_w}{3} + F_U \cdot \frac{2W}{3}$ $\quad M_o = 1414.4 \frac{lbf \cdot ft}{ft}$

Resisting Moment About Point A, $M_R := F_C \cdot \frac{W}{2}$ $\quad M_R = 2086.834 \frac{lbf \cdot ft}{ft}$

Distance of Resultant from point A, $x := \frac{M_R - M_o}{F_C - F_U}$ $\quad x = 0.661\,ft$ Resultant within base

- Since the Concertainer units may become skewed during installation, deterimine the maximum unit tilt angle while maintaining the resultant force 3-inches within the base. USACE guidance states that the resultant force must be sufficiently within the base to maintain bearing pressures within allowables. Due to the relatively light weight of the Concertainers as compared to the bearing pressures of the controlled fill they are placed on and that they will not act as rigid bodies as the analysis assumes, 3-inches within the base will maintain bearing pressures within allowables.

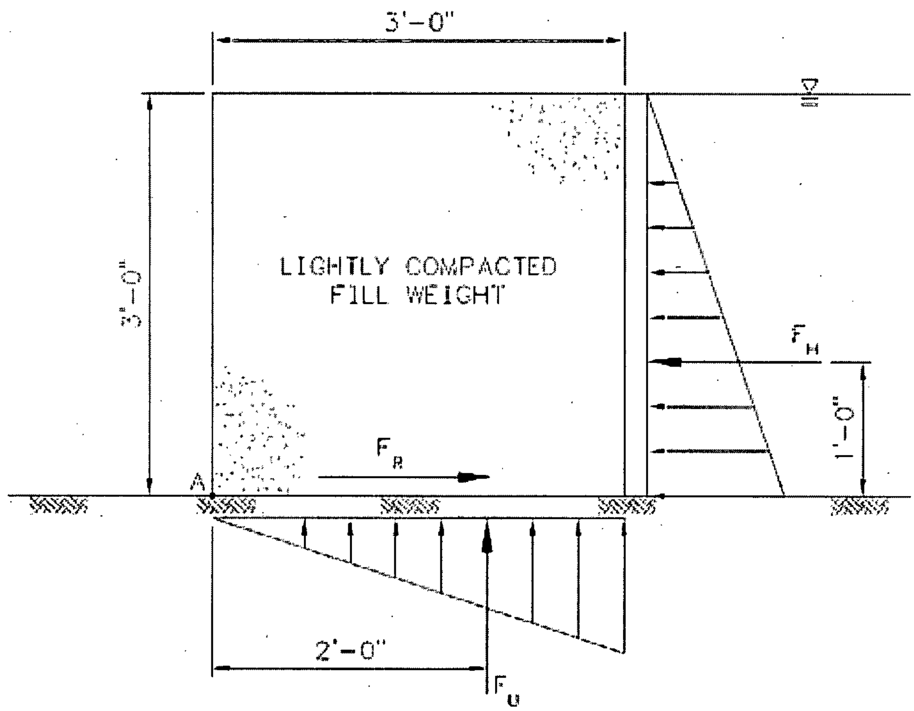


Minimum Resisting Moment, $M_{R\_min} := 0.25ft \cdot (F_C - F_U) + M_o$ $\quad M_{R\_min} = 1668.606 \frac{lbf \cdot ft}{ft}$

Minimum Moment Arm About Pont A, $D_{arm} := \frac{M_{R\_min}}{F_C}$ $\quad D_{arm} = 1.199\,ft$

Maximum Tilt Angle, $\Theta_{tilt} := \tan\left(\frac{\frac{W}{2} - D_{arm}}{\frac{H_4}{2}}\right)$ $\quad \boxed{\Theta_{tilt} = 8.678\,deg}$

6.2 Single 3-Foot Concertainer Flood Wall



6.2.1 Sliding Stability

Depth of Water, $D_w := 3ft$

Hydrostatic Resultant Force, $F_H := \frac{\gamma_w \cdot D_w^2}{2}$ $\quad F_H = 280.8\,plf$

Concertainer Weight, $F_C := W \cdot H_3 \cdot \gamma_{fill\_L} \cdot (2 \cdot f_{volume} + 1.0)$ $\quad F_C = 1043.417\,plf$

Uplift Force, $F_U := \frac{\gamma_w \cdot D_w \cdot W}{2}$ $\quad F_U = 280.8\,plf$

Resisting Force on Grass, $F_{R\_grass} := \mu_{grass} \cdot (F_C - F_U)$ $\quad F_{R\_grass} = 495.701\,plf$

- Since the pavement is impervious, uplift pressure does not act on the Concertainer units on pavement.

Resisting Force on Pavement, $F_{R\_PCC} := \mu_{PCC} \cdot (F_C - F_U)$ $\quad F_{R\_PCC} = 434.692\,plf$

Factor of Safety on Grass, $FS_{grass} := \frac{F_{R\_grass}}{F_H}$ $\quad \boxed{FS_{grass} = 1.765}$

Factor of Safety on Pavement, $FS_{PCC} := \frac{F_{R\_PCC}}{F_H}$ $\quad \boxed{FS_{PCC} = 1.548}$

6.2.2 Overturning Stability

Compute the location of the resultant force under the Concertainers considering the Concertainer installed without any unit tilt loaded with the full hydrostatic and uplift pressures. This analysis assumes that the unit acts as a rigid body.

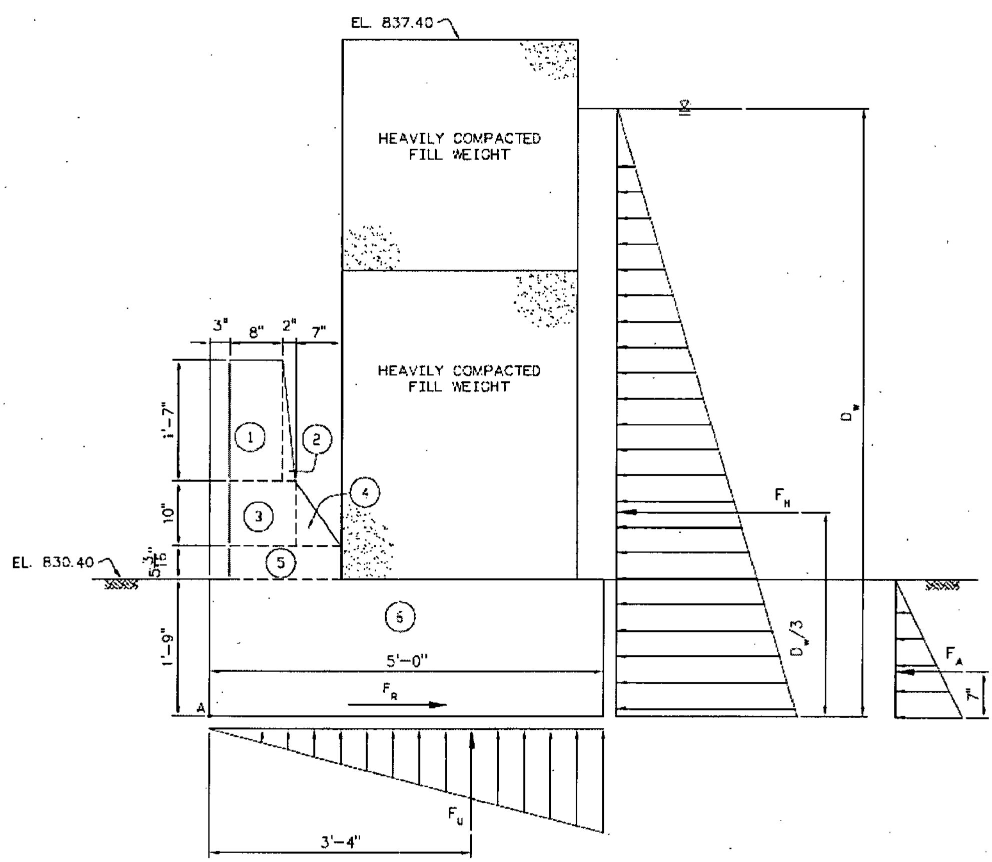
Overturning Moment About Point A, $M_o := F_H \cdot \frac{D_w}{3} + F_U \cdot \frac{2W}{3}$ $\quad M_o = 842.4 \frac{lbf \cdot ft}{ft}$

Resisting Moment About Point A, $M_R := F_C \cdot \frac{W}{2}$ $\quad M_R = 1565.125 \frac{lbf \cdot ft}{ft}$

Distance of Resultant from point A, $x := \frac{M_R - M_o}{F_C - F_U}$ $\quad x = 0.948\,ft$ Resultant within base

6.3 Double Concertainer Flood Wall Along US 321

A concrete flood wall was installed along the downstream side of US 321 in the 1990's to raise the elevation of Fort Loudoun Dam to El. 833.25. The temporary modification to Fort Loudoun Dam must raise the elevation above El. 835.00. Therefore, a 4-foot and 3-foot will be required to achieve this revised elevation. Hesco Bastion's recommended configuration for a double height flood wall utilizes two Concertainers at the base and one stacked on top. Due to limited space on the shoulder of the road it is desired to utilize the existing concrete flood wall to aid the stability of the double height Concertainer configuration as shown in the figure below and avoid using the second unit on the base level.



- Compute weight and center of gravity of existing flood wall.

Part	Shape Factor	Width (in)	Height (in)	Area (SF)	Unit Weight (PCF)	Weight (lb/ft)	Moment Arm (in)	Moment (lb-in/ft)
1	1	8	19	1.06	145	153.06	7	1071.39
2	0.5	2	19	0.13	145	19.13	11.667	223.21
3	1	10	10	0.69	145	100.69	8	805.56
4	0.5	7	10	0.24	145	35.24	15.333	540.38
5	1	17	5.1875	0.61	145	88.80	11.5	1021.20
6	1	60	21	8.75	145	1268.75	30	38062.50
Totals				11.49		1665.67		41724.24

Weight of Existing Flood Wall, $F_{wall} := 1665.67\frac{lbf}{ft}$

Moment of Wall Parts About Point A, $M_{wall} := 41724.24\frac{lbf\cdot in}{ft}$

Center of Gravity of Concrete Flood Wall from Point A, $CG_{wall} := \frac{M_{wall}}{F_{wall}}$ $CG_{wall} = 2.087\,ft$

6.3.1 Sliding Stability

Maximum Pool Elevation, $EL_{max} := 836.2ft$

Depth of Soil, $D_{soil} := 1.75ft$

Depth of Water, $D_w := EL_{max} - 830.4ft + D_{soil}$ $D_w = 7.55\,ft$

Hydrostatic Resultant Force, $F_H := \frac{\gamma_w \cdot D_w^2}{2}$ $F_H = 1778.478\,plf$

Concertainer Weight, $F_C := W\cdot(H_4 + H_3)\cdot\gamma_{fill\_H}\cdot(2\cdot f_{volume} + 1.0)$ $F_C = 2862.611\,plf$

Footing Width, $W_{footing} := 5ft$

Uplift Force, $F_U := \frac{\gamma_w \cdot D_w \cdot W_{footing}}{2}$ $F_U = 1177.8\,plf$

Resisting Force of Concrete on Earth, $F_{R\_slab} := \mu_{slab}\cdot(F_{wall} + F_C - F_U)$

$F_{R\_slab} = 2010.289\,plf$

- Compute active earth pressure on slab. Active pressure based on soil properties obtained from the original design calculation for the flood wall (Ref. 2.5).

Effective Weight of Soil, $\gamma_{soil\_eff} := 65pcf$

Internal Friction Angle, $\phi_{soil} := 26deg$

Active Pressure Coefficient, $K_a := \tan\left(45 \cdot deg - \frac{\phi_{soil}}{2}\right)^2$ $K_a = 0.39$

Active Earth Pressure Resultant, $F_A := 0.5 \cdot \gamma_{soil\_eff} \cdot D_{soil}^2 \cdot K_a$ $F_A = 38.863\,plf$

Factor of Safety, $FS_s := \frac{F_{R\_slab}}{F_H + F_A}$ $\boxed{FS_s = 1.106}$

6.3.2 Overturning Stability

Compute the location of the resultant force under the Concertainer considering the Concertainer installed without any unit tilt loaded with the full hydrostatic and uplift pressures. This analysis assumes that the unit acts as a rigid body.

Distance From Point A to Centerline of Concertainters, $D_{Conc} := 38in$

Overturning Moment About Point A, $M_o := F_H \cdot \frac{D_w}{3} + F_U \cdot \frac{2W_{footing}}{3} + F_A \cdot \frac{D_{soil}}{3}$

$$M_o = 8424.506 \frac{lbf \cdot ft}{ft}$$

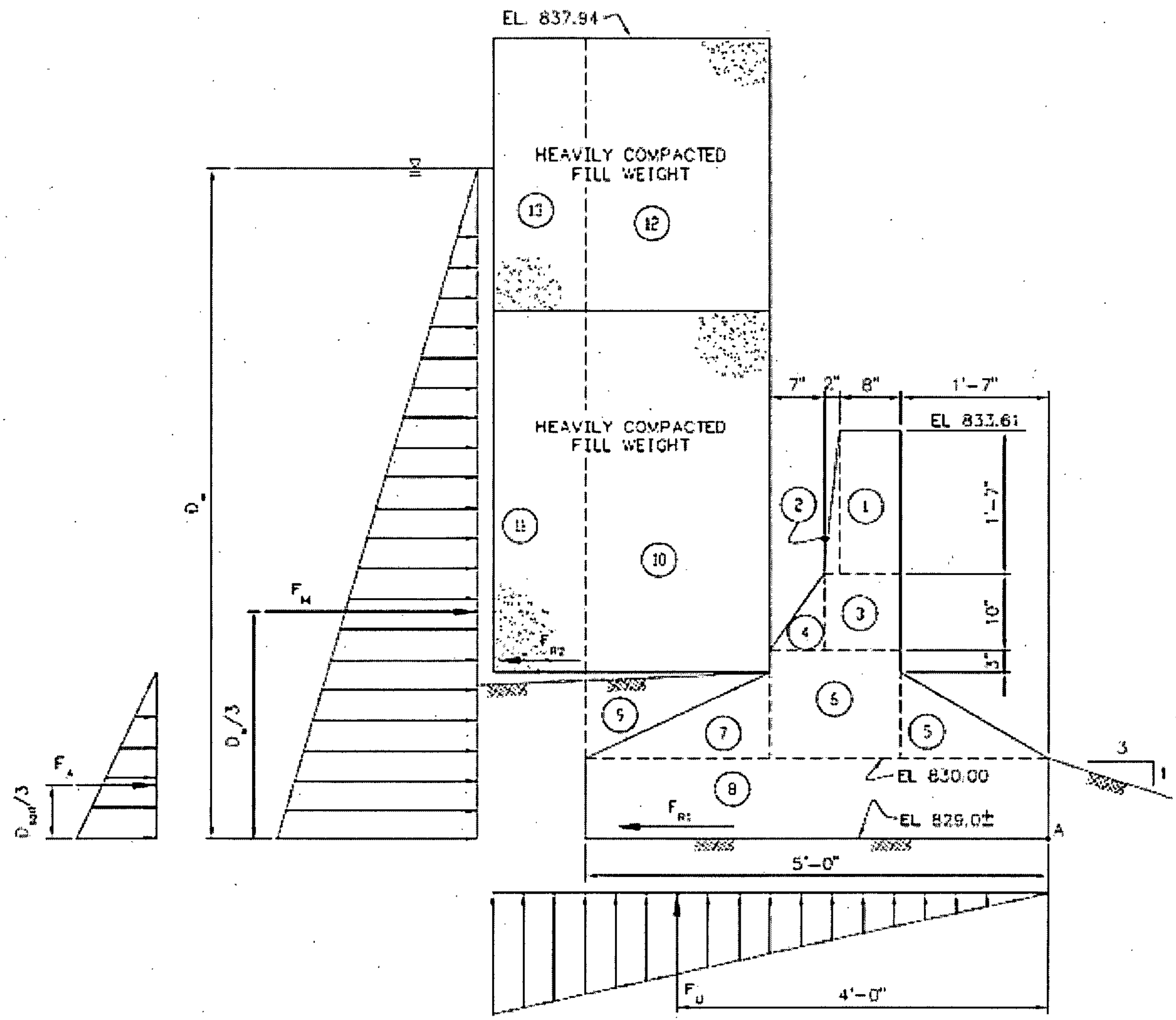
Resisting Moment About Point A, $M_R := F_C \cdot D_{Conc} + F_{wall} \cdot CG_{wall}$

$$M_R = 12541.955 \frac{lbf \cdot ft}{ft}$$

Distance of Resultant from point A, $x := \frac{M_R - M_o}{F_{wall} + F_C - F_U}$ $x = 1.229\,ft$ Resultant within base.

6.4 Double Concertainer Flood Wall Along City Park Drive

A concrete flood wall was installed along the along the edge of City Park Drive which runs along the top of the saddle dam in the 1990's to raise the elevation of Fort Loudoun Dam to El. 833.25. As with the flood wall located on the downstream side of US 321, it is desired to utilize the existing concrete flood wall to aid the stability of the double height Concertainer configuration as shown in the figure below and avoid using the second unit on the base level to maintain two lanes of traffic along City Park Drive.



- Compute weight and center of gravity of parts 1 through 10 and 12.

Part	Shape Factor	Width (in)	Height (in)	Area (SF)	Unit Weight (PCF)	Weight (lb/ft)	Moment Arm (in)	Moment (lb-in/ft)
1	1	8	19	1.06	145	153.06	23	3520.28
2	0.5	2	19	0.13	145	19.13	27.667	529.32
3	1	10	10	0.69	145	100.69	24	2416.67
4	0.5	7	10	0.24	145	35.24	31.333	1104.27
5	0.5	19	11.32	0.75	145	108.29	12.667	1371.67
6	1	17	14.32	1.69	145	245.13	27.5	6741.09
7	0.5	24	11.32	0.94	145	136.78	44	6018.47
8	1	60	12	5.00	145	725.00	30	21750.00
9	0.5	24	11.32	0.94	127.4	120.18	52	6249.39
10	1.068	24	48	8.54	117.625	1004.99	48	48239.42
12	1.068	24	36	6.41	117.625	753.74	48	36179.57
Totals				26.40		3402.24		134120.15

Table Notes:
1. Unit weight for part 9 accounts for effective weight of soil and water weight.
2. Shape factors for parts 10 and 12 accounts for deformation of one side of the Concertainers

Weight of Existing Flood Wall, $F_{wall} := 3402.24 \frac{lbf}{ft}$

Moment of Wall Parts About Point A, $M_{wall} := 134120.15 \frac{lbf \cdot in}{ft}$

Center of Gravity of Concrete Flood Wall from Point A, $CG_{wall} := \frac{M_{wall}}{F_{wall}}$ $\quad CG_{wall} = 3.285\,ft$

6.4.1 Sliding Stability

Maximum Pool Elevation, $EL_{max} := 836.47ft$

Depth of Water, $D_w := EL_{max} - 829.0ft \quad D_w = 7.47\,ft$

Hydrostatic Resultant Force, $F_H := \frac{\gamma_w \cdot D_w^{\,2}}{2} \quad F_H = 1740.988\,plf$

Total Width, $W_{total} := 6ft$

Width of Footing, $W_{footing} = 5\,ft$

Concertainer Weight, $F_C := (W_{total} - W_{footing}) \cdot (H_4 + H_3) \cdot \gamma_{fill\_H} \cdot (f_{volume} + 1.0)$

$F_C = 896.857\,plf$

Uplift Force Under Flood Wall, $F_{U1} := \frac{\gamma_w \cdot D_w \cdot \frac{W_{footing}}{W_{total}} \cdot W_{footing}}{2} \quad F_{U1} = 971.1\,plf$

Uplift Force Under Concertainers, $F_{U2} := \frac{\gamma_w \cdot D_w \cdot W_{total}}{2} - F_{U1}$ $\quad F_{U2} = 427.284\,plf$

Resisting Force of Concrete on Earth, $F_{R1} := \mu_{slab} \cdot (F_{wall} - F_{U1})$ $\quad F_{R1} = 1458.684\,plf$

Resisting Force of Concertainer on Pavement, $F_{R2} := \mu_{PCC} \cdot F_C$ $\quad F_{R2} = 511.208\,plf$

- Compute active earth pressure on slab. Active pressure based on soil properties obtained from the original design calculation for the flood wall (Ref. 2.5).

Effective Weight of Soil, $\gamma_{soil\_eff} := 65pcf$

Internal Friction Angle, $\phi_{soil} := 26deg$

Active Pressure Coefficient, $K_a := \tan\left(45 \cdot deg - \frac{\phi_{soil}}{2}\right)^2$ $\quad K_a = 0.39$

Depth of Soil, $D_{soil} := 22in$

Active Earth Pressure Resultant, $F_A := 0.5 \cdot \gamma_{soil\_eff} \cdot D_{soil}^2 \cdot K_a$ $\quad F_A = 42.653\,plf$

Factor of Safety, $FS_s := \frac{F_{R1} + F_{R2}}{F_H + F_A}$ $\quad \boxed{FS_s = 1.104}$

6.4.2 Overturning Stability

Compute the location of the resultant force under the Concertainer considering the Concertainer installed without any unit tilt loaded with the full hydrostatic and uplift pressures. This analysis assumes that the unit acts as a rigid body.

Distance From Point A to Centerline of Parts 11 & 13, $D_{Conc} := 66in$

Total Uplift Pressure, $F_{Utotal} := \frac{\gamma_w \cdot D_w \cdot W_{total}}{2}$ $\quad F_{Utotal} = 1398.384\,plf$

Overturning Moment About Point A, $M_o := F_H \cdot \frac{D_w}{3} + F_U \cdot \frac{2W_{total}}{3} + F_A \cdot \frac{D_{soil}}{3}$

$$M_o = 9072.326 \frac{lbf \cdot ft}{ft}$$

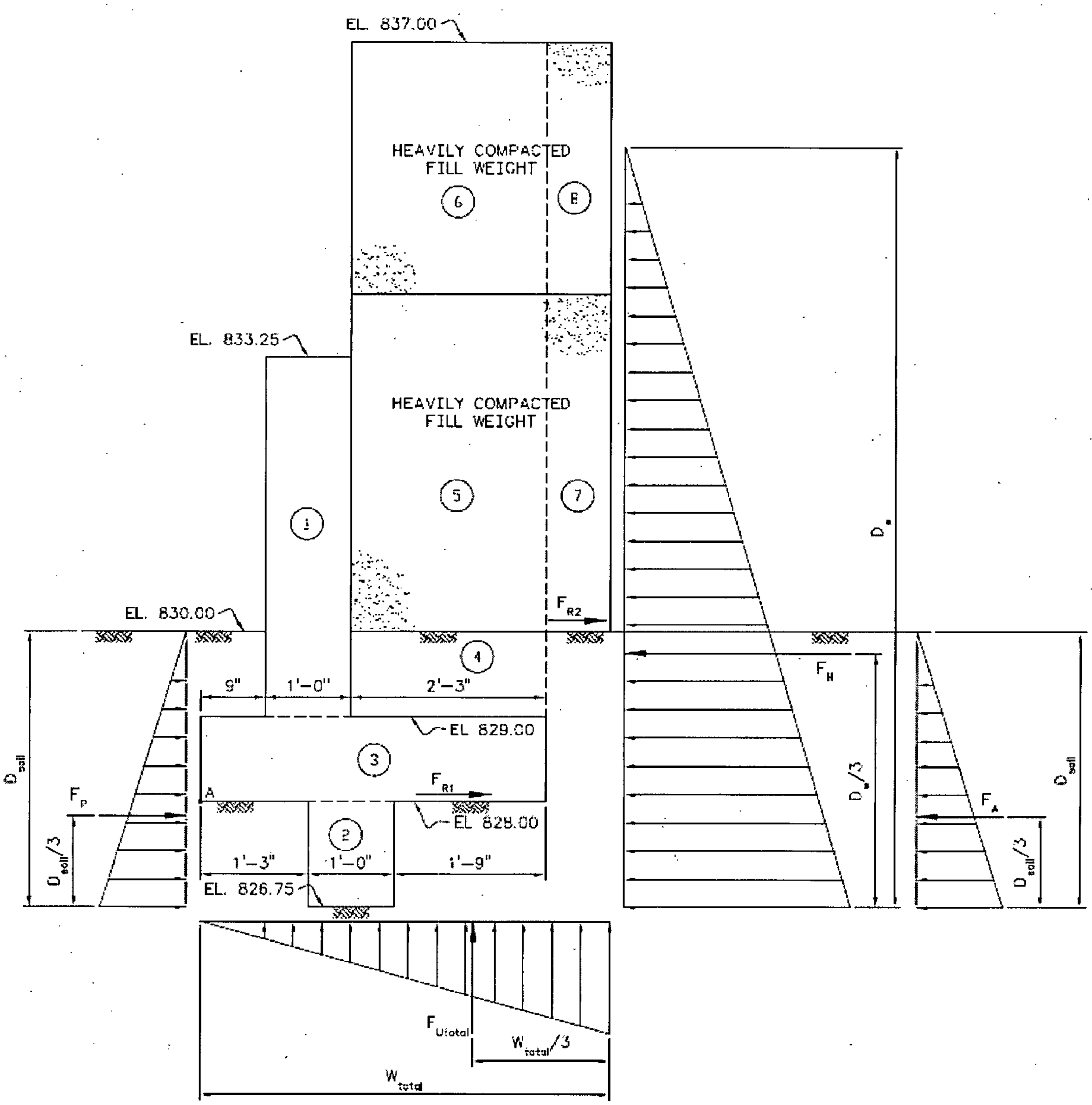
Resisting Moment About Point A, $M_R := F_C \cdot D_{Conc} + F_{wall} \cdot CG_{wall} + F_{R2} \cdot 1.943ft$

$$M_R = 17102.67 \frac{lbf \cdot ft}{ft}$$

Distance of Resultant from point A, $x := \frac{M_R - M_o}{F_{wall} + F_C - F_{U1} - F_{U2}}$ $\quad x = 2.768\,ft$ Resultant within base

6.5 Double Concertainer Flood Wall Under US 321 Bridge

A concrete flood wall was installed along the under the US 321 bridge over the dam in the 1990's to raise the elevation of Fort Loudoun Dam to El. 833.25. As with the flood wall located on the downstream side of US 321 and along City Park Drive, it is desired to utilize the existing concrete flood wall to aid the stability of the double height Concertainer configuration as shown in the figure below and avoid using the second unit on the base level to maintain two lanes of traffic along City Park Drive. The wall configuration is shown in Section A4-A4 on TVA Drawing 23W230-4.



- Compute weight and center of gravity of parts 1 through 6.

Part	Shape Factor	Width (in)	Height (in)	Area (SF)	Unit Weight (PCF)	Weight (lb/ft)	Moment Arm (in)	Moment (lb-in/ft)
1	1	12	51	4.25	145	616.25	15	9243.75
2	1	12	15	1.25	145	181.25	21	3806.25
3	1	48	12	4.00	145	580.00	24	13920.00
4	1	27	12	2.25	127.4	286.65	34.5	9889.43
5	1.068	27	48	9.61	117.625	1130.61	34.5	39006.10
6	1.068	27	36	7.21	117.625	847.96	34.5	29254.57
Totals				28.57		3642.72		105120.09

Table Notes:
1. Unit weight for part 4 accounts for effective weight of soil and water weight.
2. Shape factors for parts 5 and 6 accounts for deformation of one side of the Concertainers

Weight of Existing Flood Wall, $F_{wall} := 3642.72\frac{lbf}{ft}$

Moment of Wall Parts About Point A, $M_{wall} := 105120.09\frac{lbf \cdot in}{ft}$

Center of Gravity of Concrete Flood Wall from Point A, $CG_{wall} := \frac{M_{wall}}{F_{wall}}$ $CG_{wall} = 2.405\,ft$

6.5.1 Sliding Stability

Maximum Pool Elevation, $EL_{max} := 835.8ft$

Depth of Water, $D_w := EL_{max} - 826.75ft$ $D_w = 9.05\,ft$

Hydrostatic Resultant Force, $F_H := \frac{\gamma_w \cdot D_w^{\,2}}{2}$ $F_H = 2555.358\,plf$

Total Width, $W_{total} := 4.75ft$

Width of Footing, $W_{footing} := 4ft$

Concertainer Weight, $F_C := (W_{total} - W_{footing}) \cdot (H_4 + H_3) \cdot \gamma_{fill\_H} \cdot (f_{volume} + 1.0)$

$F_C = 672.643\,plf$

Uplift Force Under Flood Wall, $F_{U1} := \frac{\gamma_w \cdot D_w \cdot \frac{W_{footing}}{W_{total}} \cdot W_{footing}}{2}$ $F_{U1} = 951.107\,plf$

Uplift Force Under Concertainers, $F_{U2} := \frac{\gamma_w \cdot D_w \cdot W_{total}}{2} - F_{U1}$ $F_{U2} = 390.103\,plf$

Resisting Force of Concrete on Earth, $F_{R1} := \mu_{slab} \cdot (F_{wall} - F_{U1})$ $F_{R1} = 1614.968\,plf$

Resisting Force of Concertainer on Pavement, $F_{R2} := \mu_{PCC} \cdot F_C$ $F_{R2} = 383.406 \text{plf}$

- Compute active earth pressure on slab. Active pressure based on soil properties obtained from the original design calculation for the flood wall (Ref. 2.5).

Effective Weight of Soil, $\gamma_{soil\_eff} := 65 \text{pcf}$

Internal Friction Angle, $\phi_{soil} := 26 \text{deg}$

Active Pressure Coefficient, $K_a := \tan\left(45 \cdot \text{deg} - \dfrac{\phi_{soil}}{2}\right)^2$ $K_a = 0.39$

Depth of Soil, $D_{soil} := 3.25 \text{ft}$

Active Earth Pressure Resultant, $F_A := 0.5 \cdot \gamma_{soil\_eff} \cdot D_{soil}^{\,2} \cdot K_a$ $F_A = 134.038 \text{plf}$

- Compute passive earth pressure on slab. Passive pressure based on soil properties obtained from the original design calculation for the flood wall (Ref. 2.5).

Effective Weight of Soil, $\gamma_{soil\_eff} := 65 \text{pcf}$

Internal Friction Angle, $\phi_{soil} := 26 \text{deg}$

Active Pressure Coefficient, $K_p := \tan\left(45 \cdot \text{deg} + \dfrac{\phi_{soil}}{2}\right)^2$ $K_p = 2.561$

Depth of Soil, $D_{soil} := 3.25 \text{ft}$

Active Earth Pressure Resultant, $F_P := 0.5 \cdot \gamma_{soil\_eff} \cdot D_{soil}^{\,2} \cdot K_p$ $F_P = 879.168 \text{plf}$

Factor of Safety, $FS_s := \dfrac{F_{R1} + F_{R2} + F_P}{F_H + F_A}$ $\boxed{FS_s = 1.07}$

6.5.2 Overturning Stability

Compute the location of the resultant force under the Concertainer considering the Concertainer installed without any unit tilt loaded with the full hydrostatic and uplift pressures. This analysis assumes that the unit acts as a rigid body.

Distance From Point A to Centerline of Parts 7 & 8, $D_{Conc} := 52.5 \text{in}$

Total Uplift Pressure, $F_{Utotal} := \dfrac{\gamma_w \cdot D_w \cdot W_{total}}{2}$ $F_{Utotal} = 1341.21 \text{plf}$

Overturning Moment About Point A, $M_o := F_H \cdot \dfrac{D_w}{3} + F_U \cdot \dfrac{2W_{total}}{3} + F_P \cdot \dfrac{D_{soil}}{3}$

$$M_o = 12390.795 \frac{lbf \cdot ft}{ft}$$

Resisting Moment About Point A, $M_R := F_C \cdot D_{Conc} + F_{wall} \cdot CG_{wall} + F_A \cdot \frac{D_{soil}}{3} + F_{R2} \cdot 2ft$

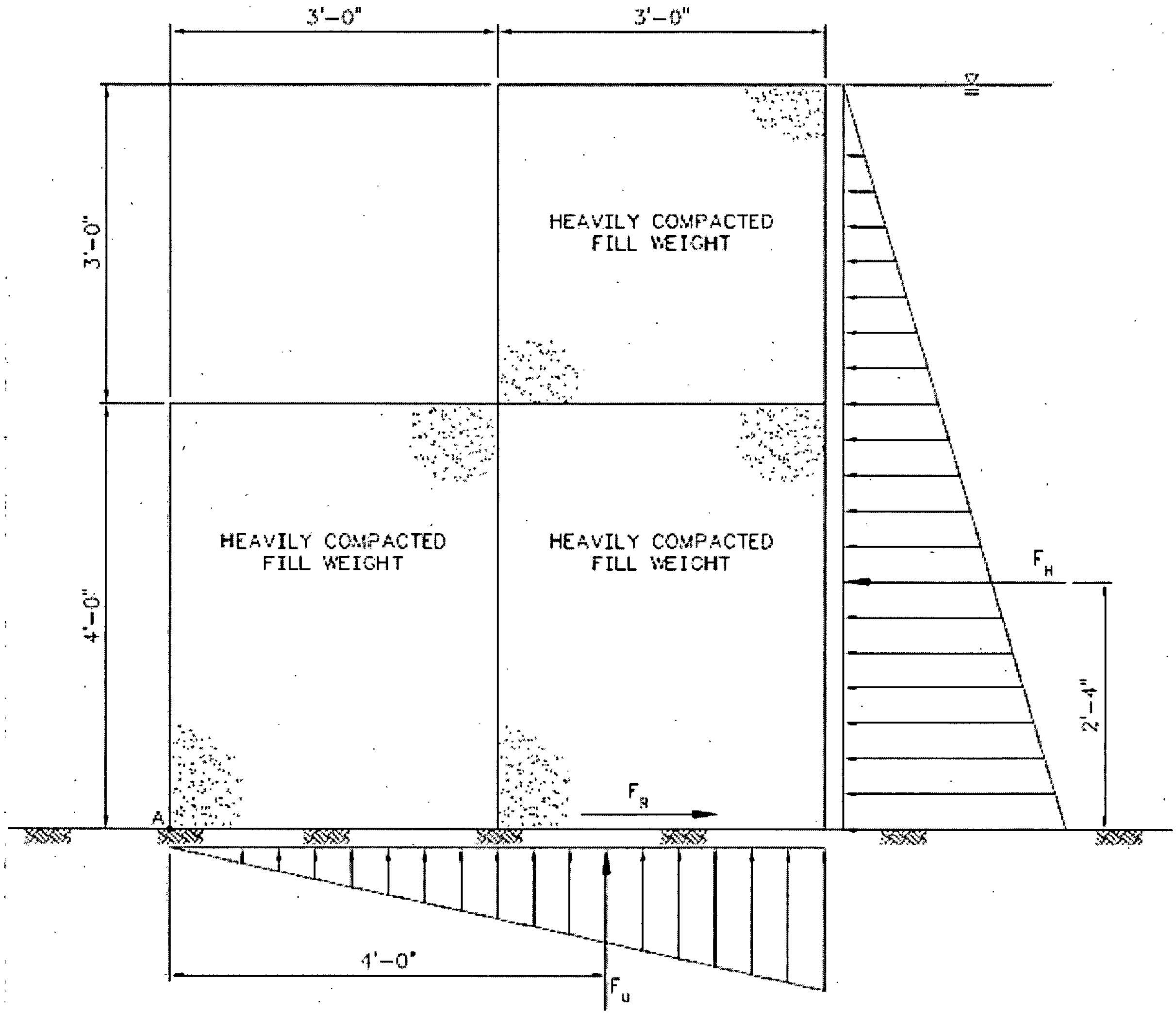
$$M_R = 12614.84 \frac{lbf \cdot ft}{ft}$$

Distance of Resultant from point A, $x := \frac{M_R - M_o}{F_{wall} + F_C - F_{Utotal}}$ $x = 0.075\,ft$ Resultant within base

This overturning analysis neglects the passive pressure that would be developed behind the shear key as the wall rotated which has the effect of shifting the resultant location closer to point A. Therefore, the configuration has adequate overturning stability.

6.6 L-Shaped Concertainer Configuration

If the pool elevation for Fort Loudoun exceeds the values calculated above for the concrete flood wall configurations, the L-shaped double stacked Concertainer configuration shown below will need to be utilized.



6.6.1 Sliding Stability

Depth of Water, $D_W := 7ft$

Hydrostatic Resultant Force, $F_H := \frac{\gamma_W \cdot D_W^{\ 2}}{2}$ $\qquad F_H = 1528.8\,plf$

Concertainer Weight, $F_C := W \cdot H_3 \cdot \gamma_{fill\_H} \cdot (2 \cdot f_{volume} + 1.0) + 2W \cdot H_4 \cdot \gamma_{fill\_H} \cdot (f_{volume} + 1.0)$

$F_C = 4301.771\,plf$

Width of Base, $W_{base} := 2W$ $\quad W_{base} = 6\,ft$

Uplift Force, $F_U := \dfrac{\gamma_w \cdot D_w W_{base}}{2}$ $\quad F_U = 1310.4\,plf$

Resisting Force on Grass, $F_{R\_grass} := \mu_{grass} \cdot (F_C - F_U)$ $\quad F_{R\_grass} = 1944.391\,plf$

Resisting Force on Pavement, $F_{R\_PCC} := \mu_{PCC} \cdot (F_C - F_U)$ $\quad F_{R\_PCC} = 1705.081\,plf$

Factor of Safety on Grass, $FS_{grass} := \dfrac{F_{R\_grass}}{F_H}$ $\quad \boxed{FS_{grass} = 1.272}$

Factor of Safety on Pavement, $FS_{PCC} := \dfrac{F_{R\_PCC}}{F_H}$ $\quad \boxed{FS_{PCC} = 1.115}$

6.6.2 Overturning Stability

Compute the location of the resultant force under the Concertainer considering the Concertainer installed without any unit tilt loaded with the full hydrostatic and uplift pressures. This analysis assumes that the unit acts as a rigid body.

Overturning Moment About Point A, $M_o := F_H \cdot \dfrac{D_w}{3} + F_U \cdot \left(\dfrac{2}{3}\right) \cdot \dfrac{2W_{base}}{3}$ $\quad M_o = 7061.6 \dfrac{lbf \cdot ft}{ft}$

Resisting Moment About Point A, $M_R := F_C \cdot \dfrac{W_{base}}{2}$ $\quad M_R = 12905.313 \dfrac{lbf \cdot ft}{ft}$

Distance of Resultant from point A, $x := \dfrac{M_R - M_o}{F_C - F_U}$ $\quad x = 1.954\,ft$ Resultant within base.

7.0 Summary and Conclusions

As stated in Attachment 6 the pool elevation associated with the Probable Maximum Flood (PMF) is 835.65 feet. At Fort Loudoun Dam, there are no locations that required the single 4-foot or single 3-foot Concertainer configurations and one location at the Lock Operations Building that- requires the L-shaped Concertainer configuration as shown on drawings 10W222-1 through 10W222-3. The maximum computed pool elevations for the flood wall configurations above were all above the PMF elevation of 835.65-feet. The maximum pool elevation for the L-shaped Concertainer configuration adacent to the Lock Operations Building is also above the PMF elevation because the L-shaped configuration has a maximum allowable water depth of 7-feet and the ground elevation is 829.92-feet (23W230-4).

Attachment 1

Engineering Evaluation of Hesco Barriers Performance in Fargo, ND 2009
(Reference 2.1)

Engineering Evaluation of Hesco Barriers Performance at Fargo,ND 2009

Wenck File #2283-01

Prepared for:

Hesco Bastion, LLC
47152 Conrad E. Anderson Drive
Hammond, LA 70401

Prepared by:

WENCK ASSOCIATES, INC.
3310 Fiechtner Drive;
Suite 110
Fargo, North Dakota
(701) 297-9600

May 2009

Wenck

Table of Contents

ATTACHMENTS

A – Red River Flood Protection Plan – 3/26/2009
(Map of Levee System)

B – "Engineering Analysis" from Report to:
United States Senate Committee on Appropriations, June 29-30, 2004.

1.0 Introduction

The Fargo, North Dakota area, along with its sister city across the river, Moorhead, Minnesota was recently faced with massive flooding from the Red River of the North. Due to an unusually wet fall, followed by a cold, snowy winter, the normally placid Red River was forecasted by the National Weather Service (NWS) to reach a flood crest elevation of 37 to 39 feet in Fargo by late March.[1] Unfortunately, unusual conditions continued to dominate, forcing the NWS to revise their forecast up to 41 feet, higher than any flood level on record, and predicted for only 7 to 10 days from then, instead of the originally estimated three weeks. As the river rapidly rose to nearly 39 feet, new forecasts predicted the river might even go as high as 42 or even 43 feet.

This situation, of course, generated intense concerns, and forced the rapid evaluation and subsequent use of several different methods of flood protection. To protect the City of Fargo, temporary clay dikes, which are the most common form of flood protection in the area, were used to the greatest extent possible to raise existing flood protection to at first 42 feet, but later raised to 44 feet in response to the revised forecasts. Traditional sandbag dikes were also widely used, but due to time constraints, reliability, height limitations, and availability of volunteers, the length of dikes that could be deployed could not meet all the area needs. Therefore, the US Army Corps of Engineers, who were assisting the City of Fargo, turned to Hesco Bastion, LLC for help, and to provide the remaining flood barriers. Hesco barriers have been well-tested by the U.S. Army Corps of Engineers (COE) for use as temporary flood protection, and widely used in many flood situations around the country, including New Orleans for temporary hurricane protection.

Footnote: (1) For reference, "normal" flood stage here is considered to be anything over 18 feet. The 1997 flood, called the "Flood of the Century" and which inundated Grand Forks that year, reached a stage height in Fargo of 39.6 feet. The highest level on record was 40.1 feet, reached back in 1897.

Hesco barriers are able to be installed relatively quickly, and due to this speed of deployment, approximately 10 miles of barriers were installed over a 4 to 5 day period. Fortunately, the combination of clay dikes, sandbag dikes, and Hesco barriers, together with the tremendous efforts by City of Fargo staff, National Guard, and volunteers, worked, and the City of Fargo largely escaped serious flooding.

In the aftermath of these efforts, various comments surfaced regarding the performance of the Hesco Barriers. In particular, concerns were raised about the possibility of the Hesco's sliding laterally over the ground surface due to water pressure, and the increased rate of seepage through the barriers over expected rates. Also, concerns were raised about two specific locations where Hesco's had been deployed, and significant leakage had occurred requiring emergency actions to prevent possible breaching.

Hesco Bastion, concerned about these comments, approached Wenck Associates, Inc. (Wenck) to provide an independent engineering evaluation of the performance of the Hesco barriers during the Fargo floodfighting efforts. The scope of this engineering evaluation was agreed to be as follows:

- Meet with Mr. Dennis Barkemeyer of Hesco Bastion to discuss installation procedures that were used at the various sites in Fargo.
- Evaluate photographs taken during installation of the barriers.
- Visit selected Hesco sites around Fargo to evaluate post-flood barriers.
- Interview City of Fargo/COE staff to discuss product use, problems they encountered or noted, comparison to other dikes (sand bag and clay), and comments on the products.
- Revisit dike locations where the City/COE indicated they had problems or issues.
- Evaluate product and its uses in Fargo for floodfighting in light of the frozen ground oftentimes encountered, and the soft clay soils.
- Prepare a letter report or technical memorandum that outlines the findings of the above work, and provides recommendations for future use in this environment.

1.1 PURPOSE OF EVALUATION

This independent engineering evaluation was requested by Hesco Bastion, LLC to address comments raised after Hesco units were installed in Fargo, North Dakota for combating flooding by the Red River of the North in March, 2009.

1.2 BACKGROUND INFORMATION

Hesco Bastion Concertainers® (hereinafter referred to as "Hesco"), are a structural system of linked baskets containing fill material. They provide a way of positioning and containing large volumes of earth, sand, gravel, or rock to form either temporary or long term structures. They may then be used for a variety of projects, including emergency flood protection (as at Fargo, North Dakota in 2009). The units are manufactured in various sizes and are made of welded galvanized steel mesh that are assembled with coiled joints. A polypropylene, nonwoven geotextile liner retains the fill material that is placed into the open top basket. The baskets are initially flat-packed on pallets, then extended and joined with joining pins, filled with fill material, and placed in various configurations depending on the end use. The units are lightweight, portable, and are easily deployed.

2.0 City of Fargo Uses of Hesco Barriers

2.1 NUMBER OF MILES USED VERSUS TOTAL

The City of Fargo constructed a total of approximately 80 miles of dikes using a combination of clay, sandbags, and Hesco barriers. There were approximately 10 miles of Hesco barriers deployed within the City of Fargo. The barriers were used for both primary and contingency dikes, and were placed on paved and non-paved surfaces, as well as on top of existing levees. The location and types of dikes are shown in Attachment A.

2.2 SIZES

The City of Fargo deployed two types of Hesco units, the standard and flood barriers in the 3' deep by 3' high by 15' long, and 3' deep by 4' high by 15' long sizes. The standard barriers have separation fabric dividing each barrier into five equal compartments, 3' long each. The flood barriers do not utilize the separation **fabric** between compartments, so each unit is able to have continuous fill material.

2.3 INSTALLATION RATES

Installation rates varied greatly due to the availability of fill material, and the access to the area in terms of both men, equipment, and physical access. Field personnel that were interviewed stated they typically could deploy and fill 400 to 600 feet of Hesco's per hour.

2.4 COMPLICATING FACTORS

The City of Fargo is located in the Red River Valley, formed at the bottom of former glacial Lake Agassiz. This lake deposited thick sequences of lacustrine clays and silts, which form the soils here. These soils are very fertile, but have poor engineering properties, and are prone to slippage and soil failures. Weather conditions prior to the rising of the Red River had completely saturated these soils, making conditions very muddy. Dikes then had to built on top of, and using, these saturated clay soils, often while rains continued. Subgrade conditions were thus far from ideal.

Later in the week prior to the predicted flood crest for the Red River, the weather changed again to very cold, and the area received nearly one foot of snow. These cold conditions persisted for awhile, and caused many of the soils to freeze at the surface, further complicating subgrade conditions on which the Hesco barriers (and all temporary dikes) were built.

3.0 Interviews with City and City Representatives

3.1 ISSUES RAISED AND AREAS OF CONCERN

During interviews with personnel involved with the Hesco barriers, concerns were brought up about the stability of the barriers. Some had thought that the barriers were kicking out at the bottom, leaning and possibly sliding over the soil surface. Comments were made about lines of Hesco barriers installed "straight", but then becoming ragged looking over subsequent days as though some were sliding and/or leaning, even though no floodwater had reached them as yet. Concerns were also raised about possible sliding and/or overturning of the Hesco barriers if floodwaters came up over their half-way level (approximately 2 feet vs. the 4-foot height of the barriers), especially since many were already leaning toward the water (although not installed with a noticeable "lean"). Additionally, seepage through some of the barriers had caused some concern.

Two specific locations of concern were also brought up by the City of Fargo. The first was on 5th Street South (just south of I-94), and the second was along the south side of Drain 27 (just east of I-29). It was stated that the 5th Street area had to be buttressed with material on the back side of the Hesco barriers and a section of sandbag dike, after a significant leak was found in the transition area from Hesco barrier to sandbag dike. The second was the Drain 27 area, which had shown settlement in one area where the Hesco barriers were placed on top of an existing earthen levee.

3.2 COMMENTS FROM INTERVIEWS

After hearing the above issues, Wenck interviewed several engineers that were directly involved in erecting Hesco barriers during the flood fight, as well as clay or sandbag dikes. These engineers included representatives from the City Engineering Department, and local firms, Ulteig Engineers, and SRF Engineers. Questions were asked directly about the issues raised above, and what the engineers observed, as well as about the two locations of concerns.

The first location of concern, 5th Street South, was thought to be a problem due to a sandbag dike being butted up directly to a Hesco barrier, with little or no overlap. This transition area had started to leak, so emergency crews buttressed the back side with clay to stop the excessive leakage. Field personnel thought the problem was due to the poor transition, not the Hesco units.

The second location of concern, along the south side of Drain 27, was due to water pushing through a stormwater structure and discharging rapidly out the top of a manhole. This water saturated the dike around the structure and caused a section of the dike to slump (sag), including the Hesco units on top of the dike. Field personnel packed clay over the top of the manhole, and built a small cofferdam around it to stop the leakage. Field personnel thought this issue had nothing to do with the Hesco units, only the stormwater structure.

Field personnel indicated that the amount of dikes constructed with the Hesco barriers in a short time was instrumental in protecting the city. Building the 10 miles with sandbag dikes would have been very difficult with the time and volunteers available, plus the miles already built with sandbags. Additionally, the uniformity of the dikes erected with Hesco units was thought to be very important, especially relative to sandbag dikes raised on an emergency basis by volunteers. They also noted that the units adapted to terrain changes very well. Most thought that seepage under the units was less than what a sandbag dike would be, even without the poly-sheeting used in most locations. Field personnel believed that any leaning or apparent sliding of the units was most likely due to settlement of the units into the saturated clay subsoils, as subgrade locations were often poor due to the saturated conditions, and then the snow and freezing conditions, rather than actual sliding.

4.0 On Site Evaluations by Wenck Associates, Inc.

4.1 VISUAL INSPECTION

Visual observations were made of Hesco barrier installations at 5th Street South, Drain 27, the Fargo Country Club golf course, 40th Avenue South, Timberline addition, and the Harwood Groves area.

The 5th Street Hesco barriers were difficult to inspect for the concerns that were brought up by City staff. The area behind the Hesco barriers and sandbag dike had been filled in with sand after issues were first brought up. During interviews with field personnel it was determined that this area most likely didn't have the sandbag dike tied in sufficiently to the Hesco barriers. It was stated that the sandbags butted up directly to the Hesco units, instead of using a sufficient overlap to adequately protect the transition.

In the Drain 27 area, Hesco barriers were placed on top of an existing earthen levee. Settlement was noted in a section of the earthen levee just east of I-29 on the south side of the drain. Interviews and inspection showed that this appeared to be due to an existing storm sewer running through the existing earth levee and discharging to the drain. After installation of the Hesco barriers and noticeable settlement in part of the dike, it became apparent that a storm sewer structure located within the earthen levee was discharging water through the top of the structure and onto the earthen levee. This leakage completely saturated the area and allowed the Hesco barriers to settle into the earthen levee, as well as causing settlement of the levee itself.

The Fargo Country Club area consisted of Hesco barriers being deployed through the golf course. Evidence of soft soils were noted from the ruts left by equipment used to deploy and fill the Hesco barriers.

The 40th Avenue south area had Hesco barriers installed on top of an existing earthen levee. A concern of the barriers leaning was made during interviews. Measurements showed that the barriers were leaning approximately 3.5" in 4 vertical feet. These barriers have also settled on the water side approximately a ½", and none on the dry side.

The Timberline area consisted of Hesco barriers that were used for primary temporary protection. Barriers were placed in residential backyards along a drainage channel. It was noted that some of the Hesco barriers were leaning. Measurements were made at a few locations, which showed 6.5" of lean in four feet. Field personnel indicated that the barriers were leaning during installation because of the lay of the land, and that they had performed field measurements over a couple of days and determined that the units had not shown any movement.

The Harwood Groves area consisted of Hesco barriers installed in a 2 – 1 configuration (base 2 barriers wide with a single unit placed on top). The Hesco barriers were providing secondary protection is this area. Clay had been placed up to the top of the base units on the backside.

5.0 Technical Evaluations

This section discusses three different traditional failure mechanisms for retaining structures such as the Hesco barriers; sliding, overturning/tipping, and seepage, and the Hesco barriers resistance to them. Within each of this subsections, references to previous studies are introduced and discussed (if available), followed by an independent review.

5.1 SLIDING

Sliding of a retaining system (i.e., the Hesco barriers) is most simply defined by Equation 1, which relates the resisting and driving forces for sliding to the overall factor of safety against sliding. For long-term situations (i.e. permanent walls), it is considered good practice to have a factor of safety (FS) against sliding equal at least 1.5, meaning that the resisting forces are 50% greater than the driving forces. For short-term situations, applicable to temporary flood protection dikes, this acceptable factor of safety is 1.3.

$$FS = \frac{\text{Resisting Forces}}{\text{Driving Forces}} = \frac{(F_v \tan\delta) + cL}{F_h} \qquad \text{(Equation 1)}$$

Where: F_v = Weight of basket (1' "slice" of basket) minus uplift force (lbs/ft)

δ= Interface friction coefficient

c = Cohesion, or undrained shear strength (lbs/ft^2)

L = Length, or basket depth (ft)

F_h = Horizontal force from water (lbs/ft)

5.1.1 Review of Available Information

A report issued for the United States Senate Committee on Appropriations, dated June 29-30, 2004, discussed possible sliding, and is provided in Attachment B for reference. This report discusses the resistance to sliding based on different types of fill soils (fine sand, coarse sand, and gravel), and different types of surfacing materials (earth, concrete, and grass).

Table 1 of the referenced report, shown below, gives the interface coefficients of friction for the fill soils and surfacing materials.

Table 1. Interface friction information

Fill Type	Interface Coefficient of Friction					
	Earth		Concrete		Grass	
	tan δ	δ	tan δ	δ	tan δ	δ
Fine Sand	0.58	30	0.35	19	0.30	17
Coarse Sand	0.67	34	0.45	24	0.35	19
Gravel	0.78	38	0.60	31	0.40	22

Note: tan δ = Tangent (δ) = μ or the friction coefficient.

Using the above information, the authors of the referenced report compiled factors of safety against sliding for 30 different load cases, considering various structure heights, flood heights, fill types, and surfacing materials. This information is provided in Attachment B, but most of the cases are shown again (albeit in a different order) in Table 2 below. Information not included in Table 2 are the cases where the flood height was higher than the structure height, and also cases where the fill material was gravel (because site observations in Fargo noted that only sand was used to fill the Hesco Concertainers).

Table 2. Factor of safety against sliding for various load cases organized by flood height[1]

Case	Flood Height	Structure Height	Surface Type	Fill Type	FS (full uplift)	FS (no uplift)
4	3	3	Concrete	Fine sand	0.8	1.2
5	3	3	Concrete	Coarse sand	1.2	1.6
1	3	3	Earth	Fine sand	1.3	1.9
2	3	3	Earth	Coarse sand	1.7	2.4
7	3	3	Grass	Fine sand	0.7	1.0
8	3	3	Grass	Coarse sand	0.9	1.3
13	3	4	Concrete	Fine sand	1.6	2.0
14	3	4	Concrete	Coarse sand	2.3	2.9
10	3	4	Earth	Fine sand	2.6	3.3
11	3	4	Earth	Coarse sand	3.3	4.2

Case	Flood Height	Structure Height	Surface Type	Fill Type	FS (full uplift)	FS (no uplift)
16	3	4	Grass	Fine sand	1.4	1.8
17	3	4	Grass	Coarse sand	1.8	2.2
22	4	4	Concrete	Fine sand	0.8	1.2
23	4	4	Concrete	Coarse sand	1.2	1.6
19	4	4	Earth	Fine sand	1.3	1.9
20	4	4	Earth	Coarse sand	1.7	2.4
25	4	4	Grass	Fine sand	0.7	1.0
26	4	4	Grass	Coarse sand	0.9	1.3

(1) Red highlighting means the factor of safety is not acceptable (below 1.0).
Yellow highlighting means that the factor of safety is only marginally acceptable (between 1.0 and 1.3)
Blue highlighting means that the factor of safety is acceptable (greater than 1.3) for short-term conditions.

This table indicates that the authors found acceptable or marginally acceptable factors of safety against sliding are achievable for many of the cases analyzed, including **all** of the cases where the flood height was 3 feet, the containers were 4 feet high, and the containers were placed on earth.

5.1.2 Independent Review

The above calculated factor of safety values assume that uplift pressures exist and will reduce the available resisting force. This is a conservative opinion, because it is likely that even if a layer of sand is frozen at the base of the Hesco Concertainer, enough pore-water pressure would be dissipated so as to minimize or negate the resulting uplift pressures (from buoyancy of the structure vs. the underlying soils), simply based on the fact that the drainage path for seepage beneath the unit is no longer than about 3 feet.

An analysis was also completed for a two layer Hesco system (consider a double container base and a single container top) using the same theories as used to develop Table 2 (i.e., forces acting on a one-foot cross-section of barrier). This information is provided in Table 3, below.

Table 3. Factor of safety against sliding for various load cases organized by flood height[1,2]

Case	Flood Height	Structure Height	Surface Type	Fill Type	FS (full uplift)	FS (no uplift)
4	6	6	Concrete	Fine sand	0.5	0.9
5	6	6	Concrete	Coarse sand	0.8	1.2
1	6	6	Earth	Fine sand	0.9	1.4
2	6	6	Earth	Coarse sand	1.1	1.8
7	6	6	Grass	Fine sand	0.5	0.8
8	6	6	Grass	Coarse sand	0.6	1.0
13	7	8	Concrete	Fine sand	0.7	1.1
14	7	8	Concrete	Coarse sand	1.1	1.6
10	7	8	Earth	Fine sand	1.2	1.9
11	7	8	Earth	Coarse sand	1.6	2.4
16	7	8	Grass	Fine sand	0.6	1.0
17	7	8	Grass	Coarse sand	0.9	1.3
22	8	8	Concrete	Fine sand	0.5	0.9
23	8	8	Concrete	Coarse sand	0.8	1.2
19	8	8	Earth	Fine sand	0.9	1.4
20	8	8	Earth	Coarse sand	1.1	1.8
25	8	8	Grass	Fine sand	0.5	0.8
26	8	8	Grass	Coarse sand	0.6	1.0

(1) Red highlighting means the factor of safety is not acceptable (at or below 1.0).
Yellow highlight means that the factor of safety is only marginally acceptable (between 1.0 and 1.3).
Blue highlighting means that the factor of safety is acceptable (greater than 1.3) for short-term conditions.
(2) Overall system is set up as 2 containers on the bottom and one on top.

This table shows that careful engineering is needed before installing such a two-tier system, as acceptable factors of safety are achievable for much fewer cases than the single tier system. These relatively low calculated factors, together with some concerns about the single tier system, especially with flood height equal to barrier height, showed that some actual field testing of the barriers should be done. This was largely due to actual field experience not squaring well with theory.

5.1.3 Field Testing

In response to the concerns raised above, field test analyses were performed on partial sections of the Hesco units to determine the sliding resistance to lateral forces. These analyses were done in Fargo on April 9, 2009, and reported on in a separate technical memo to Hesco Bastion LLC, dated April 30, 2009. Tests on 3' deep by 3' wide by 4' high sections were conducted with the filled units placed on various base surfaces. The total amount of force required to move the unit was recorded, along with the volume and weight of the filled unit. This allowed the actual friction coefficient and factor of safety to be computed in a real-life environment. An independent soils laboratory performed soil analyses on submitted fill samples, and gave a unit weight and gradation of the fill sand for both uncompacted and medium compacted samples (see Tables 4, 5 and 6). [Note: The field tests did not consider overturning, bearing capacity of the underlying soils, or seepage rates of the units.]

Table 4. Field Data Collected

Test Surface	Test #	Hesco Unit ("Basket") Volume	Load Cell Reading	Basket Weight @ 89.5 PCF Sand	Basket Weight @ 102.0 PCF Sand	Calculated Friction Coefficient if Sand is 89.5 PCF	Calculated Friction Coefficient if Sand is 102.0 PCF
		(ft^3)	(lbs)	(lbs)	(lbs)		
Grass	1	48.8	2700	4387	4978	0.62	0.54
Grass - Muddy	2	44.0	3300	3956	4488	0.83	0.74
Grass - Muddy/ Saturated	3	51.4	3400	4621	5243	0.74	0.65
PCC Street	4	46.7	2700	4198	4763	0.64	0.57
PCC Street	5	49.8	2600	4477	5080	0.58	0.51

Notes: Weights of sand are from laboratory tests on samples obtained during field testing – 89.5 PCF is average uncompacted, and 102.0 PCF is average of compacted samples to approximately 88% Standard Proctor.
PCF = Pounds per Cubic Foot
PCC = Portland Concrete Cement

Table 5: Summary of Factor of Safety Calculations for Water 3' High Against 3' x 3' x 4' High Baskets

	Forces Resisting Sliding				Force Causing Sliding	
Test Surface	Basket Weight (γ_s x ~~V~~)	F_{Uplift} (γ_w x H x $w^2/2$)	μ	F_R	F_w (γ_w x H^2 x w)/2	Factor of Safety
	(lbs)	(lbs)		(lbs)	(lbs)	
Grass	4387	842	0.58	2056	842	2.44
Grass - Muddy	3956	842	0.78	2429	842	2.88
Grass - Muddy/Saturated	4621	842	0.70	2645	842	3.14
PCC Street	4338	842	0.58	2028	842	2.40

Table 6: Summary of Factor of Safety Calculations for Water 4' High Against 3' x 3' x 4' High Baskets

	Forces Resisting Sliding				Force Causing Sliding	
Test Surface	Basket Weight (γ_s x ~~V~~)	F_{Uplift} (γ_w x H x $w^2/2$)	μ	F_R	F_w (γ_w x H^2 x w)/2	Factor of Safety
	(lbs)	(lbs)		(lbs)	(lbs)	
Grass	4387	1123	0.58	1893	1497	1.26
Grass - Muddy	3956	1123	0.78	2209	1497	1.47
Grass - Muddy/Saturated	4621	1123	0.70	2449	1497	1.63
PCC Street	4338	1123	0.58	1865	1497	1.24

1) Summary of Calculated Friction Coefficient Data

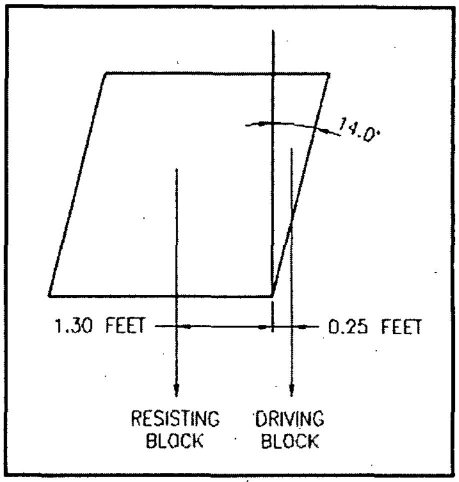
Friction coefficients were calculated for each of the field tests performed.

The field data showed significantly higher friction coefficients than the original engineering calculations, which used published friction coefficients for the different base materials. This is believed to be due to the deformation of the bottom edge of the basket, which was observed as it began to slide. This deformation cannot be discounted, however, as it would occur in the event that the lateral loads applied to the basket were enough to cause lateral movement. Therefore, the field measured friction coefficients are believed to be valid for the specific situations in which the baskets were tested.

These higher friction coefficients, in turn, show that the actual performance of the Hesco units in resisting sliding is higher than the calculated resistance using published friction coefficients, as shown by the factors of safety calculated in Tables 5 and 6.

5.2 OVERTURNING/TIPPING

Traditional methods of overturning analysis are not truly applicable for this type of container because of the general deformability of the system, plus the possibility that uplift pore-water pressures can often be dissipated through the sandy infill material. Even if a layer of sand is frozen at the base of the Hesco Concertainer, enough pore-water pressure would be likely dissipated so as to minimize or negate the uplift pressures, simply based on the fact that the drainage path is no longer than about 3 feet.



Therefore, the overturning and tipping will most likely be related to either 1) installation issues (where infill material is placed in a manner such that initial container tilt occurs), or 2) thaw of the subsoil on only one side of the container occurs, such that some differential settlement occurs (e.g., rising water on one side of barrier thaws the soil beneath one side).

As discussed earlier in this report, some of the units were experiencing some tilt, with angles nearing 6 or 7 degrees. Based on information obtained during the field work, it is believed that the units showing some tilt were either installed that way, or settlement of the units into the base soils occurred.

For a single layer system, a tilt of less than 14 degrees is a reasonable maximum value. The reason for this is because the system tends to operate as a block. Therefore, at an angle of 14 degrees or less, there is at least 7 times the mass holding back the container from tipping. When considering this angle, the resisting mass and the associated moment arm of the units, based on the resisting and driving forces, a calculated system factor of safety against overturning is greater than 30. This is shown by Equation 2 below. Overturning or tipping is not considered to be a significant problem, therefore, unless the entire subgrade fails.

$$FS = \frac{\text{Resisting Forces}}{\text{Driving Forces}}$$

$$FS = \frac{\text{Resisting Block Area} \times \text{Resisting Moment Arm} \times \text{Density}}{\text{Driving Block Area} \times \text{Driving Moment Arm} \times \text{Density}} \qquad \text{(Equation 2)}$$

5.3 SEEPAGE

Field personnel's input on the issue of seepage by the Hesco barriers varied greatly. Some thought that the seepage was excessive, while others thought it was less than a traditional sandbag levee would be. Most areas had poly-sheeting placed on the wet side of the Hesco units. However, an area of Drain 27 did not receive poly-sheeting, and the field staff thought that the seepage wasn't excessive and was easily managed.

Assessment of actual seepage rates in the field were not part of this report.

5.3.1 Review of Available Information

In 2004, the USACOE conducted tests on Hesco units in regards to seepage rates. These initial tests showed higher seepage rates than other levee systems. Most of the seepage occurred through the seams between adjacent units. Hesco learned after these test that the end panels on adjacent units should be removed to decrease the amount of seepage. Retesting of the units for seepage rates was conducted by USACOE in July and August, 2005. In this retest, the end panels of units butted up against on another were removed. This allowed for a continuously filled sand unit with no gaps between units. This retest showed seepage rates of 0.04 gpm/ft at 1' of head, and 0.14 gpm/ft and a head of 2.85'

6.0 Summary and Recommendations

Overall, the consensus of opinion among users of the Hesco barriers for the Fargo floodfight is that the barriers are well-designed, and were vital to the success of the effort to contain the flooding from the Red River of the North. They were appreciative of the speed of deployment (vital in emergency situations such as this), their ability to adapt to irregular subgrades, and the uniformity of results compared to sandbag dikes. Some cautions oftentimes repeated were to be careful with proper filling of the barriers, and to pay particular attention to the subgrade the barriers are placed on, as this can cause significant problems. Additionally, transitions between Hesco's and other types of dikes need to be done carefully, allowing an adequate overlap to prevent a weak spot in the resulting dike. Adequate monitoring of the completed barrier wall must be done, just as for any temporary dike, throughout the emergency period. Most users, especially those who used them in the field, declared they would use them again, given the same situation.

Some useful recommendations were made, however, and should be considered by Hesco Bastion.

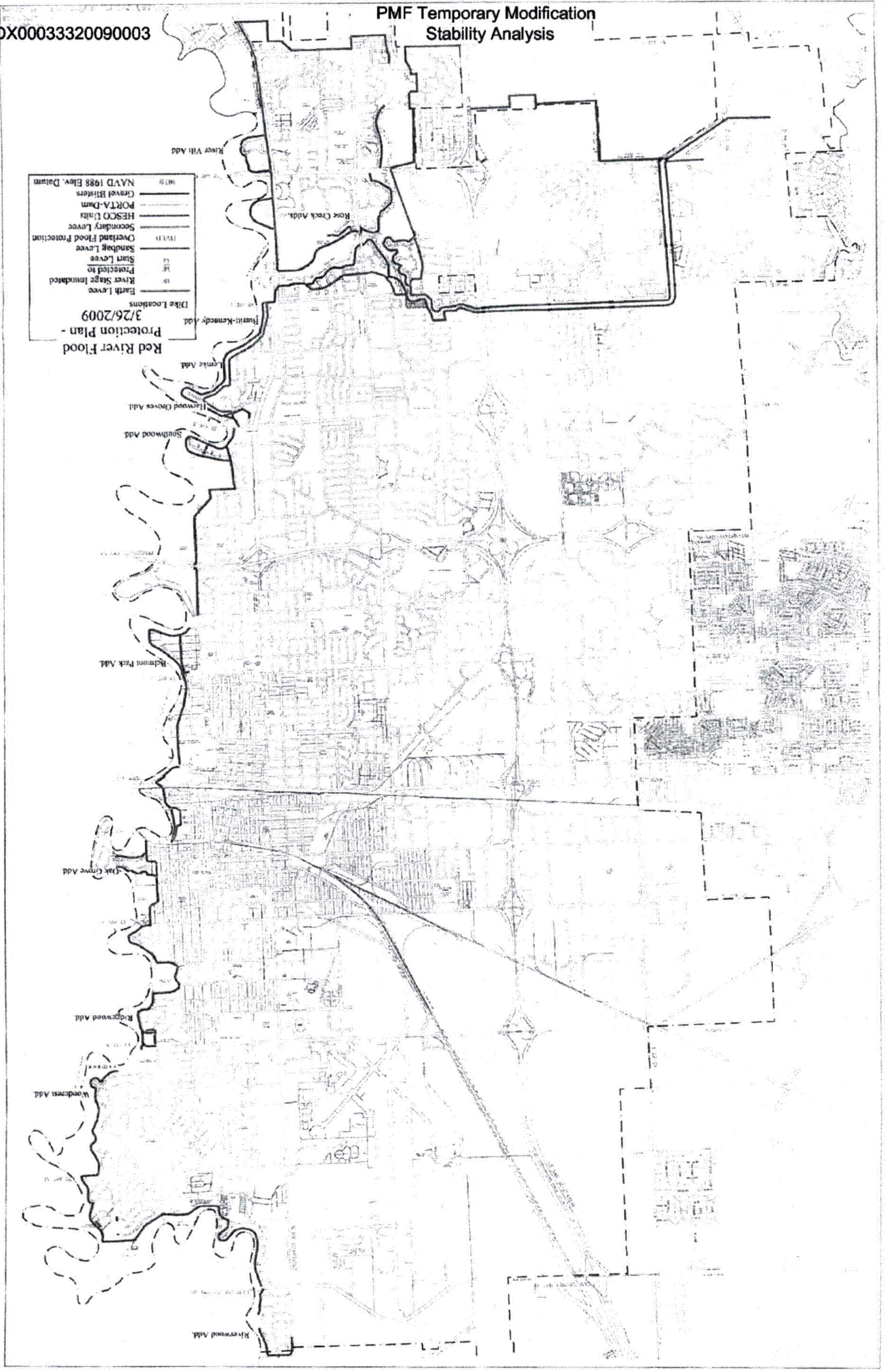
- Consider use of **colored** hinge pins to join the units together. This would make the visual inspection of finished units easier and faster, particularly at night (i.e., Were the baskets properly joined during installation?).

- Additional training. Several users reported receiving only very minimal training in how to properly install the units. This caused considerable problems and delays in getting the various installations properly started, especially as new workers arrived to help.

- Preparation of a Guidance Document for communities considering using the Hesco barriers. Like most products being considered to fight a flood, proper engineering needs to be done prior to installing them. Such a guidance document could be given to communities prior to their using Hesco's, recommending the type of engineering needed, the considerations that need to be made, and procedures to follow for such things as needed site preparation, height of barriers needed for the predicted flood elevations and their configuration (e.g., 2-4' barriers with 1-4' stacked over them, or 2-4' with 2 more 4' barriers stacked over them), lessening seepage with plastic sheeting and how to do it (front face of barrier, back face, how anchored, etc.), joining Hesco barrier walls to sandbag or clay dikes (necessary overlap, tie-ins, etc.), and proper installation procedures.

Attachment A

Red River Flood Protection Plan – 3/26/2009
(Map of Levee System)


Red River Flood
Protection Plan -
3/26/2009
Dike Locations
Earth Levee
River Stage Inundated
Protected to
Stan Levee
Sandbag Levee
Overland Flood Protection
Secondary Levee
HESCO Units
PORTA-Dam
Gravel Blisters
NAVD 1988 Elev. Datum
River Vlh Add
Rose Creek Adds.
Harwood Groves Add.
Southwood Add.
Belmont Park Add.
Oak Grove Add.
Ridgewood Add.
Riverwood Add.

Attachment B

"Engineering Analysis" from Report to:
United States Senate Committee on Appropriations,
June 29 – 30, 2004

ENGINEERING ANALYSIS

The ability of the Concertainer® structure to withstand hydrostatic and uplift forces, as well as other forces, results primarily from a combination of shape and weight of the structure and the frictional resistance generated along its base. The linkages between the units also allows for the load on a single unit to be distributed over several adjacent units. The structure is compliant and deforms slightly as a response to applied loads. This particularly important when the structure responds to uplifting forces. The Concertainer® basket is basically a shell and will experience almost no uplifting forces. Since the basket is open at the bottom, if the unit is raised the fill material remains in contact with the ground surface. The uplifting force on the fill will be due to buoyancy and not from any mechanical force of the basket. Therefore, the conventional analysis of stability based upon overturning is not applicable to the Concertainer® structure. However, because the basket and fill could be displaced laterally, the analysis of the stability of the structure to sliding is appropriate.

The ability of the structure to resist lateral forces it can be theoretically analyzed based upon the assumption that the structure will respond as a rigid body to hydrodynamic forces. A general load case is shown in Figure 1.

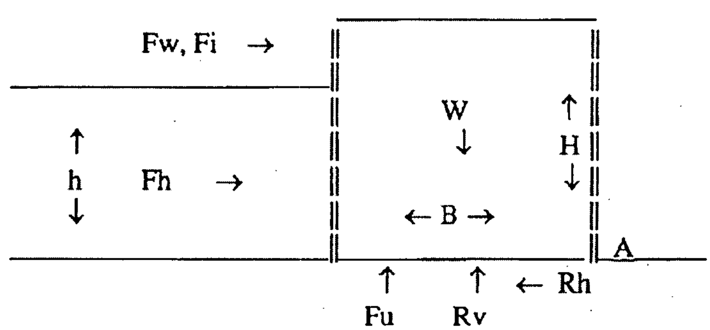


Figure 1. Schematic diagram of forces on a Concertainer® unit.

The Figure illustrates that the force per foot of structure on the Concertainer® can result from several sources:

W = weight of the basket and fill
Fh = hydrostatic pressure force
Fw = wave forces
Fi = impact force
Fu = uplift force
Rv = vertical reaction force of the soil
Rh = horizontal reaction force of the soil, with a maximum value equal to Cf Rv, where Cf is the coefficient of friction along the interface

The formulas for the static forces for the load case shown in Figure 1 and their lines of action from point A, are as follows:

$W = \frac{1}{2} \gamma_{fill}\ B\ H$	@ B/2
$Fh = \frac{1}{2} \gamma\ h^2$	@ h/3
$Fu = \frac{1}{2} \gamma\ B\ h$	@ 2/3 B
$Rv = W - Fu$	@ 1/3 B
$Rh = Fh$ with maximum value of Cf Rv	

Where:

B = width of the Concertainer®
H = height of the Concertainer®
γ_{fill} = unit weight of the fill or S γ
γ = unit weight of water
S = specific gravity of the fill
h = height of water above the base of the structure

The resistance to sliding can be expressed as a factor of safety, which is the ratio of the resisting forces to the applied forces. The horizontal resisting force is the frictional resistance generated along the base of the structure, given by Cf Rv. The applied hydrostatic force is Fh. Thus factor of safety against sliding can then be defined by

$$SF = Cf\ Rv/Fh \ = \ Cf(W\text{-}FU)/Fh$$

The analysis presented is based upon treating the structure as a rigid body, the Concertainer® is actually deformable and it would affect the impact loads and the overturning. The Concertainer® is highly resistant to impact loads because the basket and fill deform when the load is applied, thus lengthening the time over which the impacting object is stopped, and hence reducing the force. The amount of deformation would depend upon the where the impact occurred, with more deformation occurring near the top of the structure.

The Concertainer® structure is well suited to resist impact loads. The structure is compliant such that it will deform under loads. This property means that a unit will absorb debris loads and actually experience a lower force from debris than rigid structures would experience for the same debris. This can be explained because debris loads resulting from floating objects such as vegetation, logs and lumber are impact loads. In an impact load the force produced by the impacting object depends upon the initial momentum of the object, its mass time its velocity, and the time over which the objects velocity is reduced to zero by the impact; that is its deceleration. The compliancy of the structure thus extends the time over which the impacting object is stopped. This results in a reduced deceleration and hence a reduced force on the structure. The performance of the structure under debris loads would also depend upon the water depth relative to the top of the structure, the fill in the structure and the shape of the debris object. Impact tests for specific objects of interest for various fill types would need to be conducted. The effect of debris loads on the performance of the Concertainer® can be accounted for by including impact loads in the analysis of the factor of safety against sliding.

Waves can affect the Concertainer® structure in several different manners: as an additional horizontal force, as a carrier of debris, and as a mechanism for removing material from the structure. The effects of waves will depend upon whether the waves hitting the structure are non-breaking, breaking or broken waves. The horizontal force on a structure produced by each type of wave can be computed from standard coastal engineering design procedures, e.g., the Shore Protection Manual, and included in the analysis of the resistance of a structure to sliding for various unit sizes and types of fill. Debris loads could be severely increased under wave action. While the movement of water resulting from a current will be generally parallel to the structure, wave action causes water movement that is more generally perpendicular to the structure. The velocity of the water at the crest of a breaking wave approaches the phase speed of the wave and, even in shallow water, can reach a value of several feet/sec. Thus velocity of the debris could be greatly increased by the presence of waves. The property of the Concertainer® to absorb impact loads clearly becomes an advantage in resisting this wave enhanced threat from debris. The effect of waves on the erosion of material from the structure will depend upon the height of the mean water at the structure, the height and type of wave hitting the structure, and the fill material. When the combined mean water height and incoming wave height is lower than the top of the structure, no erosion would occur. For mean water levels below the top of the structure, but with wave height high enough to overtop the structure, erosion would be minimal. Water would be thrown onto the top of the fill with little horizontal velocity and wet the fill. For higher waves, waves that break into the structure, there would be some initial suspension and transport of fill out of the structure. When the mean water height exceeds the height of the structure so that it becomes submerged all types of waves would suspend some fill material. The amount of fill removed would depend upon the intensity of the wave action and the type of fill. These various effects of wave action on the structure would need to be considered in the selection of the Concertainer® size and fill so as to maintain an acceptable factor of safety against sliding under expected field conditions.

GROUND SURFACE PERFORMANCE

The performance of the Concertainer® on various surfaces will depend both on the type of surface and the type of fill used in the structure. This is because the same fill will interact differently with different surface materials. The net effect of the surface/fill interaction can be expressed through the interface friction coefficient. As shown above, the friction coefficient directly affects the resistance of the structure to sliding. Other factors that may need to be considered concerning the surface upon which the structure is placed are the permeability of the surface and it's bearing capacity. Given the test conditions described in the solicitation, the bearing capacity and permeability of the test surfaces should present no problems. However, in actual usage, these issues would need to be investigated at each field site.

The actual coefficient of friction between different fill materials and the different test surfaces will depend upon the detailed characteristics each. Since these are not known at this time, representation values of the friction coefficient can be taken from published values. The following values were used in the stability analysis:

Table 1. Soil parameters used in the analysis.

Fill Type	Specific Gravity	Interface Coefficient of Friction		
		Earth	Concrete	Grass
Fine Sand	1.60	.58	.35	.30
Coarse Sand	1.76	.67	.45	.35
Gravel	1.92	.78	.60	.40

The coefficients of friction between concrete and for various fill types are taken from the Shore Protection Manual (Table 7-15 and 7-16). Table 7-16 gives the friction coefficients for concrete dams on sand and gravel. For freshly graded surfaces, earthen material is present both in the container and on the surface. The friction resistance will depend upon the angle of internal friction for the each material. The values used in the analysis for the various fills on an earthen surface are based upon the angles of internal friction for firmly packed sediments as given in Table 7-15 of the SPM. For the grass surface case, the approach taken is that the coefficient of friction will be assumed to be smaller than for a concrete surface. Thus the concrete values were reduced for gravel, coarse sand, and fine sand by factor of .67, .77 and .86 respectively.

FIELD REPAIR AND MAINTENANCE

Depends on the location.

TEST CONDITION ANALYSIS

The performance of the Concertainer® under a particular set of test conditions can be determined using the formulas presented above. Various load cases were considered based upon the type of surface at the test site, the height of the floodwater, the size of the structure and the fill material. The results of these calculations are given in Table 2.

A single load case will be used to illustrate the methodology used in computing the factor of safety against sliding. The structure will be assumed to be placed on either grass, earth or concrete. A 3 foot by 3 foot unit will be subject to a 3 foot flood, with no waves or impact loads. The structure will be fill with either fine sand, coarse sand, or gravel. The formula for the factor of safety against sliding for a 3 foot Concertainer® unit (b=H=3 feet) as

$$FS = Cf\,(W\text{-}Fu)/Fh$$

or

$$FS = Cf\,(HBS\gamma\text{-}hB\gamma/2)/(h^2\gamma/2) = 2BCf\,(HS\text{-}h/2)/(h^2)$$

This can be simplified for H = B = 3 ft, and h=3 feet to

$$FS = .67\,Cf\,(3S - 1.5)$$

For the various fill materials and surface types the values of Cf and S can be specified. For example, for an earthen surface and with a fine sand fill, S= 1.60 and Cf = .58. The computed factor of safety is

$$FS = .67\,(.58)\,(3(1.60) - 1.5) = 1.28$$

This is the result shown in Table 2 for load case 1. For coarse sand S = 1.76 and Cf = .67, and the resulting factor of safety is 1.69, as shown in Table 2 for load case 2. For gravel, S = 1.92 and Cf = .78, and the factor of safety is 2.22, as shown in Table 2 as load case 3.

The other load cases listed in Table 2 were based upon changing the surface types, flood water depth and unit size. A second set of calculations were performed based upon increasing the flood water depth to 4 feet, and placing a 2 foot by 2 foot Concertainer® on top of a 3 foot by 3 foot unit. The factors of safety against sliding for different surfaces are given in load cases 28, 29 and 30.

Overall the analysis indicates that for the fill types and surface types considered, large changes in the factor of safety can occur. For example, for a 3 foot by 3 foot unit on a concrete surface the factor of safety changes from .77 to 1.13, to 1.70 as the fill is changed from fine sand, to coarse sand and then to gravel.

Table 2. Factor of safety against sliding for various load cases.

Load Case	Surface Type	Structure Hgt	Flood Hgt	Fill Type	Factor of Safety Against Sliding
1	E	3'	3'	FS	1.28
2	E	3'	3'	CS	1.69
3	E	3'	3'	GR	2.22
4	C	3'	3'	FS	.77
5	C	3'	3'	CS	1.13
6	C	3'	3'	GR	1.70
7	G	3'	3'	FS	.66
8	G	3'	3'	CS	.88
9	G	3'	3'	GR	1.14
10	E	4'	3'	FS	2.53
11	E	4'	3'	CS	3.30
12	E	4'	3'	GR	4.28
13	C	4'	3'	FS	1.52
14	C	4'	3'	CS	2.22
15	C	4'	3'	GR	3.29
16	G	4'	3'	FS	1.31
17	G	4'	3'	CS	1.72
18	G	4'	3'	GR	2.20
19	E	4'	4'	FS	1.28
20	E	4'	4'	CS	1.69
21	E	4'	4'	GR	2.22
22	C	4'	4'	FS	.77
23	C	4'	4'	CS	1.13
24	C	4'	4'	GR	1.70
25	G	4'	4'	FS	.66
26	G	4'	4'	CS	.88
27	G	4'	4'	GR	1.14
28	E	4'	5'	FS	1.07
29	E	4'	5'	CS	1.41
30	E	4'	5'	GR	1.85

Note: E = Earthen surface
C = Concrete surface
G = Grass surface
FS = Fine sand fill
CS = Coarse sand fill
GR = Gravel fill

The data presented herein by HESCO Bastion USA, LLC from the Rapid Deployment Flood Wall Testing at Engineering Research and Development Center (ERDC) Water Experimental Station (WES) in preliminary information from Dr. Joseph Suhayda.

HESCO Bastion USA, Inc.
47152 Conrad E. Anderson Drive
Hammond, LA 70401
Tel 985-345-73..
Email: info@hesco-usa.com
Web: www.hesco-usa.com

MEMORANDUM

TO: RECIPIENTS OF TECHNICAL REPORTS AND INFORMATION

The information provided by Hesco herewith is intended solely to provide general guidance to a purchaser or potential purchaser of its products, who accepts full responsibility for the engineering and other design, installation and use of structures incorporating the Hesco Concertainer and associated products. While reasonable care has been taken to ensure that the information provided is accurate and has been obtained from reliable sources, and the information is provided in good faith based upon that which is available at the time of production, Hesco provides no guarantee or warranty as to the accuracy, completeness or effectiveness of the information. Nothing herein shall be construed as a substitute for the need for the purchaser to exercise or employ adequate independent technical expertise and judgment. Purchaser acknowledges that risks and dangers may arise from foreseeable and unforeseeable causes and assumes all risk and danger and all responsibility for any losses and/or damages to person or property that may result from purchaser's use of Hesco's products. HESCO PROVIDES NO GUARANTEE OR WARRANTY, WHETHER EXPRESS OR IMPLIED BY LAW, IN CONNECTION WITH ITS SALE OR THE THIRD PARTY INFORMATION PROVIDED HEREWITH, INCLUDING WITHOUT LIMITATION MERCHANTABILITY OR FITNESS FOR A PARTICULAR PURPOSE, EXCEPT AS EXPRESSLY STATED IN ITS STANDARD TERMS AND CONDITIONS OF SALE.

Attachment 2

Excerpts from USACE ERCD Report
(Reference 2.2)

Environmental aspects

The only material used (sand) is considered to be nonhazardous and nontoxic, so there were no exposure hazards during these tests.

If the floodwater is contaminated with bacteria or pollutants, the sand fill inside the bags also may be contaminated. The sandbag itself should provide some filtering protection, especially for nonwater-soluble and small contaminants such as floating oil, but water-soluble contaminants would likely seep into the sand fill.

Hesco Bastion Concertainer® Levee Tests

Design

Hesco Bastion Concertainer® (hereinafter referred to as "Hesco®"), listed under U.S. Patents 3333970, 5472297, and European Patent 046626, is a structural system of linked baskets containing fill material. Hesco® systems have been used around the world for military operations as well as for combating natural disasters (Hesco 2004). The corporate Web site is http://www.hesco-usa.com.

The units (Figure 2-44) are manufactured in various sizes and are made of welded galvanized steel mesh that is assembled with coiled joints. A polypropylene nonwoven geotextile liner retains the fill material (sand, gravel, or other fill) that is dumped into the open (top and bottom) basket using minimal labor and commonly available equipment. The baskets are flat-packed on pallets, extended and joined with joining pins, filled with fill material, and stacked in various configurations depending on the end-use. The units are lightweight, portable, and are easily handled.

Engineering analysis of the system was provided by Hesco®, and listed the ability of the structure to withstand hydrostatic and uplift forces. The ability of the structure to resist lateral forces was analyzed based on the assumption that the structure will respond as a rigid body to hydrostatic forces. A free-body diagram of the hydrostatic forces showed the resistance to lateral sliding on a concrete floor with a given water height of 3 ft and a coarse-grained fill material.

A test-condition analysis for a 3-ft by 3-ft unit on a concrete floor subjected to a 3-ft-high flood was given for various load cases with given basket and fill weights, given sand unit weight, vertical and horizontal reaction forces, hydrostatic pressure force, and uplift force. Assuming an interface coefficient of friction between coarse sand and concrete floor of 0.45, the safety factor against lateral sliding was calculated to be 1.13 (Load Case 5). No floor anchoring system was accounted for, and no floor anchoring was planned for the ERDC tests.

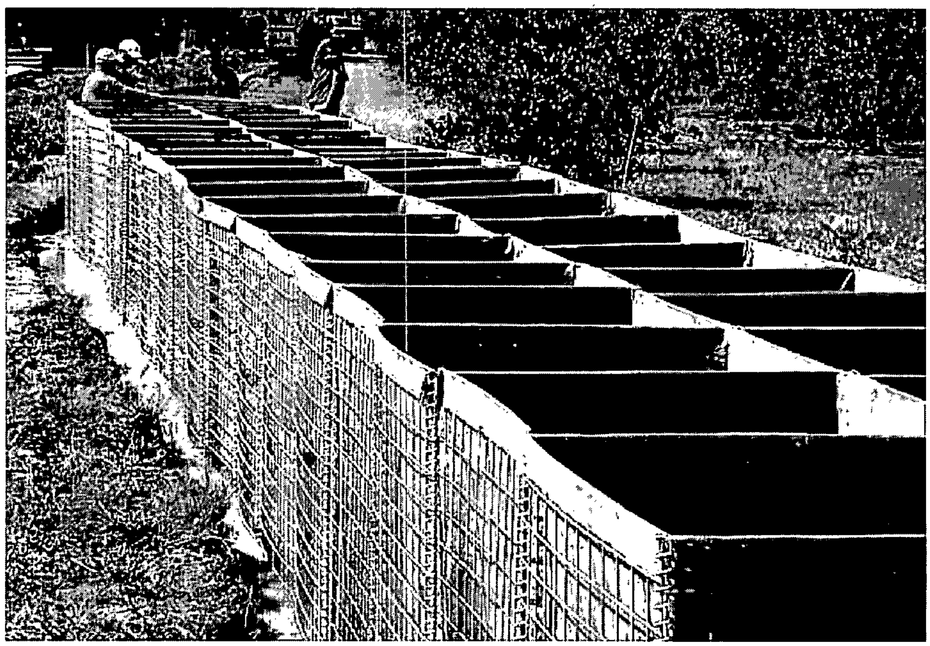


Figure 2-44. Hesco Bastion Concertainer® basket units, assembled and empty

For the ERDC tests, the Hesco® Flood Unit system (General Services Administration (GSA) No. GS-07F5369P) was furnished, with unfolded unit dimensions of 3 ft height by 3 ft depth by 12 ft width, and commercial price of $295 per unit (approximately $25 per linear foot). End panels (3 ft × 3 ft × 3 ft), connecting joining pins (3 ft) and connecting coil hinges (3 ft) were also furnished. The wire mesh, joining pins, and coil hinges were manufactured from 8-gauge steel and coated with a proprietary galvanizing. Wire mesh size was 3 in. by 3 in. The nonwoven geotextile liner was GEOTEX® 641. Fill sand was provided by ERDC (delivered price of $7 per cubic yard) and was classified as poorly graded sand (USCS "SP") with approximate moisture content of 6 percent.

Construction

Layout of the Hesco® levee built at the ERDC test facility is shown in Figure 2-45.

The stacked units were shipped to the laboratory on a wooden pallet. Construction commenced on 4 May 2004. Relatively cool ambient air temperatures (approximately 60 to 70 deg) provided comfortable working conditions inside the hangar.

Personnel needed to construct the levee included a Hesco® supervisor and four laborers unfamiliar with the product. A 5-min training session commenced (Figure 2-46), the supervisor handed out gloves to the workers, and they began unloading and expanding the units onto the concrete floor (Figure 2-47).

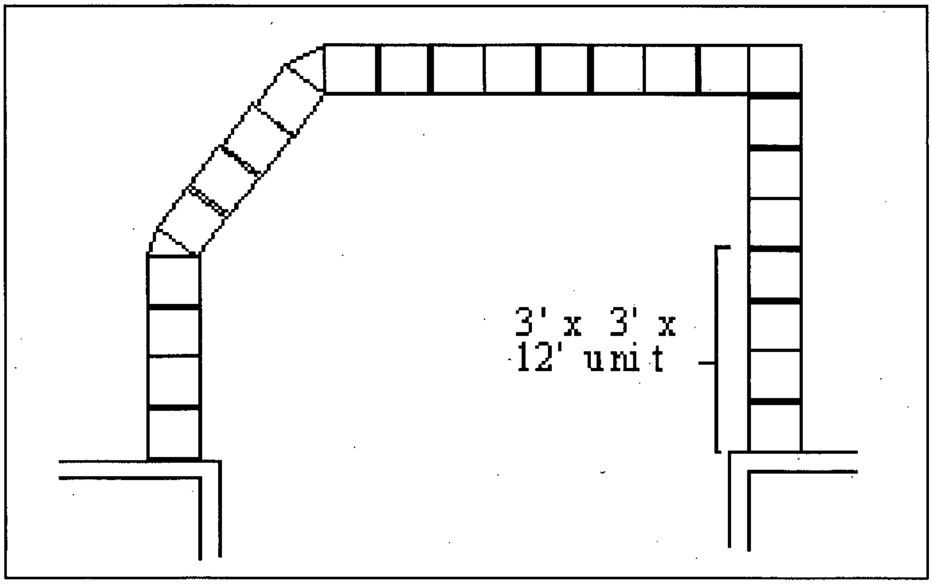


Figure 2-45. Hesco® levee layout

Figure 2-46. Training session for Hesco® assembly team

Figure 2-47. Expanding and positioning units

The expanded units were sequentially positioned on the layout footprint, and the coil hinges were fastened together with the joining pins (Figure 2-48). At angled connections (the intersection of the left and center walls), the supervisor folded and attached end panels to achieve proper unit geometry (Figure 2-49), and the workers continued pinning the units together. Nylon cable ties were also used for securing units together at critical locations determined by the supervisor (Figure 2-50). Initial treatments at concrete wall abutments were also installed (Figure 2-51). Total installation time for offloading, laying out, aligning, and connecting the levee structure was 60 min (approximately 1 lft/min).

The next construction phase consisted of filling the units with sand and completing the installation. The bottom flaps were flattened against the concrete floor (Figure 2-52). A front-end loader top-dumped sand into each unit (Figure 2-53). The supervisor and four workers continued securing the units, filling with sand, compacting, and leveling sand within the units with shovels while the sand-fill operation was ongoing, until all units were full and leveled (Figures 2-54 through 2-57). Approximately 24 cu yd of sand was required to fill the units.

No floor anchoring system was used at the concrete wall abutment connections. To seal the joint between the unit and the concrete wall abutment, expandable foam was dispensed into the joint by the supervisor (Figures 2-58 and 2-59).

Total installation time for the Hesco® levee was 3.5 hr (approximately 3.4 min per linear foot of levee). Labor required was a six-man crew (total 20.8 man-hours), and equipment required was a Cat® 916 front-end loader, sand, and aerosol foam. On a linear foot basis, the construction required 20.8 man-hours per 62 lft (measured along the protected toe), or 0.3 man-hours per linear foot.

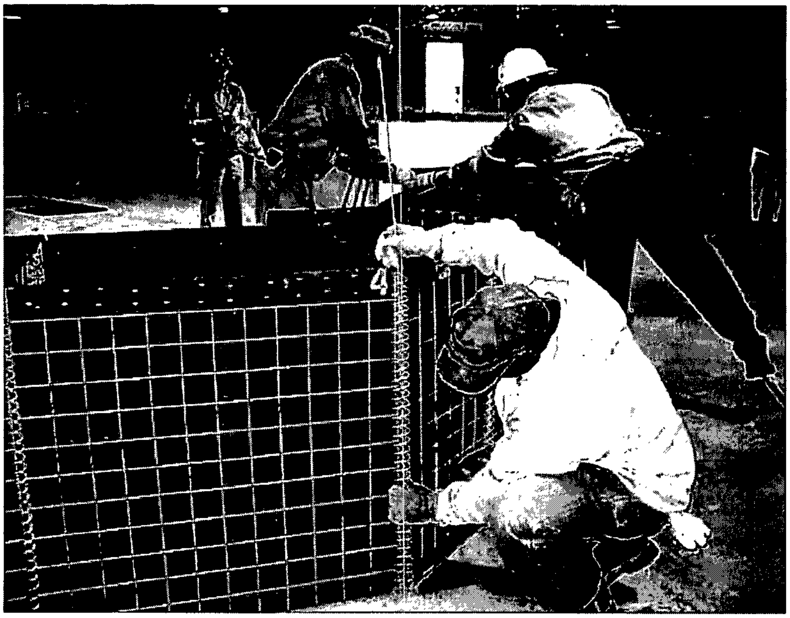


Figure 2-48. Pinning units together

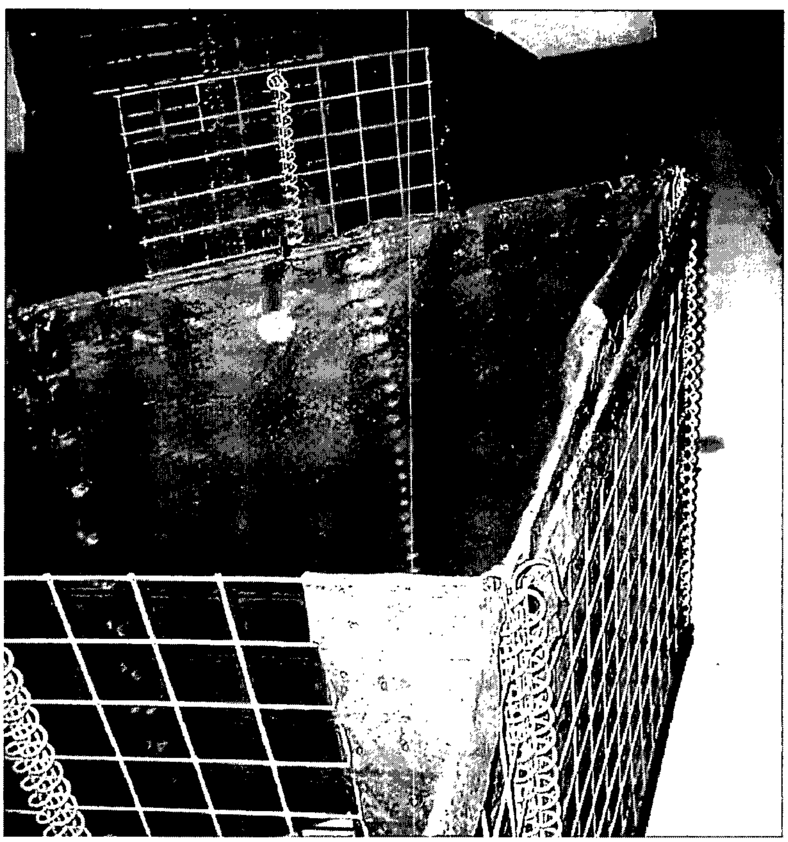


Figure 2-49. Top view of angled unit at intersection of left and center walls

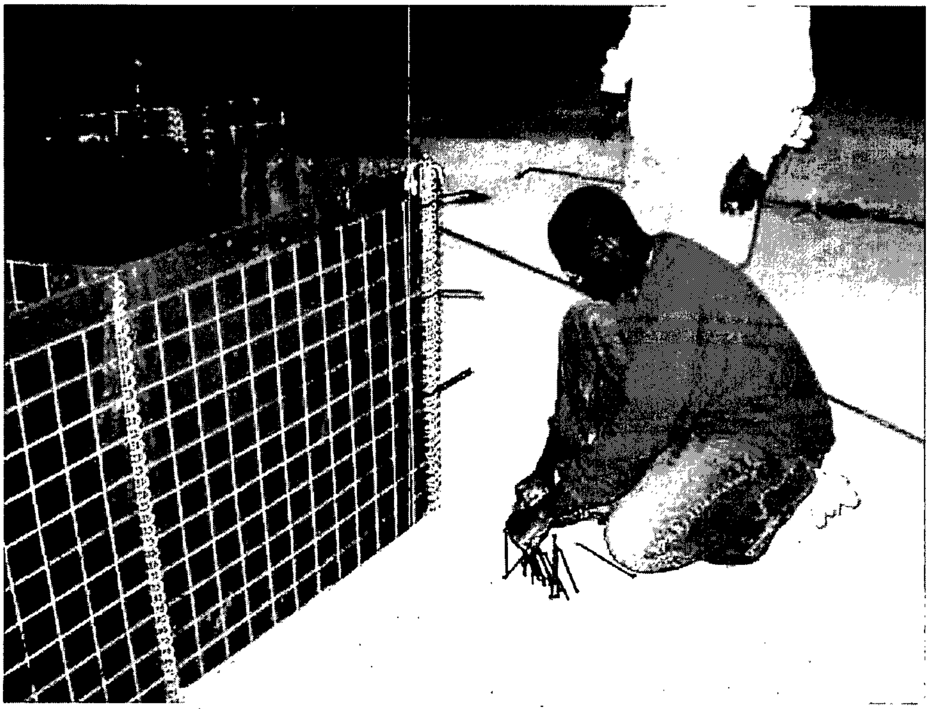


Figure 2-50. Cable ties at joint connections

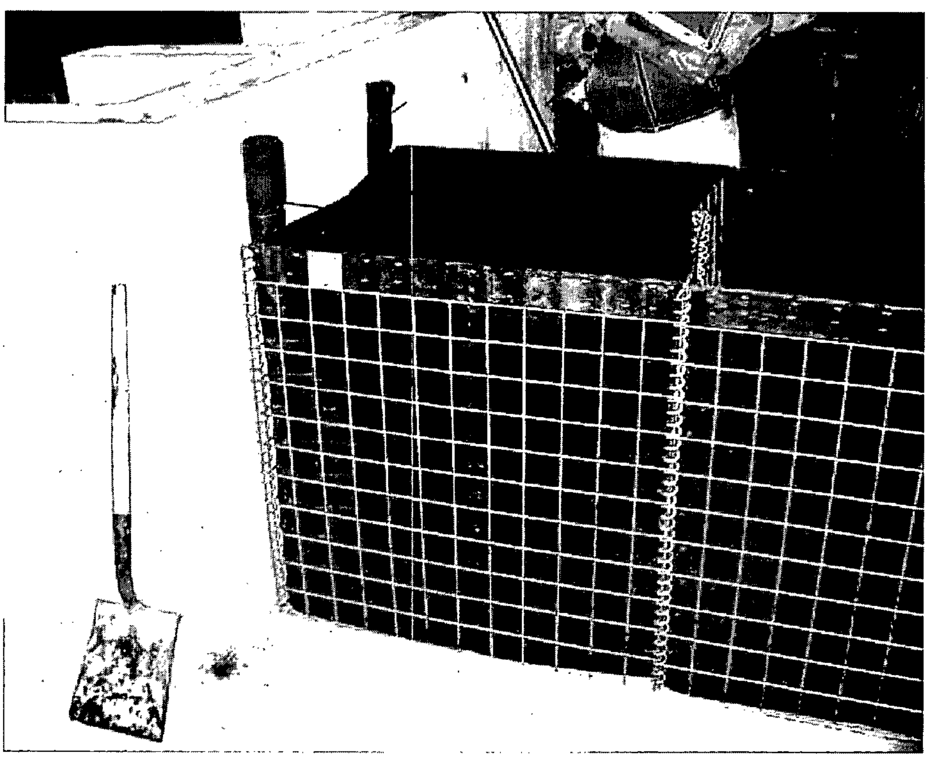


Figure 2-51. Right concrete wall abutment

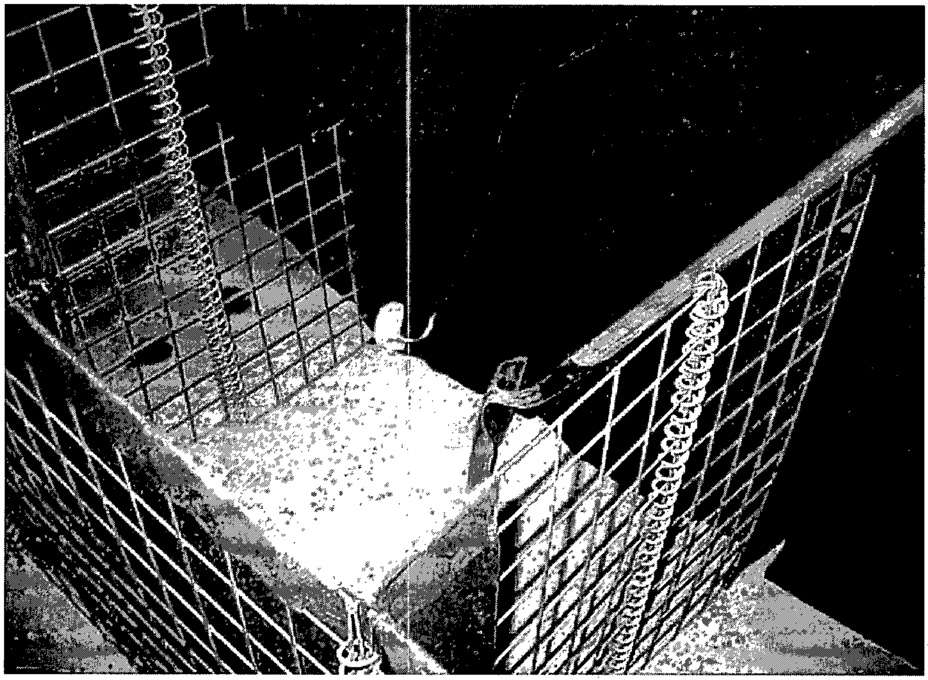


Figure 2-52. Securing flaps against concrete floor. Note center coils which are prefastened at factory

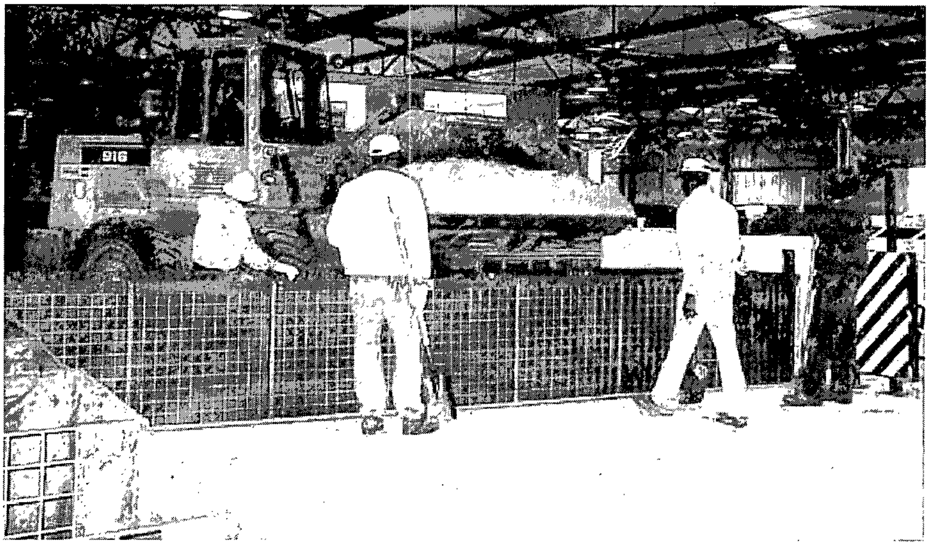


Figure 2-53. Filling with sand

Figure 2-54. Shoveling sand into unit

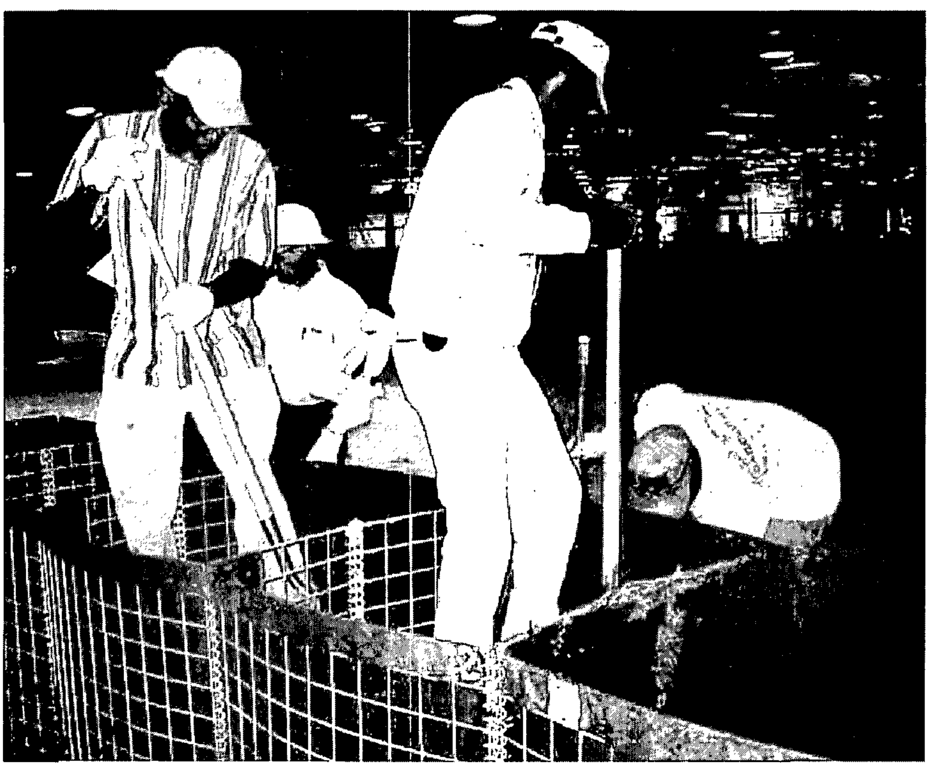


Figure 2-55. Leveling and compacting sand within each unit

Figure 2-56. Filled with sand, view from left concrete wall abutment

Figure 2-57. View from pool side

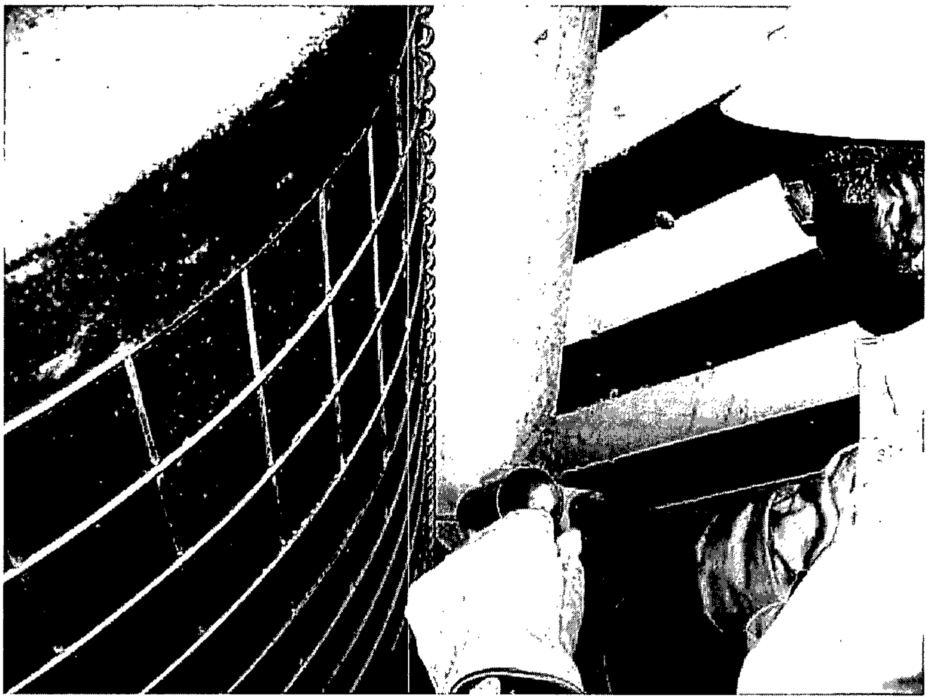


Figure 2-58. Sealing concrete wall abutment with aerosol foam

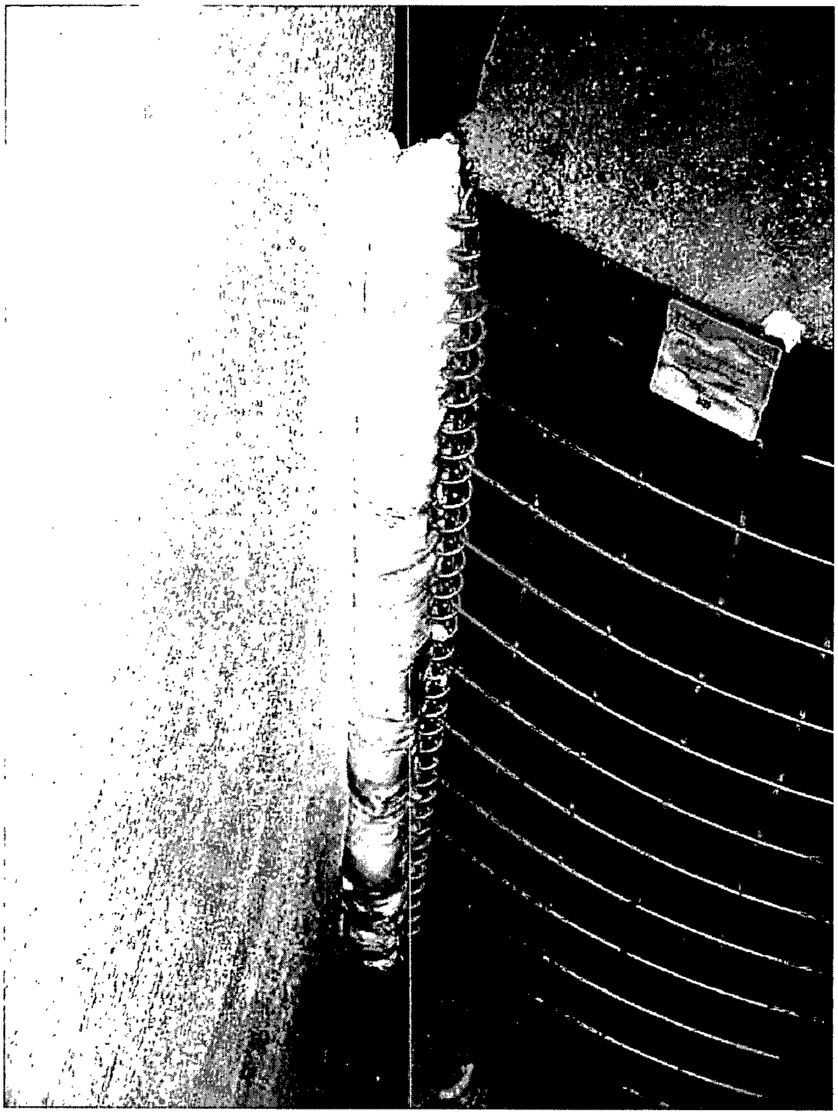


Figure 2-59. Expanded foam at abutment with concrete wall

Prior to filling the reservoir to begin the hydrostatic tests, laser targets were positioned in the levee walls and sealed with expandable foam (Figure 2-60). The completed structure was instrumented with the center-wall displacement monitoring system and was readied for static testing (Figure 2-61). The vendor representative agreed in writing that the levee had been constructed properly and was ready for testing.

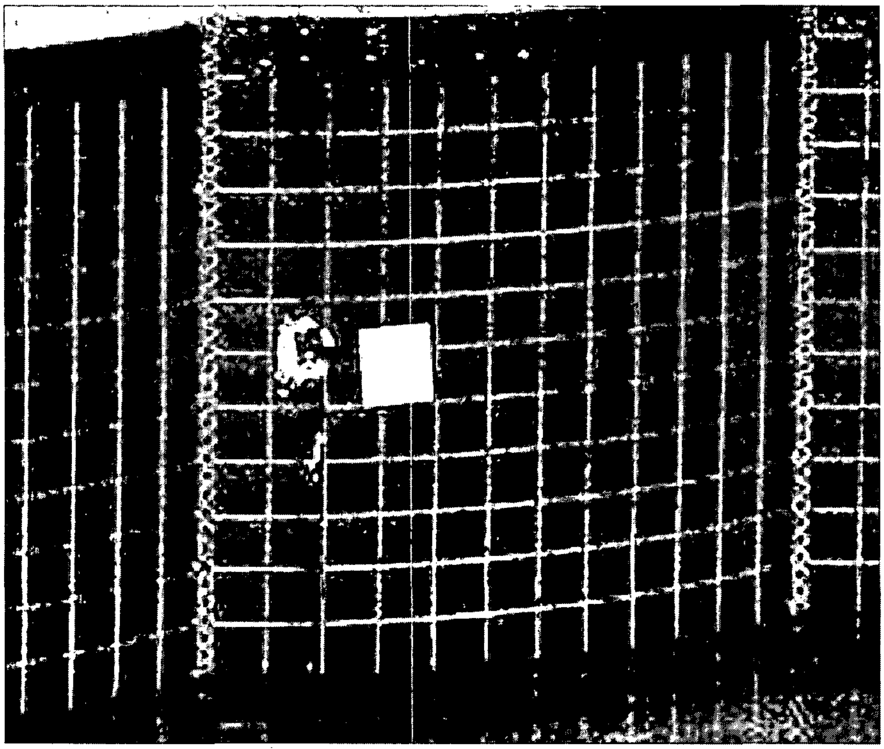


Figure 2-60. Laser target

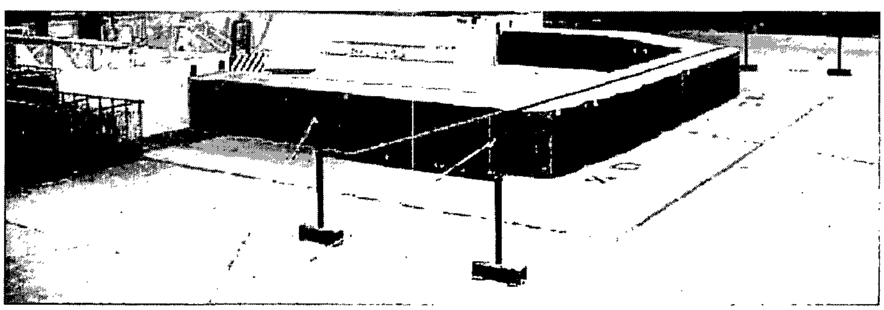


Figure 2-61. Center wall displacement monitoring system

Performance

Testing of the Hesco barrier began after construction was completed and was documented in the same manner as testing of the sandbag structure. Three minor repairs were allowed within seven windows of opportunity during the tests, as described in Appendix C. After the overtopping test, one final repair (or rebuild) was allowed prior to the impact tests.

Disassembly and removal of the barrier was performed after testing was completed and the test basin was drained. An environmental evaluation was also performed for the barrier system, to assess environmental hazards of construction and disposal.

Hydrostatic head tests

The pool elevation was raised to three different elevations for a minimum of 22 hr at each predetermined elevation. During the testing period, levee movement and seepage values were recorded. During and after each test the levee was inspected for weakness and/or failure before the pool elevation was raised to the next level.

Hydrostatic head test, 1-ft reservoir (33 percent height). The water level in the reservoir on the pool side of the levee was raised to a height of 1 ft (33 percent of the levee height). Seepage flow rate ranged from 0.36 to 0.42 gpm/lft (Figure 2-62), and no displacement was observed. Most of the flow rate was observed coming from the wall corners, and the vertical joint between unit ends.

Figure 2-63 shows the wetting front observed on top of the structure as the water saturated the dry sand. Figure 2-64 is a close-up of seepage occurring at a vertical joint between units.

Hydrostatic head test, 2-ft reservoir (66 percent height). The water level in the reservoir on the pool side of the levee was raised to a height of 2 ft (66 percent of levee height). Seepage flow rate ranged from 0.90 to 0.97 gpm/lft (Figure 2-65), and no displacement was observed. Most of the flow was observed coming from the wall corners and the vertical joint between unit ends. Figure 2-66 shows the structure from the front.

Hydrostatic head test, 3-ft reservoir (95 percent height). The water level in the reservoir on the pool side of the levee was raised to an approximate height of 34 in. (95 percent of levee height). Seepage flow rate ranged from 1.76 to 1.86 gpm/lft (Figure 2-67). Lateral displacement ranged from 3 to 9 mm. Vertical deformation was observed to range from 0.24 to 2.28 in., and was assumed to be a result of units "barreling" as the sand became completely saturated. Most of the flow was observed coming from the wall corners and the vertical joint between unit ends.

Hydrodynamic tests

The testing protocol specified that packets of monochromatic waves with a wave period T = 2.0 sec would be generated to impact the levee hydrodynamically. Tests were performed at two different pool elevations (66 percent and 80 percent of levee height). At the 66 percent height, 3-in. waves (measured from trough to crest) were generated continuously for a period of 7 hr. Waves ranging from 7 to 9 in. were then allowed to impact the structure a total of 30 min (three10-min intervals with 15 min calming periods between). Next, wave heights ranging from 10 to 13 in. were allowed to impact the structure for 10 min. The water was then raised to a level of 80 percent levee height and the tests were repeated. At the end of each 10-min increment of wave testing (excluding the 7 hr of 3-in. waves), the testing basin was stilled for up to 45 min to allow the waves to dissipate.

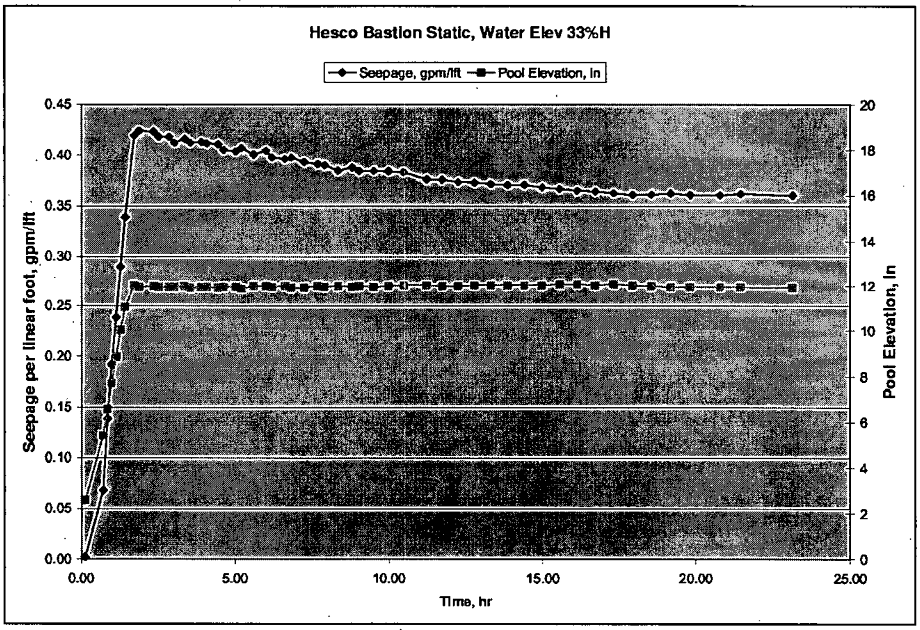


Figure 2-62. Seepage-flow rate per linear foot at 1-ft pool elevation (33% H)

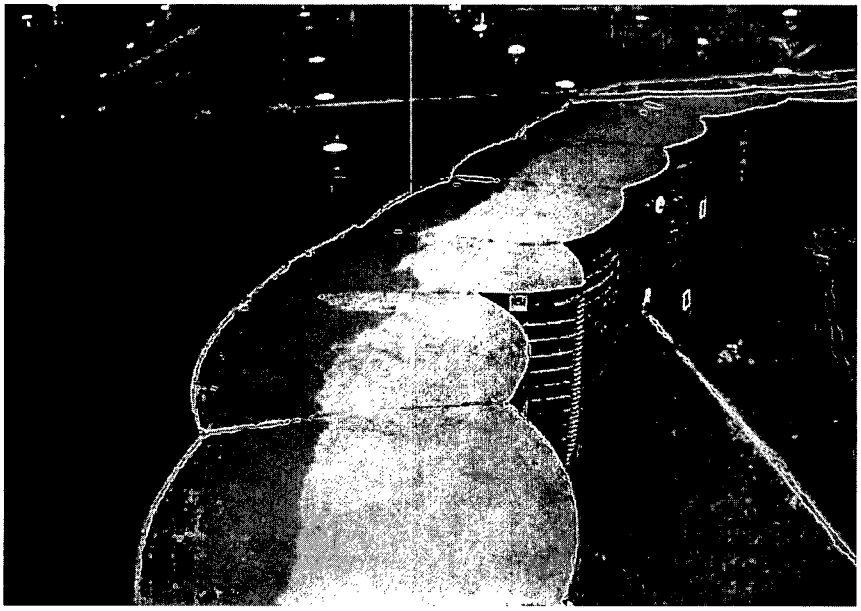


Figure 2-63. View of left wall water saturation

Figure 2-64. Close-up of seepage through vertical joint between units

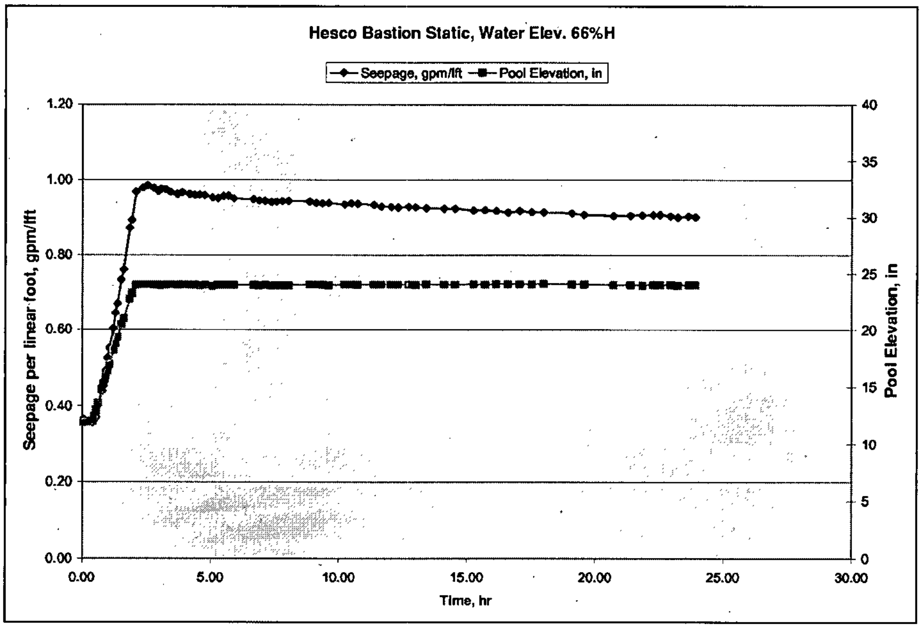


Figure 2-65. Seepage flow rate per linear foot at 2-ft pool elevation (66% H)

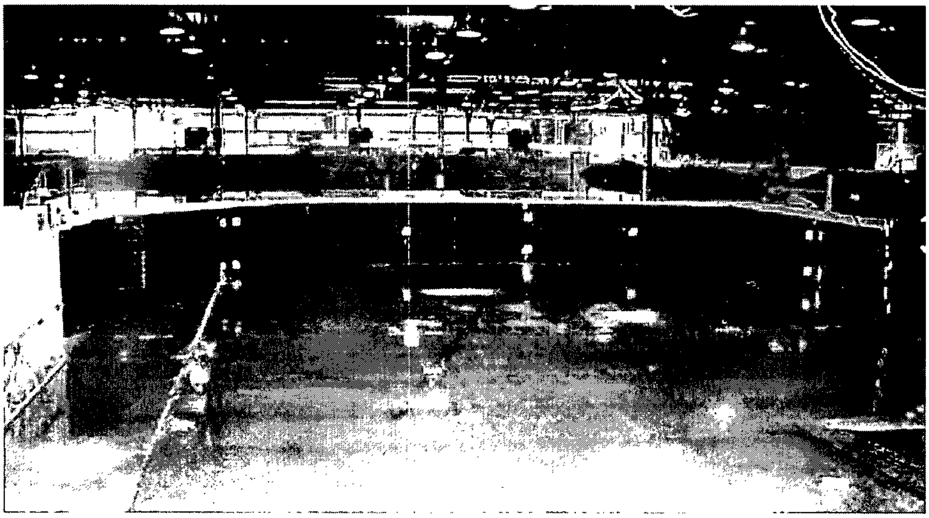


Figure 2-66. View from front

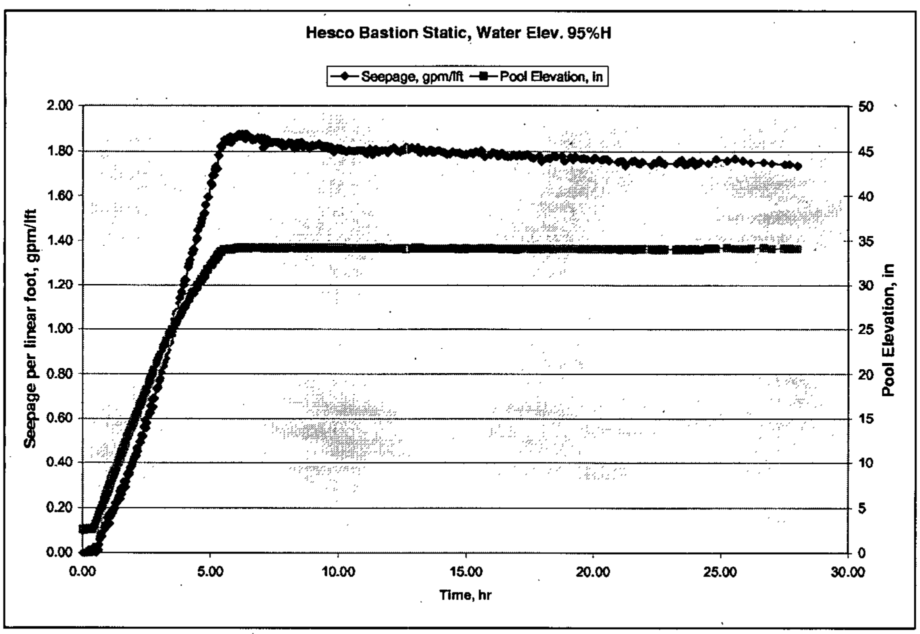


Figure 2-67. Seepage flow rate per linear foot at 95 percent pool elevation

3-in. wave test, reservoir level at 66 percent levee height. The water level in the reservoir of the levee was lowered from the 95 percent level to a height of 24 in. within an interval of about 2 hr. The wave generator was activated and the waves began to impact the levee. Flow rate was observed to range from 0.81 to 0.83 gpm/lft (Figure 2-68), with no displacement. No wave overtopping was observed. Figure 2-69 is a view of the left wall and center wall intersection showing seepage at the wall base.

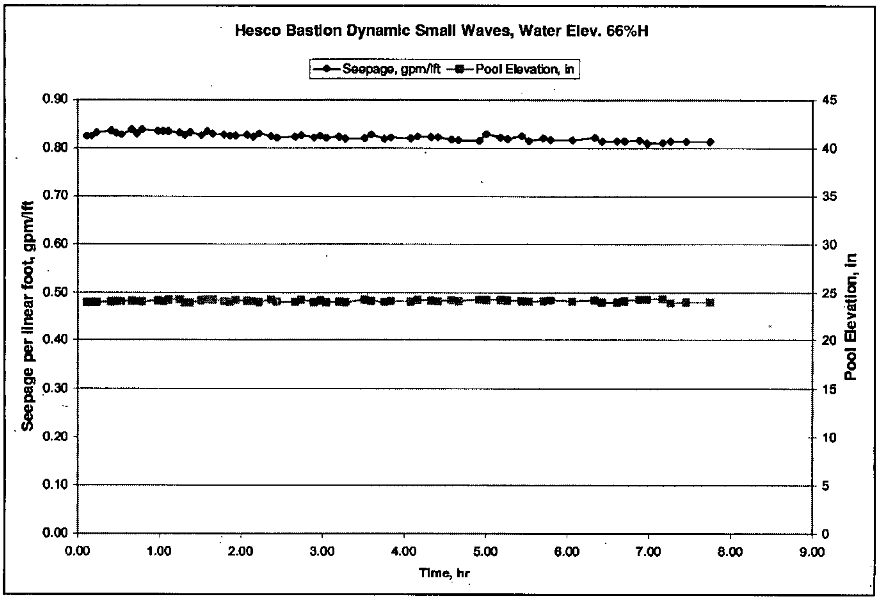


Figure 2-68. Seepage flow rate per linear foot, small wave at 66 percent pool elevation

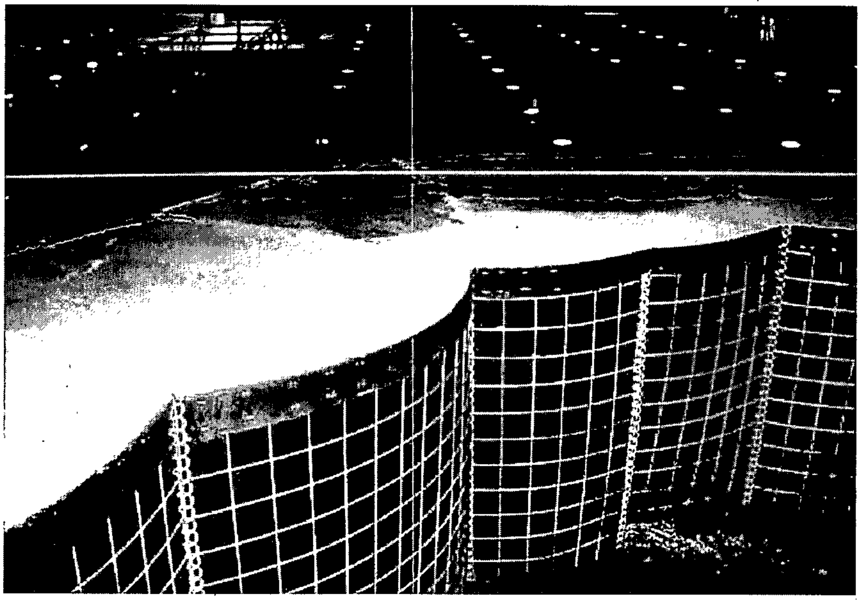


Figure 2-69. Left wall and center wall intersection

7- to 9-in. wave test, reservoir level at 66 percent levee height. The water level in the reservoir on the pool side of the levee was held at a height of 24 in., the wave generator was activated, and the waves began to impact the levee. Flow rate was observed to subside within a range of 0.77 to 0.78 gpm/lft (Figure 2-70), with no levee displacement. No wave overtopping was observed.

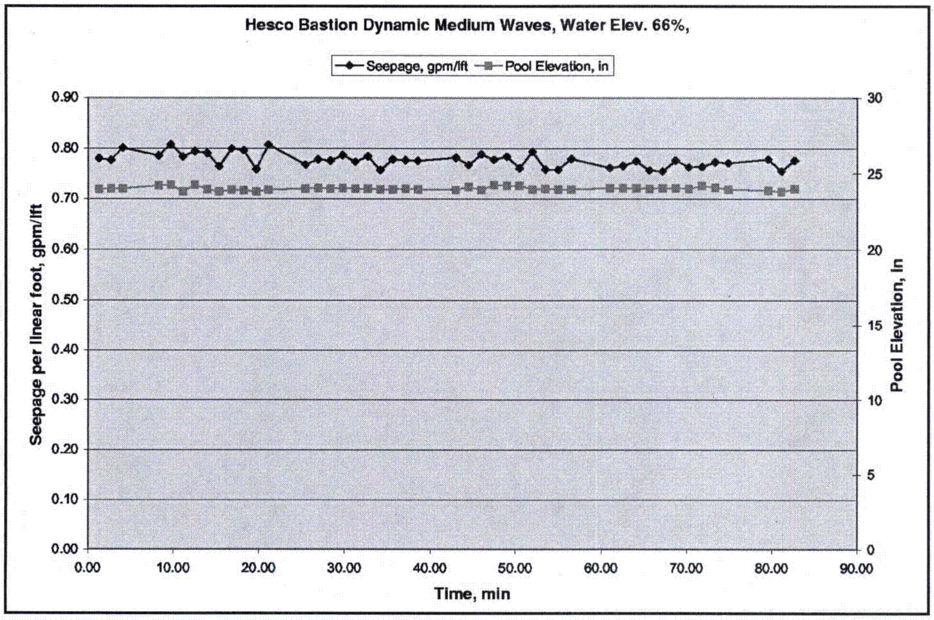


Figure 2-70. Seepage flow rate per linear foot, medium wave at 66 percent pool elevation

10- to 13-in. wave test, reservoir level at 66 percent levee height. The water level in the reservoir on the pool side of the levee was held at a height of 24 in., the wave generator was activated, and the waves began to impact the levee. Flow rate was observed to range from 0.78 to 0.98 gpm/lft (Figure 2-71), with no displacement. Minor sporadic wave overtopping was observed, primarily along the center wall (Figure 2-72).

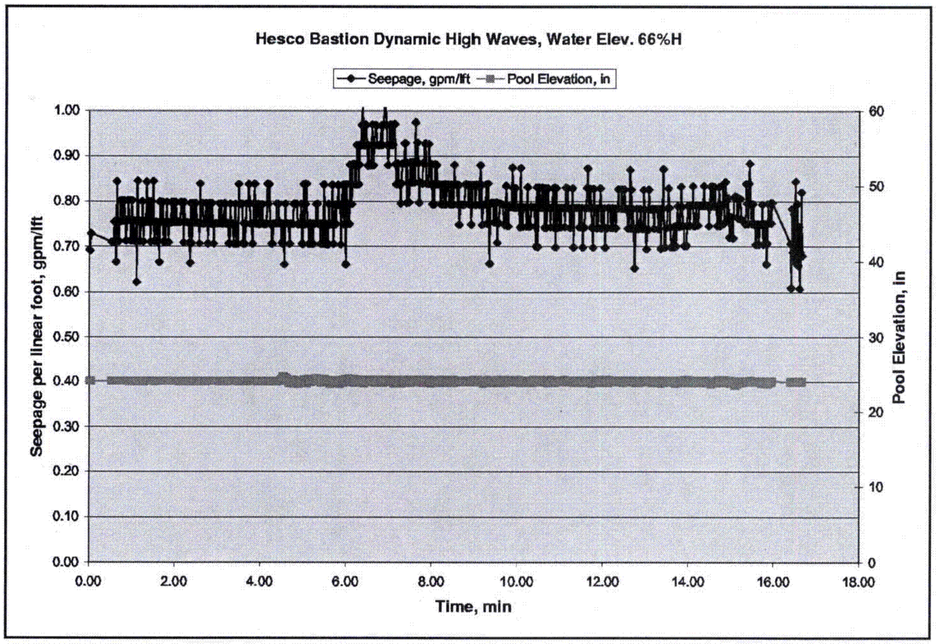


Figure 2-71. Seepage flow rate per linear foot, high wave at 66 percent pool elevation

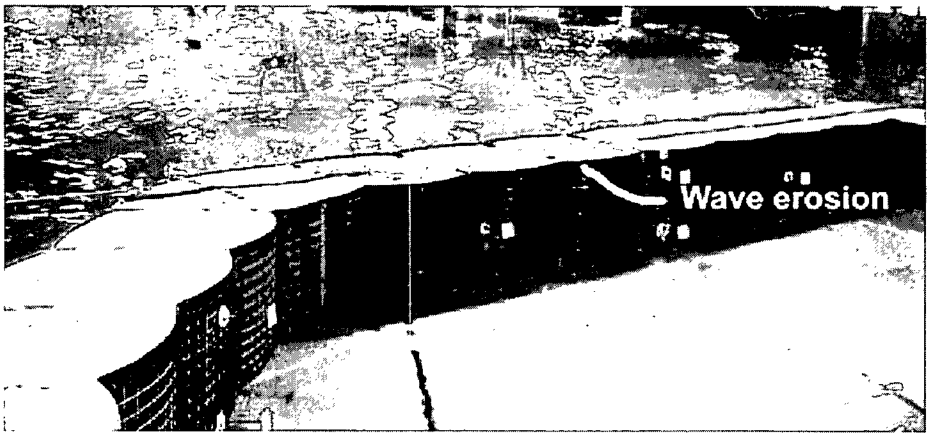


Figure 2-72. Center wall wave-induced erosion

At the conclusion of the test, sand had eroded and settled from the top of the center wall (Figure 2-73), and a solution was devised to prevent further erosion during subsequent testing. As shown in Figures 2-74 and 2-75, a tarp covering was placed on the wall top and secured with cable ties.

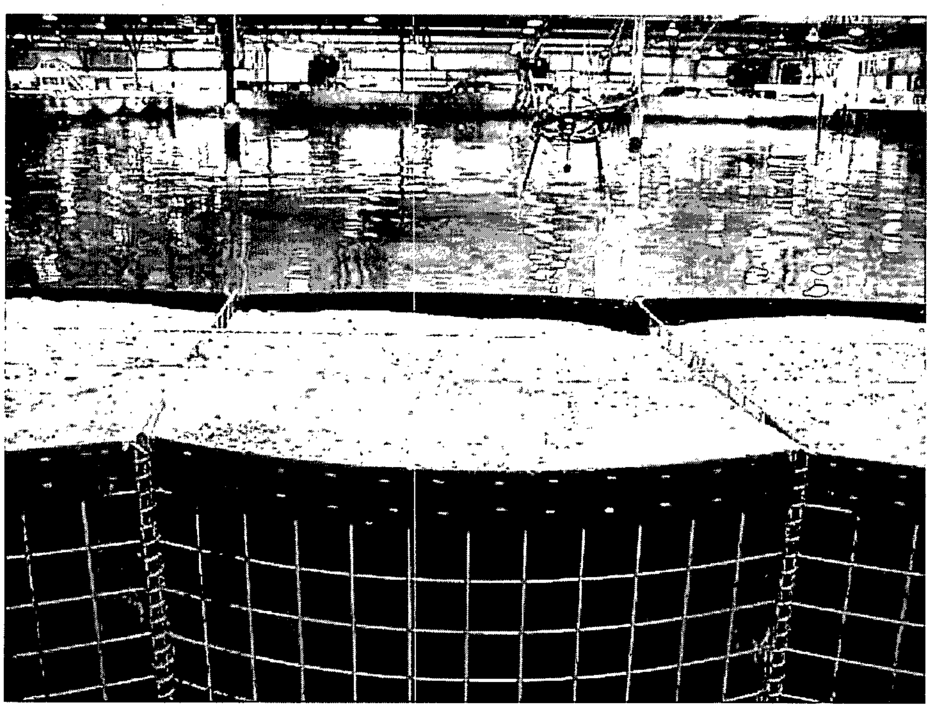


Figure 2-73. Sand eroded from top of center wall

Figure 2-74. Covering top of wall with tarp to prevent further erosion

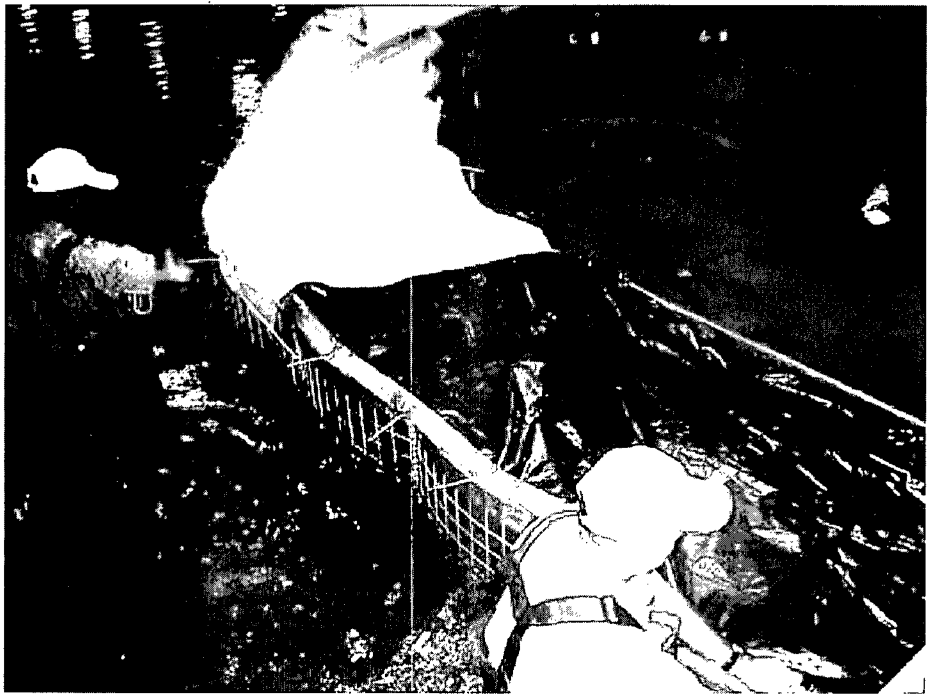


Figure 2-75. Securing with cable ties

3-in. wave test, reservoir level at 80 percent levee height. The water level in the reservoir on the pool side of the levee was raised to a height of 29 in., the wave generator was activated, and the waves began to impact the levee. Flow rate was observed to range from 1.03 to 1.04 gpm/lft (Figure 2-76), with no displacement. No wave overtopping was observed. Figure 2-77 shows seepage under the center wall base.

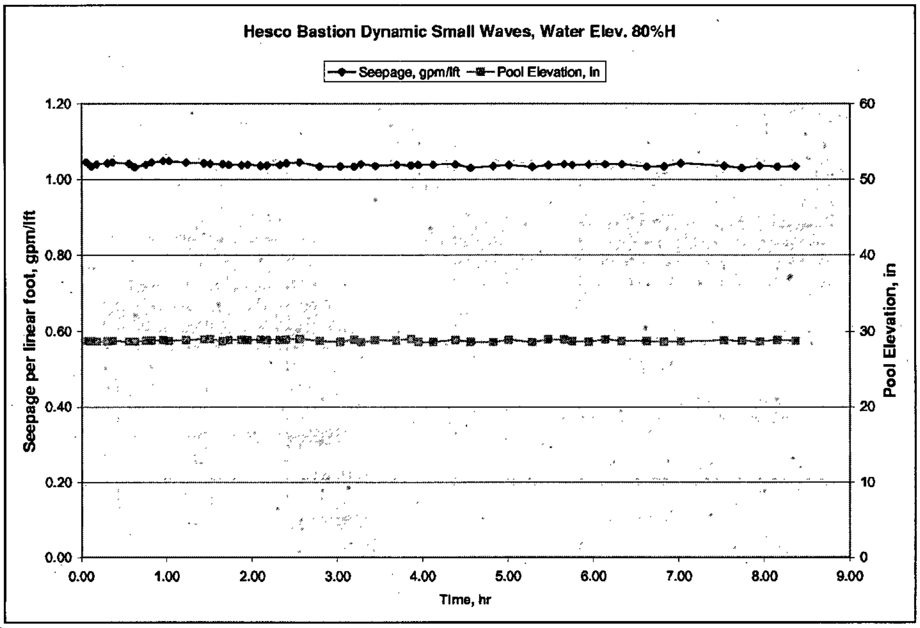


Figure 2-76. Seepage rate per linear foot, small wave at 80 percent pool elevation

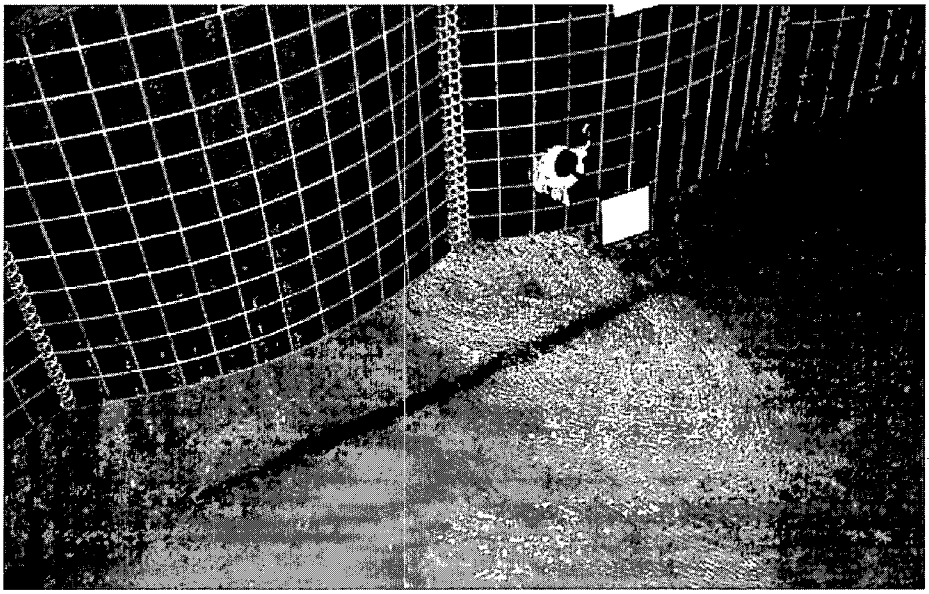


Figure 2-77. Seepage at vertical joint and wall base

7- to 9-in. wave test, reservoir level at 80 percent levee height. The water level in the reservoir on the pool side of the levee was held at a height of 29 in., the wave generator was activated, and the waves began to impact the levee. Flow rate was observed to range from 1.03 to 1.07 gpm/lft (Figure 2-78), with no displacement. No wave overtopping was observed. Figure 2-79 shows a view of the structure.

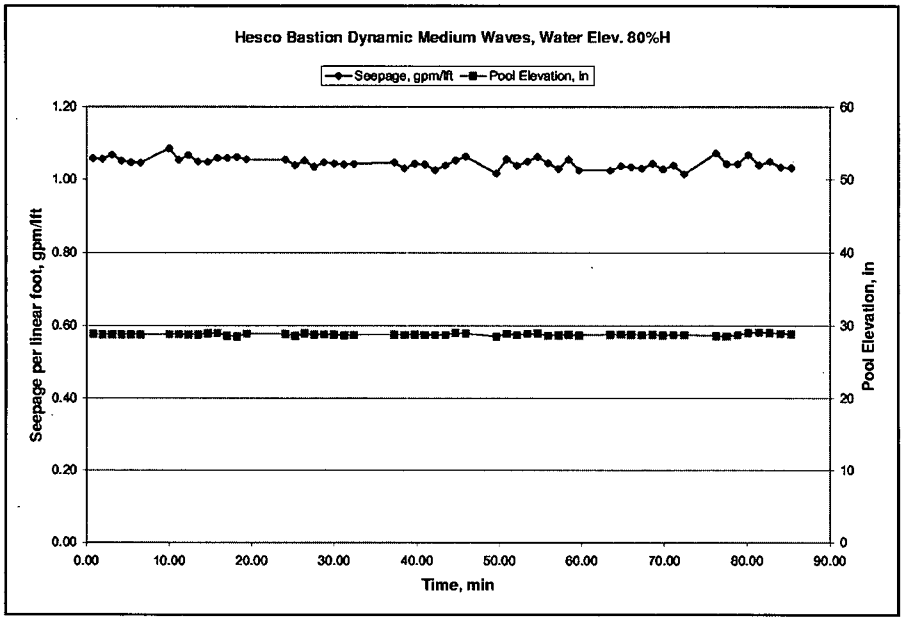


Figure 2-78. Seepage flow rate per linear foot, medium wave at 80 percent pool elevation

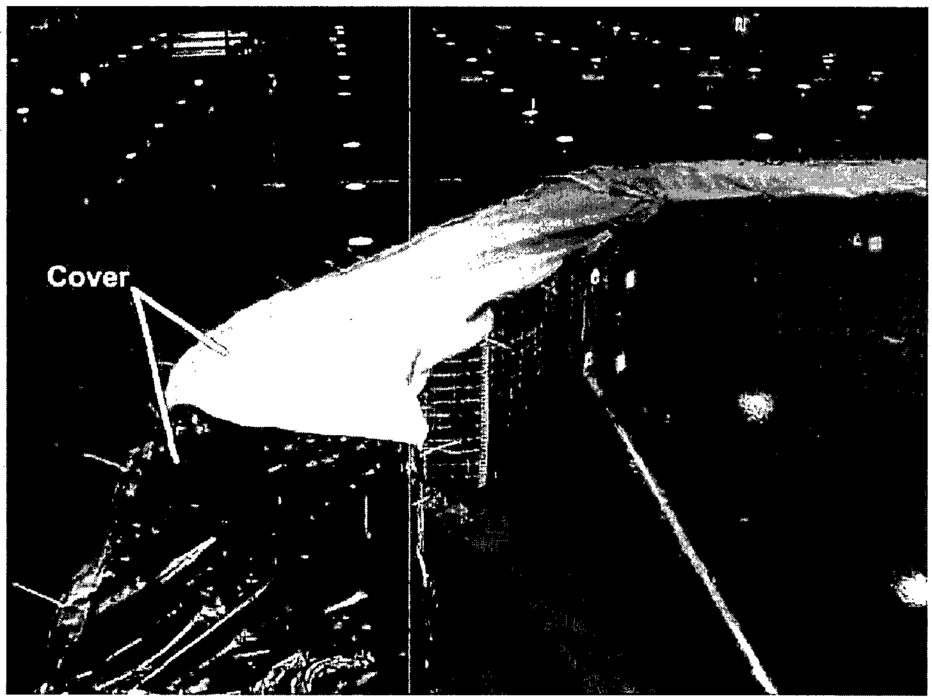


Figure 2-79. View of left and center walls

10- to 13-in. wave test, reservoir level at 80 percent levee height. The water level in the reservoir on the pool side of the levee was held at a height of 29 in., the wave generator was activated, and the waves began to impact the levee. Flow rate was observed to range from 1.05 to 3.14 gpm/lft (Figure 2-80), with no displacement. Wave overtopping was observed at each wave front, which contributed to the significant flow rate increase. Figure 2-81 shows wave overtopping.

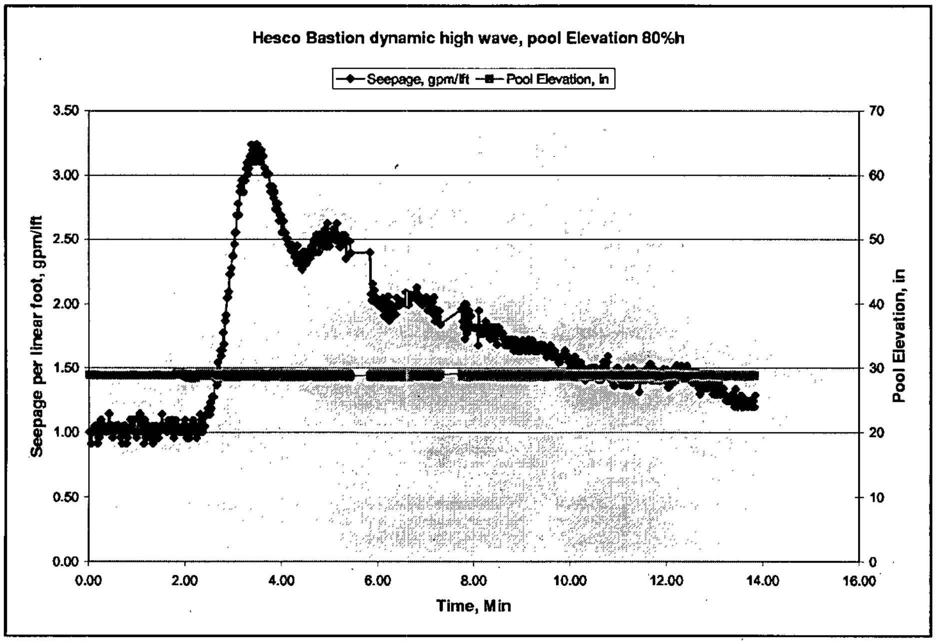


Figure 2-80. Seepage flow rate per linear foot, high wave at 80 percent pool elevation

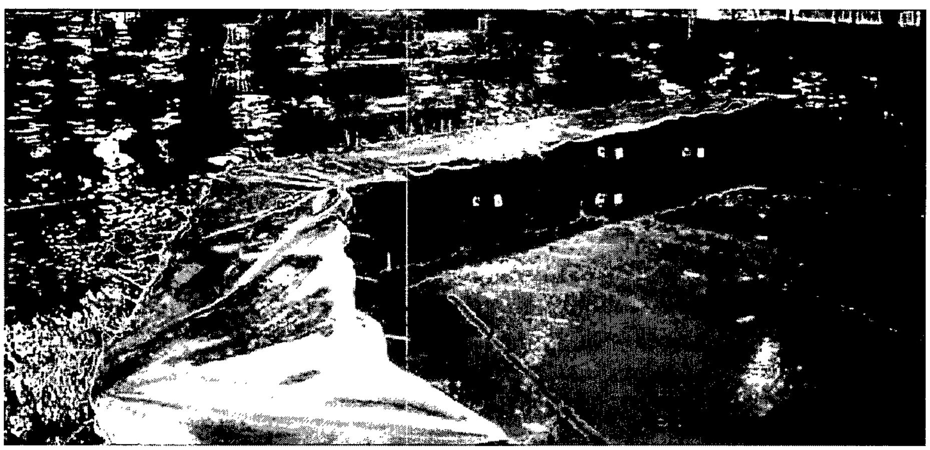


Figure 2-81. Wave overtopping along center wall

Levee-overtopping test

The reservoir level was raised from a height of 37.6 in. to a height of 38.8 in. After the water level reached the top of levee, overtopping occurred. The structure successfully withstood overtopping without failure. Overtopping water combined with seepage water to increase the measured flow rate within a range of 25.2 to 35.0 gpm/lft (1,800 to 2,500 gpm) in the span of 1 hr as shown in Figure 2-82. The overflow was uniform due to the uniform levee height. Figures 2-83 and 2-84, show the overtopped levee.

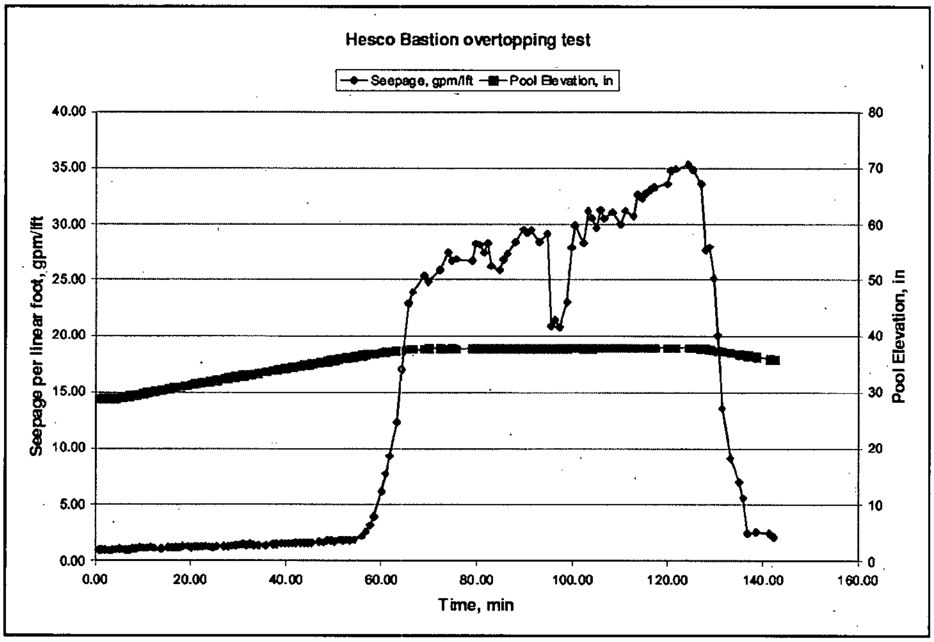


Figure 2-82. Seepage flow rate per linear foot during overtopping

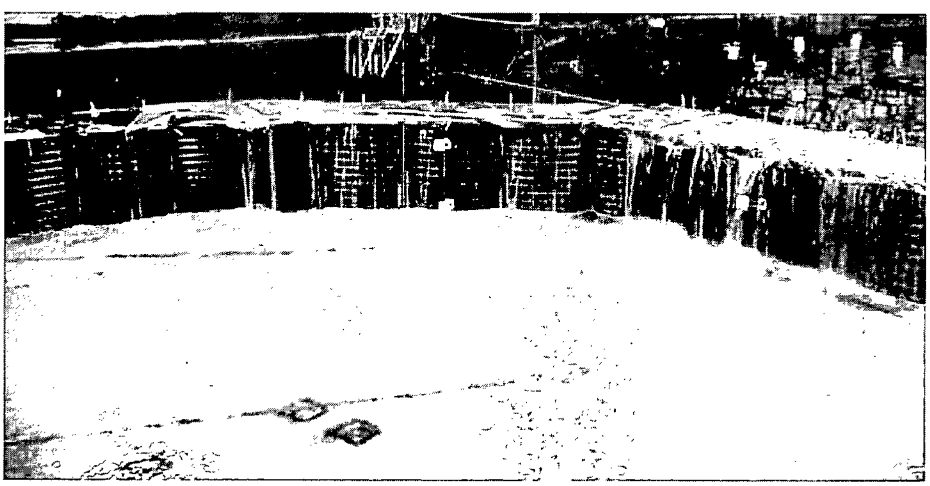


Figure 2-83. Overtopped levee structure, view from right wall

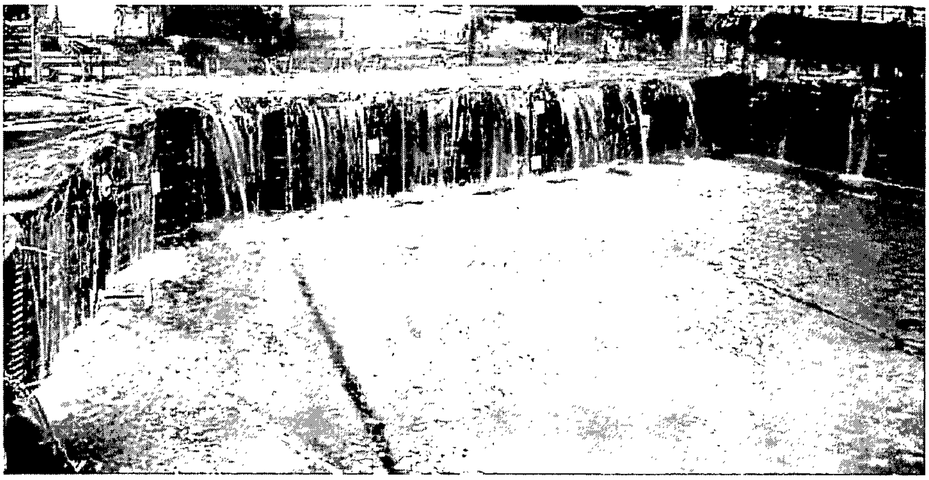


Figure 2-84. Overtopped levee structure, view from left wall

Debris impact test

With reservoir level at 24 in., the log impact tests were begun. The 12-in. log impacted the structure and bounced back without causing noticeable damage. The structure displaced slightly and recovered to its original position. The 16-in. log impacted the structure and bounced back also without causing any noticeable damage. The structure displaced slightly and recovered to its original position, but vertical deformations of the sand fill ranging from 4.02 to 0.72 in. were noted. Figure 2-85 shows the minor change in seepage flow rate during impact testing and Figure 2-86 shows the area where the logs hit, viewed from the pool side.

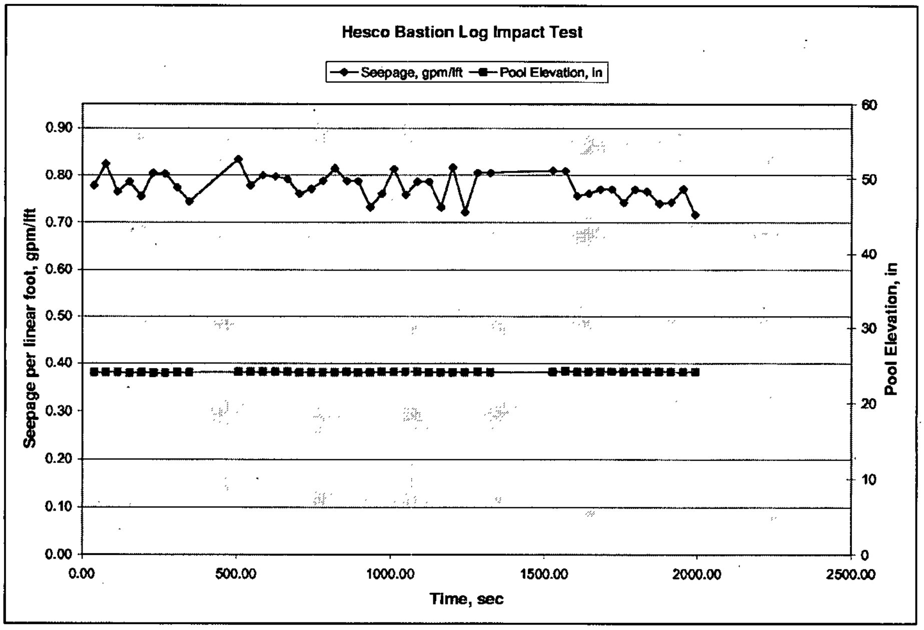


Figure 2-85. Seepage flow rate per linear foot during impact tests

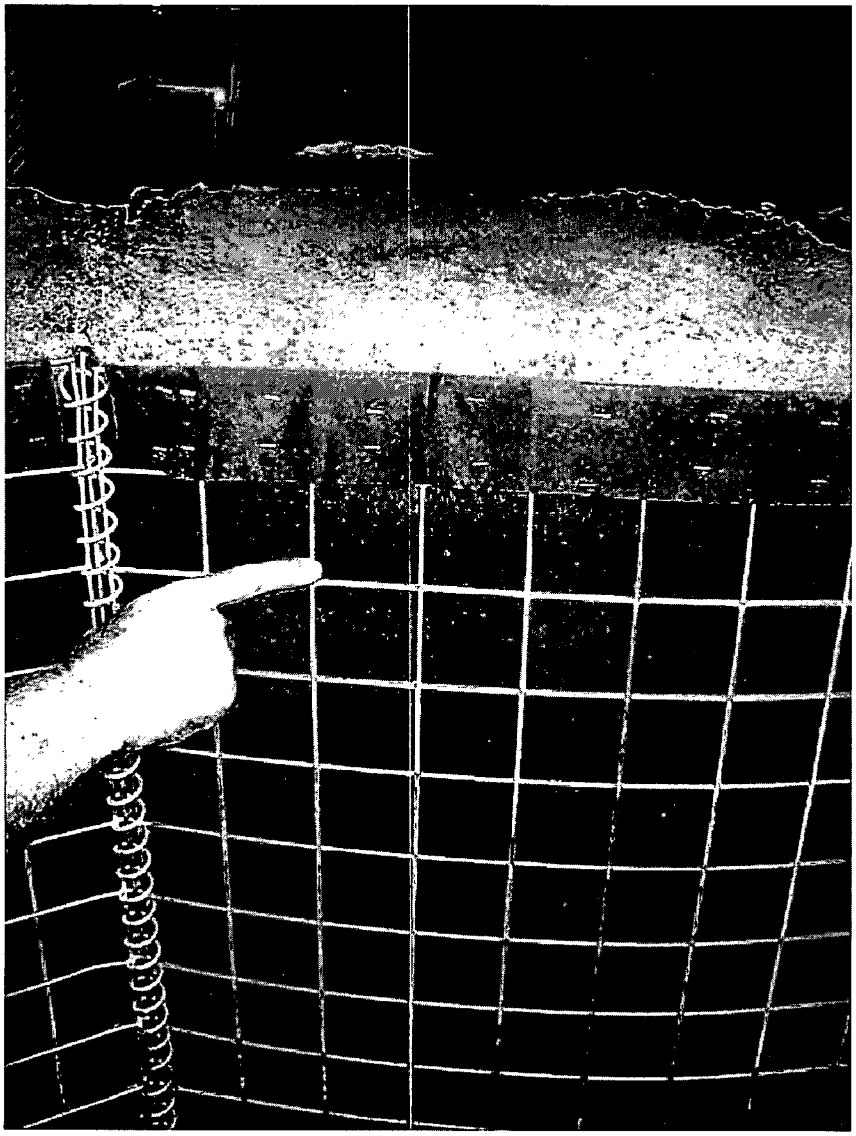


Figure 2-86. Log impact zone on center wall, pool side

Maintenance and repair

Repair 1 was performed prior to the 80 percent small (2- to 3-in.) wave test. It consisted of adding a top membrane fabric over the units, and adding cable ties and wire ties. A four-man crew took 24 min (1.6 man-hours) to do this work. Figure 2-87 shows this work (see also Figures 2-74 and 2-75).

Repair 2 was performed prior to overtopping. It took three men 5 min (0.25 man-hours) to add prefilled sandbags on the pool side for additional protection against joint seepage (Figure 2-88). Repairs 3 and 4 were not needed.

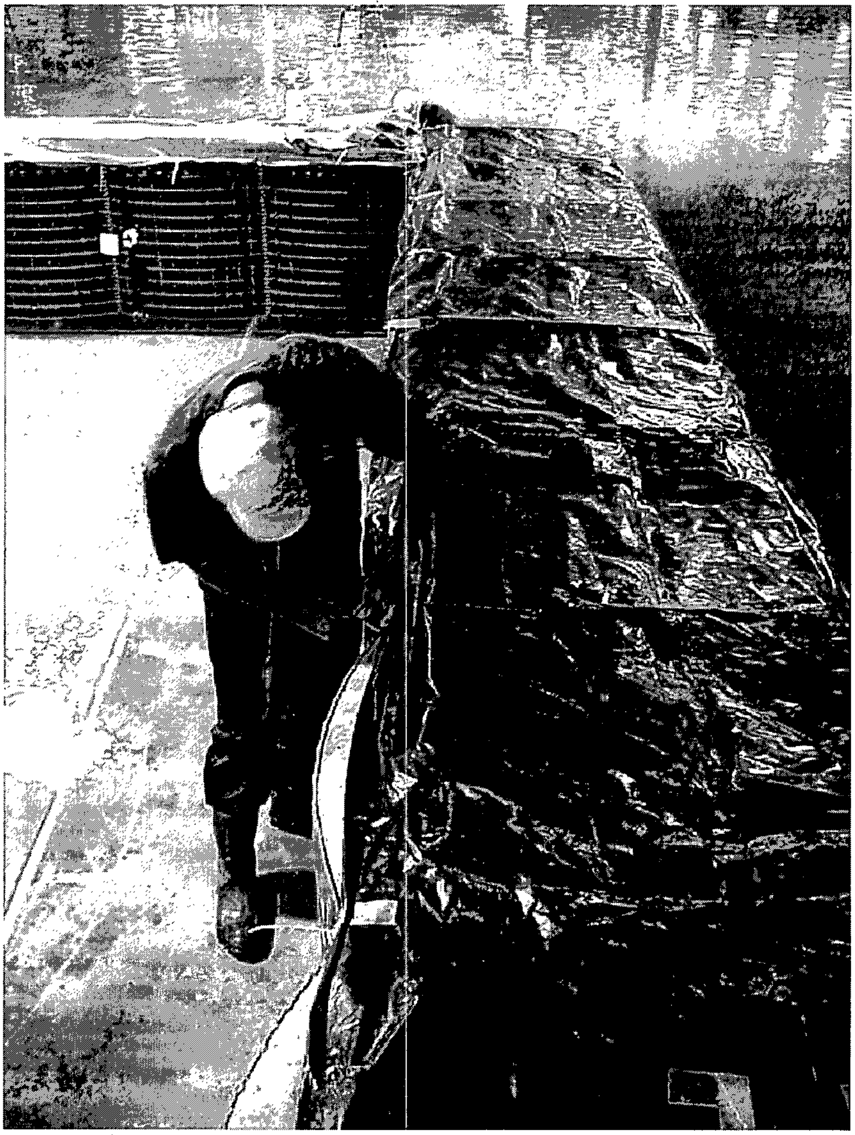


Figure 2-87. Repair 1, view along right wall

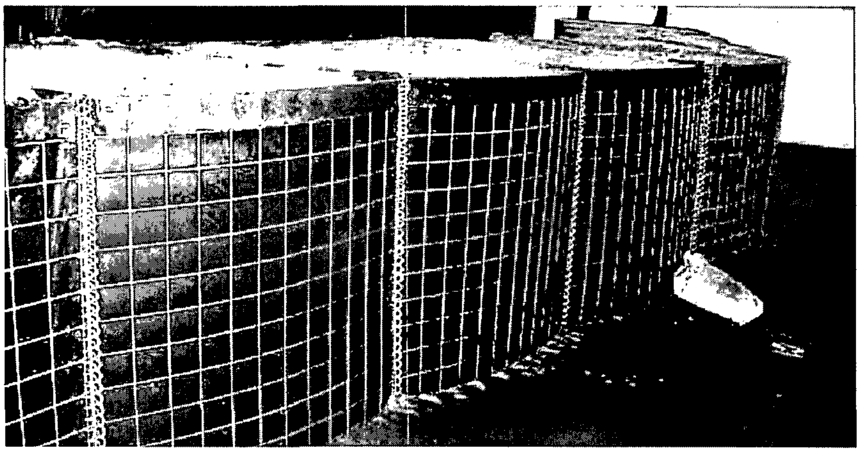


Figure 2-88. Added sandbag along left wall

Disassembly and reusability

At test conclusion, with a dry concrete floor, the Hesco® levee was disassembled and removed from the test facility on 24 May 2004. Disassembly consisted of three laborers and a supervisor to unpin the units, and a Cat® 916 front-end loader with operator to remove the sand. This five-man crew took 2 hr and 41 min (total 13.4 man-hours) to disassemble and remove the levee.

Disassembly consisted of removing all cable ties, removing the top cover (Figure 2-89), unhinging the inner and outer walls held with pins in each center partition (Figures 2-90 and 2-91, manually pulling each wall apart (Figures 2-92, 2-93, and 2-94), removing the sand pile (Figure 2-95), and restacking the units onto a pallet (Figure 2-96).

The sand was stockpiled for reuse, and the folded units were placed on wooden pallets for reuse. The only nonreusable items were the fabric panels at either end of the 12-ft units. During disassembly, the panels were slit with a knife to facilitate separation after the center partition pin was pulled out. The fabric end panels would then be repaired or replaced prior to reuse.

Figure 2-89. Cutting cable ties and removing top cover

Figure 2-90. Preparing to remove center partition pin

Figure 2-91. Removing center partition pin

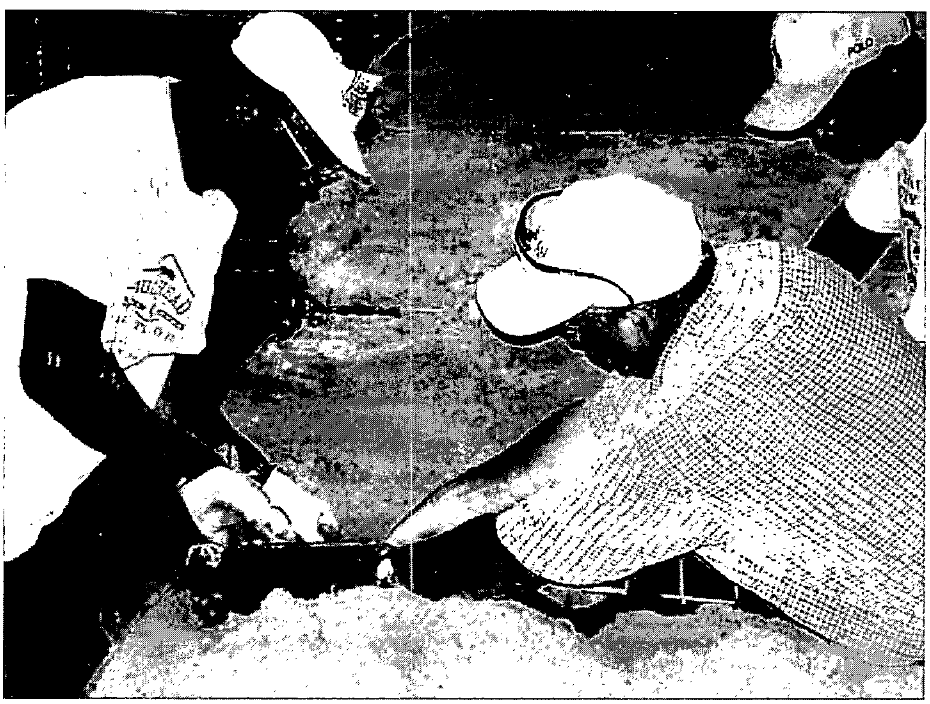


Figure 2-92. Preparing to pull unit apart

Figure 2-93. Pulling unit apart

Figure 2-94. Outer wall removed from one unit on right wall

Figure 2-95. Removing sand pile

Figure 2-96. Stacked units ready for reuse

Environmental aspects

From an environmental standpoint, when the HESCO Bastion Concertainer is used as designed, the barrier does not present any threats to the environment. Material Safety and Data Sheets provided by Hesco® indicated no exposure hazards due to everyday usage of the construction materials. The wire baskets are constructed from galvanized steel. If modifications are made to the baskets that involve welding of the wire mesh, then precautions should be made to prevent inhalation of the particulates created while welding. The baskets are constructed primarily of iron, greater than 90 percent, but do contain other metals, less than 3 percent, such as chromium, copper, manganese, nickel, and zinc. Since some of these metals are considered carcinogens, some form or respiratory protection should be used when welding the baskets.

Sand is placed in the baskets using machinery such as front-end loaders or bobcats. This machinery can damage the soil or foundation around the structure. Care should be taken when filling the baskets so that minimal damage is done to the area around the structure and repairs should be made to prevent erosion.

While being used as a flood barrier, the HESCO Bastion Concertainer does not pose any environmental hazards. Upon completion of the use of the barrier there are several issues that need to be addressed to ensure that no environmental hazards occur. Should the floodwater be contaminated with waterborne bacteria or pollutants, it may be possible for the sand fill inside the units to also become contaminated. The outer fabric should provide filtering and physical barrier protection, especially for nonwater-soluble contaminants such as floating oil, but water-soluble and suspended contaminants would likely be adsorbed by the sand fill. Should the levee materials (fabric and/or sand) become contaminated due to flood water contaminants, measures to properly decontaminate and/or dispose of those materials would be necessary. Like the sandbag structure, the sand used to fill the basket does not pose an environmental threat and should be disposed of in the appropriate manner. If the floodwater was contaminated the sand would have to be tested before disposal. The geotextile filter cloth would probably filter out most of the fine soil particles where most of the contamination is found. Still the sand would have to be tested to ensure no contaminants were in the sand that could present an environmental hazard. The filter cloth would have to also be disposed of in an appropriate manner. The wire baskets present the most danger to wildlife if left in the field. Small animals could become trapped in the mesh if left in the field. Also, if the baskets are left where water covers them, fish could become trapped in the mesh, similar to any other wire debris present in water bodies.

RDFW® Levee Tests

Design

The Rapid Deployment Flood Wall (RDFW®) was originally developed from the concept of expandable plastic grid system ("sand grid") which was invented at ERDC-GSL in the 1980s (U.S. Patent 4,797,026). The original RDFW® proponents licensed the sand grid patent from the Corps and developed a refined version of the technology which was later researched at ERDC with a Cooperative Research and Development Agreement (CRADA) in 1996.

The RDFW system is commercially available through the Geocell Systems Corporation (http://www.geocellsystems.com) and is also sold through the GSA procurement schedule #GS-07F-0340M, with a unit price of $100 (Geocell 2004). Figure 2-97 is a sketch of the unit grid dimensions. Each unit is a modular, lightweight, and collapsible plastic grid that allows for several stacking configurations and connections. The plastic material is a polyester polymer manufactured by Eastman Inc. (EastarTM copolyester 5445).

Table 3-2
Costs for Sandbag Structure

Item	Sandbag Structure
Product	$0.25 per bag for 120,000 bags = $30,000
Shipping	No $ estimated
Installation Laborers Operators Equipment Fill	 Built by volunteer labor = $0 1 man for 40 hours = $480 Sandbagger 1 loader for 5 days = $1,650 800 cu yd = $6,400
Removal Laborers Operators Equipment	 None required 3 men for 8 hr = $288 2 loaders for 1 day = $650 2 dump trucks for 1 day = $650
Training by vendor for installation and removal	By volunteers
Technical support during installation and removal	By volunteers

Based on the field testing, strengths and weaknesses of each product were observed. The strengths of the sandbag structure include low cost primarily because sandbag structures in a real-world flood are generally constructed by volunteer and/or prison labor. Because of the small size of the individual bags, sandbags conform well to varying terrain. For the field tests, the sandbag structure performed well with low seepage rates. Also, sandbag structures can be raised if needed by simply placing additional sandbags. The weaknesses of a sandbag structure are that they are labor intensive and time-consuming to construct. Also, sandbags are not reusable. All the sandbags used in the field-testing were disposed. For the field tests, the sandbags structure was constructed during the middle of May 2004 and removed during the middle of July 2004. Therefore, the structure was exposed to the elements for 2 months. During that time, the sandbags began to deteriorate. In fact, at the time of removal, walking on the bags would easily tear them and if you picked one up by the open end, the weight of the sand in the bag would tear the closed end out of the bag. The Vicksburg District Emergency Management personnel have determined that the bags used for the field test did not meet their sandbag specifications for weave count.

Field Installation and Performance of Hesco Bastion Concertainer

Introduction

The Hesco Bastion Concertainers are manufactured in the United States by Hesco Bastion – USA of Hammond, LA. The concertainers are described by Hesco as "a prefabricated, multi-cellular system, made of galvanized steel Weldmesh and lined with non-woven polypropylene geotextile." In common terms, the concertainers are granular-filled, geotextile-lined wire baskets. The Hesco Bastion Concertainers have several uses but primarily have been used since the early 1990s (Persian Gulf War) as military force protection.

The Hesco Bastion Concertainers are manufactured in a wide range of sizes. For the Vicksburg Harbor field test, units 3-ft wide by 3-ft high by 12-ft long were used to provide the required 3-ft flood protection. For the required 1-ft raise, units 3-ft wide by 2-ft high by 12-ft long were placed on top of the 3-ft high base row units. Since the concertainers are a multicellular system, each unit contained four individual 3-ft-long cells. The units were pinned together to form a u-shaped structure with a riverward face of 98 ft with tieback sections of 48 ft.

Field construction

The concertainer units as delivered to the Vicksburg Harbor were stacked flat on wood pallets and wrapped with plastic (Figure 3-67). Prior to installation, concertainer pallets were prepositioned adjacent to the construction site. The construction crew included a Hesco Bastion representative, four government-furnished laborers, and two government-furnished equipment operators. The government also furnished two tracked Bobcat front-end loaders. None of the government laborers or operators had any prior knowledge of the Hesco Bastion product.

Construction of the Hesco Bastion Concertainer structure began on the morning of 12 May 2004, in constant rain and mild temperatures. Figure 3-68 is a photograph of the Hesco Bastion site prior to construction. Because the Government laborers and operators were unfamiliar with the product, the Hesco Bastion representative conducted a 23-min training session on the installation process (Figure 3-69). At the completion of the training session, the workers began placing the base row units along the desired alignment (Figure 3-70). In accordance with the construction protocol, about half of the site was graded to bare ground while the other half was left undisturbed with the natural grass and weeds (Figure 3-71). The units were installed according to Hesco instructions as follows.

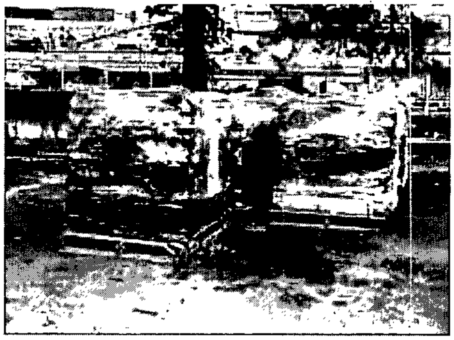


Figure 3-67. Hesco Bastion as delivered to Vicksburg

Figure 3-68. Hesco Bastion field site prior to construction

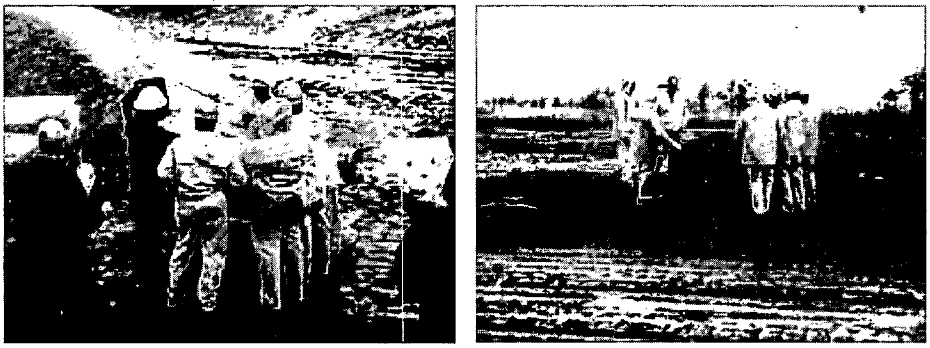


Figure 3-69. Hesco Bastion training session

Figure 3-70. Installation of base row units

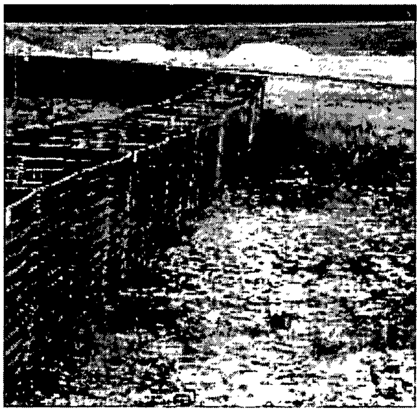


Figure 3-71. Structure constructed on graded ground and grass/weeds

Units were pinned together to form a continuous barrier by inserting joint pins through the coils of adjacent units (Figure 3-72). The units also were connected with zip ties placed along the top of adjacent unit end panels. Riverward face units of the structure were placed first, followed by the tieback sections (Figure 3-73). Each unit has a 5-in. liner flap on the bottom. Care was taken to ensure that these flaps were turned to the inside of each unit prior to filling, so that the weight of the sand on the flaps secured the units in place. Once the base row units were placed, the units were filled with sand to within approximately 5 in. of the top (Figure 3-74). The units were not completely filled because the bottom flaps on the top row are turned down and buried into the sand in the base row units. The sand had previously been stockpiled adjacent to the Hesco Bastion site and was placed in the units by two tracked front-end loaders. The laborers spread the sand within the units with shovels and manually compacted the sand by walking on it. Sand was placed in the concertainers primarily from the protected side of the structure. However, due to the location of the seepage-collection tank in the northeast corner of the structure, the sand in the vicinity of the tank was placed from the riverside.

Figure 3-72. Installation of joint pins

Figure 3-73. Construction of base row tieback section

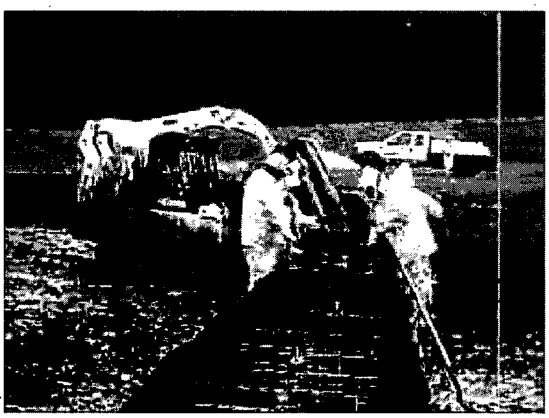


Figure 3-74. Filling base row with sand

Once the base row was filled, the required 3-ft-high structure was finished. The construction crew of one Hesco Bastion representative, four government laborers, and two government equipment operators took 5.1 hr and 34.7 man-hours to construct the 3-ft-high structure. The only equipment used to construct the base row was shovels and the two tracked Bobcats.

Once the required 3-ft-high structure was finished, work began on installing the 1-ft raise required by the construction protocol. Hesco Bastion accomplished the raise by adding a second row of units on top of the base row (Figure 3-75). The units for the second row were 3 ft wide by 2 ft high by 12 ft long. Due to the natural ground slope at the Hesco Bastion site, the top row tieback sections were only 27.6 ft and 15.25 ft long.

The construction crew installed two of the top row units before work ended on the afternoon of 12 May. Work on the required raise resumed on the morning of 13 May. The weather that morning was sunny and humid. Since the tieback sections were placed on sloping ground, the top row was only needed on the riverward face and portions of the tieback sections. The top units were unfolded and placed directly on top of the base row units. Joint pins were added to the top row and these units were zip-tied together at the top of the end panels of adjacent units. The top row and base row units were also zip-tied together. Once the top-row units were secured, sand was placed in the units. Initially, the sand was placed in the top row units from the protected side except for the northeast corner, to avoid the seepage collection tank.

During the time that the units were being filled, the ground around the structure was extremely muddy and slick. Because the riverward front of the structure was constructed on sloping ground, the Hesco Bastion representative was concerned that during filling, the Bobcats would slide into and damage the structure.

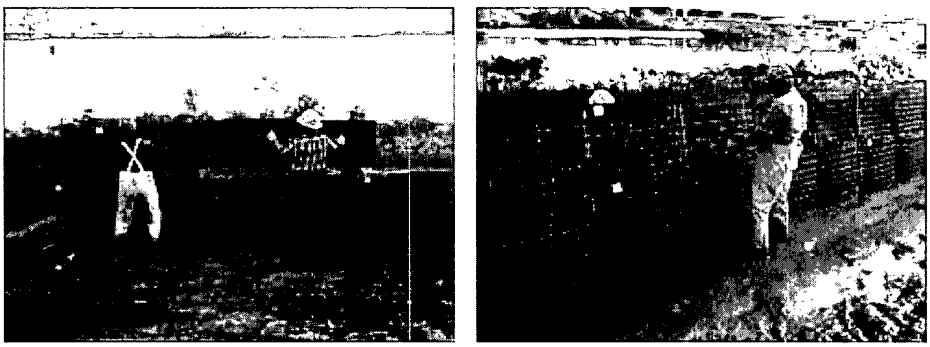


Figure 3-75. Installing top row units (required raise)

Therefore, he requested and was granted permission to fill portions of the riverward front from the riverside (Figure 3-76). Since the top row units were 2 ft high and the required raise was only 1 ft, the top row units were not completely filled. The amount of fill varied in the top row units but averaged about 18 in. (Figure 3-77).

Figure 3-76. Filling top row units with sand

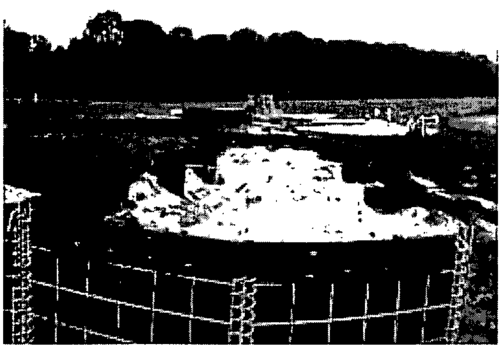


Figure 3-77. Sand fill in top row units

The construction crew of one Hesco Bastion representative, four government laborers, and two government equipment operators took 3.8 hr and 22.8 man-hours to construct the required raise. The total time to construct the Hesco Bastion structure was 8.9 hr or 57.5 man-hours. Construction of the Hesco Bastion structure was completed just prior to noon on 13 May. The equipment used to construct the top row was the same shovels and the two tracked Bobcats that were used to construct the base row.

The Hesco Bastion Concertainer units used at the Vicksburg Harbor test site were 3 ft wide when empty. However, as sand was placed in the units, the units began to expand. The cells within the units ranged from 40 to 48 in. wide when the structure was finished. Therefore, the units used for the field test have a footprint of 4 ft. The Hesco Bastion structure required 91 cu yd of sand fill. Also, Hesco Bastion was allowed a 25-ft right of way to construct their structure. Because the structure was filled from the side with tracked Bobcats, the entire 25-ft right of way was used. Figures 3-78 and 3-79 are photographs of the completed Hesco Bastion structure. Once the construction was completed, the Hesco Bastion representative signed a certification that the structure was constructed according to his onsite directions and in accordance with Hesco Bastion's installation specifications.

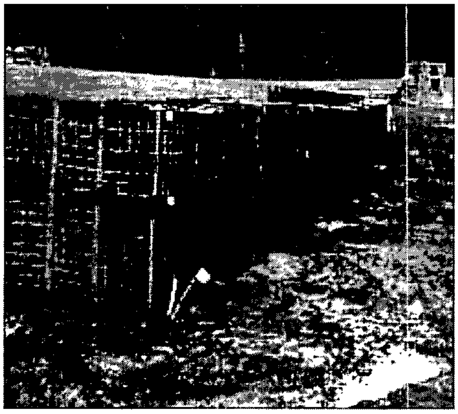


Figure 3-78. Riverward face of completed structure

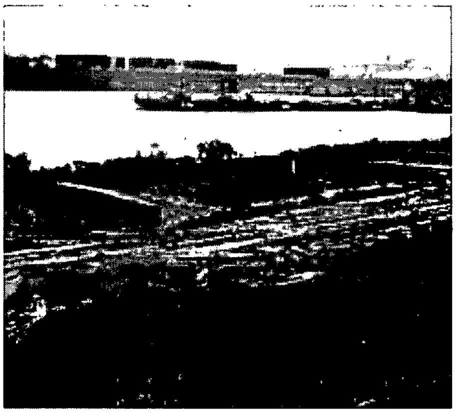


Figure 3-79. Completed structure from protected side

Testing

The Hesco Bastion Concertainer structure was constructed during a time when the river levels were falling. However, by early June, as predicted, the river had begun to rise and by the morning of 5 June approximately 0.3 ft of water was standing against the structure. Figures 3-80 through 3-87 show the Hesco Bastion structure during field testing. As the river continued to rise, the Hesco Bastion structure was subjected to higher water levels. The daily water levels against the structure are given in the figure captions. These water levels were based on 8 a.m. readings for the Mississippi River at the Vicksburg gage. The testing of the Hesco Bastion structure ended on 11 June 2004. The river never rose high enough to overtop the top row units. However, sand in five of the riverside top row cells was at the level to provide exactly 4 ft of protection. On 11 June, the river level rose high enough to overtop the sand in those five cells. The decision was made in collaboration with the Hesco Bastion representative to stop the tests at that point even though the pump capacity had not been exceeded.

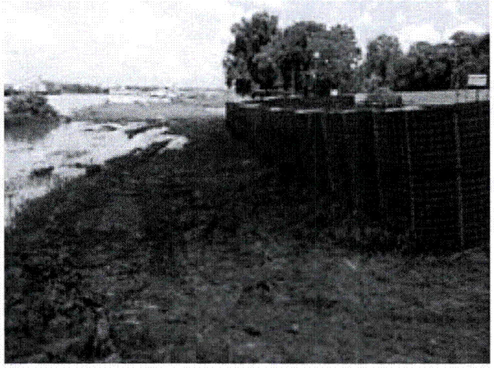


Figure 3-80. 4 June 2004, no water against structure

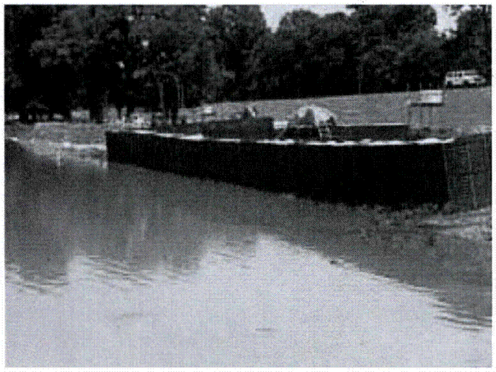


Figure 3-81. 5 June 2004, 0.3 ft of water against structure

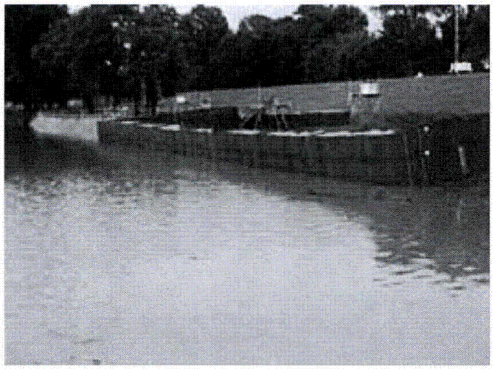


Figure 3-82. 6 June 2004, 1.3 ft of water against structure

Figure 3-83. 7 June 2004, 2.1 ft of water against structure

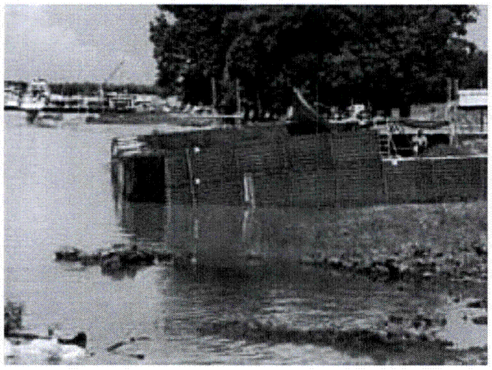


Figure 3-84. 8 June 2004, 2.7 ft of water against structure

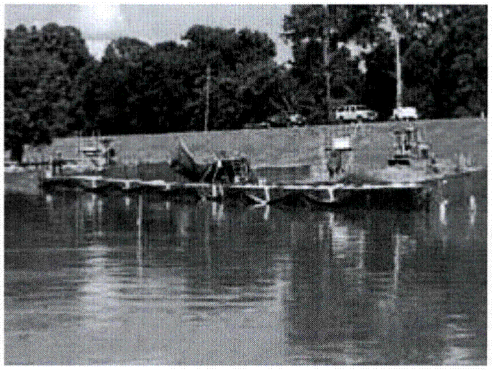


Figure 3-85. 9 June 2004, 3.1 ft of water against structure

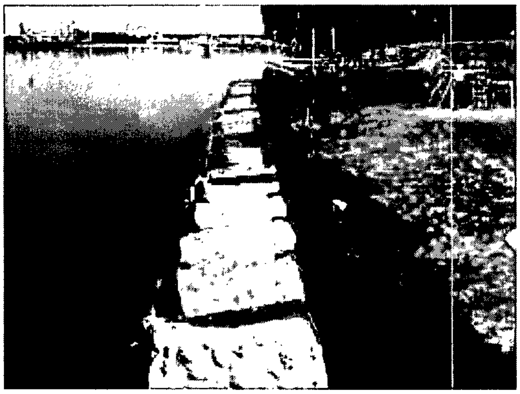


Figure 3-86. 10 June 2004, 3.5 ft of water against structure

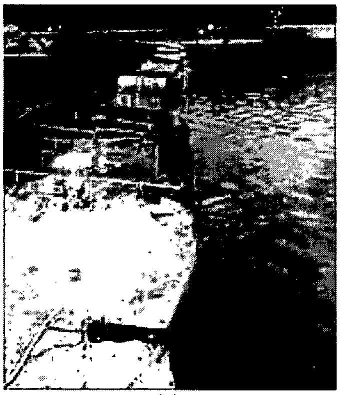
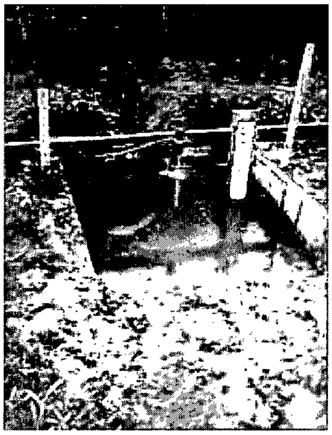
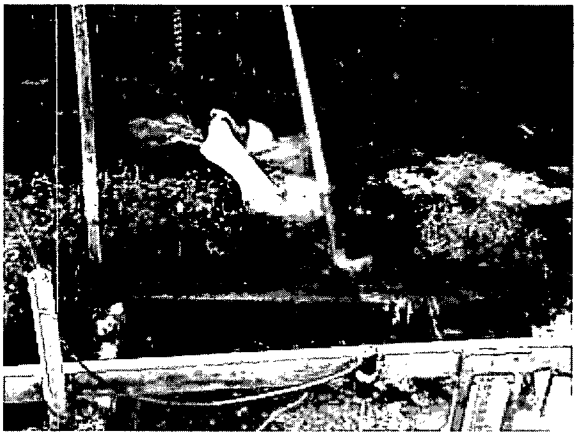


Figure 3-87. 11 June 2004, 4.0 ft of water against structure

During the field test, seepage was collected in a buried concrete tank located on the protected side of the structure. Seepage rates were determined by computing the change in volume in the tank over a specific time. As the water level rose against the structure, seepage rates increased. Figure 3-88 shows two photographs of the Hesco Bastion structure seepage tank. The first photograph was taken on 6 June 2004 while the seepage rate was low. The second photograph was taken on 10 June 2004 when the seepage rate had increased noticeably. Figure 3-89 is a photograph of the seepage observed through the joint between adjacent units. Figure 3-90 shows the seepage water on the protected side of the structure. To determine seepage rates, the wetted area for each structure for a given water surface elevation was computed. Table 3-3 provides the seepage rates for the Hesco structure. The seepage rates for the Hesco Bastion structure were high. The seepage rates were high enough that the Hesco Bastion representative attempted repairs to try to reduce through seepage.



a. 6 June 2004



b. 10 June 2004

Figure 3-88. Hesco Bastion seepage collection tank

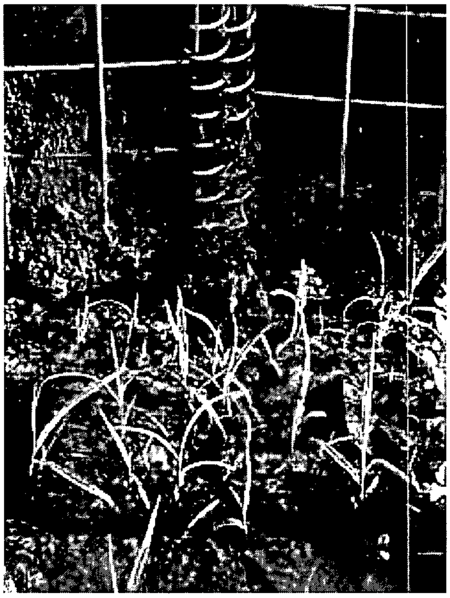


Figure 3-89. Seepage through joints

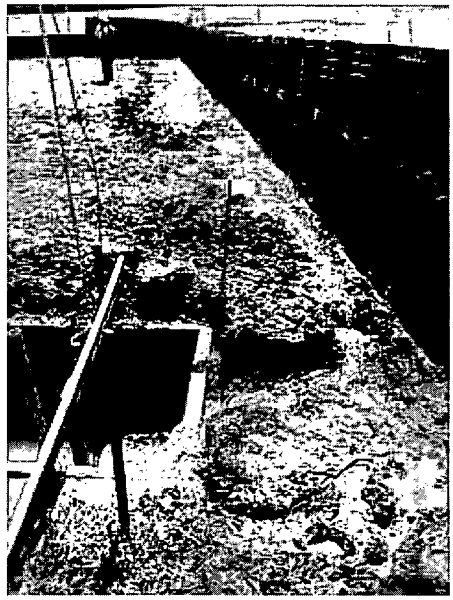


Figure 3-90. Seepage on protected side

Table 3-3 FieldTest Seepage Rates - Hesco Bastion	
Wetted Area of Structure (sq ft)	Seepage Rate (gal/hr)
100	300
200	2,300
300	3,900
400	6,000

The first repair was made on 8 June and included the addition of plastic sheeting to the riverward face of the structure (Figure 3-91). This repair was made with 2.5 to 3.0 ft of water against the structure. The plastic sheeting was rolled out and attached to the top of the top layer units with zip-ties. The sheeting was weighted and held against the bottom of the base row units with sandbags. At the time that the repair was made, the seepage rate was approximately 4,000 gal/hr. The repair temporarily reduced seepage, with the rate falling to approximately 3,000 gal/hr. The repair was made on the afternoon of 8 June. By the morning of 9 June, the seepage rate had risen to approximately 4,300 gal/hr with only a few tenths of a foot rise in the river level.

Figure 3-91. Attaching plastic sheeting to riverward face of Hesco Bastion structure

The second repair was made on 9 June. This repair consisted of attaching half sections of 4-in. PVC sewer pipe across the unit joints with zip ties. Bentonite slurry, dry powder, and pellets along with sand was poured into the top of the pipes and packed down (Figure 3-92). Hesco representatives expected the bentonite in the pipes to swell and seal the joints. This repair was made with just over 3 ft of water against the structure. After the pipes were installed, the seepage rate continued to increase. Once the river levels dropped after the testing was completed, the Hesco Bastion structure was visually inspected. Apparently, an excess of bentonite was packed into the pipes. As the bentonite swelled, the pipes were pushed away from the joints thus providing no sealing of the joints.

Removal

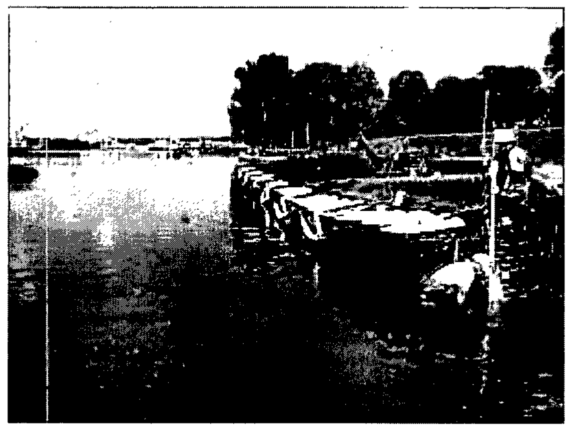
Removal of the Hesco Bastion structure was initiated on the morning of 14 July. The weather was hot and humid with a heat index near 105 deg F. Due to the extreme heat, the work crew took frequent breaks. Only the time that the crew was physically working to remove the structure was included in the removal time (the clock stopped during breaks). The removal began with a three-man Hesco Bastion crew removing the top row layer. Hesco Bastion requested and was allowed to remove the top row layer since the government-furnished crew was unavailable at that time.

The first action in the removal process was removing the joint connection pins between the units and the center connection pins within each unit. To remove the center connection pins from the unit ends, the liner material had to be cut to expose the pins. Prior to reusing the units, this liner material has to be replaced. The removal of the center connection pins is required to break each unit into a front face half and a back face half. The pins were removed by two men using a pin removal bar and a chain (Figure 3-93). Once the pins were removed, the zip-ties between the top row units and the bottom row units were cut (Figure 3-94). This allowed the work crew to lift and pull the half units from the sand (Figure 3-95).

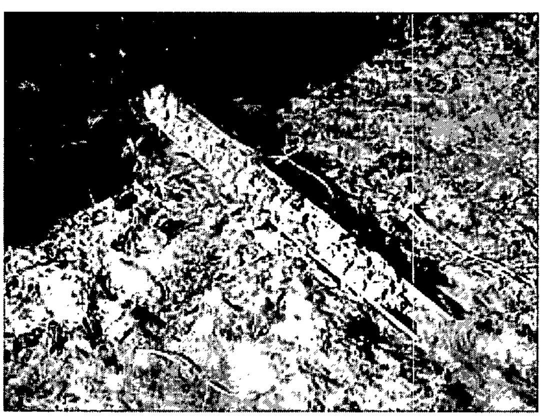
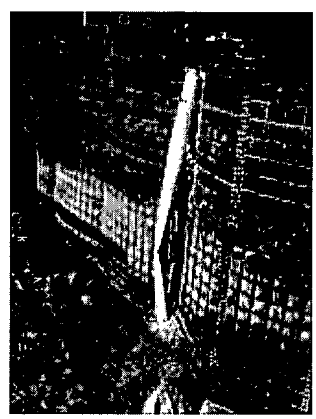
Figure 3-96 is a photograph of the riverward face of the structure after the outer half unit sections were removed from the top row. Once the top row units were removed, the sand from those units was scraped off of the base row units with a front-end loader



a. Attaching pipe to joints b. Bentonite slurry c. Bentonite pellets

d. Pipe with bentonite e. Packing bentonite into pipes

f. Bentonite-filled pipes after water receeded

Figure 3-92. Attempt to reduce seepage using bentonite

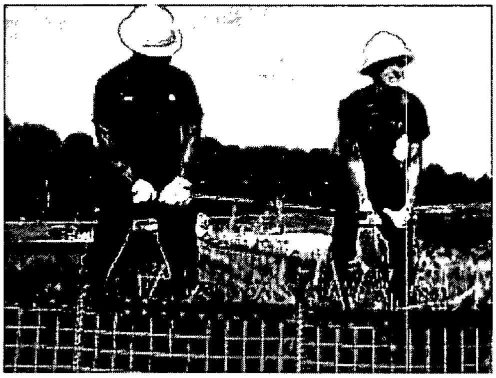


Figure 3-93. Removing center connection pins

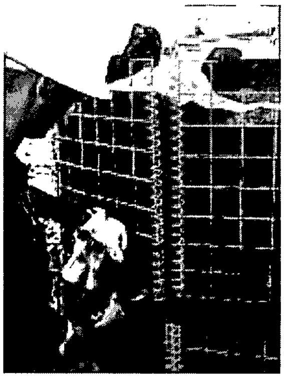


Figure 3-94. Removing zip ties

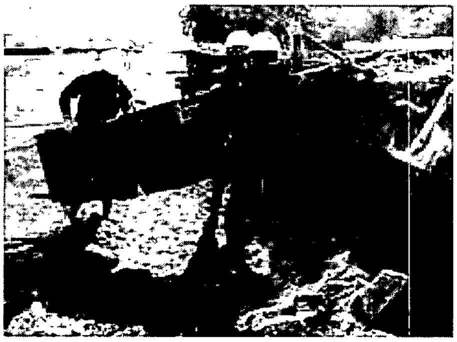


Figure 3-95. Removal of top row half units

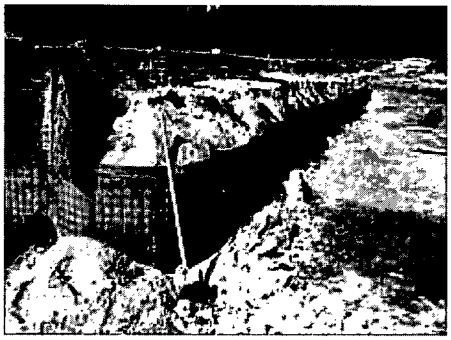
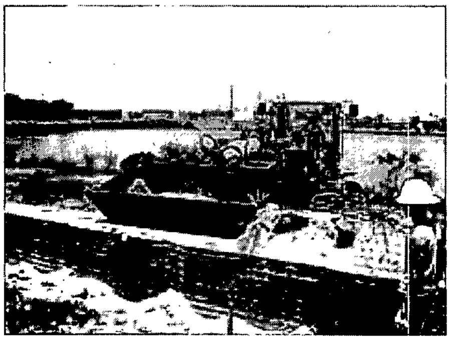


Figure 3-96. Riverward face of structure

(Figure 3-97). This sand was then removed from around the base row units so that they could be removed (Figure 3-98). The base row units were removed by a crew of two Hesco Bastion representatives and four government laborers plus a government equipment operator. The same process was used to remove the base row units that were used to remove the top row units. Most of the base row half units were physically lifted and pulled from the sand by hand (Figure 3-99). However, when the joint-connection pins were pulled from the riverward face of the base row, two half sections were pushed over by the weight of the sand because these units were on sloping ground. The removal crew used the front-end loader and four chains to remove these half sections (Figure 3-100). They also used the front-end loader to pull some of the joint-connection and center-connection pins from the base row units (Figure 3-101).

Once the units were removed, the front-end loader was used to remove the sand to a disposal site on the extreme west end of the Vicksburg Harbor testing site. The average haul distance from the Hesco Bastion structure was approximately 550 ft. By the end of the day (14 July), most of the structure had been removed. The remainder of the structure was removed during the early morning on 15 July. Since the weather that day was extremely hot and humid, work began at 6:10 a.m. The entire structure including the sand fill was removed from the site by late morning. The removal of the Hesco Bastion structure and sand fill took a total of 8.7 hr or 36.3 man-hours. The equipment used to remove the Hesco Bastion structure included shovels, a joint-pin-removal bar and chain, and a front-end loader. Once the structure was removed, the Hesco Bastion representative signed a certification that the structure was removed according to his onsite directions and in accordance with Hesco Bastion's removal specifications.



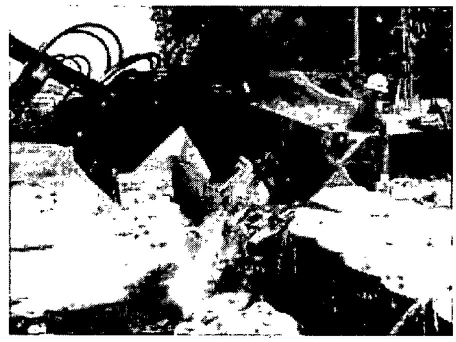


Figure 3-97. Removal of top row sand

Figure 3-98. Removal of sand from around base row units

Figure 3-99. Removal of base row half units

Figure 3-100. Removal of half units with front-end loader

Figure 3-101. Removal of joint-connection pins with front-end loader

Reusability

Once removed, the Hesco Bastion units were inspected for damage, folded, and placed on pallets for transport offsite. All of the Hesco Bastion units used for field testing were folded and strapped to four pallets (Figure 3-102). The removed units were stacked to a height of 36 in. on three pallets and to 40 in. on the fourth pallet. All four pallets were loaded onto a standard 16-ft trailer (Figure 3-103) for transport back to the Hesco Bastion plant.

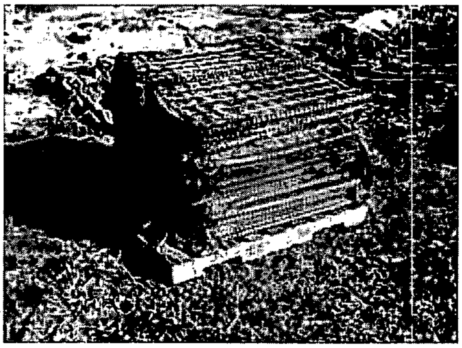


Figure 3-102. Removed units on pallet (pallets 48 × 40 in.)

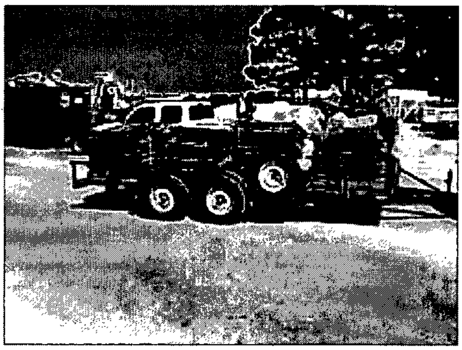


Figure 3-103. Removed units on trailer

None of the top row units (2 ft × 3 ft × 12 ft) sustained any damage. Some limited damage was noted to base-row units. Each of the Hesco Bastion base row units was made up of eight side panels (36 in. × 36 in.), 10 cross panels (36 ft × 18 in.) and 20 coils. Table 3-4 provides an inventory of the damage.

Table 3-4
Hesco Bastion Damage

Units	No. Units	Side Panels		Cross Panels		Coils	
		Used	Damaged	Used	Damaged	Used	Damaged
3 ft x 3 ft x12 ft	16	128	9	160	10	320	6
2 ft x 3 ft x 12 ft	11	88	0	110	0	220	0

Table 3-4 shows that the Hesco Bastion units received limited damage with over 95 percent of the side panels, over 96 percent of the cross panels, and over 98 percent of the coils reusable. Damaged or cut pieces can be replaced, making the unit reusable. All damage to the Hesco Bastion units occurred during removal. The damage can be directly attributed to the use of heavy machinery. Once the top row units were removed, a front-end loader was used to scrape the remaining sand from these units off of the bottom row units, which damaged some panels and coils. Also, the front-end loader and chains were used to hoist some of the bottom row sections that were heavily weighted with sand. This lifting damaged some panels to which the chains were attached. Figure 3-104 provides examples of the damage that the units experienced during the removal process.

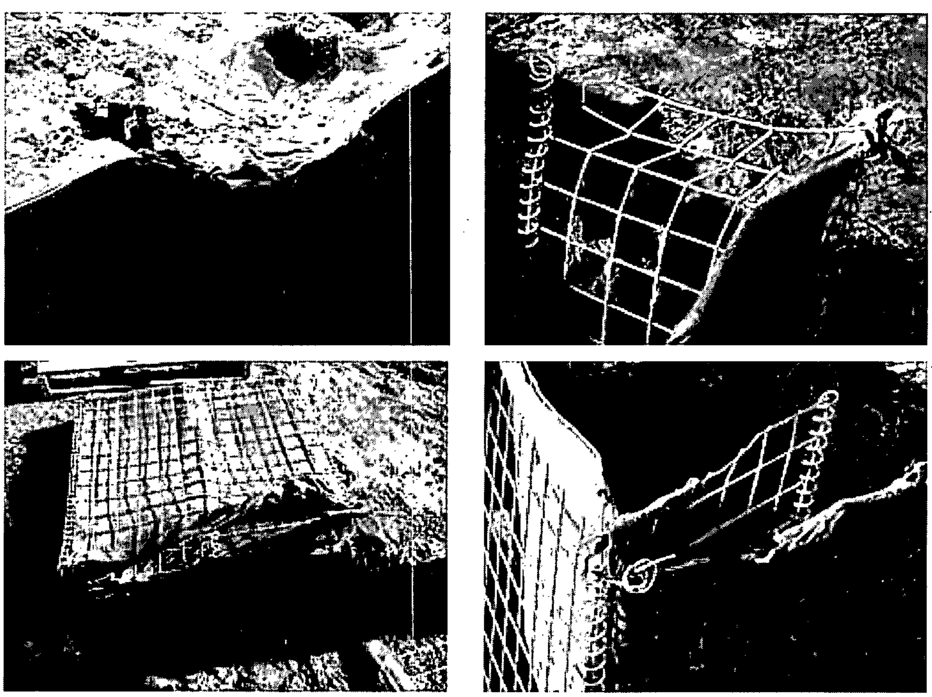


Figure 3-104. Units damaged during removal process

The units can be cleaned by washing the sand, mud, and debris off the units with a garden hose. If the units are washed, the liner should be completely dry before folding and storing. If the soil on the units is dry, the soil can be swept off the liner with a broom. In this project, the units were not cleaned at the field site, but were packed for shipping immediately after disassembly.

Summary

For the field testing, various construction, removal, and performance parameters were evaluated. Table 3-5 provides a summary for the field testing of the Hesco Bastion Concertainer structure.

Table 3-5
Hesco Bastion Field Testing Summary

Item	Hesco Bastion
ROW Used (ft)	25
Footprint Width (ft)	4 (includes bulge in 3-ft wide units)
Structure Length (ft) Riverward Face East Tieback West Tieback	 98 48 48
Ease of Construction Time (hr) Effort (man-hours) Manpower (no. men) Equipment	 8.9 57.5 7 Shovels 2 Bobcat Loaders
Fill (cu yd)	91
Durability	The Hesco Bastion structure stayed in the field for 2 months and was subjected to hot, wet weather. The structure showed no signs of deterioration.
Varying Terrain	The field test site was relatively flat with a mild slope from the protected side of the Hesco Bastion structure to the riverward side.
Ease of Removal Time (hr) Effort (man-hours) Manpower (no. men) Equipment	 8.7 36.3 6 Shovels Pin Removal Bar Front End Loader Forklift
Seepage (gal / hr) For 100 sq ft Wetted Area For 200 sq ft Wetted Area For 300 sq ft Wetted Area For 400 sq ft Wetted Area	 300 2,300 3,900 6,000
Repairs	All Minor – Structural Integrity Not Threatened Attempted to Seal Joints with Plastic Sheeting and Bentonite
Reusability (percent)	> 95

Even if a product performs well, the flood-fighting community is not likely to use the product unless it is cost-effective. In order to make a fair comparison of costs, each product vendor was asked to submit the cost of constructing and removing 1,000 lft of their product, 3 ft high in Vicksburg, MS. This cost included the purchase of the product plus fill material, labor, and equipment based on Vicksburg rates. The cost for shipping the products were not provided. For this cost determination, sand fill delivered to the site was estimated at $8 per cu yd. Labor rates were $8/hr for laborers and $12/hr for equipment operators. Table 3-6 provides a summary of the costs furnished by Hesco Bastion. The Hesco Bastion Concertainers are reusable. However, Hesco Bastion does not provide a guarantee that would provide for no cost replacement of damaged units.

Table 3-6
Costs for Hesco Bastion Concertainer

Item	Hesco Bastion Provided Cost
Product	67 3'x3'x15' units at $394/unit = $26,398.
Shipping	No $ provided
Installation Laborers Operators Equipment Fill	 6 men for 20 hr = $960 2 men for 20 hr = $480 2 loaders for 2 days = $1,300 425 cu yd = $3,400
Removal Laborers Operators Equipment	 6 men for 20 hr = $960 2 men for 20 hr = $480 2 loaders for 2 days = $1,300
Training by vendor for installation and removal	No charge for initial installation
Technical support during installation and removal	No charge for initial installation

Based on the field testing, strengths and weaknesses of each product were observed. Hesco Bastion's strengths include ease of both construction and removal for time and manpower. The field testing showed that a Hesco Bastion structure can be constructed quickly and with a limited labor force as compared to a comparable sandbag structure. Another of Hesco Bastion's strengths is low product cost. The cost for a Hesco Bastion concertainer structure is comparable to the cost of a sandbag structure. That comparison includes labor to construct a Hesco Bastion structure and only limited labor for a sandbag structure since during real-world flood events, sandbags are typically constructed by volunteer and/or prison labor. However, with all the products tested, the cost of the product is the large majority of the total cost. The installation cost including labor, equipment, and materials is minor as compared to the purchase price of the products. A Hesco Bastion structure can be raised if required by placing additional units to the top of the structure. If the required raise is more than 1-½ to 2 ft, then stability becomes an issue. In that instance, the structure should be raised by first placing a second row of units along the original base row to increase the width of the structure. A second row can be placed in a pyramid shape on top of the base rows. Hesco Bastion units proved in the field tests to be reusable. Inspection of Hesco Bastion units subsequent to completion of the removal process showed that over 95 percent of the unit pieces were reusable. A small number of panels and coils were damaged during the removal process. However, these pieces are easily replaced. The observed weaknesses of the Hesco Bastion product include the need for significant construction right of way. Hesco Bastion structures are granular filled. At present, the fill material is placed in the units with a loader that works perpendicular to the structure. This operation results in additional right of way needed to fill the units. The Hesco Bastion structure tested in the field had high seepage rates relative to the other structures. Since completion of the testing, Hesco Bastion has evaluated their seepage rates. Their evaluation concluded that they installed the concertainer units incorrectly. Their standard installation protocol includes removing the permeable liner from the ends of adjoining units so that the sand fill can flow freely between the adjacent cells. For the field testing, the liner was not removed. If installed correctly, the seepage rates for a Hesco Bastion structure should be significantly reduced.

Field Installation and Performance of Rapid Deployment Flood Wall (RDFW)

Introduction

Rapid Deployment Flood Wall (RDFW) units are manufactured in the United States by Geocell Systems, Inc. The RDFW is described by Geocell as "a modular, collapsible plastic grid." In common terms, the units are plastic grids filled with granular material, interlocked and stacked together to form a wall.

Field construction

One RDFW unit is 41.5 lin. and holds approximately 0.3 cu yd of fill material. Each unit contains 35 individual cells. For the Vicksburg Harbor field test, the units were connected end to end by the interlocking tabs. A structure high enough to hold back 3 ft of water was accomplished by stacking five units (40 in.) to form the wall. In accordance with the construction protocol, a raise of the structure to hold back 4 ft of water was required. RDFW accomplished the raise by adding a single row of units (8 in. high) on top of the initial 40-in.-high structure.

The RDFW units were delivered to the Vicksburg Harbor in crates. Six crates were delivered containing 100 units each. Figure 3-105 shows the RDFW units as delivered to the field testing site. Prior to installation, the crates were prepositioned adjacent to the construction site. The construction crew included a Geocell representative, four government-furnished laborers, and two government-furnished equipment operators. The government also furnished two tracked Bobcat front-end loaders. None of the government laborers or operators had any prior experience with the RDFW product. Construction of the RDFW structure began on the morning of 13 May 2004.

During site preparation, the RDFW testing area was left partly undisturbed (grass and weeds remaining) and partly graded to bare ground. Because of the rainy weather conditions on the day of construction, the testing area was back-dragged with a Bobcat front-end loader to bring the moisture to the surface to assure direct contact with the ground and proper seating of the product (Figure 3-106).

Attachment 3

Amendment A: Re-Test of HESCO Bastion
(Reference 2.3)

AMMENDMENT A

Re-Test of HESCO Bastion

By Don Ward, ERDC-CHL, October 25, 2005

During the 2004 tests of flood fighting structures, seepage rates through the HESCO Bastion Concertainer™ barrier were higher than seepage rates through the other structures being tested. CHL was therefore requested by HESCO Bastion to retest the Concertainers™ in the laboratory for seepage rates to demonstrate the effectiveness of an alternate construction method. A Testing Evaluation Agreement was prepared between CHL and HESCO Bastion, with HESCO Bastion paying for all costs of the retesting. The alternate construction method consisted of wrapping plastic sheeting around the river-side wall of the structure.

A double line of putty roofing tape was placed on the floor around the outer edge of where the Concertainers™ would be erected. Plastic sheeting was placed over the putty and carefully folded at the corners to allow a single roll of sheeting to extend around the outer perimeter of the Concertainers™. At the corners, 12-in-wide duct tape was placed over the folds in the plastic. The Concertainers™ were then erected on top of the plastic, and the plastic sheeting was folded up over the outer face of the Concertainers™ and down inside the Concertainers™ prior to filling with sand. The plastic sheeting was cut where the wire mesh extended between the inner and outer walls of the Concertainers™ so the plastic could be folded inside the baskets. Plastic wire ties secured the plastic sheeting to the top of the Concertainers™. Expanding foam sealed the Concertainers™ to the wing walls and duct tape sealed the plastic sheeting to the wing walls.

Laboratory Testing – Results

The following three tables (Tables A1-A3) present the pertinent laboratory testing results. Construction of the Concertainer™ wall with plastic sheeting took slightly longer than construction without the sheeting. Other differences in construction were that the 2005 structure was assembled by an experienced HESCO Bastion team (the 2004 structure was built by laborers under the supervision of HESCO Bastion), and the 2005 structure was not covered during the large wave tests. No repairs were made to the 2005 structure during testing.

Table A-1. Effort Required to Construct, Repair, and Remove The Flood-Fighting Structures			
Structure	Construction (man-hrs)	Repairs (man-hrs)	Removal (man-hrs)
HESCO Bastion	20.8	1.8	13.4
HESCO Bastion retest	23.2	N/A	4.72

Seepage rates with the plastic sheeting were reduced by about 90 percent. Small holes in the plastic caused by the debris impact tests had no noticeable impact on seepage rates.

Table A-2. Seepage Rates During Static Head Tests				
Structure	1 ft Head (gpm / ft)	2 ft Head (gpm / ft)	95% Head (gpm / ft)	Average (gpm / ft)
HESCO Bastion	0.39	0.94	1.81	1.05
HESCO Bastion retest	0.04	0.09	0.14	0.09

gpm / ft = gallons per minute per linear foot of structure

Table A-3. Structure Damage During Laboratory Testing	
Structure	Observed Damage
HESCO Bastion	Minor Sand Settling and Washout, Some Bending of Wire During Debris Impact
HESCO Bastion Retest	Minor Sand Settling and Washout, Some Bending of Wire and Minor Tears in Plastic Sheeting During Debris Impact

Attachment 4

TVA Calculation "Modify Dams – Dam Safety" Fort Loudoun & Tellico (Reference 2.5)

TVA 10697 (OE-4-85)

OE CALCULATIONS

TITLE		PLANT/UNIT
MODIFY DAMS - DAM SAFETY		FT LOUDOUN & TELLICO
PREPARING ORGANIZATION: HEP	KEY NOUNS (Consult RIMS DESCRIPTORS LIST): CONCRETE SLABS AND WALLS	
BRANCH/PROJECT IDENTIFIERS	Each time these calculations are issued, preparers must ensure that the original (RO) RIMS accession number is filled in.	

Rev	(for RIMS' use)	RIMS accession number
R0	880923F0078	B66 '88 0122 102
R __		
R __		
R __		

APPLICABLE DESIGN DOCUMENT(S)

SAR SECTION(S) | UNID SYSTEM(S)

57

Revision 0		R1	R2	R3
ECN No. (Indicate if Not Applicable)				
Prepared	[signature]			
Checked	B. Parks			
Reviewed	[signature]			
Approved	[signature]			
Date				
Use form TVA 10534 if more room required.	List all pages added by this revision.			
	List all pages deleted of this revision.			
	List all pages changed by this revision.			

Statement of Problem

MODIFY FT LOUDOUN DAM TO PREVENT OVERTOPPING DUE TO THE PMF FLOOD. CONCRETE WALLS WILL BE DESIGNED TO RESIST HYDROSTATIC PRESSURE. SLABS WILL BE PROVIDED TO PROTECT THE EMBANKMENT ALONG THE LOCK OPERATION BLDG AREA. A PARKING AREA WILL BE PROVIDED FOR THE LOCK OPERATION BLDG. AT TELLICO, WALLS AT ENDS OF WEIR ARE DESIGNED TO WITHSTAND EARTH & SURCHARGE PRESSURES.

Abstract

cc: RIMS, SL26 C-K

TVA

REVISION LOG

Title: MODIFY DAMS

Revision No.	DESCRIPTION OF REVISION	Date Approved

TVA 10534 (EN DES-4-78)

MODIFY DAMS

SHEET 1 OF
FT LOUDOUN & TELLICO
COMPUTED GLPA DATE 14 JUL 87
CHECKED DATE

CONTENTS

TVA 11030 (WM-7-75)

MODIFY DAMS

SHEET 2 OF

FT LOUDOUN & TELLICO

COMPUTED SLPH DATE 14 JUL 87
CHECKED BP DATE NOV 3 87

PURPOSE:

MODIFY THE DAMS TO MEET HYDROLOGIC CRITERIA FOR NEW DAMS.

ASSUMPTIONS:

PMF WATER LEVELS ARE 833.1' AT FT LOUDOUN AND 829.3' AT TELLICO.

ADD WALL & BARRIER TO RAISE TOP OF EMBANKMENT AT FT LOUDOUN.

ADD UNGATED SPILLWAY AT TELLICO SADDLE DAM 1.

USE OF 3000 PSI CONC & 60 KSI REBAR.

ROLLER COMPACTED CONCRETE IN SPILLWAY.

REF:

ACI 318-83
FT LOUDOUN AND TELLICO PROJECTS - DAM SAFETY ANALYSIS REPORT = HEP - C86 - 2
AISC MANUAL
ROLLER-COMPACTED CONCRETE FOR DAMS = NORRIS-KNUDSEN ENGINEERS = EPRI AP-4715 = PROJECT 1745-16 = INTERIM REPORT = SEPT 1986
DS-C9.3.3 = DG-C1.4.3 = DS C1.5.4

3

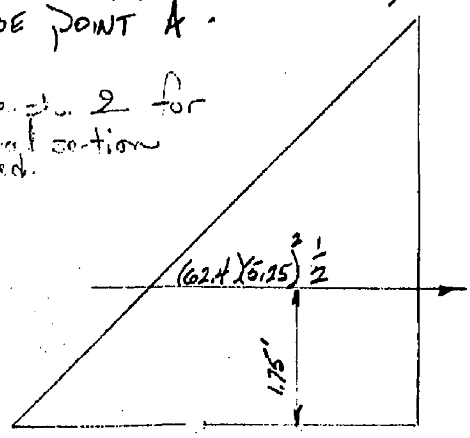
MODIFY DAMS — FT LOUDOUN & TELLICO

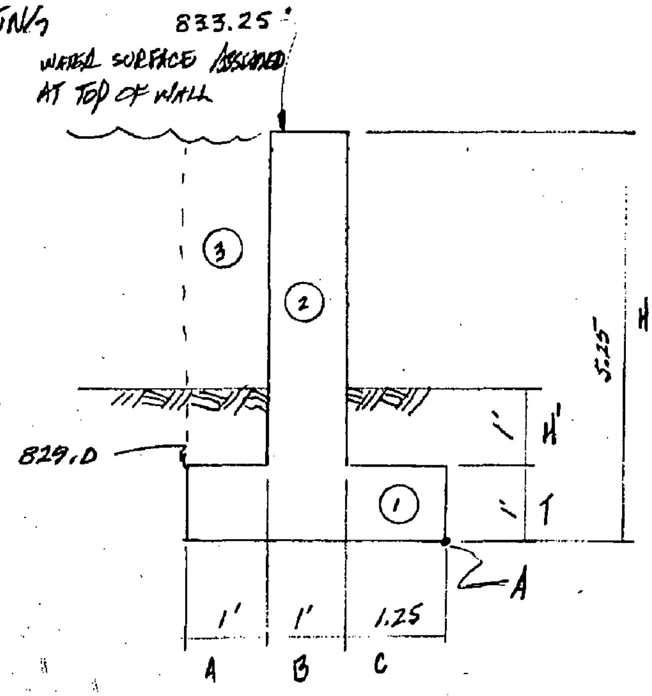
WALL STA ..+50± TO 124+40± — COMPUTED GLDA DATE 14 JUL 87

℄ SADDLE DAM ? MARINA — CHECKED BP DATE NOV 2 87

DISCOUNT EARTH FILL OVER FOOTING AND CHECK STABILITY OF WALL WITH WATER LOAD, USE POINT A.

See sh. 2 for final section used.





USE 26° AS ϕ

$H = 5.25$
$H' = 0$
$q_s = 0$
$C_s = .5$

	FORCES	DIST	M_s	LF	W_u	M_u
①	$.15 \times 1 \times 3.25 = .49$	1.625	.80	1.4	.69	1.12
②	$.15 \times 1 \times 4.25 = .64$	1.75	1.12	1.4	.90	1.57
③	$.0624 \times 1 \times 4.25 = .27$	2.75	.74	1.7	.46	1.26
	1.4		2.66		2.05	3.95

$.0624(5.25)^2 .5 (1.75) = 1.5$

$\bar{x} = \frac{2.66 - 1.5}{1.4} = .83$

$FS_{OT} = \frac{2.66}{1.5} = 1.77 > 1.5$ OK

$e = \frac{3.25}{2} - .83 = .8$

0

SHEET 4 OF

MODIFY DAMS — FT LOUDOUN & TELLICO

WALL STA 116+50± to 124+40± — COMPUTED GNPA DATE 16 JUL 87

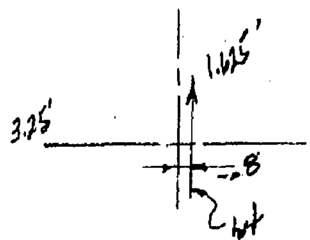
℄ SADDLE DAM @ MARINA — REV CHECKED BP DATE NOV 3 87

See sh. 2 for final section used.

1' STRIP

$A = (1)(3.25) = 3.25\ FT^2$

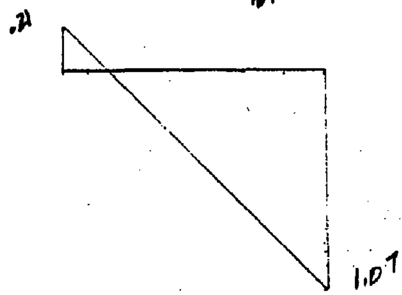
$I = \frac{3.25^3}{12} = 2.86\ FT^4$



$\frac{P}{A} \pm \frac{Mc}{I}$

$\frac{1.4}{3.25} \pm \frac{1.4(.8)(1.625)}{2.86}$

$.43 \pm .64$



ALLOWABLE BEARING FOR THIS INSTANCE IS ASSUMED AS 2.5 KSF

1.07 < 2.5 OK

CHECK SLIDING

$f_Y = 1.4^K$ (WITH WATER) $\tan\phi = .49 = 4/5$ USE .5

$f_H = .86^K$

$(.5)\frac{1.4}{.86} = .81 < 1.5$ NOT OK FOR SLIDING

CHANGE GEOMETRY OF WALL AND CHECK AGAIN WITH DIFFERENT ASSUMPTIONS.

TVA 11030 (WM-7-75)

(2)

SHEET 5 OF

MODIFY DAMS — FT LOUDOUN & TELLICO

WALL STA 116+50± to 124+40± & SADDLE DAM @ MARINA

COMPUTED GLPH DATE 17 JUL 87
REV. CHECKED EP DATE NOV 3 87

SAY $\phi = 26°$
$K_a = .4$
$K_p = \frac{1}{K_a} = 2.5$ (USE 1.12 FOR SLOPE)

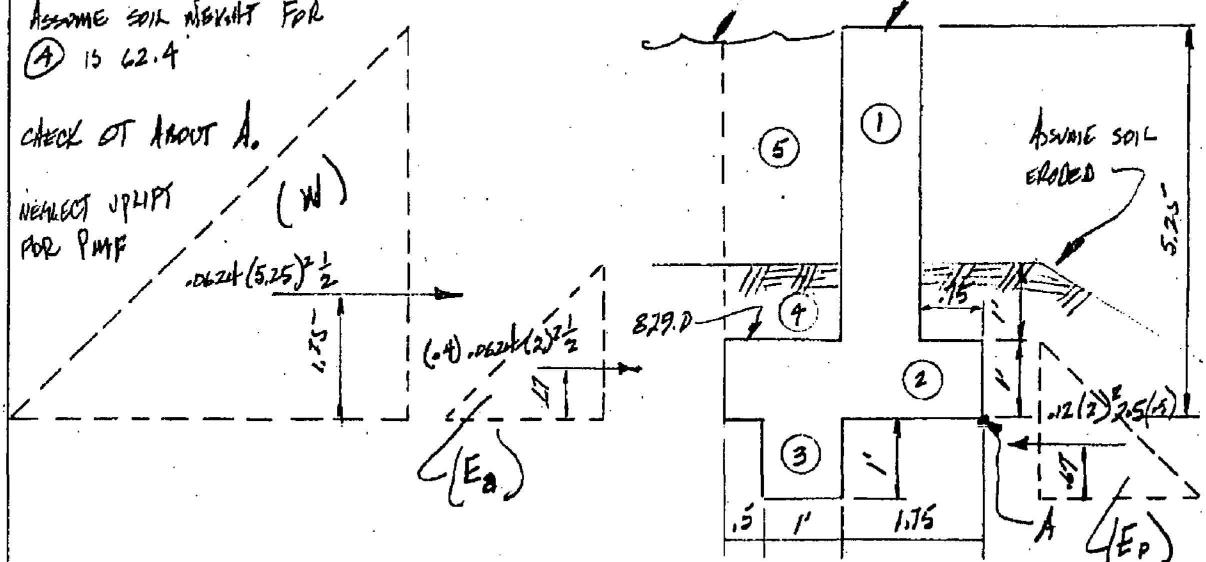
ASSUME SOIL WEIGHT FOR (4) IS 62.4

CHECK OT ABOUT A.

NEGLECT UPLIFT FOR PMF

WATER SURFACE ASSUMED AT TOP OF WALL

See sh 9 for final section used.



FORCES	DIST	M_S	LF	W_U	M_U
① .15(4.25)(1) = .64	1.25	.8	1.4	.9	1.12
② .15(3.25)(1) = .49	1.625	.8	1.4	.69	1.12
③ .15(1)(1) = .15	2.25	.34	1.4	.21	.48
④ .0624(1)(1.5) = .09	2.5	.23	1.4	.13	.32
⑤ .0624(1.5)(4.25) = .40	2.5	1.0	1.7	.68	1.7
1.77		3.17		2.61	4.74

$.0624(5.25)^2\frac{1}{2}(1.75) + .0624(2)^2\frac{1}{2}(.67)(.4) = M_{OT} = 1.54$

$FS_{OT} = \frac{3.17}{1.54} = 2.1 > 1.5$ OK

$\bar{x} = \frac{3.17 - 1.54}{1.77} = .92$

$e = \frac{3.25}{2} - .92 = .71$

SHEET 6 OF

MODIFY DAMS — FT LOUDOUN & TELLICO

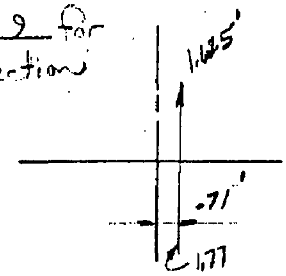
WALL STA 116+50± to 124+40± & SADDLE DAM @ MARINA — COMPUTED GLPH DATE 17 JUL 87 — REV CHECKED BP DATE NOV 3 '87

1' STRIP (NEGLECT KEY)

See sh. 9 for final section used.

$A = (1)(3.25) = 3.25\ FT^2$

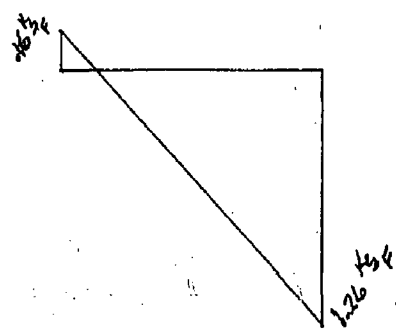
$I = \frac{3.25^3}{12} = 2.86\ FT^4$



$\frac{P}{A} \pm \frac{Mc}{I}$

$\frac{1.77}{3.25} \pm \frac{1.77(.71)(1.625)}{2.86}$

$.55 \pm .71$



MAX BEARING IS $1.26\ ^{k}/_{F} < 2.5\ ^{k}/_{F}$ OK

CHECK SLIDING

$f_V = 1.77$ (WITH WATER) $\tan\phi = .49$ $f_H = W + E_a$

$= .86 + .05 = .91$

$\frac{.49(1.77) + .27}{.91} = 1.3 < 1.5$

NOT GOOD ENOUGH!

$E_p = .12(2)^2(1.12)(.5) = .27$

SHEET 7 OF

MODIFY DAMS — FT LOUDOUN & TELLICO

WALL STA 116+50± to 124+40± — COMPUTED GRA DATE 17 JUL 87

℄ SADDLE DAM @ MARINA — CHECKED BP DATE NOV 3 '87

SIZE RESTEEL

$\Sigma M_U = 4.74 - 1.7(1.54) = 2.12$

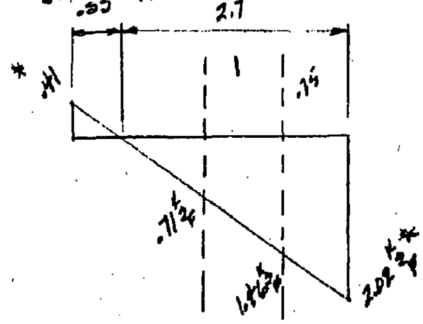
$\bar{x} = \frac{2.12}{2.61} = .81$

$e = \frac{3.25}{2} - .81 = .82$

$q_{MAX} = \frac{2.61}{3.25}\left[1 + \frac{6(.82)}{3.25}\right] = 2.02^{k/\square F}$

$q_{MIN} = \frac{2.61}{3.25}\left[1 - \frac{6(.82)}{3.25}\right] = -.41^{k/\square F}$

* From similar separate analysis using final cross-section, the results are very close to those shown.



RE DS-C9.3.3

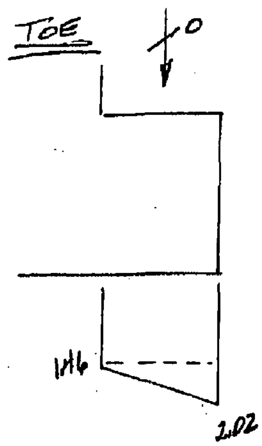
$V = 2.61 \quad M_U = 2.12 \quad H = .82 \quad H/B = \frac{.82}{3.25} = .25 \quad N = 2.7$

$P_{MAX} = 2.17^{k/\square F} \quad K/B = .74 \quad K = .74(3.25) = 2.41'$

$\frac{1}{2}(2.17)(2.41) = 2.62^k$

RESULTS ARE SIMILAR

TOE



$SHEAR = 1.46(.75) + (.56)(.75).5 = 1.31^k$, $v_u = V_u/\phi bd = .014$ ksi ok

$M = 1.46(.75)\frac{.75}{2} + .56(.75).5\left(\frac{.75(2)}{3}\right) = .52'^k$

$a_u = 4.49$ $\quad A_s = \frac{.52}{4.49(9)} = .01\ \text{in}^2$ $\quad d = 9$

TVA 11030 (WM-7-75)

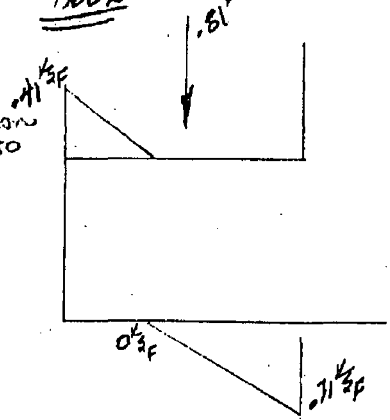
8

MODIFY DAMS — FT LOUDOUN & TELLICO

WALL — STA 116+50± to 124+40± — COMPUTED GPA DATE 12 JUL 87
E SADDLE DAM @ MARINA — CHECKED BP DATE NOV 3 87

HEEL

Assume no adhesion of conc. to soil.

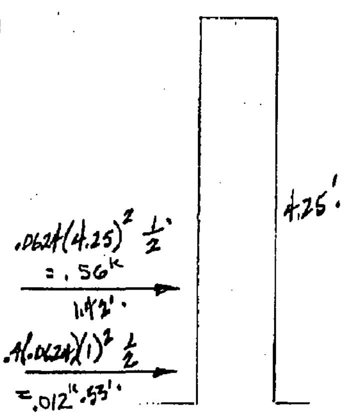


$SHEAR = .81^k + .41(.55)(.5) - .71(.95)(.5) = .59^k$

$M = .81(.75) + .41(.55).5(1.32) - .71(.95)(.5).32 = .65^{'k}$

$a_u = 4.49 \qquad A_s = .02\ in^2$

STEM



$SHEAR = .0624(4.25)^2 \frac{1}{2} + .4(.0624)(1)^2 \frac{1}{2} = .58^k$

$M = .0624(4.25)^2 \frac{1}{2}(1.42) + .4(.0624)(1)^2 \frac{1}{2}(.33) = .8^{'k}$

$a_u = 4.49 \qquad A_s = .02\ in^2$

$V_u = \frac{1.7(.58)}{.85(12)9} = .011^{ksi}$ ok

$A_s \cong \frac{.8(1.7)12}{.9(.9\times)9)60} = .04\ in^2$

$\frac{4}{3}$ OF CALC STEEL IS LESS THAN $.2\ in^2$ USE #4 @ 12 VERTICAL ok

FE DS C1.5.4

$L' = \cancel{B2B}^{\sim 4.0} \lessgtr 1.5H = 6.4'$ CASE I Pg 9
BASED ON 40' LENGTH OF WALL

USE MINIMUM STEEL:

$A_s = \rho\, bd = .0015(12)(9) = .162$ SAY #4 @ 12 EF HORIZONTAL

ACT $A_{s\,HOR} = .0025(12)12 = .36\ in^2$ or $.18\ in^2/face$

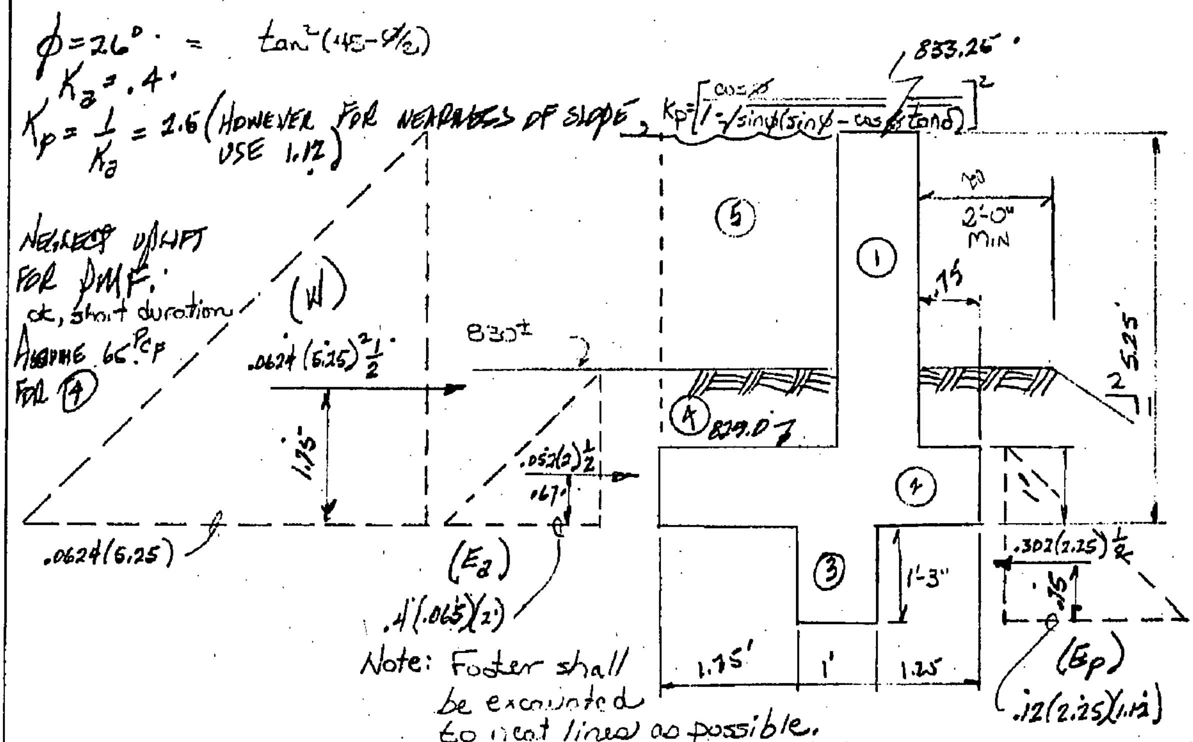
TVA 11030 (WM-7-75)

SHEET 9 OF

MODIFY DAMS — FT LOUDOUN & TELLICO

WALL STA 116+50± to 124+40± — COMPUTED GLRH DATE 20 JUL 87

℄ TELLICO DAM & MARINA — CHECKED BP DATE NOV 2 87



FORCES	DIST	M_s	LF	M_u	M_u
(1) $.15(4.25)(1) = .64$	1.25	.8	1.4	.9	1.12
(2) $.15(4)(1) = .6$	2	1.2	1.4	.84	1.68
(3) $.15(1)(1.25) = .19$	1.75	.33	1.4	.27	.47
(4) $.065(2.25)(1) = .15$	2.875	.43	1.4	.21	.60
(5) $.0624(2.25)(4.25) = .60$	2.875	1.73	1.7 or 1.4	1.02, .84	2.94, 2.42
2.18		4.49	whichever is worst	3.24, 3.06	6.81, 6.29

$.0624(6.25)^2\frac{1}{2}(1.75) + .052(2)\frac{1}{2}(.67) = M_{OT} = 1.54$

$FS_{OT} = \frac{4.49}{1.54} = 2.92 > 1.5$ OK

$\bar{x} = \frac{4.49 - 1.54}{2.18} = 1.35 \quad e = 2 - 1.35 = .65$

TVA 11030 (WM-7-75)

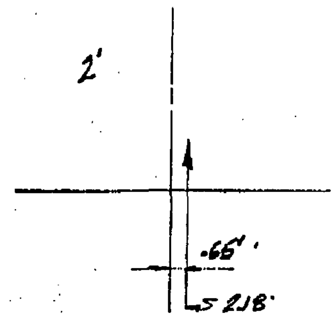
(7)

SHEET 10 OF

MODIFY DAMS — FT LOUDOUN Z TELLICO

WALL — STA 116+50± to 124+40± — COMPUTED GLRH DATE 20 JUL 87
℄ SADDLE DAM @ MARINA — CHECKED BP DATE NOV 2 87

1' STRIP (NEGLECT KEY)



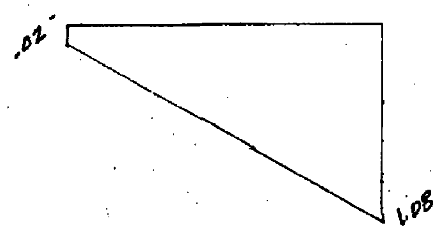
$A = 4\ FT^2$

$I = \frac{4^3}{12} = 5.33\ FT^4$

$\frac{P}{A} \pm \frac{Mc}{I}$

$\frac{2.18}{4} \pm \frac{2.18(.65)(2)}{5.33}$

$.55 \pm .53$



MAX BEARING IS $1.08\ ^{K}/_{F}$ < $2.5\ ^{K}/_{F}$ OK

CHECK SLIDING

$f_v = 2.18$ $\quad\tan\phi = .49$

$f_h = W + E_a = .86 + .05 = .91$ $\quad E_p = .302(2.25)(.5) = .34$

$\frac{(.49)2.18 + .34}{.91} = 1.55$

$FS_{SL} = 1.55 > 1.5$ OK

CHANGES IN GEOMETRY OF FDN ARE SMALL ENOUGH NOT TO ALTER CONCLUSION FOR RESTEEL FIGURED PREVIOUSLY.

SHEET 11 OF

MODIFY DAMS — FT LOUDOUN & TELLICO

WALL STA 116+50± to 124+40± & SADDLE DAM @ MARINA

COMPUTED GLEH DATE 3 SEP 87
REV BP DATE NOV 4 87

RE-CHECK WALL FOR DEEPER FOOTING & LONGER STEM

FORCES	DIST	M_S	LF	W_U	M_U
① .15 (5.75)(1) = .86	1.25	1.08	1.4	1.2	1.51
② .6	2	1.2	1.4	.84	1.68
③ .19	1.75	.33	1.4	.27	.47
④ .065 (2.25)(2.5) = .37	2.875	1.05	1.4	.52	1.47
⑤ .0624 (2.25)(5.75) = .81	2.875	2.32	1.4	1.13	3.25
2.83		5.98		3.96	8.38

$$M_{OT} = .0624(6.75)^2 .5(2.25) + .091(3.5)\tfrac{1}{2}(1.17) = 3.38$$

$$FS_{OT} = \frac{5.98}{3.38} = 1.77 > 1.5 \quad \underline{OK}$$

$$FS_{SL} = \frac{.49(2.83) + .71}{1.42 + .16} = 1.33 < 1.5 \quad \text{UNSAT}$$

DON'T TAKE FOOTING DEEPER — USE FILL MATERIAL TO CHANGE SLOPE. - DEEPER FOOTING MUST BE WIDER FOR STABILITY AND THIS MOVES WALL & END OF FOOTING AWAY FROM BRIDGE BENT FOOTINGS - THIS LATERAL MOVEMENT CAUSES FOOTING TO BE NEARER TO SLOPE, THUS LOSING THE BENIFIT OF PASSIVE RESISTANCE GAINED BY GOING DEEPER, STAY WITH ORIGINAL APPROACH

TVA 11030 (WM-7-75)

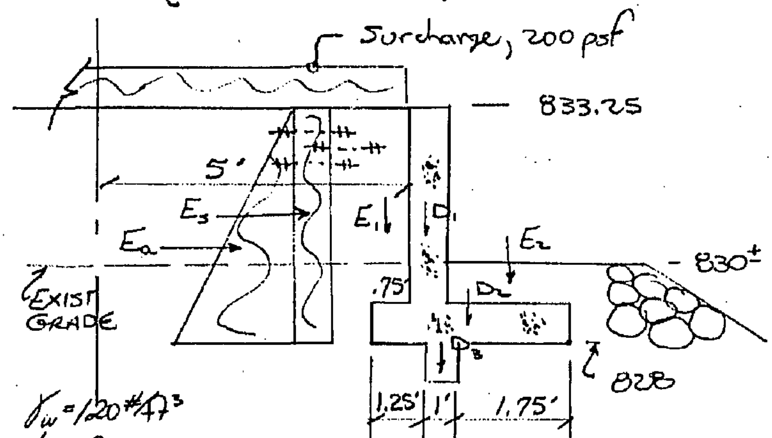
SHEET 11A OF

MODIFY DAMS
RAMP ACCESS

FORT LOUDOUN
PROJECT

COMPUTED BB DATE JAN 25 '88
CHECKED DATE 27 Jan 88

A portion of the wall will be backfilled on the downstream side to provide access across wall. (THIS IS FOR TEMPORARY ACCESS ONLY)



$\gamma_w = 120\#/ft^3$
$\phi = 30°$
$K_a \cong .33$

M_o:

				M_o
$E_a = \frac{1}{2}(.12).33(5.25)(5.25) = .55^k$	@	$5.25/3$	=	$.96^{'k}$
$E_s = .2(.33)(5.25) =$	$.35^k$	$5.25/2$	=	$.92^{'k}$
	$.90^k$			$1.88^{'k}$

M_R:

$D_1 = 1(4.25).15 = .64$	@ 2.75	=	1.75
$D_2 = 4(1).15 = .6$	2	=	1.20
$D_3 = 1(1.25).15 = .19$	2.25	=	.42
$E_1 = 4.25(.12).75 = .38$	3.63	=	1.40
1.81^k			$4.77^{'k}$

$\tan\phi = .577$

Calculate $FS_{SLIDING} = \frac{1.81(.577)}{.9} = 1.16$
" $FS_{OT} = 4.77/1.88 = 2.5$

} ok, considering this case will be only encountered at full height of fill at a corner in the wall.

$M_{IN\ STEM} \cong 1.7(1.88^{'k}) = 3.2^{k'}$
$A_s \cong 3.2(12)/.9(.9\times9)60 = .09\ in^2/ft.$ #4@12" o.c.

TVA 11030 (WM-7-75)

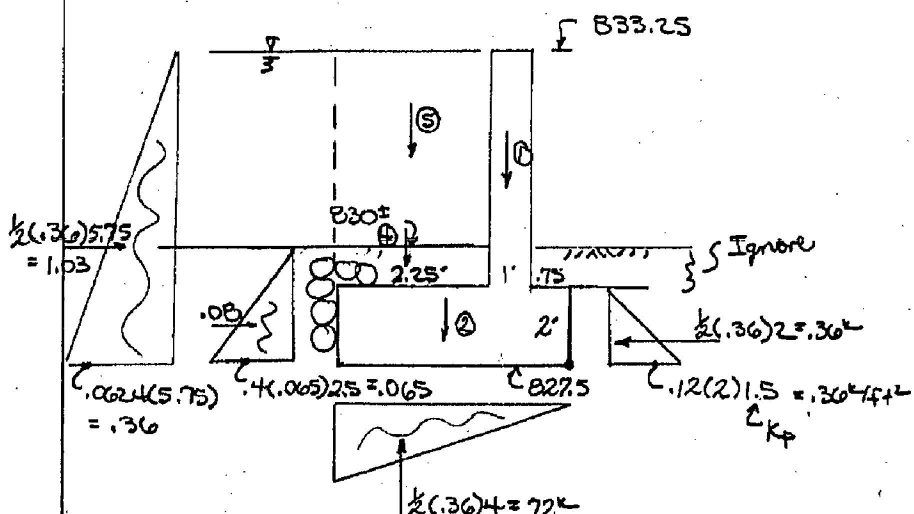
SHEET 11B OF

MODIFY DAM
WALL - TOP OF DAM
BLKS 5 & 6
Construction Request

FT LOUDOUN
PROJECT

COMPUTED BP DATE APR 14 '88
CHECKED DATE 27 JUL 88

Refer to comps, sh. 9;



Force	x or y	M_R	M_O
① 3.75(.15)1 = .56	1.25	.7	
② 4(2).15 = 1.2	2	2.4	
④ .065(2.25).5 = .07	2.875	.2	
⑤ .0624(2.25)3.75 = .51	"	1.5	
Uplift .72^k ↑	2.67		1.92
Σ = 1.61^k ↓			
1.03^k		1.9	1.96
.08^k		.8	.06
Σ 1.11^k →		7.5	3.94

$FS_{OT} = 7.5/3.94 = 1.9$ $\quad FS_{SLIDING} = \frac{1.61(.49)+.36}{1.11} = 1.04$

Make ftg. wider or deeper, if possible.
if 5' wide

$\Delta DL = [1'(.0624)3.75 + 1'(.065)(.5) + 2'(1').15] - .18$ ← ΔUplift
$= .4^k$

$FS_{SLIDING} = \frac{2.01(.49)+.36}{1.11} = 1.21$ ∴ say ok for PMF condition

TVA 11030 (WM-7-75)

11C

GATE TO HANDICAP ACCESS RAMP AT LOCK OPERATIONS BLDG AREA

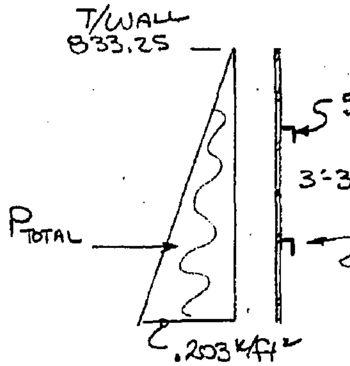
FT LOUDOUN PROJECT

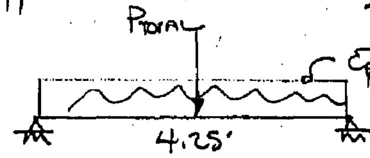
COMPUTED BP DATE APR 8 '88
CHECKED [illegible] DATE 29 Jul 88

Calculate size of PL req'd to withstand water pressure,

$L_{PL} = 4'\text{-}3''$ Opening

PL supported on two sides,





$= 1.4^k / 4.25' = .33^{k}/ft.$

$P_{TOTAL} = \frac{1}{2}(.203)3.25(4.25')$
$= 1.4^k$

$M = \frac{wl^2}{8} = \frac{.33(4.25)^2}{8} = .75'^k$

$I_{PL} = 3.25(12)\, t^3/12 = 3.25t^3 \qquad c = t/2 \qquad S = I/c = 6.5t^2$

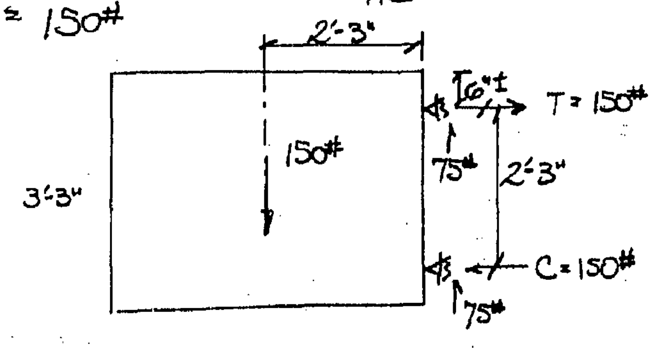
$f = Mc/I = M/S$, use $f_{all} = 24$ ksi

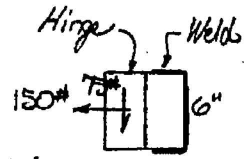
$24 = M/S = .75(12)/6.5t^2$
$t \geq .24''$

Chk deflection if 200# force is applied,

$\Delta = Pl^3/48EI = .2^k(4.25 \times 12)^3 / 48(29000)(3.25 \cdot .25^3)$
$= \frac{3}{8}''$, ok

$Wgt = \frac{1}{4}''(4.5')3.25'(490\#/ft^3)/12$
$\approx 150\#$





Worst case is w/ gate opened, shear & bending in weld.

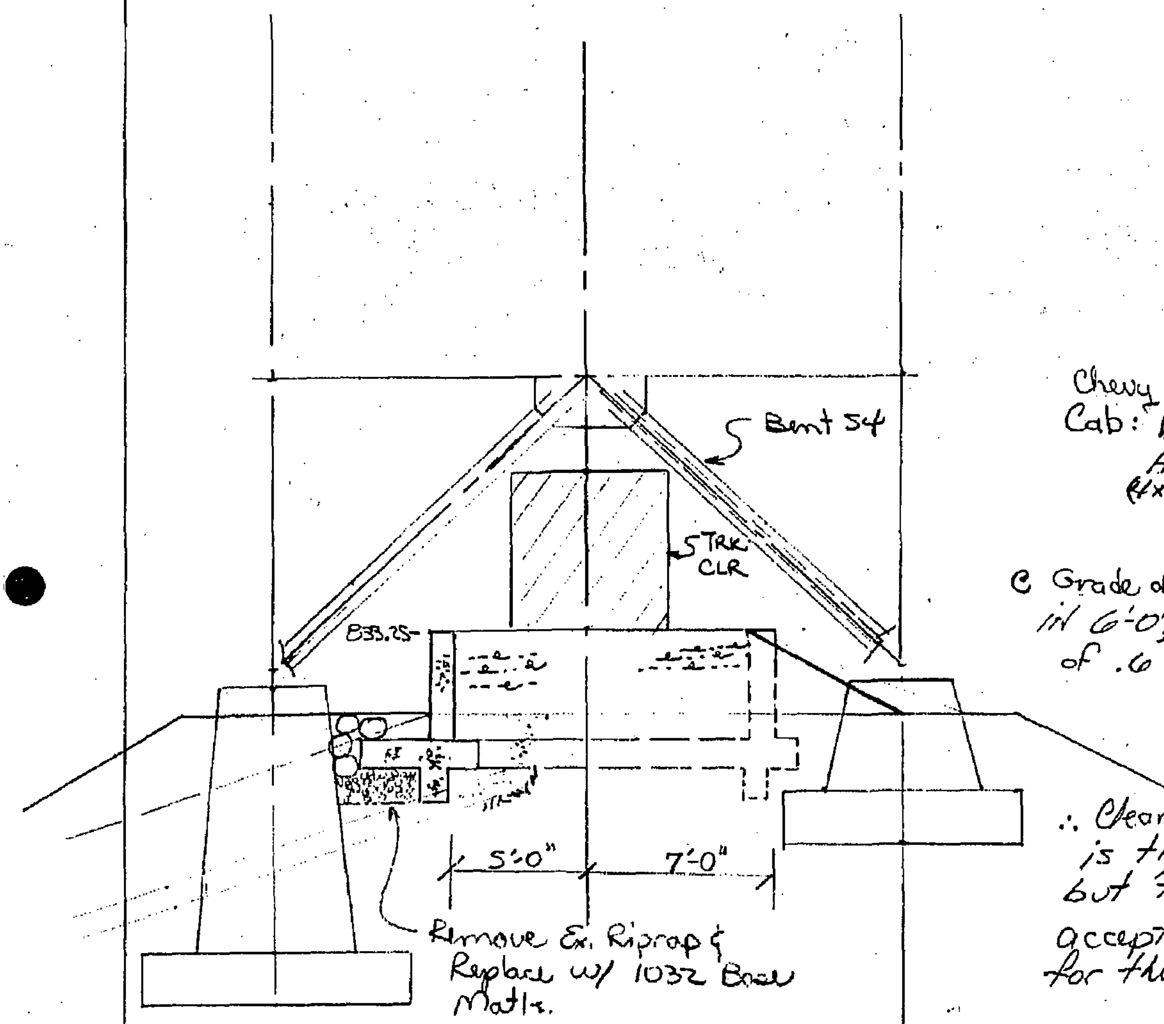
TVA 11030 (WM-7-75)

11D

Modify Dam
Flood Wall
Access Area

Fort Loudoun
Project

COMPUTED [signature] DATE JAN 25 88
CHECKED [signature] DATE 27 JUL 88



Chevy Truck,
Cab: Width = 76.8"
Height = 74.3"
(4x4)

℄ Grade of ~10%
in 6'-0", drop
of .6 ft.

∴ Clearance
is tight,
but this is
acceptable
for this area.

Section @ 54±
1" = 6'

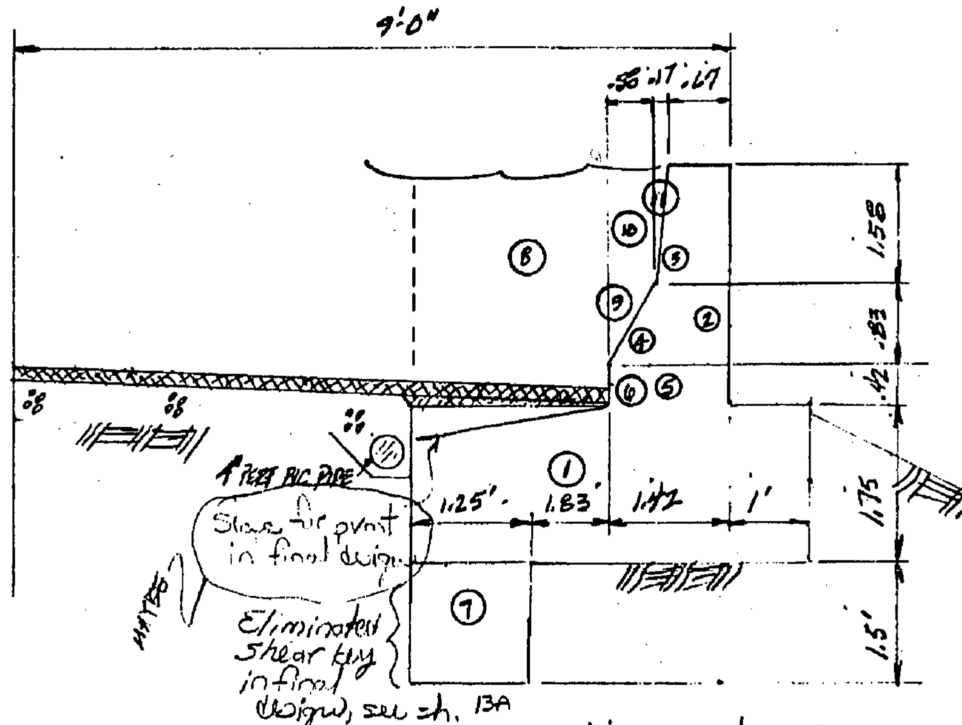
TVA 11030 (WM-7-75)

SHEET 12 OF

MODIFY DAMS — FT LOUDOUN & TELLICO

BARRIER

COMPUTED GLW DATE 2 NOV 87
CHECKED BP DATE NOV 3 87



FORCES	DIST	Ms	LF	Wu	Mu
① .15(5.5)(1.75) = 1.44	2.75	3.96	1.4	2.02	5.54
② .15(.67)2.83 = .28	1.34	.38	1.4	.39	.53
③ .15(.17)1.58(.5) = .02	1.78	.04	1.4	.03	.06
④ .15(.83).58(.5) = .04	2.03	.08	1.4	.06	.11
⑤ .15(1.25).17 = .05	1.76	.09	1.4	.07	.13
⑥ .15(.58).42 = .04	2.13	.09	1.4	.06	.13
⑦ .15(1.25)(1.5) = .28	4.875	1.37	1.4	.39	1.72
⑧ .0624(3.08)(1.83) = .54	3.96	2.14	1.7	.92	3.64
⑨ .0624(.83).58(.5) = .02	2.23	.05	1.7	.03	.09
⑩ .0624(.58)1.58 = .06	2.13	.13	1.7	.1	.22
⑪ .0624(.17)1.58(.5) = .01	1.78	.02	1.7	.02	.03
2.78		8.35		4.09	12.4

TVA 11030 (WM-7-75)

SHEET 13 OF

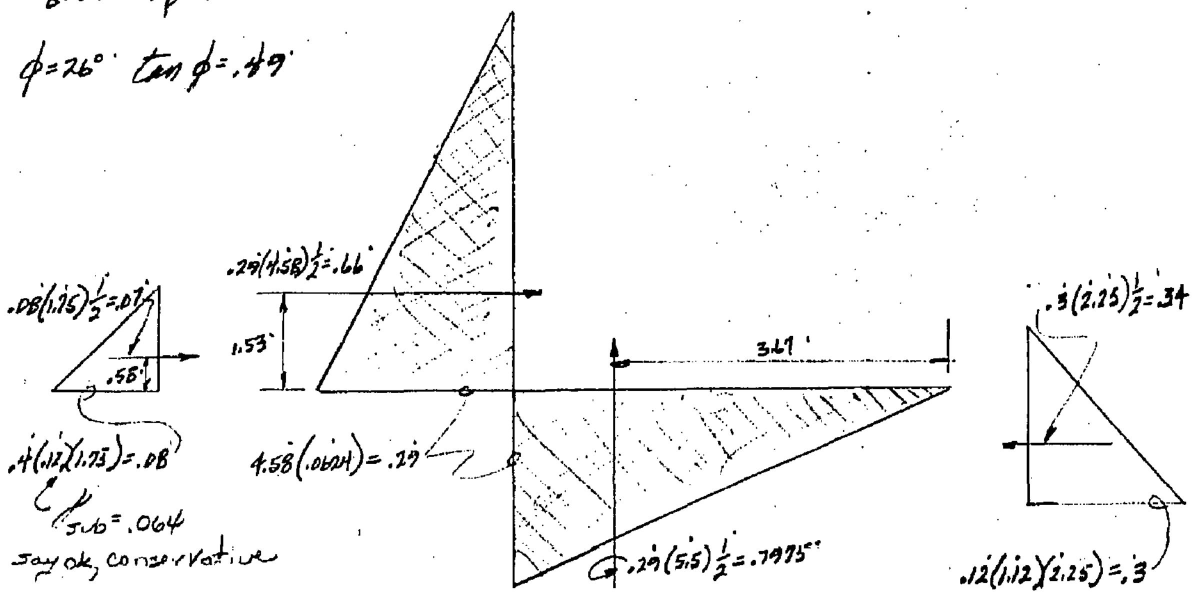
MODIFY DAMS — FT LOUDOUN & TELLICO

BARRIER

COMPUTED GLRH DATE 23 JUL 87
CHECKED BP DATE NOV 4 87

$K_a = .4 \quad K_p = 1.12$

$\phi = 26° \quad \tan\phi = .49$



$\Sigma M_{OT} = .66(1.53)+.07(.58)+.80(3.67) = 4^{1\text{-}k}$

$FS_{OT} = \dfrac{8.35 - 2.95}{\cancel{1.01+.04}\ 4} = \dfrac{\cancel{5.2}}{2.1} > 1.5$ ok

$FS_{SL} = \dfrac{.49(2.78 - .7975) + .34}{(.66) + (.07)} = 1.8 > 1.5$ ok

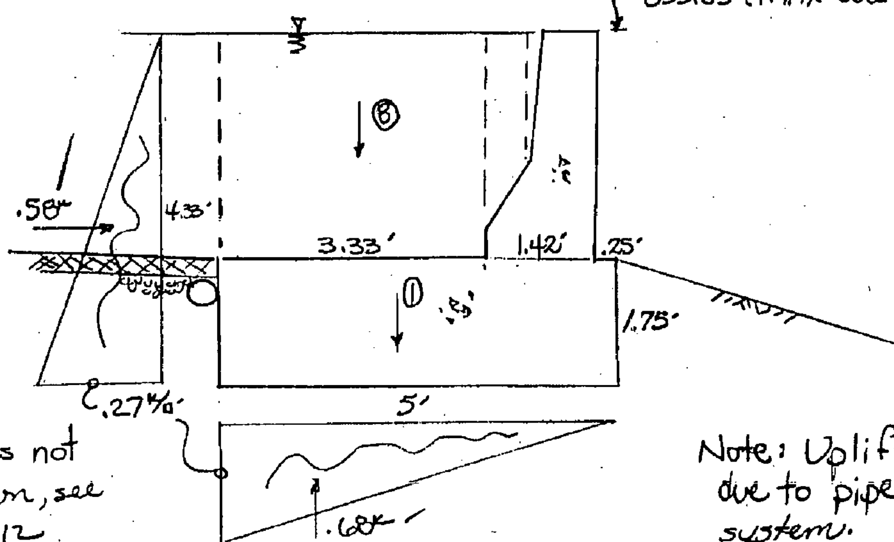
TVA 11030 (WM-7-75)

13

Modify Dam
Roadway Barrier

13A
FT LOUDOUN
PROJECT
COMPUTED BP DATE MAY 12 88
CHECKED GLDW DATE 23 MAY 88

Stability calculations,



Forces not shown, see sh. 12

	Forces	Dist.	M_R	M_O
①	1.75'(5').15 = 1.31k	2.5'	3.28	
②	= .28	.59	.16	
③	= .02	1.03	.02	
④	= .04	1.28	.05	
⑤	= .05	1.01	.05	
⑥	= .04	1.38	.06	
⑧	3.33(.0624)2.83 = .59	3.34	1.97	
⑨	= .02	1.48	.03	
⑩	= .06	1.38	.08	
⑪	= .01	1.03	.01	
	2.43k		5.71 1-k	
W_u	= .68k↑	3.33		2.27
W_L	= .58k→	1.44		.84
E_L	= .07k→	.58		.04
	1.75k↓, .65k→			3.15 1-k

$FS_{OT} = 5.71/3.15 = 1.81$ ok

$FS_{SLIDING} \Rightarrow$ Use $\mu \cong .4$

$FS_{SL} = 1.75(.4)/.65 = 1.1$ ok for PMF per RDM 4-11-88

TVA 11030 (WM-7-75)

SHEET 14 OF

MODIFY DAMS — FT LOUDOUN & TELLICO

BARRIER — COMPUTED GLRH DATE 24 JUL 87

CHECKED BP DATE NOV 3 87

$$\bar{x} = \frac{8.35' - 2.93' - 1.01' - .04}{2.78 - .8} = \cancel{1.97}\ 2.2'$$

$$e = 2.75 - \cancel{1.57}^{\,2.2} = \cancel{1.18}\ .55$$

1' STRIP

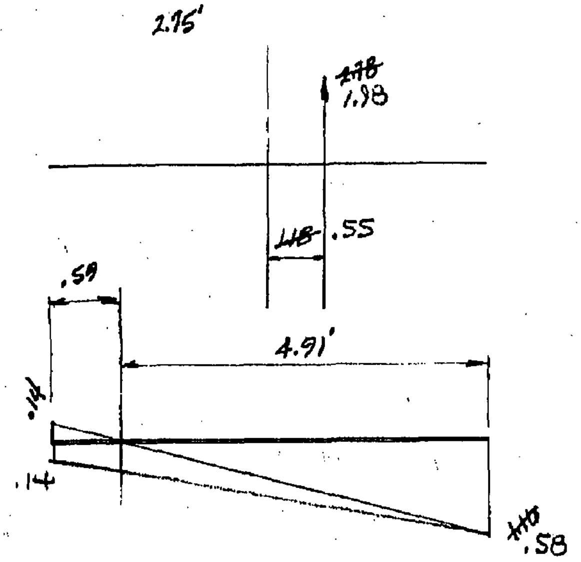
$A = 5.5'\ FT^2$

$$I = \frac{5.5^3}{12} = 13.86'\ FT^4$$

$$\frac{P}{A} \pm \frac{Mc}{I}$$

$$\frac{\cancel{2.78}\ 1.98}{5.5} \pm \frac{\cancel{2.78}\ 1.98\ (\cancel{1.18}\ .55)(2.75)}{13.86}$$

$\cancel{.51}\ .36 \pm \cancel{.65}\ .22$



CHECK TRAFFIC LOAD WITH THESE DIMENSIONS.

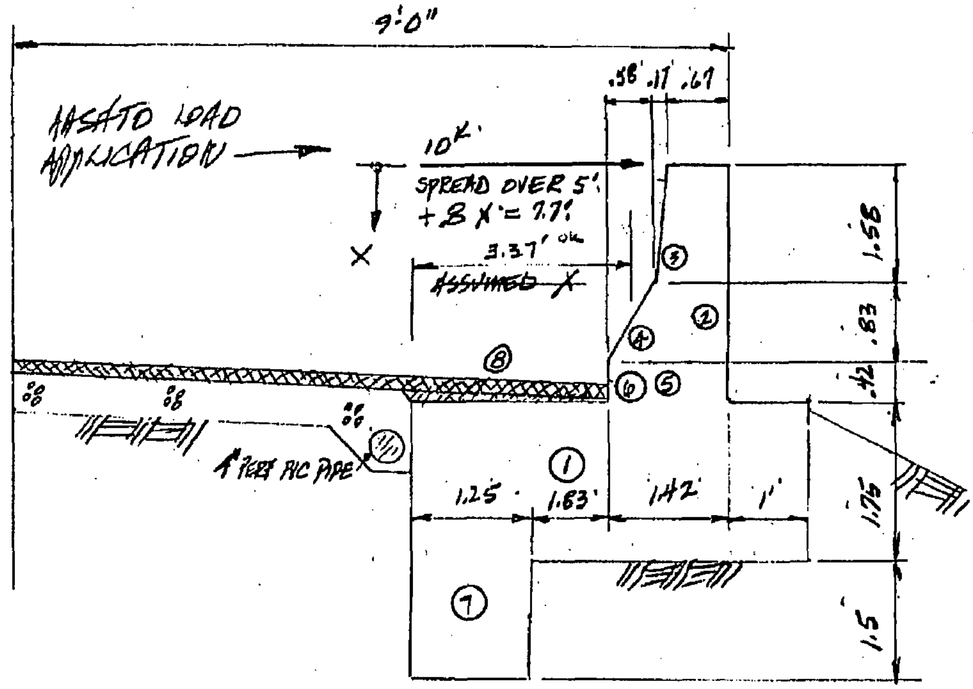
TVA 11030 (WM-7-75)

SHEET 15 OF

MODIFY DAMS — FT LOUDOUN & TELLICO

BARRIER (TRAFFIC) — COMPUTED GHH DATE 21 Oct 87

CHECKED BP DATE Nov 4 87



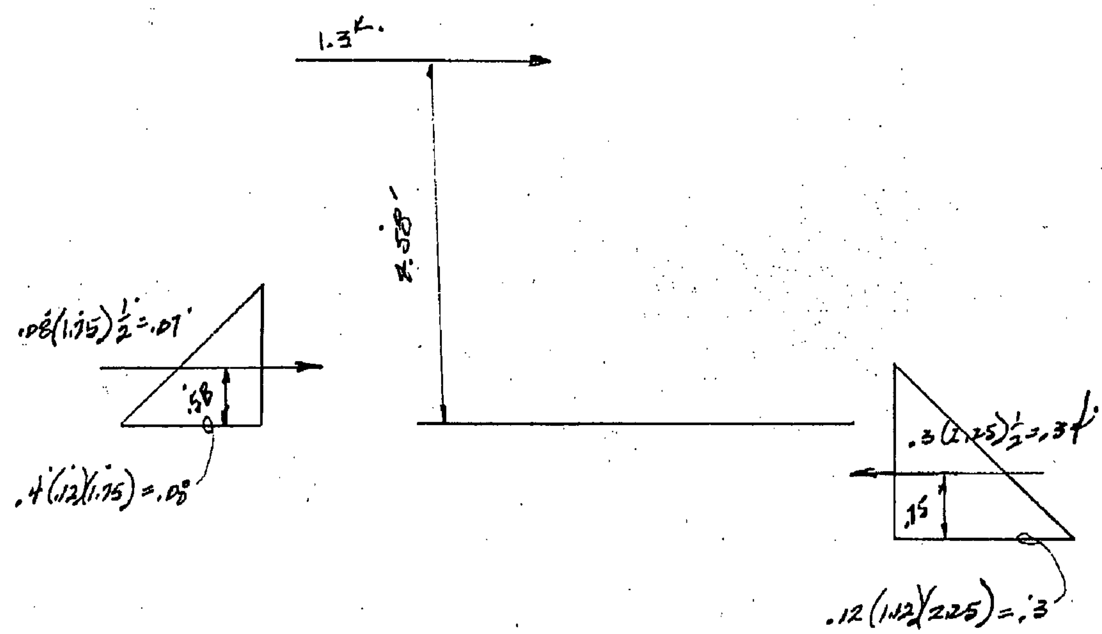
FORCES		DIST	M_S	LF	W_U	M_U
1	1.44	2.75	3.96	1.4	2.02	5.54
2	.28	1.34	.38	1.4	.39	.53
3	.02	1.78	.04	1.4	.03	.06
4	.04	2.03	.08	1.4	.06	.11
5	.05	1.76	.09	1.4	.07	.13
6	.04	2.13	.09	1.4	.06	.13
7	.28	4.875	1.37	1.4	.39	1.92
8	.15(3.08)(.25)=.12	3.96	.48	~~1.7~~ 1.4	.2	~~.82~~ .7
	2.27		6.49		3.22	9.24

SHEET 16 OF

MODIFY DAMS — FT LOUDOUN & TELLICO

BARRIER (TRAFFIC)

COMPUTED GLPH DATE 24 JUL 87
CHECKED BP DATE NOV 4 87



$FS_{OT} = \dfrac{6.49'}{5.95+.04'} = 1.1' < 1.5$ ≡ HOWEVER, THIS LOAD SITUATION IS OF SHORT DURATION FROM VEHICLE IMPACT.

$FS_{SL} = \dfrac{.49(2.27)+.34}{1.3+.07} = 1.1' < 1.5$ ≡ HOWEVER, THIS LOAD SITUATION IS OF SHORT DURATION FROM VEHICLE IMPACT.

SINCE THE SHORT DURATION LOAD WILL NOT CAUSE A SHEAR FAILURE THROUGH THE WHOLE BARRIER AND SINCE THE LOAD APPROACHES BEING A CONCENTRATED LOAD IT IS NECESSARY FOR A COMPLETE SECTION OF THE BARRIER TO OVERTURN OR SLIDE. A 40' ± SECTION OF BARRIER WILL NOT SLIDE OR OVERTURN.

(SEE NEXT PAGE)

TVA 11030 (WM-7-75)

SHEET 17 OF

MODIFY DAMS — FT LOUDOUN & TELLICO

BARRIER (TRAFFIC)

COMPUTED GLPH DATE 2 JUL 87
CHECKED BP DATE NOV 4 87

WITH A SECTION OF 40'± BARRIER, ESTIMATE A MORE REALISTIC SET OF FACTORS FOR OVERTURNING AND SLIDING:

$\frac{40'}{7.7} = 5.2$ APPROX 5 TIMES MORE BARRIER EXISTS THAN IS TREATED TO CALCULATE FACTORS ORIGINALLY.

$$FS_{OT} = \frac{6.49(40)}{45.8 + .24(40)} = 5.5$$

$$FS_{SL} = \frac{.49(2.27)(40) + .34(40)}{10' + .07(40)} = 4.5$$

THIS SECTION PROPERLY REINFORCED WILL BE SATISFACTORY.

SHEET 18 OF

MODIFY DAMS — FT LOUDOUN & TELLICO

BARRIER (TRAFFIC)

COMPUTED GHBA DATE 27 JUL 87
CHECKED BP DATE NOV 4 '87

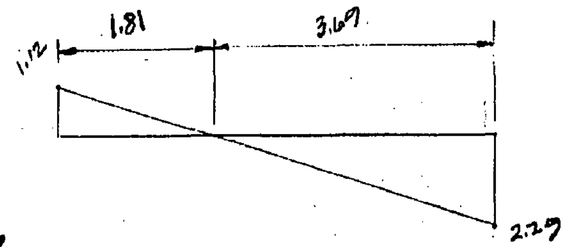
$\Sigma M_v = 9.24 - 5.95(1.1) - .04(1.4) = \cancel{.26}\ .93$

$\bar{x} = \frac{\cancel{.26}\ .93}{3.22} = \cancel{.08}\ .29$

$e = \frac{5.5}{2} - \cancel{.08}\ .29 = \cancel{2.67}\ 2.46$

$q_{MAX} = \frac{3.22}{5.5}\left[1 + \frac{6(2.67)}{5.5}\right] = 2.29$

$q_{MIN} = \frac{3.22}{5.5}\left[1 - \frac{6(2.67)}{5.5}\right] = -1.12$



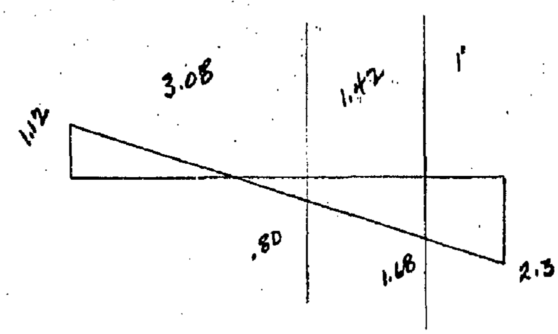
$A = 5.5\ FT^2$

$I = \frac{5.5^3}{12} = 13.86\ FT^4$

$\frac{P}{A} + \frac{Mc}{I}$

$\frac{3.22}{5.5} \pm \frac{3.22\left(\cancel{2.67}\ 2.46\right)(2.75)}{13.86}$

$.59 \pm \cancel{1.71}\ 1.57$



RESULTS COMPARE   From "Eccentrically Loaded Bases"

$e/B = 2.46/5.5 = .45 \quad \therefore N \cong 12$

$P_{max} = NV/AB = \frac{12(3.22)}{1(5.5)} = 7\ k/ft^2$

Short term loading, ok

TVA 11030 (WM-7-75)

SHEET 19 OF

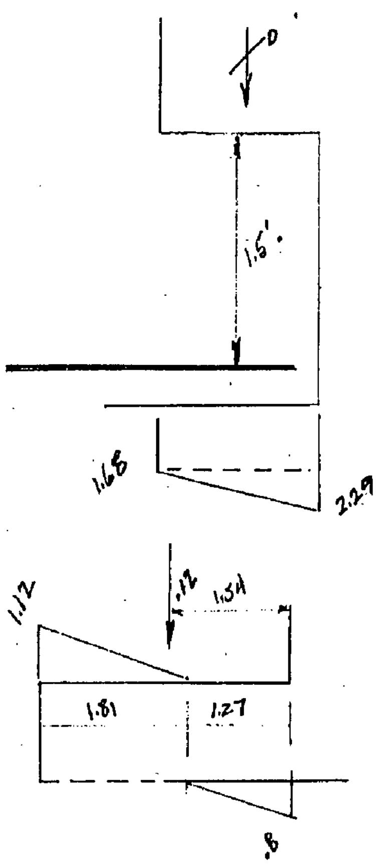
NORRIS DAMS — FT LOUDOUN & TELLICO

BARRIER (TRAFFIC) — COMPUTED GLEH DATE 27 JUL 87

REV BP DATE NOV 4 '87

Footing reinf. ok due to shape.

SIZE RE STEEL



$$SHEAR = 1.68(1) + .61(1)\tfrac{1}{2} = 1.99^{k}$$

$$M = 1.68(1)\tfrac{1}{2} + .61(1)\tfrac{1}{2}\left(\tfrac{1(2)}{3}\right) = 1.04^{'k}$$

$$a_u = \frac{4.5 + \left(20.25 - \frac{212.43(1.04)}{12(18)^2}\right)^{\frac{1}{2}}}{2}$$

$$a_u = 4.5$$

$$A_s = \frac{1.04}{4.5(18)} = .01 \text{ in}^2 \quad \text{USE MIN STEEL}$$

$$SHEAR = 1.12(1.81)\tfrac{1}{2} + .12 - .8(1.27)\tfrac{1}{2} = .63^{k}$$

$$M = 1.12(1.81)\tfrac{1}{2}(2.48) + .12(1.54) - .8(1.27)\tfrac{1}{2}(.42) = 2.49^{'k}$$

$$a_u = 4.49 \qquad A_s = \frac{2.49}{4.49(18)} = .03 \text{ in}^2$$

USE MIN STEEL ✓

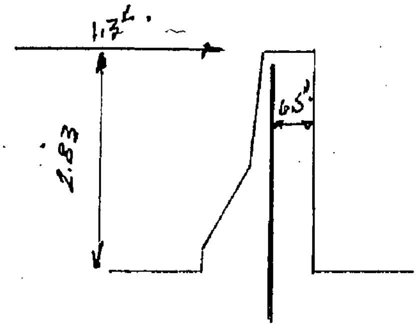
TVA 11030 (WM-7-75)

SHEET 20 OF

MODIFY DAMS — FT LOUDOUN & TELLICO

BARRIER (TRAFFIC)

COMPUTED SJRH DATE 27 JUL 87
CHECKED BP DATE Nov 4 '87



$SHEAR = 1.3^k (1.7) = 2.21^k$

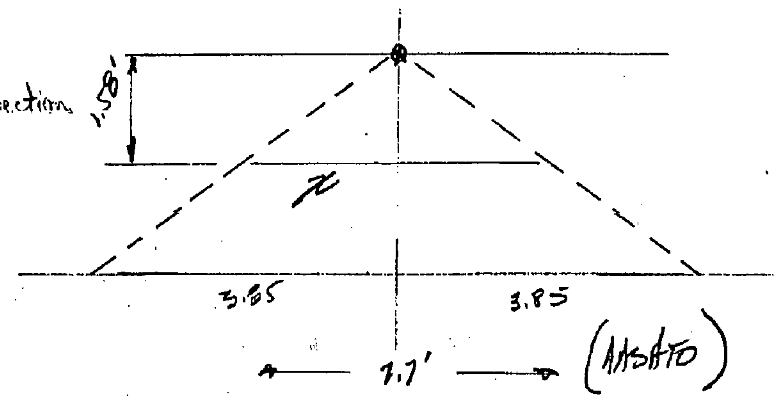
$M_u = 1.3(2.83)(1.7) = 6.25'^k$

$a_u = 4.35 \qquad A_s = \dfrac{6.25}{4.35(6.5)} = .22\ in^2$

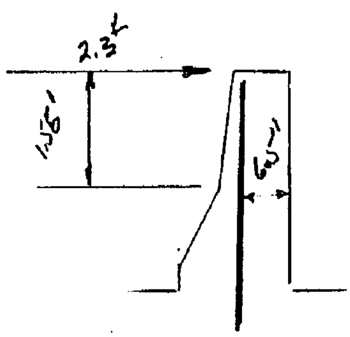
CHECK UPPER SECTION
ok by inspection

$x = \dfrac{1.58(3.85)}{2.83} = 2.15$

$\dfrac{10^k}{2.15(2)} = 2.3^k$



LOAD TRANSFER DIAGRAM



$M = 2.3(1.58)(1.7) = 6.18'^k$

$a_u = 4.35 \qquad A_s = \dfrac{6.18}{4.35(6.5)} = .22\ in^2$

#4 BARS @ 10"
OR EQUAL A_s

TVA 11030 (WM-7-75)

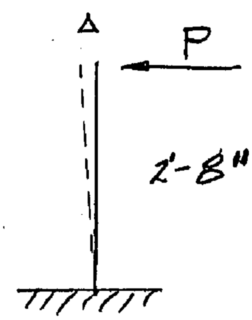
SHEET 21 OF

MODIFY DAMS — FT LOUDOUN & TELLICO

BARRIER

COMPUTED GLPH DATE 27 JUL 87
CHECKED BP DATE NOV 4 '87

DETERMINE THE AMOUNT OF DEFLECTION OF VERTICAL STEEL DURING THE EXTRUSION PROCESS. THIS DEFLECTION CAUSES CRACKS ALONG THE TOP OF THE WALL



\# 4 BAR

$$\sigma = \frac{M}{S} = \frac{P\ell}{S} \quad \therefore P = \frac{\sigma \cdot S}{\ell}$$

$$P = 60 \frac{\pi D^3}{(32)^2} = \frac{60\pi(.5)^3}{(32)^2} = .023^k$$

$$\Delta = \frac{.023(32)^3(64)}{3(29000)\pi(.5)^4} = 2.8''$$

\# 6 BAR

$$P = \frac{60\pi(.75)^3}{(32)^2} = .078^k$$

$$\Delta = \frac{.078(32)^3(64)}{3(29000)\pi(.75)^4} = 1.89''$$

ASSUME LONGITUDINAL REBARS TRANSFER LOAD AT LEAST OVER 10 BARS.

$$\Delta = \frac{.008(32)^3(64)}{3(29000)\pi(.75)^4} = .19''$$

CONSIDER INVERTED "4U" BARS POSITIONED IN THE LONG. DIRECTION & LONGITUDINAL STEEL TIED AS A SOLUTION

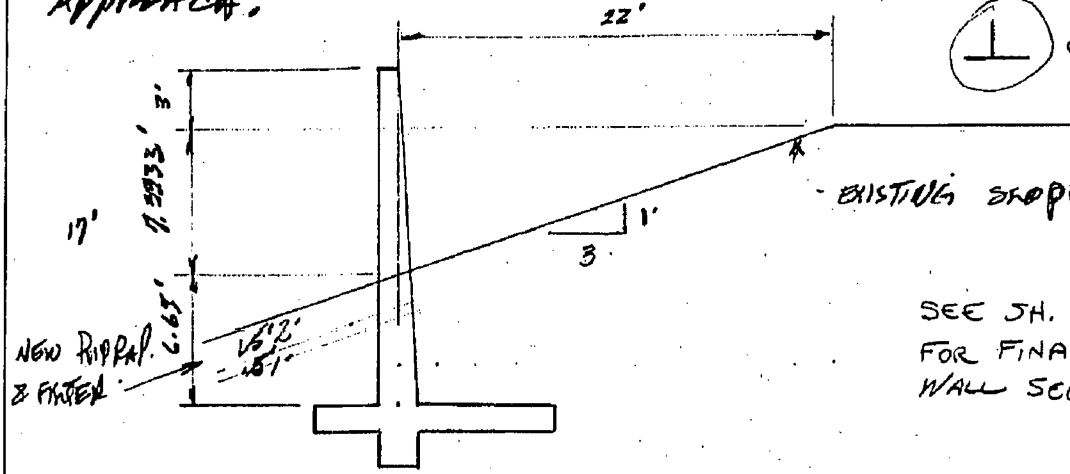
21

SHEET 22 OF

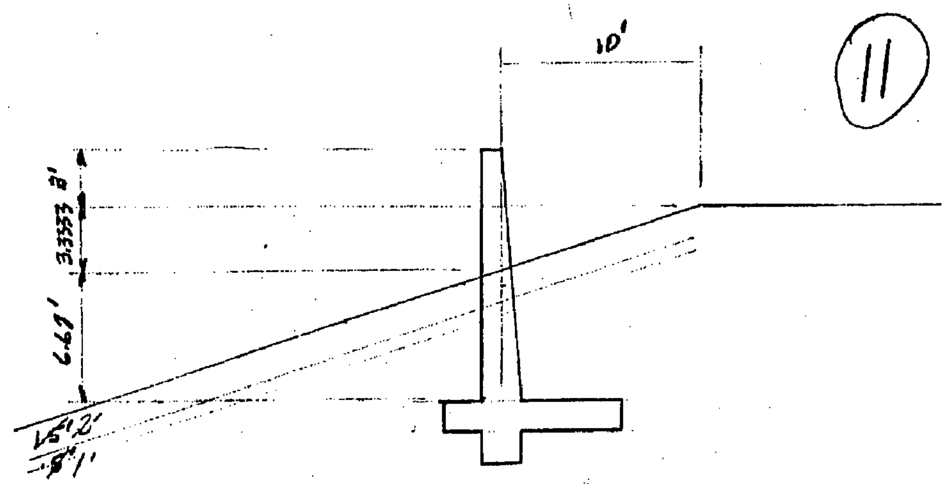
LOCK OPERATION BLDG — MELTON HILL & TELLICO
WIDEN ACCESS ROAD

COMPUTED GLPA DATE 25 NOV 87
REV CHECKED BP DATE JAN 22 88

DISTANCE BETWEEN S1 & S2 BRIDGE BENTS IS 160' — 6 SPACES @ 23' COULD BE PROVIDED PARALLEL TO ROAD BY *9' WIDE OR 16 SPACES @*10 x 20 COULD BE PROVIDED. CHECK SIZE OF RETAINING WALL FOR BOTH CASES AND DETERMIN MOST ECONOMICAL APPROACH.



SEE SH. 25 FOR FINAL WALL SECTION.



* MUST BE WIDER IF HANDICAPPED PARKING IS PROVIDED.

0

SHEET 22a OF

MODIFY DAM

FT LOUDOUN

COMPUTED SLRH DATE 1 DEC 87
CHECKED BP DATE July 29 '88

CALCULATE ALLOWABLE BEARING FOR SOIL IN SOUTH EMBANKMENT

$$q_o = c N_c \psi_c + \gamma_1 D N_q \psi_q + .5 \gamma_2 B N_\gamma \psi_\gamma$$

WHERE $\psi_c = \psi_q = \psi_\gamma = 1$ $\phi = 26°$

$N_c = 22.25$ $\gamma_1 = \gamma_2 = 120$
$N_q = 11.85$ $B = 10$ *
$N_\gamma = 12.54$ $D = 8$ *
$C = 0$

* VALUES MAY VARY USE F.S. OF 4

$q_o = 18900\ LB/FT^2$ $q = 4725\ LB/FT^2$ ok

$B = 5$
$D = 2$

$q_o = 6606\ LB/FT^2$ $q = 1652\ LB/FT^2$

LET 2500 LB/FT^2 CONTROL FOR MORE SHALLOW FOOTINGS.

NOTE C = 0, BUT IT VARIES BETWEEN 0 & 500 IN THE TEST HOLES. USING THE VALUE OF 0 IS VERY CONSERVATIVE OVERALL.

TVA 11030 (WM-7-75)

SHEET 23 OF

LOCK OPERATIONS BLDG
WIDEN ACCESS ROAD

FT LOUDOUN & TELLICO

COMPUTED GLPH DATE 30 NOV 87
REV BP DATE JAN 22 88

SEE SH. 25 FOR
FINAL WALL SECTION.

$K_a = .4$

$\gamma = .135$

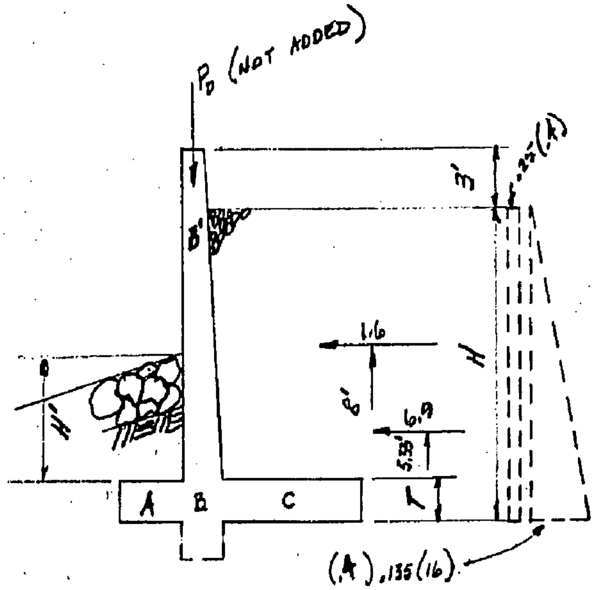
$H = 16$

$H' = 6$

$q = .25$

$c_g = .6$

$A = 3$
$B = 2$
$B' = 1.17$
$C = 9$
$T = 2$



* IGNORE & ADD KEY

0.4	
0.135	
16.	
6.	
0.25	
0.5	
3.	
2.	
1.17	
9.	
2.	
5.14027437	FS_{OT}
.8223684211	FS_{SL} *
1.045811343	e_U
4.451341076	q_{TU}
11.3764	V_{SU}
53.08986667	M_{SU}
14.04240568	M_{TU}
4.756634377	V_{HU}
3.86084837	M_{HU}

USE THESE FIGURES ONLY TO SET INITIAL GEOMETRY OF WALL

TVA 11030 (WM-7-75)

LOCK OPERATIONS BLDG
WIDEN ACCESS ROAD

FT LOUDOUN & TELLICO

COMPUTED GRH DATE 30 NOV 87
REV CHECKED EP DATE JAN 22 88

SEE SH. 25 FOR FINAL WALL SECTION.

W		D	M	LF	W_U	M_U
.135 (9)(14)	= 17.01	9.5	161.6	1.4	23.81	226.24
.135 (.83)(14)(.5)	= .78	4.72	3.68	1.4	1.09	5.15
.135 (3)(6)	= 2.43	1.5	3.65	1.4	3.40	5.11
.15 (1)(17)(.5)	= 1.28	4.33	5.54	1.4	5.54	7.76
.15 (1)(17)	= 2.55	3.5	8.93	1.4	8.93	12.50
.15 (2)(14)	= 4.2	7	29.4	1.4	5.88	41.16
q_{SUR} .25 (9.83)	= 2.46	9.09	22.36	1.7	4.18	38.01
	30.71		235.16		52.83	335.93

$M_{OT} = 6.9(5.33) + 1.6(8) = 49.58$

$FS_{OT} = \frac{235.16}{49.58} = 4.74 > 1.5$ OK

$FS_{SL} = \frac{(30.71)(.49)}{1.6 + 6.9} = 1.77 > 1.5$ OK

FS_{OT} IS VERY HIGH & FS_{SL} IS ACCEPTABLE CONSIDERING NO RESISTANCE FROM PASSIVE PRESSURE. IT MAY BE POSSIBLE TO CUT DOWN BASE AND ADD SHEAR KEY TO CONSERVE MATERIAL. —

$\bar{x} = \frac{235.16 - 49.58}{30.71} = 6.04$ $\quad e = \frac{14}{2} - 6.04 = .957$

$A = 14\ ft^2$, $I = \frac{14^3}{12} = 228.67\ ft^4$.

$P/A \pm Mc/I$

$\frac{30.71}{14} \pm \frac{30.71(.957)7}{228.67} = 2.19 \pm .90 = \begin{cases} 3.09\ k/ft^2 \\ 1.29\ " \end{cases}$ ok

TVA 11030 (WM-7-75)

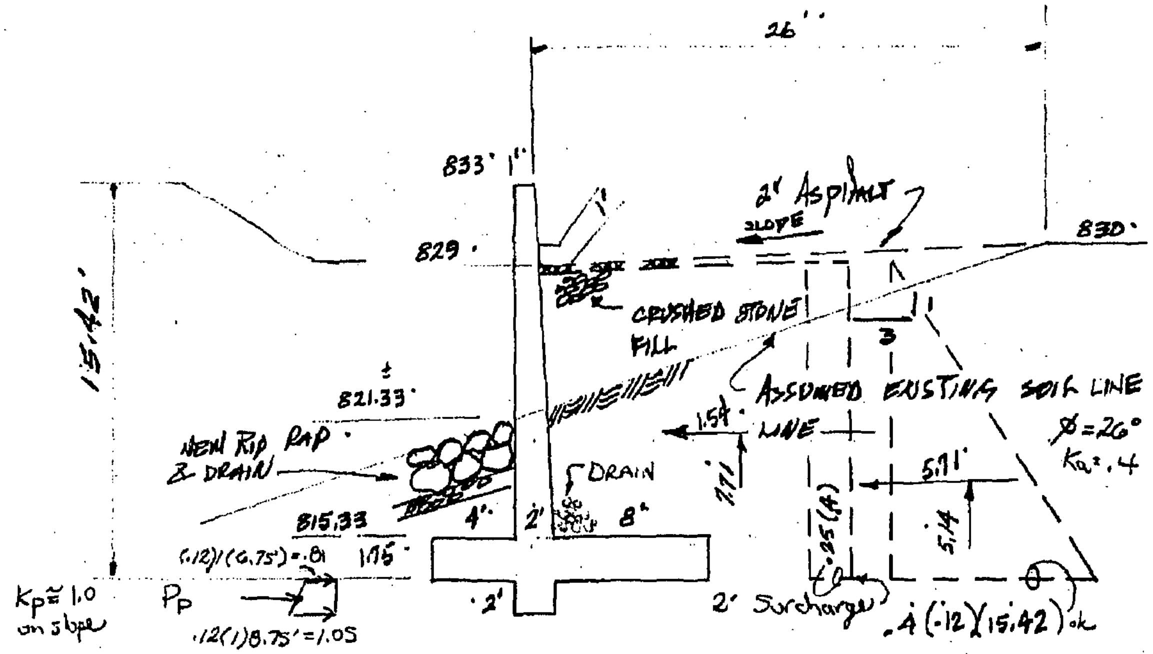
③

SHEET 25 OF

LOCK OPERATIONS BLDG
WIDEN ACCESS ROAD

FT LOUDOUN/TELLICO

COMPUTED GRPH DATE 22 DEC 87
CHECKED BP DATE JAN 15 88



W	D	M	LF	W_u	M_u
.12(8)(13.67) = 13.12	10	131.23	1.4	18.37	183.7
.12(.83)(13.67)(.5) = .68	~~6.85~~ 5.72	~~4.46~~ 3.9	1.4	.95	6.24
.12(5.5)(4) = 2.64	2	5.28	1.4	3.7	7.39
.15(17.67)(1) = 2.65	4.5	11.93	1.4	3.7	16.7
.15(17.67)(1)(.5) = 1.33	5.33	7.1	1.4	1.86	9.9
.15(1.75)(14) = 3.68	7	25.73	1.4	5.2	36.0
.25(8.83) = 2.2	9.59	21.1	1.7 (ok 1.4 Vert.)	3.74	35.9
26.3		206.83		37.52	295.83

$M_{OT} = 1.54(7.71) + 5.71(5.14) = 41.22$

$FS_{OT} = \frac{206.83}{41.22} = 5 > 1.5$ ok

$FS_{SL} = \frac{26.3(.49)}{1.54 + 5.71} = 1.78 > 1.5$ ok

w/ Passive resistance

$P_P = \frac{1}{2}(1.05 + .81)2 = 1.86^k$

Ignored surcharge vertical

$FS_{SL} = \frac{24(.4) + 1.86}{7.25} = 1.58 > 1.5$ ok

TVA 11030 (WM-7-75)

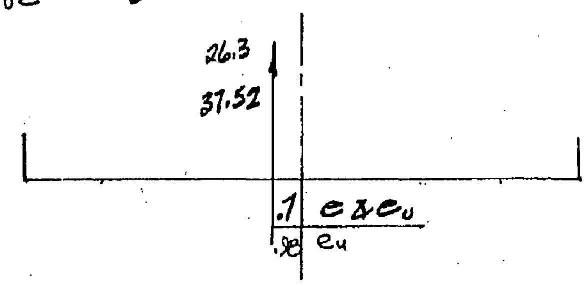
SHEET 26 OF

LOCK OPERATIONS BLDG — FT LOUDOUN/TELLICO
WIDEN ACCESS ROAD

COMPUTED GWH DATE 23 DEC 87
CHECKED BP DATE JAN 1987

$\bar{x} = \dfrac{206.83 - 41.22}{26.3} = 6.3 \qquad e = \frac{14}{2} - 6.3 = .7$

$\bar{x}_U = \dfrac{296.83 - \cancel{61.27}}{37.52} = \cancel{6.3}\ 6.02 \qquad e_U = \cancel{.7}\ .98$ $41.22(1.7) = 70.1$

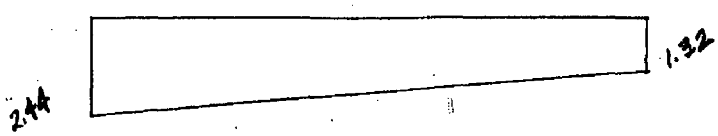


$A = 14\ FT^2$

$I = \dfrac{14^3}{12} = 228.67\ FT^4$

$\dfrac{P}{A} \pm \dfrac{Mc}{I}$

SERVICE



$\dfrac{26.3}{14} \pm \dfrac{26.3(.7)(7)}{228.67}$

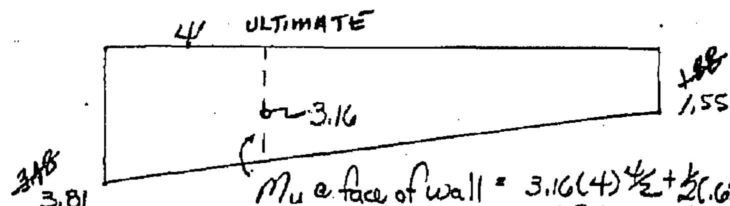
$1.88 \pm .56$

$2.44 < 2.5\ ^{k}/_{F}$ OK

ULTIMATE

$\dfrac{37.52}{14} \pm \dfrac{37.52(\cancel{.7}\ .98)(7)}{228.67}$

$2.68' \pm \cancel{.8}\ 1.13$



M_u @ face of wall $= 3.16(4)\frac{4}{2} + \frac{1}{2}(.65)\,4(\frac{8}{3}) = 28.8$

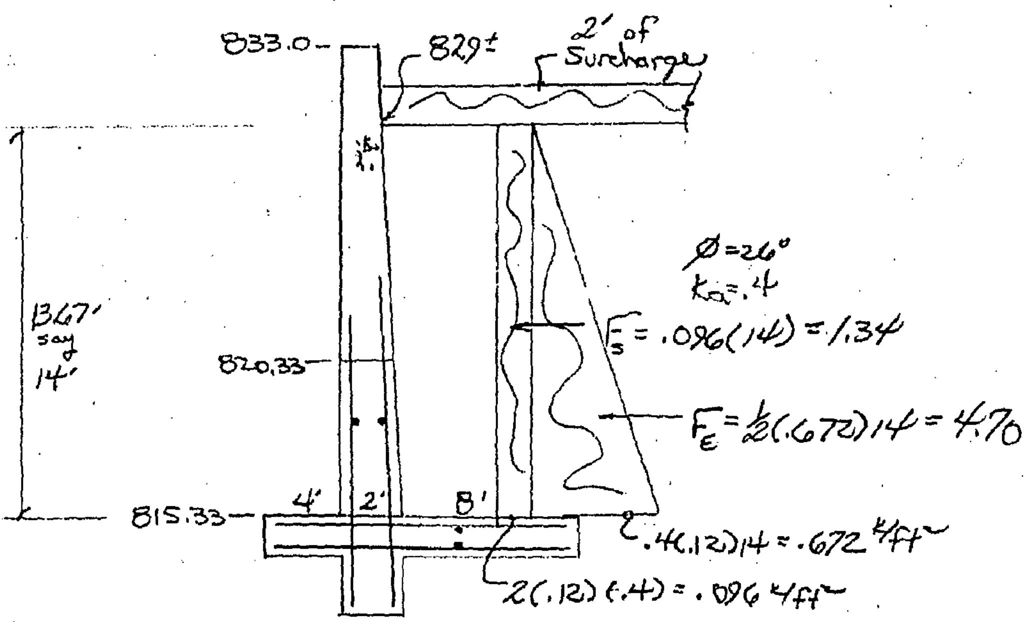
$V_u = 3.16(4) + \frac{1}{2}(.65)4 = 14^k$, $v_u = .07\ ksi$ ok $\qquad A_s \cong .4\ in^2/ft$

ADD SHEAR KEY FOR ADDITIONAL SAFETY FACTOR

$M_U^{STEM} = 53\ FT\text{-}K \qquad A_U = 4.36$ (4.33 IS MIN STEEL) $\qquad A_{ST} = .59\ in^2$ ~~#6@9~~ #8@12

$M_U^{SLAB} = 55\ FT\text{-}K \qquad A_U = 4.33 \qquad A_{ST} = .45\ in^2$ #6@12

SHEET 27 OF
LOCK OPERATIONS BLDG — FT LOUDOUN
PARKING AREA — PROJECT
COMPUTED BP DATE JAN 19 '88
CHECKED DATE



$V_u @ 815.33 = 1.7(1.34 + 4.7) = 10.3^k$ (6.04) $\quad d \cong 21''$

$v_u = 10.3 / .85(12)21 = .048\ ksi < 2\sqrt{f'_c}$ ok

$M_u @ 815.33 = 1.7(1.34 \times 14/2 + 4.7 \times 14/3) = 53.2^{'k}$

$A_s \cong 53.2(12) / .9(.9 \times 21)60 = .63\ in^2/ft$

Try #8@12"
$A_s = .79\ in^2/ft$

$p = .79 / 12(21) = .00313 > 200/f_y$ ok

$M_{u\,cap} \Rightarrow \quad a = .79(60) / .85(3)12 = 1.55''$

$M_{u\,cap} = .9(.79)60(21 - 1.55/2) / 12$
$= 71.9^{'k} > 53.2^{'k}$ ok

If no shear key provided, shear friction

$V_n = A_{vf} f_y \mu \Rightarrow A_v \geq 10.3^k / .85(60)1 = .20\ in^2/ft$ $\therefore$ $A_{s\,TOT} = (\frac{53.2}{71.9}).79 + .20 = .79\ in^2_{ok} = .79\ in^2$

TVA 11030 (WM-7-75)

SHEET 28 OF

LOCK OPERATIONS BLDG PARKING AREA — FT. LOUDOUN PROJECT

COMPUTED BP DATE JAN 19 '88

CHECKED DATE

A_s req'd @ EL. 820.33 , H = 8.67', say 9'

$M_u = 1.7(.4(.12)\overset{3.9}{2}(9)\frac{9}{2} + \frac{1}{2}(.4).\overset{5.83}{12}(9)^2(\frac{9}{3}))$
$= 16.5^{1k}$

$d \cong 17"$

$A_s \cong 16.5(12) / .9(.9 \times 17)60$

$= .24 \text{ in}^2/\text{ft}$ Use #6 @ 12"

Splice length = 33" × .8
= 26"
say 2'-3"

Temp & shrinkage Stl., ACI Code & TVA Design Std. DS-C.1.5.4

$\Delta T \Rightarrow V/S_{PLACING} = \frac{3}{4}(2')$
$= 1.5$ Fig. 1, T = 100°F

$V/S_{IN\text{-}SERVICE} = \frac{2}{3}(2') = 1.33$ Fig. 3
2' Against earth T = 35°F

ΔT = 65°F , L = 56' Fig. 6
$p = .0028$

$A_s = 460(.0028)12(2')(\frac{.42}{32.2})$
$= .4 \text{ in}^2$

#6 @ 12", ok

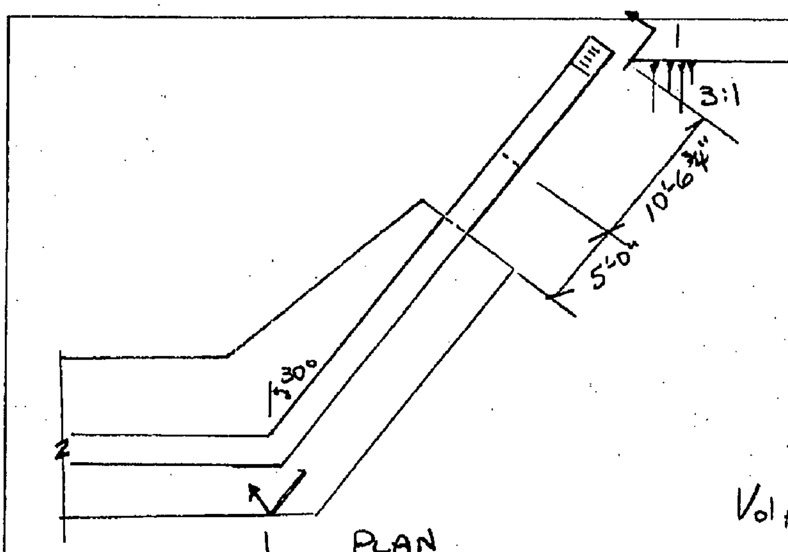
SHEET 29 OF

PARKING AREA

FT. LOUDOUN PROJECT

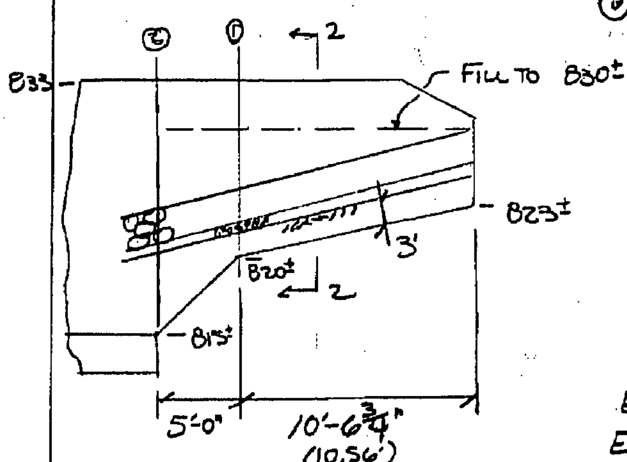
COMPUTED BP DATE JAN 22 '88

CHECKED DATE



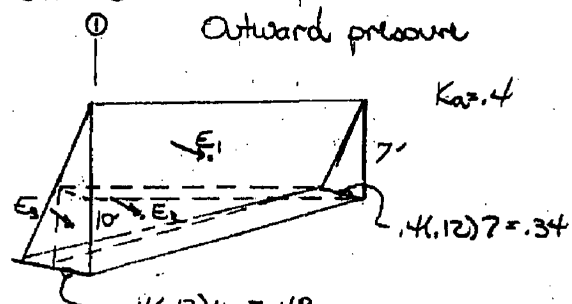
PLAN
NTS

$Vol_{PRISMATOID} = \frac{1}{6}h(B_1 + 4m + B_2)$



@ Section ①

Outward pressure



		x	M
$E_1 = .34(\frac{1}{2})10.56(7') = 12.6^k$		$10.56/2$	66.5
$E_2 = .34(3)(\frac{1}{2})10.56 = 5.4^k$		$10.56/3$	19
$E_3 = \frac{1}{6}(10.56)(\frac{1}{2}(.14) + 4(\frac{1}{2}).07)$			
$= .4^k$		$\sim 10.56/4$	1
18.4^k			$86.5^{'k}$

Depth of wall = 13'

$V_u/ft. = 1.7(18.4/13) = 2.41^k$

$v_u = 2.41/.85(12)"12$

$= .020$ ksi, ok

$M_u/ft. = 1.7(86.5/13) = 11.3^{'k}$

$A_s \cong 11.3(12)/.9(.9 \times 12)60 = .23$

#6@12", ok

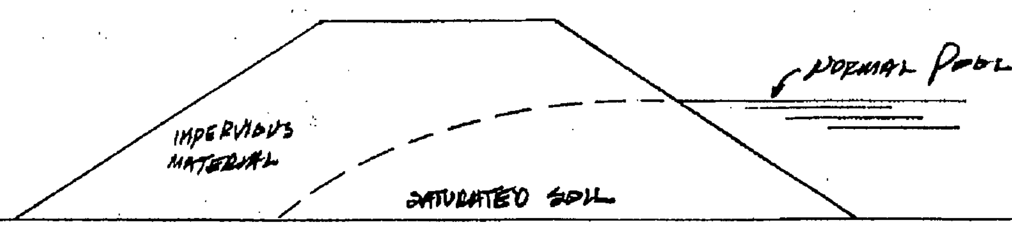
By inspection, if section ① is analyzed and reinforced then same reinf. for section ② would be ok especially since passive resistance was neglected.

TVA 11030 (WM-7-75)

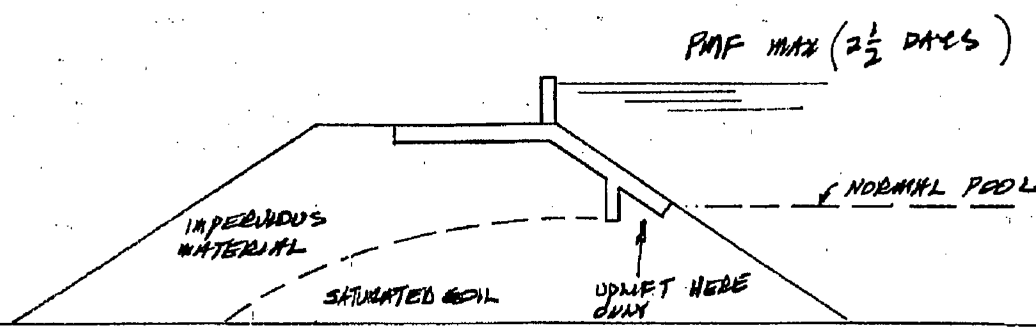
SHEET 30 OF

MODIFY DAMS — FT LOUDOUN Z TELLICO
LOCK OPERATIONS BLDG
BASIS FOR LOADING

COMPUTED GLEA DATE 15 OCT
CHECKED ____ DATE ____



SOUTH EMBANKMENT
(ADJACENT TO LOCK OPERATIONS BLDG)

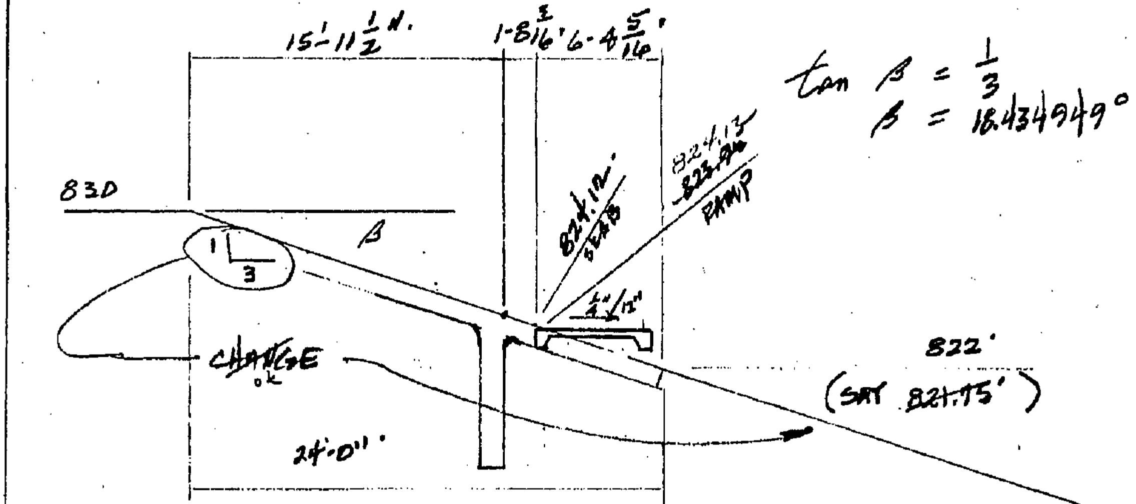


SOUTH EMBANKMENT WITH PART OF ARMOR SHOWN
(ADJACENT TO LOCK OPERATIONS BLDG)

SHEET 31 OF

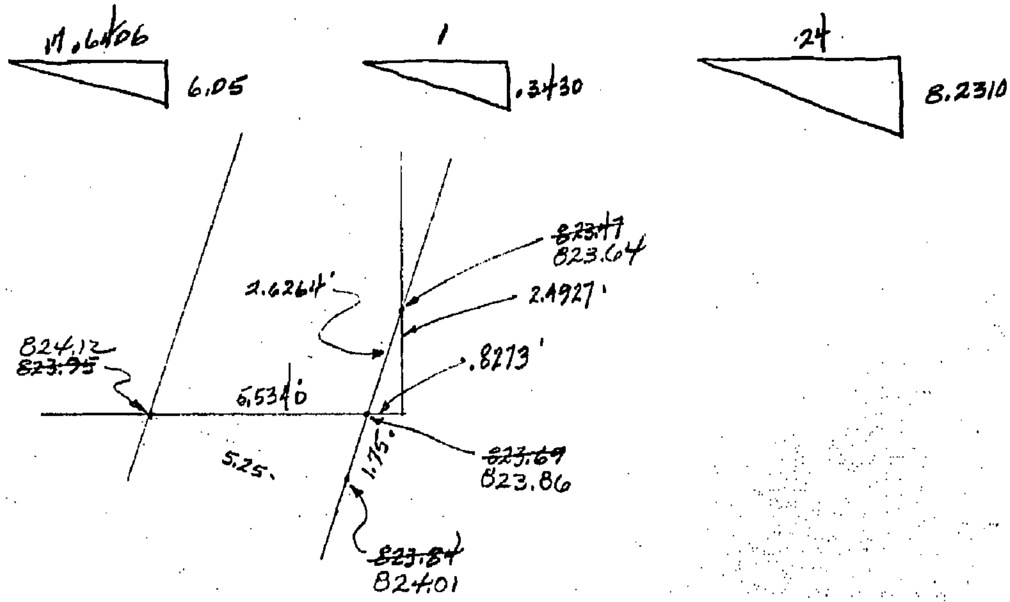
LOCK OPERATION BLDG — FT LOUDOUN & TELLICO
~~SLABS~~
HANDICAPPED ACCESS RAMP

COMPUTED GLPH DATE 9 DEC 87
CHECKED BP DATE DEC 21 '87



$\tan\beta = \frac{1}{3}$
$\beta = 18.434949°$

CHANGE SLOPE OF ARMOR SLAB TO MAKE INTERSECTION OF RAMP AND SLAB THE SAME.



TVA 11030 (WM-7-75)

SHEET 32 OF

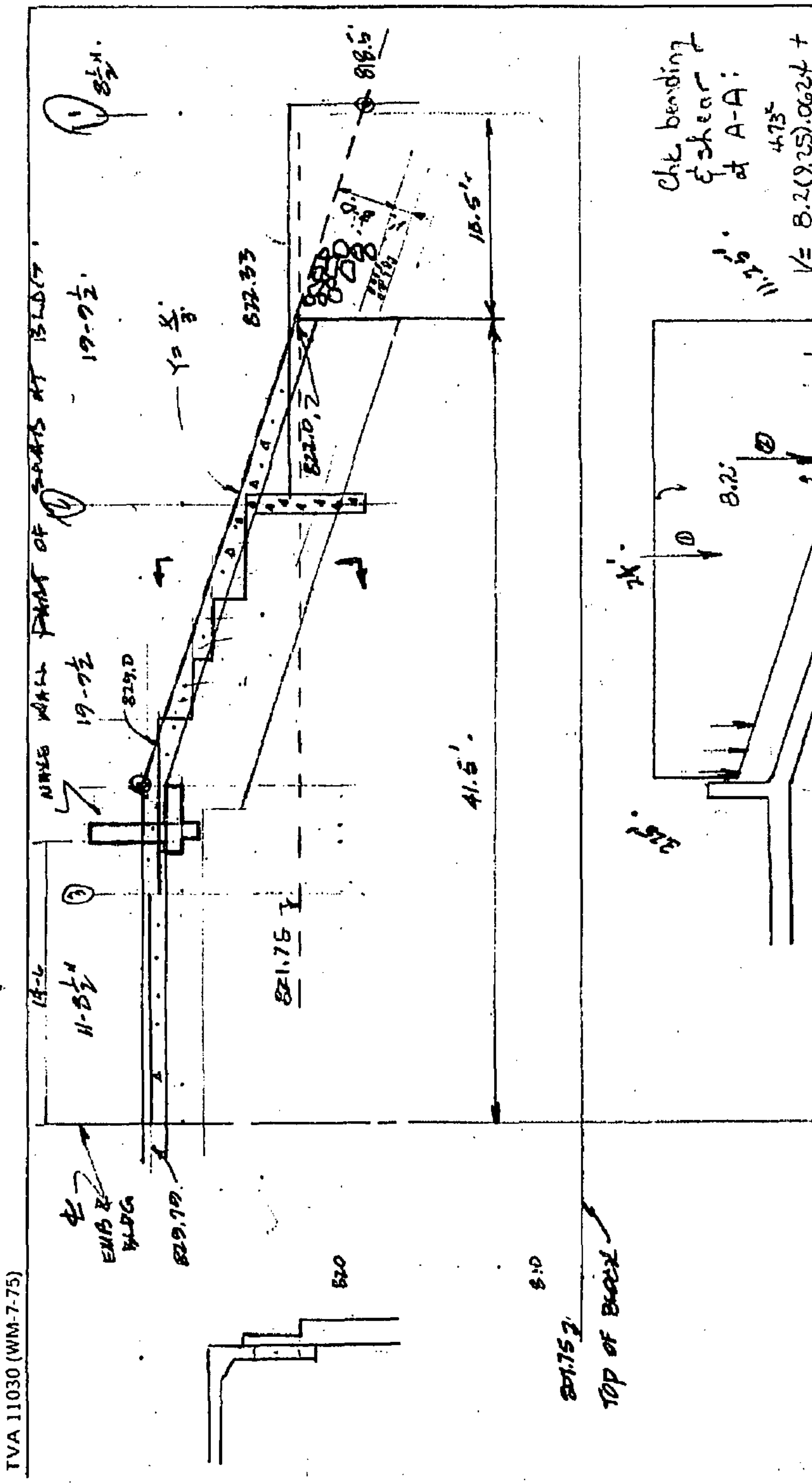
MODIFY DAMS — FT LOUDOUN & TELLICO

LOCK OPERATIONS BLDG

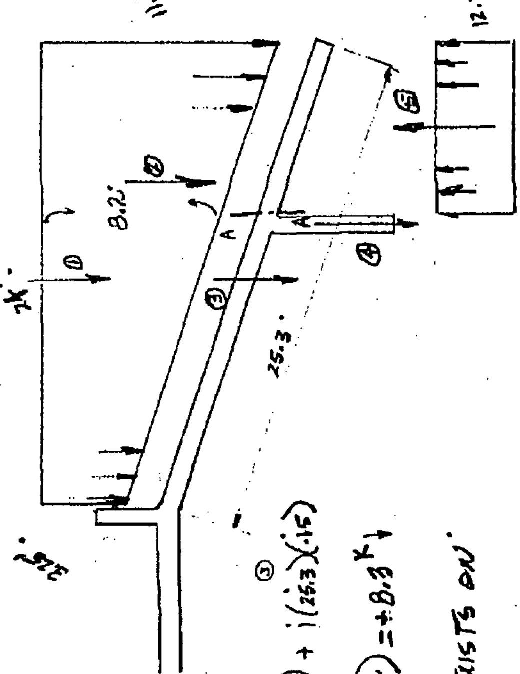
SOUTH WALL

COMPUTED BY: [illegible] DATE: 9 OCT 87

CHECKED: BP DATE: Dec. 21 '87



Chk bending & shear at A-A:



$V = 8.2(9.25)(.0624) + \frac{1}{2}(3.08)(9.25)(.0624) + 9.25(.15) - 9.25(12.25)(.0624) = .03^k$ ∴ shear ok

$M = 4.73(9.25/2) + .9(10.5/3) + 1.4(9.25/2) - 12.25(.0624) \cdot 7 \cdot 9.25(9.25/2) = 1.5^{1k}$ ok

$\overset{+\uparrow}{\Sigma F_y} = 3.25(21)(.0624) + 8(24/2)(.0624) + 1(25.3)(.15)$

$+ 5(1)(.15) - 9.25(12.25)(.0624) = +8.3^k \uparrow$

THEREFORE NO NET UPLIFT EXISTS ON

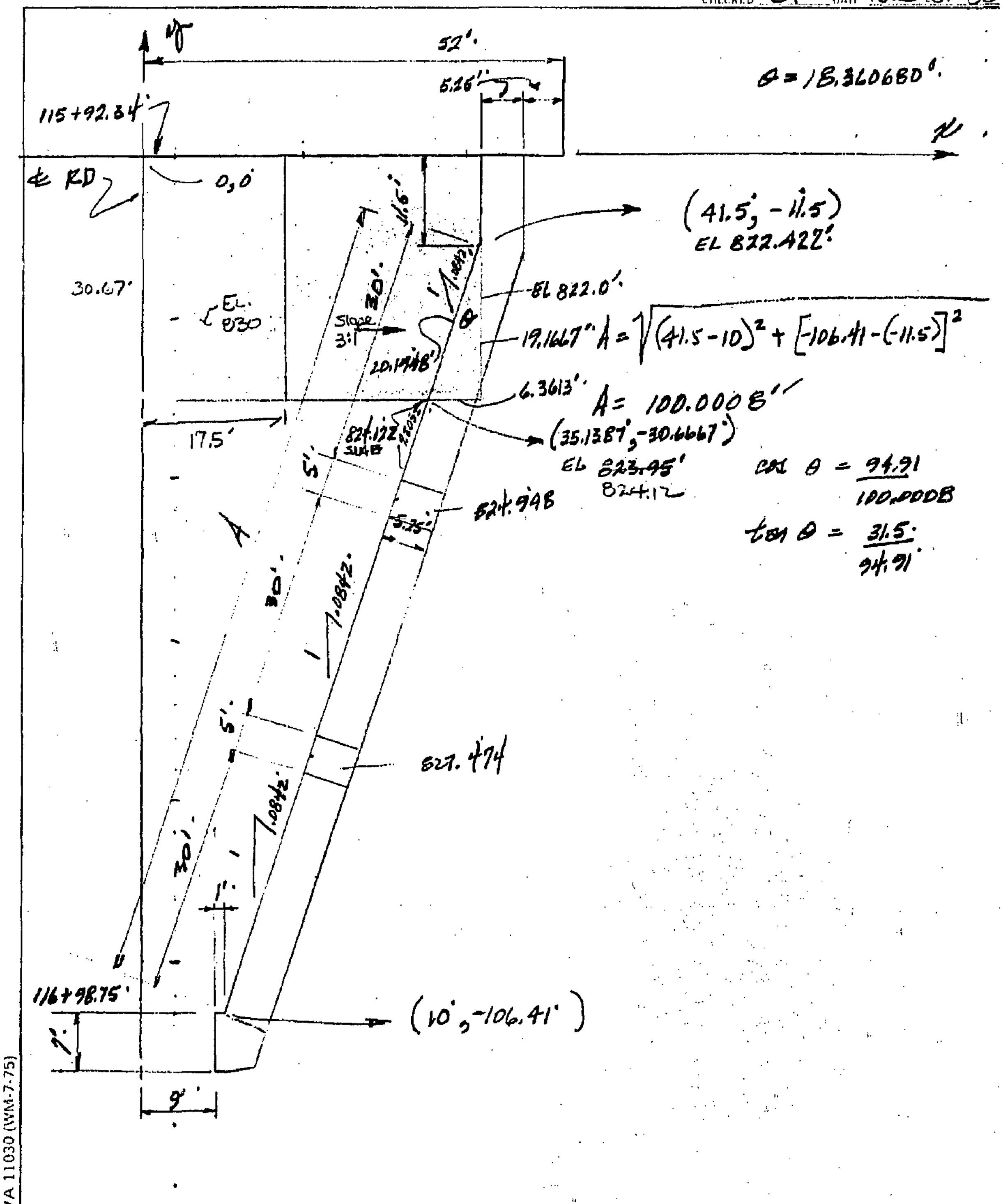
SLAB

SHEET 33 OF

LOCK OPERATION BLDG
SLABS
HANDICAPPED ACCESS RAMP

FT LOUDOUN & TELLICO

COMPUTED GLPH DATE 8 DEC 87
CHECKED BP DATE Dec 21 88



SHEET 34 OF

MODIFY DAMS — FT LOUDOUN & TELLICO
LOCK OPERATIONS BLDG
SOUTH WALL - SLABS

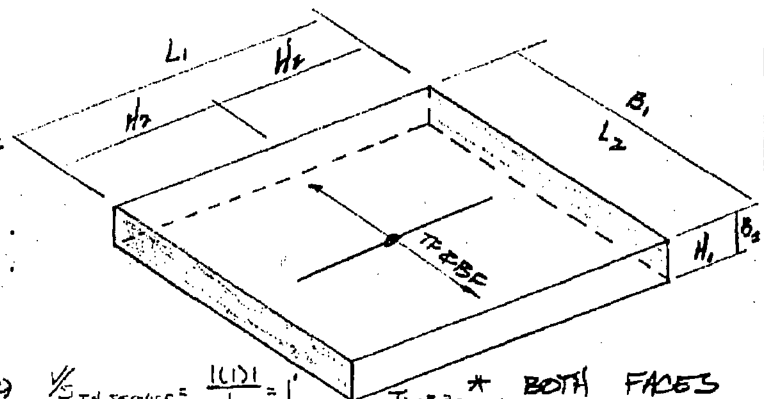
COMPUTED GWH DATE 15 OCT 87
CHECKED BP DATE DEC 21 '87

TES CIVIL DESIGN STANDARD DS-C1.5.4

CASE III.

$W_c = \cancel{.009}$ .013
$K_R = 1$
$T_E = \cancel{75}$ see below
$C_T = .000005$
$f_t' = .004\sqrt{f_c}$
$E_c = 57\sqrt{f_c}$

d = 3" clr. c = 4.5" to ℄
g = 12"
N = 50 *
$L_1 = 26$
$B_1 = 25$
$H_1 = 1$



$T_E \Rightarrow$ 1/S SPACING = 3/3(12), T = 90° ; 1/S IN SERVICE = 1(1)1/1 = 1', T IN SERV = 32° ; TDS = 30-24 = 6 * BOTH FACES ∴ T_E = 64°

$$L' = \frac{.65\, W_c}{12\left(K_R T_E C_T - \frac{f_t}{E_c}\right)} = \cancel{.889}\ 2.82 < \frac{L}{2}$$

$$A_b = \frac{f_t}{f_s}\ \frac{BH'}{N'}\ 144 = \cancel{1.02}\ .66\ in^2$$

f_s = 24 ksi

A_{min} = .0018 bh
= .0018(12)12
= .26 in²/ft < A_b ok.

PUT BARS IN BOTH FACES ~~1.02/2 = .51 in² @ #6 @ 9~~

THIS REINF IS PARALLEL TO AXIS OF EMBANKMENT

USE # 6 @ 12 ⊥ TO THESE BARS.

Used #6@12" BOTH FACES & BOTH DIRECTIONS. NOT A PURELY FIXED DIRECTION DUE TO MOVEMENT REQD FOR PASSIVE RESISTANCE.

Slab on Non-Bonding Subgrade, Case IV

$A_b = \frac{1.5(.15)\, l_e B}{24(N)} = .88$ $\quad l_e = \frac{24(.88)}{1.5(.15)} = 94'$

∴ #6 @12" ok

MODIFY DAMS
LOCK OPERATION BLDG
EAST WALL

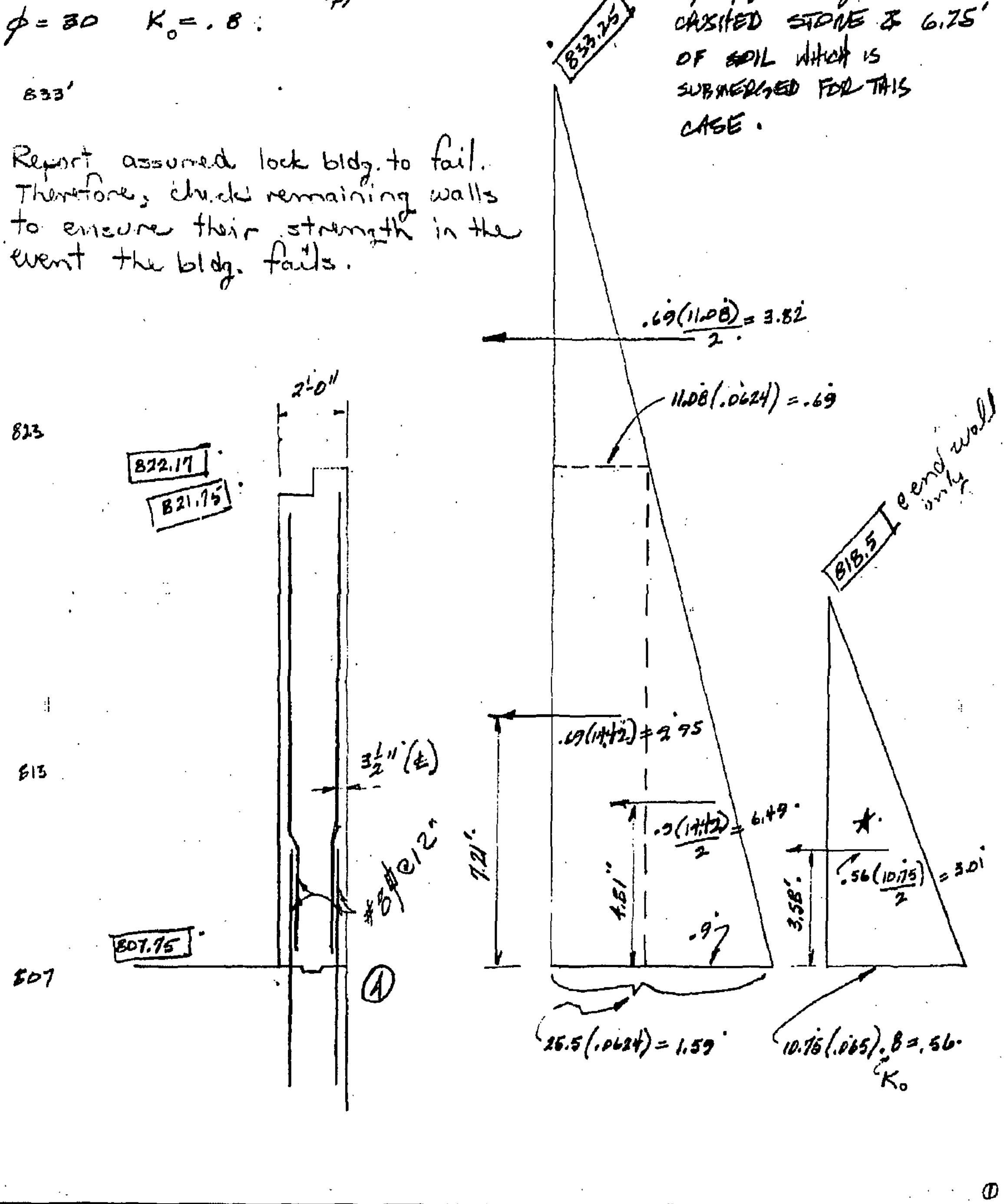
35
FT LOUDOUN & TELLICO

COMPUTED GLRH DATE 28 SEP 87
CHECKED BP DATE JAN 23 88

WATER — 62.4 LB/FT³
★ SUBMERGED — 65 LB/FT³
$\phi = 30 \quad K_0 = .8$

★ LOAD CONSISTS OF 3' RIPRAP, 1' COMPACTED CRUSHED STONE & 6.75' OF SOIL WHICH IS SUBMERGED FOR THIS CASE.

Report assumed lock bldg. to fail. Therefore, check remaining walls to ensure their strength in the event the bldg. fails.



SHEET 36 OF

MODIFY DAMS
LOCK OPERATIONS BLDG
EAST WALL

FT. LOUDOUN & TELLICO

COMPUTED GLDH DATE 6 OCT 87
CHECKED BP DATE JAN 23 88

$d = 24'' - 3\frac{1}{2}'' = 20\frac{1}{2}''$. Assumes wall above to already have failed.

$M_{u @ (A)} = 9.95(7.21)(1.7) + 6.49(4.81)(1.7) + 3.01(3.58)(1.4) = 190.1'^{K}$

PMF Case, LF = 1.0

$V_u = 9.95(1.7) + 6.49(1.7) + 3.01(1.4) = 32.2^{K}$.

$$\frac{M_u}{\phi b d^2 F'_c} = \frac{190.1(12)}{.9(12)(20.5)^2(3)} = .168$$

$A_s \doteq \frac{190.1(12)}{.9(.9\times 20.5)\,40} \doteq 3.43\ in^2$

$.168 = \omega(1 - 0.59\omega)$

$.168 = \omega - 0.59\omega^2$

$0.59\omega^2 - \omega + .168 = 0 \implies \frac{-b \pm \sqrt{b^2 - 4AC}}{2A}$

$$\frac{-(-1) \pm \sqrt{(-1)^2 - 4(.59)(.168)}}{2(.59)}$$

$\frac{1 \pm .777}{1.18} \implies 1.51$ or $.19$

$\rho = \frac{.19(3)}{40} = .0143$

$\rho_{MIN} = \begin{cases} \frac{200}{40000} = .005 < \rho_{REQD} \\ \frac{4}{3}\rho \end{cases}$

$A_s = \rho b d = .0143(12)(20.5) = 3.52\ in^2$.

TVA 11030 (WM-7-75)

SHEET 37 OF

MODIFY DAMS
LOCK OPERATIONS BLDG
EAST WALL

FT LOUDOUN & TELLICO

COMPUTED GRA DATE 7 OCT 87
CHECKED BP DATE JAN 23 88

SINCE THE REQUIREMENT OF 3.52 in^2 OF REINFORCEMENT EXCEEDS THE EXISTING 1 in^2 THE ANALYSIS OF THE WALL AS A CANTILEVER APPEARS TO BE WAY TOO CONSERVATIVE. CHANGE THE APPROACH TO A SLAB FIXED ON THREE SIDES AND FREE AT THE TOP. REFER TO THE TECHNICAL PUBLICATION "MOMENTS & REACTIONS FOR RECTANGULAR PLATES" BY W.T. MOODY, BUREAU OF RECLAMATION.

THE SAME LOADING INFORMATION WILL APPLY.

TVA 11030 (WM-7-75)

SHEET 38 OF

MODIFY DAMS — FT. LOUDOUN & TELLICO
LOCK OPERATION BLDG
EAST WALL

COMPUTED GLEN DATE 7 OCT 87
CHECKED BP DATE JAN 23 88

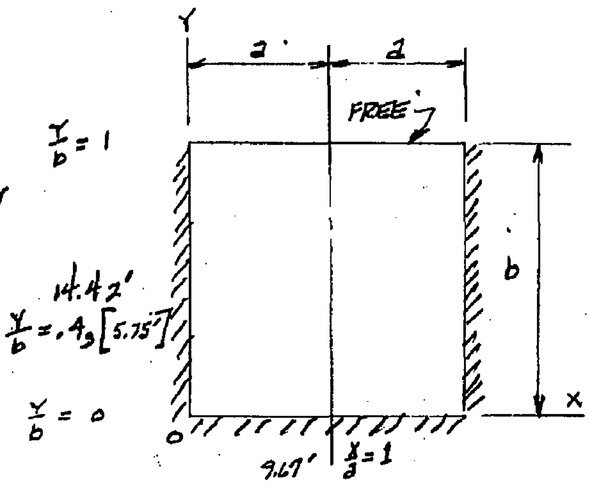
FIGURE 1.

$\frac{a}{b} = \frac{4.835}{14.42} = .335'$

USE $\frac{a}{b} = .375 = \frac{3}{8}$ ON MONOGRAPH

$M = c\,p\,b^2$
$R = c\,p\,b$

$(P = .69)$



$\left.\begin{matrix}\frac{Y}{b}=0 \\ \frac{X}{a}=0\end{matrix}\right\}$ $M_x = 0$, $R_x = -.0015$ | $M_y = 0$, $R_y = -.0015 \Rightarrow R = .0015\,p\,b \Rightarrow R_x = -.01$ | $M_x = 0$ | $M_y = 0$, $R_y = -.01$

$\left.\begin{matrix}\frac{Y}{b}=0 \\ \frac{X}{a}=1\end{matrix}\right\}$ $M_x = +.0058$, $R_x = -.0015$ | $M_y = +.0288$, $R_y = +.3410$ | $M_x = .83$, $R_x = -.01$ | $M_y = 4.13$, $R_y = 3.39$

$\left.\begin{matrix}\frac{Y}{b}=.4 \\ \frac{X}{a}=0\end{matrix}\right\}$ $M_x = +.0379$, $R_x = +.3541$ | $M_y = +.0076$, $R_y = -.0015$ | $M_x = 5.44$, $R_x = 3.~~55~~$ | $M_y = 1.09$, $R_y = -.01$

$\left.\begin{matrix}\frac{Y}{b}=.4 \\ \frac{X}{a}=1\end{matrix}\right\}$ $M_x = -.0186$, $R_x = +.3541$ | $M_y = -.0082$, $R_y = +.3410$ | $M_x = -2.67$, $R_x = 3.~~55~~$ | $M_y = 1.18$, $R_y = 3.39$

$\left.\begin{matrix}\frac{Y}{b}=1 \\ \frac{X}{a}=0\end{matrix}\right\}$ $M_x = +.0476$, $R_x = +.3711$ | $M_y = 0$, $R_y = -.0015$ | $M_x = 6.83$, $R_x = 3.69$ | $M_y = 0$, $R_y = -.01$

$\left.\begin{matrix}\frac{Y}{b}=1 \\ \frac{X}{a}=1\end{matrix}\right\}$ $M_x = -.0247$, $R_x = +.3711$ | $M_y = 0$, $R_y = +.3410$ | $M_x = -3.54$, $R_x = 3.69$ | $M_y = 0$, $R_y = 3.39$

TVA 11030 (WM-7-75)

SHEET 39 OF

MODIFY DAMS — FT LOUDOUN & TELLICO
LOCK OPERATION BLDG
EAST WALL

COMPUTED GLBH DATE 7 OCT 87
CHECKED BP DATE JAN 23 88

FIGURE #4 $(P = .9)$

$\frac{y}{b} = 0$, $\frac{x}{a} = 1$: $M_x = +.0040$, $M_y = +.0200$, $R_x = +.0102$, $R_y = +.2645$; $M_x = .75$, $R_x = .13$; $M_y = 3.74$, $R_y = 3.43$

$\frac{y}{b} = .4$, $\frac{x}{a} = 0$: $M_x = +.0208$, $M_y = +.0042$, $R_x = +.2107$, $R_y = +.0102$; $M_x = 3.89$, $R_x = 2.73$; $M_y = .79$, $R_y = .13$

$\frac{y}{b} = 1$, $\frac{x}{a} = 0$: $M_x = +.0066$, $M_y = 0$, $R_x = +.0189$, $R_y = +.0102$; $M_x = 1.24$, $R_x = .25$; $M_y = 0$, $R_y = .13$

THE REMAINING LOAD IS ONLY ON 75% OF THE WALL, HOWEVER, 67% LOAD CASE IS NOT CONSIDERED GOOD ENOUGH. THEREFORE USE FIGURE #4 AGAIN FOR LOAD ON ALL OF WALL. COEFFICIENTS ARE THE SAME. $(P = .56)$

$\frac{y}{b} = 0$, $\frac{x}{a} = 1$: $M_x = .47$, $R_x = .08$; $M_y = 2.33$, $R_y = 2.13$

$\frac{y}{b} = .4$, $\frac{x}{a} = 0$: $M_x = 2.42$, $R_x = 1.20$; $M_y = .49$, $R_y = .08$

$\frac{y}{b} = 1$, $\frac{x}{a} = 0$: $M_x = .77$, $R_x = .16$; $M_y = 0$, $R_y = .08$

SUM MAXIMUMS FOR DESIGN MOMENT & SHEAR.

TVA 11030 (WM-7-75)

(5)

SHEET 40 OF

MODIFY DAMS
LOCK OPERATION BLDG
EAST WALL

FT LOUDOUN & TELLICO

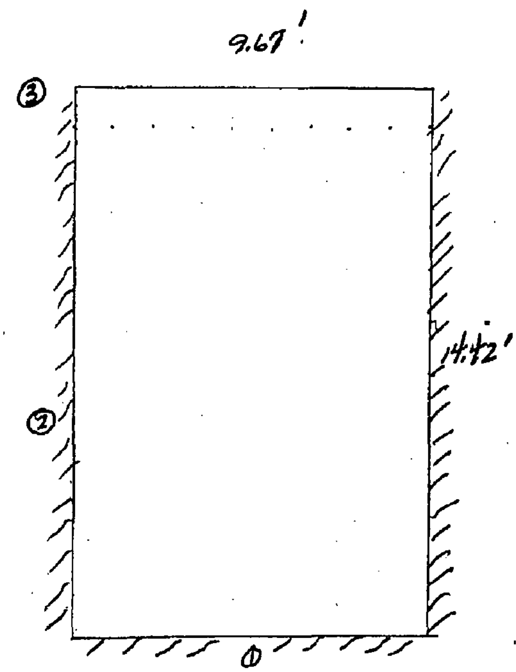
COMPUTED GLDA DATE 7 OCT 87
CHECKED BP DATE JAN 23 88

NO LOAD FACTORS INCLUDED

$M_{①} = 4.13 + 3.74 + 2.33 = 10.2$
$V_{①} = 3.39 + 3.43 + 2.13 = 8.95$

$M_{②} = 5.44 + 3.89 + 2.42 = 11.75$
$V_{②} = 3.55 + 2.73 + 1.70 = 7.98$

$M_{③} = 6.83 + 1.24 + .77 = 8.84$
$V_{③} = 3.69 + .25 + .16 = 4.1$



$$\frac{M_{①}}{\phi b d^2 f'_c} = \frac{10.2(12)}{.9(12)(20.5)^2 3} = .009$$

$$A_s \cong \frac{10.2(12)}{.9(.9 \times 20.5)40} \cong .18 \text{ in}^2$$

$.59\omega^2 - \omega + .009 = 0$

$\omega = .009$

$$\rho = \frac{.009(3)}{40} = .0007$$

$$\rho_{MIN} = \begin{cases} \frac{200}{40000} = .005 \\ \frac{4}{3}(.0007) = .0009 \end{cases}$$

$A_s = 1 \text{ in}^2 \Longrightarrow$ #8 ϕ BAR

$A_s = \rho b d$

$$\rho = \frac{A_s}{bd} = \frac{1}{12(20.5)} = .0041$$

$\rho_{ACTUAL} = .0041 > .0009$

OK

TVA 11030 (WM-7-75)

⑥

SHEET 41 OF

MODIFY DAMS
LOCK OPERATION BLDG
EAST WALL

FT LOUDOUN & TELLICO

COMPUTED GLPA DATE 7 OCT 87
CHECKED BP DATE JAN 23 88

$$\frac{M_{②}}{\phi b d^2 f'_c} = \frac{11.25\,(12)}{.9(12)(20.5)^2\,3} = .01$$

$A_s = \frac{11.75(12)}{.9(.9 \times 20.5)40} = .21 \text{ in}^2/\text{ft}$

$.59w^2 - w + .01 = 0$

$w = .01$

$\rho = \frac{.01(3)}{40} = .0008$

$$\rho_{min} = \begin{cases} \frac{200}{40000} = .005 \\ \frac{4}{3}(.0008) = .001 \end{cases}$$

$\rho_{ACTUAL} = .002 > .001$ ok

$A_s = .44 \Rightarrow \#6\,\phi$ BAR

$A_s = bd\rho$

$\rho = \frac{A_s}{bd} = \frac{.44}{12(20.5)} = .002$

TVA 11030 (WM-7-75)

0

SHEET 42 OF
MODIFY DAMS — FT LOUDOUN & TELLICO
LOCK OPERATIONS BLDG
EAST WALL
COMPUTED SLPH DATE 8 OCT 87
CHECKED BP DATE JAN 23 88

FIGURE 9 ($F = 3.82'$)

$\frac{a}{b} = .335$ SAY $3/8$

Assume wall above El. 822.17 remains in place. Therefore, wall will transfer reaction but no moment due to small fixity.

$\left.\begin{array}{l}\frac{y}{b} = 0 \\ \frac{x}{a} = 1\end{array}\right\}$ $M_x = 0'$ $M_y = -.0002'$ $R_x = -.0006'$ $R_y = -.0091'$ $M_x = 0$ $R_x = -.002$ $M_y = -.011$ $R_y = -.03'$

$\left.\begin{array}{l}\frac{y}{b} = .4 \\ \frac{x}{a} = 0\end{array}\right\}$ $M_x = +.0020'$ $M_y = +.0004'$ $R_x = -.0433'$ $R_y = -.0006'$ $M_x = .11'$ $R_x = -.17'$ $M_y = .02$ $R_y = -.002$

$\left.\begin{array}{l}\frac{y}{b} = 1 \\ \frac{x}{a} = 0\end{array}\right\}$ $M_x = +.2723'$ $M_y = 0'$ $R_x = +3.3048'$ $R_y = -0.0006$ $M_x = +14.99'$ $R_x = +\cancel{41.66}$ $+12.6$ $M_y = 0$ $R_y = 0$

NO LOAD FACTORS INCLUDED

$M_{①} = 10.2' + (-.11') = 10.09'$
$V_{①} = 8.95' + (-.03) = 8.92'$

$M_{②} = 11.75' + (.11) = 11.86'$
$V_{②} = 7.98' + (-.17) = 7.81'$

$M_{③} = 8.84' + (14.99) = 23.83'$
$V_{③} = 4.1' + (\cancel{41.66}\ 12.6) = \cancel{51.76}\ 16.7^{k}$

TVA 11030 (WM-7-75)

SHEET 43 OF

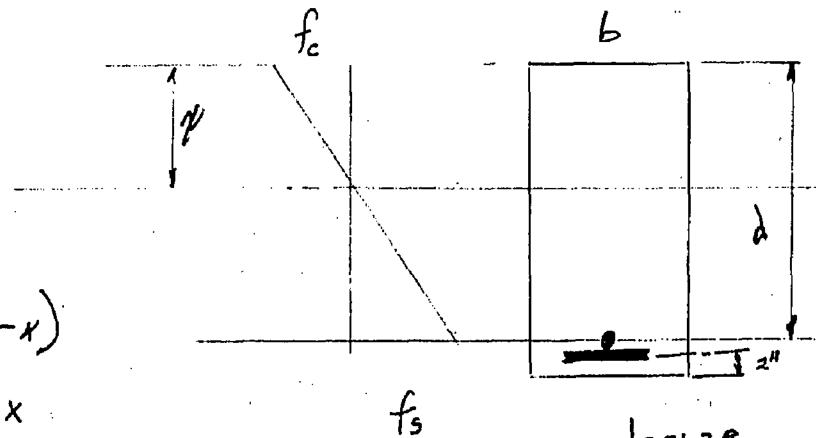
MODIFY DAMS — FT LOUDOUN/TELLICO
LOCK OPERATIONS BLDG
EAST WALL

COMPUTED GLRH DATE 8 OCT 87
CHECKED BP DATE JAN 23 88

$M = 23.83'$ $f'_c = 3000'$ GRADE 40 STEEL $n = 9$

$f_{c_{All}} = 1350$ $f_s = 20\,000 \Longleftrightarrow f_c = 3000$ $f_s = 40000$

$V = \cancel{51.46}$ 16.7



$d = 21.28$
$d = 24 - (2) - (\frac{1}{2}) - (.22)$

#6@12", A_s=.44

$(12x)\frac{x}{2} = 3.96(21.28 - x)$

$6x^2 = 84.27 - 3.96x$

$6x^2 + 3.96x - 84.27 = 0$

$x = 3.43$

$I = \dfrac{12(3.43)^3}{3} + 3.96(21.28 - 3.43)^2 = 1423.2$

$f_c = \dfrac{M_c}{I} = \dfrac{(23.83)12(3.43)}{1423.2} = .69\ ^k/_{"}$ < 1.35 ksi, ok

$f_s = \dfrac{M_c}{I} = \dfrac{(23.83)12(17.85)}{1423.2} = 3.59\ (9) = 32.31\ ^k/_{"}$ ok

$f_v = \dfrac{\cancel{51760}\ 16{,}700}{12\ (21.28)} = \cancel{202.7}\ 65.3$ psi $\lessgtr 2\sqrt{f'_c} = 109.5$ ok

$f_{V_{ed}} = \dfrac{32776}{12(21.28)} = 128.35$ psi

IF CONC STRENGTH HAS INCREASED AT LEAST TO 4118 psi THEN $2\sqrt{f'_c} = 128.35$
OK

The previous check was made for the east wall only! However, the southern-upstream walls would be exposed to the same type of loadings and these walls span horizontally greater distances. The previous "a" distance was 4.835', and these other walls would have a "a" distance of approx. $17/2 = 8.5'$.

Refer to Fig. 1 for $a/b = .5$ & $y/B = 1$, $x/A = 0$

Coeff. $M_x = +.0852$

∴ Loads approx. double in magnitude

from sh. 43, if loadings were doubled

$M = 23.8(2) = 47.6^k$

$R = 16.7(2) = 33.4^k$

Also, from Fig. 1 the moment coeff's @ $y/b = 1.0$ to $y/b = 0$ drop from $-.0432$ to $+.0107$. Therefore, even though the top part of the wall in the long direction may be overstressed the lower portions would help prevent failure. Assume 70% of this moment would need to be resisted by the top portion of this wall.

$M = .70(47.6) = 33^{\prime k}$

$a = .44(40)/.85(3)12 = .6"$

M_u #6@12" $= .44(40)(21.28 - .6/2)/12$

$= 30.8^{\prime k}$

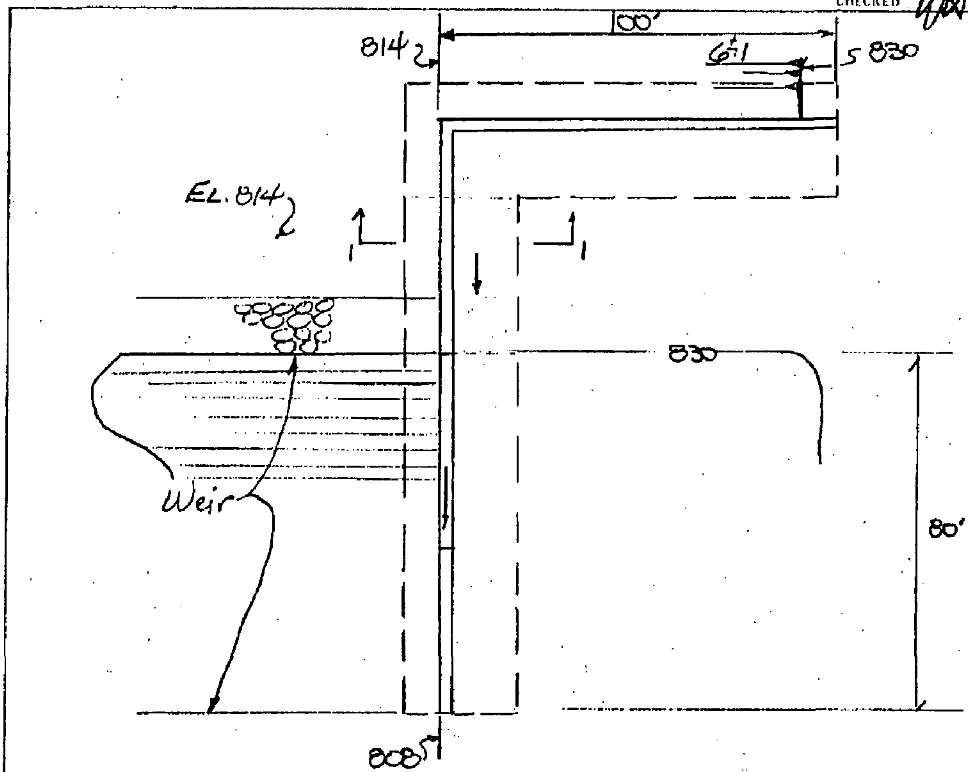
∴ Existing reinf. is very close to failure at this point. The upstream slab will be tied into the upper portion of this wall to provide strength in the event of a PMF flood.

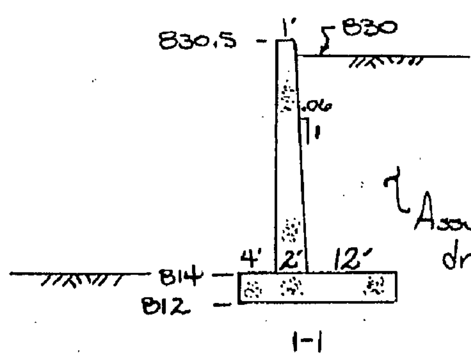
TVA 11030 (WM-7-75)

Modify Saddle Dam No. 1
Concrete Retaining Walls

45
TELLICO
PROJECT

COMPUTED BP DATE JUN 21 '88
CHECKED DATE 29 Jun 88





Surcharge @ 200 psf
$\phi = 26°$, $K_a = .4$
$\gamma_s = 120\ \#/ft^3$

Assume backfill drained

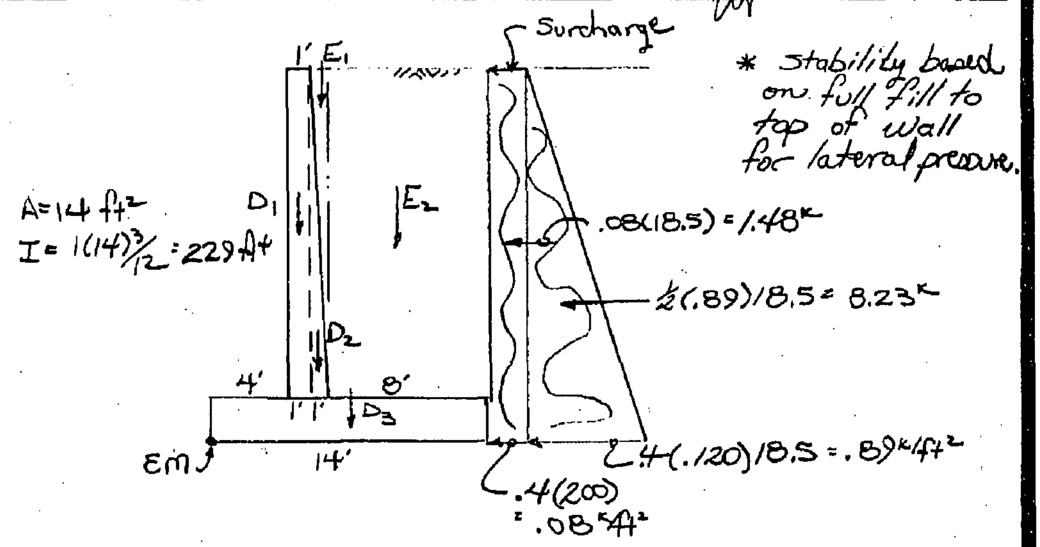
TVA 11030 (WM-7-75)

Modify Saddle Dam No. 1
Concrete Retaining Walls
Stability

SHEET 45 OF

TELLICO PROJECT

COMPUTED BP DATE JUN 22 '88
CHECKED DATE 27 Jun 88



* stability based on full fill to top of wall for lateral pressure.

Force	x or y	M_R	M_O
$D_1 = 1'(16.5).15 = 2.48^k$	4.5'	11.1	
$D_2 = \frac{1}{2}(1')16.5(.15) = 1.24^k$	5.33'	6.6	
$D_3 = 2'(14').15 = 4.2^k$	7'	29.4	
$E_1 = \frac{1}{2}(1')16'(.12) = 1.0^k$	5.66'	5.6	
$E_2 = 8'(16').12 = 15.4^k$	10'	154	
$24.3^k\downarrow$			
Lateral Force -1.48^k	9.25'		13.7
-8.23^k	6.17'		50.8
9.71^k		$207^{k'}$	$64.5^{k'}$

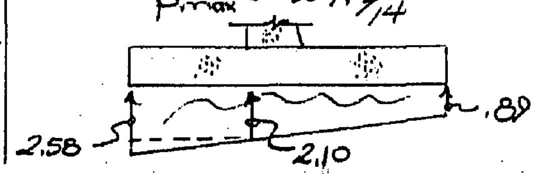
$FS_{OT} = 207/64.5 = 3.21$

Use friction factor of .45

$FS_{SLIDING} = \frac{24.3(.45)}{9.71} = 1.12$ ∴ provide shear key

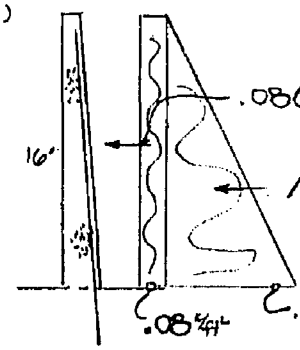
$x = (207-64.5)/24.3 = 5.86 \quad e = 1.14'$

$P_{max} = 24.3/14 + \frac{24.3(1.14)7}{229} = 2.58\ k/ft^2$ ∴ ok



Modify Saddle Dam No. 1 — SHEET 47
Concrete Retaining Walls — TELLICO
Reinf. — PROJECT
COMPUTED BP DATE JUN 22 '88
CHECKED DATE 29 Jun '88

Moment @ footing, Load Factor Design



M_u @ BASE $= 2.17^k(16/2) + 10.5(16/3)$
$= 17.4 + 57.4$
$= 74.8$ ^k-ft/ft.

$A_s \cong \dfrac{74.8(12)}{.9(.95\times22)60} = .79 in^2$

Try #8@12, $A_s = .79 in^2$, $d = 21.5"$

$p = .79/12(21.5) = .003$

$a = .79(60)/12(.85)3 = 1.55"$

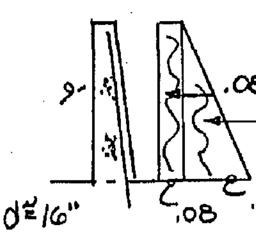
$(d - a/2) = 20.7"$

$M_u = [.79(60)(20.7)/12].9 = 73.5$ ^k-ft/ft. ≈ 74.8 ^k-ft/ft. ok

Chk shear, $V_u = 2.17 + 10.5 = 12.7^k$

$v_u = 12.7/.85(12)21.5 = .058$ ksi $< 2\sqrt{f'_c}$, ok

Calculate A_s Req'd @ CJ EL. 821,



$M_u = 1.22(9/2) + 3.3(9/3) = 15.5$ ^k-k

$A_s \cong \dfrac{15.5(12)}{.9(.92\times16)60} = .23 in^2$

Chk. shear, $V_u = 4.5^k$

$v_u = 4.5/.85(12)16 = .028$ ksi ok

#6@12", ok

TVA 11030 (WM-7-75)

48

Modify Saddle Dam No. 1
Concrete Retaining Walls
Reinf.

TELLICO PROJECT

COMPUTED BP DATE JUN 23 88
CHECKED [illegible] DATE 29 JUN 88

Calculate footing reinf.,

$V_u @ face = \frac{1}{2}(2.58 + 2.10) 4' (1.7) \cong 16^k$

$v_u = 16 / .85(12)20.5 = .077 \text{ ksi} < 2\sqrt{f_c'}$ ok

$M_u @ face = [\frac{1}{2}(.48)4(8/3) + 2.10(4)(2)] 1.7$
$= 33^{1\text{-}k}$

$A_{s_{reqd}} \cong 33(12) / .9(.9 \times 20.5)60$
$= .4 \text{ in}^2/\text{ft}.$

Try #6@12", $A_s = .44 \text{ in}^2/\text{ft}.$

$p = .44 / 12(20.5)$
$= .0018$ ok

Calculate temperature & shrinkage stl.

300:75 AFW, $d_c = 3"$
$w_c = .013"$ $T_{PLACING} = 80°F$
Placing: $V/S \cong \frac{3}{4}(2') = 1.5$ $T_{max} = 100°F$ $> 65°F$
In Service $\cong \frac{2}{3}(2') = 1.33$ $= 35°F$

$f_t = .42 \text{ ksi}$
$f_s = 28.5 \text{ ksi}$ w/ $s = 12"$ $L_{wall} = 80'$ $p = .0033$

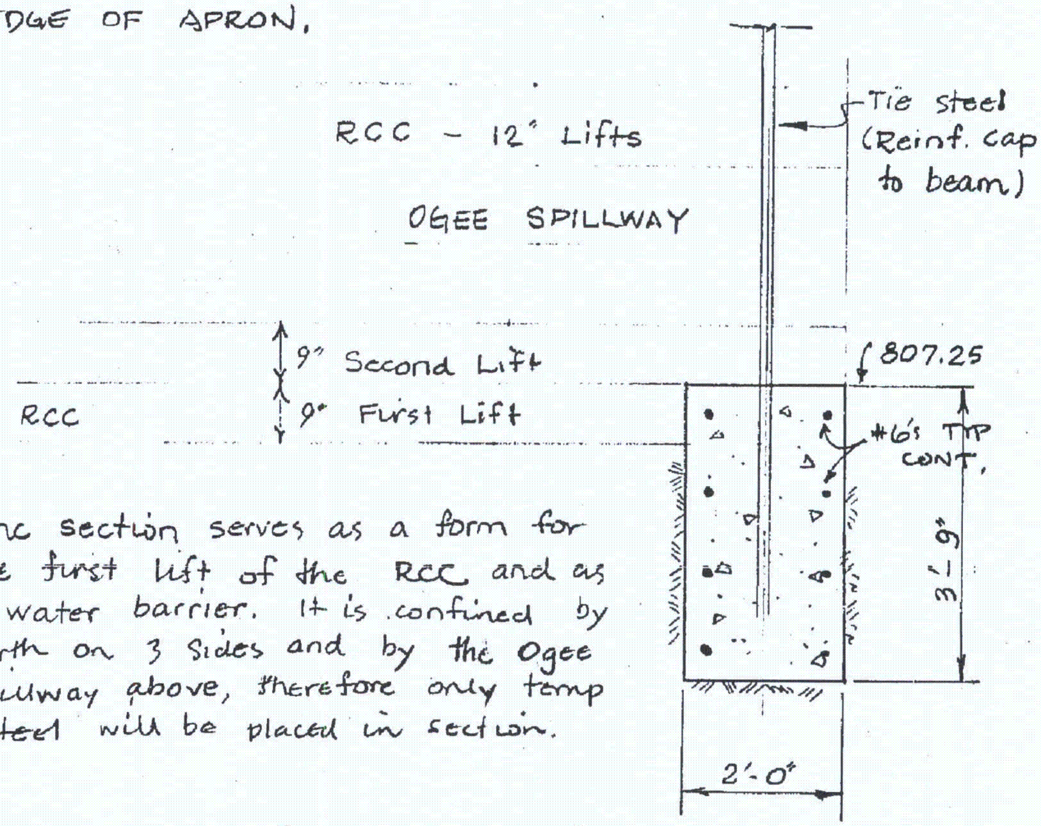
$A_b = 460 \, p \, B_s (f_t / f_s)$
$= 460(.0033) \, 2 \, (12)(.42 / 28.5)$
$= .54 \text{ in}^2/\text{ft}.$ say #6@12 ok, due to thickness varies

SHEET 49 OF

MODIFY SADDLE DAM 1 TELLICO PROJECT

COMPUTED AE DATE Aug 1, 1988
CHECKED DATE

CONC BEAM UNDER OGEE SPILLWAY & DWNSTREAM EDGE OF APRON.



Conc section serves as a form for the first lift of the RCC and as a water barrier. It is confined by earth on 3 sides and by the Ogee Spillway above, therefore only temp steel will be placed in section.

Assume ratio of A_s to A_c = .2%,

therefore $A_s = 0.002(24)(45) = 2.16 \text{ in}^2 = 1.08 \text{ in}^2$ ea. face

GLB says use 8 #6's. $A_s = 8(0.44) = 3.52 \text{ in}^2$

$$\frac{3.52}{(24)(45)} = 0.0033$$

Attachment 5

Table 6.3 from Reference 2.7

Table 6.3 Porosity, Void Ratio, Density, and Unit Weight of Typical Soils in Natural State

Description	Porosity, n (%)	Void ratio (e)	Water content, w (%)	Density (Mg/m^3) ρ_d	Density (Mg/m^3) ρ_{sat}	Unit Weight (kN/m^3) γ_d	Unit Weight (kN/m^3) γ_{sat}
1. Uniform sand, loose	46	0.85	32	1.43	1.89	14.0	18.5
2. Uniform sand, dense	34	0.51	19	1.75	2.09	17.2	20.5
3. Mixed-grained sand, loose	40	0.67	25	1.59	1.99	15.6	19.5
4. Mixed-grained sand, dense	30	0.43	16	1.86	2.16	18.2	21.2
5. Glacial till, very mixed-grained	20	0.25	9	2.12	2.32	20.8	22.7
6. Soft glacial clay	55	1.2	45		1.77	12.0	17.4
7. Stiff glacial clay	37	0.6	22		2.07	16.7	20.3
8. Soft slightly organic clay	66	1.9	70		1.58	9.1	15.5
9. Soft very organic clay	75	3.0	110		1.43	6.7	14.4
10. Soft bentonite	84	5.2	194		1.27	4.2	12.5

w = water content when saturated, in percent of dry weight.
ρ_d = density in dry state.
ρ_{sat} = density in saturated state.
γ_d = unit weight in dry state.
γ_{sat} = unit weight in saturated state.

2. A sample of hardpan had a weight of 129.1 g and a volume of 56.4 cm^3 in its natural state. Its dry weight was 121.5 g. The specific gravity of the solid constituents was found to be 2.70. Compute the water content, the void ratio, and degree of saturation.

Ans. $w = 6.3\%$; $e = 0.25$; $S_r = 67\%$.

3. The density of a sand backfill was determined by field measurements to be 1.75 Mg/m^3. The water content at the time of the test was 8.6%, and the specific gravity of solid constituents was 2.60. In the laboratory the void ratios in the loosest and densest states were found to be 0.642 and 0.462, respectively. What were the void ratio and the relative density of the fill?

Ans. $e = 0.616$; $D_r = 14\%$.

4. A dry quartz sand sample weighs 1.54 Mg/m^3. What is its density when saturated?

Ans. $\rho = 1.96$ Mg/m^3.

5. A sample of silty clay had a volume of 14.88 cm^3. Its weight at the natural water content was 28.81 g and after oven-drying was 24.83 g. The specific gravity of solid constituents was 2.70. Calculate the void ratio and the degree of saturation of the sample.

Ans. $e = 0.617$; $S_r = 70\%$.

6. Given the values of porosity n for the soils in Table 6.3, check the values of void ratio e, water content w, density ρ, and unit weight γ. For soils 1 to 5, $G_s = 2.65$; for soils 6 to 10, $G_s = 2.70$.

ARTICLE 7 CONSISTENCY OF FINE-GRAINED SOILS

7.1 Consistency and Sensitivity of Undisturbed Soils

The consistency of clays and other cohesive soils is usually described as *soft, medium, stiff,* or *hard.* The most direct quantitative measure of consistency is the load per unit of area at which unconfined prismatic or cylindrical samples of the soil fail in a compression test. This quantity is known as the *unconfined compressive strength* of the soil. Values of the compressive strength corresponding to the various degrees of consistency are given in Table 7.1.

Table 7.1 Consistency of Clay in Terms of Unconfined Compressive Strength

Consistency	Unconfined Compressive Strength, q_u(kPa)
Very soft	Less than 25
Soft	25–50
Medium	50–100
Stiff	100–200
Very stiff	200–400
Hard	Over 400

Attachment 6

PMF Headwater and Tailwater Elevations

Long, J. Justin

From: Hasan, Husein A
Sent: Wednesday, September 30, 2009 6:10 PM
To: 'Booth, Paul'; 'Nathan.Mathis@bwsc.net'; 'Zimmerman, Peter'; Duncan, Johnathan D; Long, J. Justin; Hoskins, Coleman E; 'McClung, Nicholas'; Lennon, Kendal R; Dodd, Charles B Jr; Lowe, Mark Christopher; Chaffee, Duane E
Cc: Tompkins, Russell W; Lowe, Gregory W; Maddux, Perry D; chris.triplett@bwsc.net
Subject: RE: New HW/TW PMF Elevations

Sensitivity: Private

FYI - See e-mail from Greg Lowe below: We will use WBH TW elevation 739.0 as recommended by Greg. With this, we have all the information we need to proceed with final stability calculations and design. We need to finalize all design calculations within the next few days as we are starting construction Monday.

Husein

From: Lowe, Gregory W
Sent: Wed 09/30/2009 4:56 PM
To: Hasan, Husein A
Cc: Maddux, Perry D; 'Chris Triplett'
Subject: RE: HW/TW PMF Elevations - Watts Bar

Husein,

We have taken a look at available data. Our best estimate for Watts Bar with HW at elevation 768.27 is a **TW of 739.0**. If you get any indication that this is creating an instability please let us know. We will confirm the final TW at Watts Bar as soon as the information is available.

Thanks Greg

From: Hasan, Husein A
Sent: Tue 09/29/2009 6:17 PM
To: 'Booth, Paul'; 'Nathan.Mathis@bwsc.net'; 'Zimmerman, Peter'; Duncan, Johnathan D; Long, J. Justin; Hoskins, Coleman E; 'McClung, Nicholas'; Lennon, Kendal R; Dodd, Charles B Jr
Cc: Tompkins, Russell W; Lowe, Gregory W; Maddux, Perry D; chris.triplett@bwsc.net
Subject: RE: New HW/TW PMF Elevations

In a meeting this afternoon, Perry, Greg, and Chris have provided us with the HW and TW elevations for the following dams:

Dam	HW	TW
FLH	835.65	821.2
WBH	768.27	will be provided on 9/30
TEH	833.34	820.3
CRH	1090.68	981.5

The stability analyses that you are performing that are related to the Temporary Modifications should be based on the above elevations.

Husein Hasan
Project Manager, Dam Safety
865-632-4194
423-751-7340

From: Hasan, Husein A
Sent: Tue 09/29/2009 12:59 PM
To: 'Booth, Paul'; 'Nathan.Mathis@bwsc.net'; 'Zimmerman, Peter'; Duncan, Johnathan D; Long, J. Justin; Hoskins, Coleman E; 'McClung, Nicholas'; Lennon, Kendal R; Dodd, Charles B Jr
Cc: Tompkins, Russell W; Lowe, Gregory W
Subject: WH/TW PMF Elevations

To help finalize the design of the Temporary Modifications, Greg provided us with the elevations shown below this morning. He will provide us with the CRH (HW/TW) elevations and TW elevations later today.

Dam	HW	TW
FLH	835.65	Later Today
WBH	768.27	Later Today
THE	833.34	Later Today
CRH	Later Today	Later Today

Husein Hasan
Project Manager, Dam Safety
865-632-4194 (knoxville Office)
423-751-7340 (knoxville Office)

Attachment 7

Moisture-Density Relationships for Sand Utilized in the Temporary Dams Modification Project

Moisture-Density Relationships for Sand Utilized in the Temporary Dams Modification Project

October 29, 2009

Samples Collected and Calculations Performed By: John Lane

Calculations Reviewed and Report Written By: J. David Lane Jr.

Moisture-Density Relationships for Sand Utilized in the Temporary Dams Modification Project

Purpose

The determination of moisture-density relationships of sand utilized in conjunction with HESCO Concertainers® at Cherokee, Fort Loudoun, Tellico, and Watts Bar Dams. The moisture-density relationships calculated will provide an optimum moisture content percentage to maximum unit weight.

After the optimum moisture content is reached, the unit weight of sand decreases as the moisture content increases for well-graded sands. In the case of poorly-graded sands, the optimum moisture content is not able to be determined because of their free-draining nature. Sand gradation was not performed in this study because it was beyond the scope of work, but plays a role in the differing results between sand types.

Methods

Sand samples were collected onsite at Cherokee and Watts Bar Dams for laboratory analysis. The Fort Loudoun and Tellico Dam sand was collected from the Vulcan Materials quarry located on Watt Road. Initial moisture content was obtained by ASTM D 2216-05 "Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass." Moisture-Density relationships for each type of sand were established using Method A of ASTM D 698-07 "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lb/ft^3 (600 kN-m/m^3))."

Report Organization

This report will be divided into sections by dam location with results, moisture-density graph, and analysis of results. A summary and conclusion of results will be presented prior to the Appendices.

Appendices

Appendix A contains information submitted from two quarries about the gradation of the material delivered to Watts Bar Dam, Fort Loudoun, and Tellico Dams. There is no data pertaining to the density of either sand material.

Appendix B contains calculations for the determination of data points for the moisture-density curve plots. This includes the moist density, dry density, and dry unit weight calculations.

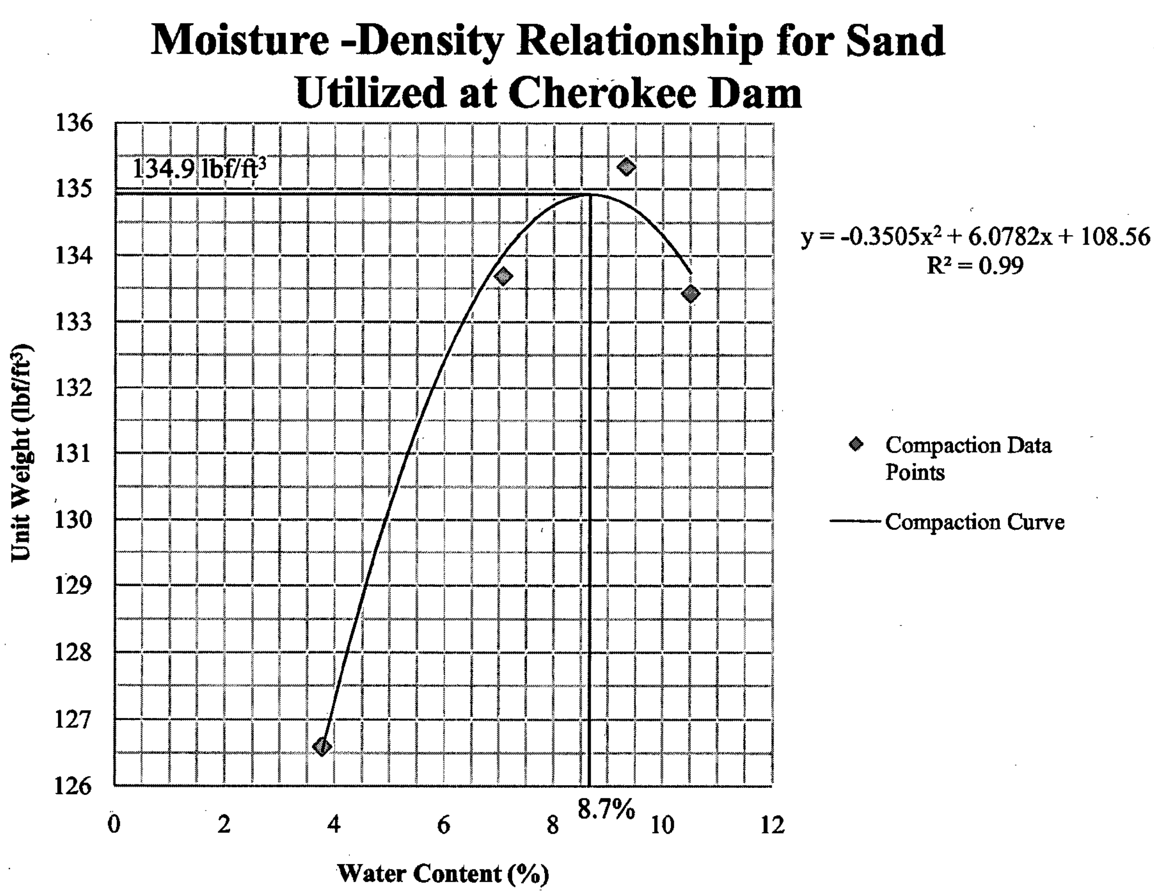
Cherokee Dam

Material

Obtained onsite, delivered from Vulcan Materials Morristown (River Bend) quarry, Morristown, TN.

Results

Initial moisture content of sand: 4.1%
Unit weight at initial moisture content: 127.6 lbf/ft^3
Optimum moisture content: 8.7%
Unit weight at optimum moisture: 134.9 lbf/ft^3
Dry unit weight: 108.6 lbf/ft^3



Analysis of Results

Although a specific test to determine sand gradation was not performed, the results obtained from the compaction test show that the sand is well-graded and therefore an optimum moisture-density relationship is obtained. The compaction curve is determined from a second-order polynomial function to best fit the compaction data points. The R^2 value of 0.99 shows that the regression compaction curve fits the actual data points well. The equation can then be used to determine additional unit weights of this sand at various moisture contents with confidence.

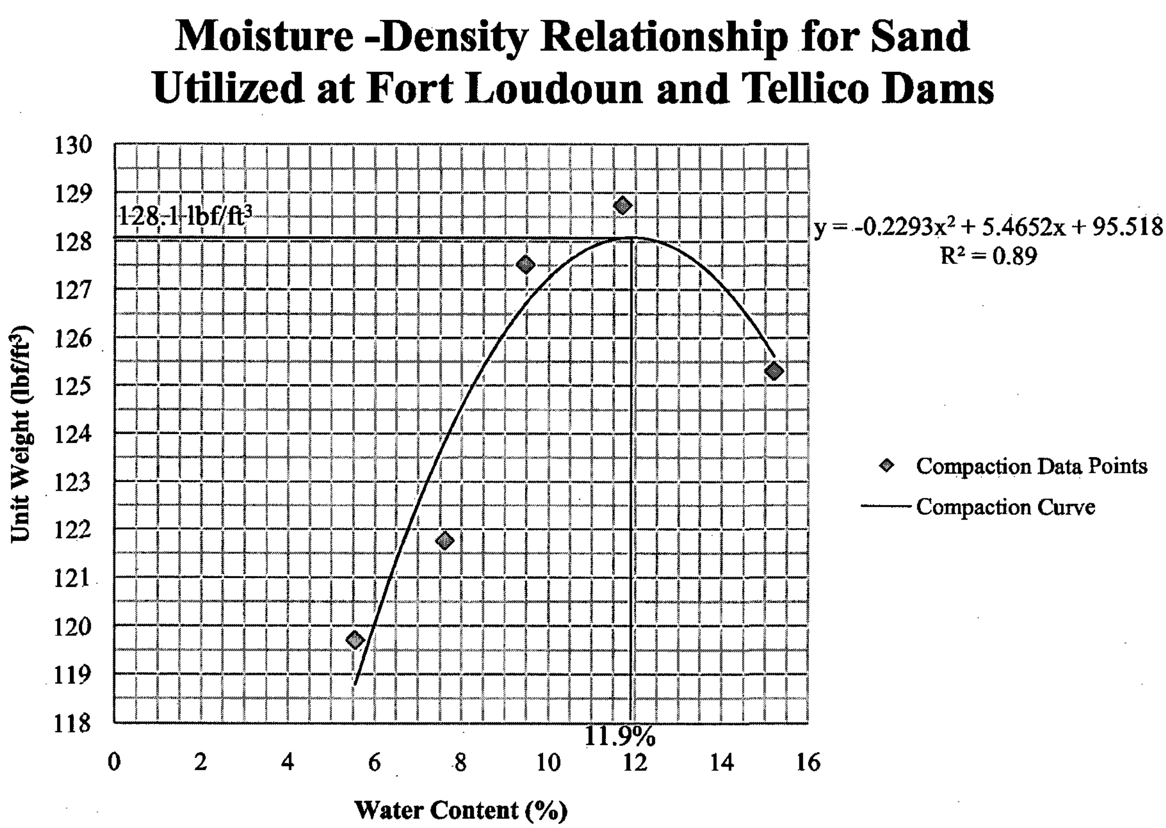
Fort Loudoun and Tellico Dams

Material

Obtained from Vulcan Materials quarry located on Watt Road, Knoxville, TN.

Results

Initial moisture content of sand: 3.5%
Unit weight at initial moisture content: 111.8 lbf/ft^3
Optimum moisture content: 11.9%
Unit weight at optimum moisture: 128.1 lbf/ft^3
Dry unit weight: 95.5 lbf/ft^3



Analysis of Results

Although a specific test to determine sand gradation was not performed, the results obtained from the compaction test show that the sand is well-graded and therefore an optimum moisture-density relationship is obtained. The compaction curve is determined from a second-order polynomial function to best fit the compaction data points. The R^2 value of 0.89 shows that the regression compaction curve fits the actual data points with relative confidence. The equation can then be used to determine additional unit weights of this sand material at various moisture contents.

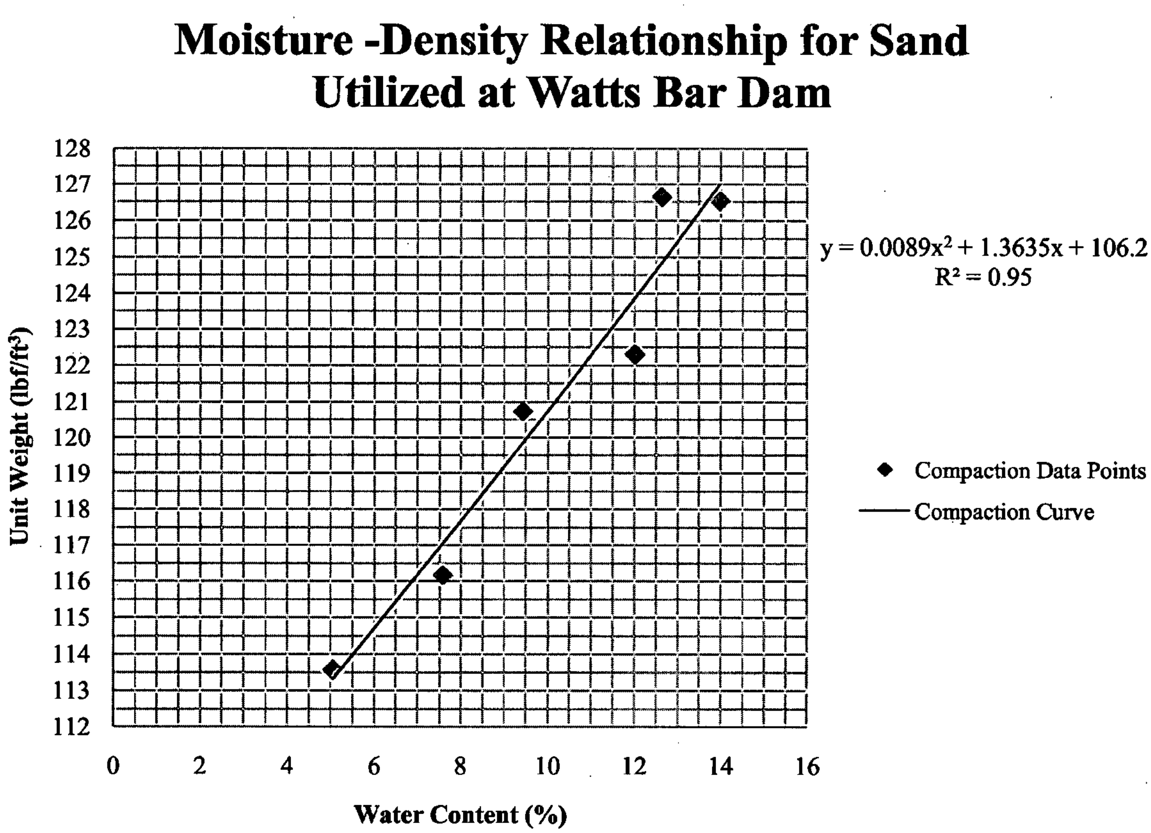
Watts Bar Dam

Material

Obtained onsite, delivered from Vulcan Materials quarry located in Dayton, TN.

Results

Initial moisture content of sand: 4.7%
Unit weight at initial moisture content: 112.8 lbf/ft^3
Optimum moisture content: NA
Unit weight at optimum moisture: NA
Dry unit weight: 106.2 lbf/ft^3



Analysis of Results

Although a specific test to determine sand gradation was not performed, the results obtained from the compaction test point toward sand that is poorly-graded and therefore an optimum moisture-density relationship cannot be obtained. The graph presented above is indicative of free draining sand that does not compact at optimum moisture content.

Conclusions

The table of results presented below was established conforming to ASTM D 2216-05 "Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass." Moisture-Density relationships for each type of sand were established using Method A of ASTM D 698-07 "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lb/ft^3 (600 kN-m/m^3))."

The laboratory tests performed on the sand located at the Cherokee Dam indicate an in-situ unit weight of over 128,1 lbf/ft^3 on the day of sampling. The laboratory tests performed on the sand located at Fort Loudoun and Tellico Dams indicate an in-situ unit weight of 111.8 lbf/ft^3. The laboratory tests performed on the sand located at Watts Bar Dam indicate an in-situ unit weight of 112.8 lbf/ft^3. The moisture content is indeterminate on the sand located at Watts Bar Dam due to its free-draining, poorly-graded nature.

Location of Sand	Dry Unit Weight (lbf/ft^3)	Optimum Moisture Content (%)	Unit Weight at Optimum Moisture Content (lbf/ft^3)	Initial Moisture Content (%)	Unit Weight at Initial Moisture Content (lbf/ft^3)
Cherokee Dam	108.6	8.7	134.9	4.1	127.6
Fort Loudoun & Tellico Dams	95.5	11.9	128.1	3.5	111.8
Watts Bar Dam	106.2	NA*	NA*	4.7	112.8

* Moisture Content Indeterminate

Appendix A

TEH
FTL

STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATION
DIVISION OF MATERIALS AND TESTS
REPORT ON SAMPLES FOR FINE AGGREGATE

Form DT-0321
Re: T-202
Revised 7-1-82

Project Reference No. ________ County ________ Region 1
Project No. ________
Serial No. 09P5223 Contract No. ________ Date Sampled 3-10-09
Report No. ________ Plant Letter ________ Date Received 3-18-09
Material MFG Sand Date Reported 5-6-09
Pit No. ________ County Pit Located in ________ Submitted by Pam Porter
Producer Vulcan MATLS Dixie Lee Sampled by Heathel C Hale Jr
Producer Location Everett Rd Concord, TN Sampled from Stockpile
Exact Location in Quarry where sample obtained ________
Type of Construction to be used in ________

SCREEN ANALYSIS					SOUNDNESS TEST				
SIEVE	WT. RET.	% RET.	TOTAL % RET.	TOTAL % PASSING	WT.	WT. NET	LOSS	% LOSS	CORRECTED TOTAL % LOSS
4	0.0	0	0	100					
8	20.9	6	6	94	100	98.9	1.1	1.1	0.1
16	117.6	33	39	61	100	99.0	1.0	1.0	0.3
28	84.7	23	62	38	100	99.4	0.6	0.6	0.1
48	50.9	14	76	24	100	99.4	0.6	0.6	0.1
100	43.2	12	88	12					0.6
200									
PAN	43.7	12	100	0	%Loss	1			
TOTAL	361.0	100			%Sound	99			

Water used: Ottawa ________ C.C. Sample ________ cc.
Brand Cement used ________
Loss on Washing ________ F.M. ________
Color Plate No. ________
Specific Gravity 2.82 SSD
Wt. per cubic ft. (Dry Rodded) ________
Clay Lumps ________
Coal and Lignite ________
No. Sodium Sulfate Soundness Cycles 5

COMPRESSIVE STRENGTH PSI			
Standard Ottawa Sand		Sample Sand	
7-Day	28-Day	7-Day	28-Day
Aver.			
Str. Ratio			

RECEIVED
MAY 1 2009
STATE OF TENNESSEE
BUREAU OF ...

This material met the requirements of the specifications for TDOT

Remarks

Materials Testing Supervisor

10/20/2009 14:09 8656915950 PAGE 02/02

Watts Bar

#10

Vulcan
Materials Company

Material Inspection Report Summary

10/23/2009

Plant: Dayton

Product: 30090-10

Sieve Size	Average % Passing	Specifications
3/8" (9.5mm)	100.0	100-100
#4 (4.75mm)	92.9	85-100
#8 (2.36mm)	64.9	
#16 (1.18mm)	41.0	
#30 (0.6mm)	26.2	
#50 (0.3mm)	17.5	
#100 (0.15mm)	12.9	10-30
#200 (75um)	10.55	

Appendix B

11.2.2.1 *Moist Density:*

$$\rho_m = K \times \frac{(M_t - M_{md})}{V} \quad (4)$$

where:

ρ_m = moist density of compacted subspecimen (compaction point), four significant digits, g/cm^3 or kg/m^3,
M_t = mass of moist soil in mold and mold, nearest g,
M_{md} = mass of compaction mold, nearest g,
V = volume of compaction mold, cm^3 or m^3 (see Annex A1), and
K = conversion constant, depending on density units and volume units.
Use 1 for g/cm^3 and volume in cm^3.
Use 1000 for g/cm^3 and volume in m^3.
Use 0.001 for kg/cm^3 and volume in m^3.
Use 1000 for kg/m^3 and volume in cm^3.

11.2.2.2 *Dry Density:*

$$\rho_d = \frac{\rho_m}{1 + \frac{w}{100}} \quad (5)$$

where:

ρ_d = dry density of compaction point, four significant digits, g/cm^3 or kg/m^3, and
w = molding water content of compaction point, nearest 0.1 %.

11.2.2.3 *Dry Unit Weight:*

$$\gamma_d = K_1 \times \rho_d \text{ in lbf/ft}^3 \quad (6)$$

or

$$\gamma_d = K_2 \times \rho_d \text{ in kN/m}^3$$

where:

γ_d = dry unit weight of compacted specimen, four sign cant digits, in lbf/ft^3 or kN/m^3,
K_1 = conversion constant, depending on density units,
Use 62.428 for density in g/cm^3, or
Use 0.062428 for density in kg/m^3,
K_2 = conversion constant, depending on density units,
Use 9.8066 for density in g/cm^3, or
Use 0.0098066 for density in kg/m^3.

Reference

ASTM D 698-07, "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3))" pp. 8-9

Attachment 8

Applicability of Fort Loudoun Dam Calculation to Tellico, Cherokee, and Watts Bar Dams

Attachment 8:
Applicability of Fort Loudoun Dam Calculation to Tellico, Cherokee, and Watts Bar Dams

1.0 Introduction

Of the four projects that require temporary modification to meet their new PMF headwater elevations, Fort Loudoun is the only project that has atypical applications of the Hesco Concertainer flood walls. Therefore, the purpose of this attachment is to address the applicability of the Fort Loudoun calculation to Tellico, Cherokee, and Watts Bar Dams.

2.0 Tellico Dam

As shown on drawings 10W222-1 and 10W222-2, the temporary flood wall at Tellico Dam is composed of approximately 6,500-feet of 4-foot tall Concertainers. A survey of the route performed of September 25, 2009, showed that the minimum ground surface elevation along the flood wall route was 829.75-feet. As shown in Section 6.1 of this calculation, the single 4-foot Concertainer flood wall has adequate stability to withstand a pool elevation up to its full 4-foot height. Therefore, the Concertainer flood wall at Tellico Dam can withstand a headwater elevation of 833.75-feet which exceeds the PMF headwater elevation of 833.34-feet.

3.0 Cherokee Dam

As shown on drawings 10W222-1 and 10W222-2, the temporary flood wall at Cherokee Dam is composed of approximately 7,000-feet of 3-foot tall Concertainers. A survey of the route showed that the minimum ground surface elevation along the flood wall route was 1,088.54 -feet. As shown in Section 6.2 of this calculation, the single 3-foot Concertainer flood wall has adequate stability to withstand a pool elevation up to its full 3-foot height. Therefore, the Concertainer flood wall at Cherokee Dam can withstand a headwater elevation of 1,091.54-feet which exceeds the PMF headwater elevation of 1,090.68-feet.

4.0 Watts Bar Dam

As shown on drawing 10W222-1, the temporary flood wall at Watts Bar Dam is composed of approximately 1,800-feet of 3-foot tall Concertainers. A survey of the route performed of August 31, 2009, showed that the minimum ground surface elevation along the flood wall route was 766.65-feet. As shown in Section 6.2 of this calculation, the single 3-foot Concertainer flood wall has adequate stability to withstand a pool elevation up to its full 3-foot height. Therefore, the Concertainer flood wall at Watts Bar Dam can withstand a headwater elevation of 769.65-feet which exceeds the PMF headwater elevation of 768.27-feet.

The use of Concertainers as additional weight on the heel of the existing flood wall on the lock side of Watts Bar Dam is shown in calculation RSOWBHROGCDX00033320090006.

5.0 Conclusion

In conclusion, the temporary flood wall configurations at Tellico, Cherokee, and Watts Bar Dams have adequate stability to withstand their site specific PMF headwater elevations.