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CHATFIELD DAM AND LAKE
SOUTH PLATTE RIVER BASIN
LITTLETON, COLORADO

DAM SAFETY EVALUATION REPORT

PROJECT MANAGEMENT PLAN

1. PRODUCT DEFINITION

It has been proposed to raise the normal reservoir elevation of the Chatfield Reservoir by up to 12 feet, from El. 5432 Ft. MSL to El. 5444 Ft. MSL, for the purpose of water supply. The water re-allocation study will actually evaluate three options, (1) no raise, (2) a five foot raise, and (3) a twelve foot raise. The final adopted plan will not increase the maximum surcharge reservoir elevation. The historic maximum reservoir elevation at Chatfield Dam is 5447.58. Ft. MSL. The purpose of this report is to evaluate potential dam safety concerns based on a permanent increase in the reservoir elevation. The evaluation is based strictly on static loading; however, historic information on previous seismic evaluations will be discussed. It is vital to address various aspects of design and performance to assure that the proposed modifications do not impact the continued safe operation of the dam and do not pose dam safety concerns.

2. PRODUCT DEVELOPMENT TEAM

The product development team consists of:

Michael T. Kelly, CENWO-ED-GB (Geotechnical Engineer)

Larkin Whistler, CENWO-ED-DF (Structural Engineer)

Eric Laux, CENWO-PM-C (Project Manager)

3. QUALITY CONTROL PROCESS

The quality control process will consist of a peer review for each discipline invoved in the product, a quality control review and an independent technical review. All comments will be resolved to the satisfaction of the Peer Review Team, Quality Control Review Team and the Independent Technical Review Team. Those participating on these teams are the following:

PERR REVIEW TEAM

Bob Worden, CENWO-ED-GB (Geotechnical Engineer)

Lyle Peterson, CENWO-ED-DF (Structural Engineer)

QUALITY CONTROL REVIEW TEAM

Richard Taylor, Chief , CENWO-ED-GB

Bruce Harris, Chief, CENWO-ED-GB

INDEPENDENT TECHNICAL REVIEW TEAM

Joseph Topi, CENWK-EC-GD (Geotechnical Engineer)

4.0 SCHEDULE

The proposed schedule milestone dates for the quality control reviews are given below.

Daft Report Complete by   15 Nov 08

Independent Technical Reviews Complete by 1 Dec 08

Peer Reviews Complete by 1 Jan 09

Quality Control Review Complete by 15 Jan 09

Final Report Complete by 1 Feb 09

5.0 DOCUMENTATION

Documentation of the quality control process will be recorded on the following forms. Reviewers will sign and date the forms after their review is complete.

CHATFIELD DAM AND LAKE
SOUTH PLATTE RIVER BASIN
LITTLETON, COLORADO

DAM SAFETY EVALUATION REPORT
PEER REVIEW CERTIFICATION

The undersigned certify that the Chatfield Dam Safety Evaluation report has been reviewed and that all significant comments generated have been addressed.

DATE

APPROVED BY: ______________________________

BOB WORDEN, P.E. SOILS SECTION B,
GEOTECHNICAL ENGINEERING AND SCIENCES BRANCH,
ENGINEERING DIVISION

APPROVED BY: ______________________________

LYLE PETERSON, P.E. STRUCTURAL/ INTERIOR DESIGN SECTION, ENGINEERING DIVISION

CHATFIELD DAM AND LAKE
SOUTH PLATTE RIVER BASIN
LITTLETON, COLORADO

DAM SAFETY EVALUATION REPORT
QUALITY CONTROL REVIEW CERTIFICATION

The undersigned certify that Chatfield Dam Safety Evaluation Report has been reviewed and that all significant comments generated by the quality control review team have been addressed.

DATE

APPROVED BY: ______________________________
RICHARD TAYLOR, P.E.. CHIEF, SOILS SECTION B
GEOTECHNICAL ENGINEERING AND SCIENCES BRANCH,
ENGINEERING DIVISION

APPROVED BY: ______________________________
BRUCE HARRIS, P.E. CHIEF, STRUCTURAL/INTERIORS SECT.,
DESIGN BRANCH, ENGINEERING DIVISION

CHATFIELD DAM AND LAKE
SOUTH PLATTE RIVER BASIN
LITTLETON, COLORADO

DAM SAFETY EVALUATION REPORT

INDEPENDANT TECHNICAL REVIEW CERTIFICATION

The undersigned certify that the Chatfield Dam Safety Evaluation Report has been reviewed.

DATE

APPROVED BY: ______________________________
Joseph Topi, CENWK-EC-GC

POST-LIQUEFACTION STABILITY ANALYSES DECEMBER 2010

CHATFIELD DAM & LAKE SOUTH PLATTE RIVER BASIN LITTLETON, COLORADO

U.S. Army Corps Of Engineers Omaha District Final Report

Post-Liquefaction Stability Analyses
Chatfield Dam
Littleton, Colorado

Table of Contents

Post-Liquefaction Stability Analyses
Chatfield Dam
Littleton, Colorado

1.0 Introduction This report presents the results of stability analyses performed on zones of the Chatfield Dam foundation that have been identified as susceptible to liquefaction (CENWO, 2009a).

2.0 Background A liquefaction assessment was performed in April 2009 as part of the water supply reallocation study for Chatfield Dam. The reallocation study is evaluating three options for a reservoir raise including: 1) no raise, 2) a five-feet raise in the multi-purpose pool, and 3) a 12-feet raise in the multi-purpose pool.

The scope of the liquefaction assessment (CENWO, 2009a) included an evaluation of the liquefaction potential of the Chatfield Dam embankment and foundation for both the existing multi-purpose reservoir (El. 5432) and the proposed 12-feet raise (El 5444). Granular soils were evaluated using the "Simplified Seed Method" (Youd et.al., 2001) which defines a Factor of Safety (F.S.) as the Cyclic Resistance Ratio (CRR) divided by the Cyclic Stress Ratio (CSR). The CRR is based on Standard Penetration Test (SPT) blow counts, corrected for fines content and hammer efficiency and normalized to an effective overburden pressure of one ton per square foot $(N_1)_{60}$. For the purpose of the liquefaction assessment, Factors of Safety less than 1.1 were deemed to liquefy.

The liquefaction assessment concluded that:

- The Chatfield Dam embankment would most likely be safe against liquefaction for a 6.0 (Mw) maximum credible earthquake with a PGA of 0.32g. Evaluation of the Chatfield Dam foundation indicates zones of liquefaction are likely along the upstream and downstream slope in the valley section.

- Review of soil conditions along the right abutment tends to indicate saturated zones of relatively loose silty sand and sand that may be prone to liquefaction.

- Recommendations included conducting post-earthquake limit equilibrium slope stability analyses to evaluate the stability of the upstream and downstream slopes in the valley section and right abutment section due to the potential for liquefaction of the foundation soils.

3.0 Cases Considered Two cross-sections were selected for post-liquefaction slope stability analyses: a maximum valley section at Station 95+00 and a right abutment section at Station 57+60. For each combination of section and slope, two multi-purpose pool levels were considered: the existing pool level (EL. 5432) and a 12-feet raise (El. 5444).

The maximum valley section was evaluated in the previously discussed liquefaction assessment. Based on the F.S. results at Station 95+00 and adjacent borings, an upstream liquefied zone was identified from El. 5375 to El. 5380 and a downstream liquefied zone was identified from El. 5352 to El. 5361.

The right abutment section was evaluated in the 1986 Seismic Evaluation (CENWO, 1986). The SPT values for this section were taken in holes advanced with a churn drill, so there is some concern that the SPT results were influenced by the drilling operation. However, the values were used only for relative comparisons with adjacent values and to identify low SPT zones. Based on the SPT results at Station 57+60 and adjacent borings, an upstream liquefied zone was identified from El. 5426 to El. 5432 and a downstream liquefied zone was identified from El. 5411 to El. 5432. The residual shear strength was determined from the blow counts of nearby SPT-12, which was evaluated in the liquefaction assessment (CENWO, 2009a).

For each section and pool level, two conditions for the horizontal extent of the potential liquefied zone were assumed: a) a zone extending beneath the entire upstream or downstream slope, and b) a zone extending ±100 feet of the embankment toes. Sketches of these liquefied zones are shown on Figures 1a and 1b. The zone under the entire slope is considered conservative, while the zone under the embankment toes is considered more realistic, given that the overburden confining pressure increases with increasing embankment fill height, which reduces liquefaction potential under the slope.

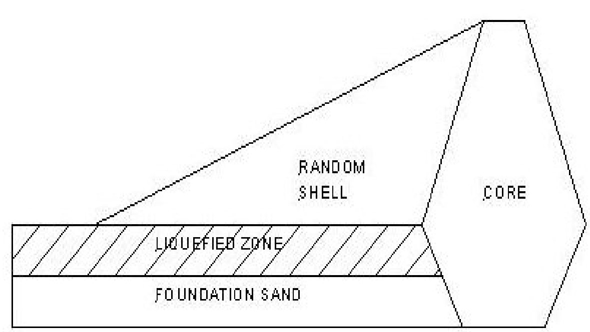


Figure 1a: Liquefied Zone Under Embankment Slope

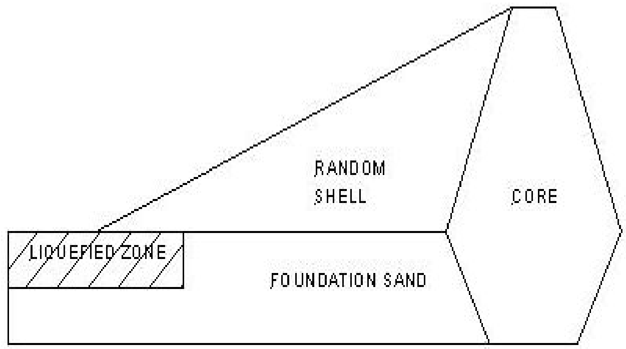


Figure 1b: Liquefied Zone Under Embankment Toe

Prior to performing the post-liquefaction stability analyses, the original design static stability analyses for the steady state seepage condition were reanalyzed using Spencer's method. The embankment geometry, foundation conditions, pool levels, and peak effective shear strengths (CD or S) were obtained from the Embankment Criteria and Performance Report (CENWO, 1980). Excerpts from this report are presented in Appendix A.

4.0 Analytical Approach The analytical approach was based on the guidance provided in the draft EM 1110-2-6001, "Seismic Stability of Earth and Rock Fill Dams," Chapter 8, "Post-Earthquake Stability Analysis" (USACE, 1998). This approach uses the effective stress parameters for the non-liquefied materials and an undrained residual strength for the liquefied zones. Stability analyses are conducted using circular arcs or non-circular surfaces and Spencer's method.

5.0 Adopted Design Parameters

5.1 Embankment Geometry The embankment geometry for the valley section (Sta. 95+00) was obtained from the "Embankment Criteria and Performance Report" (CENWO, 1980). The embankment geometry for the right abutment section (Sta. 57+60) was obtained from the 1986 Seismic Evaluation (CENWO, 1986).

5.2 Embankment Zoning The Chatfield Dam is a rolled, zoned, earthfill embankment. The embankment zoning consist of a symmetrical central impervious core extending to the Dawson formation bedrock, upstream and downstream random material shells, and a downstream pervious inclined sand drain with continuous outlets adjacent to the impervious core. The outer portion of the downstream random zone includes a zone specifically for all Dawson formation materials excavated from the spillway and outlet works excavations. This was done to keep the Dawson formation material least susceptable to saturation. A cross-section of the maximum valley section (station 95+00) is presented in Figure 2.

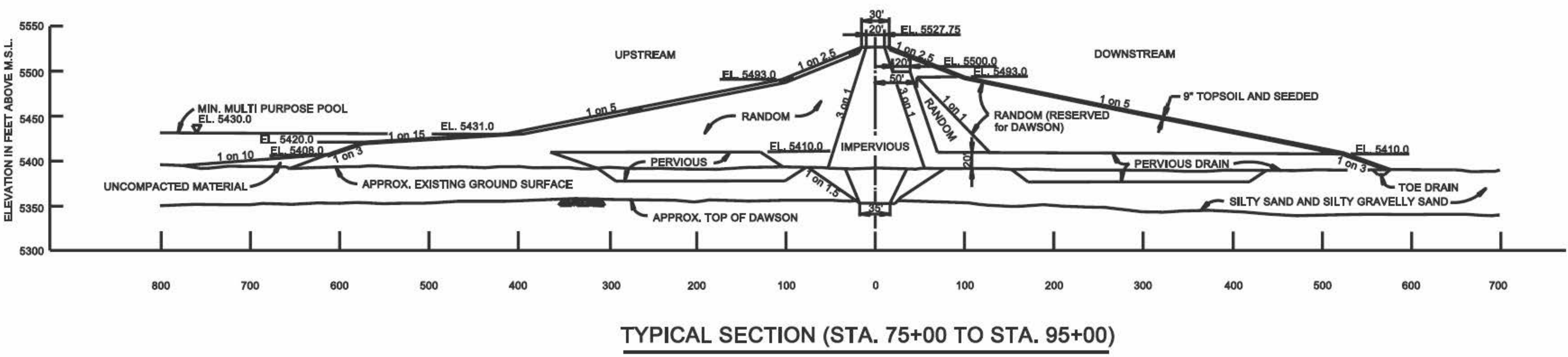


Figure 2: Embankment Zoning and Foundation Conditions in Valley

5.3 Foundation The dam foundation consists of sands, gravels, and sandy clay alluvium derived from the weathering and erosion of the parent materials of the mountains to the west, underlain by uncemented sand and sandy gravel and the Dawson formation bedrock. This study focuses on the granular alluvium and overburden soils overlying the Dawson. A cross-section of the maximum valley section is presented in Figure 2.

5.4 Shear Strengths The peak effective stress parameters (CD or S strengths) were selected from the "Embankment Criteria and Performance Report" (CENWO, 1980) for the non-liquefied materials (See Appendix A). These values are presented in Table 1.

Table 1: Adopted Peak Effective Stress (CD or S) Shear Strengths

Material	Cohesion (psf)	Friction angle (degrees)
Embankment		
Impervious	0	24.3
Random	0	24.3
Pervious	0	36.1
Dawson Fm.	0	22.3
Foundation		
Clay	0	20.8
Sand	0	33.0
Dawson Fm.	0	15.0

For the liquefied zones, an undrained residual strength was determined from a plot developed by Seed et.al., as reproduced in a report by Marcuson, Hynes and Franklin (1990). This plot (Figure 3) relates the undrained residual strength (in pounds per square foot) to the corrected, equivalent clean sand, SPT blow counts $(N_1)_{60}$.

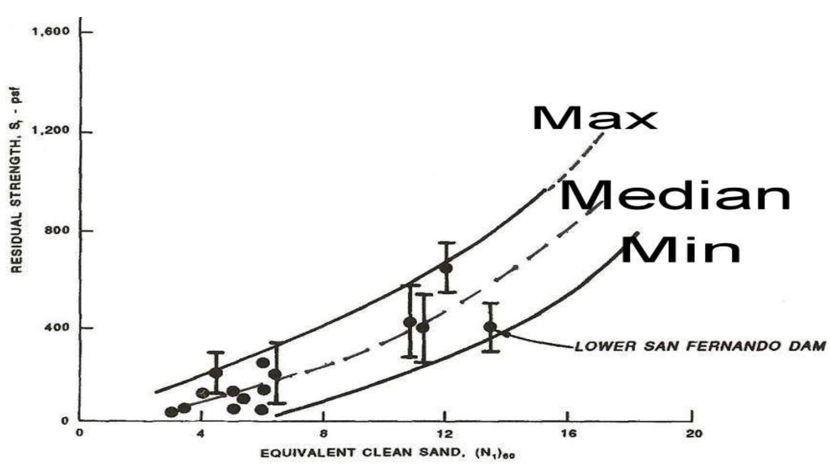


Figure 3: Relationship Between Residual Strength and SPT N-values (Marcuson, Hynes and Franklin, after Seed et.al.)

Corrected SPT values corresponding to Factors of Safety ≤ 1.1 were used to determine the residual shear strength for the liquefied zones. Three residual strength levels were considered:

1. Residual Strength Level 1 (RSL-1). Residual strengths for corrected SPT values were determined from the "Median " line of Figure 3. Individual residual shear strength values were ranked and an adopted design value was selected such that two-thirds (67 percent) of the test values exceed the adopted design value (USACE, 1970).

2. Residual Strength Level 2 (RSL-2). Residual strengths for corrected SPT values were determined from the "Median " line of Figure 3. Individual residual shear strength values were ranked and an adopted design value was selected such that one half (50 percent) of the test values exceed the adopted design value.

3. Residual Strength Level 3 (RSL-3). Residual strengths for corrected SPT values were determined from the "Max" line of Figure 3. Individual residual shear strength values were ranked and an adopted design value was selected such that one half (50 percent) of the test values exceed the adopted design value.

The corrected SPT values, selected residual strengths, and calculation for the adopted design residual strength for each strength level are presented in Appendix B. The adopted residual shear strengths are summarized in Table 2.

Table 2: Adopted Undrained Residual Shear Strengths

Location	**Residual Strength, psf**		
	RSL-1	**RSL-2**	**RSL-3**
Station 95+00			
Upstream	470	480	680
Downstream	540	560	790
Station 57+60			
Upstream	760	880	1120
Downstream	760	880	1120

5.5 Phreatic Surfaces The phreatic surfaces for the maximum valley section (Station 95+00) were obtained from the Liquefaction Assessment (CENWO, 2009). The phreatic surface for the right abutment section (Station 57+60) was obtained from the Seismic Evaluation (CENWO, 1986).

6.0 Results Non-circular, block failure surfaces were analyzed for each upstream or downstream stability case with Spencer's method, utilizing the 2007 version of SLOPE/W, developed by Geo-Slope International, Ltd.

For the re-analysis of the steady state seepage cases, the critical slide plane and peak effective strengths were used, as presented in the Embankment Criteria and Performance Report (CENWO, 1980). Excerpts from this report are presented in Appendix A. The intent of these re-analyses was to compare the current methodology (Spencer's method) with the method used for the original design.

For each post-liquefaction stability case, the sliding elevation was assumed horizontal and the location of the active and passive wedges were iterated until the coordinates corresponding to the critical F.S. were bounded by higher F.S. values. A minimum value of 1.30 was adopted for the post liquefaction factor of safety (CENWO, 2009a).

6.1 Steady State Seepage Cases Two embankment sections were considered in the original design, i.e., Station 95+00 and Station 68+50. Station 95+00 is typical of the valley section from Station 75+00 to Station 95+00. The embankment attains a height of about 117 feet and the foundation sand and clay are about 55 feet thick. Station 68+50 is typical of the right valley section. The embankment is about 131 feet high, with the Dawson Formation at the ground surface under the downstream slope and 30 to 40 feet below the ground surface under the upstream slope.

The results of the re-analysis of the steady seepage cases at Station 95+00 and Station 68+50 are presented on Table 3. These results indicate the factors of safety determined with Spencer's method exceed those factors of safety determined during the original design. Figures C-1 and C-2 in Appendix C present a comparison of the original stability analysis geometry and loading with the graphical output from the Slope/W re-analysis of each case.

Table 3: Steady State Seepage Factors of Safety

Station	Factor of Safety		
	Original	Re-Analysis	
		Janbu's Method	Spencer's Method
95+00	1.43	1.46	1.74
68+50	1.62	1.57	1.84

The difference in the Factor of Safety between the original analyses and the re-analyses using Spencer's method is due to the side force assumptions and statics of each method. The USACE method used in the original design analyses assumes a side force orientation and solves only for horizontal force equilibrium. Spencer's method iteratively determines the side force orientation and is statically determinate. Janbu's Generalized method, which makes assumptions similar to the USACE method, was used to analyze the same Slope/W model. The results, shown in Table 3, substantiate the original design analyses and confirm the difference in factor of safety is due to the analytical method.

6.2 Post-Liquefaction Cases

6.2.1 Station 95+00 The stability analysis results for the maximum valley section are summarized on Table 4. The upstream embankment factors of safety range from 1.18 to 2.38, depending on conditions and assumptions utilized. The downstream embankment factors of safety range from 1.17 to 1.85, depending on conditions and assumptions utilized.

Table 4: Post Earthquake Factors of Safety, Station 95+00

Liquefied Zone Extent	Residual Strength		
	RSL-1	RSL-2	RSL-3
Pool El. 5432			
Upstream Slope	1.18	1.19	1.40
" Toe	1.72	1.74	2.15
Downstream Slope	1.18	1.19	1.33
" Toe	1.68	1.70	1.85
Pool El 5444			
Upstream Slope	1.24	1.25	1.44
" Toe	1.92	1.95	2.38
Downstream Slope	1.17	1.18	1.32
" Toe	1.66	1.67	1.83*

* Further details of this case are presented in Appendix D.

The case representing pool El. 5444, with a liquefied zone limited to the downstream toe and the RSL-3 residual shear strength (F.S. = 1.83) is considered representative of the post-earthquake stability at this station since the overburden confining stress is lowest at the embankment toe. This case is presented on Figure D-1 in Appendix D.

6.3.1 Station 57+60 The stability analysis results for the right abutment section are summarized on Table 5. The upstream embankment factors of safety range from 1.15 to 2.75, depending on conditions and assumptions utilized. The downstream embankment factors of safety range from 1.58 to 3.75, depending on conditions and assumptions utilized.

Table 5: Post Earthquake Factors of Safety, Station 57+60

Liquefied Zone Extent	Residual Strength		
	RSL-1	RSL-2	RSL-3
Pool El. 5432			
Upstream Slope	1.16	1.26	1.44
" Toe	2.18	2.18	2.75
Downstream Slope	1.58	1.67	1.84
" Toe	3.68	3.70	3.75

Pool El 5444			
Upstream Slope	1.15	1.24	1.42
“ Toe	2.10	2.29	2.68*
Downstream Slope	1.58	1.67	1.84
“ Toe	3.62	3.64	3.69

* Further details of this case are presented in Appendix D.

The case representing pool El. 5444, with a liquefied zone limited to the upstream toe and the RSL-3 residual shear strength (F.S. = 2.68) is considered representative of the post earthquake stability at this station since the overburden confining stress is lowest at the embankment toe. This case is presented on Figure D-2 in Appendix D.

7.0 Conclusions The results of this study suggest the following conclusions:

- Cases representing the liquefied zone at the toes of the upstream and downstream slopes are considered the most representative of the field conditions.

- The conservative cases combining the lowest residual strength (RSL-1) and a liquefied zone extending under the entire slope length have a F.S. of nominally 1.2.

- All F.S. determined using the RSL-3 residual strength exceed the minimum F.S. of 1.3, regardless of the horizontal extent of the liquefied zone.

- The combination of the toe liquefied zones and the RSL-3 residual strength best represent the liquefaction potential of the Chatfield Dam foundation. The minimum Factor of Safety for these cases is 1.83 for the Station 95+00 section and 2.68 for the Station 57+60 section.

- None of the critical failure surfaces breach the embankment crest.

8.0 References

CENWO (1980). Embankment Criteria and Performance Report, Chatfield Lake, Colorado, South Platte River Basin. U.S. Army Corps of Engineers, Omaha District, April 1980.

CENWO (1986). Design Memorandum No. PC-44, South Platte River Basin, Chatfield Lake, Colorado, Seismic Evaluation. U.S. Army Corps of Engineers, Omaha District, November 1986.

CENWO (2009a). Liquefaction Assessment, Chatfield Dam & Lake, South Platte River Basin, Littleton, Colorado. U.S. Army Corps of Engineers, Omaha District, April 2009.

CENWO (2009b) Draft Water Supply Re-Allocation Study, Dam Safety Evaluation, Chatfield Dan, Littleton, Colorado. U.S. Army Corps of Engineers, Omaha District.

Marcuson III, W. F, Hynes, M. E. and Franklin A. G. (1990). "Evaluation and Use of Residual Strength in Seismic Safety Analysis of Embankments", Earthquake Spectra, Volume 6, No.3, pp. 529-572.

USACE (1998). Seismic Stability of Earth and Rock Fill Dams (Draft), EM 1110-2-6001. Department of the Army, Corps of Engineers, Office of the Chief of Engineers, 6 May 1998.

USACE (1970). Stability of Earth and Rock-Fill Dams, EM 1110-2-1902, Department of the Army, Corps of Engineers, Office of the Chief of Engineers, 1 April 1970.

Youd, T.L., et.al, (2001). "Liquefaction Resistance of Soils: Summary Report From the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Volume 127, No. 10, pp 817-833.

APPENDIX A

EXCERPTS FROM EMBANKMENT CRITERIA AND PERFORMANCE REPORT

SOUTH PLATTE RIVER BASIN

CHATFIELD LAKE, COLORADO

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

United States Army
Corps of Engineers
...Serving the Army
...Serving the Nation

Omaha District

APRIL 1980

3.11.2.6. Seepage Under or Around Cutoff. Investigations were made to determine whether seepage could be a problem either under or around the cutoff trench. It was found that any seepage that might occur under the cutoff in the sandstones of around the end of the cutoff in the right abutment would be of such a small quantity that it would not be particularly noticeable. Piezometers were installed in the sandstones of the Dawson Formation near the downstream toe of the embankment to monitor seepage pressures that occur in the sandstones.

3.11.2.7. Underground Water Rights. In studies concerning the positive cutoff trench extending to bedrock, it was assumed that the groundwater table immediately downstream of the dam would be adversely affected and may produce serious problems for downstream holders of underground water rights. In connection with this, the Colorado Water Conservation Board was asked to assist in determining the criteria that should be considered for underground water requirements in the area. They concluded that with one or two possible exceptions, an impervious cutoff would have no noticeable effect on the majority of wells in existence and that even if some effect could be demonstrated on the remainder of the wells, the well owners could not prove a legal injury since they were illegally diverting water from the South Platte River. Their conclusion was based on a study area of five miles downstream from the dam, which included 35 wells on record, 18 of which were able to be actually located in the field. Their study indicated that most of the wells were located a distance of a mile or more from the dam site and were supplied largely from irrigation ditches.

3.12. Design Shear Testing.

3.12.1. Remolded Testing. Shear tests were conducted on remolded samples of materials which were representative of materials to be encountered in the required excavations and borrow areas. The remolded samples were tested at 95 percent of maximum density, as determined by AASHO T-99-57A, at optimum and optimum plus 3 percent moisture content. A single test usually consisted of three specimens tested at different confining stresses. A total of 16 unconsolidated undrained "Q" tests; 20 consolidated-undrained "R" tests; and 18 consolidated-drained "S" tests were performed. Samples for "R" tests were saturated by back pressure before shearing. Triaxial specimens were 1.4 inch diameter except for a few at 3.9 inch diameter for coarse grained material. Direct shear tests were 3 in. by 3 in. by 0.5 in. and were sheared about 9.5 inch at a rate of about 0.0038 inch per minute. Test summaries showing adopted strengths are shown on Plates B-55 thru B-57.

3.12.2. Undisturbed Testing. Triaxial ("Q" and "R") and direct shear ("S") tests were performed on selected undisturbed samples of the clays in the valley foundation, and abutments plus the Dawson formation of bedrock. A single test usually consisted of 3 specimens tested at different confining stresses. Size of specimens were similar to that performed for remolded testing. Summaries of the foundation soils testing are shown on Plates B-58 thru B-60, and summaries of the shear tests on the Dawson Formation are on Plates B-61 thru B-69.

The total number of undisturbed tests conducted are as follows:

No. of Undisturbed Tests

Type of Test	No. of Shear Tests Foundation Soil	Foundation Bedrock
Q	18	11
R	15	4
S	14	19

3.12.3. Residual Testing. A total of 28 direct shear residual tests were performed on undisturbed Denison samples and on specimens obtained from cubic foot box samples of the Dawson Formation materials. Discussion and results are given in paragraph 3.12.4.3. The test summaries are shown on Plates B-70 and B-71.

3.12.4. Test Results.

3.12.4.1. Embankment Materials. Most of the remolded testing was concentrated on the weaker materials from each area of excavation. The impervious and random materials were chosen to have identical strengths and the final adopted strength was based primarily on the lowest strength of all the material types tested. Additionally, since the majority of the material would come from excavations where the moisture content was from 2 to 6 percent dry of optimum, the final adopted shear strengths were based on optimum moisture content specimen testing.

It was determined that "Q" tests conducted at optimum moisture were about twice the strength of those conducted at optimum plus 3 percent.

3.12.4.2. Foundation Soils. The majority of tests performed on foundation materials were on the weaker surface clays in the valley. Results indicated material of relatively low shear strength which ultimately was required to be removed to satisfy embankment stability requirements.

Results of testing indicated considerable ranges in shear strength which gave evidence to the heterogeneous nature of the foundation soils in general. Adopted strengths were based on the lowest test strengths obtained.

3.12.4.3. Bedrock. Tests on the Dawson Formation materials was generally confined to the clay-shale material which was thought to be weaker than the silty or sandy shales, sandstones and siltstones. Specific concentration was made of the more weathered bedrock. Additionally, since the Dawson Formation contains numerous slickensides of all sizes and degrees, the testing and correlation concerned this effect upon the strength. Results of the tests indicated that the slickensides in the bedrock appeared to have little effect on the strength while the affects of weathering appeared to give lower strengths.

Stress-strain curves on direct-shear tests of the Dawson Formation exhibited sharp peaks and substantial differences between peak strengths and strength at the end of the strain, commonly referred to as ultimate strength. Because of the relatively sharp drop in strength with strain and the slickensided nature of the Formation, it was thought that slight additional amounts of horizontal movement may lower the strength toward a residual strength which could be considerably lower than the ultimate strength. As a consequence, a residual testing program was developed. Tests were performed on precut samples both by MRD and SWD laboratories in direct shear boxes having a size of 3"x3"x1". Total displacements ranged from 5 to 11.6 inches with residual strength usually being reached at from 4.4 to 11 inches. The range in residual strengths ranged from $\emptyset = 5°$ to $\emptyset = 32°$.

The final adopted strength of the Dawson Formation "S" strength was a practical value selected as a reasonable and conservative strength for use in stability computations. The value $\emptyset = 15°$ is approximately midway between a peak value of $\emptyset = 24°$ and a residual value of $\emptyset = 8°$.

A range of direct shear strengths by type of material for the Dawson Formation is given below and is also shown on Plate B-69:

Type of Material	Range of Direct Shear Strength	
	Peak	Ultimate
Clay shale	19° to 27°	12° to 23°
Silty shale & Siltstone	27° to 38°	23° to 33°
Sandy shale & Sandstone	38° to 42°	33° to 37°

A summary of residual shear strength test data and mineral composition of the Dawson Formation are shown on Plate B-71.

3.12.5. Summary of Adopted Design Strengths. The adopted shear strengths of the embankment and foundation materials which have been used in stability analyses computations are as follows:

Material	Unconsolidated-Undrained "Q" Strength			Consolidated-Undrained "R" Strength			Consolidated Drained "S" Strength		
	tan Ø	Ø	coh-T/SF	tan Ø	Ø	coh-T/SF	tan Ø	Ø	coh-T/SF
Embankment									
Impervious	0.042	2.4°	1.5	0.17	9.7°	0.50	0.45	24.3°	0.0
Random	0.042	2.4°	1.5	0.17	9.7°	0.50	0.45	24.3°	0.0
Pervious	0.73	36.1°	0.0	0.73	36.1°	0.00	0.73	36.1°	0.0
Dawson Fm.	0.00	0.0°	2.0	0.20	11.3°	0.40	0.41	22.3°	0.0
Foundation									
Clay	0.00	0.0°	0.28	0.15	8.5°	0.30	0.38	20.8°	0.0
Sand	0.65	33.0°	0.00	0.65	33.0°	0.00	0.65	33.0°	0.0
Dawson Fm.	0.00	0.0°	2.70	0.35	19.3°	0.40	0.27*	15.0°*	0.0*

*Value used in lieu of "S" strength.

3.12.6. Additional Shear Testing of Dawson Fm. A post-design testing program was done for soft seam materials of the Dawson Formation. The results of the testing was reported in a Supplement to Design Memorandum No. PC-24, December 1970. The testing was a result of concerns voiced during a Board of Consultants meeting and the objective was to ascertain whether shear strengths of soft seam material might be lower than shear strengths previously obtained for the Dawson Formation. Testing was also done to investigate consolidation and mineralogical characteristics of the seam. Samples were obtained from box samples which were cut from test pits 8 and 9 located in the outlet works area. In addition, some specimens were cut directly into shear boxes. Photos 8 and 9, Plate A-8, show typical undisturbed sampling operations.

The testing program consisted of twelve (12) residual shear tests; and eleven (11) direct shear tests on the soft seam material. Only three of the tests were "precut" residuals similar to that done previously for the Dawson Formation.

Results of the tests indicated that both the residual and normal direct shear strengths of the soft seam materials were within the range of their respective strengths previously obtained for the Dawson Formation. Plate B-72 shows summary results of the testing.

Atterberg Limits and moisture content tests of the soft seam material was compared to the material above and below the seam and the following was found:

(1) The moisture content of the seam material was slightly higher than the surrounding shale.
(2) There was no appreciable difference between Atterberg Limits for the seam material and surrounding shale.
(3) Moisture contents of the seam material exceeded the plastic limit more consistently than the weathered shale or the shale above and below the seam.

Mineralogical tests on the shale and soft seam material indicated that percentages of various clay minerals in the seam material do not differ significantly from that found in the shale material above and below the seam. The dominant absorbed ion was the calcium ion which was also found to be dominant in previous tests on clay shale materials. The results of the mineralogical tests were highly supportive of the conclusion that the seam material is a gouge or fracture type of material and not a separately deposited material, such as bentonite.

3.13. Record Shear Tests. Twenty two undisturbed cubic-foot box samples were taken on the embankment materials by the Stage III Earthwork Contractor during the contract period. Plates C-1 thru C-6, Appendix "C", show the locations of the box samples and the test results compared to design shear strengths.

The total number of record shear test series on the different materials are presented below:

No. of Record Shear Tests

Type of Test	Embankment Materials: Imp & Random	Embankment Materials: Dawson Fm.
"Q"	12	3
"R"	9	2
"S"	9	2

3.14. Embankment Stability Analyses.

3.14.1. General. Stability analyses were performed for three embankment sections: (1) embankment section at station 95+00 where the embankment attains a maximum height of 137 feet and the depth of the alluvial material is about 55 feet. (2) The outlet works section, where the embankment attains a height of about 117 and alluvial material is 25 feet deep; and (3) the right valley embankment section at station 68+50 where the embankment is about 131 feet high but where the Dawson Formation is at the ground surface for the downstream portion of the section and 30 to 40 feet below the surface under the upstream portion of the section. Analyses consisted of four types of cases which simulate conditions of stress during the life of the structure. The cases are: (1) end of construction; (2) sudden drawdown; (3) partial pool and (4) steady seepage.

3.14.2. Method of Analysis. The sliding wedge method was used for the stability analyses. The factor of safety in the analyses is defined as the ratio of the available shear strength to the average necessary to maintain equilibrium. Most of the studies were performed with an RCA 301 computer using slope stability program 41-R3-1302C, "Slope Stability, Wedge Method."

Most of the analyses were performed in the conventional manner; however, some special cases were performed using at rest pressure conditions for driving forces when failure was assumed in the Dawson Formation. This was done as a means to account for strain incompatibility between the brittle Dawson Formation and the embankment materials. The earth pressure coefficient used for computing the driving forces was 0.5.

3.14.3. Seismic Coefficient. The stability analyses include an allowance for earthquake forces for all potential failure surfaces and cases studied except sudden drawdown from maximum pool. This was done by the addition of a horizontally directed static force in the computations with no change in strengths. The additional earthquake force is the product of a seismic coefficient and the weight of the sliding mass. The coefficient assumed for the analyses was 0.1.

3.14.4. Summary of Results. The following table summarizes the stability analyses results and Plates B-73 thru B-78 summarize each analyses.

CASE	ELEV. OF FAILURE PLANE	NORMAL FACTOR OF SAFETY	REQUIRED FACTOR OF SAFETY	EARTHQUAKE FACTOR OF SAFETY	REQUIRED EARTHQUAKE FACTOR OF SAFETY	SHEAR STRENGTH
END OF CONSTRUCTION						
Upstream-Sta 95+00	5410	2.62	1.3	1.82	1.0	"Q"
Downstream-Sta 95+00	5335	2.53	1.3	1.62	1.0	"Q"
PARTIAL POOL						
Sta 95+00	5410	1.49	1.5	1.04	1.0	"R"
Sta 104+35	5353	1.46	1.5	0.90	1.0	*
SUDDEN DRAWDOWN						
Max. Pool-Sta 95+00	5410	1.23	1.0	-	Not Req'd	"R"
Spillway Pool-Sta 95+00	5410	1.33	1.20	0.92	1.0	"R"
STEADY SEEPAGE						
Sta 95+00 (Conventional)	5300	1.43	1.5	0.86	1.0	*
Sta 95+00 (At-rest pressure)	5335	1.13	-	-	-	*
Sta 104+35(Conventional)	5320	1.58	1.5	0.92	1.0	*
Sta 68+50	5320	1.62	1.5	0.94	1.0	*

* The "S" strength was used for all materials except the Dawson Formation. A lower strength was adopted for the Dawson Formation in lieu of an "S" Strength.

APPENDIX B

DETERMINATION OF RESIDUAL SHEAR STRENGTH

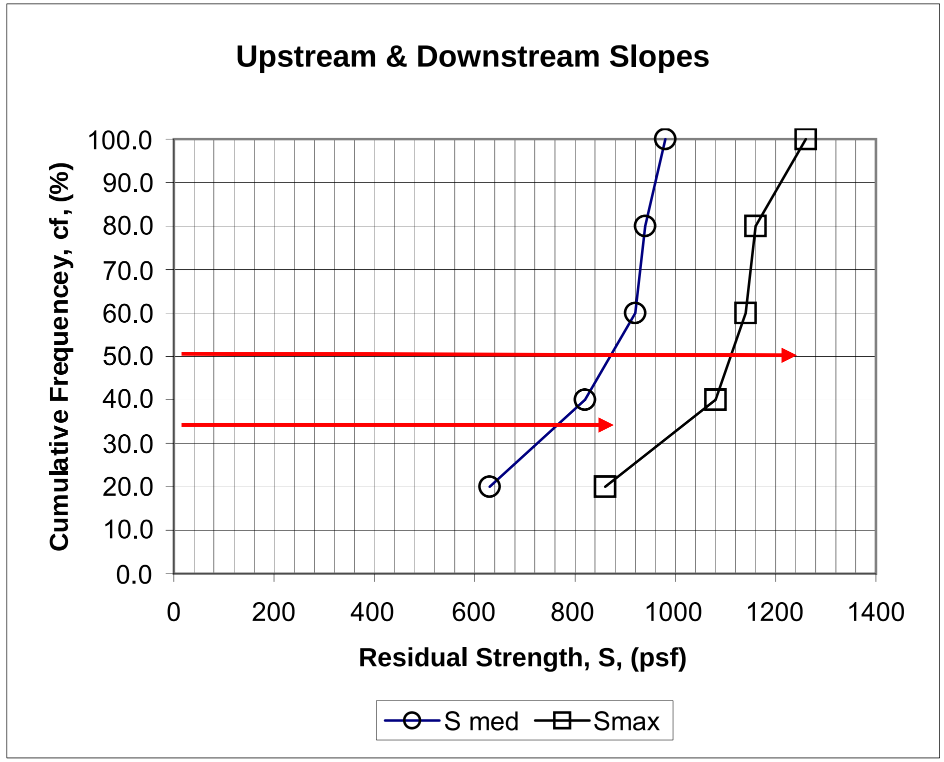
CHATFFIELD DAM
POST-LIQUEFACTION SHEAR STRENGTH

Station 57+60					
Upstream & Downstream					
$(N1)_{60}$	S_{med}	S_{max}	n	$\sum n$	cf
(bpf)	(psf)	(psf)	(-)	(-)	(%)
14.0	630	860	1	1	20.0
16.3	820	1080	1	2	40.0
16.5	920	1140	1	3	60.0
16.6	940	1160	1	4	80.0
17.6	980	1260	1	5	100.0

Notes:

1. $(N1)_{60}$ data from Boring SPT-12 for F.S. < 1.1
2. S values obtained from Seed et al. plot

Adopted Residual Shear Strength, (psf)			
Location	**RSL-1**	**RSL-2**	**RSL-3**
Upstream	760	880	1120
Downstream	760	880	1120



CHATFFIELD DAM
POST-LIQUEFACTION SHEAR STRENGTH

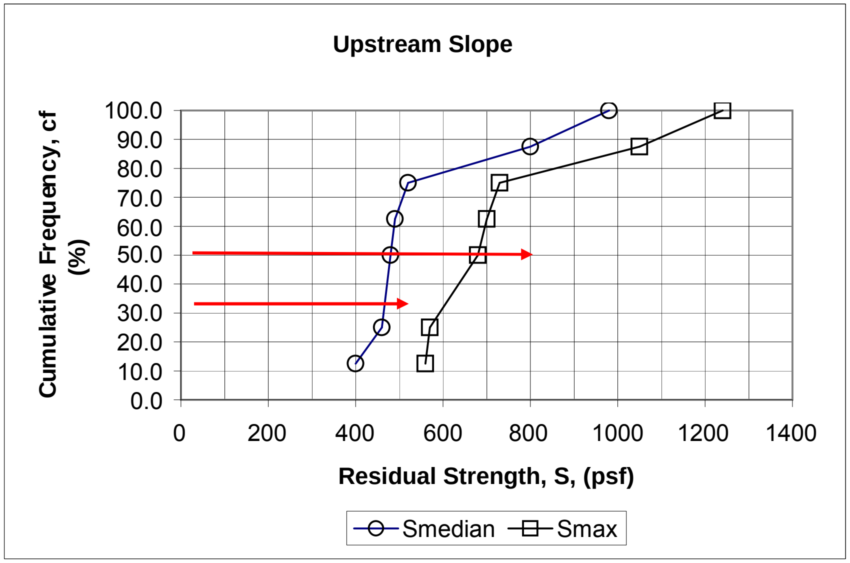
Station 95+00					
Upstream					
$(N1)_{60}$	S_{median}	S_{max}	n	$\sum n$	cf
(bpf)	(psf)	(psf)	(-)	(-)	(%)
10.1	400	560	1	1	12.5
10.3	460	570	1	2	25.0
11.8	480	680	2	4	50.0
12.0	490	700	1	5	62.5
12.4	520	730	1	6	75.0
16.0	800	1050	1	7	87.5
17.6	980	1240	1	8	100.0
Downstream					
11.7	460	680	1	1	11.1
11.8	480	690	1	2	22.2
12.8	540	760	1	3	33.3
13.0	550	780	1	4	44.4
13.2	570	800	1	5	55.6
13.8	610	850	1	6	66.7
14.3	650	890	1	7	77.8
14.4	660	900	1	8	88.9
14.6	680	910	1	9	100.0

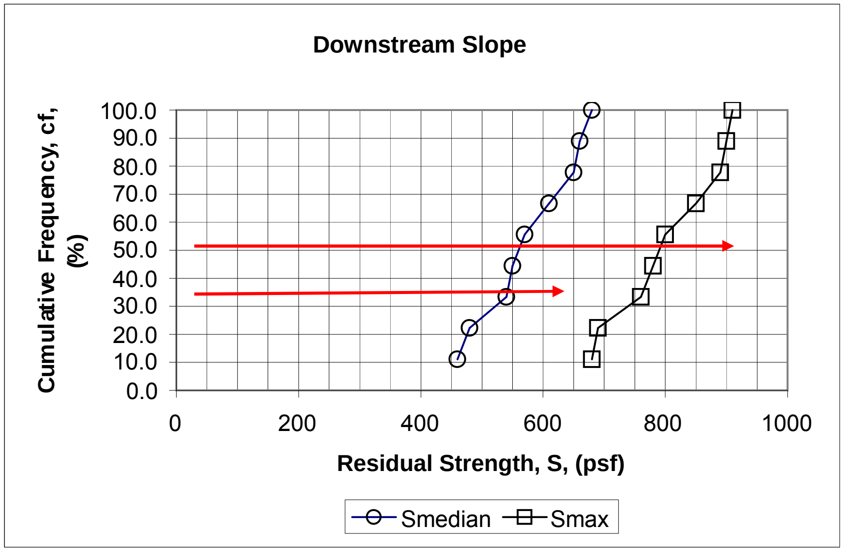
Notes:

1 Upstream $(N1)_{60}$ data from Borings SPT-10 & 14 for F.S. < 1.1

2 Downstream $(N1)_{60}$ data from Boring SPT 08-03 for F.S. < 1.1

3 S values obtained from Seed et.al. plot





Adopted Residual Shear Strength. (psf)			
Location	**RSL-1**	**RSL-2**	**RSL-3**
Upstream	470	480	680
Downstream	540	560	790

APPENDIX C

STEADY-STATE SEEPAGE STABILITY ANALYSES

CHATFIELD DAM
Steady-Seepage Stability Re-Analysis

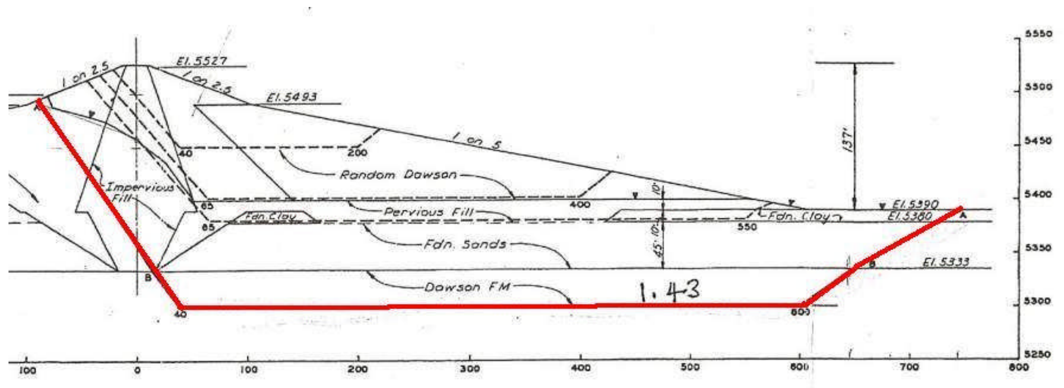


Figure C-1a: Original Stability Analysis from "Embankment Criteria and Performance Report," April, 1980.

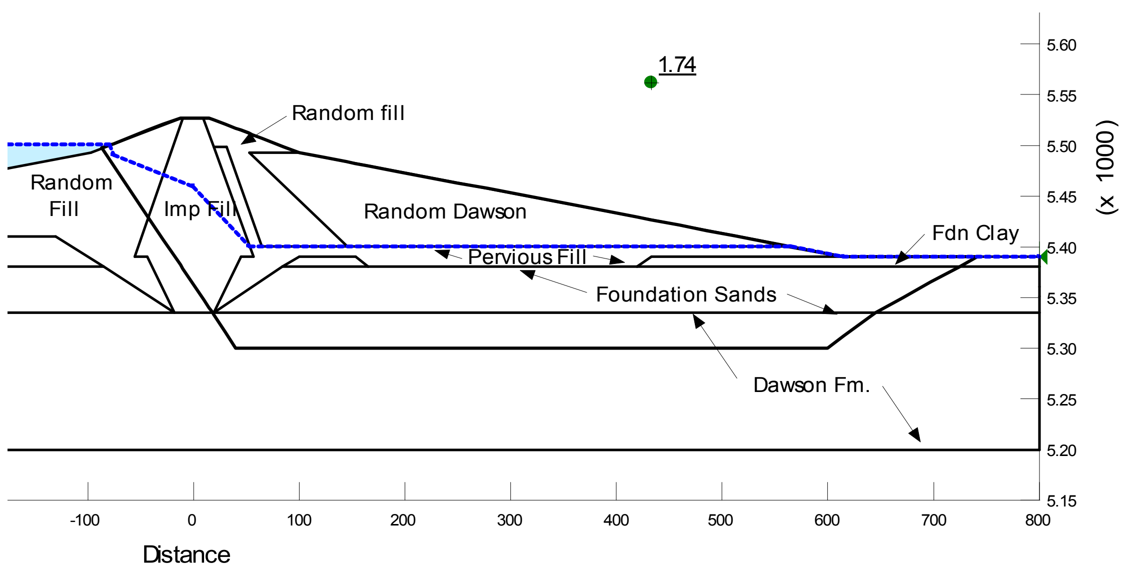


Figure C-1b: Re-Analysis Output from Slope/W

Figure C-1: Steady-Seepage Re-Analysis, Station 95+00

Station 95+00, Steady Seepage, Pool 5500

File Information

Title: Chatfield Dam, Post Liquefaction Stability Assessment
Comments:
Created By: Stacey, Teryl L NWO
Revision Number: 45
Last Edited By: R.L.Donovan
Date: 09/04/2009
Time: 10:32:50 AM
File Name: Chatfield Sta 95 Steady Seepage.gsz
Directory: F:\Projects\Other\Chatfield Dam\
Last Solved Date: 09/04/2009
Last Solved Time: 10:32:56 AM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Station 95+00 Pool 5500

Description: See Plate B-73 for referenced data.
Kind: SLOPE/W
Method: Spencer
Settings
- Apply Phreatic Correction: No
- PWP Conditions Source: Piezometric Line
- Use Staged Rapid Drawdown: No

SlipSurface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Fully-Specified

Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No
FOS Distribution
FOS Calculation Option: Constant
Advanced
Number of Slices: 30
Optimization Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Embankment Impervious

Model: Bilinear
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi 1: 24.3 °
Phi 2: 17.2 °
Bilinear Normal: 3500 psf
Pore Water Pressure
Piezometric Line: 1

Embankment Dawson

Model: Bilinear
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi 1: 22.3 °
Phi 2: 17 °
Bilinear Normal: 3700 psf
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Embankment Random

Model: Bilinear
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi 1: 24.3 °
Phi 2: 17.2 °
Bilinear Normal: 3500 psf

Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Embankment Pervious
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 36.1 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Foundation Clay
Model: Bilinear
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi 1: 20.8 °
Phi 2: 14.8 °
Bilinear Normal: 2500 psf
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Foundation Sand
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 33 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Foundation Dawson
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 15 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Limits
Left Coordinate: (-800, 5390) ft
Right Coordinate: (800, 5390.0789) ft

Fully Specified Slip Surfaces

Fully Specified Slip Surface 1

	X (ft)	Y (ft)
	-88	5499
	18	5335
	40	5300
	600	5300
	645	5335
	745	5392

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	-800	5499.9853
	-79.5642	5500.1743
	-75.8814	5490.203
	0.3135	5460.2447
	52.5913	5400.1504
	565	5400
	615	5390
	800	5390.0789

Critical Slip Surfaces

	Slip Surface	FOS	Center (ft)	Radius (ft)	Entry (ft)	Exit (ft)
1	1	1.70	(337.761, 5524.05)	378.06	(-86.8702, 5497.25)	(741.586, 5390.05)

Slices of Slip Surface: 1

	Slip Surface	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)

1	1	-83.21718	5491.6	534.96891	726.42301	86.444845	0
2	1	-77.7228	5483.099	754.36791	1540.7205	355.05183	0
3	1	-59.36778	5454.701	1810.2308	4522.2899	1224.5416	0
4	1	-35.72581	5418.123	3512.5875	8561.9653	1563.0435	496.88
5	1	-20.54873	5394.6415	4605.6485	11196.467	2040.1992	496.88
6	1	-11.25	5380.255	5274.9709	12769.451	2319.9291	496.88
7	1	-4.84325	5370.3425	5736.4305	13669.747	2455.7717	496.88
8	1	5.15675	5354.8705	6227.6683	15034.834	2726.2733	496.88
9	1	12.5	5343.5095	6410.1298	16005.783	2970.3508	496.88
10	1	16.5	5337.321	6509.3953	16484.376	3087.7725	496.88
11	1	18.53	5334.157	6561.2733	17123.372	2830.1058	0
12	1	24.53	5324.6115	6726.5708	18001.409	3021.0837	0
13	1	35	5307.9545	7015.0865	19452.47	3332.5868	0
14	1	42.5	5300	6973.2	27434	5482.4548	0
15	1	48.79565	5300	6521.6761	27171.894	5533.2092	0
16	1	54.29565	5300	6249.3033	26896.764	5532.4705	0
17	1	60.5	5300	6249.2222	26443.333	5410.9958	0
18	1	75.25	5300	6248.7805	25539.024	5168.8053	0
19	1	92.75	5300	6248.6207	24588.276	4914.0958	0
20	1	111.34435	5300	6248.4904	23892.091	4727.5884	0
21	1	133.78305	5300	6247.7804	23342.533	4580.5252	0
22	1	148.9387	5300	6247.5916	22971.567	4481.1757	0
23	1	158.75	5300	6247.4783	22792.174	4433.1378	0
24	1	178.66665	5300	6247.0596	22366.238	4319.1208	0
25	1	207	5300	6246.7066	21672.355	4133.2902	0
26	1	235.33335	5300	6246.0007	20978.473	3947.5541	0
27	1	263.66665	5300	6245.6478	20284.944	3761.818	0
28	1	292	5300	6244.9419	19591.061	3576.0819	0
29	1	320.33335	5300	6244.589	18897.532	3390.3458	0
30	1	348.66665	5300	6243.8831	18203.649	3204.6097	0
31	1	377	5300	6243.5301	17510.12	3018.8736	0
32	1	405.33335	5300	6242.8243	16816.237	2833.1375	0
33	1	426.25	5300	6242.5185	16242.963	2679.611	0
34	1	446.2	5300	6242.0455	15693.182	2532.4244	0
35	1	472.6	5300	6241.6667	15046.97	2359.3738	0
36	1	499	5300	6241.2879	14400.379	2186.2218	0

37	1	525.4	5300	6240.9091	13754.167	2013.1713	0
38	1	551.8	5300	6240.1515	13107.576	1840.1208	0
39	1	582.5	5300	6021.7143	12324.857	1688.922	0
40	1	607.5	5305.8335	5345.4943	13202.182	2105.193	0
41	1	630	5323.3335	4160.4121	10174.223	1611.3958	0
42	1	658.1579	5342.5	2965.1433	7669.3734	3054.9628	0
43	1	684.4737	5357.5	2029.837	5216.7982	2069.6368	0
44	1	710.7895	5372.5	1094.5636	2764.0579	1084.1823	0
45	1	732.76665	5385.027	313.44999	703.89445	148.31593	0

CHATFIELD DAM
Steady-Seepage Stability Re-Analysis

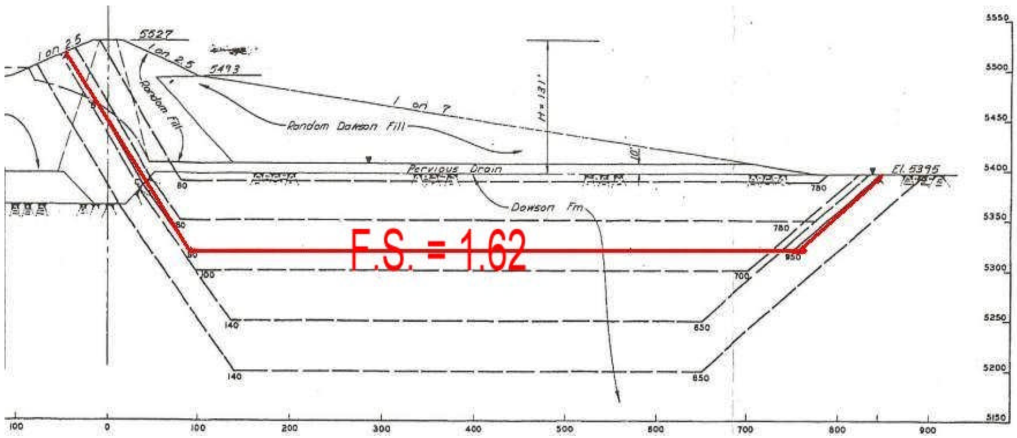


Figure C-2a: Original Stability Analysis from "Embankment Criteria and Performance Report," April, 1980.

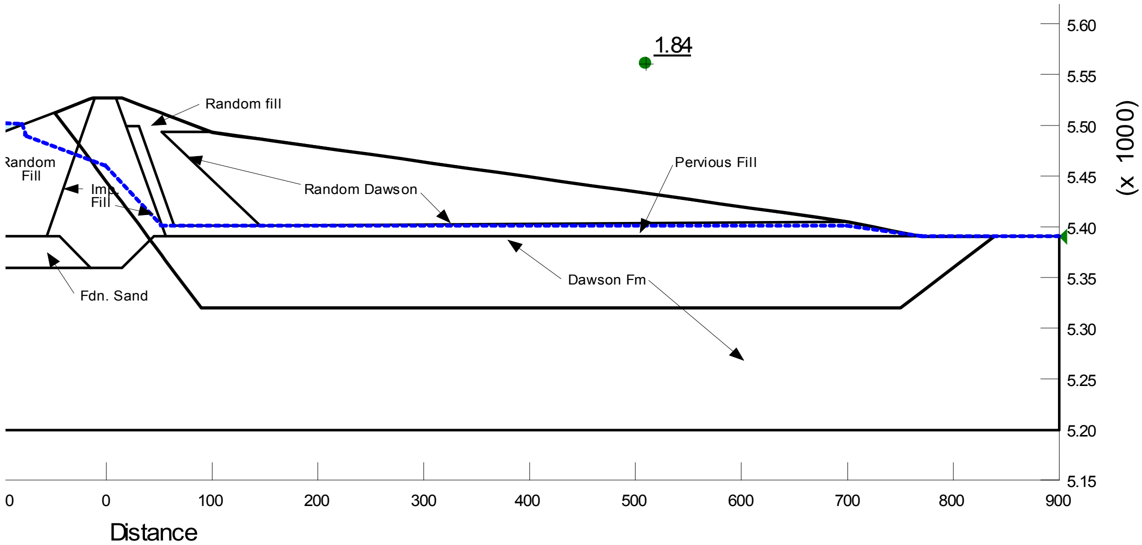


Figure C-2b: Re-Analysis Output from Slope/W

Figure C-2: Steady-Seepage Re-Analysis, Station 68+50

Station 68+50, Steady Seepage, Pool El. 5500

File Information

Title: Chatfield Dam, Post Liquefaction Stability Assessment.
Comments:
Created By: R.L. Donovan
Revision Number: 50
Last Edited By: R. L. Donovan
Date: 09/08/2009
Time: 10:10:16 AM
File Name: Chatfield Sta 68+50 Steady Seepage.gsz
Directory: F:\Projects\Other\Chatfield Dam\
Last Solved Date: 09/08/2009
Last Solved Time: 10:10:22 AM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Station 68+50 Pool 5500

Description: See Plate B-6 of Draft Water Supply Re-Allocation study.
Kind: SLOPE/W
Method: Spencer
Settings
- Apply Phreatic Correction: No
- PWP Conditions Source: Piezometric Line
- Use Staged Rapid Drawdown: No

SlipSurface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Fully-Specified

Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No

FOS Distribution

FOS Calculation Option: Constant

Advanced

Number of Slices: 30
Optimization Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Embankment Impervious

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 24.3 °
Pore Water Pressure

Piezometric Line: 1

Embankment Dawson

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 22.3 °
Phi-B: 0 °
Pore Water Pressure

Piezometric Line: 1

Embankment Random

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 24.3 °
Phi-B: 0 °
Pore Water Pressure

Piezometric Line: 1

Embankment Pervious

Model: Mohr-Coulomb

Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 36.1 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Foundation Sand

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 33 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Foundation Dawson

Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 15 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Limits

Left Coordinate: (-800, 5390) ft
Right Coordinate: (900, 5390.0789) ft

Fully Specified Slip Surfaces

Fully Specified Slip Surface 1

	X (ft)	Y (ft)
	-50	5514
	90	5320
	750	5320
	845	5395

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	-800	5499.9853
	-79.5642	5500.1743
	-75.8814	5490.203
	0.3135	5460.2447
	52.5913	5400.1504
	700	5400
	770	5390
	900	5390.0789

Critical Slip Surfaces

	Slip Surface	FOS	Center (ft)	Radius (ft)	Entry (ft)	Exit (ft)
1	1	1.84	(407.58, 5543.05)	396.191	(-48.88, 5512.45)	(838.72, 5390.04)

Slices of Slip Surface: 1

	Slip Surface	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
1	1	-37.280165	5496.374	-1332.1226	1666.7171	752.55163	0
2	1	-20.725875	5473.4345	-306.85143	4045.3081	1826.5266	0
3	1	-14.13571	5464.3025	101.30585	5013.0599	2217.742	0
4	1	-11.25	5460.3035	280.02287	5424.4337	2322.7906	0
5	1	-4.84325	5451.4255	676.84842	6220.9457	2503.2559	0
6	1	5.15675	5437.5685	1067.6088	7416.2321	2866.5133	0
7	1	12.5	5427.393	1175.8714	8257.5499	3197.5005	0
8	1	17.03	5421.116	1242.5819	8711.1899	3372.2058	0
9	1	24.53	5410.723	1353.1476	9437.8529	3650.3844	0
10	1	35.898205	5394.9695	1520.6556	10488.426	4049.1038	0
11	1	43.398205	5384.5765	1631.2195	12009.697	2780.9045	0

12	1	48.79565	5377.0975	1710.7767	12558.904	2906.7468	0
13	1	54.29565	5369.476	1913.9937	13123.106	3003.4726	0
14	1	60.5	5360.8785	2450.4315	13703.093	3015.1414	0
15	1	77.5	5337.3215	3920.2734	15449.36	3089.2095	0
16	1	95	5320	5000.8	22129	4589.4874	0
17	1	111.21795	5320	5000.4747	21683.581	4470.2248	0
18	1	133.6538	5320	5000.029	21280.655	4362.3805	0
19	1	151.9117	5320	4999.9254	20953.82	4274.8333	0
20	1	173.98085	5320	4999.6274	20559.753	4169.3232	0
21	1	204.0391	5320	4999.2947	20023.128	4025.6241	0
22	1	234.0973	5320	4998.6293	19486.504	3882.0142	0
23	1	264.15555	5320	4998.2966	18949.879	3738.3151	0
24	1	294.2138	5320	4997.964	18413.254	3594.6161	0
25	1	324.27205	5320	4997.2986	17876.629	3451.0061	0
26	1	354.3303	5320	4996.9659	17340.004	3307.3071	0
27	1	384.3885	5320	4996.6332	16803.379	3163.608	0
28	1	414.44675	5320	4996.3005	16266.754	3019.9089	0
29	1	444.505	5320	4995.6351	15730.129	2876.299	0
30	1	474.5632	5320	4995.3025	15193.504	2732.6	0
31	1	504.62145	5320	4994.9698	14656.879	2588.9009	0
32	1	534.6797	5320	4994.3044	14120.255	2445.291	0
33	1	564.73795	5320	4993.9717	13583.63	2301.5919	0
34	1	594.7962	5320	4993.639	13047.005	2157.8928	0
35	1	624.8544	5320	4992.9736	12510.38	2014.2829	0
36	1	654.91265	5320	4992.641	11973.755	1870.5839	0
37	1	684.9709	5320	4992.3083	11437.13	1726.8848	0
38	1	712.5	5320	4880.4	10816	1590.4392	0
39	1	737.5	5320	4657.6	10110.4	1461.0734	0
40	1	760	5327.8945	3964.4432	10272.939	1690.3564	0
41	1	787.17985	5349.3525	2536.9533	6496.1171	1060.8548	0
42	1	821.5396	5376.479	845.67393	2165.3419	353.60397	0

APPENDIX D

POST-LIQUEFACTION STABILITY ANALYSES

Table D-1: Adopted Shear Strengths

Material	Cohesion (psf)	Friction angle (degrees)
Embankment		
Impervious	0	24.3
Random	0	24.3
Pervious	0	36.1
Dawson Fm.	0	22.3
Foundation		
Clay	0	20.8
Sand	0	33.0
Dawson Fm.	0	15.0
Liquefied Zones		
Upstream	680	0
Downstream	790	0

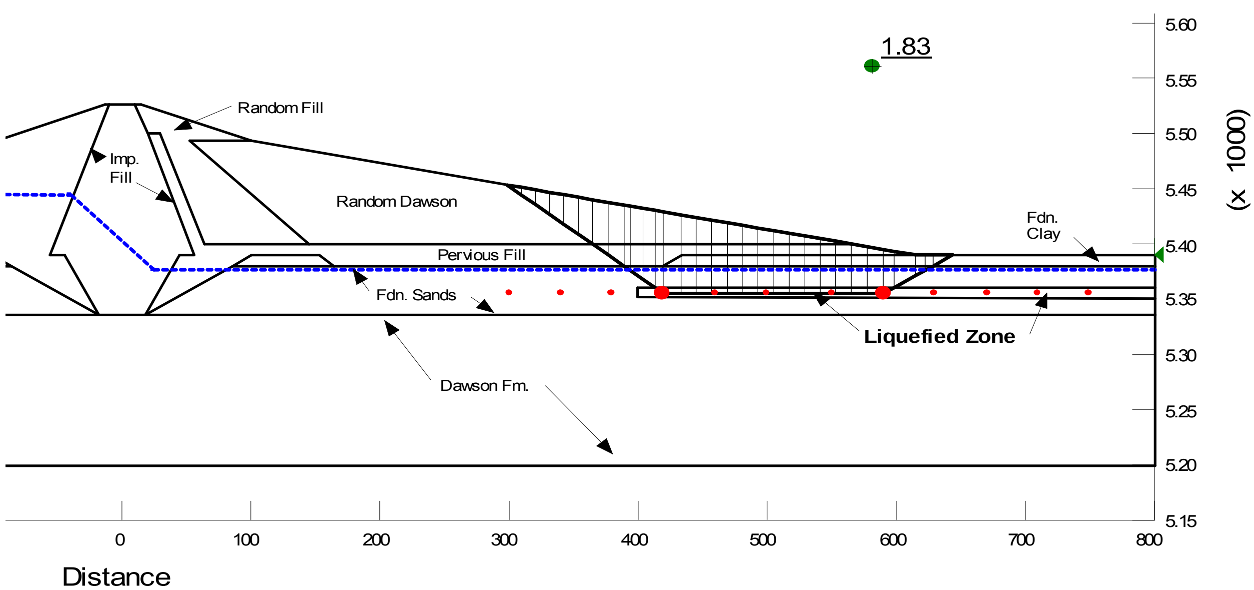


Figure D-1: Post-Liquefaction Slope Stability at Downstream Toe

Figure D-1: Sta 95+00, Pool El. 5444, Downstream Toe

Station 95+00 Pool El. 5444 Downstream Toe

File Information

Title: Chatfield Dam, Post-Liquefaction Stability Assessment.
Comments:
Created By:
Revision Number: 49
Last Edited By: R.L. Donovan
Date: 09/01/2009
Time: 10:57:48 AM
File Name: Chatfield Sta 95 Pool 5444 - Min Liq - Max S - DS.gsz
Directory: F:\Projects\Other\Chatfield Dam\
Last Solved Date: 09/01/2009
Last Solved Time: 10:57:58 AM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Station 95+00 Pool 5444

Description: See Plate B-73 for referenced data.
Kind: SLOPE/W
Method: Spencer
Settings
- Apply Phreatic Correction: No
- PWP Conditions Source: Piezometric Line
- Use Staged Rapid Drawdown: No

SlipSurface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Block

Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No
FOS Distribution
FOS Calculation Option: Constant
Restrict Block Crossing: No
Advanced
Number of Slices: 30
Optimization Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Embankment Impervious

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 24.3 °
Pore Water Pressure
Piezometric Line: 1

Embankment Dawson

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 22.3 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Embankment Random

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 24.3 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Embankment Pervious

Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 36.1 °
Phi-B: 0 °
Pore Water Pressure
    Piezometric Line: 1

Foundation Clay

Model: Mohr-Coulomb
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 20.8 °
Phi-B: 0 °
Pore Water Pressure
    Piezometric Line: 1

Foundation Sand

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 33 °
Phi-B: 0 °
Pore Water Pressure
    Piezometric Line: 1

Foundation Dawson

Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 15 °
Phi-B: 0 °
Pore Water Pressure
    Piezometric Line: 1

Liquefied U.S. Foundation Sand

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 680 psf
Phi: 0 °
Phi-B: 0 °
Pore Water Pressure
    Piezometric Line: 1

Liquefied D.S. Foundation Sand

Model: Mohr-Coulomb

Unit Weight: 125 pcf
Cohesion: 790 psf
Phi: 0 °
Phi-B: 0 °
Pore Water Pressure
    Piezometric Line: 1

Slip Surface Limits

Left Coordinate: (-800, 5390) ft
Right Coordinate: (800, 5390.0789) ft

Slip Surface Block

Left Grid
- Upper Left: (300, 5355) ft
- Lower Left: (300, 5355) ft
- Lower Right: (500, 5355) ft
- X Increments: 5
- Y Increments: 0
- Starting Angle: 135 °
- Ending Angle: 147 °
- Angle Increments: 2

Right Grid
- Upper Left: (550, 5355) ft
- Lower Left: (550, 5355) ft
- Lower Right: (750, 5355) ft
- X Increments: 5
- Y Increments: 0
- Starting Angle: 33 °
- Ending Angle: 45 °
- Angle Increments: 3

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	-800	5444
	-40	5444
	25	5376
	800	5376

Critical Slip Surfaces

	Slip Surface	FOS	Center (ft)	Radius (ft)	Entry (ft)	Exit (ft)
1	245	1.83	(479.972, 5469.09)	159.547	(298.646, 5453.27)	(643.914, 5390.01)

Slices of Slip Surface: 245

	Slip Surface	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
1	245	304.1276	5448.8315	-4544.7103	320.35611	131.38762	0
2	245	315.0916	5439.953	-3990.7003	961.0825	394.16866	0
3	245	326.0556	5431.0745	-3436.6904	1601.7805	656.93808	0
4	245	337.0196	5422.196	-2882.6096	2242.5495	919.73656	0
5	245	347.9836	5413.3175	-2328.5996	2883.2475	1182.506	0
6	245	358.9476	5404.439	-1774.5897	3523.9455	1445.2754	0
7	245	370.6041	5395	-1185.5765	3873.4034	2824.5344	0
8	245	382.9531	5385	-561.59913	4604.7365	3357.8316	0
9	245	391.5974	5378	-124.80053	5223.8312	3392.3956	0
10	245	398.8723	5372.109	242.80206	5686.5415	3535.2057	0
11	245	408.4825	5364.327	728.41427	6327.8948	3636.3452	0
12	245	416.3938	5357.9205	1128.1772	7860.5251	0	790
13	245	426.25	5355.2025	1297.7868	8974.4833	0	790
14	245	439	5355	1310.4167	8680.8333	0	790
15	245	451	5355	1310.4167	8393.3333	0	790
16	245	463	5355	1310.4167	8105.1667	0	790
17	245	475	5355	1310.4167	7817.1667	0	790
18	245	487	5355	1310.4167	7529.1667	0	790
19	245	499	5355	1310.4167	7241.1667	0	790
20	245	511	5355	1310.4167	6953.1667	0	790
21	245	523	5355	1310.4167	6665.1667	0	790
22	245	535	5355	1310.4167	6377.1667	0	790
23	245	547	5355	1310.4167	6089.1667	0	790
24	245	559	5355	1310.4167	5801.1667	0	790
25	245	571.25	5355	1310.4	5494.64	0	790
26	245	583.75	5355	1310.4	5169.68	0	790

27	245	594.2483	5357.759	1138.2794	5243.8634	0	790
28	245	606.7483	5365.8765	631.7207	4610.5295	2583.8687	0
29	245	618.6686	5373.6175	148.66438	2990.2047	1845.3178	0
30	245	625.4169	5378	-124.79995	2207.9666	1433.8703	0
31	245	636.20545	5385.006	-561.97347	775.20898	294.47427	0

CHATFIELD DAM
Post Liquefaction Stability Assessment

Table D-2: Adopted Shear Strengths

Material	Cohesion (psf)	Friction angle (degrees)
Embankment		
Impervious	0	24.3
Random	0	24.3
Pervious	0	36.1
Dawson Fm.	0	22.3
Foundation		
Clay	0	20.8
Sand	0	33.0
Dawson Fm.	0	15.0
Liquefied Zones		
Upstream	1120	0
Downstream	1120	0

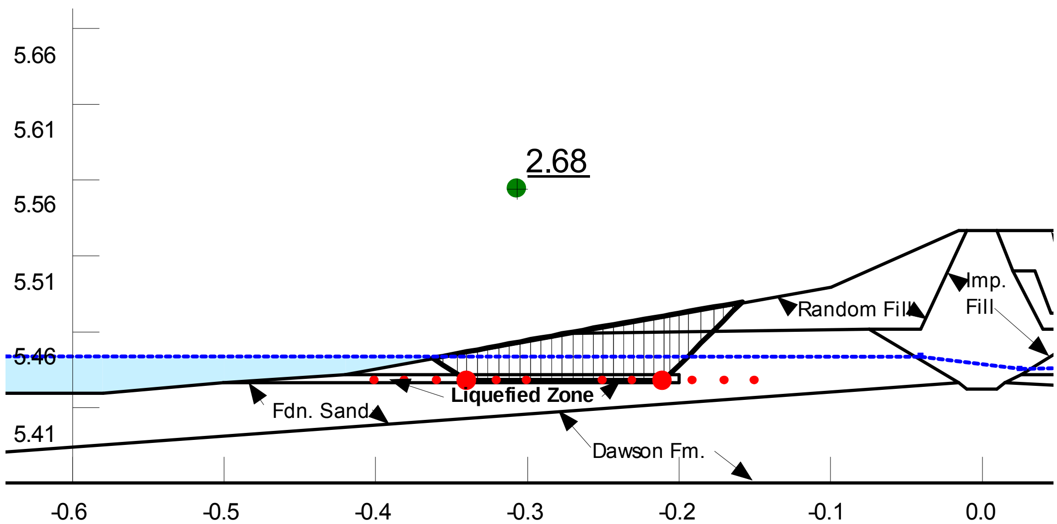


Figure D-1: Post-Liquefaction Slope Stability at Upstream Toe

Figure D-2: Sta 57+60, Pool El. 5444, Upstream Toe

Chatfield Sta 57+60 Pool El 5444 Upstream Toe

File Information

Revision Number: 21
Date: 09/03/2009
Time: 11:02:31 AM
File Name: Chatfield St 57+60 Pool 5444 - Min Liq - Max S US.gsz
Directory: F:\Projects\Other\Chatfield Dam\
Last Solved Date: 09/03/2009
Last Solved Time: 11:02:42 AM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Chatfield Sta 57+60

Kind: SLOPE/W
Method: Spencer
Settings
- Apply Phreatic Correction: No
- PWP Conditions Source: Piezometric Line
- Use Staged Rapid Drawdown: No

SlipSurface
- Direction of movement: Right to Left
- Use Passive Mode: No
- Slip Surface Option: Block
- Critical slip surfaces saved: 1
- Optimize Critical Slip Surface Location: No

FOS Distribution
- FOS Calculation Option: Constant

Restrict Block Crossing: No

Advanced

Number of Slices: 30

Optimization Tolerance: 0.01

Minimum Slip Surface Depth: 0.1 ft

Optimization Maximum Iterations: 2000

Optimization Convergence Tolerance: 1e-007

Starting Optimization Points: 8

Ending Optimization Points: 16

Complete Passes per Insertion: 1

Driving Side Maximum Convex Angle: 5 °

Resisting Side Maximum Convex Angle: 1 °

Materials

Embankment Impervious

Model: Mohr-Coulomb

Unit Weight: 120 pcf

Cohesion: 0 psf

Phi: 24.3 °

Phi-B: 0 °

Pore Water Pressure

Piezometric Line: 1

Embankment Dawson

Model: Mohr-Coulomb

Unit Weight: 120 pcf

Cohesion: 0 psf

Phi: 22.3 °

Phi-B: 0 °

Pore Water Pressure

Piezometric Line: 1

Embankment Random

Model: Mohr-Coulomb

Unit Weight: 120 pcf

Cohesion: 0 psf

Phi: 24.3 °

Phi-B: 0 °

Pore Water Pressure

Piezometric Line: 1

Embankment Pervious

Model: Mohr-Coulomb

Unit Weight: 130 pcf

Cohesion: 0 psf

Phi: 36.1 °

Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Foundation Sand
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 33 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Foundation Dawson
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 15 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Liquefied Foundation Sand
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 1120 psf
Phi: 0 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Limits
Left Coordinate: (-800, 5420) ft
Right Coordinate: (800, 5435) ft

Slip Surface Block
Left Grid
Upper Left: (-400, 5428) ft
Lower Left: (-400, 5428) ft
Lower Right: (-300, 5428) ft
X Increments: 5
Y Increments: 0
Starting Angle: 135 °
Ending Angle: 147 °
Angle Increments: 3

Right Grid

Upper Left: (-250, 5428) ft

Lower Left: (-250, 5428) ft

Lower Right: (-150, 5428) ft

X Increments: 5

Y Increments: 0

Starting Angle: 45 °

Ending Angle: 45 °

Angle Increments: 3

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	-800	5444
	-40	5444
	25	5435
	800	5435

Critical Slip Surfaces

	Slip Surface	FOS	Center (ft)	Radius (ft)	Entry (ft)	Exit (ft)
1	372	2.68	(-265.492, 5488.97)	92.704	(-158.286, 5479.71)	(-362.646, 5442.71)

Slices of Slip Surface: 372

	Slip Surface	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
1	372	-359.1799	5440.4555	221.1737	504.06889	183.71428	0
2	372	-350.9369	5435.1025	555.20056	1559.0521	651.90882	0
3	372	-343.07975	5430	873.60099	2615.2204	0	1120
4	372	-336.5	5428	998.4	2507.5714	0	1120
5	372	-329.5	5428	998.4	2670.8571	0	1120
6	372	-322.5	5428	998.4	2834.1429	0	1120

7	372	-315.5	5428	998.4	2997.5714	0	1120
8	372	-308.5	5428	998.4	3160.8571	0	1120
9	372	-301.5	5428	998.4	3324.1429	0	1120
10	372	-294.5	5428	998.4	3487.5714	0	1120
11	372	-287.5	5428	998.4	3650.8571	0	1120
12	372	-280.5	5428	998.4	3814.1429	0	1120
13	372	-273.5	5428	998.4	3977.5714	0	1120
14	372	-266.66665	5428	998.39995	4129.9498	0	1120
15	372	-260	5428	998.39995	4271.3998	0	1120
16	372	-253.33335	5428	998.39995	4412.9998	0	1120
17	372	-246.66665	5428	998.39995	4554.4498	0	1120
18	372	-240	5428	998.39995	4696.0498	0	1120
19	372	-233.33335	5428	998.39995	4837.4998	0	1120
20	372	-226.66665	5428	998.39995	4979.0998	0	1120
21	372	-220	5428	998.39995	5120.5497	0	1120
22	372	-213.33335	5428	998.39995	5262.1497	0	1120
23	372	-208	5430	873.59511	4124.2005	0	1120
24	372	-203	5435	561.59602	3456.456	1879.944	0
25	372	-197	5441	187.19474	2937.3217	1785.9533	0
26	372	-189.76165	5448.2385	-264.47634	2351.7998	1527.2767	0
27	372	-181.28495	5456.715	-793.42067	1699.8892	1103.921	0
28	372	-173.9198	5464.08	-1253.0556	1201.382	542.44479	0
29	372	-167.6662	5470.334	-1643.2645	720.84054	325.47198	0
30	372	-161.41255	5476.5875	-2033.4734	240.27641	108.48896	0