DAM SAFETY ASSESSMENT REPORT
PLANT CRISP
COAL COMBUSTION WASTE IMPOUNDMENT

14-5232
REVISION 0
JANUARY 16, 2015

SUBMITTED TO:

CRISP COUNTY POWER COMMISSION

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APPROVALS

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Report Name:	Dam Safety Assessment Report Plant Crisp Coal Combustion Waste Impoundment Crisp County Power Commission
Date:	January 16, 2015
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Approval by the responsible manager signifies that the document is complete, all required reviews are complete, and the document is released for use.

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CHANGE MANAGEMENT RECORD

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Crisp County Power Commission

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0	January 16, 2015	Initial Submittal

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1.0 INTRODUCTION

RIZZO Associates (RIZZO) has prepared this dam safety assessment report on the Plant Crisp Coal Combustion Waste (CCW) Impoundment for the Crisp County Power Commission (CCPC). The scope of this report is to identify potential issues with dam performance, compare current conditions with those documented in the February 2014 report by CDM Smith, analyze slope stability and hydrologic and hydraulic conditions at the site, provide recommendations for correcting deficiencies, and identify additional investigations or remedial measures that may be required to meet the Georgia Safe Dams Program and United States Environmental Protection Agency (EPA) dam safety requirements.

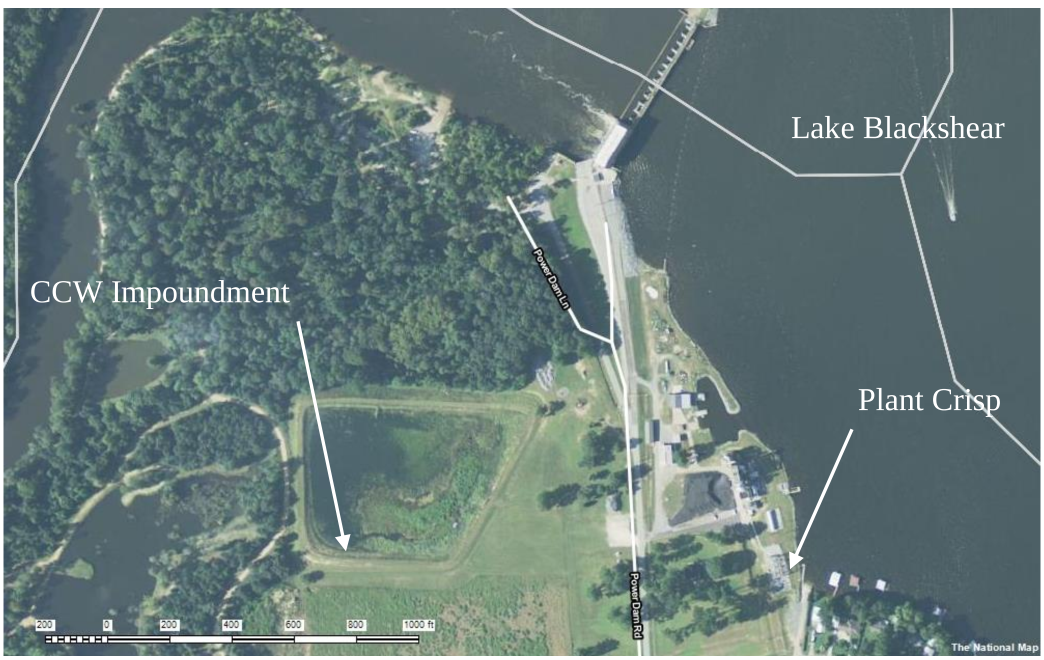

FIGURE 1-1
AERIAL PHOTOGRAPH - VIEW OF PLANT CRISP AND CCW IMPOUNDMENT

2.0 PROJECT DESCRIPTION

The CCW Impoundment at the Plant Crisp fossil plant is located west of the fossil plant and southwest of the Lake Blackshear Hydroelectric Project. The trapezoidal impoundment consists of built-up earthen embankments on all sides, ranging from 2 feet (ft) to 5 ft high (East and South Embankments) to approximately 22 ft high (West and North Embankments). The bottom of the impoundment generally slopes down from east to west. The West Embankment runs against the CCPC property line, with a sand-clay road along its toe on the adjacent property. ***Table 2-1*** summarizes the general details of the CCW Impoundment.

TABLE 2-1
CCW IMPOUNDMENT DETAILS

ITEM	INFORMATION
Geographical Location:	Worth County, GA Latitude: 31º 50’ 40.81’ N Longitude: 83º 56’ 28.74” W
GA Safe Dams Program Size Classification:	Small
EPA-Recommended Hazard Classification:	Low Hazard
Drainage Area:	6.5 Acres
Dam Type:	Earthen Embankment
Maximum Dam Height:	22 ft
Dam Length (Approximate):	Total Embankment: 2,222 ft North Embankment: 720 ft East Embankment: 570 ft South Embankment: 448 ft West Embankment: 484 ft
Design Slopes: (Upstream and Downstream)	2H:1V
Crest Elevation:	245 ft
Normal Pool Elevation:	< 240.95 ft
Reservoir Area:	6.5 Acres
Normal Storage Capacity:	29 ac-ft
Primary Spillway Type	Corrugated metal pipe drop inlet
Primary Spillway Diameter	12” inlet with 24” diameter screen 12” discharge
Primary Spillway Inlet Elevation	240.95 ft
Required Spillway Design Flood (SDF)	0.25 PMP (Based on Georgia Safe Dams Program Criteria)
Primary Spillway Capacity	> 3.2 cfs
Auxiliary Spillway Type	Earth chute at NE corner
Auxiliary Spillway Dimensions	Approximately 6” deep by 80’ long

Two discharge lines empty into the CCW Impoundment: a ductile iron pipe that carries water and CCW byproducts from the fossil plant during plant operations and a Polyvinyl chloride (PVC) line that carries miscellaneous runoff and process water from the bag house sump. Plant Crisp is rarely operated, with less than 100 operating hours in the last year; thus, the deposition and accumulation of CCW materials is very limited.

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3.0 DAM SAFETY INSPECTION AND TOPOGRAPHIC SURVEY

Site visits and dam safety inspections were performed by RIZZO personnel on several days in March and May of 2014. ***Table 3-1*** identifies visit and inspection dates and key participants.

TABLE 3-1
DAM SAFETY RECONNAISSANCE AND INSPECTION PARTICIPANTS AND SCHEDULE

EVALUATION TYPE AND DATE	RIZZO PERSONNEL
March 27, 2014 (Site Visit & Reconnaissance)	John P. Osterle, P.E. A. Hans Hasnay, P.E.
May 1-2, 2014 (Detailed Inspection)	Conrad H. Ginther, P.E.
May 15, 2014 (Detailed Inspection)	Conrad H. Ginther, P.E.

In general, the embankment slopes and crest are in good condition, with no sloughs, cracking, or other evidence of active slope instability. The downstream slopes of the West Embankment are somewhat irregular, but no sloughs or signs of instability were noted. According to plant personnel, the slopes of the North and West Embankments were cleared of small trees and brush several years ago, and the slopes were not groomed to achieve a uniform surface after the clearing operation concluded. Overall vegetal cover is good, with a few bare areas on the North Embankment and small animal trails that lead down to the water on the interior slopes of the impoundment.

Based on RIZZO's visual safety inspection and review of available documents, the Plant Crisp CCW Impoundment, with a suggested "Low Hazard" classification, was determined to be in fair overall condition. Subsequent stability, liquefaction, and hydrological analyses have been performed to further evaluate the structure.

The Dam Safety Inspection Check List included in ***Appendix A*** provides a comprehensive listing of the items checked and photograph references. The following findings are of high importance:

- The exterior slopes of the West Embankment are irregular, with hummocky areas and some short vertical surfaces near the crest, and lack sufficient vegetal cover to prevent surface erosion at several locations.

- An area of ponded water and soft material was noted along the toe near the midpoint of the North Embankment. This area starts approximately 100 ft east of the spillway discharge pipe and historically has not been wet and soft. Several days of heavy rain preceded the RIZZO May inspections, so it is unclear whether the wet, soft zone is due to new seepage from the impoundment or from collected surface runoff.
- The spillway discharge pipe was found to be completely buried and plugged. It appears that over the years, the mouth of the pipe was buried as a result of deposition of silt, and possibly, disturbance of the area by a contractor hired to clear the dam toe. During the inspections, the pipe was partially excavated using hand tools, and the discharge flow was observed until it had dropped to near zero. It appears that the pipe and low-level inlet valve are still intact as the reservoir level was lower than the spillway inlet, and the flow stopped when the trapped water was released.

4.0 GEOTECHNICAL INVESTIGATION

4.1 SUBSURFACE EXPLORATION

RIZZO retained Geotechnical & Environmental Consultants, Inc. to perform the drilling, sampling, and laboratory testing required for evaluation of the CCW impoundment. Subsurface exploration was performed with a truck-mounted drill rig from May 1, 2014 to May 2, 2014 and on May 15, 2014 with a track-mounted drill rig. Borings were logged by Mr. Conrad Ginther of RIZZO. Two borings were drilled in the North Embankment, and two were drilled in the West Embankment–one from the crest and one near the toe of the slope in each embankment. Borings were advanced using a hollow-stem auger, and Standard Penetration Test (SPT) samples were taken at 2-ft intervals. ***Table 4-1*** summarizes the location of the borings and the depths drilled. Boring locations are shown on Drawing 1 in ***Appendix B***.

TABLE 4-1
SUBSURFACE EXPLORATION SUMMARY

BORING	DATE DRILLED	APPROXIMATE LOCATION	SURFACE ELEVATION[1] (ft)	DRILLED DEPTH (ft)
N-1	May 1, 2014	North Embankment crest near principal spillway	245.2	49.5
N-2	May 15, 2014	Toe of North Embankment near principal spillway	226 [1]	24.5
W-1	May 1, 2014	West Embankment crest approximately 140' south of northwest corner of Impoundment	243.9	49.5
W-2	May 2, 2014	Center of sand-clay road at toe of West Embankment, offset approximately 60'west of W-1	228.64	35.5

Note:

1. Approximate surface elevation, estimated using topographic survey.

The subsurface stratigraphy encountered in the borings was consistent across the site. In the borings drilled through the embankment crest, 18 ft to 20 ft of orange- to orange-brown silty sand and silty-clayey sand fill was encountered, followed by approximately 5 ft of mottled natural silty sand and silty-clayey sand. The sandy materials were underlain by a thin layer of light grey-blue clay marl and white or light tan calcareous clay with limestone fragments. The borings at the toe encountered similar materials, with the exception of the fill. ***Table 4-2*** summarizes the general subsurface profile of the site.

TABLE 4-2
SUMMARY OF SUBSURFACE CONDITIONS

MATERIAL	AVERAGE TOP ELEVATION (ft)	AVERAGE BOTTOM ELEVATION (ft)	MATERIAL DESCRIPTION
Fill	244.5	223.8	Moist, loose to medium-dense, orange and orange-brown silty sand and silty-clayey sand (SM, SM-SC)
Silty Sand & Silty-Clayey Sand	223.8	218.9	Moist to wet, loose to medium-dense, mottled grey-brown and orange silty sand and silty-clayey sand with trace organics (SM, SM-SC)
Clay Marl & Calcareous Clay (decomposed limestone)	218.9	Below terminal depth	Wet, grey-blue stiff clay transitioning to light tan calcareous clay with limestone fragments–decomposed limestone (CH, CL)

Boring logs are included in ***Appendix C***.

4.2 LABORATORY TESTING

Laboratory testing was performed on selected samples to determine soil engineering properties and to confirm field-performed soil classifications. Tests included sieve analysis, moisture content, organic content, and Atterberg limits. ***Table 4-3*** summarizes the results of the laboratory testing program. Laboratory test result sheets are provided in ***Appendix C,*** and testing results are included on the boring logs.

TABLE 4-3
SUMMARY OF LABORATORY TEST RESULTS

SAMPLE ID (BORING, SAMPLE #)	SAMPLE DEPTH (ft)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	ATTERBERG LIMITS			% PASSING			USCS CLASSIFICATION
				LL	PL	PI	10	40	200	
N-1, #04	6-7.5	9.3	*	19	14	5	*	*	*	*
N-1, #05	8-9.5	9.5	*	*	*	*	94.2	65.2	22.5	SM
N-1, #07	12-13.5	13.0	*	*	*	*	*	*	19.5	*
N-1, #09	16-17.5	15.0	2.8	*	*	*	*	*	*	*
N-1, #10	18-19.5	10.0	*	*	*	*	98.6	70.3	28.2	SM
N-1, #13	24-25.5	23.1	*	63	20	43	*	*	*	*

TABLE 4-3
SUMMARY OF LABORATORY TEST RESULTS
(CONTINUED)

SAMPLE ID (BORING, SAMPLE #)	SAMPLE DEPTH (ft)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	ATTERBERG LIMITS			% PASSING			USCS CLASSIFICATION
				LL	PL	PI	10	40	200	
N-1, #14	16-17.5	13.4	*	41	13	28	*	*	*	*
N-1, #18	34-35.5	23.8	*	*	*	*	*	*	56.9	*
N-1, #23	44-45.5	44.5	*	*	*	*	*	*	50.9	*
N-1, #24	46-47.5	32.3	*	34	27	7	*	*	*	*
N-1, #25	48-49.5	53.3	*	*	*	*	100	98.4	60.9	ML
W-1, #03	4-5.5	12.5	*	*	*	*	95.4	60.0	24.2	SM
W-1, #07	12-13.5	9.8	*	*	*	*	96.8	70.5	27.5	SM
W-1, #11	20-21.5	13.1	*	*	*	*	*	*	27.3	*
W-1, #12	22-23.5	15.5	3.0	*	*	*	*	*	30.0	*
W-1, #13	24-25.5	17.7	*	61	19	42	*	*	*	CH
W-1, #19	36-37.5	28.7	*	*	*	*	*	*	57.3	*
W-2, #03	8-9.5	15.1	*	*	*	*	*	*	45.3	*
W-2, #05	12-13.5	14.7	*	46	15	31	*	*	44.3	SC
W-2, #08	18-19.5	21.8	*	*	*	*	*	*	64.7	*
W-2, #11	24-24.5	28.2	*	*	*	*	*	*	57.8	*

Note:

* Not Tested

4.3 MATERIAL PROPERTIES

Engineering properties for the materials encountered in the borings were developed using the SPT data collected in the field and the laboratory test results. The SPT and laboratory test results were compared to published correlations (Peck, 1974) among N values, internal friction angle, and cohesion. These properties were used in the slope stability analyses of the CCW Impoundment. ***Table 4-4*** summarizes the material properties developed.

TABLE 4-4
SUMMARY OF MATERIAL PROPERTIES

MATERIAL	UNIT WEIGHT (pcf)	COHESION (psf)	PHI (deg)
SM	125	0	31
SC-SM	125	0	30
CL	135	1000	0
CL-Decomposed Limestone	140	5000	0

5.0 SLOPE STABILITY

The slope stability of the North and West Embankments was evaluated using a subsurface conditions model developed from the subsurface investigation and site survey results. SLOPE/W was used to perform static limit-equilibrium stability analyses of the CCW Impoundment North Embankment dam downstream slope. The North Embankment is the highest of the embankment sections comprising the CCW Impoundment. Analyses were also performed for the upstream slope of the dam, including an analysis for rapid drawdown loading conditions. The Morgenstern-Price method was used to perform the limit-equilibrium analysis, which satisfies both force and moment equilibrium.

The following load conditions were evaluated:

- Steady seepage with normal pool level
- Steady seepage with normal pool level and earthquake loading
- Steady seepage with maximum pool level
- Rapid drawdown from maximum pool level (upstream slope)
- Steady seepage with post-seismic strengths
- Steady seepage with normal pool level and 2 ft of water on the downstream toe

Minimum factors of safety for slope stability were taken from Georgia Safe Dams Program guidelines. Under these guidelines, the CCW Impoundment is classified as a Category II small dam because it is less than 25 ft in height and has less than 500 acre-ft of storage. The guidelines also specify that the dam must be capable of withstanding seismic accelerations defined in the most current U.S. Geological Survey (USGS) map for peak acceleration with a 2 percent exceedance in 50 years or a minimum seismic acceleration of 0.05g, whichever is greater. According to current USGS maps, the calculated seismic acceleration for the site is 0.08g.

To support the load case including post-seismic strengths, the potential for subsurface material liquefaction was analyzed. Soils that experience full or partial liquefaction during seismic events exhibit reduced strength. The factor of safety (FS) against liquefaction of each soil layer for a given seismic acceleration was calculated using empirical methods that account for the fines content, relative density, and distance to bedrock of the layer analyzed. This calculation showed that the majority of the materials are not expected to be subject to liquefaction, with the exception of a loose (N=4) layer of silty-clayey sand encountered in boring W-1 at the base of the dam. Strength reductions were applied to the layer of interest in the stability analysis to evaluate the effect of the weakened layer on slope stability.

The stability analysis indicates that the North and West embankments in their current configuration does not meet Georgia Safe Dams Program requirements for any of the downstream slope stability cases. While the embankment is relatively short, the downstream slopes are too steep and will require modification to comply with the Georgia Safe Dams Program minimum Factors of Safety. The upstream (interior) slopes of the embankments meet the requirements as they are much flatter than the downstream slopes. ***Table 5-1*** summarizes the minimum FS for the stability cases analyzed. The complete results of the liquefaction and stability analyses are provided in ***Appendix D***.

TABLE 5-1
FACTOR OF SAFETY FOR SLOPE STABILITY

LOAD CASE	FAILURE TYPE	CALCULATED FS	GEORGIA SAFE DAMS PROGRAM MINIMUM FS
Steady State Seepage (Normal Pool)	Circular	1.26	1.5
Steady State Seepage (Normal Pool)	Block	1.26	1.5
Steady State Seepage with Seismic Loading (Normal Pool)	Circular	0.92	1.1
Steady State Seepage (Maximum Pool)[1]	Circular	1.15	1.3
Rapid Drawdown (Upstream Slope)	Circular	1.71	1.3
Steady State Seepage with Post-Seismic Strengths	Block	1.23	1.1
Steady State Seepage (Normal Pool with 2' of Water at Downstream Toe)	Circular	1.26	1.5

Note:

1. This case is not required by Georgia Safe Dams Program guidelines but is included because it is in the realm of possibility.

6.0 HYDROLOGIC AND HYDRAULIC ANALYSIS

Coal ash is pumped into the impoundment via a discharge pipe from the coal plant as a slurry (i.e., an ash-water mixture). Coal ash settles in the impoundment, and water slowly infiltrates into the ground. During periods of high water in the impoundment, excess water is discharged from the pond via a vertical 12-inch-diameter corrugated metal pipe (CMP). The CMP acts as a morning glory spillway with a crest elevation of 240.95 ft. Hydrologic and hydraulic analyses were performed using the characteristics of the CCW Impoundment to determine if impoundment outflow structure capacity is sufficient to pass the design flood for the dam without overtopping of the embankment. The following sections summarize the analyses performed. The full text and output from the analyses is presented in ***Appendix E***.

6.1 HAZARD CLASSIFICATION AND DESIGN FLOOD

The design flood event for the dam depends upon the dam's classification, which is based upon the threat of potential damage to life and property from dam failure. Two classification systems were considered: Federal Emergency Management Agency (FEMA) and Georgia state regulations. Under FEMA's hazard classification system, the CCW Impoundment is a low-hazard structure. This indicates that a hypothetical failure would not result in loss of life or major economic and/or environmental losses. According to FEMA, the design flood event for a low-hazard dam is the 100-year flood event.

The state of Georgia determines hazard class based upon dam storage capacity and height. The CCW Impoundment has a maximum embankment height of 22 ft and a maximum storage volume of 42.1 acre-ft. Therefore, according to the state of Georgia, the structure is considered a small dam (e.g., a dam with storage capacity less than 500 acre-ft and a height not exceeding 25 ft). According to Georgia guidelines, the design flood event for a small dam is the flood that results from a precipitation event equal to 25 percent of the Probable Maximum Precipitation (PMP).

The analysis compared the design flood events for the CCW Impoundment based upon the 100-year event (FEMA criteria) and the 25 percent-of-PMP event (Georgia guidelines). As shown in ***Appendix E***, the 25 percent-of-PMP event produces greater precipitation depths; therefore, this event was used in the model. The site conditions were modeled using Hydrologic Modeling Software (HMS) from the Hydrologic Engineering Center (HEC) of the U.S. Army Corps of

Engineers. Among the 25 percent-of-PMP events, the 24-hour event has greater rainfall depths than the 6-hour or 12-hour events. Therefore, the incremental precipitation values for the 24-hour event were used in this analysis and incorporated into the HEC-HMS model.

6.2 IMPOUNDMENT STORAGE AND DISCHARGE RELATIONSHIPS

The elevation-storage and elevation-area curves were calculated for the CCW Impoundment based on the site topographic survey and are shown on ***Figure 6-1***. Contours from the site survey are provided in the drawings in ***Appendix B***. It should be noted that inflow into the impoundment consists of direct rainfall and discharge from Plant Crisp, and it has been assumed that the 25 percent of the PMP inflow does not occur simultaneously with discharge from the plant.

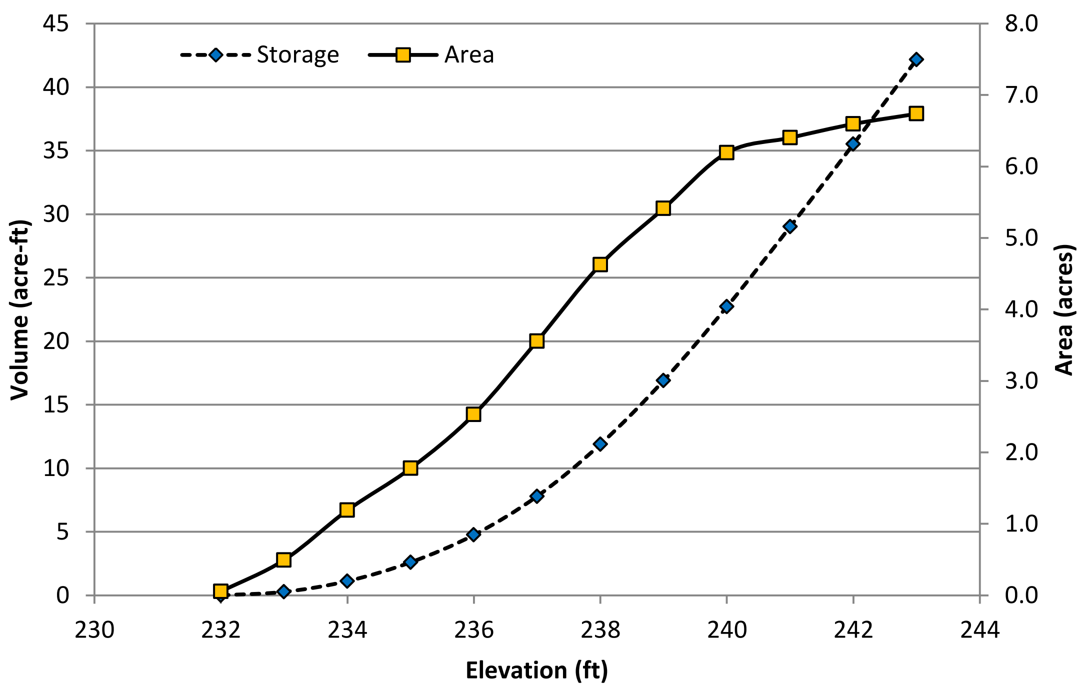


FIGURE 6-1
STORAGE AND AREA CURVES FOR CCW IMPOUNDMENT

The vertical CMP that serves as the outlet structure for the CCW Impoundment functions as a morning glory spillway. Flow regimes developed for a morning glory spillway are dependent on the water head above the spillway crest and the dimensions of the different geometric features of the spillway. In the case of small heads, flow over the spillway is governed by the characteristics of crest discharge. With respect to larger discharges, submergence begins to affect the weir flow, ultimately the crest will drown out, and orifice control flow (throat control) will govern until full pipe flow conditions develop. A spillway rating curve (***Figure 6-2***) showing the relationship between reservoir elevation and spillway discharge was calculated using United States Bureau of Reclamation methodology, which accounts for the different flow regimes that develop for the spillway, including the effects of submergence and back pressure.

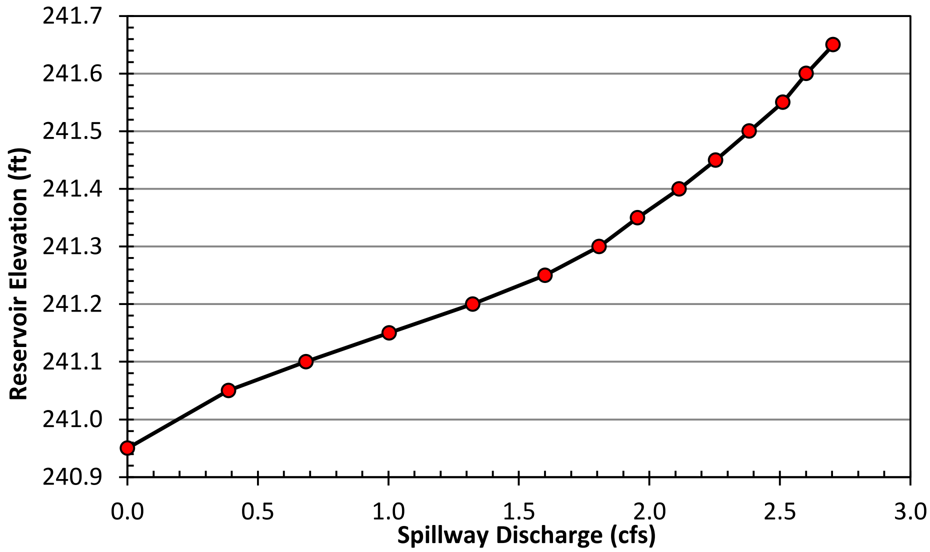


FIGURE 6-2
CCW IMPOUNDMENT SPILLWAY RATING CURVE

6.3 RESULTS OF HEC-HMS MODELING

The hydrologic and hydraulic analyses performed demonstrate that the capacity of the outflow structure at the CCW Impoundment is sufficient to pass the design flood event for the dam (25 percent of the PMP) without overtopping of the embankment. The total precipitation to fall in the reservoir during the 25 percent PMP is 11.03 inches. From a starting elevation at normal pool (Elevation 240.95 ft), this precipitation causes the water surface in the CCW Impoundment to peak at Elevation 241.6 ft (0.65 ft rise) and results in a peak outflow through the spillway of 2.7 cubic feet per second (cfs). The peak water surface elevation is approximately 1.4 ft below the lowest portion of the embankment. ***Figure 6-3*** shows the results of the modeling in plots of precipitation, reservoir elevation, and spillway discharge.

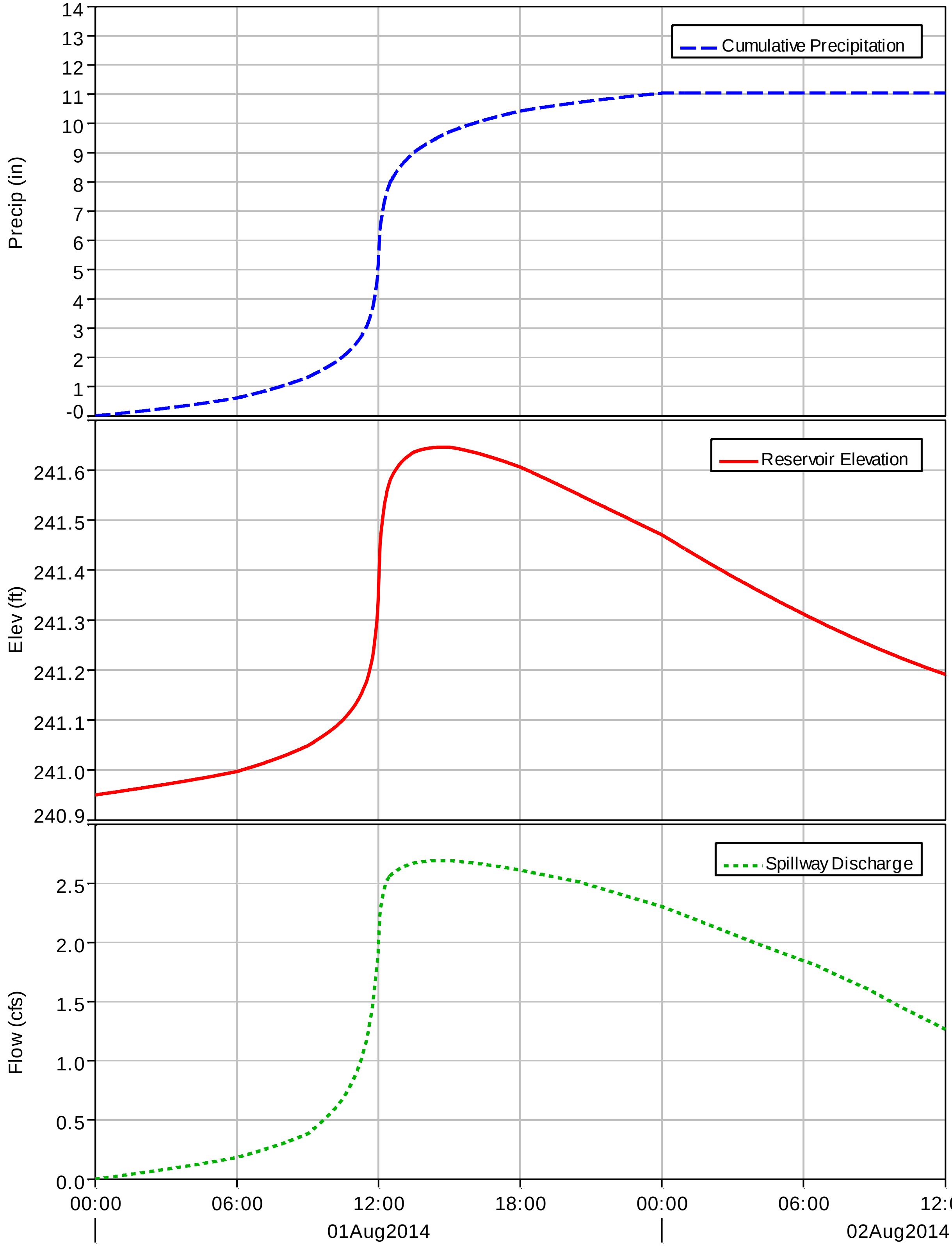


FIGURE 6-3
CCW IMPOUNDMENT HYDRAULIC MODEL RESULTS

7.0 RECOMMENDATIONS AND CONCLUSION

Overall, the CCW Impoundment is in good condition, with adequate vegetal cover and no signs of active slope instability or other conditions that require immediate action so that the impoundment can continue to operate safely. Spillway capacity is adequate for the design flood event, and the spillway outlet has been improved to ensure that flow will not be obstructed when needed. At this time, the CCW Impoundment does not appear to fall under the control of dam safety regulatory agencies, including the Georgia Safe Dams Program, EPA, and the U.S. Army Corps of Engineers. The minimum factors of safety for slope stability required by the Georgia Safe Dams Program are generally consistent with common regulatory standards for embankments that are similar in size and hazard classification to those of the CCW Impoundment. For this reason, RIZZO recommends adoption of the Georgia Safe Dams Program minimum requirements for the CCW Impoundment. To meet these minimum factors of safety, the embankment slopes must be modified, as discussed below.

RIZZO offers the following recommendations to assist the CCPC in ensuring that the dam and its appurtenant works continue to operate safely.

R-1 The exterior slopes of the North and West Embankments should be flattened to provide adequate FS against instability. This may be accomplished by stripping and adding compacted engineered fill against the existing downstream face or by cutting back the slope. The latter option will result in a narrower crest and may require the addition of fill on the interior slopes to ensure suitable crest width and continuing access to the embankment. A minimum crest width of 10 ft is recommended. After the wider section is established, the area should be seeded or sodded as appropriate to prevent further erosion in the area. The results of preliminary slope stability analysis indicate that flattening the slopes to 2.5H:1V with a 10 foot crest width will provide an adequate FS against slope instability.

R-2 The toe of the North Embankment should be monitored, and the source of the ponded water and soft conditions at the toe should be further evaluated. If the source of the water is seepage through the embankment or leakage from the spillway discharge pipe, remedial action may be necessary.

R-3 The spillway discharge pipe has been excavated and cleared to ensure that it is operable. As shown in ***Appendix A***, the spillway discharge pipe has been improved by fully excavating the pipe end, adding a flared end-section terminating with a grate, and constructing a small rip-rap and gravel splash pad. RIZZO recommends that the grate at the discharge end of the pipe be attached so that it is hinged about the top of the pipe.

This configuration will allow the grate to float open instead of blocking up if debris enters the spillway from the impoundment during high flow events.

R-4 Visual observation and inspection of the discharge pipe should be added to the existing weekly inspection tasks performed by plant personnel. Observations of discharge volume and turbidity should be recorded to provide a history of spillway performance over time.

8.0 REFERENCES

Peck, R.B., Hanson, W.E. and Thornburn, T.H., 1974, Foundation Engineering, 2nd Edition, John Wiley and Sons, Inc.

Whiteside, Stephen L., “Crisp County Power Commission, Plant Crisp, Warwick, Georgia,” CDM Smith, Raleigh, NC, Report to Environmental Protection Agency, December 2013 (Rev February 2014).

APPENDIX A

INSPECTION CHECKLISTS AND PHOTOGRAPHS

DAM SAFETY INSPECTION CHECKLIST

DAM SAFETY INSPECTION CHECKLIST
PLANT CRISP CCW IMPOUNDMENT

RESERVOIR AREA			
ITEMS	**YES**	**NO**	**REMARKS**
1. Signs of Shoreline Instability		x	***Photo 1***
2. Sedimentation	x		The rate of sedimentation at the CCW Impoundment is extremely slow due to the very limited operation of the fossil plant (~50 hours in 2013). An automatic level control device pumps all runoff and wash and process water, etc., from a sump at the plant. Under current operating conditions, stormwater runoff constitutes the majority of discharge and is delivered via an 8”-diameter PVC pipe on the northern side of the impoundment’s East Embankment. CCW is sluiced into the impoundment via an 8” ductile iron pipe on the southern side of the East Embankment. CCW solids (bottom ash and other larger granular waste products) are periodically deposited in the impoundment from the east side.
3. Debris		x	
4. Ice-Related Problems		x	
5. Operating Constraints		x	
6. Environmental Concerns		x	
7. Rim Stability		x	No issues. Some areas have poor vegetal cover.
8. Other	x		Scrub vegetation grows in the impoundment on the east side. The inside and outside slopes are generally free of brush and tree growth.

DAM SAFETY INSPECTION CHECKLIST
PLANT CCW IMPOUNDMENT
(CONTINUED)

SERVICE SPILLWAY 12" Corrugated Metal Pipe (CMP) Drop Inlet with 24" Mesh and CMP Trash Rack			
ITEMS	**YES**	**NO**	**REMARKS**
1. CMP Drop Inlet			
a. Settlements		x	None apparent. The original installation elevation data are unavailable.
b. Displacements		x	The foundation of the inlet is unknown but appears to be plumb.
c. Cracking		x	
d. Deterioration		x	The galvanized CMP and strainer appear to be in good condition, with very little corrosion. The original construction included a valved/gated opening into the reservoir; however, the actuator has since been cut off due to corrosion. While the condition of the valve/gate is unknown, it appears to be intact, based on flow through the outlet following removal of the obstruction. The condition of the outlet pipe through the embankment was not observed. The outlet pipe was found to be buried and plugged but in good condition when excavated.
e. Exposed Reinforcement			N/A
f. Downstream Boils		x	
g. Springs		x	None noted. There are ponds/swampland to the north and west of the impoundment.
2. Discharge Channel		x	A discharge channel was not provided for the outlet pipe, which contributed to the covering and plugging of the discharge____. A discharge channel should be established.
a. Deterioration		x	
b. Undercutting		x	
c. Erosion		x	
d. Obstruction		x	

EARTHEN EMBANKMENTS			
ITEMS	**YES**	**NO**	**REMARKS**
1. Alignment			
a. Alignment		x	The crest and toe alignments appear uniform.
b. Displacement		x	
c. Settlement		x	None noticeable during walkdown; to be confirmed by site survey.
2. Deterioration			
a. Erosion	x		There is some minor surface erosion at locations of concentrated runoff or missing vegetal cover.
b. Sloughs or Slumps	x		There are 1- to 1.5-ft-high vertical faces along the crest on the outside slope at several locations on the West Embankment. The exterior slopes on the West Embankment are somewhat irregular/hummocky. No circular slip surfaces or cracks were observed. Based on conversations with site personnel, the irregular surface may be due to removal of extra material during previous brush-clearing operations.
c. Riprap		x	None
d. Damage from Nuisance Wildlife		x	No burrows or undercuts along the bank were noted. At least two paths over the embankment where animals approach the impoundment were noted (North and South embankments). Armadillo burrows were identified along the treeline downstream of the North Embankment, near the spillway outlet.
3. Seepage		x	None
a. Where			The toe along the North Embankment is historically dry. During inspection in May, the section of the toe east of the outlet had standing clear water and was soft. The site had experienced several days of heavy rain at the time of inspection.

DAM SAFETY INSPECTION CHECKLIST
PLANT CCW IMPOUNDMENT
(CONTINUED)

ITEMS	YES	NO	REMARKS
b. Quantity			
4. Abutment Contacts			
a. Abutment Instability		x	
b. Erosion		x	
c. Undercutting		x	
d. Visible Displacement		x	
e. Seepage from Contact		x	
f. Downstream Boils		x	
g. Springs		x	
h. Abutment Shoreline Freeboard			>5 feet at northeast and southeast corners
5. Instrumentation		x	There is no monitoring instrumentation at this dam.

Other Comments:

- Based on the results from the topographic survey and field observations, the slopes are irregular in some locations and do not match those indicated on the available Natural Resources Conservation Service (NRCS) design drawings. Stability analyses will determine whether or not the existing slopes are sufficient to meet dam stability requirements.
- The outside slope of the West Embankment has several vertical faces near the crest and hummocky areas. While no signs of active slope movement were noted, these slopes should be regraded to even slopes and reseeded or sodded to provide adequate vegetal cover.
- Minor bare areas and a few vertical faces were observed on the outside slope of the North Embankment.
- A short overflow spillway section was constructed on the eastern end of the North Embankment. The embankment crest slopes down approximately one foot to a lower elevation, which is constant with the northeast corner of the embankment. It appears from visual observation that portions of the East Embankment crest are lower still than the overflow section. Overflow would be expected to exit on the east side of the reservoir and would likely result in relatively minor headcutting and loss of embankment material.

PLANT CRISP CCW IMPOUNDMENT PHOTOGRAPHS

PHOTO 1
PANORAMA OF EAST EMBANKMENT (LOOKING WEST)

PHOTO 2
PANORAMA OF CCW IMPOUNDMENT FROM SE CORNER OF EMBANKMENT

PHOTO 3
PANORAMA OF CCW IMPOUNDMENT FROM SW CORNER

PHOTO 4
PANORAMA OF WEST EMBANKMENT (LOOKING E)

NORTH EMBANKMENT PHOTOS

PHOTO 5
INTERIOR SLOPES OF N. EMBANKMENT (LOOKING W FROM NE CORNER)

PHOTO 6
EXTERIOR SLOPES OF N. EMBANKMENT (LOOKING W FROM NE CORNER)

PHOTO 7
EXTERIOR SLOPES, STANDING WATER, AND SOFT AREA AT TOE OF N. EMBANKMENT (LOOKING W)

PHOTO 8
EXTERIOR SLOPES, STANDING WATER, AND SOFT AREA AT TOE OF N. EMBANKMENT (LOOKING E)

PHOTO 9
EXTERIOR SLOPES OF N. EMANKMENT NEAR NW CORNER
(LOOKING W)

PHOTO 10
EXTERIOR SLOPES OF N. EMBANKMENT AT NW CORNER
(LOOKING E)

PHOTO 11
INTERIOR SLOPES OF N. EMBANKMENT FROM NW CORNER (LOOKING E)

PHOTO 12
INTERIOR SLOPES OF N. EMBANKMENT FROM BOARDWALK (LOOKING W)

PHOTO 13
INTERIOR SLOPES OF N. EMBANKMENT FROM BOARDWALK (LOOKING E)

PHOTO 14
BOARDWALK, CMP DROP INLET, AND TRASH SCREEN

PHOTO 15
12” CMP DROP INLET AND 24” TRASH SCREEN

PHOTO 16
INITIAL FLOW FROM SPILLWAY DISCHARGE AFTER UNPLUGGING

PHOTO 17
RUSTY STORED WATER DRAINING FROM UNPLUGGED DISCHARGE

PHOTO 18
SPILLWAY DISCHARGE AFTER FLOW STOPPED

PHOTO 19
FLARED END-SECTION ADDED TO SPILLWAY DISCHARGE (PHOTO BY CCPC)

PHOTO 20
GRATE BEING INSTALLED OVER DISCHARGE (PHOTO BY CCPC)

PHOTO 21
COMPLETED IMPROVEMENTS (PHOTO BY CCPC)

PHOTO 22
ANIMAL TRAILS UP AND OVER EMBANKMENT

PHOTO 23
ARMADILLOW BURROW IN TREELINE

EAST EMBANKMENT PHOTOS

PHOTO 24
NE CORNER OF EMBANKMENTS

PHOTO 25
INTERIOR SLOPE OF EAST EMBANKMENT (LOOKING S)

PHOTO 26
EXTERIOR SLOPE OF EAST EMBANKMENT (LOOKING N)

PHOTO 27
DISPOSAL OF BOTTOM ASH/GRANULAR BYPRODUCT

PHOTO 28
8” PVC SLURRY DISCHARGE PIPE

PHOTO 29
DISCHARGE PERMIT SIGN

PHOTO 30
7" DUCTILE IRON DISCHARGE PIPE

SOUTH EMBANKMENT PHOTOS

PHOTO 31
EXTERIOR SLOPES OF S EMBANKMENT FROM SE CORNER
(LOOKING W)

PHOTO 32
INTERIOR SLOPES OF S EMBANKMENT FROM SE CORNER

(LOOKING W)

PHOTO 33
INTERIOR SLOPES OF S EMBANKMENT (LOOKING W)

PHOTO 34
ANIMAL PATH ON INTERIOR SLOPE OF S EMBANKMENT

(LOOKING S)

PHOTO 35
BEGINNING OF TOE DITCH AT SW CORNER OF EMBANKMENT

WEST EMBANKMENT PHOTOS

PHOTO 36
W EMBANKMENT EXTERIOR SLOPE FROM SW CORNER (LOOKING N)

PHOTO 37
W EMBANKMENT EXTERIOR SLOPE IN HUMMOCKY/ BARE AREA (LOOKING N)

PHOTO 38
W EMBANKMENT EXTERIOR SLOPE (LOOKING S)

PHOTO 39
W EMBANKMENT BARE/ IRREGULAR EXTERIOR SLOPE (LOOKING E)

PHOTO 40
VERTICAL SURFACES AT CREST OF W EMBANKMENT

PHOTO 41
IRREGULAR EXTERIOR SLOPES ON W EMBANKMENT (LOOKING NE)

PHOTO 42
EXTERIOR SLOPES AT NW CORNER OF EMBANKMENT
(LOOKING NE)

PHOTO 43
DRILLING BORING W-2

PHOTO 44
POND ADJACENT TO TOE OF W EMBANKMENT (LOOKING W)

PHOTO 45
INTERIOR SLOPES OF W EMBANKMENT (LOOKING N)

PHOTO 46
INTERIOR SLOPES OF W EMBANKMENT (LOOKING NW)

APPENDIX B

DRAWINGS

CAD FILE NUMBER 14-5232-D400(M)

CHECKED BY | APPROVED BY

GPB | 5/23/14

DRAWN BY

APPROXIMATE STORAGE CAPACITY IS 181 AC-FT

ASH POND PLAN

SCALE: 1"= 40'-0"

S C A L E

40 0 40 80 FEET

NOTE:

POOL ELEVATION AS OF THE DATE OF THIS SURVEY MAY 2 & MAY 5, 2014 = APPROX. ELEVATION 239'

BORING LOCATION

REFERENCE:

A DRAWING ENTITLED" TOPOGRAPHIC SURVEY OF ASH POND" BY J.B. FAIRCLOTH & ASSOCIATES. DATED: MAY 2, & MAY 5, 2014 AND REVISED MAY 28, 2014.

NO.	DESCRIPTIONS	DATE	APPROVED
B	GENERAL REVISIONS	6/12/14	
A	PRELIMINARY DESIGN	5/22/14	

REVISIONS

PROJECT:

DAM SAFETY INSPECTION
PLANT CRISP CCW IMPOUNDMENT
WARWICK, GEORGIA

DRAWING TITLE:

INSPECTION REPORT SITE PLAN

PREPARED FOR:

CRISP COUNTY POWER COMMISSION
CORDELE, GA

RIZZO ASSOCIATES

CAD FILE NUMBER 14-5232-D400(M)

DRAWING NO. 1

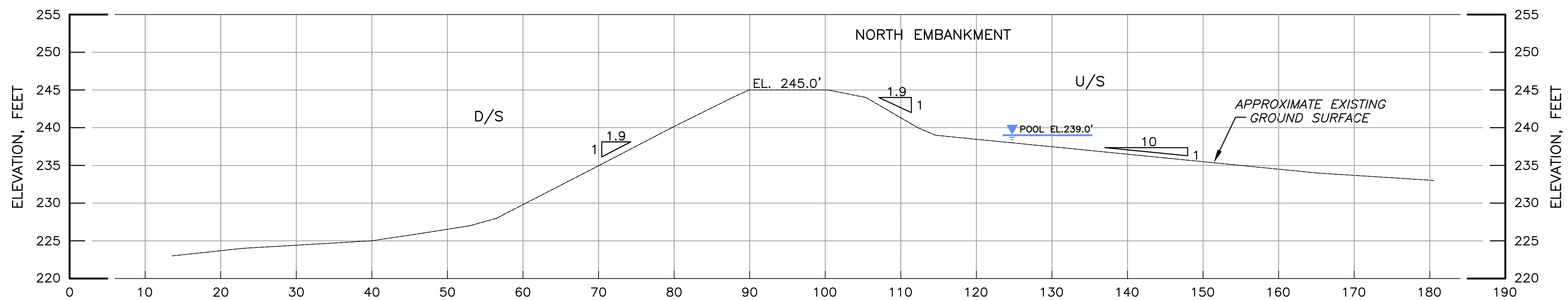
CAD FILE NUMBER 14-5232-D400(M)

CHECKED BY

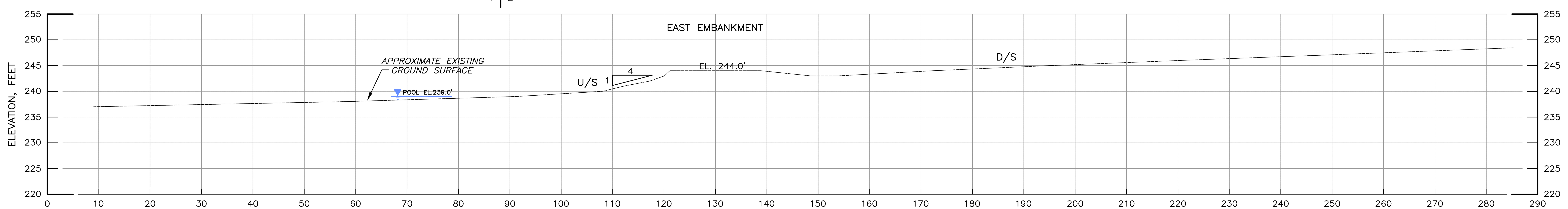
APPROVED BY

GPB 5/23/14

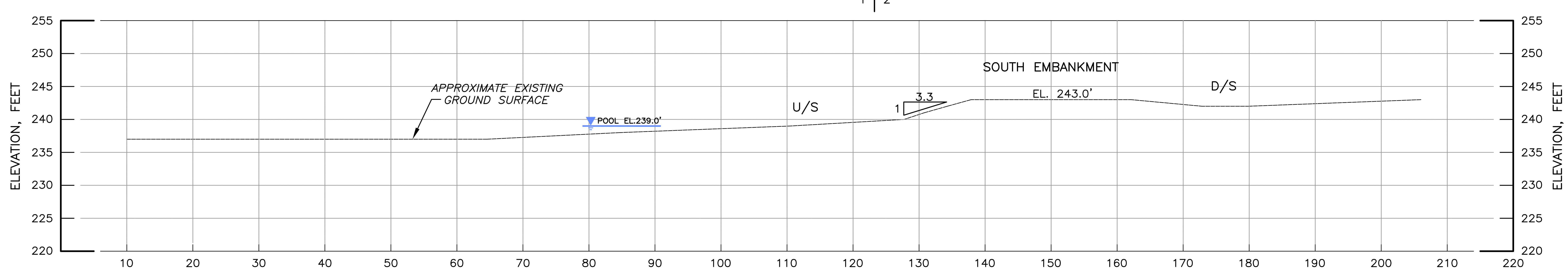
DRAWN BY



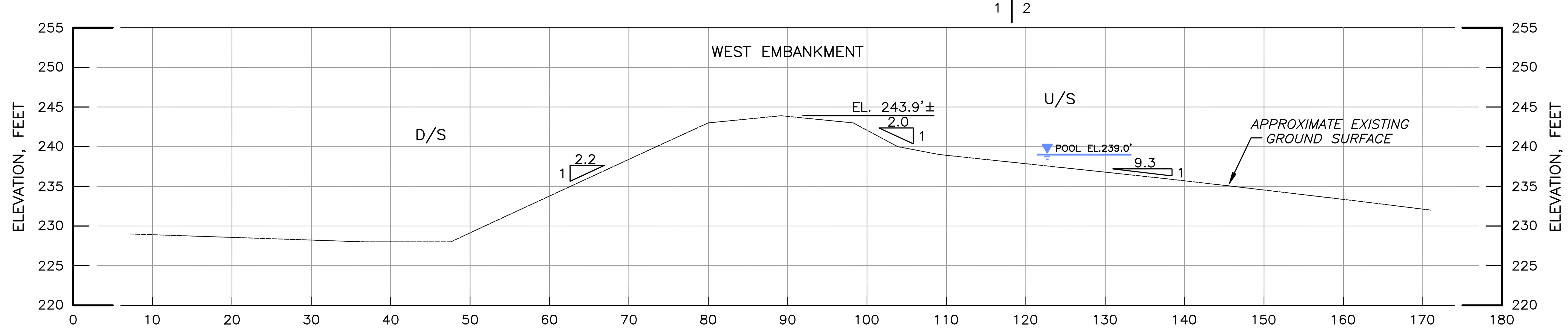
SECTION A
1 | 2



SECTION B
1 | 2



SECTION C
1 | 2



SECTION D
1 | 2

SCALE: 1"= 10'-0"
S C A L E
10 0 10 20 FEET

NOTE:

POOL ELEVATION AS OF THE DATE OF THIS SURVEY MAY 2 & MAY 5, 2014 = APPROX. ELEVATION 239'

NO.	DESCRIPTIONS	DATE	APPROVED
B	GENERAL REVISIONS	6/10/14	
A	PRELIMINARY DESIGN	5/22/14	

REVISIONS

PROJECT:

DAM SAFETY INSPECTION
PLANT CRISP CCW IMPOUNDMENT
WARWICK, GEORGIA

DRAWING TITLE:

SECTIONS A–D

PREPARED FOR:

CRISP COUNTY POWER COMMISSION
CORDELE, GA

RIZZO ASSOCIATES

CAD FILE NUMBER 14-5232-D400(M)

DRAWING NO. 2

APPENDIX C

BORING LOGS AND LABORATORY TEST DATA

Paul C. Rizzo Associates, Inc
101 Westpark Blvd Suite B
Columbia, SC
Telephone: 803-750-9773
Fax: 803-750-9116

BORING NUMBER N-1

CLIENT Crisp County Power Commision	**PROJECT NAME** CCW Impoundment Evaluation
PROJECT NUMBER 14-5232	**PROJECT LOCATION** Warwick, GA
DATE STARTED 5/1/14 **COMPLETED** 5/1/14	**GROUND ELEVATION** 245.2 ft **HOLE SIZE** 2 inches
DRILLING CONTRACTOR GEC	**GROUND WATER LEVELS:**
DRILLING METHOD Mud Rotary	**AT TIME OF DRILLING** ---
LOGGED BY CHG **CHECKED BY** JPO	**AT END OF DRILLING** ---
NOTES Borehole grouted 5/2/14	**14hrs AFTER DRILLING** 16.30 ft / Elev 228.90 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0		(sm) Loose, moist, orange, orange-brown silty sand with some clay (FILL)	SPT 1	100	2-1-2 (3)							
			SPT 2	100	1-3-6 (9)							
5		(sm) Loose, moist, orange, orange-brown silty sand with some clay (FILL)	SPT 3	100	2-7-9 (16)							
			SPT 4	100	6-10-15 (25)			9	19	14	5	
10		(SM) Medium Dense, moist, orange, orange-brown silty sand with some clay (FILL)	SPT 5	100	9-10-12 (22)			10				23
		(sp-sm) Medium Dense, moist, grey, grey-brown slightly silty to silty sand with trace organics (FILL)	SPT 6	100	3-9-9 (18)							
		(SM) Medium Dense, moist, grey, grey-brown silty sand with trace organics (FILL)	SPT 7	100	7-7-4 (11)			13				20
15		(sc-sm) Loose to Medium Dense, moist, dark grey silty clayey sand with trace organics (FILL)	SPT 8	100	6-6-7 (13)							
		(SC-SM) Loose to Medium Dense, moist to wet, dark grey silty clayey sand with trace organics (FILL)	SPT 9	100	4-3-3 (6)			15	41	13	28	
20		(sc-sm) Medium Dense, moist, dark grey silty clayey sand with trace organics (FILL)	SPT 10	100	4-7-6 (13)			10				28
		(sc-sm) Medium dense, moist, mottled grey, brown, and orange silty clayey sand	SPT 11	100	5-7-5 (12)							
			SPT 12	100	5-6-4 (10)							
25		(CH) Stiff light grey-blue clay with phosphitic pebbles and quartz gravels	SPT 13	100	4-4-7 (11)			23	63	20	43	
		(cl) Stiff light grey-blue clay with phosphitic pebbles and quartz gravels	SPT 14	100	3-4-8 (12)							
30			SPT 15	100	5-5-5 (10)							
			SPT 16	100	2-2-9 (11)							
		(cl) Hard white and light tan calcareous clay with limestone fragments and lenses (decomposed limestone)	SPT 17	100	22-24-32 (56)							
35			SPT 18	100	8-7-42 (49)			13				
			SPT 19	100	15-18-30 (48)							
40			SPT 20	100	14-17-19 (36)							

(Continued Next Page)

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BORING NUMBER N-1

CLIENT Crisp County Power Commision **PROJECT NAME** CCW Impoundment Evaluation

PROJECT NUMBER 14-5232 **PROJECT LOCATION** Warwick, GA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
40		(cl) Hard white and light tan calcareous clay with limestone fragments and lenses (decomposed limestone) *(continued)*	SPT 21	100	21-22-26 (48)							
			SPT 22	100	50/3"							
45			SPT 23	100	13-30-9 (39)			45				
		(CL) Stiff white and light tan calcareous clay with limestone fragments and lenses (decomposed limestone)	SPT 24	100	3-3-11 (14)			32	34	27	7	
			SPT 25	100	2-3-7 (10)			53				61

Bottom of borehole at 49.5 feet.

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BORING NUMBER N-2

CLIENT Crisp County Power Commision
PROJECT NAME CCW Impoundment Evaluation
PROJECT NUMBER 14-5232
PROJECT LOCATION Warwick, GA
DATE STARTED 5/15/14 **COMPLETED** 5/15/14
GROUND ELEVATION **HOLE SIZE** 2 inches
DRILLING CONTRACTOR GEC
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger 2"
AT TIME OF DRILLING 5.00 ft
LOGGED BY CHG **CHECKED BY** JPO
AT END OF DRILLING ---
NOTES Backfilled with bentonite chips 5/15
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0		(sc-sm) Very loose to loose, moist, mottled grey, brown, and orange silty clayey sand	SPT 1	100	3-2-3 (5)							
			SPT 2	100	1-1-1 (2)							
5		(sc-sm) Very loose to loose, wet, mottled grey, brown, and orange silty clayey sand	SPT 3	100	1-1-2 (3)							
			SPT 4	100	1-3-3 (6)							
10		(cl) Hard white and light tan calcareous clay with limestone fragments and lenses (decomposed limestone)	SPT 5	100	1-3-5 (8)							
			SPT 6	100	6-12-28 (40)							
			SPT 7	100	12-50							
15			SPT 8	100	12-50/5"							
		(cl) Very stiff white and light tan calcareous clay with limestone fragments and lenses (decomposed limestone)	SPT 9	100	4-10-12 (22)							
20		(cl) Hard light blueish gray clay with limestone fragments and lenses (decomposed limestone)	SPT 10	100	18-50							
			SPT 11	100	7-12-23 (35)							
			SPT 12	100	16-50/3"							

Bottom of borehole at 24.5 feet.

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BORING NUMBER W-1

CLIENT Crisp County Power Commision
PROJECT NAME CCW Impoundment Evaluation
PROJECT NUMBER 14-5232
PROJECT LOCATION Warwick, GA
DATE STARTED 5/1/14 **COMPLETED** 5/1/14
GROUND ELEVATION 243.9 ft **HOLE SIZE** 2 inches
DRILLING CONTRACTOR GEC
GROUND WATER LEVELS:
DRILLING METHOD Mud Rotary
AT TIME OF DRILLING ---
LOGGED BY CHG **CHECKED BY** JPO
AT END OF DRILLING ---
NOTES Borehole grouted 5/2/14, topped with bentonite chips on 5/15
▽ **19hrs AFTER DRILLING** 15.10 ft / Elev 228.80 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(sm) Loose to medium dense, moist, orange, Brown, Grey Silty Sand with gravels (FILL)	SPT 1	100	2-2-3 (5)							
			SPT 2	100	7-9-13 (22)							
5		(SM) Medium dense, moist, orange, Brown, Grey Silty Sand with gravels (FILL)	SPT 3	100	7-7-9 (16)			13				24
		(sm) Medium dense, moist, orange, Brown, Grey Silty Sand with gravels (FILL)	SPT 4	100	11-9-8 (17)							
			SPT 5	100	2-4-9 (13)							
10		(sm) Medium dense, moist, dark Grey Silty Sand with trace organics (wood and root fibers) (FILL)	SPT 6	100	7-10-14 (24)							
		(SM) Medium dense, moist, dark Grey Silty Sand with trace organics (wood and root fibers) (FILL)	SPT 7	100	8-8-11 (19)			10				28
15	▽	(sm) Medium dense, moist, dark Grey Silty Sand with trace organics (wood and root fibers) (FILL)	SPT 8	100	9-8-9 (17)							
			SPT 9	100	8-9-10 (19)							
			SPT 10	100	4-6-7 (13)							
20		(SM) Loose, moist, dark Grey Silty Sand with trace organics (wood and root fibers) (FILL)	SPT 11	100	3-4-4 (8)			13				27
		(SC-SM) Very Loose, moist, dark Grey Silty Clayey Sand with trace organics (wood and root fibers)	SPT 12	100	2-1-3 (4)			16				30
25		(CH) Stiff Light Grey-Blue Clay (Marl)	SPT 13	100	4-6-9 (15)			18	61	19	42	
		(cl) Stiff White and Light Tan Calcareous Clay with Limestone fragments and lenses (decomposed limestone)	SPT 14	100	4-6-7 (13)							
		(cl) Very stiff to hard White and Light Tan Calcareous Clay with Limestone fragments and lenses (decomposed limestone)	SPT 15	100	6-10-13 (23)							
30			SPT 16	100	12-25-24 (49)							
			SPT 17	100	17-15-20 (35)							
35			SPT 18	100	12-50/3"							
			SPT 19	100	14-13-19 (32)			29				57
40			SPT 20	100	10-20-50/1"							

(Continued Next Page)

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BORING NUMBER W-1

CLIENT Crisp County Power Commision
PROJECT NAME CCW Impoundment Evaluation
PROJECT NUMBER 14-5232
PROJECT LOCATION Warwick, GA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTIC LIMIT	ATTERBERG LIMITS PLASTICITY INDEX	FINES CONTENT (%)
40		(cl) Very stiff to hard White and Light Tan Calcareous Clay with Limestone fragments and lenses (decomposed limestone) *(continued)*	SPT 21	100	9-16-21 (37)							
		(cl) Stiff to very stiff White and Light Tan Calcareous Clay with Limestone fragments and lenses (decomposed limestone)	SPT 22	100	9-6-4 (10)							
45			SPT 23	100	1-2-9 (11)							
			SPT 24	100	4-3-13 (16)							
		(cl) Hard White and Light Tan Calcareous Clay with Limestone fragments and lenses (decomposed limestone)	SPT 25	100	50/2"							

Bottom of borehole at 49.5 feet.

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BORING NUMBER W-2

CLIENT Crisp County Power Commision
PROJECT NAME CCW Impoundment Evaluation
PROJECT NUMBER 14-5232
PROJECT LOCATION Warwick, GA
DATE STARTED 5/2/14 **COMPLETED** 5/2/14
GROUND ELEVATION 228.64 ft **HOLE SIZE** 2 inches
DRILLING CONTRACTOR GEC
GROUND WATER LEVELS:
DRILLING METHOD Mud Rotary
AT TIME OF DRILLING ---
LOGGED BY CHG **CHECKED BY** JPO
AT END OF DRILLING ---
NOTES Borehole grouted 5/2/14
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0	(sc) Red-orange, moist, silty clayey sand roadbase (Hand Auger to 4')										
5	(sm) Medium dense, moist, dark grey organic slightly silty to silty sand	SPT 1	100	4-8-7 (15)							
	(sc-sm) Very loose, moist, grey-brown mottled orange silty clayey sand with trace organics	SPT 2	100	2-1-2 (3)							
10	(SC-SM) Medium dense, moist, grey-brown mottled orange silty clayey sand with trace organics	SPT 3	100	4-5-6 (11)			15				45
	(sc) Dense, moist, light grey-blue clayey sand with limestone fragments and phosphitic pebbles	SPT 4	100	6-9-24 (33)							
	(SC) Medium dense, moist, light grey-blue clayey sand with limestone fragments and phosphitic pebbles	SPT 5	100	5-6-6 (12)			15	46	15	31	44
15	(sc) Loose light grey-blue clayey sand with limestone fragments and phosphitic pebbles	SPT 6	100	3-4-4 (8)							
	(cl) Hard white and light tan calcareous clay with limestone fragments and sand (decomposed limestone)	SPT 7	100	20-20-17 (37)							
	(CL) Hard white and light tan calcareous clay with limestone fragments and sand (decomposed limestone)	SPT 8	100	20-18-50/4"			22				65
20	(cl) Hard white and light tan calcareous clay with limestone fragments and sand (decomposed limestone)	SPT 9	100	12-17-20 (37)							
		SPT 10	100	50/3"							
25	(CL) Very stiff white and light tan calcareous clay with limestone fragments and sand (decomposed limestone)	SPT 11	100	4-9-14 (23)			28				58
	(cl) Hard white and light tan calcareous clay with limestone fragments and sand (decomposed limestone)	SPT 12	100	12-28-19 (47)							
		SPT 13	100	11-50/5"							
30		SPT 14	100	9-13-16 (29)							
		SPT 15	100	25-50/1"							
35		SPT 16	100	16-9-27 (36)							

Bottom of borehole at 35.5 feet.

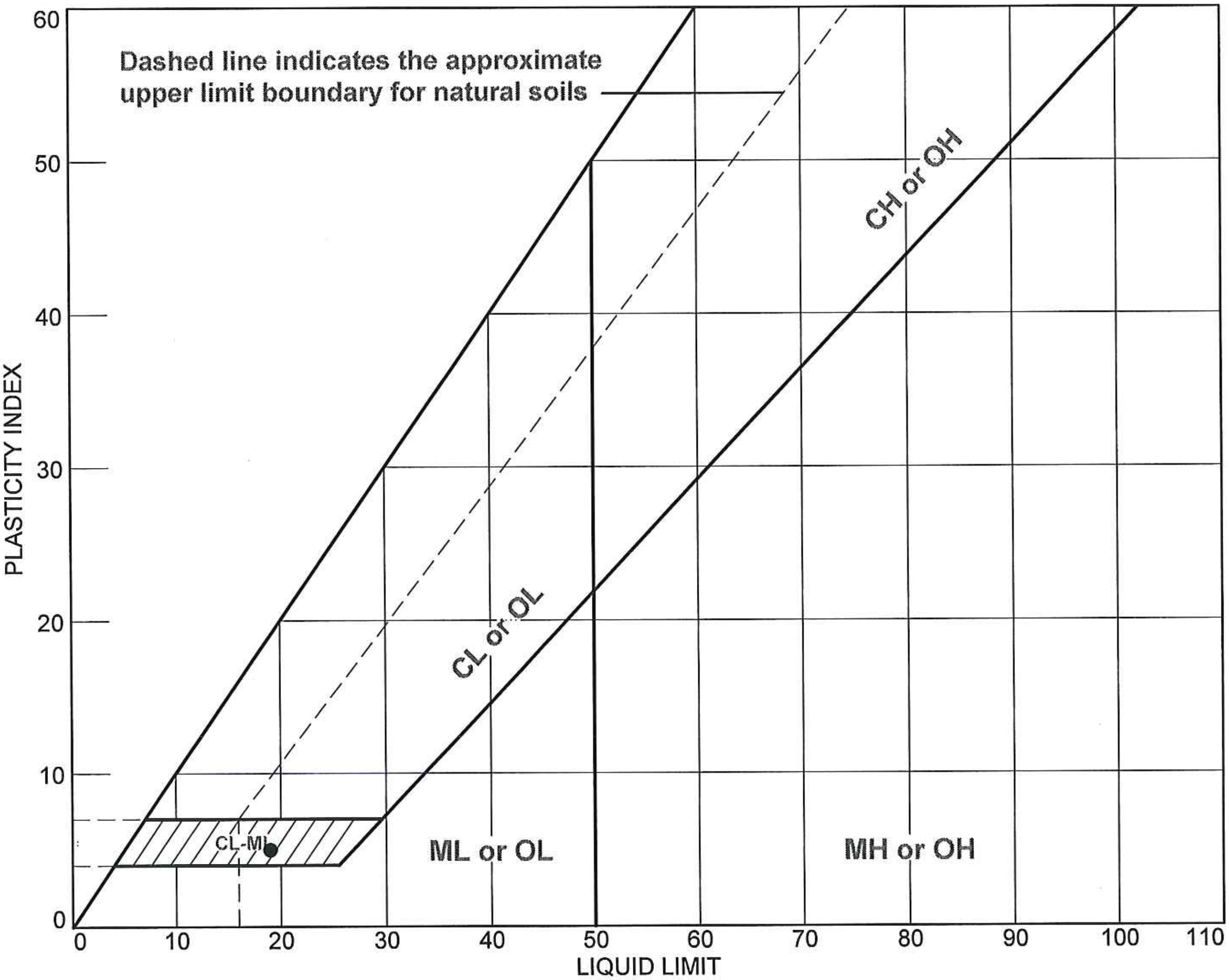
Geotechnical & Environmental Consultants, Inc.

514 Hillcrest Industrial Blvd.
Macon, Ga 31204
Summary of Results for
CCW Impoundment Evaluation
140287.220

Sample Identification	Depth (ft)	Moisture Content (%)	Atterberg Limits			% Passing			Organic Content	Max Dry Density (pcf	Optimum Moisture	USCS Class.
			L.L.	P.L.	P.I.	+ 10	+ 40	+200				
N-1, #4	6-7.5	9.3%	19.0	14.0	5.0	*	*	*	*	*	*	*
N-1, #5	8-9.5	9.5%	*	*	*	94.2	65.2	22.5	*	*	*	SM
N-1, #7	12-13.5	13.0%	*	*	*	*	*	19.5	*	*	*	*
N-1, #9	16-17.5	15.0%	*	*	*	*	*	*	2.8%	*	*	*
N-1, #10	18-19.5	10.0%	*	*	*	98.6	70.3	28.2	*	*	*	SM
N-1, #13	24-25.5	23.1%	63.0	20.0	43.0	*	*	*	*	*	*	*
N-1, #14	16-17.5	13.4%	41.0	13.0	28.0	*	*	*	*	*	*	*
N-1, #18	34-35.5	23.8%	*	*	*	*	*	56.9	*	*	*	*
N-1, #23	44-45.5	44.5%	*	*	*	*	*	50.9	*	*	*	*
N-1, #24	46-47.5	32.3%	34.0	27.0	7.0	*	*	*	*	*	*	*
N-1, #25	48-49.5	53.3%	*	*	*	100.0	98.4	60.9	*	*	*	ML
W-1, #3	4-5.5	12.5%	*	*	*	95.4	60.0	24.2	*	*	*	SM
W-1, #7	12-13.5	9.8%	*	*	*	96.8	70.5	27.5	*	*	*	SM
W-1, #11	20-21.5	13.1%	*	*	*	*	*	27.3	*	*	*	*
W-1, #12	22-23.5	15.5%	*	*	*	*	*	30.0	3.0%	*	*	*
W-1, #13	24-25.5	17.7%	61.0	19.0	42.0	*	*	*	*	*	*	*
W-1, #19	36-37.5	28.7%	*	*	*	*	*	57.3	*	*	*	*
W-2, #3	8-9.5	15.1%	*	*	*	*	*	45.3	*	*	*	*
W-2, #5	12-13.5	14.7%	46.0	15.0	31.0	*	*	44.3	*	*	*	SC
W-2, #8	18-19.5	21.8%	*	*	*	*	*	64.7	*	*	*	*
W-2, #11	24-24.5	28.2%	*	*	*	*	*	57.8	*	*	*	*

(*) Test not requested.

LIQUID AND PLASTIC LIMITS TEST REPORT



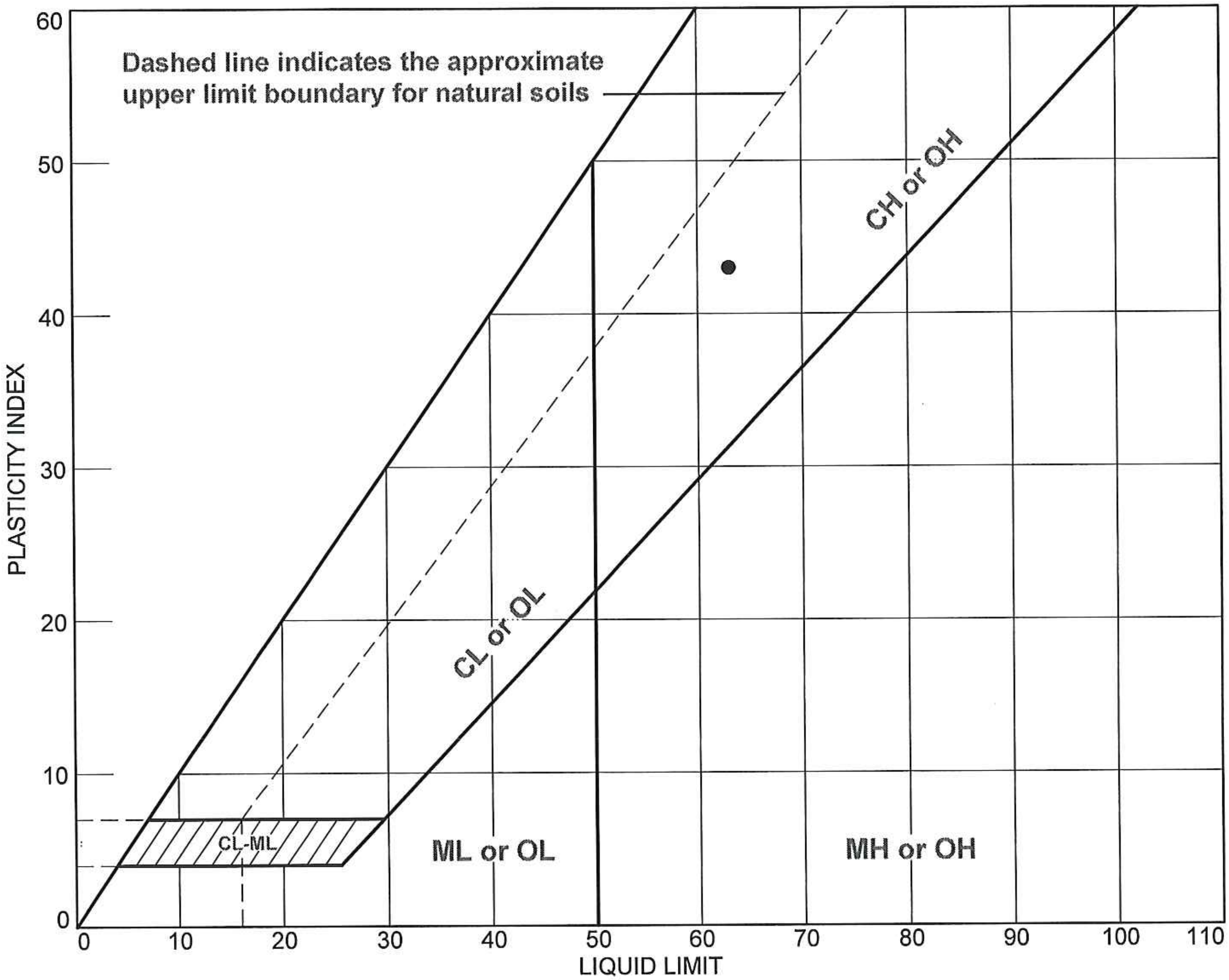
SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		N-1, #4	6-7.5'	9.3	14	19	5	

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation

Project No.: 140287.220 **Figure**

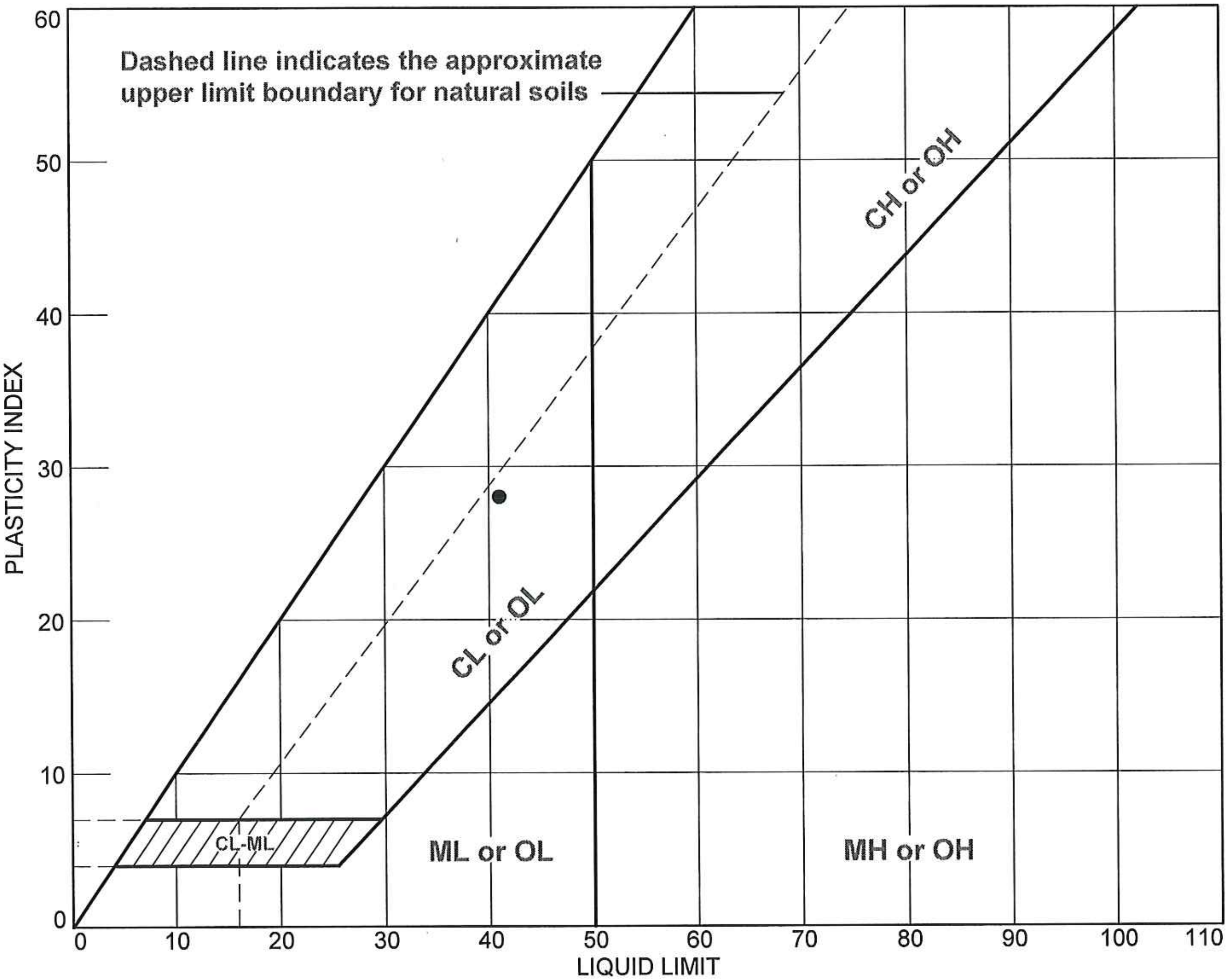
LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		N-1, #13	24-25.5'	23.1	20	63	43	

Geotechnical and Environmental Consultants	**Client:** Paul C. Rizzo Associates, Inc. **Project:** CCW Impoundment Evaluation **Project No.:** 140287.220 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



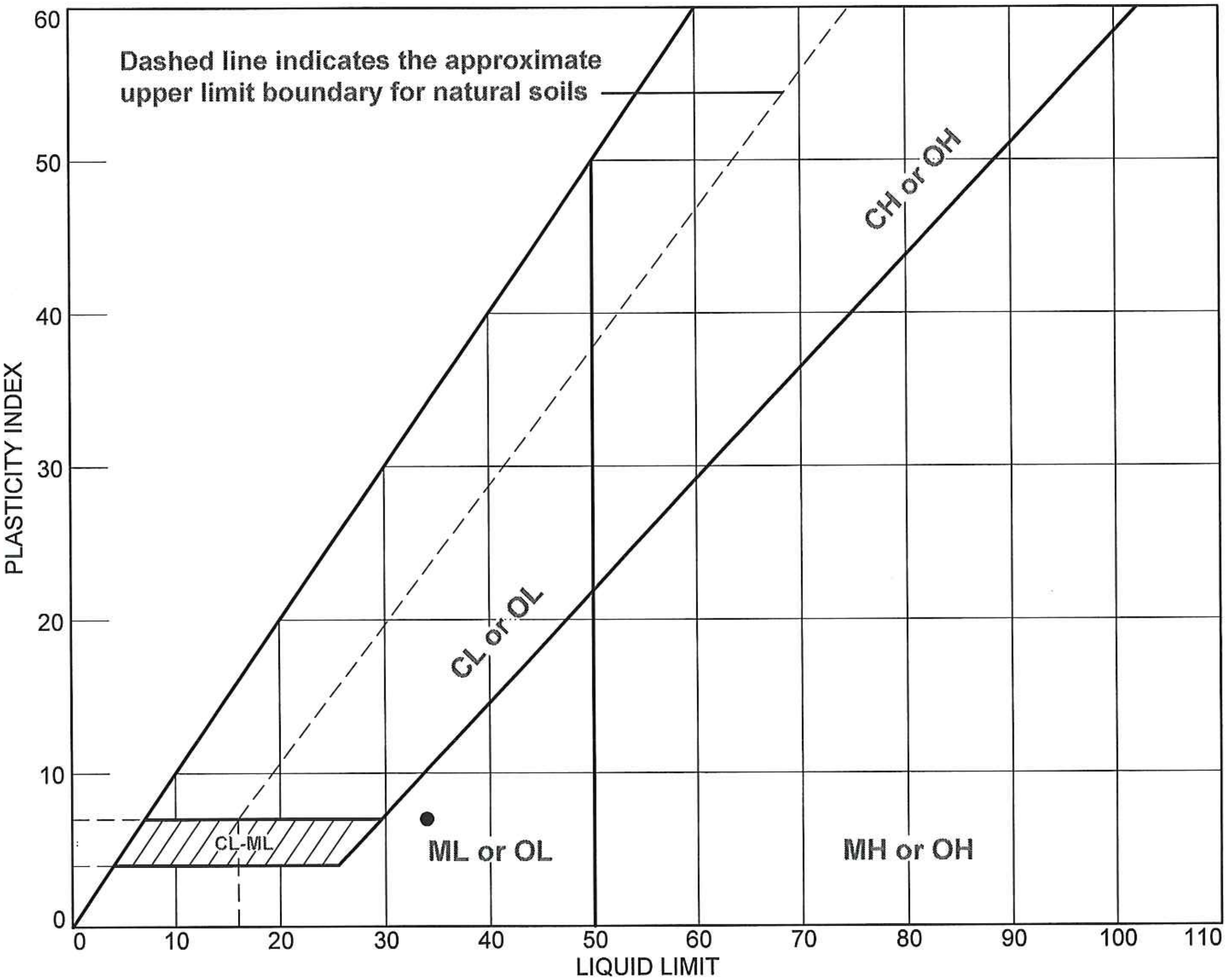
SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		N-1, #14	16-17.5'	13.4	13	41	28	

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation

Project No.: 140287.220 **Figure**

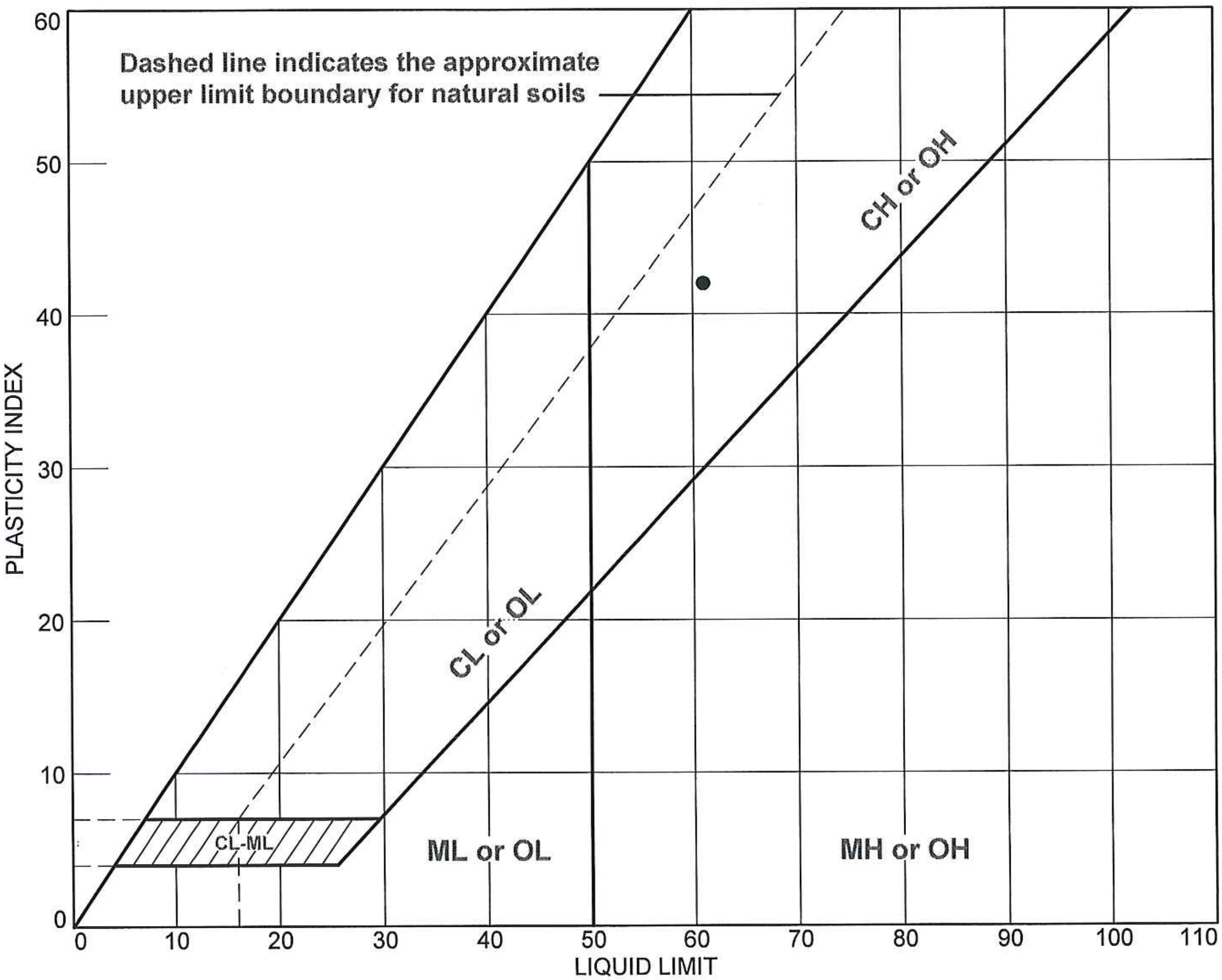
LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		N-1, #24	46-47.5'	32.3	27	34	7	

Geotechnical and Environmental Consultants	**Client:** Paul C. Rizzo Associates, Inc. **Project:** CCW Impoundment Evaluation **Project No.:** 140387.220 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT

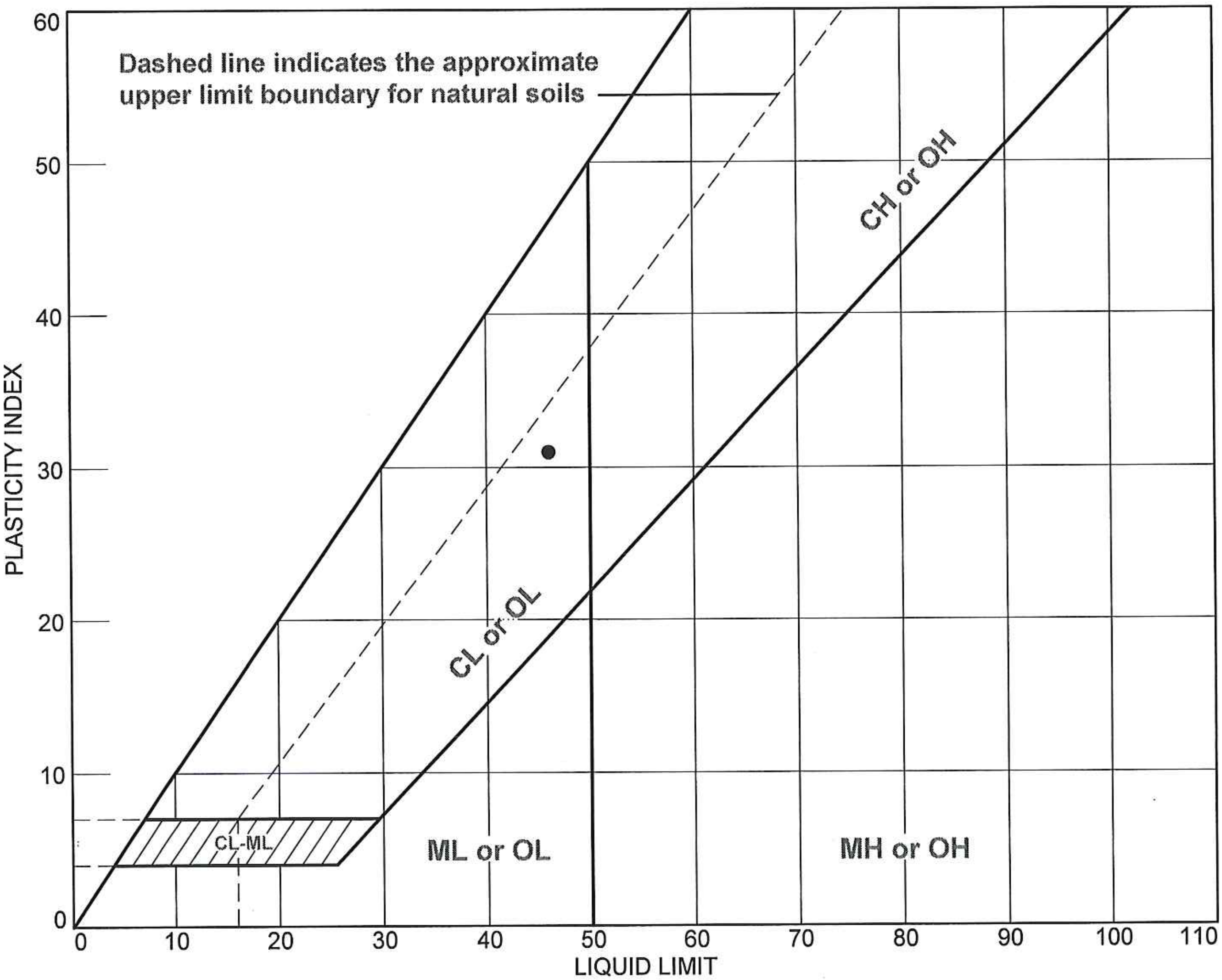


SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		W-1, #13	24-25.5'	17.7	19	61	42	

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation
Project No.: 140287.220 **Figure**

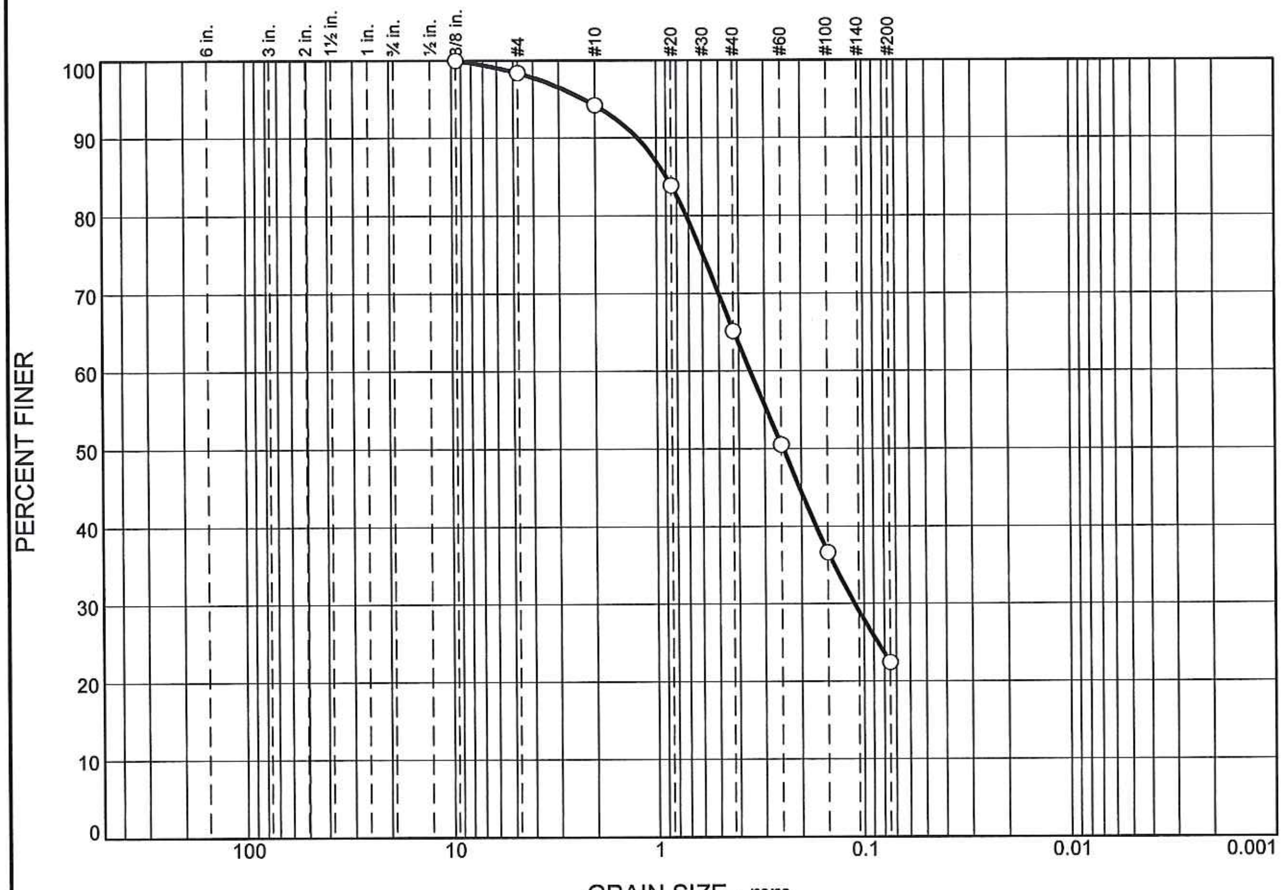
LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		W-2, #5	12-13.5'	14.7	15	46	31	

Geotechnical and Environmental Consultants	**Client:** Paul C. Rizzo Associates, Inc. **Project:** CCW Impoundment Evaluation **Project No.:** 140287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.6	4.2	29.0	42.7	22.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375	100.0		
#4	98.4		
#10	94.2		
#20	83.9		
#40	65.2		
#60	50.5		
#100	36.7		
#200	22.5		

* (no specification provided)

Material Description

Brown tan silty c-f SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.2373 D_{85}= 0.8974 D_{60}= 0.3535
D_{50}= 0.2455 D_{30}= 0.1113 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

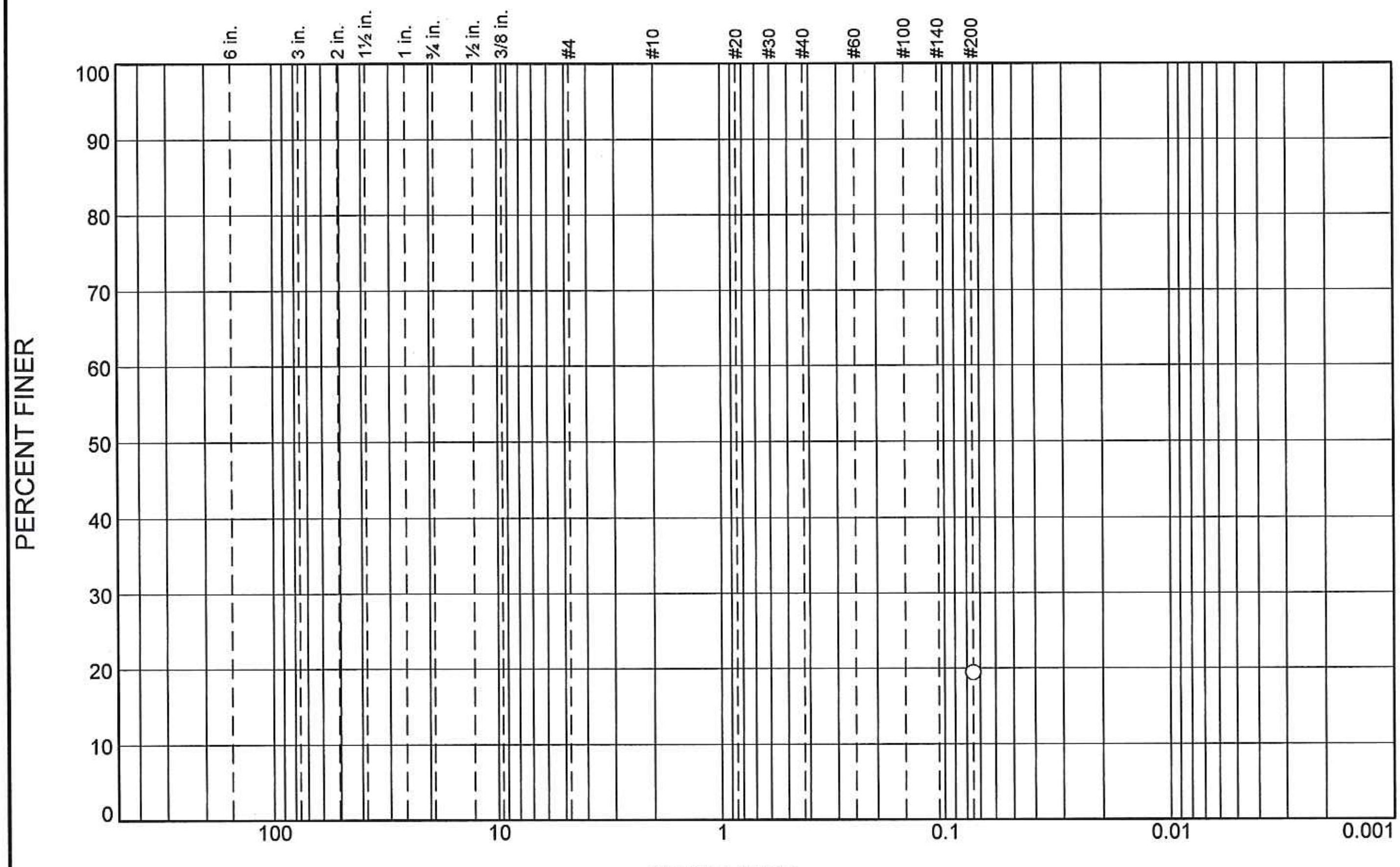
Sample Number: N-1, #5 **Depth:** 8-9.5'

Date: 5/13/14

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation
Project No: 140287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						19.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	19.5		

* (no specification provided)

Material Description

Brown tan

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

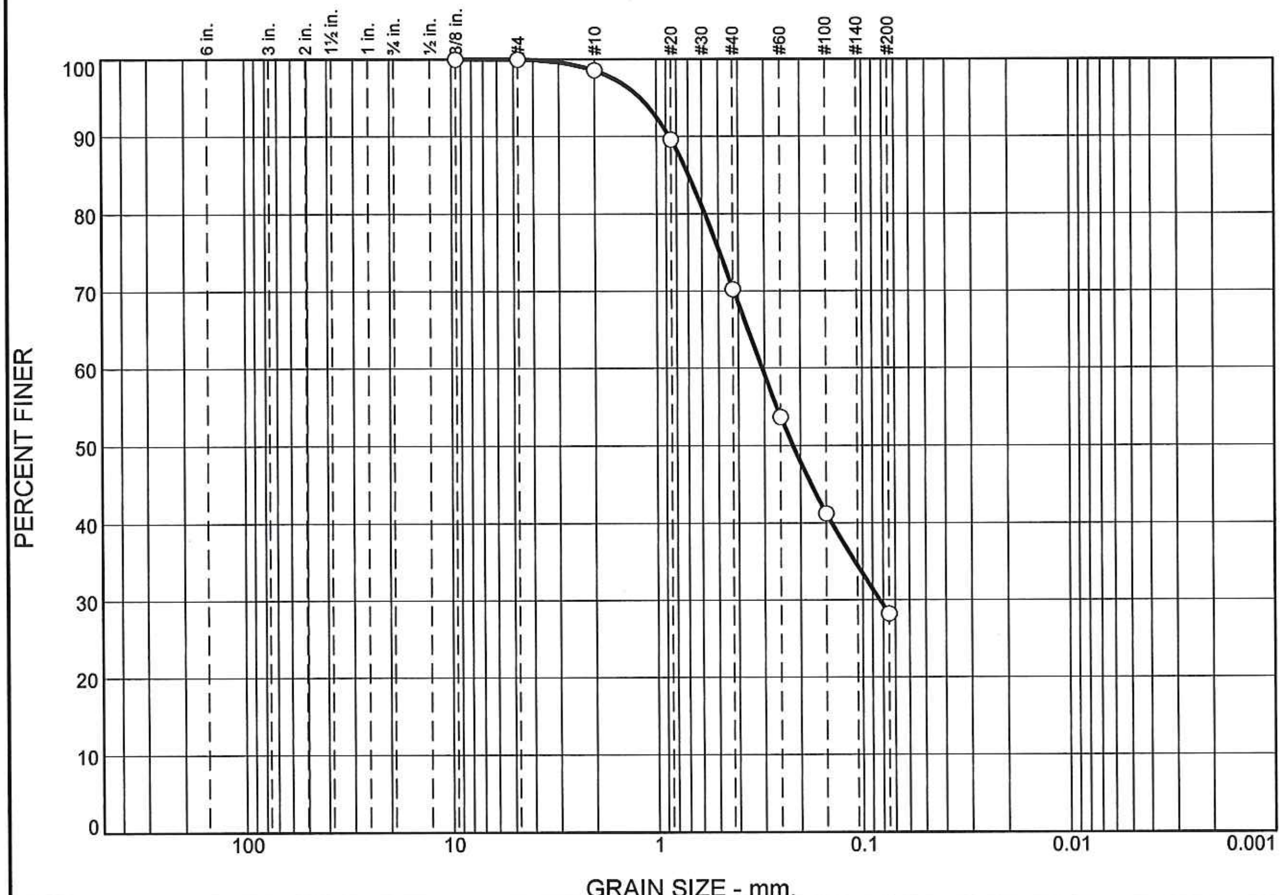
Sample Number: N-1, #7 **Depth:** 12-13.5'

Date: 5/15/14

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation
Project No.: 140287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	1.4	28.3	42.1	28.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375	100.0		
#4	100.0		
#10	98.6		
#20	89.6		
#40	70.3		
#60	53.7		
#100	41.2		
#200	28.2		

* (no specification provided)

Material Description

Gray black silty m-f SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.8667 D_{85}= 0.6968 D_{60}= 0.3087
D_{50}= 0.2183 D_{30}= 0.0829 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

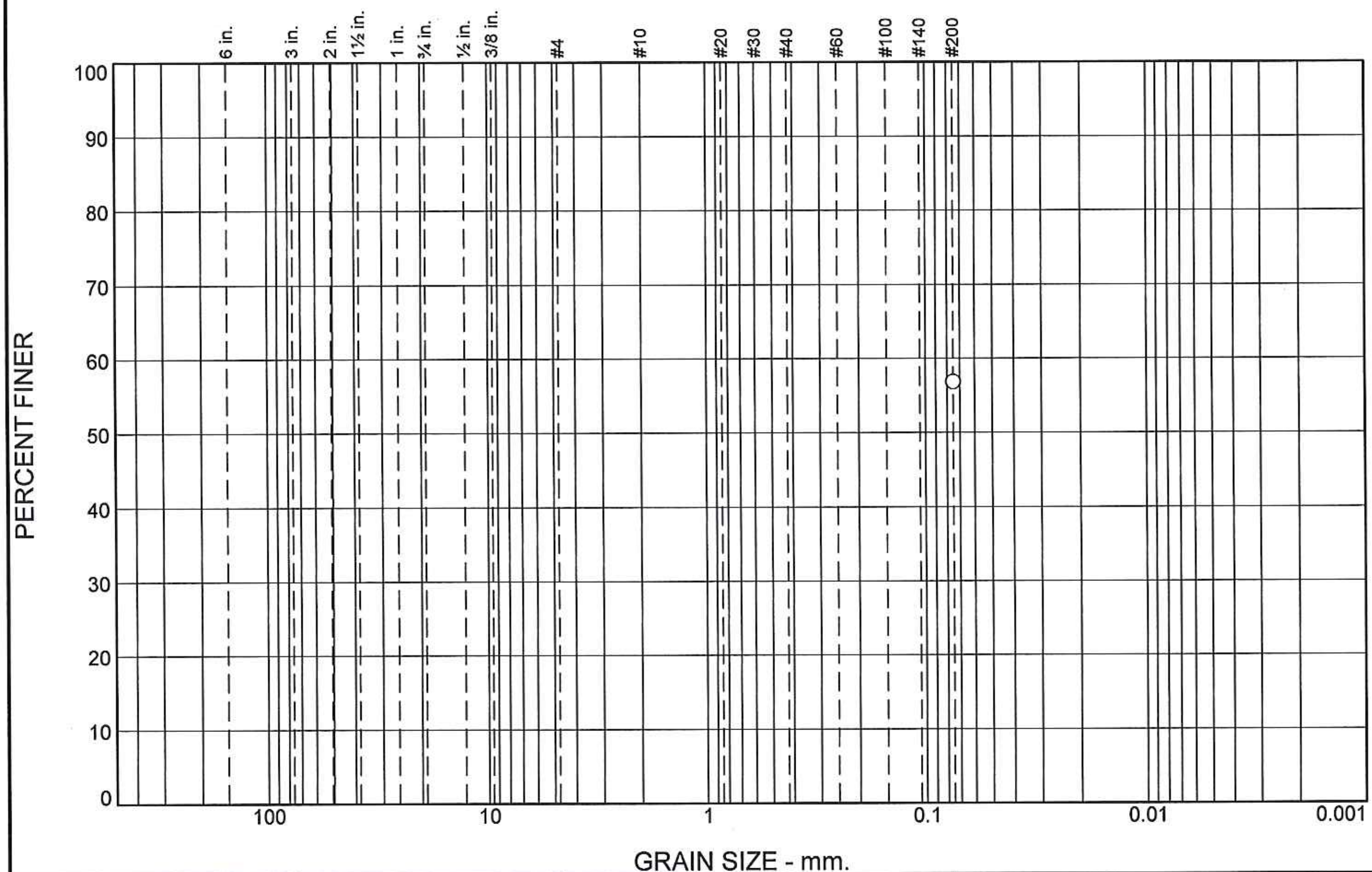
Sample Number: N-1, #10 **Depth:** 18-19.5'

Date: 5/14/14

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation
Project No: 140287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						56.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	56.9		

* (no specification provided)

Material Description

White

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample Number: N-1, #18 **Depth:** 34-35.5'

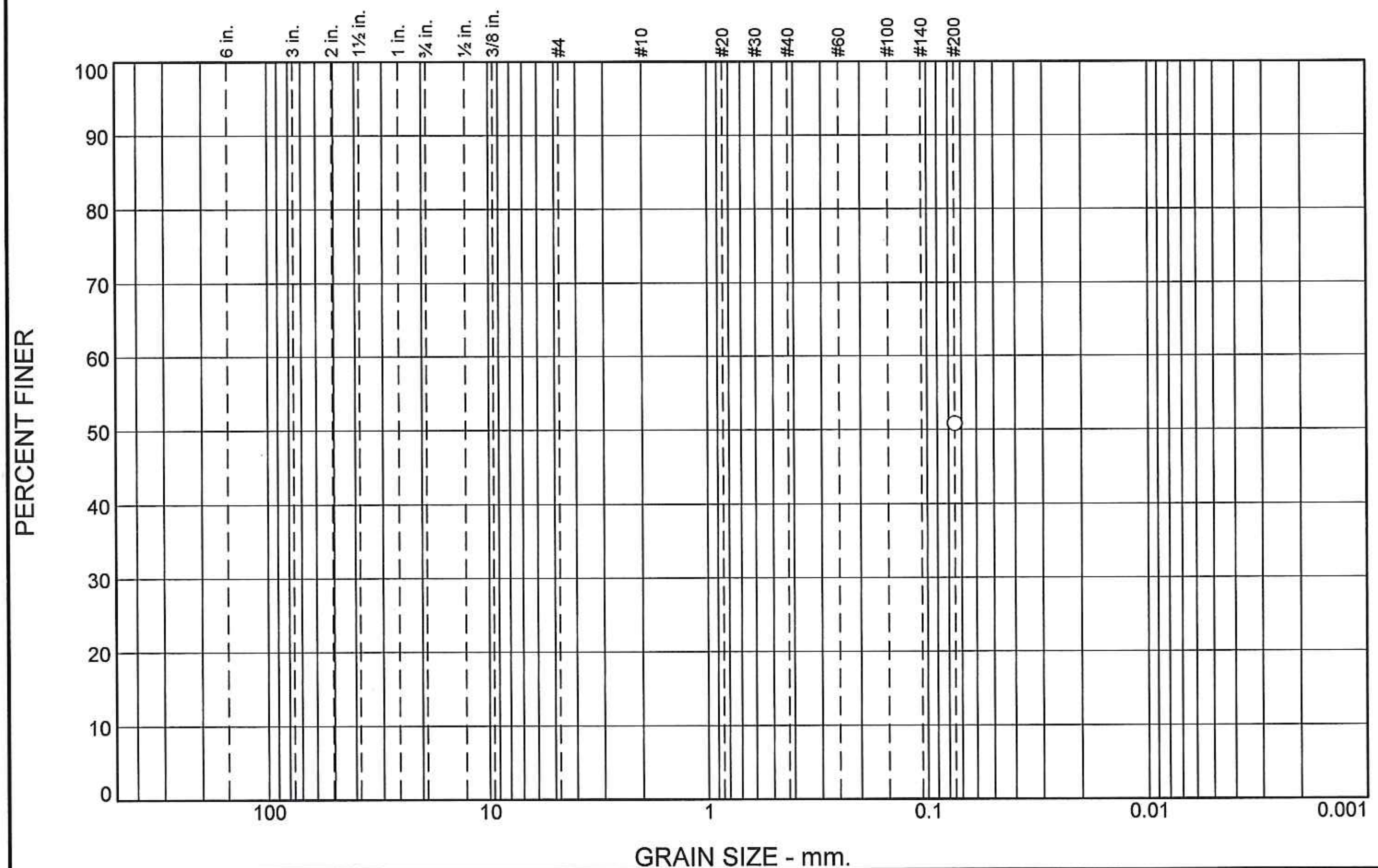
Date: 5/16/14

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation

Project No: 140287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						50.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	50.9		

* (no specification provided)

Material Description

White

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

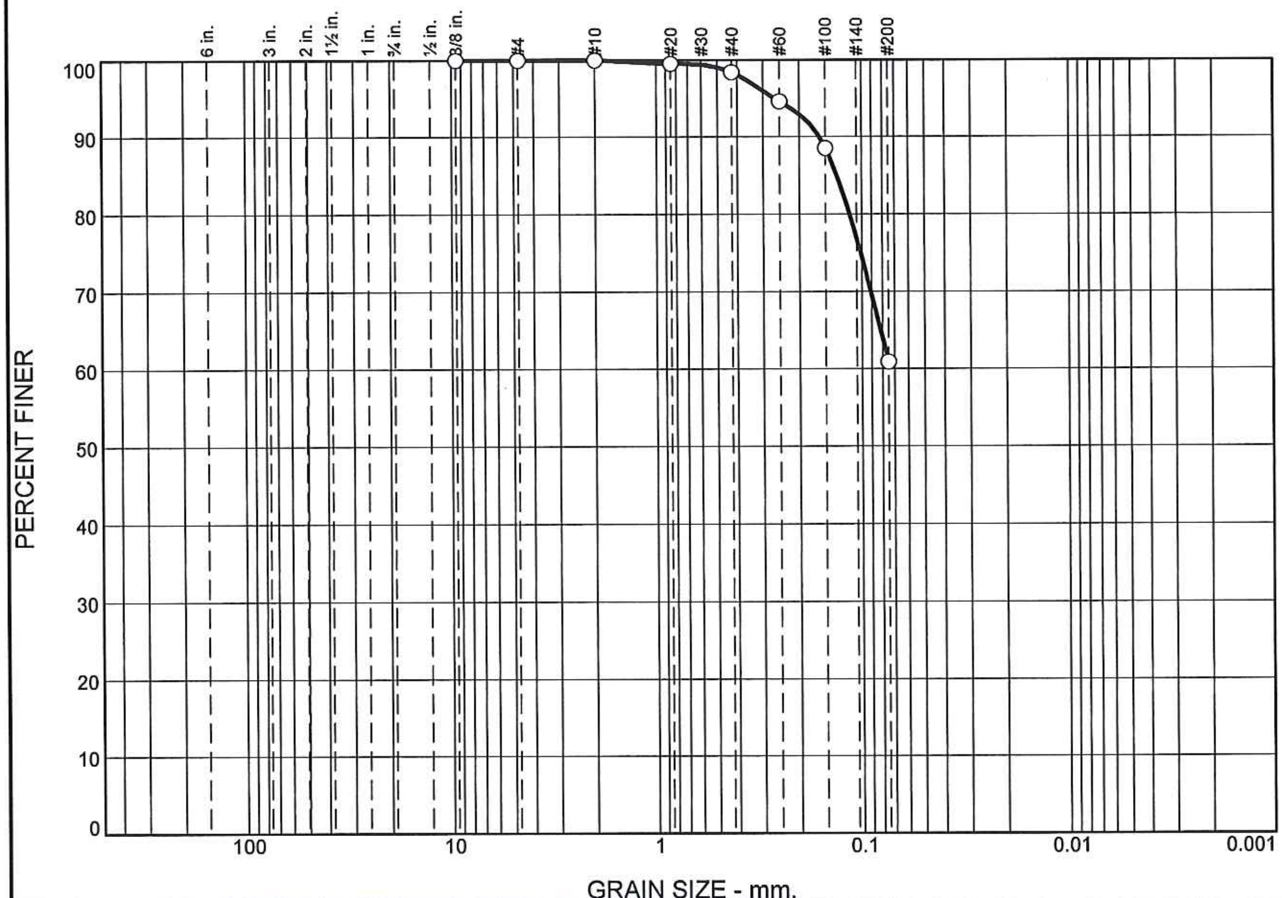
Sample Number: N-1. #23 **Depth:** 44-45.5'

Date: 5/16/14

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation
Project No: 140287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	1.6	37.5	60.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375	100.0		
#4	100.0		
#10	100.0		
#20	99.6		
#40	98.4		
#60	94.6		
#100	88.5		
#200	60.9		

* (no specification provided)

Material Description

Off white fine sandy SILT trace of clay

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1608 D_{85}= 0.1320 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

Sample Number: N-1, #25 **Depth:** 48-49.5'

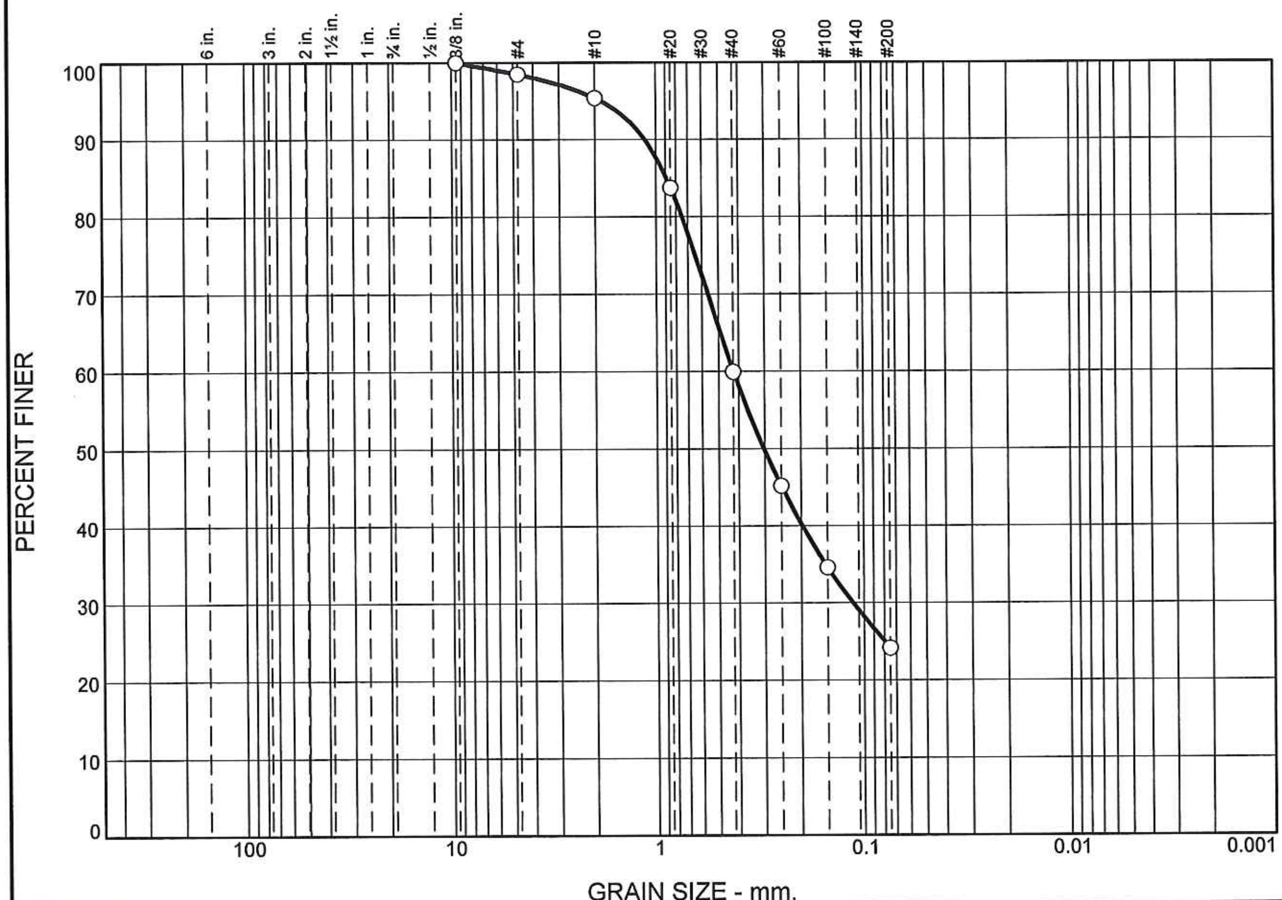
Date: 5/15/14

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation

Project No: 140287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.5	3.1	35.4	35.8	24.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375	100.0		
#4	98.5		
#10	95.4		
#20	83.8		
#40	60.0		
#60	45.2		
#100	34.6		
#200	24.2		

* (no specification provided)

Material Description

Orange brown silty c-f SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.1382 D_{85}= 0.8903 D_{60}= 0.4256
D_{50}= 0.3036 D_{30}= 0.1130 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample Number: W-1, #3 **Depth:** 4-5.5'

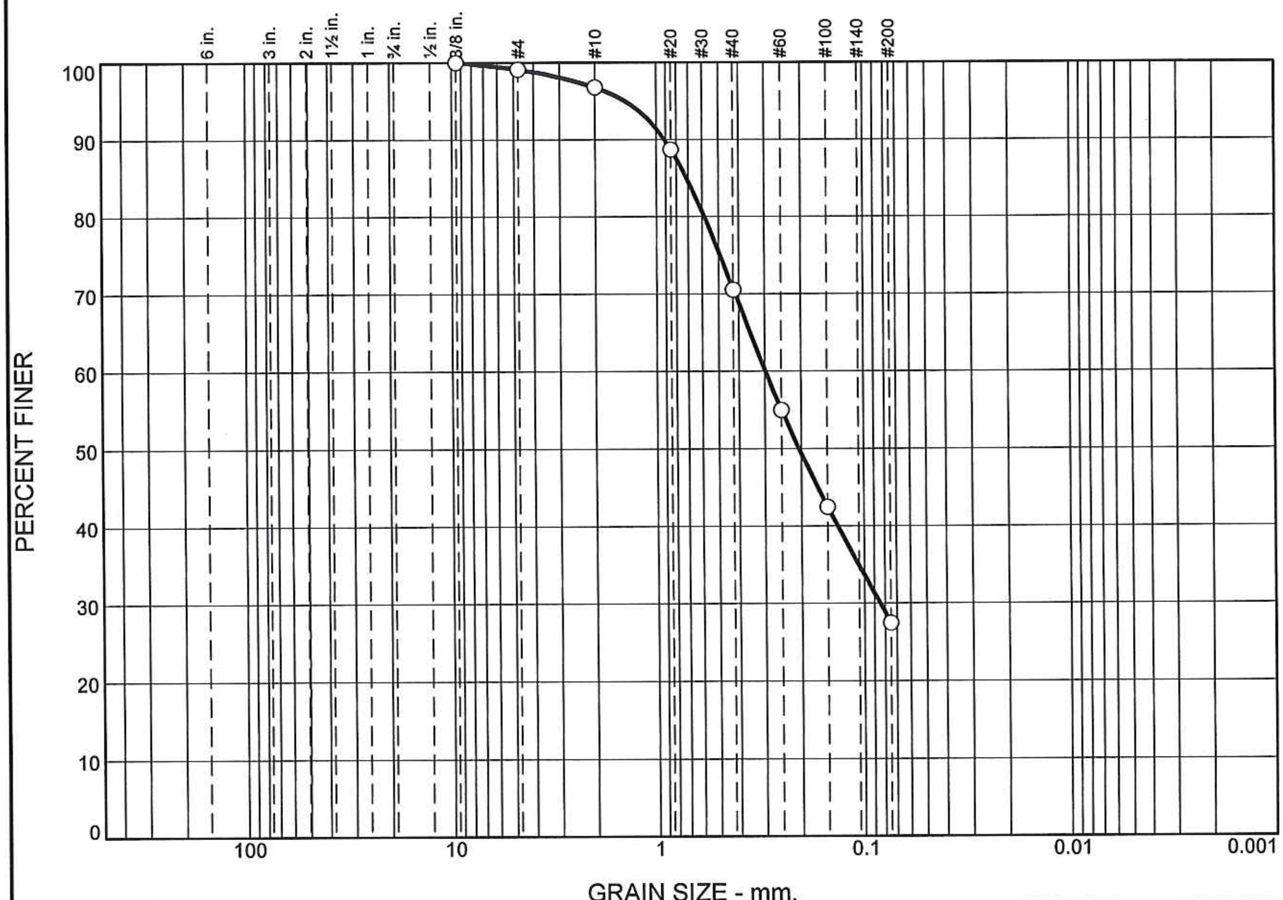
Date: 5/19/14

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation

Project No: 140287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.9	2.3	26.3	43.0	27.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375	100.0		
#4	99.1		
#10	96.8		
#20	88.8		
#40	70.5		
#60	55.0		
#100	42.5		
#200	27.5		

* (no specification provided)

Material Description

Gray brown orange silty m-f SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.9119 D_{85}= 0.7133 D_{60}= 0.2986
D_{50}= 0.2061 D_{30}= 0.0845 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample Number: W-1, #7 **Depth:** 12-13.5'

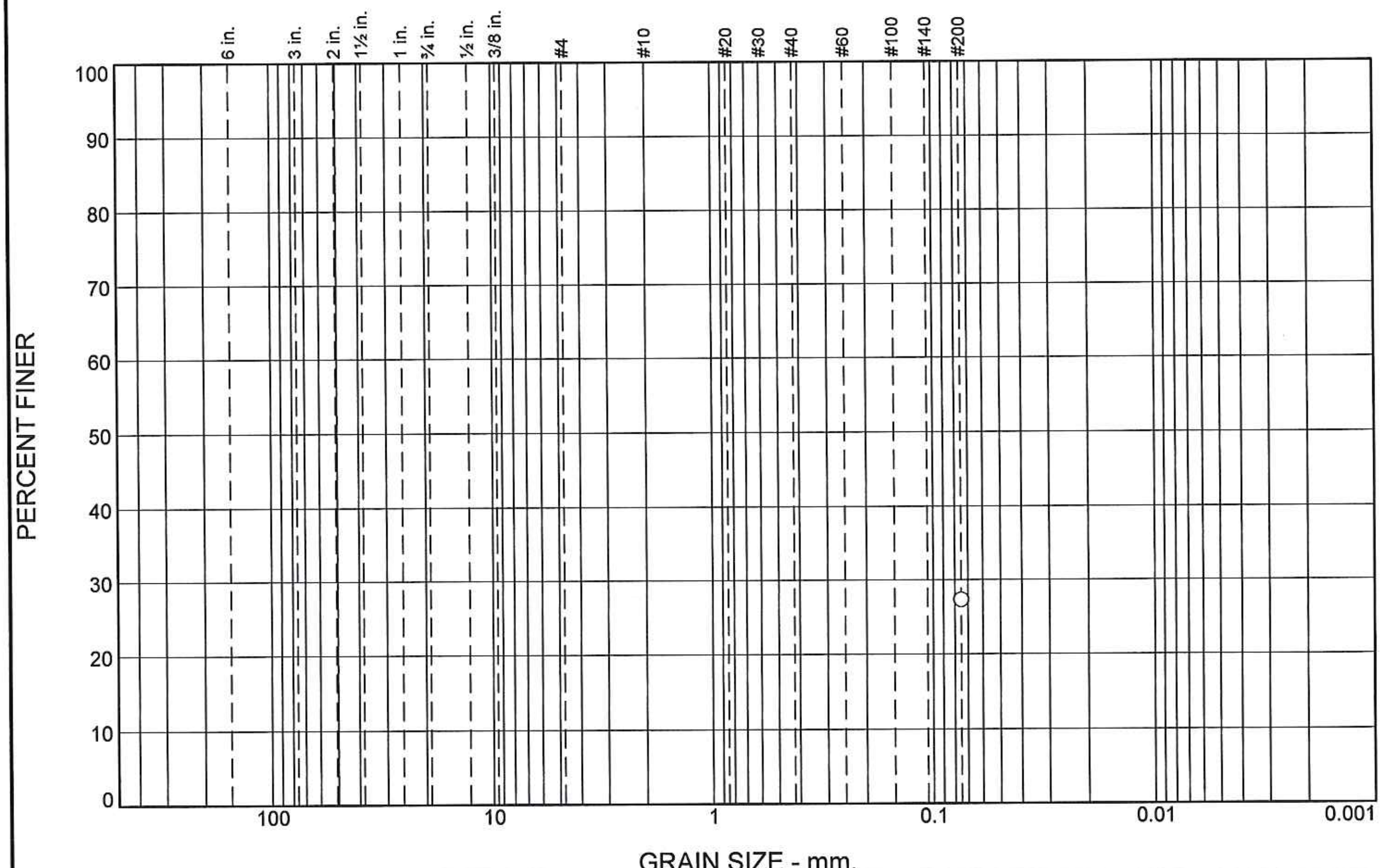
Date: 5/19/14

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation

Project No: 140287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						27.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	27.3		

* (no specification provided)

Material Description

Brown black orange

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample Number: W-1, #11 **Depth:** 20-21.5'

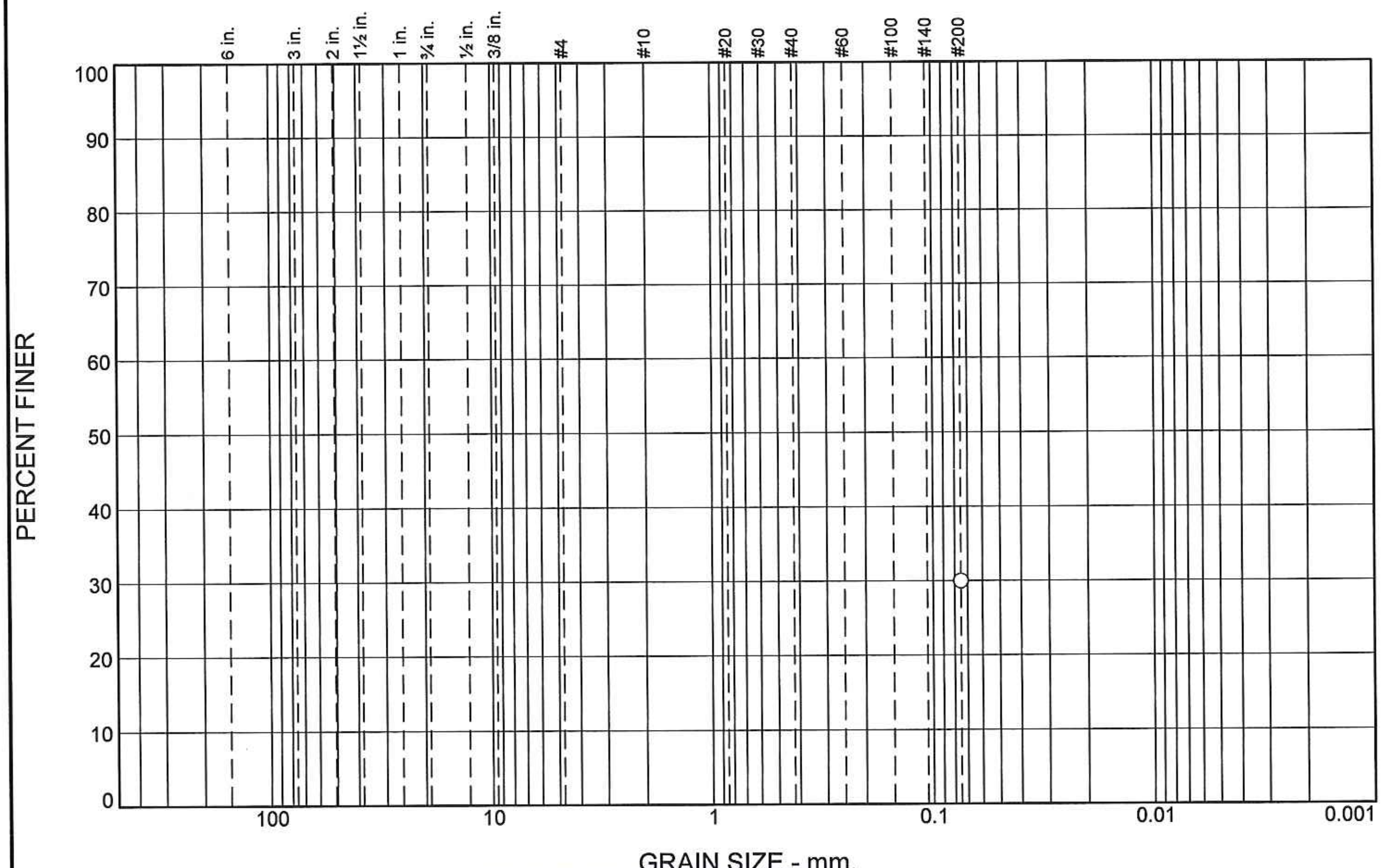
Date: 5/19/14

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation

Project No: 140287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						30.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	30.0		

* (no specification provided)

Material Description

Gray brown

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

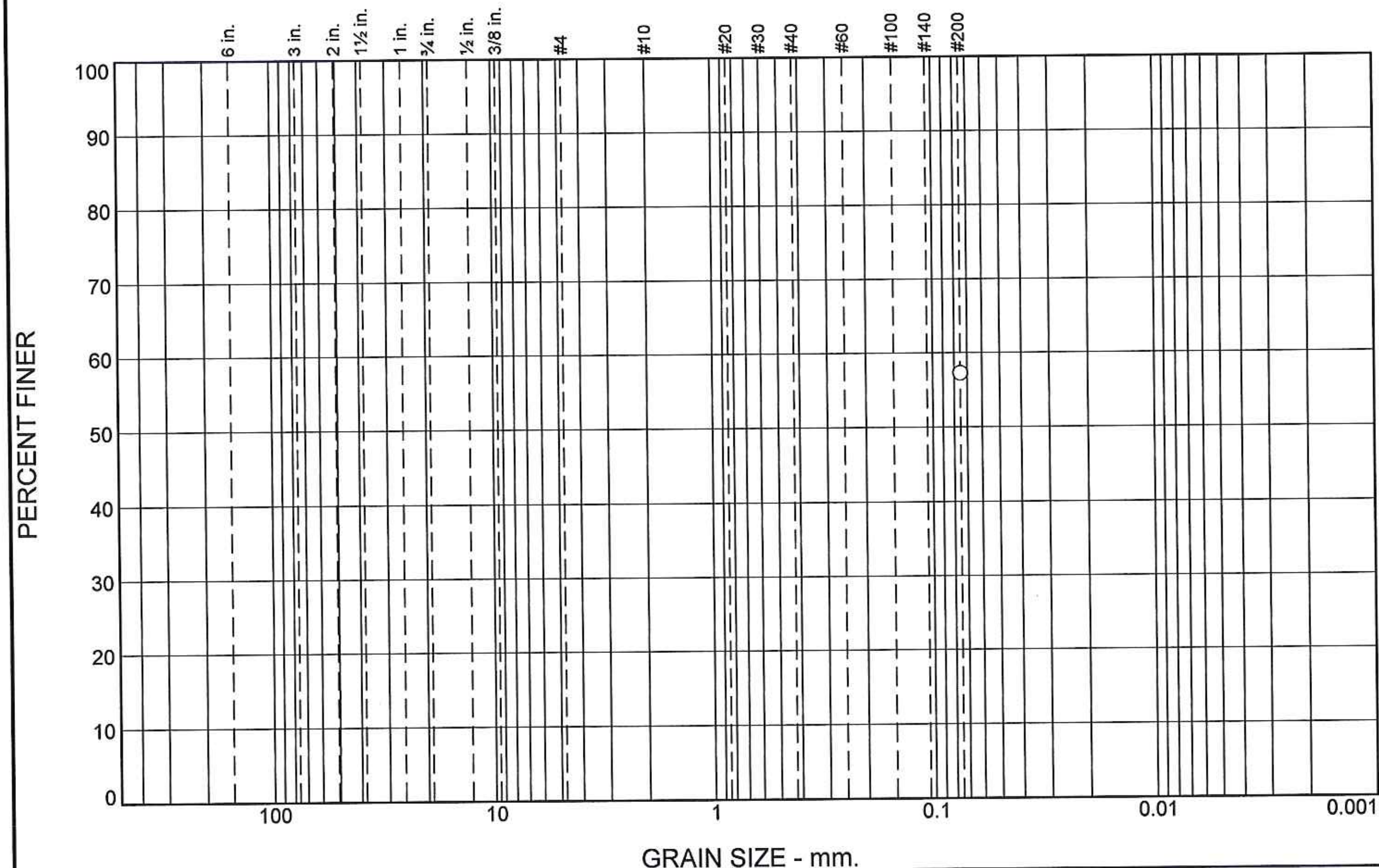
Sample Number: W-1, #12 **Depth:** 22-23.5'

Date: 5/19/14

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation
Project No: 140287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						57.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	57.3		

* (no specification provided)

Material Description

Off white

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample Number: W-1, #19 **Depth:** 36-37.5'

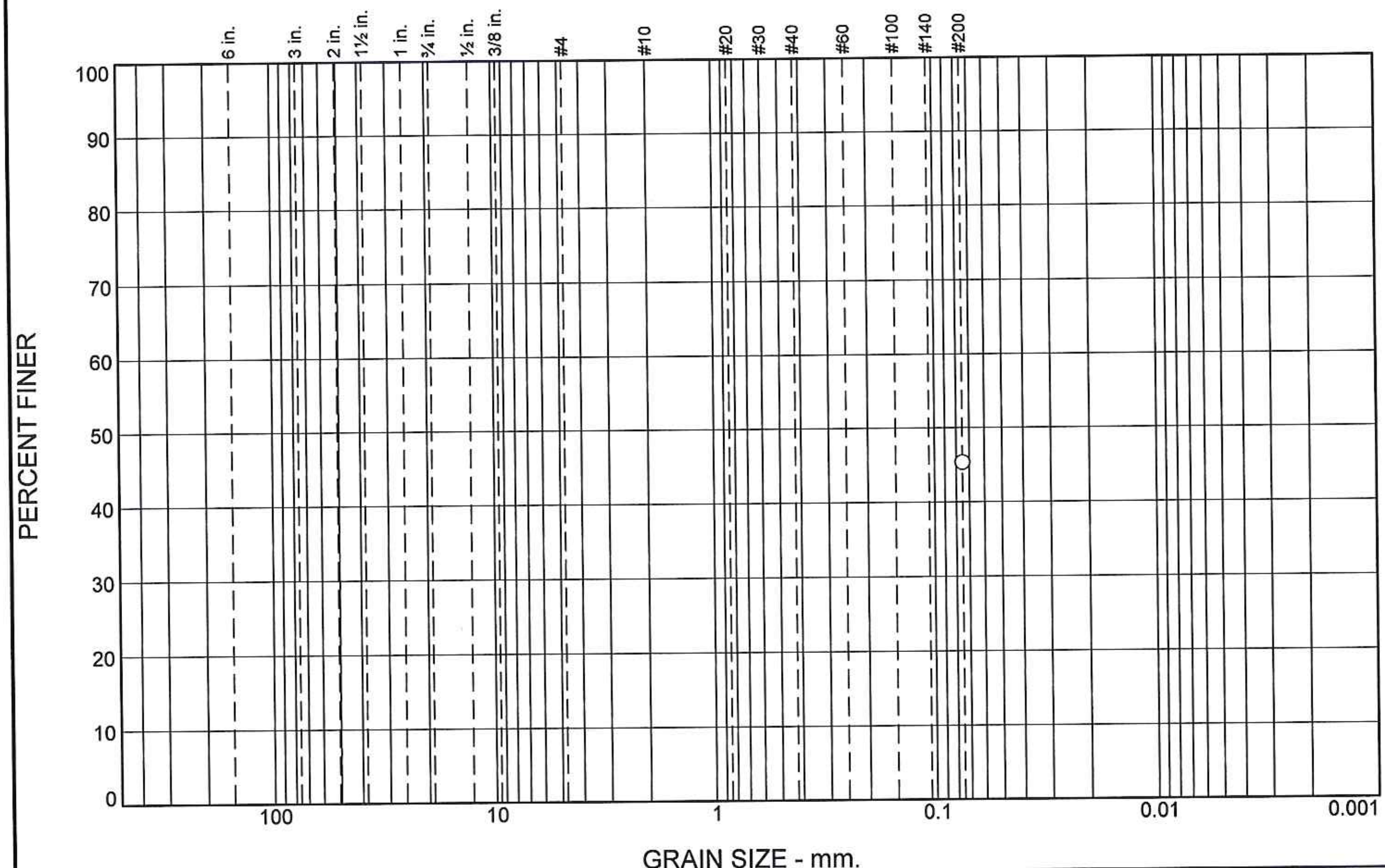
Date: 5/19/14

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation

Project No: 1690287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						45.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	45.3		

* (no specification provided)

Material Description

Gray tan brown

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample Number: W-2, #3 **Depth:** 8-9.5'

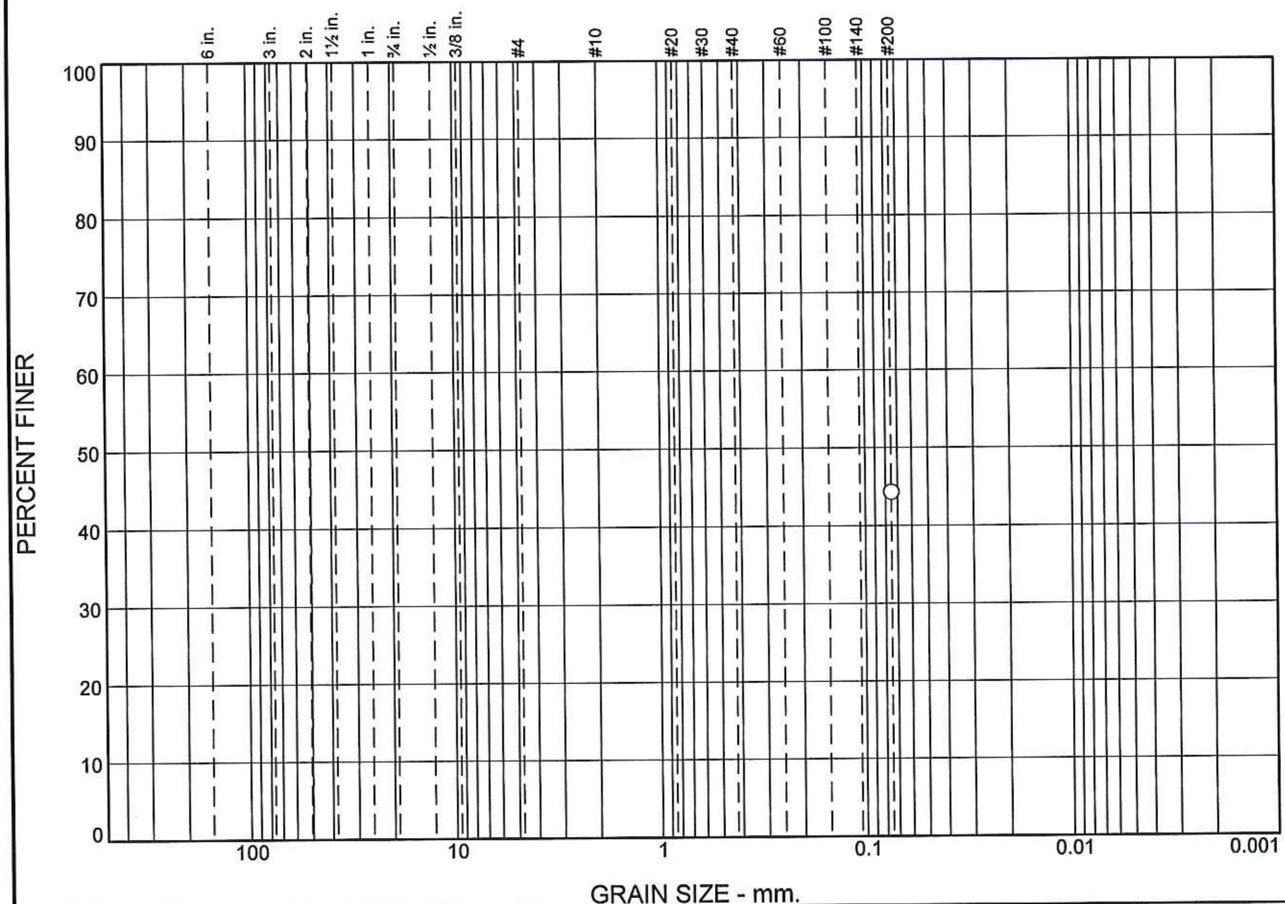
Date: 5/19/14

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation

Project No: 1690287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						44.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	44.3		

* (no specification provided)

Material Description

Brown tan clayey m-f SAND

Atterberg Limits

PL= 15 LL= 46 PI= 31

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Sample Number: W-2, #5 **Depth:** 12-13.5'

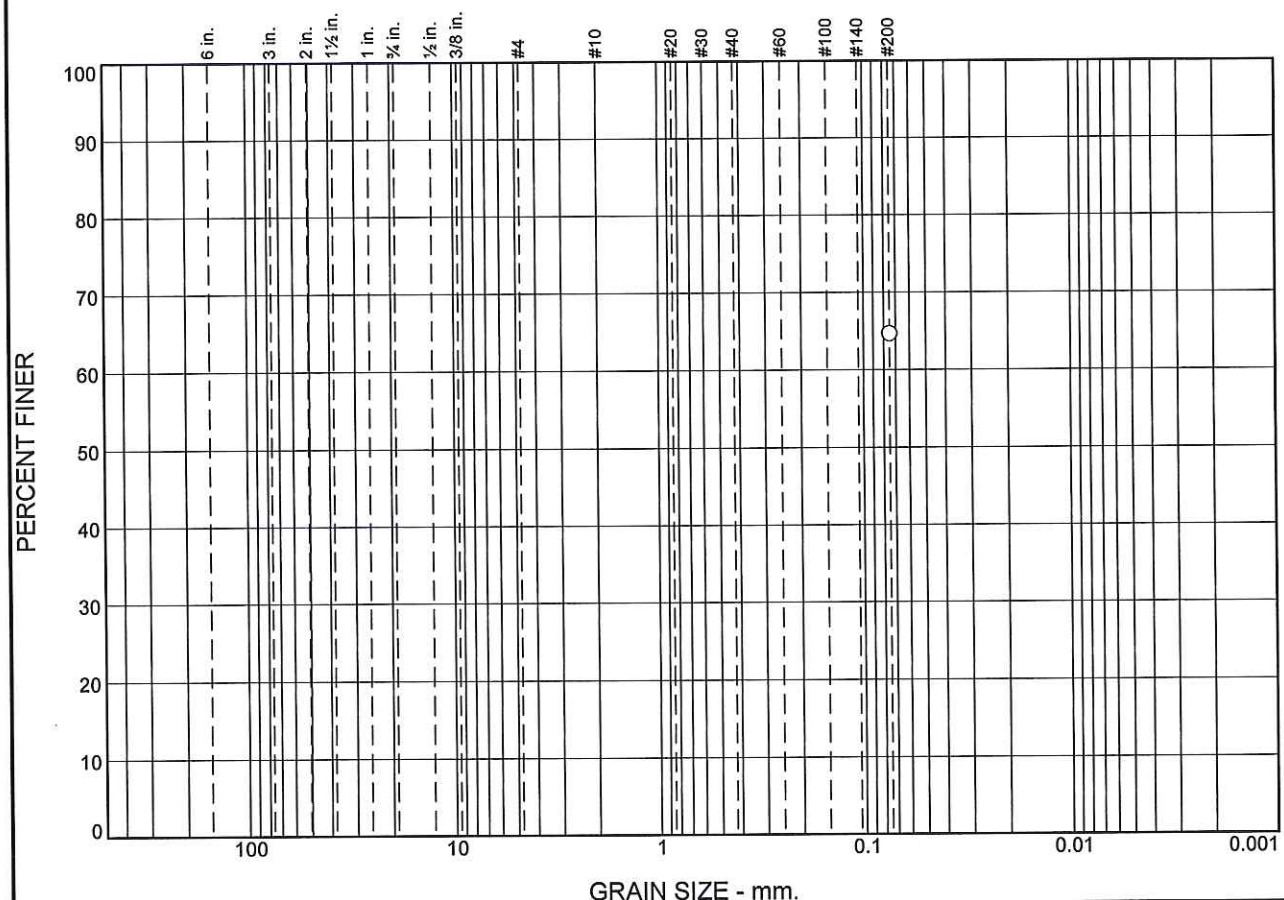
Date: 5/19/14

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation

Project No: 1690287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						64.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	64.7		

* (no specification provided)

Material Description

Off white

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

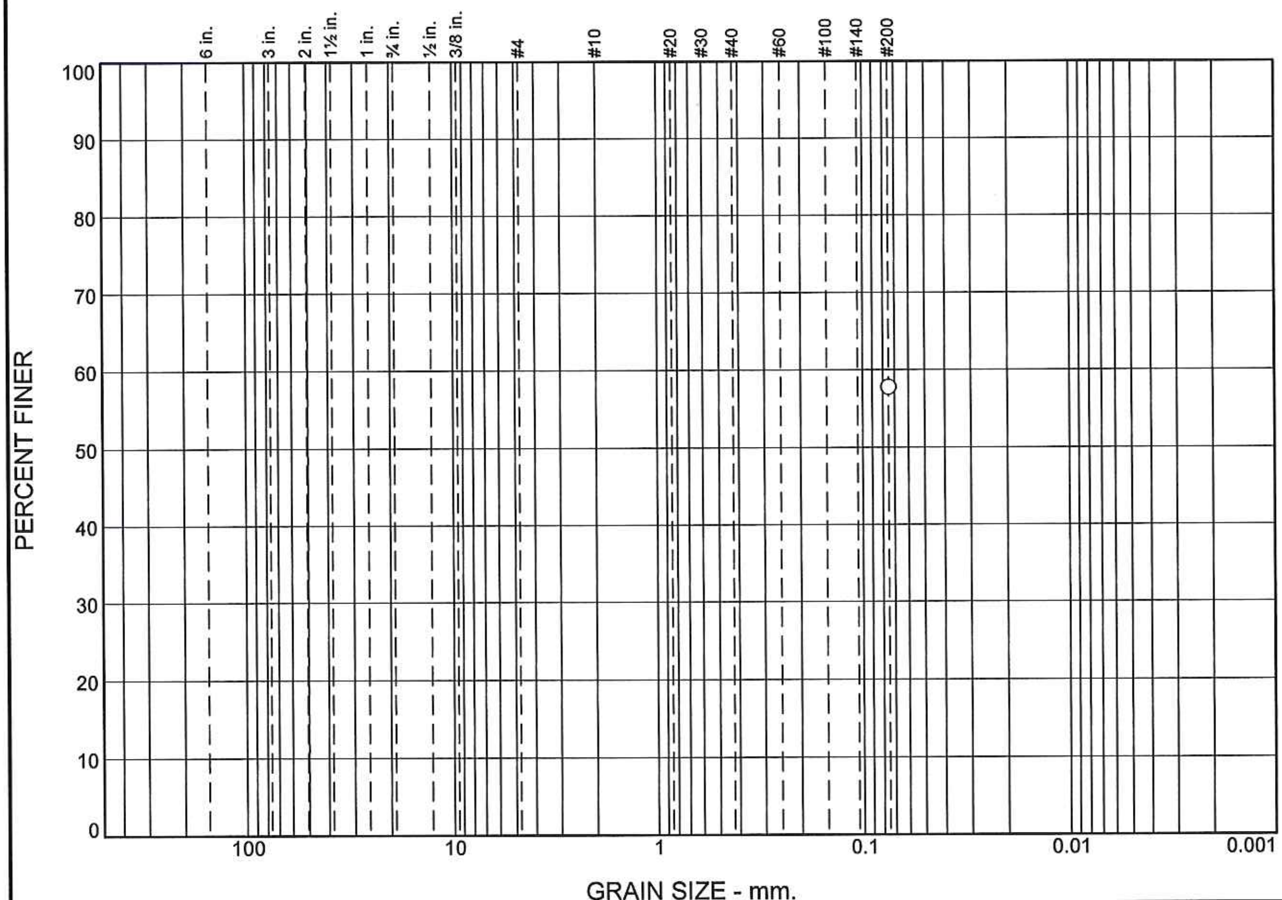
Remarks

Sample Number: W-2, #8 **Depth:** 18-19.5'

Date: 5/19/14

Geotechnical and Environmental Consultants	**Client:** Paul C. Rizzo Associates, Inc. **Project:** CCW Impoundment Evaluation **Project No:** 140287.220 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						57.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	57.8		

* (no specification provided)

Material Description

Off white

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample Number: W-2, #11 **Depth:** 24-24.5'

Date: 5/19/14

Geotechnical and Environmental Consultants

Client: Paul C. Rizzo Associates, Inc.
Project: CCW Impoundment Evaluation

Project No: 140287.220 **Figure**

APPENDIX D

SLOPE STABILITY ANALYSIS

Part I – Completed by Originator

Project Name: CCPC CCW Impoundment Evaluation

		N/A	No	Yes
1.	If this is a revision, explain reason for revision:			
2.	Have superseded versions been VOIDED or destroyed as required?	☒	☐	☐
3.	Has design or analysis software been used for this calculation?		☐	☒
3.1.	If Yes, provide the following information:			
3.2.	Software Name: Slope/W 2007 Version Number: Ver. 7.22 Build 5083			
3.3.	Computer serial number of computer used for this calculation: 000673			
3.4.	Confirm that software is listed on Form QP-7-13.		☐	☒
3.5.	Confirm that Software Usage Log has been updated to include this calculation.		☐	☒
4.	Has a thorough self-check of this calculation been completed and accurate?		☐	☒
5.	Is this calculation nuclear safety related?		☒	☐
5.1.	Has In-Use Test been performed on the computer used for this calculation?	☐	☐	☐
5.2.	If "No" or "N/A," explain:			

Part II – Completed by Verifier(s) – The Independent Reviewer shall address the following:

		N/A	No	Yes
1.	Calculation inputs were correctly selected?	☐	☐	☒
2.	Significant assumptions are adequately identified, described, justified, reasonable?	☐	☐	☒
3.	Any assumptions identified for re-verification are completed?	☐	☐	☒
4.	Calculation inputs were correctly incorporated into the design?	☐	☐	☒
5.	Numerical calculations are correct and documented?	☐	☐	☒
6.	Calculation outputs were reasonable compared to inputs	☐	☐	☒
7.	Calculation input and verification requirements for interfaces are identified (e.g., specified in the Work Plan, supporting procedures, or instructions)?	☒	☐	☐
8.	Suitable materials, parts, processes, inspection and testing criteria were specified (e.g., may be applicable to design calculations, field activities, etc.)?	☒	☐	☐
9.	Hand-annotated changes are made correctly (single line strike through, initialed, and dated)?	☒	☐	☐
10.	All pages are legible, references identified and appropriate; document identifier and revision assigned; and acceptable with respect to grammar, spelling and punctuation?	☐	☐	☒
11.	Each calculation input, information and equations from external sources referenced?	☐	☐	☒
12.	Calculation report contains the required information?	☐	☐	☒

Part III – Approval for Calculations

Originator(s) Print Name	Signature/Date	
Vinod Pillai	Vinod Digitally signed by Vinod Pillai Date: 2014.08.15 14:07:55 -04'00'	
Verifier(s)	**Signature/Date**	**Verification: Independent Design Review**
Conrad Genther	8/16/14	
Project Manager	**Signature/Date**	
Conrad Genther	8/16/14	

Approval of the Project Manager signifies that the document and all required reviews are complete, and the document is released for use.

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1.0 STATEMENT OF PURPOSE

The purpose of this calculation is to analyze the stability of the north embankment of the CCW Impoundment at Plant Crisp in Warwick, GA. Slope stability will be evaluated for the following conditions:

1. Steady Seepage w/ normal pool level
2. Steady Seepage w/ normal pool level & earthquake loading
3. Steady Seepage w/ maximum pool level
4. Sudden Drawdown from maximum pool level (upstream slope)
5. Steady Seepage w/ post seismic shear strength reduction
6. Steady Seepage w/ normal pool level and 2 feet high pool at the toe of the dam (sensitivity case).

Potential for liquefaction during seismic events at the site has been evaluated in a separate calculation.

2.0 DESCRIPTION OF METHODOLOGY USED

A subsurface conditions model for the slope stability analyses was created using the boring logs from the field investigation and geometry from the topographic survey.

SLOPE/W (GeoStudio 2007) was used to perform static, limit-equilibrium slope stability analyses of the downstream slope of the north embankment dam of the CCW Impoundment. The North Embankment is the highest of the embankment sections comprising the CCW Impoundment. An analysis was also performed for the upstream slope including a rapid drawdown case. The Morgenstern-Price method was used to perform the limit-equilibrium analysis, which satisfies both force and moment equilibrium.

Circular failures were considered for all cases except for the normal pool with post seismic strength reduction case. Block failure was considered for this case to analyze the effect of the layer with reduced shear strength on the factor of safety against failure of the slope. An additional block failure case was performed for the normal pool loading condition for comparison with the reduced strength case.

3.0 ASSUMPTIONS AND JUSTIFICATION

Assumptions used in this calculation include:

1. Phreatic surface in the embankment is taken as the level measured in the boreholes during the geotechnical exploration + the difference in elevation between the pool elevation at the time of exploration and the pool elevation for the case analyzed, as shown in ***Table 3-1***:

TABLE 3-1: Phreatic Surface Elevations

Boring	Phreatic surface during exploration (ft)	Pool elevation during exploration (ft)	Load Case Pool Elevation (ft)	Change in Pool Elevation (ft)	Load Case Phreatic Surface Elevation (ft)
N-1 (C/L of crest)	229	239	241 (Normal Pool)	2	231
			245 (Full Pool)	6	235
N-2 (toe)	221	239	241 (Normal Pool)	2	223
			245 (Full Pool)	6	226 (Ground Surface)

2. Material types are taken from the boring logs and strength properties are determined based on correlations with the measure N values. ***Table 3-2*** summarizes the material properties used in this analysis. Strength Reduction factor (R_u) for SC-SM 2 is calculated as described in ***Section 5.0***.

TABLE 3-2: Material Properties

Material	Unit Weight (pcf)	Cohesion (psf)	Phi (deg)	Strength Reduction Factor
SM	125	0	31	-
SC-SM	125	0	30	-
CL	135	1000	0	-
CL-Decomposed Limestone	140	5000	0	-
SC-SM 2	125	0	30*	0.19

*Note: A phi of 10 deg was used to verify did not have any impact on the F.S. of the slope. An informal case using a phi of 10 deg was also run to check the sensitivity of the F.S. to very low strength in the affected layer. The F.S. was not impacted.

3. Assuming that there is some seepage at the toe, a sensitivity case is analyzed with a 2 foot pool at the toe of the dam.

4.0 CALCULATION INPUT

Calculation inputs used are as follows:

1. Dam classification, seismic acceleration, and minimum Factors of Safety are taken from Georgia Safe Dam Rules:
   a. The CCW Impoundment is classified as a Category II, small dam. The dam is less than 25' in height and has less than 500 ac-ft of storage.
   b. The dam must be capable of withstanding seismic accelerations defined in the most current USGS map for peak acceleration with a 2% exceedance in 50 years, or a minimum seismic acceleration of 0.05g, whichever is greater. Seismic acceleration for the site is 0.14g (***Ref 2***).
   c. FS for Slope Stability for the load cases analyzed are as shown in ***Table 4-1***:

TABLE 4-1: Minimum FS for Slope Stability

Load Case	Minimum FS
Steady State Seepage (Normal Pool)	1.5
Steady State Seepage with Seismic Loading (Normal Pool)	1.1
Steady State Seepage (Maximum Pool)[1]	1.3
Rapid Drawdown (Upstream Slope)	1.3
Steady State Seepage w/ post seismic strengths	1.1
Sensitivity Case	1.5

1. This case is not required by GA Safe Dams guidelines.

5.0 NUMERICAL CALCULATIONS

SLOPE/W is use to perform numerical calculations. Static, limit-equilibrium slope stability analyses are performed of the existing geometry of the North (tallest) Embankment. A rapid drawdown analysis was also performed for the upstream slope. The Morgenstern-Price method was used to perform the limit-equilibrium analysis, which is considered a complete equilibrium procedure since it satisfies both force and moment equilibrium.

For the steady state seepage w/ post seismic strength reduction case, the shear strength reduction factor calculations are included in ***Appendix B***.

6.0 CALCULATION OUTPUT

Output from SLOPE/W showing the critical failure surface, material strength properties, and slope geometry is provided in ***Appendix A*** for review.

7.0 RESULTS

Table 7-1 summarizes the FS for the critical sections for each of the load cases:

TABLE 7-1: Calculation Results

Load Case	Failure Type	Calculated FS	Minimum FS
Steady State Seepage (Normal Pool)	Circular	1.26	1.5
Steady State Seepage (Normal Pool)	Block	1.26	1.5
Steady State Seepage with Seismic Loading (Normal Pool)	Circular	0.92	1.1
Steady State Seepage (Maximum Pool)[1]	Circular	1.15	1.3
Rapid Drawdown (Upstream Slope)	Circular	1.71	1.3
Steady State Seepage w/ post seismic strengths	Block	1.23	1.1
Steady State Seepage (Normal Pool) (w/ 2 ft pool at the toe of the dam).	Circular	1.26	1.5

8.0 CONCLUSION/SUMMARY

The stability analysis indicates that the embankment in its current configuration does not have sufficient FS to meet GA Safe Dams requirements for any of the downstream slope stability cases. While the embankment is relatively short, the downstream slopes are too steep and will require modification to comply with the GA Safe Dams minimum FS. The upstream slopes of the embankment meet the requirements due to the much flatter slopes of the upstream side of the embankment.

The sensitivity case performed with 2 feet of water against the downstream toe has the same F.S. as the normal pool case. This shows that the stability of the dam is relatively independent of the downstream phreatic surface and that the steepness of the slope is the main contributor to slope instability.

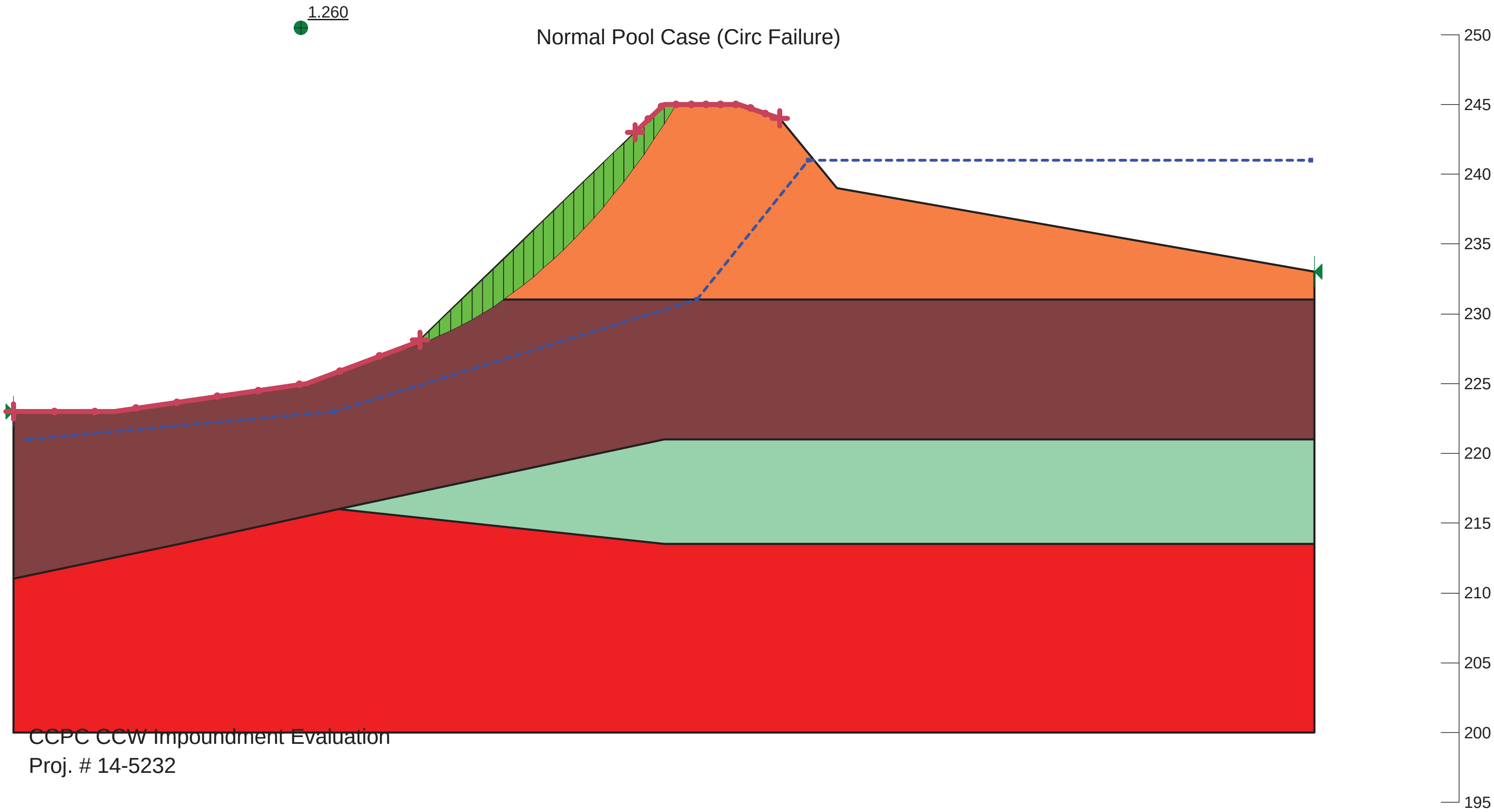
The results of the post seismic case indicate that the calculated strength reductions do not significantly impact the F.S. or failure location.

9.0 REFERENCES

1. Georgia Department of Natural Resources – Environmental Protection Division, "Georgia Stormwater Management Manual, Vol II Technical Handbook, Appendix H," October 26, 1998.
2. Rizzo Associates, July 2014, "Liquefaction Analysis," 14-5232 F-2, Pittsburgh, PA.
3. Rizzo Associates, July 2014, "Boring Logs," 14-5232, Columbia, SC.
4. Rizzo Associates, July 2014, "Topographic Drawings and Sections," 14-5232 DWG 1 and DWG 2, Columbia, SC.

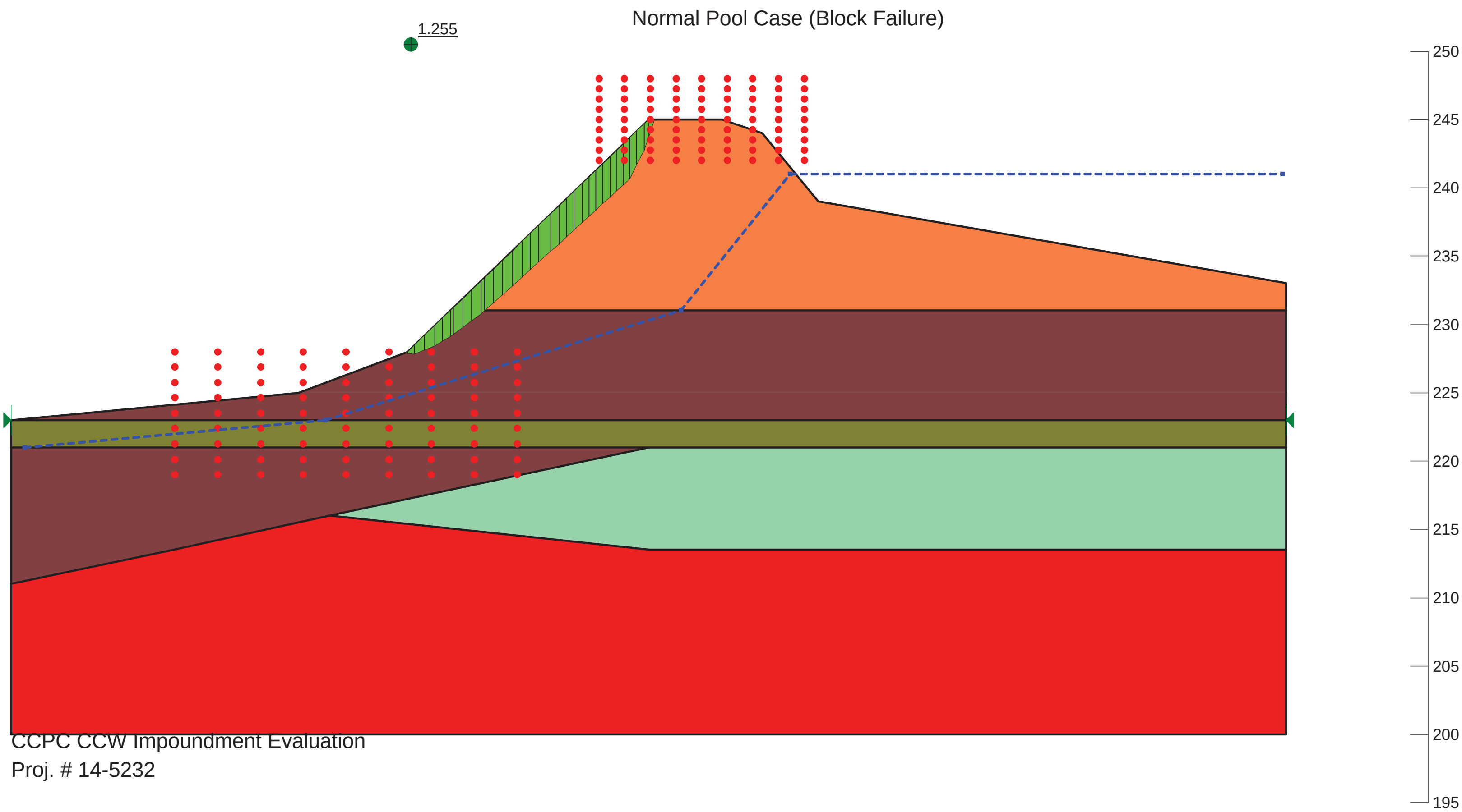
APPENDIX A

ANALYSIS RESULTS



CCPC CCW Impoundment Evaluation
Proj. # 14-5232

Name: SM Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 0 psf Phi: 31 ° Phi-B: 0 °
Name: SC-SM Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 0 psf Phi: 30 ° Phi-B: 0 °
Name: CL Model: Mohr-Coulomb Unit Weight: 135 pcf Cohesion: 1000 psf Phi: 0 ° Phi-B: 0 °
Name: CL - Decomposed LS Model: Mohr-Coulomb Unit Weight: 140 pcf Cohesion: 5000 psf Phi: 0 ° Phi-B: 0 °



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Proj. # 14-5232

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Name: CL Model: Mohr-Coulomb Unit Weight: 135 pcf Cohesion: 1000 psf Phi: 0 ° Phi-B: 0 °
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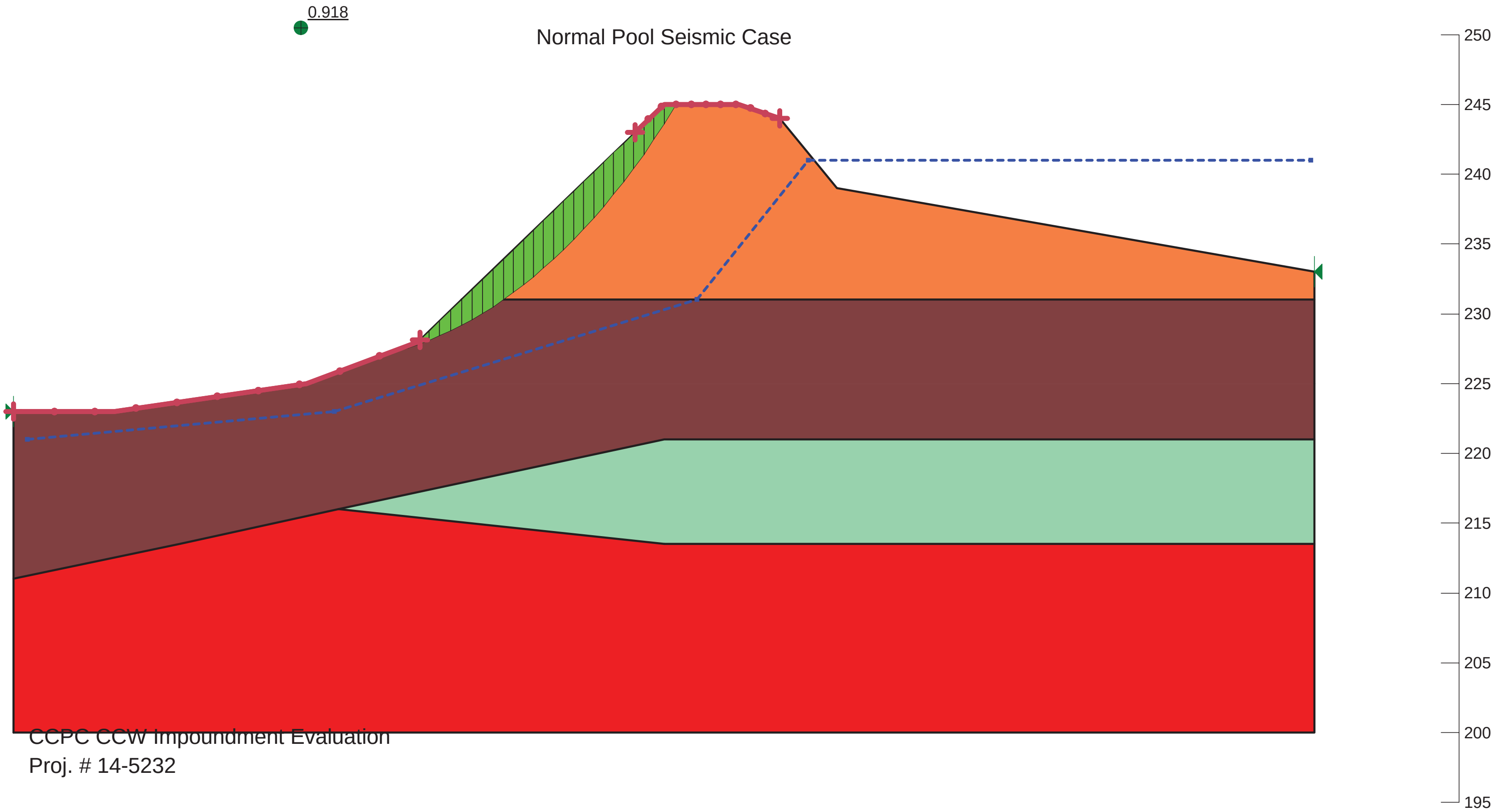

CCPC CCW Impoundment Evaluation
Proj. # 14-5232

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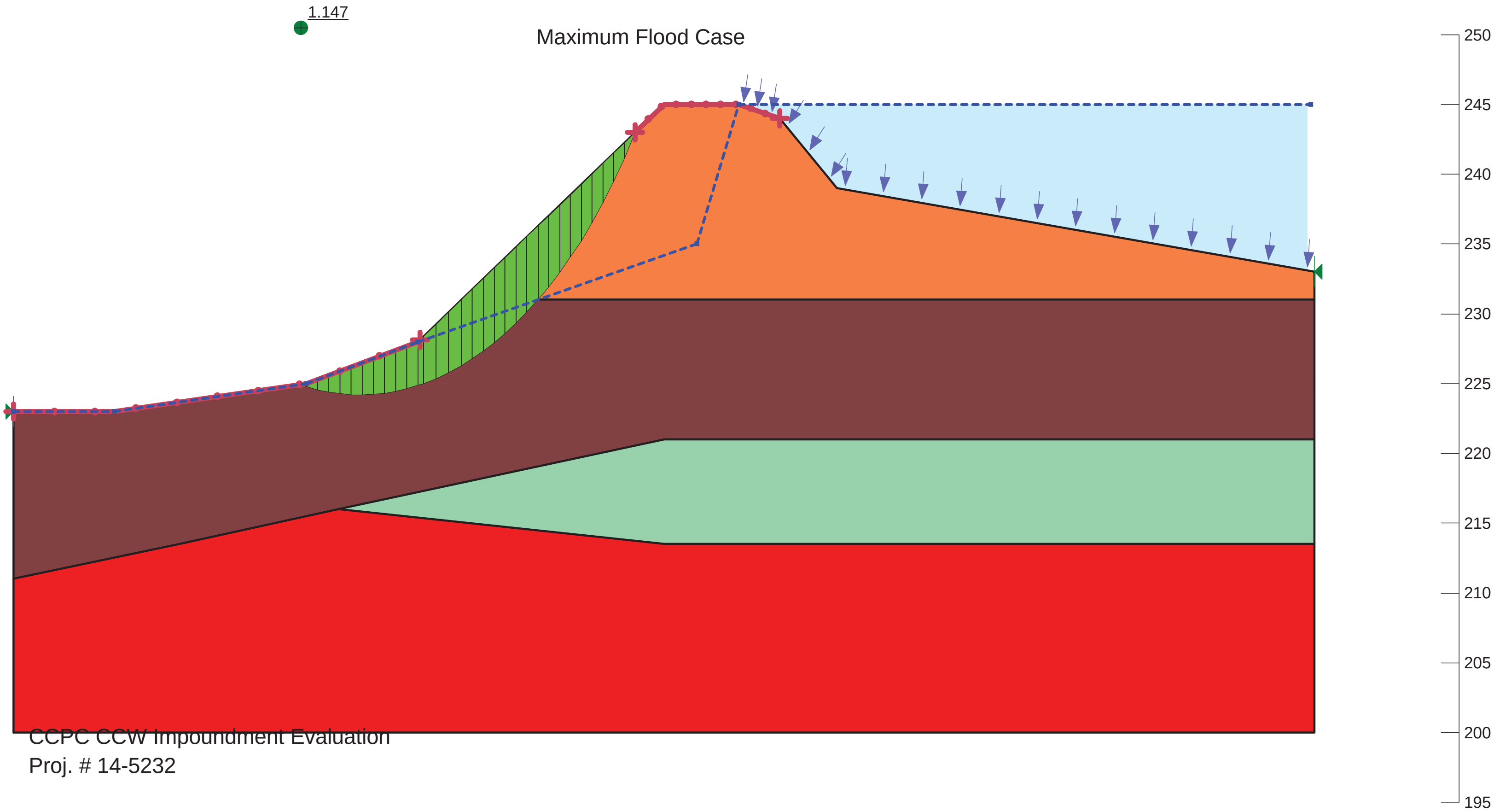
Name: SC-SM Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 0 psf Phi: 30 ° Phi-B: 0 °
Name: CL Model: Mohr-Coulomb Unit Weight: 135 pcf Cohesion: 1000 psf Phi: 0 ° Phi-B: 0 °
Name: CL - Decomposed LS Model: Mohr-Coulomb Unit Weight: 140 pcf Cohesion: 5000 psf Phi: 0 ° Phi-B: 0 °



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Name: SC-SM Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 0 psf Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1
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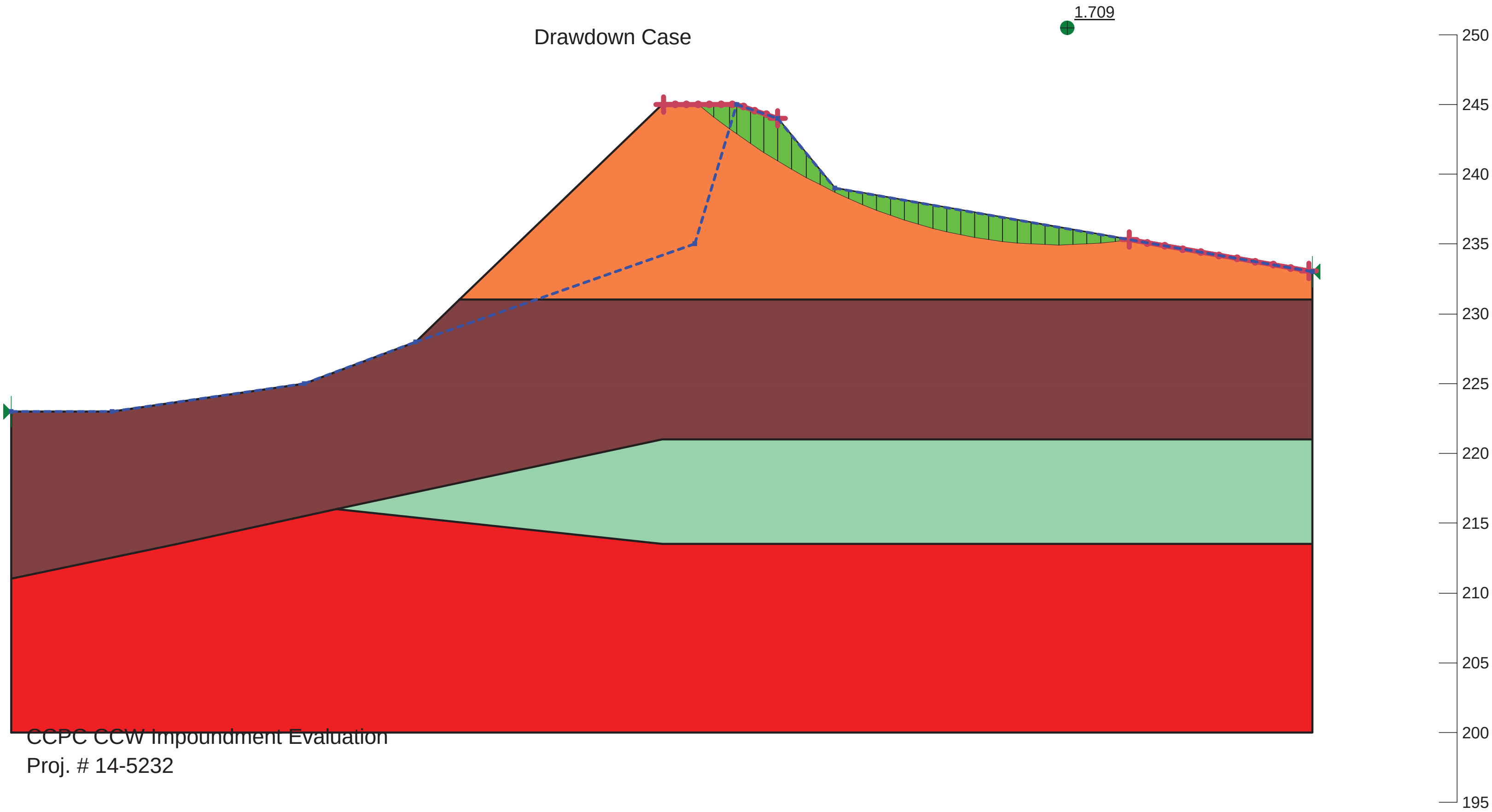

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Proj. # 14-5232

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Name: SC-SM Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 0 psf Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1
Name: CL Model: Mohr-Coulomb Unit Weight: 135 pcf Cohesion: 1000 psf Phi: 0 ° Phi-B: 0 ° Piezometric Line: 1
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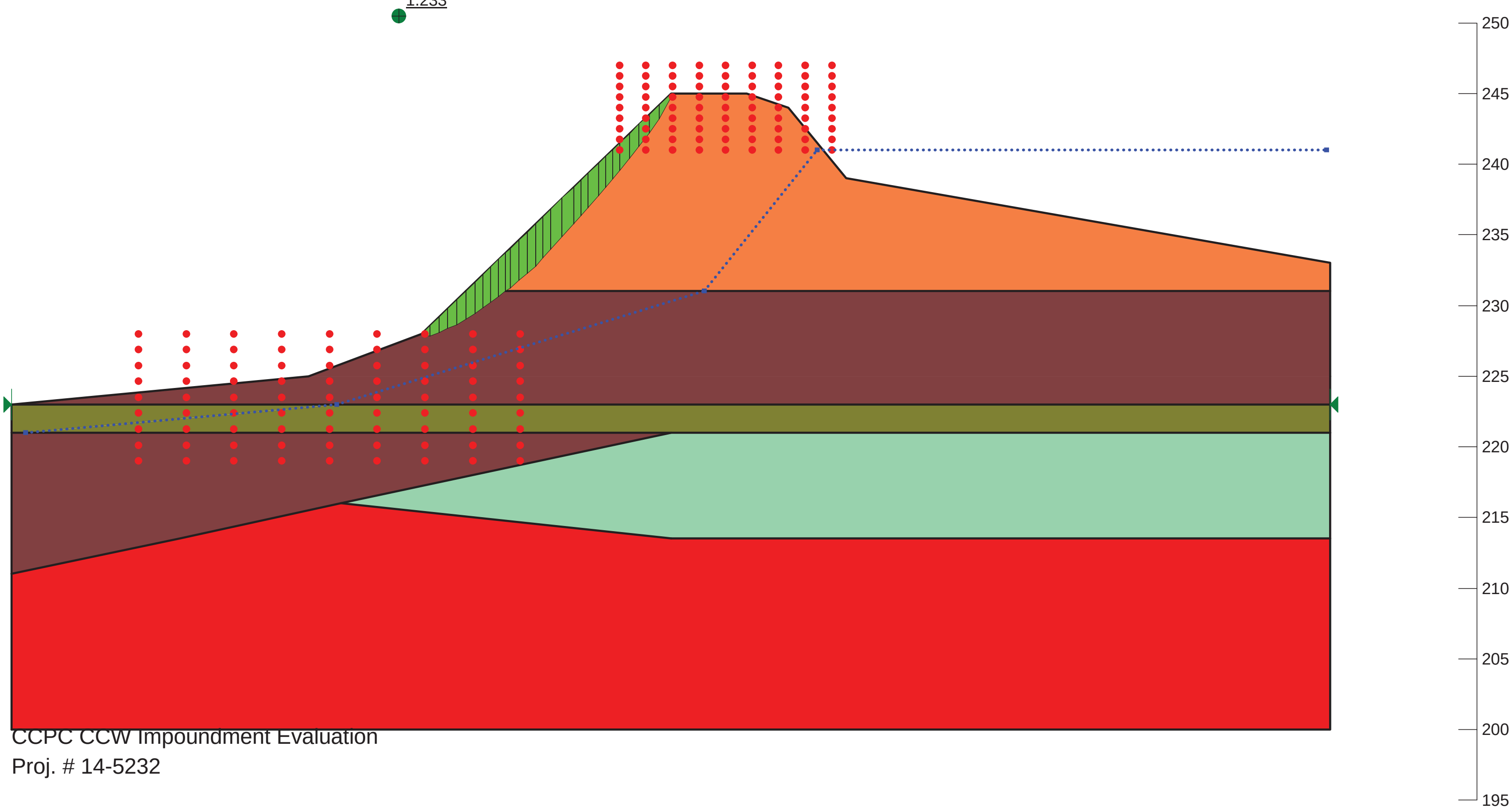
Normal Pool Post Seismic Case



Name: SM   Model: Mohr-Coulomb   Unit Weight: 125 pcf   Cohesion: 0 psf   Phi: 31 °   Phi-B: 0 °   Include in PWP: No
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Name: CL   Model: Mohr-Coulomb   Unit Weight: 135 pcf   Cohesion: 1000 psf   Phi: 0 °   Phi-B: 0 °   Include in PWP: No
Name: CL - Decomposed LS   Model: Mohr-Coulomb   Unit Weight: 140 pcf   Cohesion: 5000 psf   Phi: 0 °   Phi-B: 0 °   Include in PWP: No
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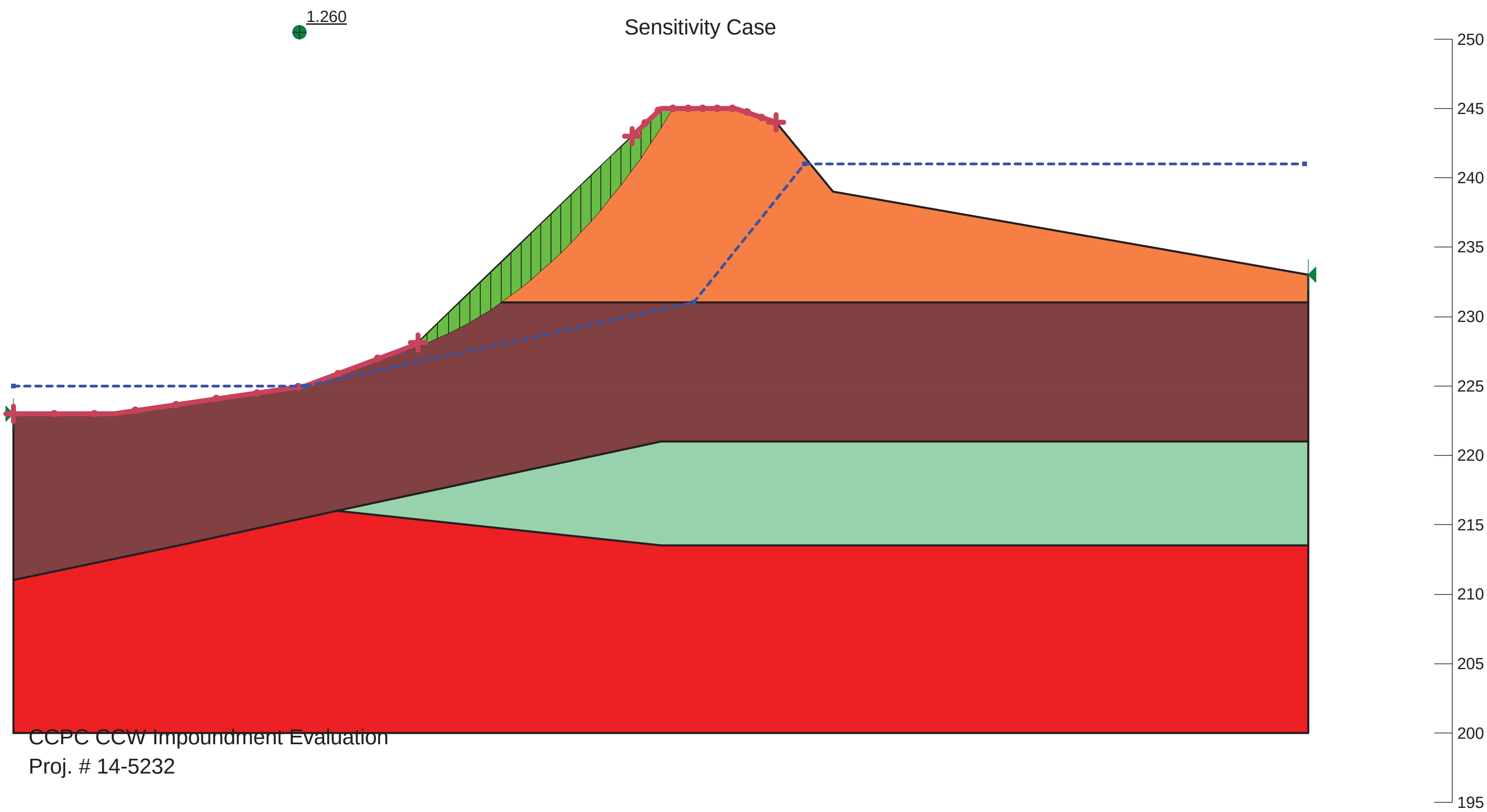

CCPC CCW Impoundment Evaluation
Proj. # 14-5232

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Name: SC-SM Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 0 psf Phi: 30 ° Phi-B: 0 °
Name: CL Model: Mohr-Coulomb Unit Weight: 135 pcf Cohesion: 1000 psf Phi: 0 ° Phi-B: 0 °
Name: CL - Decomposed LS Model: Mohr-Coulomb Unit Weight: 140 pcf Cohesion: 5000 psf Phi: 0 ° Phi-B: 0 °

APPENDIX B

STRENGTH REDUCTION FACTOR CALCULATION

Strength Reduction Factor Calculation

For R_u of very loose silty clayey sand layer @ W-1 with F.S. against liquefaction less than 1.8 (***Ref. 2***)

Elevation of dam crest	=	245	ft
Elevation of phreatic surface (from boring logs)	=	245-15	ft
	=	230	ft
Height of soil column above layer (pt1) [from boring logs], H_s	=	23	ft
Elevation of very loose silty clayey sand layer (pt1)	=	245-23	ft
		222	ft
Height of water @ pt1	=	230-222	ft
	=	8	ft
γ_t of soil (**Table 3-2**)	=	125	pcf
Therefore, $\gamma_t H_s$	=	125x23	psf
	=	2875	psf
Strength Reduction factor, R_u	=	$U/\gamma_t H_s$	
	=	8x62.4/2875	
	=	**0.17**	

Part I Completed by Originator

Project Name: Crisp County

1. If this is a revision, explain reason for revision: N.A.
2. Have superseded versions been VOIDED, or destroyed as required? ☒N/A ☐No ☐Yes
3. Has design or analysis software been used for this Calculation? ☒No ☐Yes
   3.1. If Yes, provide the following information:
   3.2. Software Name: Version Number:
   3.3. Computer Serial Number of computer used for this Calculation:
   3.4. Confirm that software is listed on Form QP-7-13. ☐No ☐Yes
   3.5. Confirm that Software Usage Log has been updated to include this calculation. ☐No ☐Yes
4. Has a thorough Self-Check of this Calculation been completed and accurate? ☐No ☒Yes
5. Is this calculation nuclear safety related? ☒No ☐Yes
   5.1. Has In-Use Test been performed on the computer used for this calculation? ☐N/A ☐No ☐Yes
   5.2. If "No" or "N/A," explain

Part II – Completed by Verifier(s)–The Independent Reviewer shall address the following:

1. Calculation inputs were correctly selected. ☐N/A ☐No ☒Yes
2. Significant assumptions are adequately identified, described, justified, reasonable? ☐N/A ☐No ☒Yes
3. Any assumptions identified for re-verification are completed? ☐N/A ☐No ☒Yes
4. Calculation inputs were correctly incorporated into the design? ☐N/A ☐No ☒Yes
5. Numerical calculations correct, and documented? ☐N/A ☐No ☒Yes
6. Calculation outputs were reasonable compared to inputs ☐N/A ☐No ☒Yes
7. Calculation input and verification requirements for interfaces are identified (e.g., specified in the Work Plan, supporting procedures, or instructions) ☒N/A ☐No ☐Yes
8. Suitable materials, parts, processes, inspection and testing criteria specified? (e.g., may be applicable to design calculations, field activities, etc.) ☒N/A ☐No ☐Yes
9. Hand-annotated changes are made correctly (single line strike through, initialed, dated)? ☒N/A ☐No ☐Yes
10. All pages are legible, references identified and appropriate; document identifier and revision assigned; and acceptable with respect to grammar, spelling and punctuation ☐N/A ☐No ☒Yes
11. Each calculation input, Information and equations from external sources referenced? ☐N/A ☐No ☒Yes
12. Calculation Report contains the required information? ☐N/A ☐No ☒Yes

Part III – Approval for Calculations

Originator(s) Print Name	Signature/Date
VINOD PILLAI	8/5/14
Verifier(s)	**Signature/Date**
KEVIN CASS	8/5/14
Project Manager	**Signature/Date**
CONRAD GINTHER	8/18/14

Approval of the Project Manager signifies that the document and all required reviews are complete, and the document is released for use.

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1.0 STATEMENT OF PURPOSE

The purpose of this calculation is to determine the factors of safety against liquefaction for the various layers at the Crisp County Dam.

2.0 DESCRIPTION OF METHODOLOGY USED

The empirical method given by Idriss and Boulanger (2008) (***Ref. 10***), and Youd et al. (2001) (***Ref. 9***) is used to calculate the factors of safety against liquefaction for the various sub-surface layers based on the SPT N-values. Selected pages from Youd (2001), and Idriss and Boulanger (2008) are attached as ***Appendix C*** and ***Appendix E*** respectively. A step-by-step procedure is outlined in ***Section 2.2***. Before starting this procedure, inputs regarding earthquake characteristics and fines content of the Loose Fill material must be determined, as described in ***Section 2.1***.

The silt content has a large impact on the factors of safety against liquefaction. A sensitivity analysis is performed by assuming Lower Bound (LB) and Best Estimate (BE) values for silt content. The piezometric levels are determined using the measured levels at borings N-1 and W-1 (***Ref. 1***) and observed seepage at the toe of the embankment.

2.1 INITIAL INPUTS

Earthquake inputs are derived in ***Section 2.1.1***. Values for the fines content of the soil layers are estimated in ***Section 2.1.2***.

2.1.1 Earthquake Inputs

The initial inputs that must be determined with respect to the design earthquake are the horizontal peak ground acceleration (PGA) and the earthquake magnitude (Mw). The PGA used is the acceleration corresponding to a 2% probability of exceedence in 50 years (i.e., 2500-year return period). This PGA can be found on the USGS 2008 National Seismic Hazard maps (USGS, 2008) (***Ref. 7***). The horizontal PGA corresponding to Crisp County Dam is 0.0513g.

Since the PGA given above is for the bedrock, an adjustment is needed to account for the acceleration experienced by the soil layers. Adjustments need to be made to account for the location within the embankment.

1. Figure 47 from FHWA (1997) (***Ref. 2***), which was reprinted from Harder (1991) (***Ref. 4***), plots historical base accelerations vs crest accelerations for a variety of earth dams. Based on the upper bound trend line in Figure 47, the peak acceleration at the crest of the embankment is estimated as 0.2g. This figure is shown below as ***Figure 2-1***.

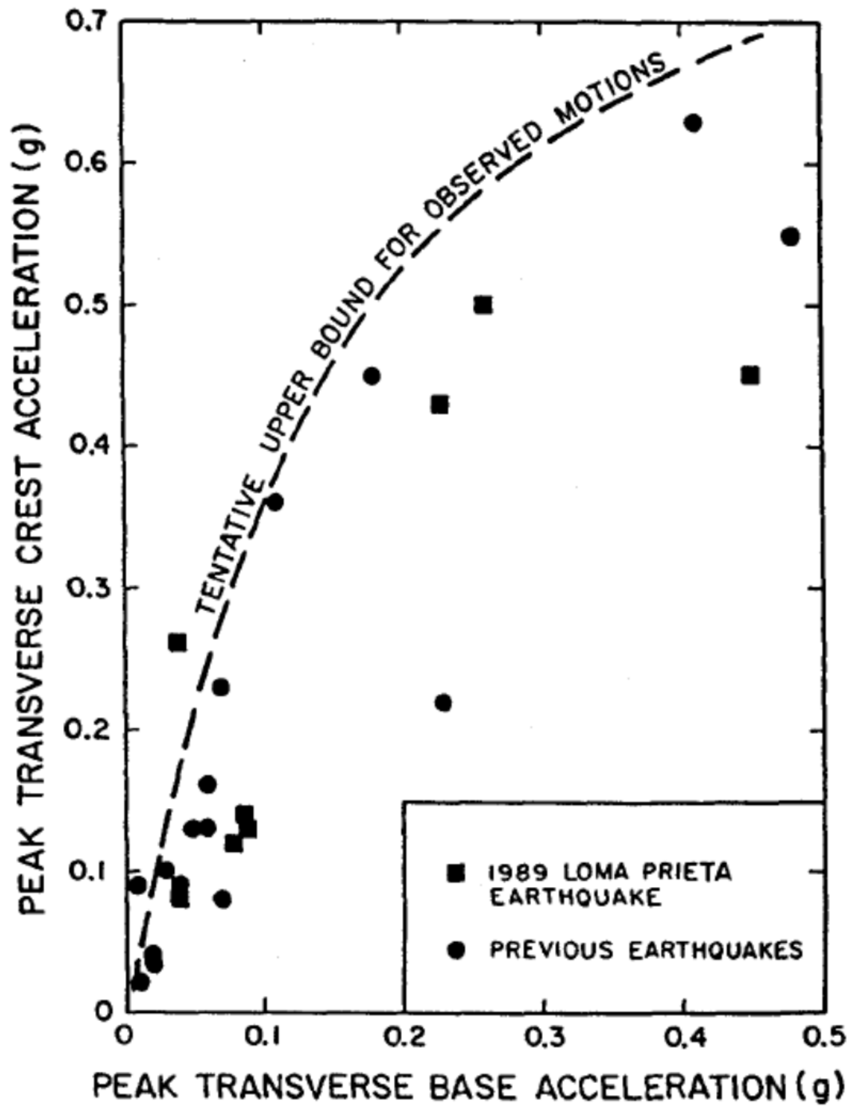

FIGURE 2-1
COMPARISONS OF PEAK BASE AND CREST ACCELERATIONS
RECORDED AT EARTH DAMS (HARDER, 1991, AS CITED IN FHWA, 1997)

2. The material with the lowest N-values boreholes N-1 and W-1 are approximately 40% to 50% (y/h) of the way down the embankment as measured from the crest. From ***Figure 2-2***, the acceleration for a y/h of 0.4 is roughly 0.7 times the acceleration at the crest of the dam. Using the peak crest acceleration from above and ***Figure 2-2*** (1997) (***Ref. 2***), the peak acceleration at layers with lowest N-values is conservatively taken as 0.2g*0.7 = 0.14g. This figure is shown below as ***Figure 2-2***.

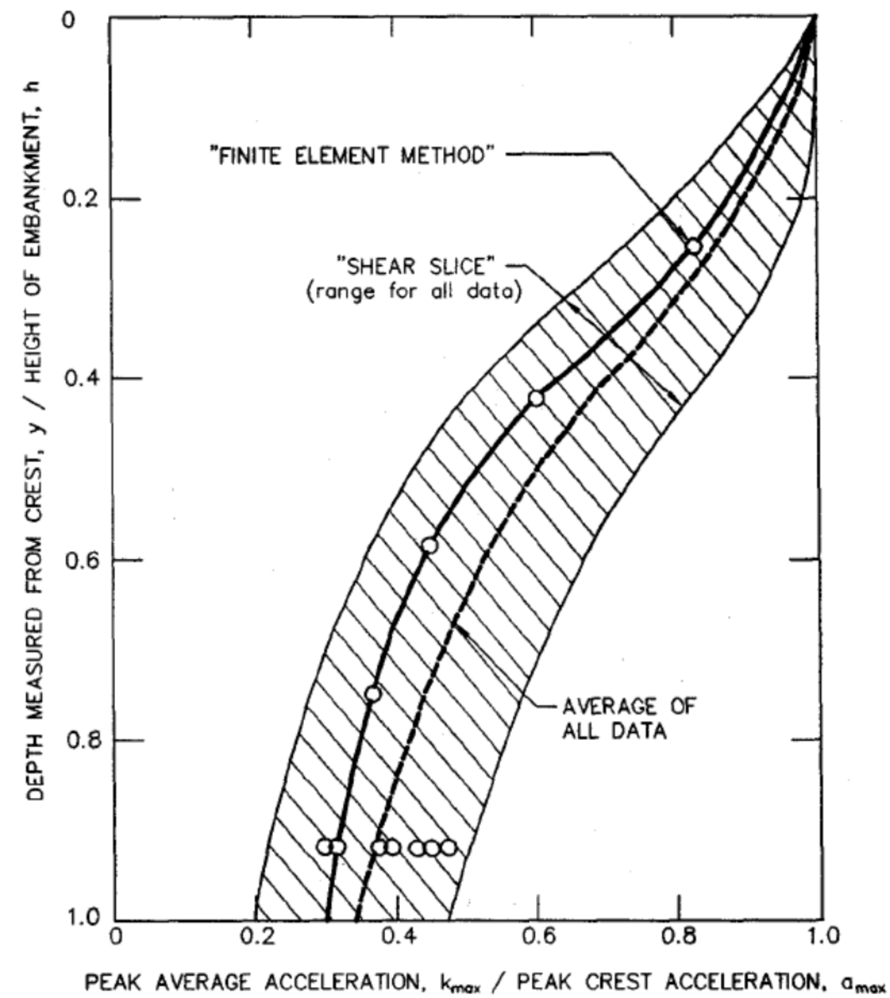

FIGURE 2-2
VARIATION OF PEAK AVERAGE ACCELERATION RATIO
WITH DEPTH OF SLIDING MASS
(FHWA, 1997)

To determine an appropriate earthquake magnitude, the USGS 2008 Interactive Deaggregations site (***Ref. 8***) was used to list of all of the earthquakes that contributed to the estimation of the PGA. This list is given in ***Appendix A***. The magnitude used in the liquefaction analysis is the average magnitude of all contributing earthquakes (Mw = 6.32).

2.1.2 Fines Content

Laboratory Test results presented in the boring logs (***Ref. 1***) in conjunction with USCS Soil classification tables (***Ref. 13***) are used to determine the fines content for each layer.
Boring N-1 is divided into three layers (that fall below the phreatic surface). The first layer mostly consists of silty clayey sand. As per the lab results the fines content of this layer is 28%. In an additional sensitivity case, the fines content of this layer is modified to 15% which is the lowest possible fines content for a silty clayey sand, to evaluate if this reduces the Factor of Safety against liquefaction significantly. The second layer mostly consists of clay with pebbles and gravel fragments. For a soil of this type, the minimum amount of fines is 30%. The third layer consists of clay with decomposed limestone fragments. The minimum amount of fines for this type of soil is 30%.

Boring W-1 is also divided into three layers (that fall below the phreatic surface). The first layer mostly consists of silty clayey sand/silty sand. As per the lab results the fines content of this layer is 29%. In an additional sensitivity case, the fines content of this layer is modified to 15% which is the lowest possible fines content for a silty clayey sand, to evaluate if this reduces the Factor of Safety against liquefaction significantly. The second layer mostly consists of clay. For a soil of this type, the minimum amount of fines is 30%. The third layer consists of clay with decomposed limestone fragments. The minimum amount of fines for this type of soil is 30%.

2.2 LIQUEFACTION ANALYSIS

The following steps describe the procedure for analyzing liquefaction potential (Youd et al., 2001, ***Ref. 9***).

1. Determine the total unit weight (γ_{total}) and saturated unit weight (γ_{sat}) to use for the embankment. The total unit weight will be used to calculate stress above the phreatic surface, and the saturated unit weight will be used to calculate the stress below the phreatic surface. These values are γ_{total} = 115 pcf and γ_{sat} = 120 pcf.

2. For both the borings, determine the ground surface elevation, phreatic surface elevation, and elevation of each sample for which SPT N-values were obtained. The ground surface, phreatic surface and soil layer elevations are taken from the boring logs (***Ref. 1***).

3. Calculate the total vertical stress on each of the samples as shown in ***Equation 1*** (assumes all samples are below phreatic surface).

$$\sigma_v = (GSE - PSE)\gamma_{total} + (PSE - LFE)\gamma_{sat} \qquad (1)$$

where,

σ_v	=	total vertical stress (psf)
GSE	=	ground surface elevation (ft)
LFE	=	average Loose Fill sample elevation (ft)
PSE	=	phreatic surface elevation (ft)
γ_{total}	=	average total unit weight of overburden soil (115 pcf)
γ_{sat}	=	average saturated unit weight of overburden soil (120 pcf)

4. Calculate the effective vertical stress as shown in Equation 2.

$$\sigma_v' = \sigma_v - (PSE - LFE)\gamma_w \qquad (2)$$

where,

σ_v'	=	effective vertical stress from Step 3 (psf)
γ_w	=	unit weight of water (62.4 pcf)

5. Calculate the average N-value in blows per foot (bpf) for each sample.
6. Remove the first and last SPT values from each split spoon sample interval.
7. Calculate the N-value correction factor for overburden pressure (Equation 10 in Youd et al., 2001, ***Ref. 9***).

$$C_N = 2.2/(1.2 + \sigma_v'/P_a) \qquad (3)$$

where,

C_N	=	overburden stress correction factor (psf)
P_a	=	atmospheric pressure (2,116.2 psf)

8. Adjust the N-value for the overburden pressure.

$$N_m = N \cdot C_N \qquad (4)$$

where,

N_m	=	N-value corrected for overburden stress (bpf)
N	=	N-value from boring logs (bpf)

9. Calculate correction factors CR (rod length correction), CS (sampling method correction), CB (borehole diameter correction), and CE (hammer energy correction). Because limited equipment information is available, only CB can be determined. From Table 2 of Youd et al. (2001), the CB factor remains at unity. CE = $ER_m/60$ where ER_m is the measured energy ratio as a percentage of the theoretical maximum. In US practice, the

delivered energy is commonly about 60% of the theoretical maximum energy (Kovacs et al, 1983, ***Ref. 11***). Because the remaining correction factors cannot be determined, their values remain at unity as well.

10. Adjust the N-value by the correction factors found in Step 9 (Equation 27 in Idriss & Boulanger, 2008, ***Ref. 10***).

$$N_{1,60} = N_1 \cdot C_E \cdot C_B \cdot C_R \cdot C_S \quad (5)$$

where,

$N_{1,60}$ = N-value corrected for overburden stress and equipment (bpf)

11. Adjust the N-value for the fines content (Equation 75 and 76 in Idriss & Boulanger, 2008, ***Ref. 10***). For fines content values greater than 35%, the value is 5.5 (Mitchell and Soga, 2005, ***Ref. 12***).

$\Delta N_{1,60}$ = exp (1.63 + (9.7/(FC+0.01)) – (15.7/(FC+0.01)2) (6)

where,

$N_{1,60cs}$ = N-value corrected for overburden stress, equipment, and fines content (bpf)

FC = Fines Content (%)

12. Calculate the cyclic resistance ration (CRR) for a standard magnitude 7.5 earthquake (Equation 70 in Idriss & Boulanger, 2008, ***Ref. 10***). The $CRR_{7.5}$ value is restricted to a maximum of 2.0 as higher values of $CRR_{7.5}$ yield extremely high values of F.S. which are meaningless for the purposes of this calculation.

$$CRR_{7.5} = \exp\left(\frac{N_{1,60cs}}{14.1} + \left(\frac{N_{1,60cs}}{126}\right)^2 - \left(\frac{N_{1,60cs}}{23.6}\right)^3 + \left(\frac{N_{1,60cs}}{25.4}\right)^4 - 2.8\right) \quad (7)$$

where,

$CRR_{7.5}$ = cyclic resistance ratio standardized for magnitude 7.5 earthquake

13. Calculate the stress reduction coefficient (Equation 22, 23, & 24 in Idriss & Boulanger, 2008, ***Ref. 10***).

$$r_d = \exp(\alpha(z) + \beta(z)M_W)$$

$$\alpha(z) = -1.012 - 1.126\,Sin\left(\frac{z}{11.73} + 5.133\right) \qquad (8)$$

$$\beta(z) = 0.106 + 0.118\,Sin\left(\frac{z}{11.28} + 5.142\right)$$

where,

M_W = Earthquake magnitude

r_d = stress reduction coefficient

z = depth below ground surface (m)

14. Calculate the cyclic stress ratio (CSR) (Equation 25 in Idriss & Boulanger, 2008, ***Ref. 10***).

$$CSR = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_v}{\sigma_v'}\right)r_d \qquad (9)$$

where,

CSR = cyclic stress ratio

a_{max} = peak ground acceleration considering location within embankment

g = gravitational constant (32.2 ft/s)

15. Calculate the overburden correction factor (Equation 54 and 56 in Idriss & Boulanger, 2008, ***Ref. 10***).

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma_v'}{P_a}\right) \le 1.1$$

$$C_\sigma = \frac{1}{18.9 - 2.55\sqrt{N_{1,60}}} \le 0.3 \qquad (10)$$

where,

K_σ = overburden correction factor

P_a = atmospheric pressure (2,116.2 psf)

16. Calculate the sloping ground correction factor (K_α), which accounts for static shear stresses on the soil layer. Youd (2001) recommends an adjustment for this, but does not supply the correction factors. Therefore, the correction factors are taken from Harder (1988, ***Ref. 3***) and Hynes (1988, ***Ref. 5***), as cited in Figure 61 of FHWA (1997). This figure is reprinted below as ***Figure 2-3***.

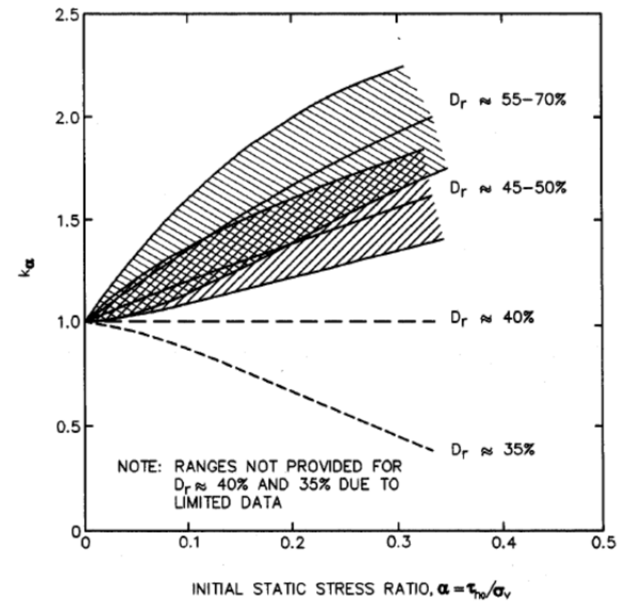

FIGURE 2 3
CURVES FOR ESTIMATION OF K□
(HARDER, 1988 AND HYNES, 1988, AS CITED IN FHWA, 1997)

The soil is assumed to be at 50% compaction, which would give a correction factor greater than or equal to 1.0. Therefore, a correction factor of 1.0 is conservatively used for this analysis.

17. Calculate the magnitude scaling factor (MSF). Youd et al. (2001) (***Ref. 9***) recommend using the revised Idriss scaling factors as a conservative lower bound.
The revised Idriss MSF is calculated according to Equation 51 in Idriss & Boulanger (2008) (***Ref. 10***).

$$MSF = 6.9\exp\left(\frac{-M_W}{4}\right) - 0.058 \le 1.8 \tag{11}$$

where,
M_w = earthquake magnitude (6.32 from ***Section 2.1.1***)

18. Calculate the factor of safety against liquefaction (Equation 30 in Youd et al., 2001, ***Ref. 9***). For the purposes of this calculation, the Factors of safety values have been restricted to a maximum of 2.0. This is because values of 1.8 and lower signify a reduction of strength in the soil layer. Factors of Safety values above 1.8 indicate that no liquefaction will occur. Therefore, high values of Factors of safety are capped at a round value of 2.0.

$$FS = \left(\frac{CRR_{7.5}}{CSR}\right) \cdot MSF \cdot K_\sigma \cdot K_\alpha \quad (12)$$

where,

FS = factor of safety against liquefaction

3.0 ASSUMPTIONS AND JUSTIFICATION

1. The relative compaction of the soil is assumed to be 50%.
2. A total (moist) unit weight of 115 pcf and a saturated unit weight of 120 pcf are used to calculate the effective vertical stress.
3. The fines content of Layer 1 is varied in the sensitivity case based on fines content values from the USCS Soil Classification tables (***Ref. 13***).
4. The phreatic surface is based on water levels on the boring logs and observed seepage at the toe of the embankment.

4.0 CALCULATION INPUT

1. The horizontal PGA for bedrock is 0.0513g (see ***Section 2.1.1***).
2. The average magnitude corresponding to the PGA 6.32 (see ***Section 2.1.1***).
3. The fines contents are based on the boring log descriptions (see ***Section 2.1.2***).
4. The phreatic surface is taken as the level observed from the boring logs (***Ref. 1***).
5. The uncorrected N-values, ground surface elevations, and elevations of the soil samples are taken from the boring logs (***Ref. 1***).

5.0 NUMERICAL CALCULATIONS

Numerical calculations are carried out in Excel to determine if the soil samples/layers at each boring will liquefy. The Excel worksheets for both the borings are attached as ***Appendix D***.

6.0 CALCULATION OUTPUT

N/A.

7.0 RESULTS

Table 7-1 summarizes the results of the liquefaction analysis. The full results are presented in the tables in ***Appendix D***.

TABLE 7-1
SUMMARY OF RESULTS OF LIQUEFACTION ANALYSIS FOR NORMAL WATER LEVEL

Boring	FS_{min}	Average Factor of Safety by layer		
		Layer 1	Layer 2	Layer 3
N-1	1.8	>2.0	>2.0	2.0
W-1	1.5	1.9	>2.0	2.0
N-1 (Sensitivity Case)	1.8	2.0	>2.0	2.0
W-1 (Sensitivity Case)	1.4	1.8	>2.0	2.0

The Factors of safety against liquefaction for all the soil layers except layer 1 at Boring W-1 are greater than 1.8. These soil layers will not liquefy. One sample in layer 1 of Boring W-1 has a F.S. less than 1.8.

8.0 CONCLUSION/SUMMARY

The Factors of safety against liquefaction for all the soil layers except layer 1 at Boring W-1 are greater than 1.8. This minimum required value is 1.8 below which there is a possibility of liquefaction or a reduction in strength of the soil. Therefore, these soil layers will not liquefy.

One sample in layer 1 of Boring W-1 has a F.S. less than 1.8. This layer will have a Reduction in Shear Strength (R_u) during the post-earthquake case of the stability analysis.

9.0 REFERENCES

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APPENDIX A

EARTHQUAKE MAGNITUDES AND DISTANCES CONTRIBUTING TO PGA (USGS, 2009)

*** Deaggregation of Seismic Hazard at One Period of Spectral Accel. ***
*** Data from U.S.G.S. National Seismic Hazards Mapping Project, 2008 version ***
PSHA Deaggregation. %contributions. site: Crisp_County long: 83.942 W., lat: 31.843 N.
Vs30(m/s)= 760.0 CEUS atten. model site cl BC(firm) or A(hard).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.05112 g. Weight * Computed_Rate_Ex 0.404E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00573
#This deaggregation corresponds to Mean Hazard w/all GMPEs

DIST(KM)	MAG(MW)	ALL_EPS	ε>2	1<ε<2	0<ε<1	-1<ε<0	-2<ε<-1	ε<-2
13.7	4.6	0.739	0.022	0.13	0.325	0.233	0.029	0
32.7	4.6	1.242	0.109	0.605	0.516	0.012	0	0
61.5	4.61	0.411	0.224	0.187	0	0	0	0
88.9	4.61	0.098	0.098	0	0	0	0	0
115.5	4.62	0.071	0.071	0	0	0	0	0
13.8	4.8	1.301	0.036	0.213	0.535	0.449	0.067	0
33.3	4.8	2.57	0.179	1.06	1.232	0.1	0	0
61.8	4.8	1.038	0.394	0.644	0	0	0	0
89.1	4.81	0.297	0.285	0.012	0	0	0	0
117.9	4.81	0.288	0.288	0	0	0	0	0
14	5.03	0.899	0.023	0.138	0.346	0.333	0.059	0.001
34	5.03	2.185	0.116	0.691	1.179	0.199	0	0
62.4	5.03	1.177	0.256	0.862	0.059	0	0	0
89.3	5.04	0.414	0.259	0.155	0	0	0	0
119.6	5.04	0.511	0.498	0.013	0	0	0	0
163.9	5.05	0.071	0.071	0	0	0	0	0
14.1	5.21	0.335	0.008	0.049	0.124	0.124	0.028	0.001
34.5	5.21	0.937	0.041	0.248	0.522	0.127	0	0
62.8	5.21	0.619	0.092	0.433	0.094	0	0	0
89.5	5.21	0.25	0.094	0.156	0	0	0	0
120.6	5.21	0.353	0.291	0.062	0	0	0	0
166.4	5.21	0.075	0.075	0	0	0	0	0
14.2	5.39	0.498	0.012	0.071	0.179	0.179	0.053	0.003
34.9	5.39	1.568	0.06	0.359	0.851	0.298	0.001	0
63.3	5.4	1.247	0.133	0.753	0.361	0	0	0
89.6	5.4	0.578	0.136	0.442	0	0	0	0
121.4	5.4	0.937	0.516	0.421	0	0	0	0
168.3	5.41	0.265	0.265	0	0	0	0	0
14.3	5.61	0.24	0.006	0.034	0.084	0.084	0.03	0.002
35.4	5.61	0.854	0.028	0.169	0.424	0.226	0.007	0
63.8	5.62	0.842	0.062	0.373	0.406	0.001	0	0
89.7	5.62	0.456	0.064	0.353	0.04	0	0	0
122.3	5.62	0.851	0.253	0.598	0	0	0	0
169.5	5.62	0.309	0.268	0.041	0	0	0	0
219.3	5.63	0.08	0.08	0	0	0	0	0
14.3	5.8	0.209	0.005	0.029	0.073	0.073	0.028	0.002
35.6	5.8	0.8	0.024	0.146	0.366	0.251	0.014	0
64.1	5.81	0.9	0.054	0.322	0.511	0.013	0	0
89.8	5.81	0.539	0.055	0.329	0.155	0	0	0
122.9	5.81	1.118	0.218	0.866	0.034	0	0	0
170.2	5.81	0.478	0.307	0.171	0	0	0	0
220.8	5.82	0.158	0.158	0	0	0	0	0
270.2	5.82	0.054	0.054	0	0	0	0	0
13.5	6.01	0.157	0.004	0.022	0.054	0.054	0.022	0.002
36.6	6.01	0.612	0.017	0.103	0.259	0.214	0.018	0
62.1	6	0.548	0.024	0.146	0.334	0.044	0	0
86.8	6.02	0.655	0.044	0.264	0.347	0	0	0
123.8	6.01	1.131	0.136	0.763	0.232	0	0	0
171.4	6.01	0.564	0.205	0.359	0	0	0	0
222.2	6.02	0.248	0.231	0.017	0	0	0	0
271.8	6.02	0.12	0.12	0	0	0	0	0
13.5	6.21	0.165	0.004	0.023	0.057	0.057	0.023	0.003
36.9	6.21	0.585	0.016	0.094	0.235	0.214	0.026	0
60.8	6.21	0.495	0.018	0.111	0.278	0.089	0	0
85.9	6.21	0.788	0.042	0.25	0.492	0.004	0	0
124.3	6.22	1.392	0.12	0.715	0.558	0	0	0
172	6.22	0.812	0.179	0.617	0.016	0	0	0
222.9	6.22	0.419	0.286	0.133	0	0	0	0
273	6.22	0.249	0.249	0	0	0	0	0
335.7	6.23	0.168	0.168	0	0	0	0	0
18	6.43	0.174	0.004	0.024	0.06	0.06	0.024	0.002
39.6	6.41	0.317	0.008	0.049	0.123	0.12	0.016	0
60.8	6.42	0.352	0.011	0.068	0.171	0.101	0	0
85.7	6.42	0.601	0.025	0.15	0.373	0.053	0	0
124.7	6.4	0.874	0.056	0.332	0.486	0	0	0
125	6.49	0.33	0.018	0.105	0.205	0.002	0	0
172.5	6.42	0.834	0.109	0.585	0.14	0	0	0
223.6	6.43	0.508	0.187	0.321	0	0	0	0
273.5	6.43	0.366	0.319	0.046	0	0	0	0
342.5	6.43	0.353	0.353	0	0	0	0	0
13	6.59	0.063	0.001	0.009	0.021	0.021	0.009	0.001
36.2	6.59	0.235	0.006	0.035	0.087	0.087	0.021	0

DIST(KM)	MAG(MW)	ALL_EPS	ε>2	1<ε<2	0<ε<1	-1<ε<0	-2<ε<-1	ε<-2
60.6	6.59	0.218	0.007	0.039	0.098	0.074	0	0
85.6	6.59	0.395	0.015	0.087	0.219	0.074	0	0
125.2	6.59	0.849	0.043	0.255	0.533	0.019	0	0
173.2	6.59	0.645	0.062	0.367	0.216	0	0	0
224	6.6	0.453	0.107	0.344	0.002	0	0	0
273.9	6.6	0.368	0.225	0.144	0	0	0	0
347.6	6.6	0.424	0.419	0.004	0	0	0	0
13.3	6.78	0.091	0.002	0.012	0.031	0.031	0.012	0.002
36.9	6.78	0.319	0.008	0.046	0.116	0.116	0.033	0.001
60.2	6.78	0.298	0.008	0.05	0.126	0.112	0.002	0
86	6.78	0.601	0.02	0.119	0.3	0.162	0	0
126	6.78	1.24	0.052	0.314	0.76	0.114	0	0
174	6.79	1.106	0.079	0.472	0.554	0	0	0
224.5	6.74	0.53	0.089	0.413	0.028	0	0	0
225.1	6.86	0.364	0.049	0.269	0.047	0	0	0
274.6	6.79	0.819	0.293	0.526	0	0	0	0
349.8	6.72	0.346	0.331	0.015	0	0	0	0
351.7	6.83	0.737	0.612	0.125	0	0	0	0
380.6	6.8	0.758	0.742	0.016	0	0	0	0
13.9	7	0.068	0.002	0.009	0.023	0.023	0.009	0.002
37	7	0.218	0.005	0.031	0.077	0.077	0.026	0.001
62.9	6.99	0.242	0.006	0.039	0.097	0.094	0.006	0
86.8	7.01	0.385	0.011	0.068	0.171	0.134	0	0
125.6	7	0.942	0.033	0.197	0.496	0.215	0	0
174.1	7	0.957	0.05	0.297	0.592	0.019	0	0
225	6.92	0.247	0.026	0.156	0.065	0	0	0
225.1	7.04	0.677	0.062	0.373	0.242	0	0	0
274.7	7.01	0.959	0.186	0.757	0.015	0	0	0
352.4	7.01	1.384	0.832	0.553	0	0	0	0
379	7.1	2.797	1.794	1.003	0	0	0	0
36.9	7.19	0.128	0.003	0.018	0.045	0.045	0.016	0.001
60.8	7.19	0.116	0.003	0.018	0.045	0.045	0.006	0
86.2	7.19	0.246	0.007	0.041	0.103	0.094	0.001	0
126	7.19	0.549	0.018	0.105	0.265	0.161	0	0
175.7	7.19	0.594	0.026	0.156	0.368	0.043	0	0
225.8	7.19	0.673	0.048	0.288	0.337	0	0	0
275.2	7.19	0.769	0.101	0.565	0.103	0	0	0
353.8	7.19	1.147	0.478	0.669	0	0	0	0
627.7	7.19	0.073	0.073	0	0	0	0	0
35.8	7.39	0.139	0.003	0.019	0.048	0.048	0.019	0.001
60.4	7.39	0.137	0.003	0.02	0.051	0.051	0.011	0
85.8	7.38	0.265	0.007	0.042	0.106	0.105	0.005	0
125.9	7.39	0.606	0.018	0.108	0.27	0.21	0	0
176.3	7.39	0.705	0.027	0.161	0.402	0.116	0	0
226.4	7.39	0.909	0.05	0.301	0.553	0.004	0	0
276.3	7.4	1.209	0.108	0.645	0.456	0	0	0
368	7.48	10.486	1.998	7.818	0.671	0	0	0
375.7	7.3	13.401	4.554	8.804	0.042	0	0	0
640.9	7.4	0.175	0.175	0	0	0	0	0
705.3	7.45	0.482	0.405	0.077	0	0	0	0
127.2	7.59	0.068	0.002	0.011	0.028	0.026	0	0
175.4	7.59	0.083	0.003	0.016	0.041	0.022	0	0
226.4	7.59	0.106	0.005	0.028	0.067	0.006	0	0
276.4	7.59	0.152	0.01	0.059	0.083	0	0	0
363.9	7.59	0.378	0.064	0.261	0.053	0	0	0
687.6	7.7	0.243	0.08	0.162	0	0	0	0
707.9	7.7	1.769	0.977	0.792	0	0	0	0
678.7	8	0.058	0.008	0.048	0.002	0	0	0
706.5	8	1.535	0.525	1.011	0	0	0	0
AVG =	6.32							

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 100.0
Mean src-site R= 221.5 km; M= 6.50; eps0= 0.51. Mean calculated for all sources.
Modal src-site R= 375.7 km; M= 7.30; eps0= 1.27 from peak (R,M) bin
MODE R*= 375.7km; M*= 7.30; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 8.804

Principal sources (faults, subduction, random seismicity having > 3% contribution)
Source Category: % contr. R(km) M epsilon0 (mean values).
New Madrid SZ no clustering 4.09 705.5 7.79 1.59
CEUS gridded 70.24 137.5 6.12 0.19
Charleston SC M<7.2; 2 zones 3.56 379.4 7.04 1.69
Charleston SC M>7.2; 2 zones 22.12 373.6 7.38 1.12
Individual fault hazard details if its contribution to mean hazard > 2%:
Fault ID % contr. Rcd(km) M epsilon0 Site-to-src azimuth(d)
New Madrid FZ, central 2.86 705.9 7.79 1.60 -48.2
#*********End of deaggregation corresponding to Mean Hazard w/all GMPEs *********#

APPENDIX B

BORING LOGS

BORING NUMBER N-1

Paul C. Rizzo Associates, Inc
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CLIENT Crisp County Power Commision
PROJECT NAME CCW Impoundment Evaluation
PROJECT NUMBER 14-5232
PROJECT LOCATION Warwick, GA
DATE STARTED 5/1/14 **COMPLETED** 5/1/14
GROUND ELEVATION 245.2 ft **HOLE SIZE** 2 inches
DRILLING CONTRACTOR GEC
GROUND WATER LEVELS:
DRILLING METHOD Mud Rotary
AT TIME OF DRILLING ---
LOGGED BY CHG **CHECKED BY** JPO
AT END OF DRILLING ---
NOTES Borehole grouted 5/2/14
14hrs AFTER DRILLING 16.30 ft / Elev 228.90 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0		(sm) Loose, moist, orange, orange-brown silty sand with some clay (FILL)	SPT 1	100	2-1-2 (3)							
			SPT 2	100	1-3-6 (9)							
5		(sm) Loose, moist, orange, orange-brown silty sand with some clay (FILL)	SPT 3	100	2-7-9 (16)							
			SPT 4	100	6-10-15 (25)			9	19	14	5	
10		(SM) Medium Dense, moist, orange, orange-brown silty sand with some clay (FILL)	SPT 5	100	9-10-12 (22)			10				23
		(sp-sm) Medium Dense, moist, grey, grey-brown slightly silty to silty sand with trace organics (FILL)	SPT 6	100	3-9-9 (18)							
		(SM) Medium Dense, moist, grey, grey-brown silty sand with trace organics (FILL)	SPT 7	100	7-7-4 (11)			13				20
15		(sc-sm) Loose to Medium Dense, moist, dark grey silty clayey sand with trace organics (FILL)	SPT 8	100	6-6-7 (13)							
		(SC-SM) Loose to Medium Dense, moist to wet, dark grey silty clayey sand with trace organics (FILL)	SPT 9	100	4-3-3 (6)			15	41	13	28	
20		(sc-sm) Medium Dense, moist, dark grey silty clayey sand with trace organics (FILL)	SPT 10	100	4-7-6 (13)			10				28
		(sc-sm) Medium dense, moist, mottled grey, brown, and orange silty clayey sand	SPT 11	100	5-7-5 (12)							
			SPT 12	100	5-6-4 (10)							
25		(CH) Stiff light grey-blue clay with phosphitic pebbles and quartz gravels	SPT 13	100	4-4-7 (11)			23	63	20	43	
		(cl) Stiff light grey-blue clay with phosphitic pebbles and quartz gravels	SPT 14	100	3-4-8 (12)							
30			SPT 15	100	5-5-5 (10)							
			SPT 16	100	2-2-9 (11)							
		(cl) Hard white and light tan calcareous clay with limestone fragments and lenses (decomposed limestone)	SPT 17	100	22-24-32 (56)							
35			SPT 18	100	8-7-42 (49)			13				
			SPT 19	100	15-18-30 (48)							
40			SPT 20	100	14-17-19 (36)							

(Continued Next Page)

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BORING NUMBER N-1

CLIENT Crisp County Power Commision **PROJECT NAME** CCW Impoundment Evaluation

PROJECT NUMBER 14-5232 **PROJECT LOCATION** Warwick, GA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
40		(cl) Hard white and light tan calcareous clay with limestone fragments and lenses (decomposed limestone) *(continued)*	SPT 21	100	21-22-26 (48)							
			SPT 22	100	50/3"							
45			SPT 23	100	13-30-9 (39)			45				
		(CL) Stiff white and light tan calcareous clay with limestone fragments and lenses (decomposed limestone)	SPT 24	100	3-3-11 (14)			32	34	27	7	
			SPT 25	100	2-3-7 (10)			53				61

Bottom of borehole at 49.5 feet.

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BORING NUMBER N-2

CLIENT Crisp County Power Commision
PROJECT NAME CCW Impoundment Evaluation
PROJECT NUMBER 14-5232
PROJECT LOCATION Warwick, GA
DATE STARTED 5/15/14 **COMPLETED** 5/15/14
GROUND ELEVATION **HOLE SIZE** 2 inches
DRILLING CONTRACTOR GEC
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger 2"
▽ **AT TIME OF DRILLING** 5.00 ft
LOGGED BY CHG **CHECKED BY** JPO
AT END OF DRILLING ---
NOTES Backfilled with bentonite chips 5/15
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(sc-sm) Very loose to loose, moist, mottled grey, brown, and orange silty clayey sand	SPT 1	100	3-2-3 (5)							
			SPT 2	100	1-1-1 (2)							
5		▽ (sc-sm) Very loose to loose, wet, mottled grey, brown, and orange silty clayey sand	SPT 3	100	1-1-2 (3)							
			SPT 4	100	1-3-3 (6)							
10			SPT 5	100	1-3-5 (8)							
		(cl) Hard white and light tan calcareous clay with limestone fragments and lenses (decomposed limestone)	SPT 6	100	6-12-28 (40)							
			SPT 7	100	12-50							
15			SPT 8	100	12-50/5"							
		(cl) Very stiff white and light tan calcareous clay with limestone fragments and lenses (decomposed limestone)	SPT 9	100	4-10-12 (22)							
20		(cl) Hard light blueish gray clay with limestone fragments and lenses (decomposed limestone)	SPT 10	100	18-50							
			SPT 11	100	7-12-23 (35)							
			SPT 12	100	16-50/3"							

Bottom of borehole at 24.5 feet.

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BORING NUMBER W-1

CLIENT Crisp County Power Commision
PROJECT NAME CCW Impoundment Evaluation
PROJECT NUMBER 14-5232
PROJECT LOCATION Warwick, GA
DATE STARTED 5/1/14 **COMPLETED** 5/1/14
GROUND ELEVATION 243.9 ft **HOLE SIZE** 2 inches
DRILLING CONTRACTOR GEC
GROUND WATER LEVELS:
DRILLING METHOD Mud Rotary
AT TIME OF DRILLING ---
LOGGED BY CHG **CHECKED BY** JPO
AT END OF DRILLING ---
NOTES Borehole grouted 5/2/14, topped with bentonite chips on 5/15
19hrs AFTER DRILLING 15.10 ft / Elev 228.80 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(sm) Loose to medium dense, moist, orange, Brown, Grey Silty Sand with gravels (FILL)	SPT 1	100	2-2-3 (5)							
			SPT 2	100	7-9-13 (22)							
5		(SM) Medium dense, moist, orange, Brown, Grey Silty Sand with gravels (FILL)	SPT 3	100	7-7-9 (16)			13				24
		(sm) Medium dense, moist, orange, Brown, Grey Silty Sand with gravels (FILL)	SPT 4	100	11-9-8 (17)							
			SPT 5	100	2-4-9 (13)							
10		(sm) Medium dense, moist, dark Grey Silty Sand with trace organics (wood and root fibers) (FILL)	SPT 6	100	7-10-14 (24)							
		(SM) Medium dense, moist, dark Grey Silty Sand with trace organics (wood and root fibers) (FILL)	SPT 7	100	8-8-11 (19)			10				28
15		(sm) Medium dense, moist, dark Grey Silty Sand with trace organics (wood and root fibers) (FILL)	SPT 8	100	9-8-9 (17)							
			SPT 9	100	8-9-10 (19)							
			SPT 10	100	4-6-7 (13)							
20		(SM) Loose, moist, dark Grey Silty Sand with trace organics (wood and root fibers) (FILL)	SPT 11	100	3-4-4 (8)			13				27
		(SC-SM) Very Loose, moist, dark Grey Silty Clayey Sand with trace organics (wood and root fibers)	SPT 12	100	2-1-3 (4)			16				30
25		(CH) Stiff Light Grey-Blue Clay (Marl)	SPT 13	100	4-6-9 (15)			18	61	19	42	
		(cl) Stiff White and Light Tan Calcareous Clay with Limestone fragments and lenses (decomposed limestone)	SPT 14	100	4-6-7 (13)							
		(cl) Very stiff to hard White and Light Tan Calcareous Clay with Limestone fragments and lenses (decomposed limestone)	SPT 15	100	6-10-13 (23)							
30			SPT 16	100	12-25-24 (49)							
			SPT 17	100	17-15-20 (35)							
35			SPT 18	100	12-50/3"							
			SPT 19	100	14-13-19 (32)			29				57
40			SPT 20	100	10-20-50/1"							

(Continued Next Page)

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BORING NUMBER W-1

CLIENT Crisp County Power Commision
PROJECT NAME CCW Impoundment Evaluation
PROJECT NUMBER 14-5232
PROJECT LOCATION Warwick, GA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
40		(cl) Very stiff to hard White and Light Tan Calcareous Clay with Limestone fragments and lenses (decomposed limestone) *(continued)*	SPT 21	100	9-16-21 (37)							
		(cl) Stiff to very stiff White and Light Tan Calcareous Clay with Limestone fragments and lenses (decomposed limestone)	SPT 22	100	9-6-4 (10)							
45			SPT 23	100	1-2-9 (11)							
			SPT 24	100	4-3-13 (16)							
		(cl) Hard White and Light Tan Calcareous Clay with Limestone fragments and lenses (decomposed limestone)	SPT 25	100	50/2"							

Bottom of borehole at 49.5 feet.

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BORING NUMBER W-2

CLIENT Crisp County Power Commision
PROJECT NAME CCW Impoundment Evaluation
PROJECT NUMBER 14-5232
PROJECT LOCATION Warwick, GA
DATE STARTED 5/2/14 **COMPLETED** 5/2/14
GROUND ELEVATION 228.64 ft **HOLE SIZE** 2 inches
DRILLING CONTRACTOR GEC
GROUND WATER LEVELS:
DRILLING METHOD Mud Rotary
AT TIME OF DRILLING ---
LOGGED BY CHG **CHECKED BY** JPO
AT END OF DRILLING ---
NOTES Borehole grouted 5/2/14
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0	(sc) Red-orange, moist, silty clayey sand roadbase (Hand Auger to 4')										
	(sm) Medium dense, moist, dark grey organic slightly silty to silty sand										
5		SPT 1	100	4-8-7 (15)							
	(sc-sm) Very loose, moist, grey-brown mottled orange silty clayey sand with trace organics	SPT 2	100	2-1-2 (3)							
	(SC-SM) Medium dense, moist, grey-brown mottled orange silty clayey sand with trace organics	SPT 3	100	4-5-6 (11)			15				45
10	(sc) Dense, moist, light grey-blue clayey sand with limestone fragments and phosphitic pebbles	SPT 4	100	6-9-24 (33)							
	(SC) Medium dense, moist, light grey-blue clayey sand with limestone fragments and phosphitic pebbles	SPT 5	100	5-6-6 (12)			15	46	15	31	44
15	(sc) Loose light grey-blue clayey sand with limestone fragments and phosphitic pebbles	SPT 6	100	3-4-4 (8)							
	(cl) Hard white and light tan calcareous clay with limestone fragments and sand (decomposed limestone)	SPT 7	100	20-20-17 (37)							
	(CL) Hard white and light tan calcareous clay with limestone fragments and sand (decomposed limestone)	SPT 8	100	20-18-50/4"			22				65
20	(cl) Hard white and light tan calcareous clay with limestone fragments and sand (decomposed limestone)	SPT 9	100	12-17-20 (37)							
		SPT 10	100	50/3"							
25	(CL) Very stiff white and light tan calcareous clay with limestone fragments and sand (decomposed limestone)	SPT 11	100	4-9-14 (23)			28				58
	(cl) Hard white and light tan calcareous clay with limestone fragments and sand (decomposed limestone)	SPT 12	100	12-28-19 (47)							
		SPT 13	100	11-50/5"							
30		SPT 14	100	9-13-16 (29)							
		SPT 15	100	25-50/1"							
35		SPT 16	100	16-9-27 (36)							

Bottom of borehole at 35.5 feet.

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APPENDIX C

SELECTED PAGES FROM YOUD ET AL. (2001)

Preface

Evaluation of soil liquefaction resistance is an important aspect of geotechnical engineering practice. To update and enhance criteria that are routinely applied in practice, workshops were convened in 1996 and 1998 to gain consensus from 20 experts on updates and augmentations that should be made to standard procedures that have evolved over the past 30 years. At the outset, the goal was to develop this state-of-the-art summary of consensus recommendations. A commitment was also made to those who participated in the workshops that all would be listed as co-authors. Unfortunately, the previous publication of this summary paper (April 2001) listed only the co-chairs of the workshop, Profs. Youd and Idriss, as authors; the remaining workshop participants were acknowledged in a footnote. In order to correct this error and to fully acknowledge and credit those who significantly contributed to the work, this paper is being republished in its entirety, at the request of the journal's editors, with all the participants named as co-authors. All further reference to this paper should be to this republication. The previous publication should no longer be cited. Also, several minor errors are corrected in this republication.

LIQUEFACTION RESISTANCE OF SOILS: SUMMARY REPORT FROM THE 1996 NCEER AND 1998 NCEER/NSF WORKSHOPS ON EVALUATION OF LIQUEFACTION RESISTANCE OF SOILS[a]

By T. L. Youd,[1] Chair, Member, ASCE, I. M. Idriss,[2] Co-Chair, Fellow, ASCE, Ronald D. Andrus,[3] Ignacio Arango,[4] Gonzalo Castro,[5] John T. Christian,[6] Richardo Dobry,[7] W. D. Liam Finn,[8] Leslie F. Harder Jr.,[9] Mary Ellen Hynes,[10] Kenji Ishihara,[11] Joseph P. Koester,[12] Sam S. C. Liao,[13] William F. Marcuson III,[14] Geoffrey R. Martin,[15] James K. Mitchell,[16] Yoshiharu Moriwaki,[17] Maurice S. Power,[18] Peter K. Robertson,[19] Raymond B. Seed,[20] and Kenneth H. Stokoe II[21]

ABSTRACT: Following disastrous earthquakes in Alaska and in Niigata, Japan in 1964, Professors H. B. Seed and I. M. Idriss developed and published a methodology termed the ‘‘simplified procedure’’ for evaluating liquefaction resistance of soils. This procedure has become a standard of practice throughout North America and much of the world. The methodology which is largely empirical, has evolved over years, primarily through summary papers by H. B. Seed and his colleagues. No general review or update of the procedure has occurred, however, since 1985, the time of the last major paper by Professor Seed and a report from a National Research Council workshop on liquefaction of soils. In 1996 a workshop sponsored by the National Center for Earthquake Engineering Research (NCEER) was convened by Professors T. L. Youd and I. M. Idriss with 20 experts to review developments over the previous 10 years. The purpose was to gain consensus on updates and augmentations to the simplified procedure. The following topics were reviewed and recommendations developed: (1) criteria based on standard penetration tests; (2) criteria based on cone penetration tests; (3) criteria based on shear-wave velocity measurements; (4) use of the Becker penetration test for gravelly soil; (4) magnitude scaling factors; (5) correction factors for overburden pressures and sloping ground; and (6) input values for earthquake magnitude and peak acceleration. Probabilistic and seismic energy analyses were reviewed but no recommendations were formulated.

[a]This Summary Report, originally published in April 2001, is being republished so that the contribution of all workshop participants as authors can be officially recognized. The original version listed only two authors, plus a list of 19 workshop participants. This was incorrect; all 21 individuals should have been identified as authors. ASCE deeply regrets the error.

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Note. Discussion open until March 1, 2002. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on January 18, 2000; revised November 14, 2000. This paper is part of the ***Journal of Geotechnical and Geoenvironmental Engineering***, Vol. 127, No. 10, October, 2001. ©ASCE, ISSN 1090-0241/01/0010-0817–0833/$8.00 + $.50 per page. Paper No. 22223.

INTRODUCTION

Over the past 25 years a methodology termed the "simplified procedure" has evolved as a standard of practice for evaluating the liquefaction resistance of soils. Following disastrous earthquakes in Alaska and in Niigata, Japan in 1964, Seed and Idriss (1971) developed and published the basic "simplified procedure." That procedure has been modified and improved periodically since that time, primarily through landmark papers by Seed (1979), Seed and Idriss (1982), and Seed et al. (1985). In 1985, Professor Robert V. Whitman convened a workshop on behalf of the National Research Council (NRC) in which 36 experts and observers thoroughly reviewed the state-of-knowledge and the state-of-the-art for assessing liquefaction hazard. That workshop produced a report (NRC 1985) that has become a widely used standard and reference for liquefaction hazard assessment. In January 1996, T. L. Youd and I. M. Idriss convened a workshop of 20 experts to update the simplified procedure and incorporate research findings from the previous decade. This paper summarizes recommendations from that workshop (Youd and Idriss 1997).

To keep the workshop focused, the scope of the workshop was limited to procedures for evaluating liquefaction resistance of soils under level to gently sloping ground. In this context, liquefaction refers to the phenomena of seismic generation of large pore-water pressures and consequent softening of granular soils. Important postliquefaction phenomena, such as residual shear strength, soil deformation, and ground failure, were beyond the scope of the workshop.

The simplified procedure was developed from empirical evaluations of field observations and field and laboratory test data. Field evidence of liquefaction generally consisted of surficial observations of sand boils, ground fissures, or lateral spreads. Data were collected mostly from sites on level to gently sloping terrain, underlain by Holocene alluvial or fluvial sediment at shallow depths (<15 m). The original procedure was verified for, and is applicable only to, these site conditions. Similar restrictions apply to the implementation of the updated procedures recommended in this report.

Liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore-water pressure and reduced effective stress (Marcuson 1978). Increased pore-water pressure is induced by the tendency of granular materials to compact when subjected to cyclic shear deformations. The change of state occurs most readily in loose to moderately dense granular soils with poor drainage, such as silty sands or sands and gravels capped by or containing seams of impermeable sediment. As liquefaction occurs, the soil stratum softens, allowing large cyclic deformations to occur. In loose materials, the softening is also accompanied by a loss of shear strength that may lead to large shear deformations or even flow failure under moderate to high shear stresses, such as beneath a foundation or sloping ground. In moderately dense to dense materials, liquefaction leads to transient softening and increased cyclic shear strains, but a tendency to dilate during shear inhibits major strength loss and large ground deformations. A condition of cyclic mobility or cyclic liquefaction may develop following liquefaction of moderately dense granular materials. Beneath gently sloping to flat ground, liquefaction may lead to ground oscillation or lateral spread as a consequence of either flow deformation or cyclic mobility. Loose soils also compact during liquefaction and reconsolidation, leading to ground settlement. Sand boils may also erupt as excess pore water pressures dissipate.

CYCLIC STRESS RATIO (CSR) AND CYCLIC RESISTANCE RATIO (CRR)

Calculation, or estimation, of two variables is required for evaluation of liquefaction resistance of soils: (1) the seismic demand on a soil layer, expressed in terms of CSR; and (2) the capacity of the soil to resist liquefaction, expressed in terms of CRR. The latter variable has been termed the cyclic stress ratio or the cyclic stress ratio required to generate liquefaction, and has been given different symbols by different writers. For example, Seed and Harder (1990) used the symbol CSRℓ, Youd (1993) used the symbol CSRL, and Kramer (1996) used the symbol CSR_L to denote this ratio. To reduce confusion and to better distinguish induced cyclic shear stresses from mobilized liquefaction resistance, the capacity of a soil to resist liquefaction is termed the CRR in this report. This term is recommended for engineering practice.

EVALUATION OF CSR

Seed and Idriss (1971) formulated the following equation for calculation of the cyclic stress ratio:

$$\mathrm{CSR} = (\tau_{av}/\sigma'_{vo}) = 0.65(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \qquad (1)$$

where a_{max} = peak horizontal acceleration at the ground surface generated by the earthquake (discussed later); g = acceleration of gravity; σ_{vo} and σ'_{vo} are total and effective vertical overburden stresses, respectively; and r_d = stress reduction coefficient. The latter coefficient accounts for flexibility of the soil profile. The workshop participants recommend the following minor modification to the procedure for calculation of CSR.

For routine practice and noncritical projects, the following equations may be used to estimate average values of r_d (Liao and Whitman 1986b):

$$r_d = 1.0 - 0.00765z \quad \text{for } z \leq 9.15 \text{ m} \qquad (2a)$$

$$r_d = 1.174 - 0.0267z \quad \text{for } 9.15 \text{ m} < z \leq 23 \text{ m} \qquad (2b)$$

where z = depth below ground surface in meters. Some investigators have suggested additional equations for estimating r_d at greater depths (Robertson and Wride 1998), but evaluation of liquefaction at these greater depths is beyond the depths where the simplified procedure is verified and where routine applications should be applied. Mean values of r_d calculated from (2) are plotted in Fig. 1, along with the mean and range of values proposed by Seed and Idriss (1971). The workshop participants agreed that for convenience in programming spreadsheets and other electronic aids, and to be consistent with past practice, r_d values determined from (2) are suitable for use in routine engineering practice. The user should understand, however, that there is considerable variability in the

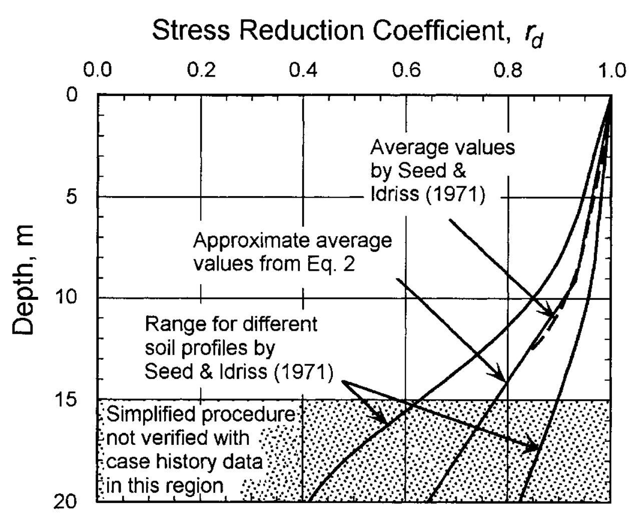

FIG. 1. r_d versus Depth Curves Developed by Seed and Idriss (1971) with Added Mean-Value Lines Plotted from Eq. (2)

flexibility and thus r_d at field sites, that r_d calculated from (2) are the mean of a wide range of possible r_d, and that the range of r_d increases with depth (Golesorkhi 1989).

For ease of computation, T. F. Blake (personal communication, 1996) approximated the mean curve plotted in Fig. 1 by the following equation:

$$r_d = \frac{(1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5})}{(1.000 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^2)} \quad (3)$$

where z = depth beneath ground surface in meters. Eq. (3) yields essentially the same values for r_d as (2), but is easier to program and may be used in routine engineering practice.

I. M. Idriss [Transportation Research Board (TRB) (1999)] suggested a new procedure for determining magnitude-dependent values of r_d. Application of these r_d require use of a corresponding set of magnitude scaling factors that are compatible with the new r_d. Because these r_d were developed after the workshop and have not been independently evaluated by other experts, the workshop participants chose not to recommend the new factors at this time.

EVALUATION OF LIQUEFACTION RESISTANCE (CRR)

A major focus of the workshop was on procedures for evaluating liquefaction resistance. A plausible method for evaluating CRR is to retrieve and test undisturbed soil specimens in the laboratory. Unfortunately, in situ stress states generally cannot be reestablished in the laboratory, and specimens of granular soils retrieved with typical drilling and sampling techniques are too disturbed to yield meaningful results. Only through specialized sampling techniques, such as ground freezing, can sufficiently undisturbed specimens be obtained. The cost of such procedures is generally prohibitive for all but the most critical projects. To avoid the difficulties associated with sampling and laboratory testing, field tests have become the state-of-practice for routine liquefaction investigations.

Several field tests have gained common usage for evaluation of liquefaction resistance, including the standard penetration test (SPT), the cone penetration test (CPT), shear-wave velocity measurements (V_s), and the Becker penetration test (BPT). These tests were discussed at the workshop, along with associated criteria for evaluating liquefaction resistance. The participants made a conscientious attempt to correlate liquefaction resistance criteria from each of the various field tests to provide generally consistent results, no matter which test is applied. SPTs and CPTs are generally preferred because of the more extensive databases and past experience, but the other tests may be applied at sites underlain by gravelly sediment or where access by large equipment is limited. Primary advantages and disadvantages of each test are listed in Table 1.

SPT

Criteria for evaluation of liquefaction resistance based on the SPT have been rather robust over the years. Those criteria are largely embodied in the CSR versus $(N_1)_{60}$ plot reproduced

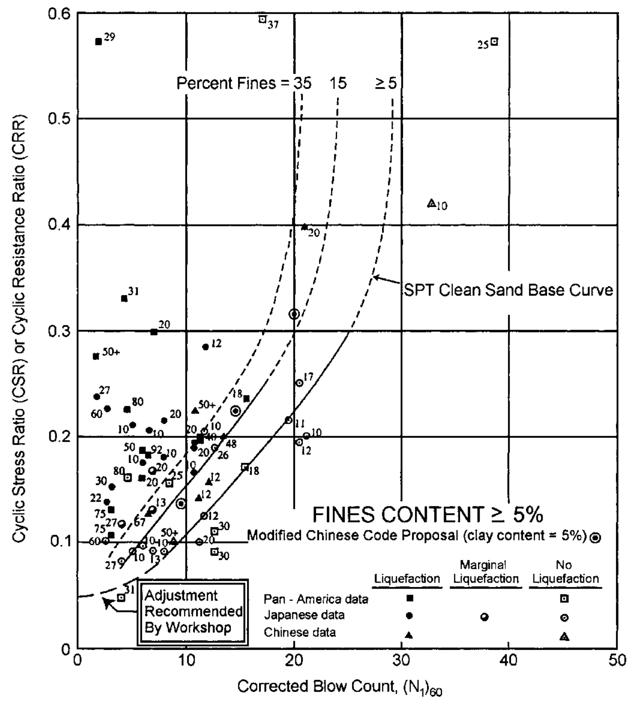

FIG. 2. SPT Clean-Sand Base Curve for Magnitude 7.5 Earthquakes with Data from Liquefaction Case Histories (Modified from Seed et al. 1985)

in Fig. 2. $(N_1)_{60}$ is the SPT blow count normalized to an overburden pressure of approximately 100 kPa (1 ton/sq ft) and a hammer energy ratio or hammer efficiency of 60%. The normalization factors for these corrections are discussed in the section entitled Other Corrections. Fig. 2 is a graph of calculated CSR and corresponding $(N_1)_{60}$ data from sites where liquefaction effects were or were not observed following past earthquakes with magnitudes of approximately 7.5. CRR curves on this graph were conservatively positioned to separate regions with data indicative of liquefaction from regions with data indicative of nonliquefaction. Curves were developed for granular soils with the fines contents of 5% or less, 15%, and 35% as shown on the plot. The CRR curve for fines contents <5% is the basic penetration criterion for the simplified procedure and is referred to hereafter as the "SPT clean-sand base curve." The CRR curves in Fig. 2 are valid only for magnitude 7.5 earthquakes. Scaling factors to adjust CRR curves to other magnitudes are addressed in a later section of this report.

SPT Clean-Sand Base Curve

Several changes to the SPT criteria are recommended by the workshop participants. The first change is to curve the trajec-

TABLE 1. Comparison of Advantages and Disadvantages of Various Field Tests for Assessment of Liquefaction Resistance

Feature	SPT (Test Type)	CPT (Test Type)	V_s (Test Type)	BPT (Test Type)
Past measurements at liquefaction sites	Abundant	Abundant	Limited	Sparse
Type of stress-strain behavior influencing test	Partially drained, large strain	Drained, large strain	Small strain	Partially drained, large strain
Quality control and repeatability	Poor to good	Very good	Good	Poor
Detection of variability of soil deposits	Good for closely spaced tests	Very good	Fair	Fair
Soil types in which test is recommended	Nongravel	Nongravel	All	Primarily gravel
Soil sample retrieved	Yes	No	No	No
Test measures index or engineering property	Index	Index	Engineering	Index

tory of the clean-sand base curve at low $(N_1)_{60}$ to a projected intercept of about 0.05 (Fig. 2). This adjustment reshapes the clean-sand base curve to achieve greater consistency with CRR curves developed for the CPT and shear-wave velocity procedures. Seed and Idriss (1982) projected the original curve through the origin, but there were few data to constrain the curve in the lower part of the plot. A better fit to the present empirical data is to bow the lower end of the base curve as indicated in Fig. 2.

At the University of Texas, A. F. Rauch (personal communication, 1998), approximated the clean-sand base curve plotted in Fig. 2 by the following equation:

$$\mathrm{CRR}_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{1}{200} \quad (4)$$

This equation is valid for $(N_1)_{60} < 30$. For $(N_1)_{60} \geq 30$, clean granular soils are too dense to liquefy and are classed as nonliquefiable. This equation may be used in spreadsheets and other analytical techniques to approximate the clean-sand base curve for routine engineering calculations.

Influence of Fines Content

In the original development, Seed et al. (1985) noted an apparent increase of CRR with increased fines content. Whether this increase is caused by an increase of liquefaction resistance or a decrease of penetration resistance is not clear. Based on the empirical data available, Seed et al. developed CRR curves for various fines contents reproduced in Fig. 2. A revised correction for fines content was developed by workshop attendees to better fit the empirical database and to better support computations with spreadsheets and other electronic computational aids.

The workshop participants recommend (5) and (6) as approximate corrections for the influence of fines content (FC) on CRR. Other grain characteristics, such as soil plasticity, may affect liquefaction resistance as well as fines content, but widely accepted corrections for these factors have not been developed. Hence corrections based solely on fines content should be used with engineering judgment and caution. The following equations were developed by I. M. Idriss with the assistance of R. B. Seed for correction of $(N_1)_{60}$ to an equivalent clean sand value, $(N_1)_{60cs}$:

$$(N_1)_{60cs} = \alpha + \beta (N_1)_{60} \quad (5)$$

where α and β = coefficients determined from the following relationships:

$$\alpha = 0 \quad \text{for FC} \leq 5\% \quad (6a)$$

$$\alpha = \exp[1.76 - (190/\mathrm{FC}^2)] \quad \text{for } 5\% < \text{FC} < 35\% \quad (6b)$$

$$\alpha = 5.0 \quad \text{for FC} \geq 35\% \quad (6c)$$

$$\beta = 1.0 \quad \text{for FC} \leq 5\% \quad (7a)$$

$$\beta = [0.99 + (\mathrm{FC}^{1.5}/1{,}000)] \quad \text{for } 5\% < \text{FC} < 35\% \quad (7b)$$

$$\beta = 1.2 \quad \text{for FC} \geq 35\% \quad (7c)$$

These equations may be used for routine liquefaction resistance calculations. A back-calculated curve for a fines content of 35% is essentially congruent with the 35% curve plotted in Fig. 2. The back-calculated curve for a fines contents of 15% plots to the right of the original 15% curve.

Other Corrections

Several factors in addition to fines content and grain characteristics influence SPT results, as noted in Table 2. Eq. (8) incorporates these corrections

TABLE 2. Corrections to SPT (Modified from Skempton 1986) as Listed by Robertson and Wride (1998)

Factor	Equipment variable	Term	Correction
Overburden pressure	—	C_N	$(P_a/\sigma'_{vo})^{0.5}$
Overburden pressure	—	C_N	$C_N \leq 1.7$
Energy ratio	Donut hammer	C_E	0.5–1.0
Energy ratio	Safety hammer	C_E	0.7–1.2
Energy ratio	Automatic-trip Donut-type hammer	C_E	0.8–1.3
Borehole diameter	65–115 mm	C_B	1.0
Borehole diameter	150 mm	C_B	1.05
Borehole diameter	200 mm	C_B	1.15
Rod length	<3 m	C_R	0.75
Rod length	3–4 m	C_R	0.8
Rod length	4–6 m	C_R	0.85
Rod length	6–10 m	C_R	0.95
Rod length	10–30 m	C_R	1.0
Sampling method	Standard sampler	C_S	1.0
Sampling method	Sampler without liners	C_S	1.1–1.3

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \quad (8)$$

where N_m = measured standard penetration resistance; C_N = factor to normalize N_m to a common reference effective overburden stress; C_E = correction for hammer energy ratio (ER); C_B = correction factor for borehole diameter; C_R = correction factor for rod length; and C_S = correction for samplers with or without liners.

Because SPT N-values increase with increasing effective overburden stress, an overburden stress correction factor is applied (Seed and Idriss 1982). This factor is commonly calculated from the following equation (Liao and Whitman 1986a):

$$C_N = (P_a/\sigma'_{vo})^{0.5} \quad (9)$$

where C_N normalizes N_m to an effective overburden pressure σ'_{vo} of approximately 100 kPa (1 atm) P_a. C_N should not exceed a value of 1.7 [A maximum value of 2.0 was published in the National Center for Earthquake Engineering Research (NCEER) workshop proceedings (Youd and Idriss 1997), but later was reduced to 1.7 by consensus of the workshop participants] Kayen et al. (1992) suggested the following equation, which limits the maximum C_N value to 1.7, and in these writers' opinion, provides a better fit to the original curve specified by Seed and Idriss (1982):

$$C_N = 2.2/(1.2 + \sigma'_{vo}/P_a) \quad (10)$$

Either equation may be used for routine engineering applications.

The effective overburden pressure σ'_{vo} applied in (9) and (10) should be the overburden pressure at the time of drilling and testing. Although a higher ground-water level might be used for conservatism in the liquefaction resistance calculations, the C_N factor must be based on the stresses present at the time of the testing.

The C_N correction factor was derived from SPT performed in test bins with large sand specimens subjected to various confining pressures (Gibbs and Holtz 1957; Marcuson and Bieganousky 1997a,b). The results of several of these tests are reproduced in Fig. 3 in the form of C_N curves versus effective overburden stress (Castro 1995). These curves indicate considerable scatter of results with no apparent correlation of C_N with soil type or gradation. The curves from looser sands, however, lie in the lower part of the C_N range and are reasonably approximated by (9) and (10) for low effective overburden pressures [200 kPa (<2 tsf)]. The workshop participants endorsed the use of (9) for calculation of C_N, but acknowledged that for overburden pressures >200 kPa (2 tsf) the results are uncertain. Eq. (10) provides a better fit for overburden

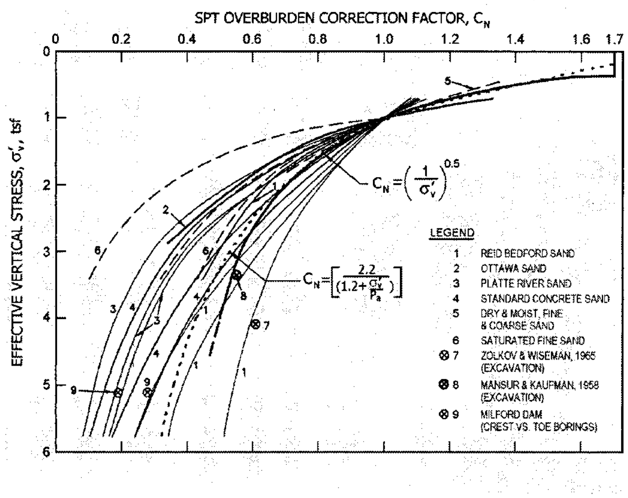

FIG. 3. C_N Curves for Various Sands Based on Field and Laboratory Test Data along with Suggested C_N Curve Determined from Eqs. (9) and (10) (Modified from Castro 1995)

pressures up to 300 kPa (3 tsf). For pressures >300 kPa (3 tsf), the uncertainty is so great that (9) should not be applied. At these high pressures, which are generally below the depth for which the simplified procedure has been verified, C_N should be estimated by other means.

Another important factor is the energy transferred from the falling hammer to the SPT sampler. An ER of 60% is generally accepted as the approximate average for U.S. testing practice and as a reference value for energy corrections. The ER delivered to the sampler depends on the type of hammer, anvil, lifting mechanism, and the method of hammer release. Approximate correction factors (C_E = ER/60) to modify the SPT results to a 60% energy ratio for various types of hammers and anvils are listed in Table 2. Because of variations in drilling and testing equipment and differences in testing procedures, a rather wide range in the energy correction factor C_E has been observed as noted in the table. Even when procedures are carefully monitored to conform to established standards, such as ASTM D 1586-99, some variation in C_E may occur because of minor variations in testing procedures. Measured energies at a single site indicate that variations in energy ratio between blows or between tests in a single borehole typically vary by as much as 10%. The workshop participants recommend measurement of the hammer energy frequently at each site where the SPT is used. Where measurements cannot be made, careful observation and notation of the equipment and procedures are required to estimate a C_E value for use in liquefaction resistance calculations. Use of good-quality testing equipment and carefully controlled testing procedures conforming to ASTM D 1586-99 will generally yield more consistent energy ratios and C_E with values from the upper parts of the ranges listed in Table 2.

Skempton (1986) suggested and Robertson and Wride (1998) updated correction factors for rod lengths <10 m, borehole diameters outside the recommended interval (65–125 mm), and sampling tubes without liners. Range for these correction factors are listed in Table 2. For liquefaction resistance calculations and rod lengths <3 m, a C_R of 0.75 should be applied as was done by Seed et al. (1985) in formulating the simplified procedure. Although application of rod-length correction factors listed in Table 2 will give more precise $(N_1)_{60}$ values, these corrections may be neglected for liquefaction resistance calculations for rod lengths between 3 and 10 m because rod-length corrections were not applied to SPT test data from these depths in compiling the original liquefaction case history databases. Thus rod-length corrections are implicitly incorporated into the empirical SPT procedure.

A final change recommended by workshop participants is the use of revised magnitude scaling factors rather than the original Seed and Idriss (1982) factors to adjust CRR for earthquake magnitudes other than 7.5. Magnitude scaling factors are addressed later in this report.

CPT

A primary advantage of the CPT is that a nearly continuous profile of penetration resistance is developed for stratigraphic interpretation. The CPT results are generally more consistent and repeatable than results from other penetration tests listed in Table 1. The continuous profile also allows a more detailed definition of soil layers than the other tools listed in the table. This stratigraphic capability makes the CPT particularly advantageous for developing liquefaction-resistance profiles. Interpretations based on the CPT, however, must be verified with a few well-placed boreholes preferably with standard penetration tests, to confirm soil types and further verify liquefaction-resistance interpretations.

Fig. 4 provides curves prepared by Robertson and Wride (1998) for direct determination of CRR for clean sands (FC ≤ 5%) from CPT data. This figure was developed from CPT case history data compiled from several investigations, including those by Stark and Olson (1995) and Suzuki et al. (1995). The chart, valid for magnitude 7.5 earthquakes only, shows calculated cyclic resistance ratio plotted as a function of dimensionless, corrected, and normalized CPT resistance q_{c1N} from sites where surface effects of liquefaction were or were not observed following past earthquakes. The CRR curve conservatively separates regions of the plot with data indicative of liquefaction from regions indicative of nonliquefaction.

Based on a few misclassified case histories from the 1989 Loma Prieta earthquake, I. M. Idriss suggested that the clean sand curve in Fig. 4 should be shifted to the right by 10–15%. However, a majority of workshop participants supported a curve in its present position, for three reasons. First, a purpose of the workshop was to recommend criteria that yield roughly equivalent CRR for the field tests listed in Table 1. Shifting the base curve to the right makes the CPT criteria generally more conservative. For example, for $(N_1)_{60} > 5$, q_{c1N}:$(N_1)_{60}$ ra-

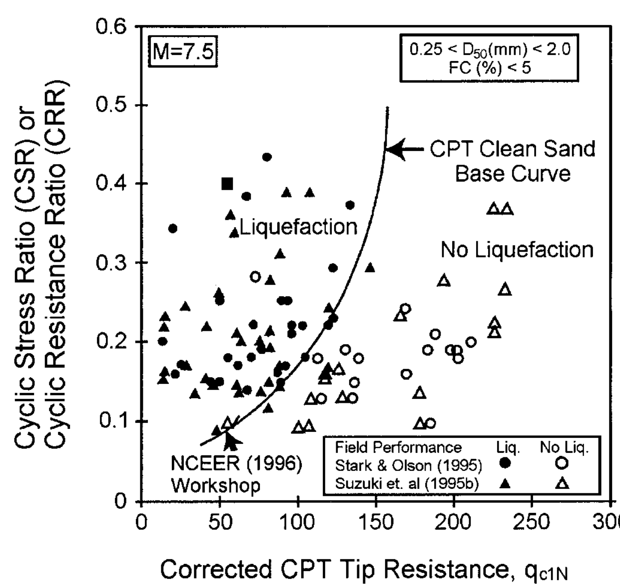

FIG. 4. Curve Recommended for Calculation of CRR from CPT Data along with Empirical Liquefaction Data from Compiled Case Histories (Reproduced from Robertson and Wride 1998)

from BPT measurements. These plots indicate that although SPT blow counts can be roughly estimated from BPT measurements, there can be considerable uncertainty for calculating liquefaction resistance because the data scatter is greatest in the range of greatest importance [N-values of 0–30 blows/300 mm (ft)].

A major source of variation in BPT blow counts is deviations in hammer energy. Rather than measuring hammer energy directly, Harder and Seed (1986) monitored bounce-chamber pressures and found that uniform combustion conditions (e.g., full throttle with a supercharger) correlated rather well with variations in Becker blow count. From this information, Harder and Seed developed an energy correction procedure based on measured bounce-chamber pressure.

Direct measurement of transmitted hammer energy could provide a more theoretically rigorous correction for Becker hammer efficiency. Sy and Campanella (1994) and Sy et al. (1995) instrumented a small length of Becker casing with strain gauges and accelerometers to measure transferred energy. They analyzed the recorded data with a pile-driving analyzer to determine strain, force, acceleration, and velocity. The transferred energy was determined by time integration of force times velocity. They were able to verify many of the variations in hammer energy previously identified by Harder and Seed (1986), including effects of variable throttle settings and energy transmission efficiencies of various drill rigs. However, they were unable to reduce the amount of scatter and uncertainty in converting BPT blow counts to SPT blow counts. Because the Sy and Campanella procedure requires considerably more effort than monitoring of bounce-chamber pressure without producing greatly improved results, the workshop participants agreed that the bounce-chamber technique is adequate for routine practice.

Friction along the driven casing also influences penetration resistance. Harder and Seed (1986) did not directly evaluate the effect of casing friction; hence, the correlation in Fig. 10(b) intrinsically incorporates an unknown amount of casing friction. However, casing friction remains a concern for depths >30 m and for measurement of penetration resistance in soft soils underlying thick deposits of dense soil. Either of these circumstances could lead to greater casing friction than is intrinsically incorporated in the Harder and Seed correlation.

The following procedures are recommended for routine practice: (1) the BPT should be conducted with newer AP-1000 drill rigs equipped with supercharged diesel hammers to drive plugged 168-mm outside diameter casing; (2) bounce-chamber pressures should be monitored and adjustments made to measured BPT blow counts to account for variations in diesel hammer combustion efficiency—for most routine applications, correlations developed by Harder and Seed (1986) may be used for these adjustments; and (3) the influence of some casing friction is indirectly accounted for in the Harder and Seed BPT-SPT correlation. This correlation, however, has not been verified and should not be used for depths >30 m or for sites with thick dense deposits overlying loose sands or gravels. For these conditions, mudded boreholes may be needed to reduce casing friction, or specially developed local correlations or sophisticated wave-equation analyses may be applied to quantify frictional effects.

MAGNITUDE SCALING FACTORS (MSFs)

The clean-sand base or CRR curves in Figs. 2 (SPT), 4 (CPT), and 10 (V_{s1}) apply only to magnitude 7.5 earthquakes. To adjust the clean-sand curves to magnitudes smaller or larger than 7.5, Seed and Idriss (1982) introduced correction factors termed ''magnitude scaling factors (MSFs).'' These factors are used to scale the CRR base curves upward or downward on CRR versus $(N_1)_{60}$, q_{c1N}, or V_{s1} plots. Conversely, magnitude weighting factors, which are the inverse of magnitude scaling factors, may be applied to correct CSR for magnitude. Either correcting CRR via magnitude scaling factors, or correcting CSR via magnitude weighting factors, leads to the same final result. Because the original papers by Seed and Idriss were written in terms of magnitude scaling factors, the use of magnitude scaling factors is continued in this report.

To illustrate the influence of magnitude scaling factors on calculated hazard, the equation for factor of safety (FS) against liquefaction is written in terms of CRR, CSR, and MSF as follows:

$$FS = (CRR_{7.5}/CSR)MSF \quad (23)$$

where CSR = calculated cyclic stress ratio generated by the earthquake shaking; and $CRR_{7.5}$ = cyclic resistance ratio for magnitude 7.5 earthquakes. $CRR_{7.5}$ is determined from Fig. 2 or (4) for SPT data, Fig. 4 or (11) for CPT data, or Fig. 9 or (22) for V_{s1} data.

Seed and Idriss (1982) Scaling Factors

Because of the limited amount of field liquefaction data available in the 1970s, Seed and Idriss (1982) were unable to adequately constrain bounds between liquefaction and nonliquefaction regions on CRR plots for magnitudes other than 7.5. Consequently, they developed a set of MSF from average numbers of loading cycles for various earthquake magnitudes and laboratory test results. A representative curve developed by these investigators, showing the number of loading cycles required to generate liquefaction for a given CSR, is reproduced in Fig. 11. The average number of loading cycles for various magnitudes of earthquakes are also noted on the plot. The initial set of magnitude scaling factors was derived by dividing CSR values on the representative curve for the number of loading cycles corresponding to a given earthquake magnitude by the CSR for 15 loading cycles (equivalent to a magnitude 7.5 earthquake). These scaling factors are listed in column 2 of Table 3 and are plotted in Fig. 12. These MSFs have been routinely applied in engineering practice since their introduction in 1982.

Revised Idriss Scaling Factors

In preparing his H. B. Seed Memorial Lecture, I. M. Idriss reevaluated the data that he and the late Professor Seed used to calculate the original (1982) magnitude scaling factors. In so doing, Idriss replotted the data on a log-log plot and suggested that the data should plot as a straight line. He noted, however, that one outlying point had strongly influenced the original analysis, causing the original plot to be nonlinear and characterized by unduly low MSF values for magnitudes <7.5. Based on this reevaluation, Idriss defined a revised set of magnitude scaling factors listed in column 3 of Table 3 and plotted

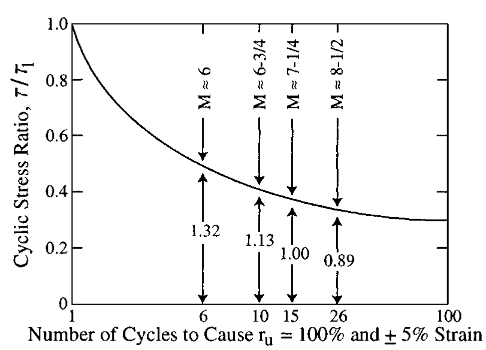

FIG. 11. Representative Relationship between CSR and Number of Cycles to Cause Liquefaction (Reproduced from Seed and Idriss 1982)

TABLE 3. Magnitude Scaling Factor Values Defined by Various Investigators (Youd and Noble 1997a)

Magnitude, M	Seed and Idriss (1982)	Idriss[a]	Ambraseys (1988)	Arango (1996) Distance based	Arango (1996) Energy based	Andrus and Stokoe (1997)	Youd and Noble (1997b) $P_L < 20\%$	Youd and Noble (1997b) $P_L < 32\%$	Youd and Noble (1997b) $P_L < 50\%$
5.5	1.43	2.20	2.86	3.00	2.20	2.8	2.86	3.42	4.44
6.0	1.32	1.76	2.20	2.00	1.65	2.1	1.93	2.35	2.92
6.5	1.19	1.44	1.69	1.60	1.40	1.6	1.34	1.66	1.99
7.0	1.08	1.19	1.30	1.25	1.10	1.25	1.00	1.20	1.39
7.5	1.00	1.00	1.00	1.00	1.00	1.00	—	—	1.00
8.0	0.94	0.84	0.67	0.75	0.85	0.8?	—	—	0.73?
8.5	0.89	0.72	0.44	—	—	0.65?	—	—	0.56?

Note: ? = Very uncertain values.
[a]1995 Seed Memorial Lecture, University of California at Berkeley (I. M. Idriss, personal communication to T. L. Youd, 1997).

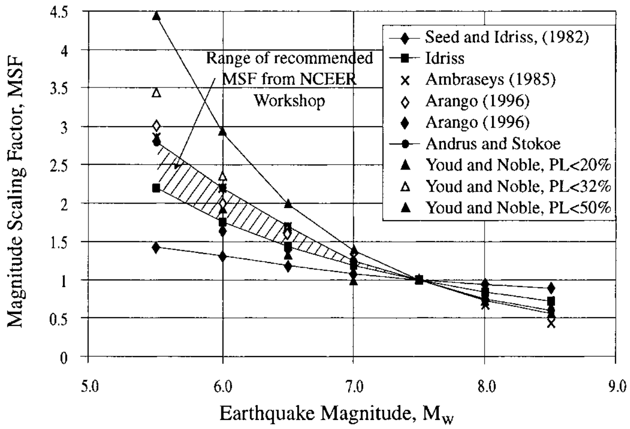

FIG. 12. Magnitude Scaling Factors Derived by Various Investigators (Reproduced from Youd and Noble 1997a)

in Fig. 12. The revised MSFs are defined by the following equation:

$$\mathrm{MSF} = 10^{2.24}/M_w^{2.56} \tag{24}$$

The workshop participants recommend these revised scaling factors as a lower bound for MSF values.

The revised scaling factors are significantly higher than the original scaling factors for magnitudes <7.5 and somewhat lower than the original factors for magnitudes >7.5. Relative to the original scaling factors, the revised factors lead to a reduced calculated liquefaction hazard for magnitudes <7.5, but increase calculated hazard for magnitudes >7.5.

Ambraseys (1988) Scaling Factors

Field performance data collected since the 1970s for magnitudes <7.5 indicate that the original Seed and Idriss (1982) scaling factors are overly conservative. For example, Ambraseys (1988) analyzed liquefaction data compiled through the mid-1980s and plotted calculated cyclic stress ratios for sites that did or did not liquefy versus $(N_1)_{60}$. From these plots, Ambraseys developed empirical exponential equations that define CRR as a function of $(N_1)_{60}$ and moment magnitude M_w. By holding the value of $(N_1)_{60}$ constant in the equations and taking the ratio of CRR determined for various magnitudes of earthquakes to the CRR for magnitude 7.5 earthquakes, Ambraseys derived the magnitude scaling factors listed in column 4 of Table 3 and plotted in Fig. 12. For magnitudes <7.5, the MSFs suggested by Ambraseys are significantly larger than both the original factors developed by Seed and Idriss (column 2, Table 3) and the revised factors suggested by Idriss (column 3). Because they are based on observational data, these factors have validity for estimating liquefaction hazard; however, they have not been widely used in engineering practice.

For magnitudes >7.5, Ambraseys factors are significantly lower and much more conservative than the original (Seed and Idriss 1982) and Idriss's revised scaling factors. Because there are few data to constrain Ambraseys' scaling factors for magnitudes >7.5, they are not recommended for hazard evaluation for large earthquakes.

Arango (1996) Scaling Factors

Arango (1996) developed two sets of magnitude scaling factors. The first set (column 5, Table 3) is based on furthest observed liquefaction effects from the seismic energy source, the estimated average peak accelerations at those distant sites, and the seismic energy required to cause liquefaction. The second set (column 6, Table 3) was developed from energy concepts and the relationship derived by Seed and Idriss (1982) between numbers of significant stress cycles and earthquake magnitude. The MSFs listed in column 5 are similar in value (within about 10%) to the MSFs of Ambraseys (column 4), and the MSFs listed in column 6 are similar in value (within about 10%) to the revised MSFs proposed by Idriss (column 3).

Andrus and Stokoe (1997) Scaling Factors

From their studies of liquefaction resistance as a function of shear wave velocity V_s Andrus and Stokoe (1997) drew bounding curves and developed (22) for calculating CRR from V_s for magnitude 7.5 earthquakes. These investigators drew similar bounding curves for sites where surface effects of liquefaction were or were not observed for earthquakes with magnitudes of 6, 6.5, and 7. The positions of the CRR curves were visually adjusted on each graph until a best-fit bound was obtained. Magnitude scaling factors were then estimated by taking the ratio of CRR for a given magnitude to the CRR for magnitude 7.5 earthquakes. These MSFs are quantified by the following equation:

$$\mathrm{MSF} = (M_w/7.5)^{-2.56} \tag{25}$$

MSFs for magnitudes <6 and >7.5 were extrapolated from this equation. The derived MSFs are listed in column 7 of Table 3, and plotted in Fig. 12. For magnitudes <7.5, the MSFs proposed by Andrus and Stokoe are rather close in value (within about 5%) to the MSFs proposed by Ambraseys. For magnitudes >7.5, the Andrus and Stokoe MSFs are slightly smaller than the revised MSFs proposed by Idriss.

Youd and Noble (1997a) Scaling Factors

Youd and Noble (1997a) used a probabilistic or logistic analysis to analyze case history data from sites where effects of liquefaction were or were not reported following past earthquakes. This analysis yielded the following equation, which was updated after publication of the NCEER proceedings (Youd and Idriss 1997):

$$\text{Logit}(P_L) = \ln(P_L/(1 - P_L)) = -7.0351 + 2.1738M_w - 0.2678(N_1)_{60cs} + 3.0265 \ln \text{CRR} \quad (26)$$

where P_L = probability that liquefaction occurred; $1 - P_L$ = probability that liquefaction did not occur; and $(N_1)_{60cs}$ = corrected equivalent clean-sand blow count. For magnitudes <7.5, Youd and Noble recommended direct application of this equation to calculate the CRR for a given probability of liquefaction. In lieu of direct application, Youd and Noble defined three sets of MSFs for use with the simplified procedure. These MSFs are for probabilities of liquefaction occurrence <20, 32, and 50%, respectively, and are defined by the following equations:

$$\text{Probability } P_L < 20\% \quad \text{MSF} = 10^{3.81}/M^{4.53} \text{ for } M_w < 7 \quad (27)$$

$$\text{Probability } P_L < 32\% \quad \text{MSF} = 10^{3.74}/M^{4.33} \text{ for } M_w < 7 \quad (28)$$

$$\text{Probability } P_L < 50\% \quad \text{MSF} = 10^{4.21}/M^{4.81} \text{ for } M_w < 7.75 \quad (29)$$

New Recommendation by Idriss

I. M. Idriss (TRB 1999) proposed a new set of MSFs that are compatible with, and are only to be used with, the magnitude-dependent r_d that he also proposed. These new MSFs have lower values than the revised MSFs listed in Table 3, but slightly higher values than the original Seed and Idriss (1982) MSFs. Because the proposed r_d and associated MSFs have not been published and the factors have not been independently verified, the workshop participants chose not to recommend the new r_d or MSFs at this time.

Recommendations for Engineering Practice

The workshop participants reviewed the MSFs listed in Table 3, and all but one (S. S. C. Liao) agree that the original factors were too conservative and that increased MSFs are warranted for engineering practice for magnitudes <7.5. Rather than recommending a single set of factors, the workshop participants suggest a range of MSFs from which the engineer is allowed to choose factors that are requisite with the acceptable risk for any given application. For magnitudes <7.5, the lower bound for the recommended range is the new MSF proposed by Idriss [column 3 in Table 3, or (23)]. The suggested upper bound is the MSF proposed by Andrus and Stokoe [column 7 in Table 3, or (26)]. The upper-bound values are consistent with MSFs suggested by Ambraseys (1988), Arango (1996), and Youd and Noble (1997a) for P_L < 20%.

For magnitudes >7.5, the new factors recommended by Idriss [column 3 in Table 3; (25)] should be used for engineering practice. These new factors are smaller than the original Seed and Idriss (1982) factors, hence their application leads to increased calculated liquefaction hazard compared to the original factors. Because there are only a few well-documented liquefaction case histories for earthquakes with magnitudes >8, MSFs in that range are poorly constrained by field data. Thus the workshop participants agreed that the greater conservatism embodied in the revised MSF by Idriss (column 3, Table 3) should be recommended for engineering practice.

CORRECTIONS FOR HIGH OVERBURDEN STRESSES, STATIC SHEAR STRESSES, AND AGE OF DEPOSIT

Correction factors K_σ and K_α were developed by Seed (1983) to extrapolate the simplified procedure to larger overburden pressure and static shear stress conditions than those embodied in the case history data set from which the simplified procedure was derived. As noted previously, the simplified procedure was developed and validated only for level to gently sloping sites (low static shear stress) and depths less than about 15 m (low overburden pressures). Thus applications using K_σ and K_α are beyond routine practice and require specialized expertise. Because these factors were discussed at the workshop and some new information was developed, recommendations from those discussions are included here. These recommendations, however, apply mostly to liquefaction hazard analyses of embankment dams and other large structures. These factors are applied by extending (23) to include K_σ and K_α as follows:

$$\text{FS} = (\text{CRR}_{7.5}/\text{CSR}) \cdot \text{MSF} \cdot K_\sigma \cdot K_\alpha \quad (30)$$

K_σ Correction Factor

Cyclically loaded laboratory test data indicate that liquefaction resistance increases with increasing confining stress. The rate of increase, however, is nonlinear. To account for the nonlinearity between CRR and effective overburden pressure, Seed (1983) introduced the correction factor K_σ to extrapolate the simplified procedure to soil layers with overburden pressures >100 kPa. Cyclically loaded, isotropically consolidated triaxial compression tests on sand specimens were used to measure CRR for high-stress conditions and develop K_σ values. By taking the ratio of CRR for various confining pressures to the CRR determined for approximately 100 kPa (1 atm) Seed (1983) developed the original K_σ correction curve. Other investigators have added data and suggested modifications to better define K_σ for engineering practice. For example, Seed and Harder (1990) developed the clean-sand curve reproduced in Fig. 13. Hynes and Olsen (1999) compiled and analyzed an enlarged data set to provide guidance and formulate equations for selecting K_σ values (Fig. 14). The equation they derived for calculating K_σ is

$$K_\sigma = (\sigma'_{vo}/P_a)^{(f-1)} \quad (31)$$

where σ'_{vo}, effective overburden pressure; and P_a, atmospheric pressure, are measured in the same units; and f is an exponent that is a function of site conditions, including relative density, stress history, aging, and overconsolidation ratio. The workshop participants considered the work of previous investigators and recommend the following values for f (Fig. 15). For relative densities between 40 and 60%, f = 0.7–0.8; for relative densities between 60 and 80%, f = 0.6–0.7. Hynes and Olsen recommended these values as minimal or conservative esti-

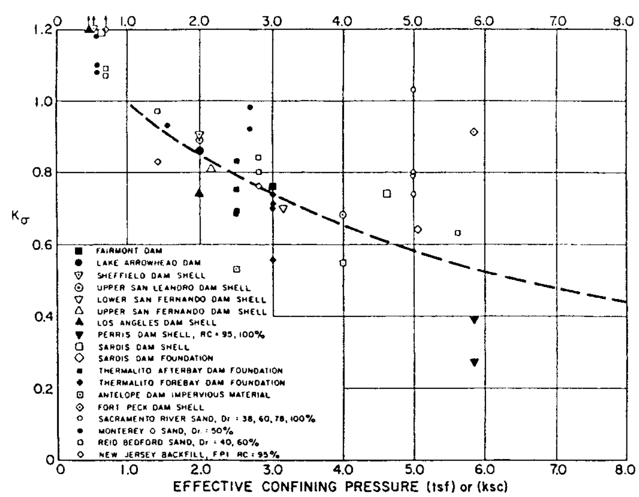

FIG. 13. K_σ-Values Determined by Various Investigators (Reproduced from Seed and Harder 1990)

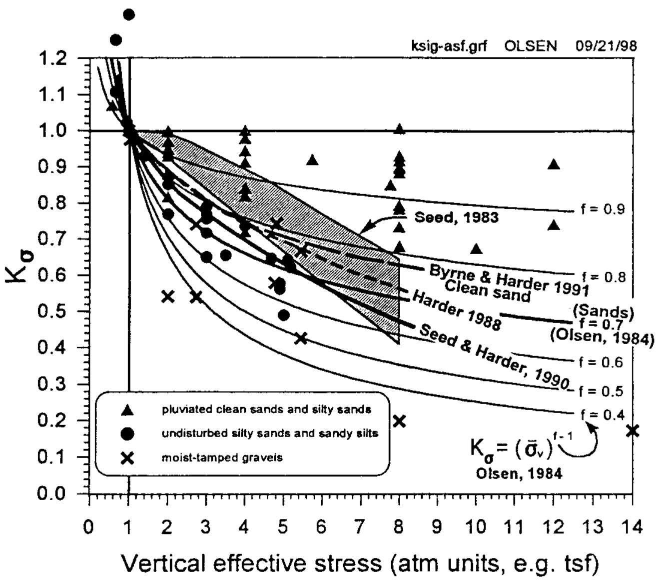

FIG. 14. Laboratory Data and Compiled K_σ Curves (Reproduced from Hynes and Olsen 1999)

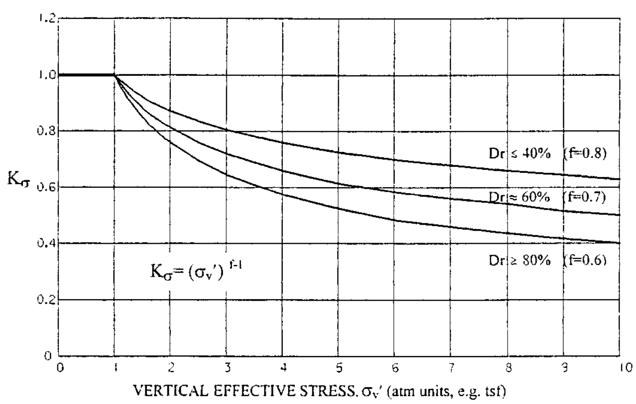

FIG. 15. Recommended Curves for Estimating K_σ for Engineering Practice

mates of K_σ for use in engineering practice for both clean and silty sands, and for gravels. The workshop participants concurred with this recommendation.

K_α Correction Factor for Sloping Ground

The liquefaction resistance of dilative soils (moderately dense to dense granular materials under low confining stress) increases with increased static shear stress. Conversely, the liquefaction resistance of contractive soils (loose soils and moderately dense soils under high confining stress) decreases with increased static shear stresses. To incorporate the effect of static shear stresses on liquefaction resistance, Seed (1983) introduced a correction factor K_α. To generate values for this factor, Seed normalized the static shear stress τ_{st} acting on a plane with respect to the effective vertical stress σ'_{vo} yielding a parameter α, where

$$\alpha = \tau_{st}/\sigma'_{vo} \tag{32}$$

Cyclically loaded triaxial compression tests were then used to empirically determine values of the correction factor K_α as a function of α.

For the NCEER workshop, Harder and Boulanger (1997) reviewed past publications, test results, and analyses of K_α. They noted that a wide range of K_α values have been proposed, indicating a lack of convergence and a need for continued research. The workshop participants agreed with this assessment. Although curves relating K_α to α have been published (Harder and Boulanger 1997), these curves should not be used by nonspecialists in geotechnical earthquake engineering or in routine engineering practice.

Influence of Age of Deposit

Several investigators have noted that liquefaction resistance of soils increases with age. For example, Seed (1979) observed significant increases in liquefaction resistance with aging of reconstituted sand specimens tested in the laboratory. Increases of as much as 25% in cyclic resistance ratio were noted between freshly constituted and 100-day-old specimens. Youd and Hoose (1977) and Youd and Perkins (1978) noted that liquefaction resistance increases markedly with geologic age. Sediments deposited within the past few thousand years are generally much more susceptible to liquefaction than older Holocene sediments; Pleistocene sediments are even more resistant; and pre-Pleistocene sediments are generally immune to liquefaction. Although qualitative time-dependent increases have been documented as noted above, few quantitative data have been collected. In addition, the factors causing increased liquefaction resistance with age are poorly understood. Consequently, verified correction factors for age have not been developed.

In the absence of quantitative correction factors, engineering judgment is required to estimate the liquefaction resistance of sediments more than a few thousand years old. For deeply buried sediments dated as more than a few thousand years old, some knowledgeable engineers have omitted application of the K_σ factor as partial compensation for the unquantified, but substantial increase of liquefaction resistance with age. For man-made structures, such as thick fills and embankment dams, aging effects are minimal, and corrections for age should not be applied in calculating liquefaction resistance.

SEISMIC FACTORS

Application of the simplified procedure for evaluating liquefaction resistance requires estimates of two ground motion parameters—earthquake magnitude and peak horizontal ground acceleration. These factors characterize duration and intensity of ground shaking, respectively. The workshop addressed the following questions with respect to selection of magnitude and peak acceleration values for liquefaction resistance analyses.

Earthquake Magnitude

Records from recent earthquakes, such as 1979 Imperial Valley, 1988 Armenia, 1989 Loma Prieta, 1994 Northridge, and 1995 Kobe, indicate that the relationship between duration and magnitude is rather uncertain and that factors other than magnitude also influence duration. For example, unilateral faulting, in which rupture begins at one end of the fault and propagates to the other, usually produces longer shaking duration for a given magnitude than bilateral faulting, in which slip begins near the midpoint on the fault and propagates in both directions simultaneously. Duration also generally increases with distance from the seismic energy source and may vary with tectonic province, site conditions, and bedrock topography (basin effects).

Question: Should correction factors be developed to adjust duration of shaking to account for the influence of earthquake source mechanism, fault rupture mode, distance from the energy source, basin effects, etc.?

Answer: Faulting characteristics and variations in shaking duration are difficult to predict in advance of an earthquake

APPENDIX D

LIQUEFACTION ANALYSIS SPREADSHEETS

Boring N-1 BE

Boring Data

GSE = 245.2 ft

PSE = 229.0 ft

C_R = 1.00 [(1)]

C_S = 1.00 [(2)]

C_B = 1.00 [(3)]

C_E = 1.00 [(4)]

Soil Data

γ_{total} = 115 pcf

γ_{sat} = 120 pcf

Layer 1 Fines content = 28% [(5)]

Layer 2 Fines content = 30% [(5)]

Layer 3 Fines content = 30% [(5)]

Seismic Data

a_{max} = 0.140 g

M_w = 6.32 [(7)]

MSF = 1.36 [(8)]

K_a = 1.00 [(9)]

Conversion Factors

1 atm = 2,116.2 psf

1 m = 3.28 ft

1 m^2 = 10.76 ft^2

Notes

(1) Rod length unknown

(2) Not known if a liner was used, or if there was space for a liner in the sampler

(3) For HW size casing with 117 mm OD

(4) Hammer type unknown

(5) Based on boring log descriptions

(6) Maximum value of 2.0 to prevent unnecessarily high Factors of safety

(7) Largest recorded earthquake from STID

(8) Revised value from Idriss (1999)

Depth (ft)	Depth (m)	El. (ft)	El. (m)	σ_v (psf)	σ_v' (psf)	σ_v' (Kpa)	$N_{6"}$ (bpf/2)	$N_{6"}$ (bpf/2)	N (bpf)	C_N	N_1 (bpf)	$N_{1,60}$ (bpf)	$\Delta N_{1,60}$	$N_{1,60cs}$ (bpf)	$CRR_{7.5}$ [(6)]	r_d	CSR	K_σ	FS
17	5.18	228.2	69.57	1,959	1,909	91	3	3	6	1.0466	6.28	6.28	5.27	11.55	0.1291	0.9237	0.0863	1.0098	2.0
19	5.79	226.2	68.96	2,199	2,024	97	7	6	13	1.0201	13.26	13.26	5.27	18.53	0.1892	0.9114	0.0901	1.0052	2.0
21	6.40	224.2	68.35	2,439	2,139	102	7	5	12	0.9950	11.94	11.94	5.27	17.21	0.1759	0.8987	0.0932	0.9983	2.0
23	7.01	222.2	67.74	2,679	2,255	108	6	4	10	0.9711	9.71	9.71	5.27	14.98	0.1560	0.8857	0.0958	0.9926	2.0
25	7.62	220.2	67.13	2,919	2,370	113	4	7	11	0.9483	10.43	10.43	5.36	15.79	0.1629	0.8725	0.0978	0.9867	2.0
27	8.23	218.2	66.52	3,159	2,485	119	4	8	12	0.9266	11.12	11.12	5.36	16.48	0.1691	0.8590	0.0994	0.9808	2.0
29	8.84	216.2	65.91	3,399	2,600	125	5	5	10	0.9058	9.06	9.06	5.36	14.42	0.1513	0.8454	0.1006	0.9773	2.0
31	9.45	214.2	65.30	3,639	2,715	130	2	9	11	0.8860	9.75	9.75	5.36	15.11	0.1570	0.8316	0.1014	0.9719	2.0
33	10.06	212.2	64.70	3,879	2,831	136	24	32	56	0.8670	48.55	48.55	5.36	53.91	2.0000	0.8178	0.1020	0.9132	2.0
35	10.67	210.2	64.09	4,119	2,946	141	7	42	49	0.8487	41.59	41.59	5.36	46.95	2.0000	0.8039	0.1023	0.9014	2.0
37	11.28	208.2	63.48	4,359	3,061	147	18	30	48	0.8313	39.90	39.90	5.36	45.26	2.0000	0.7900	0.1024	0.8901	2.0
39	11.89	206.2	62.87	4,599	3,176	152	17	19	36	0.8145	29.32	29.32	5.36	34.69	1.0393	0.7762	0.1023	0.8946	2.0
41	12.50	204.2	62.26	4,839	3,291	158	22	26	48	0.7984	38.33	38.33	5.36	43.69	2.0000	0.7624	0.1020	0.8687	2.0
43	13.11	202.2	61.65	5,079	3,407	163	25	25	50	0.7830	39.15	39.15	5.36	44.51	2.0000	0.7487	0.1016	0.8586	2.0
45	13.72	200.2	61.04	5,319	3,522	169	30	9	39	0.7681	29.96	29.96	5.36	35.32	1.1853	0.7352	0.1010	0.8631	2.0
47	14.33	198.2	60.43	5,559	3,637	174	3	11	14	0.7538	10.55	10.55	5.36	15.92	0.1640	0.7218	0.1004	0.9376	2.0
49	14.94	196.2	59.82	5,799	3,752	180	3	7	10	0.7400	7.40	7.40	5.36	12.76	0.1382	0.7086	0.0997	0.9412	1.8

Min 1.78

G:\vpillai\14-5232 Crisp County\Liquefaction Potential 1

Boring W-1 BE

Boring Data

GSE = 243.9 ft

PSE = 228.9 ft

C_R = 1.00 [(1)]

C_S = 1.00 [(2)]

C_B = 1.00 [(3)]

C_E = 1.00 [(4)]

Soil Data

γ_{total} = 115 pcf

γ_{sat} = 120 pcf

Layer 1 Fines content = 29% [(5)]

Layer 2 Fines content = 30% [(5)]

Layer 3 Fines content = 30% [(5)]

Seismic Data

a_{max} = 0.140 g

M_w = 6.32 [(7)]

MSF = 1.36 [(8)]

K_a = 1.00 [(9)]

Conversion Factors

1 atm = 2,116.2 psf

1 m = 3.28 ft

1 m^2 = 10.76 ft^2

Notes

(1) Rod length unknown

(2) Not known if a liner was used, or if there was space for a liner in the sampler

(3) For HW size casing with 117 mm OD

(4) Hammer type unknown

(5) Based on boring log descriptions

(6) Maximum value of 2.0 to prevent unnecessarily high Factors of safety

(7) Largest recorded earthquake from STID

(8) Revised value from Idriss (1999)

Depth	Depth	El.	El.	σ_v	σ_v'	σ_v'	$N_{6"}$	$N_{6"}$	N	C_N	N_1	$N_{1,60}$	$\Delta N_{1,60}$	$N_{1,60cs}$	$CRR_{7.5}$ [(6)]	r_d	CSR	K_σ	FS
(ft)	(m)	(ft)	(m)	(psf)	(psf)	(Kpa)	(bpf/2)	(bpf/2)	(bpf)		(bpf)	(bpf)		(bpf)					
17	5.18	226.9	69.18	1,965	1,840	88	9	10	19	1.0630	20.20	20.20	5.32	25.52	0.3028	0.9237	0.0898	1.0227	2.0
19	5.79	224.9	68.57	2,205	1,955	94	6	7	13	1.0358	13.47	13.47	5.32	18.79	0.1919	0.9114	0.0935	1.0097	2.0
21	6.40	222.9	67.96	2,445	2,071	99	4	4	8	1.0099	8.08	8.08	5.32	13.40	0.1431	0.8987	0.0966	1.0019	2.0
23	7.01	220.9	67.35	2,685	2,186	105	1	3	4	0.9853	3.94	3.94	5.32	9.26	0.1130	0.8857	0.0990	0.9968	1.5
25	7.62	218.9	66.74	2,925	2,301	110	6	9	15	0.9618	14.43	14.43	5.36	19.79	0.2033	0.8725	0.1009	0.9885	2.0
27	8.23	216.9	66.13	3,165	2,416	116	6	7	13	0.9395	12.21	12.21	5.36	17.58	0.1795	0.8590	0.1024	0.9835	2.0
29	8.84	214.9	65.52	3,405	2,531	121	10	13	23	0.9181	21.12	21.12	5.36	26.48	0.3298	0.8454	0.1035	0.9684	2.0
31	9.45	212.9	64.91	3,645	2,647	127	25	24	49	0.8977	43.99	43.99	5.36	49.35	2.0000	0.8316	0.1042	0.9331	2.0
33	10.06	210.9	64.30	3,885	2,762	132	15	20	35	0.8782	30.74	30.74	5.36	36.10	1.4101	0.8178	0.1047	0.9247	2.0
35	10.67	208.9	63.69	4,125	2,877	138	25	25	50	0.8595	42.98	42.98	5.36	48.34	2.0000	0.8039	0.1049	0.9084	2.0
37	11.28	206.9	63.08	4,365	2,992	143	13	19	32	0.8416	26.93	26.93	5.36	32.30	0.6751	0.7900	0.1049	0.9207	2.0
39	11.89	204.9	62.47	4,605	3,107	149	25	25	50	0.8245	41.22	41.22	5.36	46.59	2.0000	0.7762	0.1047	0.8857	2.0
41	12.50	202.9	61.86	4,845	3,223	154	16	21	37	0.8080	29.90	29.90	5.36	35.26	1.1702	0.7624	0.1043	0.8872	2.0
43	13.11	200.9	61.25	5,085	3,338	160	6	4	10	0.7921	7.92	7.92	5.36	13.28	0.1422	0.7487	0.1038	0.9522	1.8
45	13.72	198.9	60.64	5,325	3,453	165	2	9	11	0.7769	8.55	8.55	5.36	13.91	0.1472	0.7352	0.1032	0.9475	1.8
47	14.33	196.9	60.03	5,565	3,568	171	3	13	16	0.7623	12.20	12.20	5.36	17.56	0.1793	0.7218	0.1024	0.9360	2.0
49	14.94	194.9	59.42	5,805	3,683	176	25	25	50	0.7482	37.41	37.41	5.36	42.77	2.0000	0.7086	0.1016	0.8355	2.0

Min 1.55

Boring N-1 sens

Boring Data

GSE = 245.2 ft

PSE = 229.0 ft

C_R = 1.00 (1)

C_S = 1.00 (2)

C_B = 1.00 (3)

C_E = 1.00 (4)

Soil Data

γ_{total} = 115 pcf

γ_{sat} = 120 pcf

Layer 1 Fines content = 15% (5)

Layer 2 Fines content = 30% (5)

Layer 3 Fines content = 30% (5)

Seismic Data

a_{max} = 0.140 g

M_w = 6.32 (7)

MSF = 1.36 (8)

K_a = 1.00 (9)

Conversion Factors

1 atm = 2,116.2 psf

1 m = 3.28 ft

1 m^2 = 10.76 ft^2

Notes

(1) Rod length unknown

(2) Not known if a liner was used, or if there was space for a liner in the sampler

(3) For HW size casing with 117 mm OD

(4) Hammer type unknown

(5) Based on boring log descriptions

(6) Maximum value of 2.0 to prevent unnecessarily high Factors of safety

(7) Largest recorded earthquake from STID

(8) Revised value from Idriss (1999)

Depth (ft)	Depth (m)	El. (ft)	El. (m)	σ_v (psf)	σ_v' (psf)	σ_v' (Kpa)	$N_{6"}$ (bpf/2)	$N_{6"}$ (bpf/2)	N (bpf)	C_N	N_1 (bpf)	$N_{1,60}$ (bpf)	$\Delta N_{1,60}$	$N_{1,60cs}$ (bpf)	$CRR_{7.5}$ (6)	r_d	CSR	K_σ	FS
17	5.18	228.2	69.57	1,959	1,909	91	3	3	6	1.0466	6.28	6.28	3.26	9.54	0.1149	0.9237	0.0863	1.0091	1.8
19	5.79	226.2	68.96	2,199	2,024	97	7	6	13	1.0201	13.26	13.26	3.26	16.52	0.1695	0.9114	0.0901	1.0048	2.0
21	6.40	224.2	68.35	2,439	2,139	102	7	5	12	0.9950	11.94	11.94	3.26	15.20	0.1578	0.8987	0.0932	0.9984	2.0
23	7.01	222.2	67.74	2,679	2,255	108	6	4	10	0.9711	9.71	9.71	3.26	12.97	0.1398	0.8857	0.0958	0.9931	2.0
25	7.62	220.2	67.13	2,919	2,370	113	4	7	11	0.9483	10.43	10.43	5.36	15.79	0.1629	0.8725	0.0978	0.9867	2.0
27	8.23	218.2	66.52	3,159	2,485	119	4	8	12	0.9266	11.12	11.12	5.36	16.48	0.1691	0.8590	0.0994	0.9808	2.0
29	8.84	216.2	65.91	3,399	2,600	125	5	5	10	0.9058	9.06	9.06	5.36	14.42	0.1513	0.8454	0.1006	0.9773	2.0
31	9.45	214.2	65.30	3,639	2,715	130	2	9	11	0.8860	9.75	9.75	5.36	15.11	0.1570	0.8316	0.1014	0.9719	2.0
33	10.06	212.2	64.70	3,879	2,831	136	24	32	56	0.8670	48.55	48.55	5.36	53.91	2.0000	0.8178	0.1020	0.9132	2.0
35	10.67	210.2	64.09	4,119	2,946	141	7	42	49	0.8487	41.59	41.59	5.36	46.95	2.0000	0.8039	0.1023	0.9014	2.0
37	11.28	208.2	63.48	4,359	3,061	147	18	30	48	0.8313	39.90	39.90	5.36	45.26	2.0000	0.7900	0.1024	0.8901	2.0
39	11.89	206.2	62.87	4,599	3,176	152	17	19	36	0.8145	29.32	29.32	5.36	34.69	1.0393	0.7762	0.1023	0.8946	2.0
41	12.50	204.2	62.26	4,839	3,291	158	22	26	48	0.7984	38.33	38.33	5.36	43.69	2.0000	0.7624	0.1020	0.8687	2.0
43	13.11	202.2	61.65	5,079	3,407	163	25	25	50	0.7830	39.15	39.15	5.36	44.51	2.0000	0.7487	0.1016	0.8586	2.0
45	13.72	200.2	61.04	5,319	3,522	169	30	9	39	0.7681	29.96	29.96	5.36	35.32	1.1853	0.7352	0.1010	0.8631	2.0
47	14.33	198.2	60.43	5,559	3,637	174	3	11	14	0.7538	10.55	10.55	5.36	15.92	0.1640	0.7218	0.1004	0.9376	2.0
49	14.94	196.2	59.82	5,799	3,752	180	3	7	10	0.7400	7.40	7.40	5.36	12.76	0.1382	0.7086	0.0997	0.9412	1.8

Min 1.78

G:\vpillai\14-5232 Crisp County\Liquefaction Potential 1

Boring W-1 sens

Boring Data

GSE = 243.9 ft

PSE = 228.9 ft

C_R = 1.00 [(1)]

C_S = 1.00 [(2)]

C_B = 1.00 [(3)]

C_E = 1.00 [(4)]

Soil Data

γ_{total} = 115 pcf

γ_{sat} = 120 pcf

Layer 1 Fines content = 15% [(5)]

Layer 2 Fines content = 30% [(5)]

Layer 3 Fines content = 30% [(5)]

Seismic Data

a_{max} = 0.140 g

M_w = 6.32 [(7)]

MSF = 1.36 [(8)]

K_a = 1.00 [(9)]

Conversion Factors

1 atm = 2,116.2 psf

1 m = 3.28 ft

1 m^2 = 10.76 ft^2

Notes

(1) Rod length unknown

(2) Not known if a liner was used, or if there was space for a liner in the sampler

(3) For HW size casing with 117 mm OD

(4) Hammer type unknown

(5) Based on boring log descriptions

(6) Maximum value of 2.0 to prevent unnecessarily high Factors of safety

(7) Largest recorded earthquake from STID

(8) Revised value from Idriss (1999)

Depth	Depth	El.	El.	σ_v	σ_v'	σ_v'	$N_{6"}$	$N_{6"}$	N	C_N	N_1	$N_{1,60}$	$\Delta N_{1,60}$	$N_{1,60cs}$	$CRR_{7.5}$ [(6)]	r_d	CSR	K_σ	FS
(ft)	(m)	(ft)	(m)	(psf)	(psf)	(Kpa)	(bpf/2)	(bpf/2)	(bpf)		(bpf)	(bpf)		(bpf)					
17	5.18	226.9	69.18	1,965	1,840	88	9	10	19	1.0630	20.20	20.20	3.26	23.46	0.2576	0.9237	0.0898	1.0208	2.0
19	5.79	224.9	68.57	2,205	1,955	94	6	7	13	1.0358	13.47	13.47	3.26	16.73	0.1713	0.9114	0.0935	1.0090	2.0
21	6.40	222.9	67.96	2,445	2,071	99	4	4	8	1.0099	8.08	8.08	3.26	11.34	0.1276	0.8987	0.0966	1.0018	1.8
23	7.01	220.9	67.35	2,685	2,186	105	1	3	4	0.9853	3.94	3.94	3.26	7.20	0.0995	0.8857	0.0990	0.9971	1.4
25	7.62	218.9	66.74	2,925	2,301	110	6	9	15	0.9618	14.43	14.43	5.36	19.79	0.2033	0.8725	0.1009	0.9885	2.0
27	8.23	216.9	66.13	3,165	2,416	116	6	7	13	0.9395	12.21	12.21	5.36	17.58	0.1795	0.8590	0.1024	0.9835	2.0
29	8.84	214.9	65.52	3,405	2,531	121	10	13	23	0.9181	21.12	21.12	5.36	26.48	0.3298	0.8454	0.1035	0.9684	2.0
31	9.45	212.9	64.91	3,645	2,647	127	25	24	49	0.8977	43.99	43.99	5.36	49.35	2.0000	0.8316	0.1042	0.9331	2.0
33	10.06	210.9	64.30	3,885	2,762	132	15	20	35	0.8782	30.74	30.74	5.36	36.10	1.4101	0.8178	0.1047	0.9247	2.0
35	10.67	208.9	63.69	4,125	2,877	138	25	25	50	0.8595	42.98	42.98	5.36	48.34	2.0000	0.8039	0.1049	0.9084	2.0
37	11.28	206.9	63.08	4,365	2,992	143	13	19	32	0.8416	26.93	26.93	5.36	32.30	0.6751	0.7900	0.1049	0.9207	2.0
39	11.89	204.9	62.47	4,605	3,107	149	25	25	50	0.8245	41.22	41.22	5.36	46.59	2.0000	0.7762	0.1047	0.8857	2.0
41	12.50	202.9	61.86	4,845	3,223	154	16	21	37	0.8080	29.90	29.90	5.36	35.26	1.1702	0.7624	0.1043	0.8872	2.0
43	13.11	200.9	61.25	5,085	3,338	160	6	4	10	0.7921	7.92	7.92	5.36	13.28	0.1422	0.7487	0.1038	0.9522	1.8
45	13.72	198.9	60.64	5,325	3,453	165	2	9	11	0.7769	8.55	8.55	5.36	13.91	0.1472	0.7352	0.1032	0.9475	1.8
47	14.33	196.9	60.03	5,565	3,568	171	3	13	16	0.7623	12.20	12.20	5.36	17.56	0.1793	0.7218	0.1024	0.9360	2.0
49	14.94	194.9	59.42	5,805	3,683	176	25	25	50	0.7482	37.41	37.41	5.36	42.77	2.0000	0.7086	0.1016	0.8355	2.0

Min 1.36

APPENDIX E

SELECTED PAGES FROM IDRISS & BOULANGER (2008)

The upper limit for the MSF would then be computed as

$$(\mathrm{MSF})_{\mathrm{max,\ cohesionless}} = \left(\frac{15}{2.7}\right)^{0.34} \approx 1.8 \tag{50}$$

MSF values at different M values can be similarly computed by using the above expressions and the correlation between N and M in Figure 62. This approach was used by Idriss (1999) to arrive at the following relationships between the MSF and M:

$$\mathrm{MSF} = 6.9 \exp\left(\frac{-M}{4}\right) - 0.058 \tag{51}$$

$$\mathrm{MSF} \leq 1.8 \tag{52}$$

The MSF values obtained via the above recommended expressions are presented in Figure 63, together with those proposed by other researchers. These MSF values are somewhat greater (at M < 7.5) than those proposed by Seed and Idriss (1982), Tokimatsu and Yoshimi (1983), and Cetin et al. (2004). In contrast, the Idriss (1999) MSF values are significantly smaller than those proposed by Ambraseys (1988) and Arango (1996). The latter researchers had used different r_d relationships, along with empirical techniques that mixed the

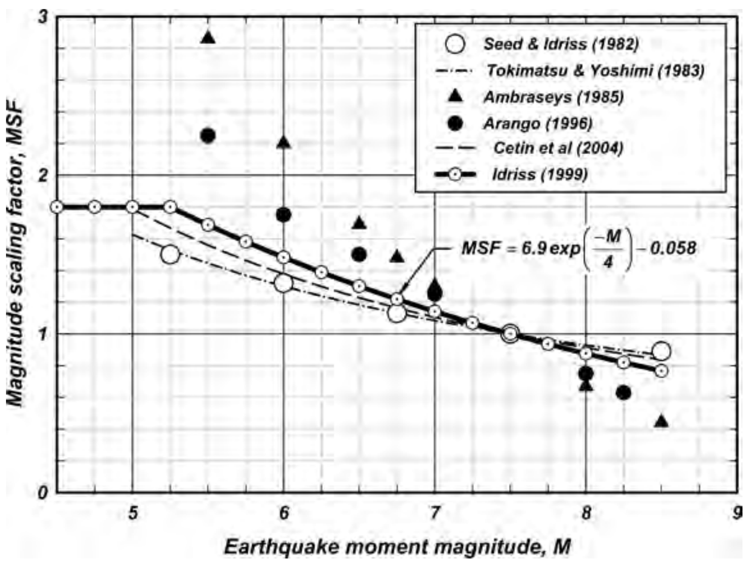

Figure 63. Magnitude scaling factor values proposed by various researchers.

3.10 SPT and CPT Correlations for Triggering of Liquefaction in Clean Sands

The compiled SPT and CPT data for clean sands are shown in Figures 66 and 67, respectively, along with the boundary lines derived by Idriss and Boulanger (2004) and those proposed by other researchers in earlier studies. The CRR-ξ_R relationships derived from the liquefaction correlations by Idriss and Boulanger are shown in Figure 68, which illustrates the consistency that was obtained between the two liquefaction correlations. These derived correlations between CRR and penetration resistances can be expressed via the following expressions for the SPT and CPT, respectively:

$$\mathrm{CRR}_{M=7.5,\sigma'_{vc}=1} = \exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right) \quad (70)$$

$$\mathrm{CRR}_{M=7.5,\sigma'_{vc}=1} = \exp\left(\frac{q_{c1Ncs}}{540} + \left(\frac{q_{c1Ncs}}{67}\right)^2 - \left(\frac{q_{c1Ncs}}{80}\right)^3 + \left(\frac{q_{c1Ncs}}{114}\right)^4 - 3\right) \quad (71)$$

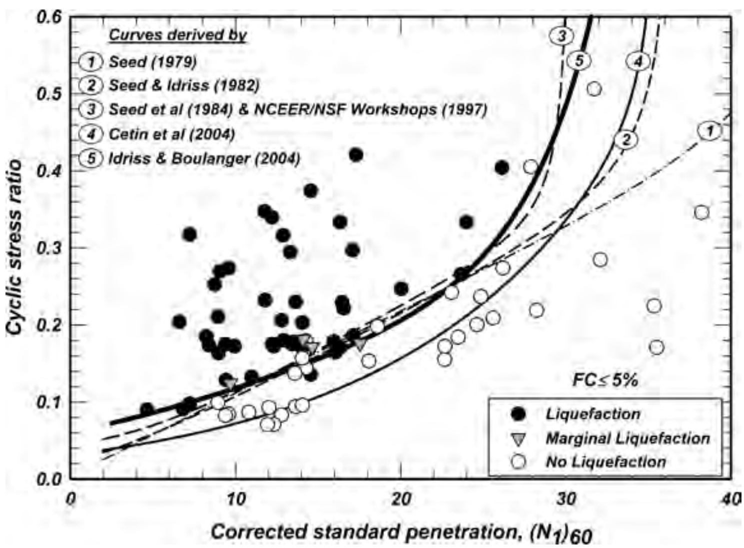

Figure 66. Curves relating the CRR to $(N_1)_{60}$ for clean sands with M = 7.5 and $\sigma'_{vc} = 1$ atm.

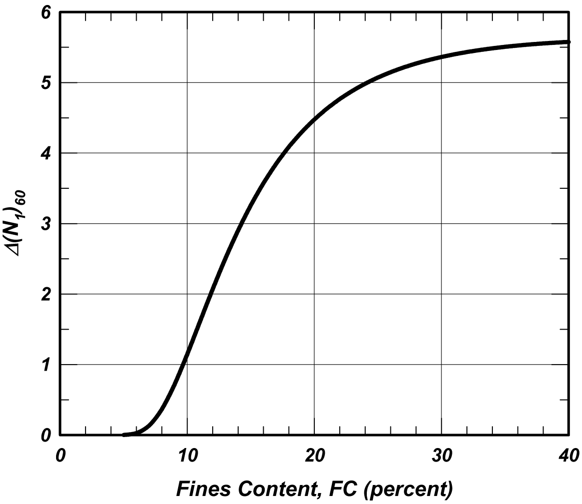

Figure 74. Variation of $\Delta(N_1)_{60}$ with fines content.

$15\% \leq$ FC $< 35\%$. Again, the revised curve is lower than the NCEER/NSF workshop curve, and this reflects the influence of the recent SPT case history data set compiled by Cetin et al. (2000).

The revised boundary curves for silty sands are horizontal translations of the boundary curve for clean sand and can therefore be conveniently represented using an equivalent clean-sand SPT penetration resistance computed as

$$(N_1)_{60cs} = (N_1)_{60} + \Delta\,(N_1)_{60} \tag{75}$$

$$\Delta(N_1)_{60} = \exp\left(1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right) \tag{76}$$

The variation of $\Delta(N_1)_{60}$ with FC, calculated via equations 75 and 76 (with FC in percent), is presented in Figure 74. Note that the correction for fines content is constant for FC values greater than about 35%, which is consistent with experimental observations that the behavior of silty sand with this level of fines content is largely governed by the matrix of fines, with the sand particles essentially floating within this matrix (e.g., Mitchell and Soga 2005). For silty sands with gravel contents of up to 15–20%, the liquefaction resistance is expected to depend primarily on the silty sand matrix. In those cases, SPT $(N_1)_{60}$ values should be carefully screened for the influence of the gravel particles (e.g., Figure 52), and then the FC used to compute the $\Delta(N_1)_{60}$ may be based on the soil fraction passing the No. 4 sieve.

where r_d is a shear stress reduction coefficient. The variations of $(\tau_{\max})_r$ and $(\tau_{\max})_d$ will typically have the form shown in Figure 50, and thus the value of r_d will decrease from a value of 1 at the ground surface to lower values at large depths.

One-dimensional dynamic site response analyses have been used to develop simplified expressions for r_d. These analyses have shown that r_d is particularly dependent on the earthquake ground motion characteristics (e.g., intensity and frequency content), the shear wave velocity profile of the site, and the nonlinear dynamic soil properties (Seed and Idriss 1971, Golesorkhi 1989, Idriss 1999, Cetin et al. 2004).

Idriss (1999), in extending the work of Golesorkhi (1989), performed several hundred parametric site response analyses and concluded that, for the purpose of developing liquefaction evaluation procedures, the parameter r_d could be adequately expressed as a function of depth and earthquake magnitude (M). The following expressions were derived by using those results:

$$r_d = \exp(\alpha(z) + \beta(z)M) \tag{22}$$

$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right) \tag{23}$$

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right) \tag{24}$$

in which z is depth in meters, M is moment magnitude, and the arguments inside the sine terms are in radians. Equations 22–24 are mathematically applicable to a depth of $z \leq 34$ m. However, the uncertainty in r_d increases with increasing depth, so these equations should actually be applied only for depths that are less than about 20 m. Liquefaction evaluations at greater depths often involve special conditions for which more detailed analyses can be justified. For these reasons, it is recommended that the CSR (or equivalent r_d values) at depths greater than about 20 m be based on site response studies—provided, however, that a high-quality site response calculation can be completed for the site. Site response analyses for this purpose require sufficient subsurface characterization of the site and must account for variability in the possible input motions.

Figure 51 shows plots of r_d calculated by using the above recommended expressions for M values of 5.5, 6.5, 7.5, and 8. Also shown in this figure is the average of the range published by Seed and Idriss (1971). The information in Figure 51 indicates that the average of that series with an equivalent number of uniform cycles that depends on the uniform cyclic stress amplitude (as described in Section 3.5).

Consequently, Seed and Idriss (1971) chose to represent earthquake-induced cyclic stresses by using a representative value (or equivalent uniform value) equal to 65% of the peak cyclic stress. The corresponding earthquake-induced CSR is therefore computed as

$$\mathrm{CSR} = 0.65\frac{\tau_{\max}}{\sigma'_{vc}} = 0.65\frac{\sigma_{vc}}{\sigma'_{vc}}\frac{a_{\max}}{g}r_d \tag{25}$$

The choice of 0.65 to represent a reference stress level is somewhat arbitrary, but it was selected in the beginning of the development of liquefaction evaluation procedures in 1966 and has been in use ever since. More importantly, the overall liquefaction evaluation procedure would be essentially unaffected by the choice of a different reference stress ratio, provided that the adjustment factors for the duration of shaking and the empirically derived liquefaction correlations were all derived for that reference stress (see Section 3.5).

3.4 In-Situ Tests as Indices for Liquefaction Characteristics

The in-situ tests that have been most widely used as indices for evaluating liquefaction characteristics include the SPT, CPT, BPT, large penetrometer test (LPT), and shear wave velocity (V_s) test. The SPT was used first in developing liquefaction correlations and was the most common in practice up through the 1990s. The CPT has a number of advantages, however, that have made it the primary site characterization tool in certain geologic settings. The BPT, LPT, and V_s tests tend to be used in special situations and thus are used less often than the SPT and CPT in liquefaction evaluations. Each of these tests is discussed separately below, after which the complementary roles of site investigation techniques and the advantages of pairing techniques (e.g., CPT soundings and SPT borings) are discussed.

SPT

The SPT is a widely available sampling method that indicates a soil's compactness or strength. The SPT measures the number of blows (N) by a 140-pound hammer falling freely through a height of 30 in. that are required to drive a standard split-spoon sampling tube (2 in. outside diameter, $1^3/_8$ in. inside diameter) to a 12-in. depth after an initial seating drive of 6 in. The thick walls of the split-spoon sampler range of delivered energy can be 30–90% of the theoretical maximum energy (the 140-lb hammer multiplied by its 30-in. drop height), depending on the amount of energy lost to frictional and mechanical resistances that depend on the type of equipment and its operating condition. The N value is essentially inversely proportional to the delivered energy (Schmertmann and Palacios 1979). In U.S. practice, the delivered energy is commonly about 55–60% of the theoretical maximum energy (Kovacs et al. 1983), and therefore Seed et al. (1984) recommended adopting N_{60} as a standard. The value of N_{60} is computed as

$$N_{60} = N_m \frac{ER_m}{60} \tag{26}$$

where N_m is the measured blow count, ER_m is the measured delivered energy ratio as a percentage, and N_{60} is the blow count for an energy ratio of 60%. The ratio of $ER_m/60$ is also referred to as an energy ratio correction factor, C_E. The energy ratio is one of the most important variables in obtaining reliable N_{60} values. Therefore, it is important that energy ratios be routinely measured as part of liquefaction evaluations.

Additional correction factors may be needed to arrive at a more standardized value of N_{60}. The resulting relationship is given by

$$N_{60} = C_E C_B C_R C_S N_m \tag{27}$$

in which C_E is the energy ratio correction factor described above, C_B is a correction factor for borehole diameter, C_R is a correction factor for rod length, and C_S is a correction factor for a sampler that had room for liners but was used without the liners. Suggested ranges for each factor are in Table 3. The borehole diameter and sampler correction factors can be important in interpreting older borings, but for future applications it is recommended that appropriate standards be followed so that the C_B and C_S factors are unnecessary (i.e., each is equal to unity).

The short rod correction factor C_R (Table 3) is intended to account for how the energy transferred to the sampling rods is affected by rod length (e.g., Schmertmann and Palacios 1979). The hammer-to-anvil impact sends a compressive stress wave down the sampling rods, which then reflects from the sampler as a tension wave. This tension wave returns to the anvil, where it causes the hammer to bounce off the anvil. Schmertmann and Palacios (1979) concluded that the energy imparted to the sampling rods during this primary impact insensitive to variations in the assumed relationships between ξ_R and penetration resistance, because the same relationships were used in mapping the field correlation to a CRR-ξ_R relationship and in mapping it back to K_σ relationships.

The recommended K_σ relationships are computed as

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_{vc}}{P_a}\right) \leq 1.1 \tag{54}$$

where the coefficient C_σ can be expressed in terms of the sand's D_R or the overburden corrected penetration resistances as (Boulanger and Idriss 2004a)

$$C_\sigma = \frac{1}{18.9 - 17.3 D_R} \leq 0.3 \tag{55}$$

$$C_\sigma = \frac{1}{18.9 - 2.55\sqrt{(N_1)_{60}}} \leq 0.3 \tag{56}$$

$$C_\sigma = \frac{1}{37.3 - 8.27(q_{c1N})^{0.264}} \leq 0.3 \tag{57}$$

The coefficient C_σ may be restricted to its maximum value of 0.3 by restricting $(N_1)_{60}$ and q_{c1N} to values $\leq$ 37 and $\leq$ 211, respectively, in these expressions. Values of K_σ computed via equations 56 and 57 are shown in Figure 64 for a range of $(N_1)_{60}$ and q_{c1N} values. These plots

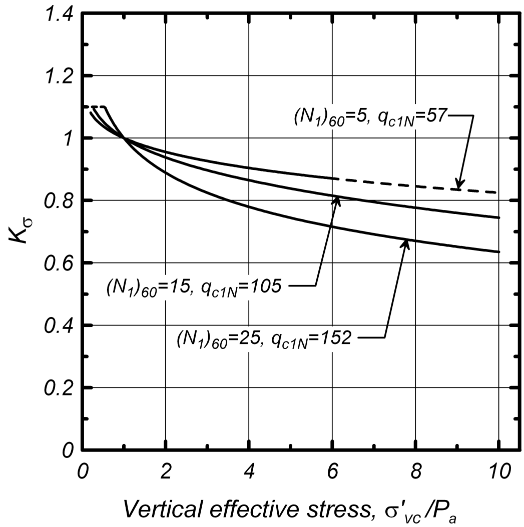

Figure 64. K_σ relationships derived from ξ_R relationships (Boulanger and Idriss 2004).

APPENDIX E

HYDRAULIC AND HYDROLOGIC ANALYSIS

Part I – Completed by Originator

Project Name: Crisp County Power Plant Action Plan/Crisp County Power Commission

1. If this is a revision, explain reason for revision: N/A
2. Have superseded versions been VOIDED or destroyed as required? ☒N/A ☐No ☐Yes
3. Has design or analysis software been used for this calculation? ☐No ☒Yes
   3.1. If Yes, provide the following information:
   3.2. Software Name: HEC HMS Version Number: 3.5
   3.3. Computer serial number of computer used for this calculation: RIZZO tag # 000519
   3.4. Confirm that software is listed on Form QP-7-13. ☐No ☒Yes
   3.5. Confirm that Software Usage Log has been updated to include this calculation. ☐No ☒Yes
4. Has a thorough self-check of this calculation been completed and accurate? ☐No ☒Yes
5. Is this calculation nuclear safety related? ☒No ☐Yes
   5.1. Has In-Use Test been performed on the computer used for this calculation? ☒N/A ☐No ☐Yes
   5.2. If "No" or "N/A," explain: Not nuclear safety related

Part II – Completed by Verifier(s) – The Independent Reviewer shall address the following:

1. Calculation inputs were correctly selected? ☐N/A ☐No ☒Yes
2. Significant assumptions are adequately identified, described, justified, reasonable? ☐N/A ☐No ☒Yes
3. Any assumptions identified for re-verification are completed? ☒N/A ☐No ☐Yes
4. Calculation inputs were correctly incorporated into the design? ☒N/A ☐No ☐Yes
5. Numerical calculations are correct and documented? ☐N/A ☐No ☒Yes
6. Calculation outputs were reasonable compared to inputs ☐N/A ☐No ☒Yes
7. Calculation input and verification requirements for interfaces are identified (e.g., specified in the Work Plan, supporting procedures, or instructions)? ☒N/A ☐No ☐Yes
8. Suitable materials, parts, processes, inspection and testing criteria were specified (e.g., may be applicable to design calculations, field activities, etc.)? ☒N/A ☐No ☐Yes
9. Hand-annotated changes are made correctly (single line strike through, initialed, and dated)? ☒N/A ☐No ☐Yes
10. All pages are legible, references identified and appropriate; document identifier and revision assigned; and acceptable with respect to grammar, spelling and punctuation? ☐N/A ☐No ☒Yes
11. Each calculation input, information and equations from external sources referenced? ☐N/A ☐No ☒Yes
12. Calculation report contains the required information? ☐N/A ☐No ☒Yes

REVIEW COMMENTS:

None

Part III – Approval for Calculations

Originator(s) Print Name	Signature/Date	
Greg Shaffer, P.E.	Digitally signed by Greg Shaffer Date: 2014.11.17 15:40:48 -05'00'	
Verifier(s)	**Signature/Date**	**Verification: Independent Design Review**
Allan Estivalet, P.E.	Allan Estivalet 2014.11.17 15:50:33 -05'00'	
Project Manager	**Signature/Date**	
Conrad Ginther, P.E.	Digitally signed by Conrad Ginther Date: 2014.12.02 15:18:46 -05'00'	

Approval of the Project Manager signifies that the document and all required reviews are complete, and the document is released for use.

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1.0 STATEMENT OF PURPOSE

The purpose of this analysis is to determine if the capacity of the outflow structure of the Plant Crisp's Coal Combustion Waste (CCW) Impoundment is sufficient to pass the design flood for the dam without overtopping the embankment.

2.0 DESCRIPTION OF METHODOLOGY USED

The design flood event for the dam depends upon its classification, which is based upon the threat of potential damage that may be posed by a dam failure to the life and property. Two classification systems are considered in this analysis.

The first classification system considered is endorsed by FEMA. According to FEMA's hazard classification system (**Reference 3**), the CCW Impoundment is a low hazard structure (**Reference 2**). This indicates that a hypothetical failure does not result in loss of life or major economic and/or environmental losses. According to FEMA, the design flood for a low hazard dam is the 100-year flood event (**Reference 4**).

The second classification system is promulgated by the state of Georgia. Georgia determines a dam's hazard class based upon the storage capacity and height of a dam. The CCW Impoundment has a maximum embankment height of 23 feet and a maximum storage volume of 42.1 acre-feet (**Reference 6**). Therefore, according to the state of Georgia, the structure is considered a small dam (e.g., a dam with storage capacity less than 500 acre-feet and a height not exceeding 25 feet (**Reference 5**). The design flood event for a small dam is a precipitation event equal to 25% of the Probable Maximum Precipitation (PMP) (**Reference 5**).

The analysis compares the design flood events for the CCW Impoundment based upon the two classification systems described above. The more conservative of the two design flood events is modeled in Hydrologic Modeling Software (HMS) (version 3.5) from the Hydrologic Engineering Center (HEC) of the US Army Corps of Engineers. The precipitation event causes the antecedent water level in the CCW Impoundment to rise, initiating outflow from the Impoundment spillway. The model results include the reservoir storage elevation, which is compared to the embankment crest elevation.

CCW Impoundment

The CCW Impoundment sits on a mild slope surrounded on three sides by an earth embankment. The earth embankment crest varies in elevation between

243 ft and 245 ft[1]. Coal ash is pumped into the impoundment via a discharge pipe from the coal plant as a slurry (i.e., an ash-water mixture). Coal ash settles in the impoundment and water slowly infiltrates into the ground. During periods of high water in the impoundment, excess water is discharged from the pond via a vertical 12-inch-diameter corrugated metal pipe (CMP). The CMP acts as a morning glory spillway with a crest elevation of 240.95 ft (**Reference 6**).

The elevation-storage and elevation-area curves for the CCW impoundment are shown on **Figure 1**. Contours from the site survey (J.B. Faircloth & Associates, 2014) are provided in **Appendix A**.

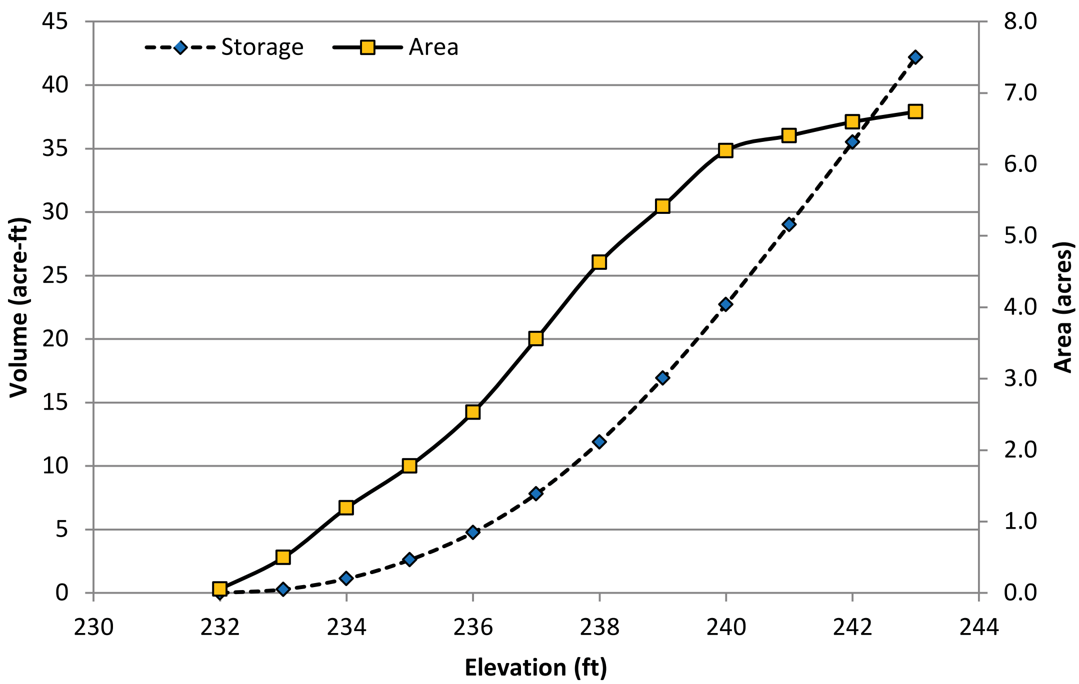

FIGURE 1: STORAGE AND AREA CURVES FOR CCW IMPOUNDMENT

<u>Design Flood Event</u>

Two design flood events are evaluated in this analysis: the 100-year flood and 25% of the PMP. The more conservative of the two is used in the hydrologic model.

To determine the 100-year flood, the 100-year rainfall depths serve as the precipitating event. The 100-year flood does not necessarily result from the 100-year precipitation event, but for a small, isolated watershed such as the CCW Impound, the rainfall is the primary driver for a flood event. See **Assumption 1** for further explanations.

[1] Elevations reported herein are in reference to North American Vertical Datum of 1988 unless otherwise noted.

The 100-year precipitation depths from 5 minutes up to 24 hours for the CCW Impoundment are presented in **Table 1** (**Reference 8**).

TABLE 1: 100-YEAR PRECIPITATION EVENT

Duration	Cumulative Precipitation (inches)
5-min	1.11
10-min	1.62
15-min	1.98
30-min	2.89
60-min	3.78
2-hr	4.67
3-hr	5.2
6-hr	6.25
12-hr	7.46
24-hr	8.57

The PMP for the CCW Impoundment is found using Hydrometeorological Report (HMR) 51 (**Reference 7**), which provides the PMP depths for select storm durations. For this analysis, 6-hr, 12-hr, and 24-hr storm events are evaluated (**Table 2**). These values are applicable for storm events up to 10 square miles in area. Each PMP depth is multiplied by 0.25 to obtain the design event according to GA DNR (**Reference 5**).

TABLE 2: PMP DEPTHS FOR CCW IMPOUNDMENT

	Storm Duration		
Event	**6 hr**	**12 hr**	**24 hr**
PMP Depth (inches)	31.3	37.5	44.2
25% of PMP (inches)	7.83	9.38	11.05

The smallest time increment presented in HMR 51 is 6 hours, which is a coarse time frame when analyzing precipitation events of 6, 12 and 24 hours. Therefore, the temporal distribution of a 500-year precipitation event (from **Reference 9**) is used to supplement the 25% PMP depths. A comparison of the 500-year event and 25% PMP event is presented in **Table 3**, along with the shorter durations available for a 500-year event.

TABLE 3: COMPARISON OF THE 25% PMP AND 500-YEAR PRECIPITATION EVENTS

Duration	25% PMP	500-Year
5-min:	-	1.41
15-min:	-	2.52
60-min:	-	4.95
2-hr:	-	6.19
3-hr:	-	6.99
6-hr:	7.83	8.39
12-hr:	9.38	9.81
24-hr:	11.05	11.03

Using the normalized temporal distribution of the 500-year event, the hyetographs for a precipitation event equal to 25% of the PMP is developed (**Table 4**).

TABLE 4: TEMPORAL DISTRIBUTION FOR 25% PMP EVENTS

	6 Hour 25% PMP		12 Hour 25% PMP		24 Hour 25% PMP	
Duration	% of 500-Year Event Cumulative Rainfall Depth (%)*	Calculated Incremental Precipitation for 25% PMP Event (inches)	% of 500-Year Event Cumulative Rainfall Depth (%)*	Calculated Incremental Precipitation for 25% PMP Event (inches)	% of 500-Year Event Cumulative Rainfall Depth (%)*	Calculated Incremental Precipitation for 25% PMP Event (inches)
5-min	17%	1.32	14%	1.35	13%	1.41
15-min	30%	2.35	26%	2.41	23%	2.52
60-min	59%	4.62	50%	4.73	45%	4.96
2-hr	74%	5.77	63%	5.92	56%	6.20
3-hr	83%	6.52	71%	6.68	63%	7.00
6-hr	100%	7.83	86%	8.02	76%	8.41
12-hr	N/A	N/A	100%	9.38	89%	9.83
24-hr	N/A	N/A	N/A	N/A	100%	11.05

* - These values represent the percentage of rainfall falling within the specified duration for 6-hr, 12-hr, and 24-hr 500-year precipitation events.
N/A – Not Applicable

Between the 100-year precipitation event and 25% of the PMP, the 25% of the PMP provides greater precipitation depths. Among the 25% of the PMP events, the 24 hour event has greater rainfall depths than the 6 hour or 12 hour events. Therefore, the 24-hour 25% of the PMP incremental precipitation values are used in this analysis and implemented into the HEC-HMS Model.

Spillway Hydraulics

The vertical CMP that serves as the outlet structure for the CCW Impoundment functions as a morning glory spillway.

Flow regimes developed for a morning glory spillway are dependent to the water head above the spillway crest and the dimensions of the different geometric features of the spillway. For small heads, flow over the spillway is governed by the characteristics of crest discharge. For bigger discharges, submergence begins to affect the weir flow and ultimately the crest will drown out and orifice control flow (throat control) will govern until the development of full pipe flow conditions.

Discharge from the spillway is calculated using **Equation 1 (Reference 1)**. The coefficient of discharge, C_0, was developed from empirical laboratory model testing, and takes into account the different flow regimes that develop for the spillway, including the effects of submergence and back pressure. C_0 is related to both H_0 and R_s. The relationship between C_0, H_0 and R_s is plotted on **Figure 2** for a ratio approach depth (P) to circular spillway radius (R_s) superior to 2.

$$Q = C_o(2\pi R_s)H_o^{3/2} \qquad (1)$$

Where:

- Q = discharge (cfs)
- C_o = circular crest coefficient of discharge (-)
- R_s = spillway radius (ft)
- H_o = head above spillway crest (ft)

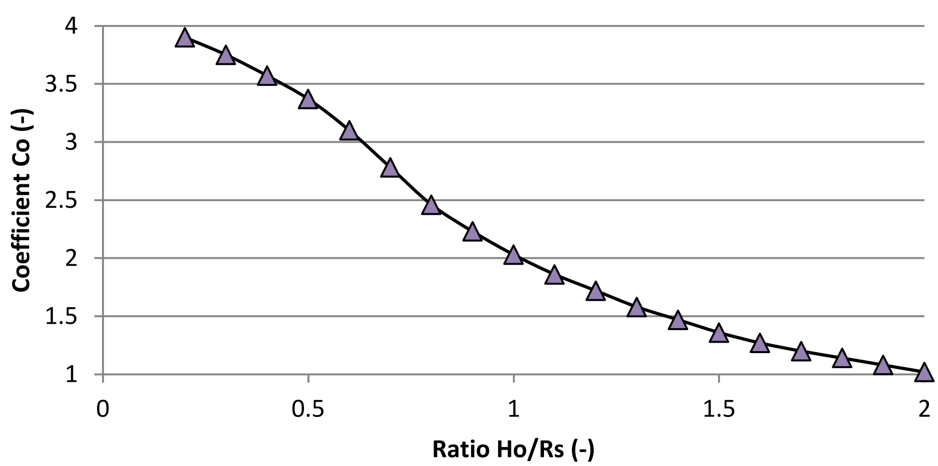

FIGURE 2: RELATIONSHIP OF CIRCULAR CREST COEFFICIENT CO TO H_O/R_S FOR RATIO P/RS EQUAL OR SUPERIOR TO 2

3.0 ASSUMPTIONS AND JUSTIFICATION

1. The 100-year flood is a flood event that occurs on average once every 100 years. The 100-year rainfall event is the rainfall event that occurs on average once every 100 years. While these two events often have a strong correlation, a flood's magnitude is not solely influenced by the precipitation. Particularly for larger watersheds, additional factors (antecedent soil moisture, existing snow pack, prior river levels, etc.) can have a significant effect on realized flood levels for a common precipitation event. For watersheds where these other factors vary, the probability of various values for each parameter (e.g., existing snowpack) must be considered.

   However, for this analysis on the CCW Impoundment, these other factors have little effect. Precipitation that falls outside of the impoundment perimeter is conveyed around the impoundment. Therefore all runoff that ends up in the impoundment comes from precipitation that falls directly within the limits of the impoundment.

   Furthermore, since the impoundment is assumed to have a starting water level at the spillway crest, the flood that results following a 100-year rainfall event would produce a more severe flood than a 100-year flood. For the purposes of this analysis, it is conservatively assumed that the 100-year flood coincides with the 100-year precipitation event.

2. It is assumed that in the event of a very large rainfall event, coal-ash slurry is not being discharged into the CCW Impoundment. It is reasonable to assume that if heavy precipitation is forecast and the pond is full, that discharging to the pond would cease.

3. It is assumed that the spillway and discharge pipe are free of debris and other obstructions that reduce the discharge capacity of the system.

4.0 CALCULATION INPUT

The following design standards apply to the CCW Impoundment:

TABLE 5: DESIGN REQUIREMENTS

Classification System	Classification	Design Requirement
FEMA	Low Hazard	100-Year Flood
State of Georgia	Small Dam	25% of Probable Maximum Precipitation Event

5.0 NUMERICAL CALCULATIONS

The spillway elevation-discharge curve calculations are calculated as shown in **Tables 6 & 7**.

TABLE 6: DAM AND SPILLWAY DIMENSIONS

Geometry	Dimension
Minimum Dam Crest Elevation (ft NAVD88)	243.0
Spillway Crest Elevation (ft NAVD88)	240.95
Spillway Diameter (ft)	1.0

The spillway rating curve for the CMP spillway computed using **Equation 1** is shown on **Figure 3** below. This relationship is implemented into HEC-HMS.

TABLE 7: SPILLWAY DISCHARGE CALCULATIONS

H_0 [ft]	Stage [ft]	C_0 [-]	H_0/R_s	$Q=C_0*\pi*D_s*H_0^{3/2}$ [cfs]
0	240.95	-	0	0.00
0.1	241.05	3.9	0.2	0.39
0.15	241.10	3.75	0.3	0.68
0.2	241.15	3.57	0.4	1.00
0.25	241.20	3.37	0.5	1.32
0.3	241.25	3.1	0.6	1.60
0.35	241.30	2.78	0.7	1.81
0.4	241.35	2.46	0.8	1.96
0.45	241.40	2.23	0.9	2.11
0.5	241.45	2.03	1	2.25
0.55	241.50	1.86	1.1	2.38
0.6	241.55	1.72	1.2	2.51
0.65	241.60	1.58	1.3	2.60
0.7	241.65	1.47	1.4	2.70
0.75	241.70	1.36	1.5	2.78
0.8	241.75	1.27	1.6	2.85
0.85	241.80	1.2	1.7	2.95
0.9	241.85	1.14	1.8	3.06
0.95	241.90	1.08	1.9	3.14
1	241.95	1.02	2	3.20

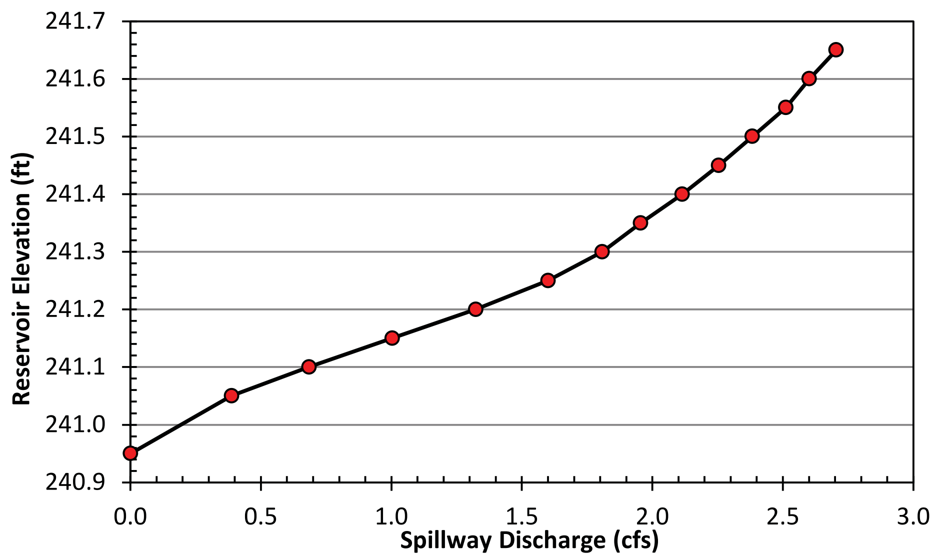

FIGURE 3: CCW IMPOUNDMENT SPILLWAY RATING CURVE

6.0 CALCULATION OUTPUT

HEC-HMS files are provided in Appendix A.

7.0 RESULTS

The total precipitation to fall in the reservoir is 11.03 inches. The precipitation causes the water surface elevation in the CCW Impoundment to peak at 241.6 ft, an increase of 0.65 ft above the starting water surface elevation and 1.4 feet below the lowest portion of the embankment crest. The peak outflow through the spillway is 2.7 cfs. Graphical results are presented in ***Figure 4***.

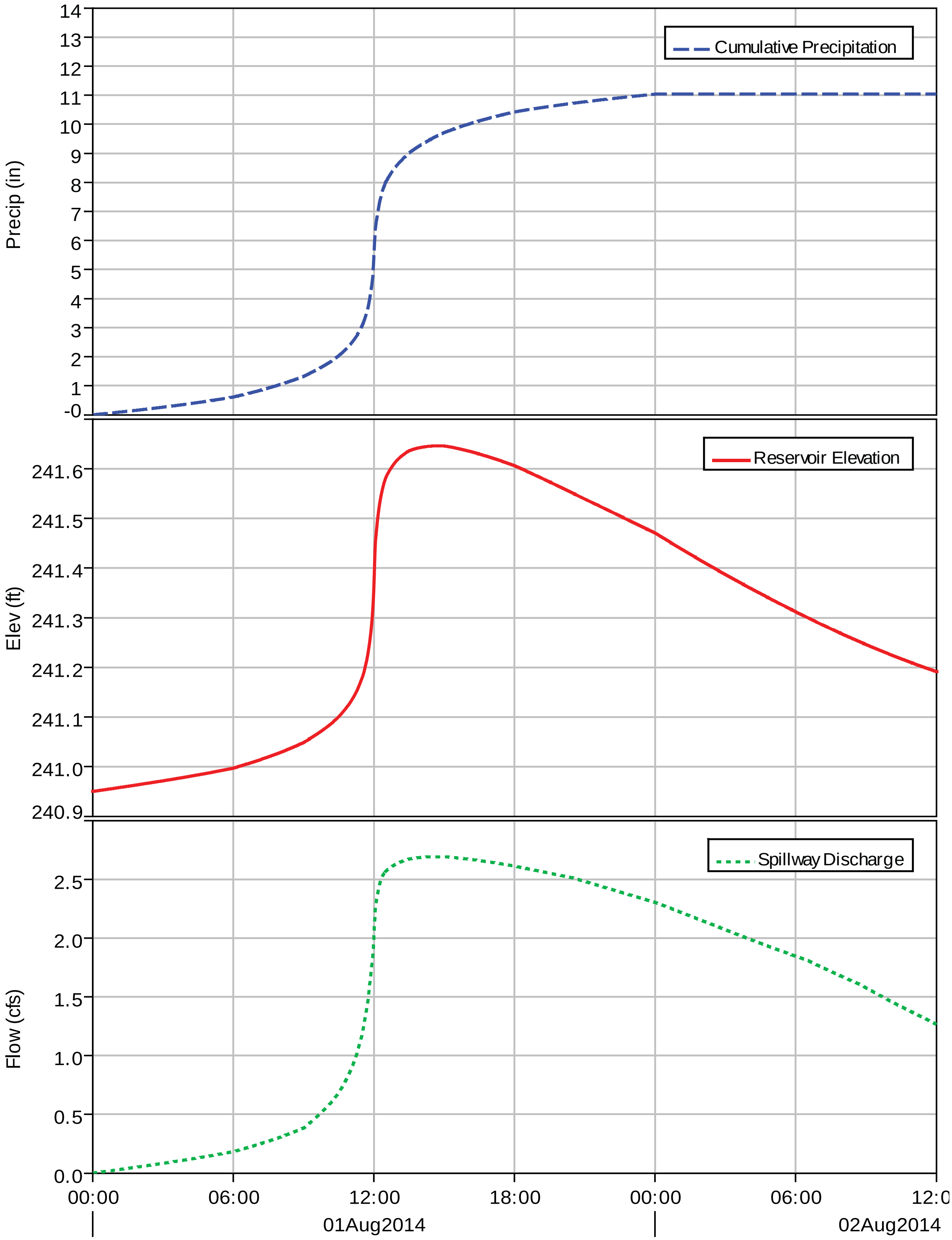

FIGURE 4: HEC-HMS RESULTS FOR CCW IMPOUNDMENT

8.0 CONCLUSION/SUMMARY

The hydrologic and hydraulic analyses performed for this calculation demonstrate that the capacity of the outflow structure of the Plant Crisp's Coal Combustion Waste Impoundment is sufficient to pass the design flood for the dam (i.e. 25% of the PMP) without overtopping the embankment and with an available freeboard of more than one foot.

9.0 REFERENCES

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Appendix A
Electronic Files