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Seismic Evaluation and Remediation of Embankment Dam

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ABTRACT

The seismic stability of the Croton Dam left embankment, was evaluated for potential earthquake ground motions. Field and laboratory test results were used to characterize the static and dynamic properties of the embankment and foundation materials. Results showed liquefaction and strength loss in certain zones of the embankment resulting in large deformation of the slope. Remediation of these areas of the dam was necessary. Various techniques were evaluated including drainage, construction of a berm at toe, vibro-grouted stone columns and compaction grouting. Compaction grouting was selected.

INTRODUCTION

The Croton Project is located on the Muskegon River in Grand rapids county, Michigan. The project is owned and operated by Consumer Power Company. The project structures include two earth embankments, a gated spillway, and a concrete and masonry powerhouse. The earth embankments of this project were constructed of sand with concrete core walls. The embankments were built using modified hydraulic fill method. This method consisted of dumping the sand and then sluicing the sand into the desired location. Croton Dam is classified as a "high-hazard" dam and is in Earthquake Zone 1. As part of the FERC Part 12 Inspection by Acres in 1996, an evaluation of the seismic stability was performed for the downstream slope of the Left Embankment at Croton Dam (Uddin 1996). The Croton embankment was analyzed in the following manner: soil parameters were chosen based on Standard penetration (N) values and laboratory test and a seismic study was carried out to obtain the design earthquake. Using the chosen soil properties, a static finite element study was made to evaluate the existing state of stress in the embankment. Then a one-dimensional dynamic analysis was conducted to determine stress induced by design earthquake shaking. The available strength was compared with expected maximum earthquake conditions so that the stability of the embankment during and immediately after an earthquake could be evaluated. The evaluation showed that the embankment had a strong potential to liquefy and fail during the design earthquake. Consumers Energy requested Acres to study options for stabilizing the slope and remediation of the liquefaction potential. After a detailed study, the minimum soil strength required to eliminate the liquefaction potential was determined and Acres recommended the embankment soils be strengthened by in-site densification. This paper will present a synopsis of the earthquake evaluation of Croton dam, and the design of remedial measure.

EARTHQUAKE STUDY

Maximum credible earthquakes have been determined for the Croton dam. The maximum credible earthquake is the maximum design earthquake used for engineering analysis. The steps involved in determining the maximum credible earthquake were 1) identifying seismic source zones capable of generating large earthquakes, 2) determining the potential horizontal acceleration at each site, and 3) selecting a recorded strong ground motion. Each design earthquake includes one standard deviation, which amounts to a factor of safety of 1.7 applied to the input acceleration. The Taft record was then used to represent the design accelerations. The Croton project is located 776, 412, 250 and 165 kilometers from New Madrid A, Wabash Valley, Northern Illinois and Ana, Ohio source zones, respectively. The recommended potential horizontal acceleration determined for Croton is 0.09 g.

SEISMIC EVALUATION

Two modes of failures were considered in the analyses namely, loss of stability and excessive deformations of the embankment. Following analyses were carried out in succession: (1) determination of pore water pressure buildup immediately following the design earthquake, (2) estimation of strength for the loose foundation layer during and immediately following earthquake, (3) analysis of the loss of stability for post-earthquake loading where loose sand layer in embankment is completely liquefied with pore water pressure ratio 100%, and (4) a liquefaction impact analysis for the loose sand layer for which the factor of safety against liquefaction is unsatisfactory.

GEOMETRY OF THE EMBANKMENT AND MATERIAL PROPERTIES

Figures 1 and 2 show the plan view and Figure 3 shows the

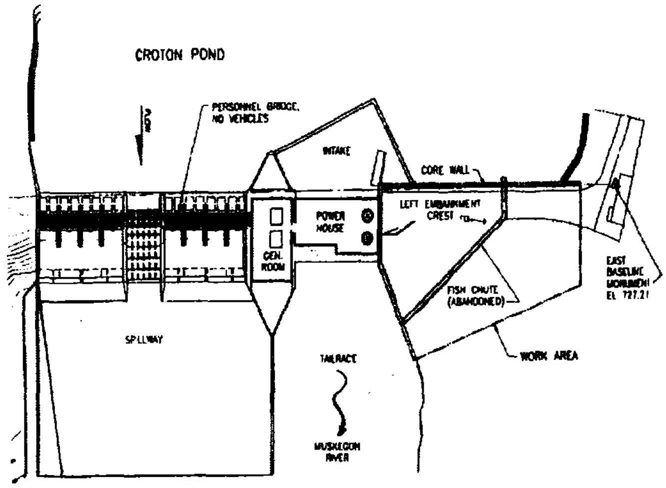


Fig. 1. Plan View of the Croton Dam

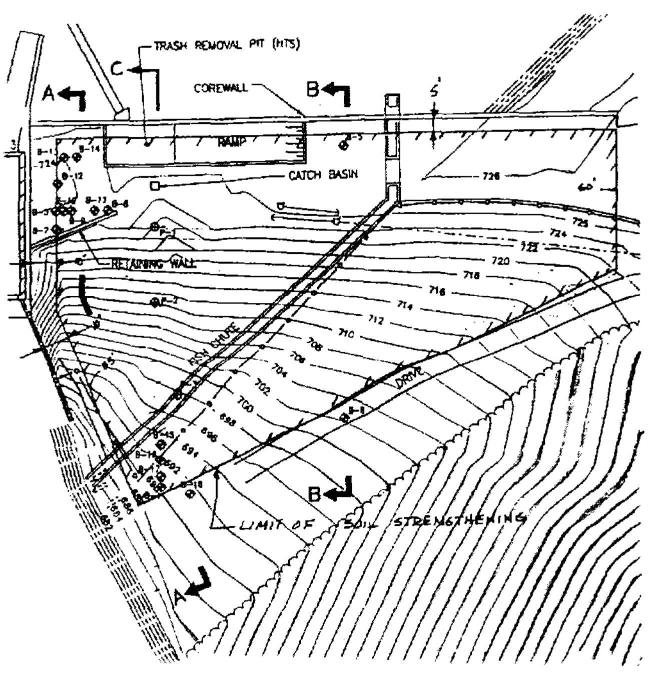
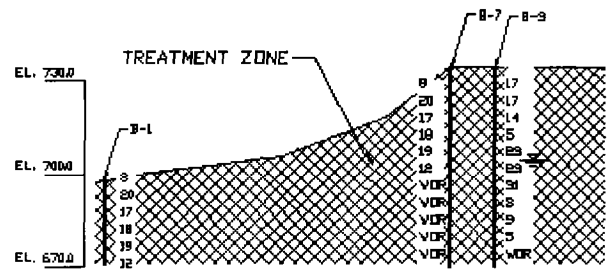
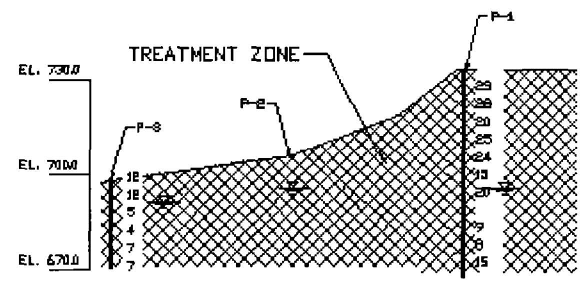


Fig. 2 Plan View of the Left Embankment of Croton Dam

sections through the embankment. Figure 3 indicates loose to very loose zone immediately below the phreatic surface and extends to the top of the embankment. Based on the SPT test and geotechnical laboratory test following soil parameters are assumed for the properties: 1) embankment soil – unit weight : dry – 108 psf and saturated – 118 pcf; Friction angle 34.3 degrees, cohesion = 0 psf. 2) Foundation soil – unit weight : dry – 125 psf and saturated – 130 pcf; Friction



Section A – A



Section B - B

Fig. 3. Cross Sections of the Left Embankment with the SPT values before the Strengthening of the Embankment

angle 38 degrees, cohesion = 0 psf. For simplicity it is conservatively assumed that loose sand zone is horizontal and continuous with a uniform thickness of 15 ft. Piezometer reading dated February 1996 is used to construct the ground water surface in the slope.

The topographic survey drawing shows that the majority of the downstream slope is 1V:2.5H (or flatter) with an area of slope near the powerhouse at 1V:2H. The 1V:2.5H slope, by inspection, has acceptable stability for the steady state seepage conditions at maximum pool (normal) and maximum surcharge (flood). The 1V:2H slope is confined to a 60-foot wide area between the powerhouse/tailrace wall and the

1V:2.5H slope area. The slope stability analysis is two-dimensional and does not account for any three-dimensional stabilizing effects at the edges. Also, not counted in the analysis is the stabilizing affect of the vegetation root system, which can add up to 3 degrees to the internal friction angle. Acres previous slope stability analysis for the left embankment showed the 1V:2H downstream slope had the following factors of safety:

	Acres Analysis	**FERC Reqd.**
Left embankment D/S slope:		
with normal water level	1.41	1.50
with high water level	1.37	1.40

The calculated factors of safeties are marginally low by less than 1/10 of a point and the associated failure surfaces are shallow. The left embankment is considered stable for the steady state seepage condition of normal and high water level and stabilization measures are not required for these conditions. The loose zones in the embankment still require treatment in order to be stable during the design earthquake.

LIQUEFACTION ANALYSIS RESULTS

The available strength in the embankment was determined from FEADAM and SHAKE analyses at the locations of the two columns "A" (at embankment centerline) and column "B" (at embankment toe). The available strength was compared with the induced cyclic stress ratio from the SHAKE study. The factor of safety against liquefaction at column "A" is only 0.075 and at column "B" is 2.7. The minimum required factor of safety against liquefaction for the project is 1.01. However, for simplicity and to be conservative, it is concluded that the loose sand layer from the centerline of the embankment (A line) up to the toe (B line) has the potential to be completely liquefied following the design earthquake.

ANALYSIS FOR THE LOSS OF STABILITY FOLLOWING THE EARTHQUAKE

The potential for loss of stability is analyzed using conventional analysis recommended by FERC (section 4-6, FERC guidelines for the evaluation of Hydropower Projects), and incorporating the residual strength values to the liquefied soil layer following the design earthquake. Three cases were considered based on the estimates of residual shear strength for the liquefied soil layer (Case A: Upper bound 300 psf, Case B: Lower bound 0 psf, and Case C: Average 150 psf). The factor of safety against flow failure following earthquake are 0.63 for Case A, 0.01 for Case B, and 0.38 for Case C, all well below acceptable factor of safety (=1).

LIQUEFACTION IMPACT ASSESSMENT

Total settlement of 15 ft thick loose embankment layer due to complete liquefaction was found to be 0.75 ft.

PERMANENT DEFORMATION ANALYSIS

Based on a procedure by Makdisi and Seed (1978), permanent deformation can be calculated using the yield acceleration, and the time history of averaged induced acceleration. Since the factor of safety against flow failure immediately following the earthquake falls well short of required FERC, the Newmark type deformation analysis is unnecessary. Therefore, it can be concluded that the embankment will undergo significant permanent deformation following the earthquake due to slope failure in excess of liquefaction-induced settlement of 0.75 ft.

LEFT EMBANKMENT LIQUEFACTION REMEDIATION

Based on the above results, Acres recommended the embankment soils be strengthened by in-situ densification (Uddin, 1996). An analysis is carried out to determine the minimum soil strength required to eliminate the liquefaction potential. The analysis is divided into 3 parts as follows:

(1) Slope Stability Analysis (using PCSTABL) of the downstream slope of the Left Embankment: Strength and geometric parameters are varied in order to determine the minimum residual shear strength and minimum zone of soil strengthening required for post earthquake stability factor of safety, F.S.>1. It is assumed that loose Zone extends form EL 695 to EL 670 (i.e. river bottom) and from core wall (at upstream crest) to daylight at downstream. The results of numerous PCSTABL runs with varying soil strengths and locations of treated zone are not included here for brevity. The presented PCSTABL run (Figure 4) shows that the minimum

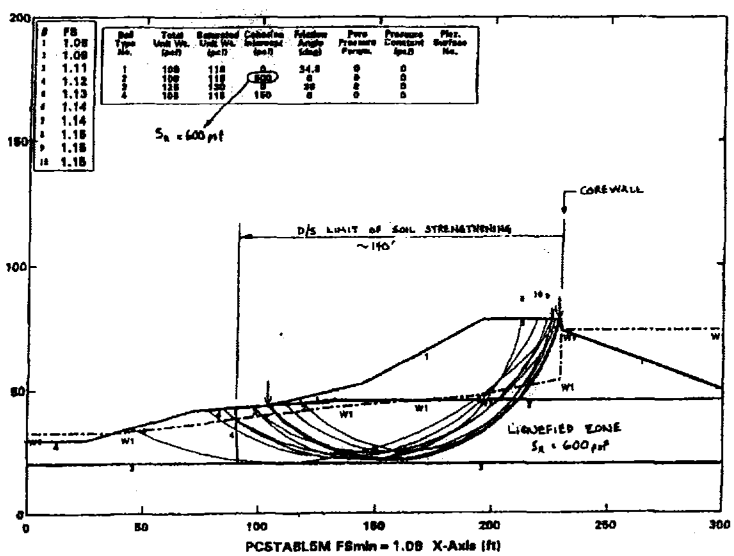


Fig. 4 Post Earthquake Stability Analysis (Residual Shear Strength = 600 psf)

shear strength of 600 psf for the assumed treated zone is adequate to obtain factor of safety of FS = 1.08.

(2) SPT Values: Based on the relationship between corrected "Clean Sand" blow count $(N_1)_{60}$ and undrained residual strength (S_R) from Case studies (Seed and Harder, 1990), equivalent clean sand blow count, $(N_1)_{60\text{-}cs} = 16$, corresponding to undrained residual shear strength, S_R = 600 psf. After the SPT corrections for the fines content the minimum residual shear strength correlates to a corrected/normalized penetration resistance value, $(N_1)_{60}$ = 15. From this value a back calculation is performed to determine the minimum field measure standard penetration resistance N-values (blows per foot) including the corrections for overburden stress and field procedures.

(3) Liquefaction Analysis: Liquefaction potential is re-evaluated based on the minimum zone of strengthening and minimum average normalized N value. The calculated factor of safety for both column A and Column B is observed to be more that one (i.e., 1.34 and 1.7 for columns A and B respectively). It is therefore, concluded that if the embankment is strengthened to the minimum then the liquefaction potential in the downstream slope of the left embankment will, for all practical purposes, be eliminated.

SELECTION OF REMEDIAL MEASURES

Maximum possible area of treatment has been estimated based on the location of Original River channel (Ref. General Design Drawings shown in Exhibit 2, Sheet 2 of 1991 Part 12 Inspection Report by Mead & Hunt), Left Embankment Investigations by Blystra, October 1990, and the 1997 Topographic Survey provided by Consumer Energy. The maximum possible area to treat extends from the core wall (upstream) to 20 ft downstream of the down stream toe of the left embankment and from the powerhouse left tailrace wall east to the original river bank. Limits of soil strengthening are shown on Figures 1 and 2. Clearances of 10 feet and 5 feet are recommended at Powerhouse/Tailrace walls and core wall respectively. However contractor will be required to determine clearances required for no damage to existing structures. The upper limit of soil strengthening should be to the ground surface to ensure that all areas of the embankment are strengthened to the same degree.

Table 1 summarizes all the remedial measures considered.

Table 1: Left Embankment Stabilization

Option	**Technical Review**	**Total Cost ($)**
1A [New Toe Drains]	Does not increase stability or reduce liquefaction. Not practical - Does not lower water level in dam	--------
1B [Slot Drains]	Reduces liquefaction potential. Conventional construction. One third of the loose sand remains saturated. Settlement will occur with earthquake.	1,235,000
2A [Vibroflotat-ion]	Eliminates liquefaction potential and increases stability. Proprietary method, surface settlement will occur during construction; possible damage to pavement; same profile.	240,000
2B [Jet Grouting]	Eliminates liquefaction potential and increases stability. Specialist method, repair pavement, same profile. No surface settlement, may raise water level.	350,000
2C [Compaction Grouting]	Eliminates liquefaction potential and increases stability. Specialist method commonly used for dams, no settlement, no repairs, same profile.	240,000
3A [Partial Reconstructi on]	Eliminates liquefaction potential and increases stability. Conventional construction, no settlement, new pavement	1,805,000
4A [Toe Berm (2H:1V) Sand]	Increases stability and reduces liquefaction potential. Does not reduce post seismic settlement. Berm changes profile. Surface settlement will occur with earthquake. Significant reinforcement of tailrace wall.	365,000
4B [Toe Berm (1.5H:1V) Gravel]	Increases stability and reduces liquefaction potential. Does not reduce post seismic settlement. Berm changes profile. Surface settlement will occur with earthquake. Significant reinforcement of tailrace wall.	435,000
4C [Flatten Slope (3H:1V) Sand]	Increases stability and reduces liquefaction potential. Does not reduce post seismic settlement. Berm change profile. Surface settlement will occur with earthquake. Significant reinforcement of tailrace wall.	740,000

The residual undrained shear strength required by analysis is on the order of 600-700 psf, which is by Seed's Chart (Seed et al, 1990), correlates to a SPT blow count of 16 (medium dense). This is an achievable value for the in-situ soil improvement Options 2A, 2B and 2C. Costs for soil strength improvement Options 2A (Vibroflotation), 2B (Jet Grouting), and 2C (Compaction Grouting) have been estimated based on the maximum plausible area of treatment which has been taken as about 70% of the Maximum Possible Area. Specialty contractors were contacted and have provided budgetary prices.

The risk of settlement damage to surface structures (i.e., pavement and retaining walls) as a result of soil strengthening operations is greatest for Vibroflotation and the least for Compaction grouting and Jet grouting. The vibration associated with the Vibroflotation method may cause uncontrollable settlement directly beneath the surface structures. The other two grouting methods work by adding material to the loose zone and there is little to no risk of uncontrollable surface settlement with these methods. Compaction Grouting involves injecting a still (1 inch slump), soil and water mixture into the loose zones. The grout displaces and densifies the loose soil and forms cylinders around the injection hole. This method has been successfully used to strengthen weak cohesionless zones in embankment dams and their foundations. It is attractive because it does not involve large vibratory forces (i.e. vibrocompaction), which induce self-settlement of loose zones and potential settlement at the surface. The grout is still and must be pumped at high pressures but at very slow rates, not more than 2 cubic feet per minute. Monitoring is commonly performed to measure ground heave and to control effects on adjacent structures. Minimum clearance distances are specified where treatment is close to structures.

Feasibility of Toe Berm Options 4A and 4B is highly dependent on the feasibility of reinforcing the left tailrace wall. The wall in its present condition is slightly overstressed according to Barr Engineering analysis prior to repair. Barr's As-Built drawing notes that wall geometry is different from Construction Issue drawings i.e. the actual wall is 7 feet higher than what Barr assumed for design. It is likely the wall will not be stable for any increased toe berm load. The toe berm required for embankment stability would significantly increase (i.e. quadruple) the load on the tailrace wall. In order to support the new toe berms, the tailrace wall would have to be extended 22 feet vertically to elevation 700 and 25 feet horizontally - downstream and would have to be extensively supported (i.e. tied back) in order to support the proposed soil load. This is a major effort that would require extensive engineering and difficult construction in the river with a large excavation of the left embankment in the toe area where the slope failure occurred in November 1997 during excavation for tailrace wall repairs. The cost of these options has been increased to reflect the more extensive work required.

CONCLUSIONS

- The left embankment downstream slope is stable for the normal and high water loading conditions and does not require treatment.

- The left embankment downstream slope is not stable for the design earthquake and stabilization is recommended.

- The in situ soil strengthening options of Vibroflotation and Compaction Grouting are the most attractive methods for improving the stability of the left embankment. Toe berms are much less attractive because the left tailrace retaining wall requires significantly more strengthening and stabilizing than previously determined - based on information from the as-built drawings of the 1997 tailrace wall restoration work. The preferred soil strengthening method is Compaction Grouting because it is less likely to cause damage to adjacent structures than Vibroflotation. However, the extent of damage from Vibroflotation cannot be assessed until the contractor visits the site to view the existing conditions.

- The embankment stabilization should be bid on a lump sum basis with the two options, Compaction Grouting and Vibroflotation and the stipulation that the contractor is responsible for repair of all damages. Compaction Grouting is preferred because the risk of settlement damage is much lower than with Vibroflotation. The cost of potential settlement damage has been included in Vibroflotation.

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