15 Apr 2004, 1:00pm - 2:45pm

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Proceedings: Fifth International Conference on Case Histories in Geotechnical Engineering
New York, NY, April 13-17, 2004

EVALUATION OF ALTERNATIVES FOR EARTHQUAKE HAZARD MITIGATION OF AN EMBANKMENT DAM IN KANSAS

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ABSTRACT

A major embankment dam, approximately 140 feet high and over one mile long, is located in a zone of moderate seismicity in Eastern Kansas. It was determined that slightly cohesive soils and fine sands in the foundation are vulnerable to significant loss of strength by liquefaction during a potential strong earthquake. Numerous seismic retrofit solutions were studied, including the extreme options of “no action” and “replace embankment”. The recommended solution, which is currently designed in detail, considers jet grouting for foundation soil stabilization under the upstream slope and deep soil mixing under the downstream slope.

INTRODUCTION

Tuttle Creek dam is located in a zone of moderate seismicity in Eastern Kansas. The reservoir covers approximately 12,500 acres at the normal pool with over two million acre-feet of storage below the flood control pool. The purposes of the project include: recreation, fish and wildlife, water supply, water quality, flood control, and supplemental releases for navigation.

The dam is a rolled earth fill and hydraulic fill embankment, 7,500 feet in length, standing 137 feet high, with a crown width of 50 feet and a base width of 1,050 feet on an alluvial foundation. Seismic and geotechnical investigations established that a strong earthquake generated from a nearby active fault zone could induce liquefaction of the foundation soil under the lower portions of both upstream and downstream slopes.

Figure 1 shows the typical cross section of the Tuttle Creek dam and the zones in the foundation that were found susceptible to significant loss of strength following a strong earthquake shaking. It is noted that the dam does not have a positive cutoff, but a line of relief wells along the downstream toe.

Numerous seismic retrofit solutions were studied under the Dam Safety Assurance Program of the Corps of Engineers, including the extreme options of “no action” and “replace embankment”. The preliminary design of the remediation alternatives was intended to ensure a factor of safety in excess of 1.2 for post-earthquake stability with liquefaction expected to be induced by the maximum credible earthquake (defined as a 6.6 moment magnitude event at 20 km from the dam site). The strong earthquake was considered to occur with the water in reservoir at the normal pool elevation, which is relatively low, as the major function of the dam is flood control.

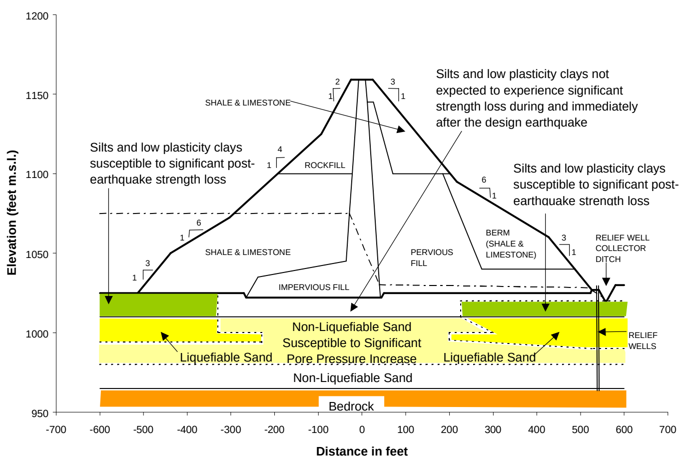

Fig. 1. Cross Section of the Dam (deformed scale).

FOUNDATION SOIL

The foundation consists of 50 to 70 feet of alluvial deposits over shale and limestone. The upper zone of about 20 feet consists of lean clay, silty clay, and clayey silt. Underneath the cohesive soil blanket are sand deposits. The upper sands are easily liquefiable: the minimum factor of safety determined with Seed-Idriss simplified procedure (Youd et al., 2001) for the maximum credible earthquake varied between 0.6 and 0.8 in free field and between 0.7 and 1.1 underneath the embankment. It was assumed that the cohesive soil in direct contact with liquefiable sand is susceptible to large deformations and, therefore, to loss of strength following a strong earthquake. It was desirable to avoid consequences of loss of strength in both sand and cohesive soil.

Figure 2 presents the gradation range of both materials, together with approximate gradation ranges where various soil improvement methods are efficiently applicable (Mitchell and Katti, 1981).

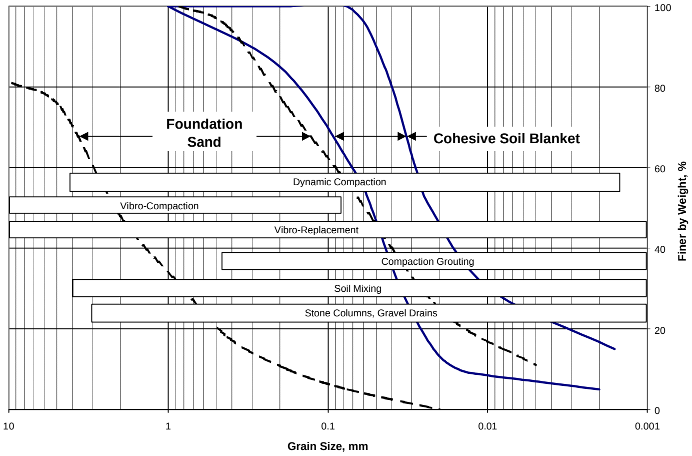

Fig. 2. Ranges of gradation of soils in foundation of the dam, compared with ranges of stabilization methods effectiveness (Mitchell and Katti, 1981).

POTENTIAL REMEDIATION ALTERNATIVES

Figure 3 summarizes the methods that were considered for improvement of seismic stability of the dam. From the beginning it was recognized that the optimum solution to be applied may include one or more methods of improvement; different methods can be applied to the upstream and to the downstream side; the selected solution (or combination of alternatives) should be the most cost effective solution that reduces the seismic hazard to a reasonable acceptable level.

An initial screening of the potential remediation alternatives was used to eliminate, form further, more detailed evaluation, the options that did not meet several critical requirements. The alternatives that met all these acceptance criteria of the initial screening were thereafter compared in view of selection of the most appropriate alternative. These two steps of the selection process and the obtained results for the Tuttle Creek dam are summarized in this paper, after a brief presentation of all considered alternatives.

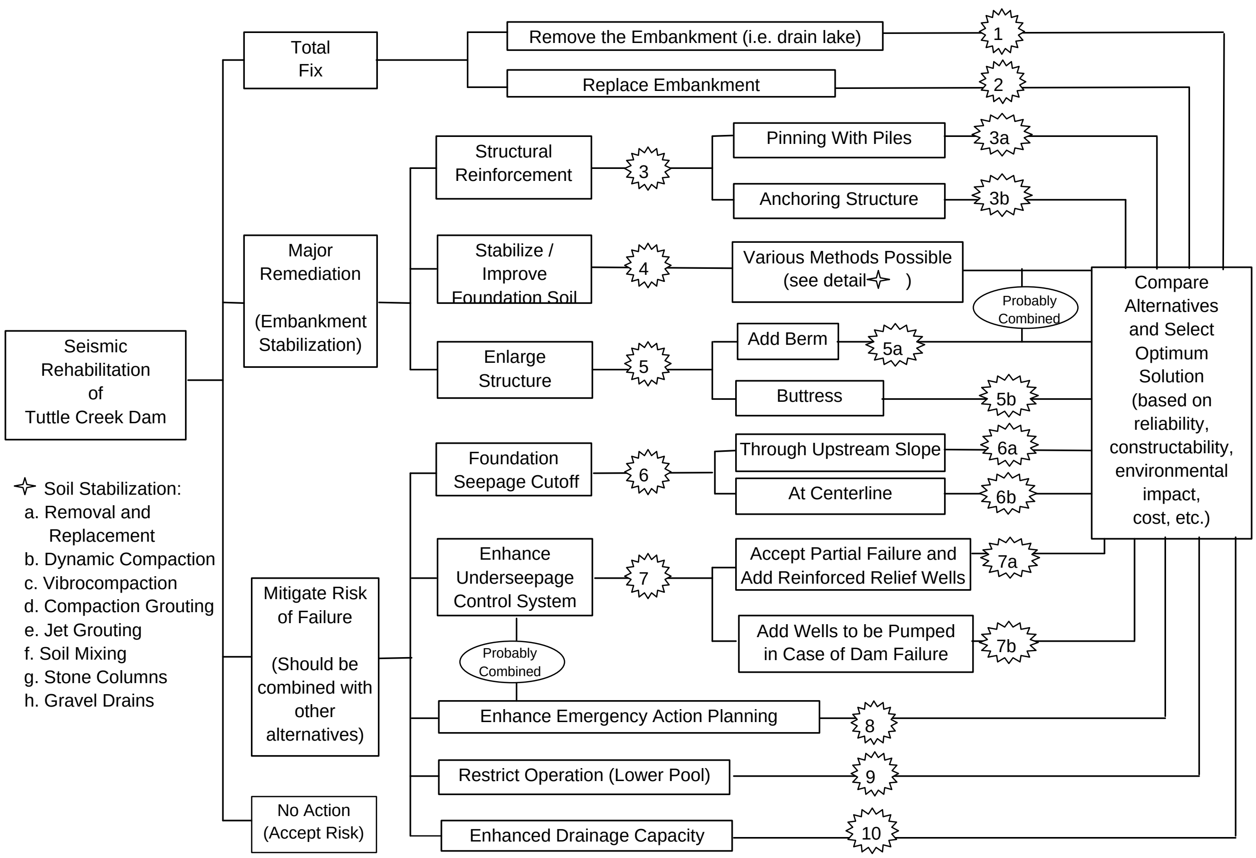

Fig. 3. Alternatives considered for improvement of seismic stability of the dam.

BRIEF PRESENTATION OF ALTERNATIVES

Remove (Breach) the Embankment

It is presented as alternative (1) in Figure 3. Examples: Lower San Fernando Dam, California (Los Angeles Department of Water and Power, LADWP, after it was heavily damaged by an earthquake in 1971), South Haiwee Dam, California (LADWP) (Markuson et al., 1996).

A portion of the dam embankment could be removed and the lake permanently drained. The breach would be wide enough (approximately 500 feet at the bottom) to safely pass a major flood event. The remaining fill would be protected against erosion. The outlet works and portions of the embankment dam, although no longer necessary, may remain in place.

Although technically sound, this alternative has major drawbacks: the annual project benefit would be completely lost and all authorized project functions (recreation, water supply, fish and wildlife, flood control, water quality, and navigation) would not be maintained. In addition, the environmental impacts would be significant and the public expressed considerable opposition to this alternative.

Replace Embankment

It is presented as alternative (2) in Figure 3. Examples: Como Dam, Montana (US Bureau of Reclamation, USBR), Echo Lake Dam, California (Pacific Gas and Electric Co.), Jackson Lake Dam, Wyoming (USBR), John Hart Dam, B.C., Canada (British Columbia Hydro and Power Authority, BCHPA), Lake Arrowhead Dam, California (California Dept. of Water Resources, CDWR), Lower San Fernando Dam, California - upper half of reservoir (LADWP), Silver Lake Dam, California (LADWP), Upper San Leandro Old Dam, California (East Bay Municipal Utility District, EBMUD) (Markuson et al., 1996 and USBR, 1987).

The existing dam could be replaced with an embankment having similar height and features. The foundation soil underneath would be stabilized, allowing the replacement embankment to have significantly steeper slopes than the original. Relief wells or a positive cut-off are also needed. If built immediately downstream of the existing structure, the new dam may use the existing spillway and outlet works.

Reinforce embankment with piles

It is presented as alternative (3a) in Figure 3. Example: Sardis Dam, Mississippi (U.S. Army Corps of Engineers, USACE) (Stacy et al., 1996).

Concrete piles would be used to pin the lower portion of the slope into the stable foundation, underneath the liquefiable layers. On the upstream side it would be necessary to drill through the embankment fill (where big stones are expected) and to drive the piles into the foundation soil. On the downstream side temporary excavation of the existing berm would be needed. It may be necessary to build a plant for manufacturing of piles in the vicinity of the dam. This alternative is presented in Figure 4.

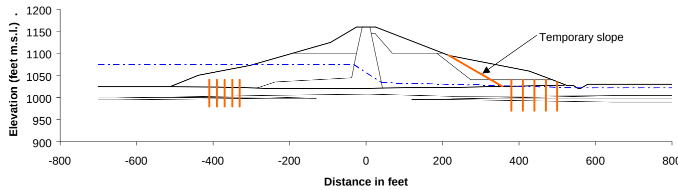

Fig. 4. Reinforcing embankment with piles.

The preliminary design of this alternative used the remediation applied at Sardis Dam, Mississippi as a guide. At Sardis Dam the maximum thickness of the weak layer was 10 feet and its stabilization required 10 rows of heavily reinforced 24x24" prestressed concrete piles at 8 to 12 feet c/c (for an average of about one pile per foot of dam). The liquefiable layer at this project is more than three times greater than that at Sardis Dam. This additional thickness resulted in such a high moment demand on individual piles that they were deemed impractical.

Reinforce embankment with anchors

It is presented as alternative (3b) in Figure 3. No example of this alternative is known.

High capacity anchors encased in concrete can prevent excessive deformation. Concrete cracking may be prevented by pre-tensioning the anchors. The forces in anchors should be distributed into the embankment fill through a reinforced slab on the slope surface. See Figure 5 for the general configuration.

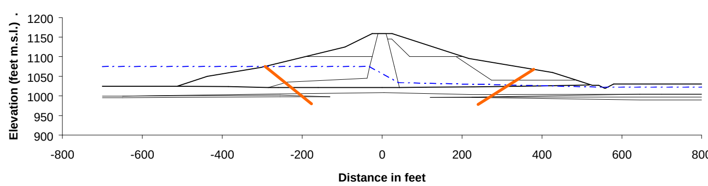

Fig. 5. Reinforcing embankment with anchors.

For both the upstream slope and downstream slope the numbers and lengths of required anchors was uneconomical as compared with the solution using piles. Assuming installation of 1-3/8" Dywadag bars, 2 to 3 anchors per foot of stabilized dam would be necessary. For an installation at approximately 45º from horizontal, the necessary length of the anchors would be between 150 and 180 feet. Therefore, this alternative was considered not technically feasible and was not studied in detail.

Stabilize Foundation Soil

It is presented as alternative (4) in Figure 3. The foundation soils could be stabilized by various means to reduce (or eliminate) its liquefaction potential and provide a strengthened soil mass to resist deformations. On the upstream side of the dam the stabilization equipment should operate from a platform through holes predrilled within the shale and limestone fill. On the downstream side an alternate option would be to temporarily remove the existing berm fill. The concept of this alternative is presented in Figure 6.

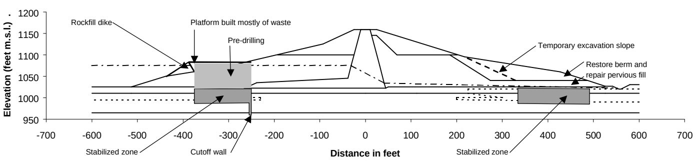

Fig. 6. Conceptual sketch of foundation soil stabilization.

Figure 2 presents the methods previously used for seismic liquefaction mitigation in the United States; the gradation range where the various methods are efficiently applicable is compared with the gradation ranges of the problem soils encountered in the Tuttle Creek dam foundation. Other more recently developed stabilization methods (jet grouting, super-jet grouting, grout piles, etc.) are currently used for seismic mitigation and may be more effective than the traditional methods. The final decision on the selected stabilization method and its optimum parameters should be based on the results of full scale test sections at the dam site.

The following methods of foundation soil stabilization were evaluated.

<u>Removal and replacement of liquefiable material.</u> Examples: Casitas Dam, California (USBR) (Parsons, 2000), Island Park Dam (USBR), Pinopolis West Dam, South Carolina (South Carolina Public Service Authority, SCPSA) (Markuson et al., 1996).

In the case of this project, deep excavation, on the order of 30-40 feet, is necessary if all problem soil is to be removed. The excavated material can be replaced, becoming non-liquefiable if properly compacted. At the downstream toe the water table is normally at a depth of 7-8 feet, so an excavation to this depth would require temporarily lowering the reservoir and a dewatering system that may include the existing wells. The removal and replacement may be restricted to the upper zone of cohesive soils (15-20 feet in depth) with in situ stabilization of the sand underneath. Removal and replacement is not an option upstream, where even temporarily draining the lake is unacceptable to the public. Also given the nature of the drainage basin, it would be impossible to keep the lake drained most of the time.

<u>Dynamic compaction (heavy tamping)</u>. Examples: Jackson Lake, Wyoming (USBR) (USBR, 1987 and Dise et al., 1994), Mormon Island Auxiliary Dam, California (USACE/USBR), Steinaker Dam, Utah (USBR) (Markuson et al., 1996 and Dise et al., 1994), Steel Creek Dam, South Carolina (USACE) (Castro et al., 1987, Rogers, 1987, Dobson, 1987, and Mitchell and Welsh, 1989).

Dynamic compaction is a competitive solution from cost and efficiency points of view, but it has restricted applicability at Tuttle Creek dam. The method is efficient only if applied at the surface of the soil to be improved or on a structural fill of selected material and relatively small thickness (sand blanket with thickness of the order of 5 feet); it is, therefore, not applicable under the upstream slope and requires temporarily removal of most of berm fill for stabilization of soil under the downstream slope.

<u>Densification by vibrocompaction</u>. Examples: Jebba Dam, Nigeria (Jebba Hydroelectric Development) (Mitchell and Welsh, 1989, Solymar et al., 1984), John Hart Dam, B.C., Canada (BCHPA), Modesto Containment Dike, California (Markuson et al., 1996).

This method is considered “not feasible” in the case of Tuttle Creek dam because of lack of efficiency in fine grained materials (blanket and upper portion of sands).

<u>Compaction grouting</u>. Examples: Mormon Island Auxiliary Dam, California (USACE/USBR, compaction grouting was studied and recommended as supplementary method, with dynamic compaction as the primary method), Pinopolis West Dam, South Carolina (SCPSA, in combination with removal and replacement) (Markuson et al., 1996.)

Compaction grouting is a displacement process: as the grout is injected a bulb grows and the soil surrounding the bulb is compacted. In the case of Tuttle Creek Dam, the more efficient jet grouting technology, which includes in the created columns part of the in situ soil, was considered for achieving similar effects.

<u>Jet Grouting</u>. Examples: Wickiup Dam, Oregon (USBR, 2000).

Jet grout segments of stabilized soil can be used to create zones of containment of the liquefiable layer. While not reducing the risk of liquefaction, containment minimizes the potential for catastrophic failure by preventing the flow of the liquefied soil. In addition, the grouted zones have increased shear strength, which opposes deformation and improves stability. Jet grouting is considered an ideal solution for the upstream slope since it can be implemented through the rockfill without lowering the lake. An additional beneficial effect of the stabilized zones is that they decrease the permeability of the foundation soil underneath the upstream slope. A full depth jet grouted wall would further assist in controlling underseepage.

<u>Soil mixing</u>. Examples: Jackson Lake Dam, Wyoming (USBR) (USBR, 1987), Lockington Dam, Ohio (Miami Conservancy District, Dayton) (Walker, 1994).

The deep soil mixing method can be used to install a wall or cells under the downstream toe, to prevent flow of the liquefied soil from under the structure. The high-productivity specialized equipment cannot work through pre-drilled holes, so that the method is not applicable to the upstream slope. However, soil mixing with Portland cement is considered one of the best solutions for the downstream slope stabilization.

<u>Densification by stone columns</u>. Examples: Hinckeley Dam, New York (New York Power Authority), John Hart Dam, B.C., Canada (BCHPA), Mormon Island Auxiliary Dam, California (USACE/USBR) (Allen et al., 1995, Baez and Martin, 1995, Boulanger et al., 1998, Kelsic et al., 1995), Salmon Lake Dam, Washington (USBR) (Luehring et al., 1998), Steel Creek Dam, South Carolina (USACE) (Castro et al., 1987, Rogers, 1987, Dobson, 1987, Mitchell and Welsh, 1989), Tolt Regulating Basin South Dam, Washington (Seattle Water Department) (Mejia et al., 1997).

There are various methods of stone columns construction, basically classified in two main categories: (1) the wet (vibro-replacement) installation method and (2) dry bottom feed stone columns. Densification is the primary mechanism of treatment, with drainage being a secondary benefit. Stone columns are not appropriate for treatment under the upstream slope because they would shorten the foundation seepage path and be detrimental for long term seepage and stability. Stone columns would be more applicable to treatment below the downstream slope, provided the column material was selected to meet filter criteria and well controlled dry bottom feed method would be used. It is uncertain if it is feasible to construct stone columns of fine enough material to meet filter criteria with the cohesive blanket material.

<u>Gravel drains</u>. Examples: Kingsley Dam, Nebraska (Central Nebraska Power and Irrigation District), Mormon Island Auxiliary Dam, California (USACE/USBR) (Markuson et al., 1996, Dise et al., 1994).

The difference between gravel drains/piles and stone columns is mainly the technology used for installation. Gravel drains/piles may be installed with impact driven casing method (Franki) or the vibro-replacement method. The Franki method is preferred as more effective in the cohesive materials of the blanket.

<u>Enlarge Embankment.</u>

It is presented as alternative (5) in Figure 3. Enlargement at the base is done by building berms either upstream or downstream, or both, using mostly dredged material from the reservoir or the lake downstream. Enlargement at the crest level by construction of a buttress, increases the width of the structure at the retention level and prevents piping even if significant cracking and displacements occur in the embankment fill.

<u>Build berm upstream</u>. Examples: Ashton Dam, Idaho (Utah Power and Light Co.), Crane Valley Dam, California (Pacific Gas and Electric Co., PGEC), Hinckeley Dam, New York (New York Power Authority), Mormon Island Auxiliary Dam, California (USACE/USBR) (Markuson et al., 1996), Sardis Dam, Mississippi (USACE) (Stacy et al., 1996).

The berm upstream would be built underwater. The top of the berm would be above multipurpose pool and will create a dry platform in normal conditions from where the soil underneath can be improved. Alternatively, the soil improvement may be performed before building the berm. The stabilizing effect of the berm is significantly decreased by submergence. Also it was assumed that the liquefiable sand extends indefinitely upstream, which is a legitimate assumption. Therefore, a relatively wide berm of 400 feet is necessary (Figure 7).

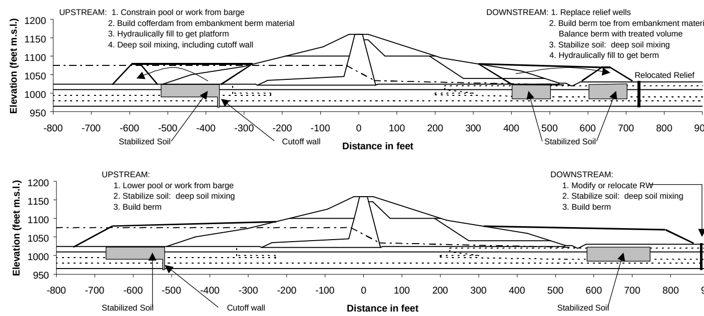

Fig. 7. Berms on stabilized soil, in two variants.

<u>Build berm downstream</u>. Examples: Casitas Dam, California (USBR) (Parsons, 2000), Henshaw Dam, California (Vista Irrigation District), Hinckeley Dam, New York (New York Power Authority), O'Neill Dam, California (USBR), Pineview Dam, Utah (USBR), Steinaker Dam, Utah (USBR) (Markuson et al., 1996).

A downstream berm, approximately 425 feet wide, would adequately control deformation. Stabilization of soil underneath is necessary to prevent damage to the pressure relief system if the berm were to fail. The existing pressure relief system would be replaced with a new system located further downstream (Figure 7).

<u>Add buttress downstream</u>. Examples: Austrian Dam, California (San Jose Water Co.), Butt Valley Dam, California (PGEC) (Verighin and Gutierrez, 1998), Calaveras Dam, California (CDWR), Casitas Dam, California (USBR) (Parsons, 2000), Chabot Dam, California (EBMUD), Lake Almanor Dam, California (PGEC) (Verighin and Gutierrez, 1998), Rye Patch Dam, Nevada (USBR) (France et al., 1994), Salmon Lake Dam, Washington (USBR) (Luehring et al., 1998), Thermalito Afterbay Dam, California (CDWR) (Markuson et al., 1996), Tolt Regulating Basin South Dam, Washington (Seattle Water Department) (Mejia et al., 1997).

The preliminary design determined the need of a buttress 100 feet wide at the crest and 300 feet wide at the ground level to seismically stabilize the dam. The upper portion should be reinforced and a strong internal drainage must be built between the new and the old embankment fill. A new pressure relief system is recommended. Soil improvement under the buttress is necessary.

Figure 8 presents this variant.

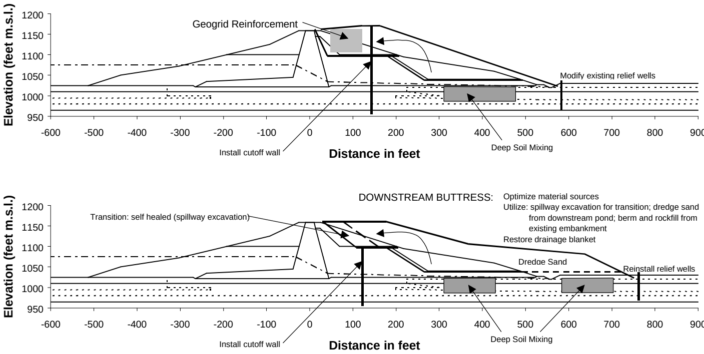

Fig. 8. Preliminary design of the buttress, in two variants.

Foundation Seepage Cutoff.

It is presented as alternative (6) in Figure 3.

For subsequently presented alternatives the risk of failure is only partially reduced, so other alternatives should be used in a combined solution.

There are no known examples of using seepage cutoff for seismic rehabilitation. However, diaphragm walls (e.g. built by jet grouting) were used for containment of the potentially liquefiable soil.

Positive control of underseepage would eliminate the necessity of pressure relief systems along the downstream toe and, therefore, the danger of piping if the existing system is destroyed by large deformations of the embankment near the downstream toe. Two variants of this alternative may be effective:

Cutoff through the upstream slope, within the limits of the upstream impervious fill. This location minimizes the thickness of the existing fill that must be penetrated and does not require temporary lowering of the pool. As seismic deformations are considered possible at this location, the allowable deformations should be coordinated with the thickness and flexibility of the cutoff wall.

Cutoff through central core. The advantage of this location is that no significant seismic deformations are probable. Therefore both cement/bentonite backfills or concrete diaphragm walls are possible options, and their thicknesses may be minimized, within the limits of constructability. A deep channel exists in the bedrock at approximately the middle of the valley so that a maximum depth of about 230 feet from the dam crest is necessary for positive cutoff.

Enhanced Underseepage Control System.

It is presented as alternative (7) in Figure 3. There are no known examples of application of this alternative. For Tuttle Creek Dam two different options were considered:

Accept partial failure and add reinforced relief wells. Fifteen reliable wells are needed to prevent piping if MCE occurs with the lake pool at multipurpose level.

Accept partial failure and add wells to be pumped. In this variant 13 additional wells would be installed 600 feet downstream from the toe, far enough to prevent damage to them if the downstream slope of the dam fails. They would not have any role in normal conditions. If some of the existing relief wells fail, their function may be taken by pumping from distant wells. A number of submersible pumps and electric generators should be operable at any time.

Enhanced Emergency Action Planning.

It is presented as alternative (8) in Figure 3. Example: Santee North Dam, South Carolina (SCPSA) (Gotzmer, 1998).

Failure of the dam would be accepted but measures taken to evacuate the population downstream before the releases can reach them. Due to populated areas immediately downstream of the Tuttle Creek Dam, any evacuation plan would not be feasible; this alternative alone is not acceptable, but is considered appropriate until and during construction.

Restricted Lake Operation (permanently lowering normal pool).

It is presented as alternative (9) in Figure 3. Examples: Ascot Dam, California (LADWP), Cuyamaca Dam, California (CDWR), Henshaw Dam, California (Vista Irrigation District), Lower Franklin Dam, California (CDWR), Phoenix Lake Dam, California (CDWR), Pleasant Valley Dam, California (LADWP), Rattlesnake Canyon Dam, California (CDWR) (Markuson et al., 1996).

Although the existing freeboard at Tuttle Creek dam (based on the multipurpose pool elevation of 1075 feet m.s.l.) is 84 feet, this is not sufficient, as large deformations and severe cracking are expected in the assumption of MCE occurrence. Prevention of failure by piping, if the relief pressure system becomes non-functional following large deformations of the dam, requires permanent lowering of the lake level by approximately 25 feet (to a normal level of 1050 feet m.s.l., see Figure 9; justification of this limiting pool was based on the requirement of factor of safety against piping in excess of 1.0, in the absence of the pressure relief system. Such a dramatic pool level reduction would result in essentially a dry flood retention structure and considerable negative ecologic consequences.

Due to the drainage basin characteristics, the lake would be above the safe elevation forty percent of the time. Additionally, the remaining storage amount would not be sufficient to provide dependable yield for any of the consumptive uses (navigation, water supply and water quality). Recreation, fish and wildlife would also be severely impacted due to the change in pool. Therefore, lowering of the pool elevation would adversely impact numerous project purposes and would require a reallocation of the project and Congressional approval.

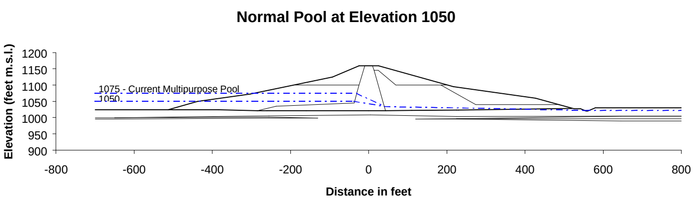

Fig. 9. Conceptual sketch of the alternative of permanently lowering normal pool.

Enhanced Drainage Capacity.

It is presented as alternative (10) in Figure 3. This alternative is intended to significantly improve the ability to drain the lake in the event of embankment failure, following a strong earthquake. This alternative was removed from detailed consideration due to the high construction uncertainty and risks, high cost to construct and maintain, failure to eliminate downstream flooding and potential loss of human life and property, and requiring human intervention to operate after a seismic event.

No Action.

The probability of occurrence of a seismic event capable to liquefy the dam foundation and, consequently, to induce major deformations of the dam and uncontrolled releases, is remote (on the order of once in 4,000 years). However, a lower seismic event (the threshold earthquake) may induce liquefaction underneath the downstream slope and, consequently, failure of the lower portion of the slope. Such a failure would fracture the existing relief wells and create piping potential that can trigger dam failure. The threshold event has a return period of about 1,800 years. Loss of life is very probable if the embankment fails by piping.

With dam failure, there would be significant impacts to the residents and users of the land and development resources downstream of the dam. The annual flood control and other benefits provided by the project would be lost and there would be additional significant downstream economic damages that would occur with a dam breach and flood wave. There would be high risk potential for loss of life in the upstream reaches below the dam. Emergency services would be impacted due to impacts to access routes and transportation infrastructure. Loss of critical services would occur, including loss of water and sewer services in the upstream reaches, and there would be environmental damages and losses. With dam failure, there would also be future costs to the Federal Government. In addition to the high cost of repairing or rebuilding the dam after failure, there could be significant costs to settle legal claims. These would include the time and resources spent in settlement negotiations and potential litigation if the Government does nothing to correct the problems identified. The dollar amount of actual claim settlements and litigation damage payouts would be significant. This alternative would also generate severe environmental impacts.

INITIAL SCREENING CRITERIA

The initial screening of the potential remediation alternatives was based on the following acceptance criteria that are either met or not met:

- ***Safety Requirement.*** In the event of the design earthquake (Maximum Credible Earthquake) occurrence, loss of life should be prevented. In other words, uncontrolled release of water in reservoir (assumed at normal, multipurpose level at the time of earthquake occurrence) must be avoided. To quantify this requirement, the following post-earthquake conditions have been defined:
  - Factor of safety for post-earthquake limit equilibrium 1.2 or greater;
  - Maximum 5 feet lost of freeboard (to prevent significant cracking of the embankment fill or overtopping of the deteriorated dam if a flood event occurs before repair work can be completed);
  - Maximum horizontal deformation of 1 foot at the downstream toe (to prevent significant damage of the relief well system that is a critical feature even in normal operation conditions);
  - Maximum horizontal deformation of 10 feet at the upstream toe (to prevent significant damage of the upstream impervious blanket and, consequently, unsafe increase in pore pressure under the downstream slope).
- ***Economic Requirement.*** The annualized cost of modification should not exceed the annual project benefit.
- ***Maintain Project Purpose.*** The dam was Congressionally authorized for: recreation, water supply, fish and wildlife, flood control, water quality, and assisting navigation on Missouri River. All these functions should be preserved unaltered after remediation.
- ***Technical Feasibility.*** This requirement includes:
  - The improvement method should be feasible under standard construction procedures and its results should be verifiable, both at a field test location and the final improvement product.

- The rehabilitation solution should be safe during construction. If soil treatment involves use of potentially hazardous and/or toxic chemical substances, Manufacturer's recommendations to protect workers against short-term hazard and environmental long-term quality should be carefully followed.
- Treatment methods and operations must be specified and monitored to prevent damage to the dam. The remediation should not create a new defect. After remedial treatment, the stability and safety of the dam under static and water loads should remain unaffected or be improved.

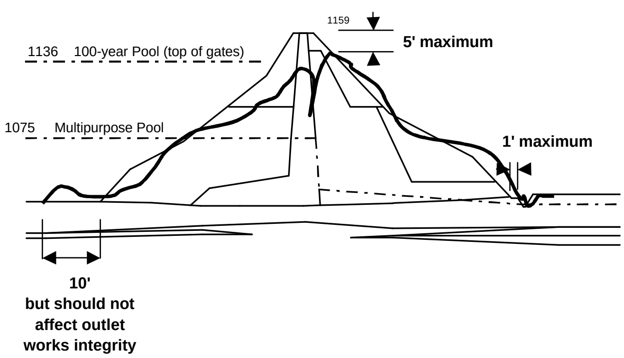

Fig. 10. Displacements considered acceptable (not to scale).

Table 1, on the next page, summarizes the results of the initial screening.

ALTERNATIVES FURTHER INVESTIGATED

Only some variants of alternatives "Stabilize/Improve Foundation Soil" and "Enlarge Structure" were determined appropriate for further evaluation, meeting all initial screening requirements. Alternatives "Foundation Seepage Cutoff", "Enhance Underseepage Control", and "Enhance Emergency Action Planning" may be selected in conjunction with other stabilization methods. It was also recognized that different methods can be applied to the upstream and to the downstream side; the selected solution should be the most cost effective alternative (or combination of alternatives) that reduces the seismic hazard to a reasonable acceptable level, is acceptable from environmental impact point of view, and meets the public preferences.

The preliminary design of the alternatives retained after the initial screening was the basis of an economic analysis. Table 2 compares the cost estimates for these alternatives, in 2001 dollars.

Table 2 – Comparison of cost estimates for potential remediation options.

Stabilization alternative	**Cost in 2001 dollars**	
	Per foot of dam length	Per cubic foot treated ground
Upstream slope:		
Jet grouting without cutoff	19,000	23
Jet grouting with cutoff	23,000	20
Berm on soil stabilized with jet grouting	17,000	N/A
Downstream slope:		
Jet grouting after pre-drilling	32,000	16
Jet grouting after temporary excavation	23,000	11
Deep soil mixing	11,000	6
Gravel columns	14,000	14
Berm on soil stabilized with soil mixing	12,000	N/A

SELECTED ALTERNATIVE

In addition to the comparison of cost estimates and technical merits, the environmental impact and the public preferences were significant criteria in selection of the recommended alternative.

The initial component of the selected alternative is the installation of a dam failure warning system. This system will monitor seismic activity, pore pressures, and deformations and will automatically provide an alert when the appropriate triggers occur. The system will be operational during design and construction to maximize the amount of time available to evacuate the 13,000 people in the downstream valley. This system is an interim measure until the activities discussed below can be implemented.

The best alternative for stabilization of the upstream slope was considered jet grouting from a platform built on the lower portion of the slope, which requires pre-drilling through the embankment fill (a mixture of shale and limestone). Although the estimate of the construction cost of this alternative was found higher than applying jet grouting from a berm built in the reservoir ($19,000 per foot of dam compared with $17,000 per foot of dam, see Table 2), it implies much less adverse environmental impact, which may become much costlier in time. Installing an underseepage cutoff at the downstream limit of the stabilization zone was also recommended. The additional cost of $4,000 per foot of dam is justified by the possibility of allowing larger deformations at the downstream toe with the final design, as the relief wells would not be critical for dam stability any more, even if the cutoff wall were not perfect.

Table 1 - Summary of the initial screening results.

Alternative / Variant	Criteria met (Y = Yes; **N** = No)					Comment
	No loss of life for MCE	Annualized cost less than project benefit	No change in project purposes	Technically feasible		
				U/S	D/S	
(1) - Breach embankment	Y	**N**	**N**	Y	Y	Eliminated
(2) - Replace embankment	Y	**N**	Y	**N**	Y	Eliminated
(3) - Reinforce embankment:						
(a) - with piles	Y	Y	Y	**N**	**N**	Eliminated
(b) - with anchors	Y	?	Y	**N**	**N**	Eliminated
(4) - Stabilize foundation soil:						
(a) - remove and replace	Y	?	Y	**N**	Y	OK d/s only
(b) - dynamic compaction	Y	Y	Y	**N**	**N**	Eliminated
(c) - vibrocompaction	Y	Y	Y	**N**	**N**	Eliminated
(d) - compaction grouting	Y	Y	Y	Y	Y	OK
(e) - jet grouting	Y	Y	Y	Y	Y	OK
(f) - soil mixing	Y	Y	Y	Y	Y	OK
(g) - stone columns	Y	Y	Y	Y	Y	OK d/s only
(h) - gravel drains	Y	Y	Y	Y	Y	OK d/s only
(5) - Enlarge embankment:						
(a) - build berms	Y	Y	Y	Y	Y	OK
(b) - add buttress	**N**	Y	Y	**N**	Y	Eliminated*
(6) - Foundation seepage cutoff:						
(a) - through u/s slope	**N**	Y	Y	Y	N/A	Eliminated*
(b) - through central core	**N**	Y	Y	Y	N/A	Eliminated*
(7) - Enhanced underseepage control system:						
(a) - add reinforced wells	**N**	Y	Y	N/A	Y	Eliminated*
(b) - add wells to be pumped	**N**	Y	Y	N/A	Y	Eliminated*
(8) - Enhanced emergency action planning	N	Y	Y	Y	Y	Eliminated *
(9) - Restricted lake operation	?	Y	**N**	Y	Y	Eliminated
(10) - Enhanced drainage capacity	**N**	**N**	Y	Y	Y	Eliminated
No action	**N**	Y	Y	Y	Y	Eliminated

Note: * Although eliminated, this variant may be used in conjunction with other alternatives.

The cheapest and recommended alternative for stabilization of the downstream slope was deep soil mixing from a platform at the surface of the existing horizontal pervious fill, obtained by excavation of the lower portion of the embankment fill. The temporary downstream slope of the portion of the dam where the fill will be excavated will be 1(v) : 2.75(h). The contamination with grout from deep soil mixing operation of the 15-foot drainage blanket material will be minimized by lining with metal pipes the holes within it. The general concept of the selected alternative is presented in Figure 6.

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