Evaluation of Structural Alternatives for Seismic Risk Mitigation at Deer Creek Dam

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ABSTRACT

Deer Creek Dam is a zoned earthen embankment on the Provo River in central Utah. The site is potentially subject to very severe earthquake loading, and some foundation materials have been identified as being likely to be liquefied by strong shaking. This could lead to instability or large dynamic deformations of the downstream slope, and possibly a breach of the dam. Several structural concepts were evaluated by the U.S. Bureau of Reclamation to determine the best method to stabilize the embankment. The selected modification concept was a rolled earthfill key under the downstream toe of the embankment, with a berm over it to weight the key and buttress the slope. This concept was selected in the fall of 2002, and in July 2003, a contract was awarded to modify the dam.

This paper presents background information on the dam and its foundation, and describes the process of determining that modification is required and designing the modification. The latter includes potential earthquake loadings, *in situ* and laboratory testing to evaluate the foundation, and analyses of liquefaction potential and the stability of the unmodified embankment. The various concepts for modification are described, along with the rationale for selecting the preferred concept.

INTRODUCTION

Deer Creek Dam, a zoned earthen embankment on the Provo River in Utah's Wasatch Range, was constructed in 1941. It was designed by the U.S. Bureau of Reclamation (Reclamation), which owns and operates the dam. According to practices of the time, overburden was removed from the foundation core trench and portions of the abutments. However, alluvial materials were left in place under the downstream portion of the embankment, and are now believed to be susceptible to liquefaction or significant loss of strength from earthquake loading. It has been estimated that Deer Creek Dam could be subjected to seismic loads of 0.6 g or larger. During the past eleven years, a series of field investigation programs have been performed, along with studies to evaluate the potential for seismic loading to cause damage to the dam. Following a determination of potentially unsatisfactory performance during a severe earthquake, several alternatives for mitigation were developed and evaluated. The preferred alternative for modifying the dam was approved, and in July 2003, construction operations began. In addition to modifications to the dam for seismic stability, U.S. Highway 189, which currently crosses the dam, is proposed for relocation, which affects the design of the modification.

BACKGROUND INFORMATION

Design of Dam

The embankment of Deer Creek Dam is comprised of 5 zones: a core of silty clay with sand and gravel (zone 2); flanked upstream and downstream by a semi-pervious zone (zone 1); another semi-pervious transition zone (zone 3); a zone of miscellaneous fill consisting of silt, sand and gravel which forms the toe of the downstream slope (zone 4), and an outer zone of coarse shell material. (See Figure 1.) Construction records indicate that the transition zones are composed of selected gravelly soils, and grade in coarseness towards the upstream and downstream slopes. A cutoff trench was excavated to bedrock 156 feet upstream from the dam axis. A shallower "auxiliary" foundation trench was excavated 140 feet downstream of the dam axis to improve the stability of the embankment. The existing slopes are 3(H):1(V) upstream and 2.25:1 downstream.

The original embankment design was modified during construction when excavation of the upstream cutoff trench re-vealed an extensive “clayey layer”, now referred to as Qals. The designers were concerned that the undrained strength of this layer could be low enough to allow instability of the downstream slope of the embankment during construction. Reclamation amended the construction contract to include excavation of the auxiliary trench to interrupt that layer's upstream-downstream continuity. The trench was excavated slightly into the dense gravelly unit, now referred to as Qalg1, which underlies the Qals, and backfilled with rolled Zone 1 material.

Construction records do not indicate the cross-canyon extent of the auxiliary trench, or of the Qals layer. The auxiliary foundation trench appears to stop several hundred feet short of the actual right boundary of the Qals layer, which was later determined to extend nearly to the abutment rock on both sides of the valley. The auxiliary trench may have been made shorter than desired due to logistical problems with the Denver and Rio Grande Railroad track, which then crossed the dam's footprint. (Drilling to determine whether it was indeed shortened was not feasible due to of lack of access.) Because the auxiliary trench provides critical support to the downstream embankment slope during earthquake loading, its location and depth were important to the evaluation of dynamic slope stability.

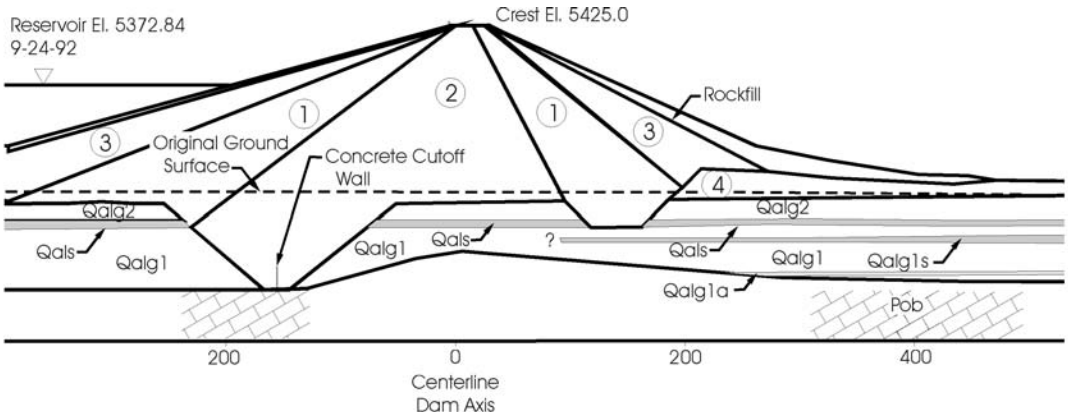

Figure 1. Cross section of Deer Creek Dam and Foundation (Existing Condition)

Site Geology

Bedrock below Deer Creek Dam is a sequence of alternating limestone and sandstone strata, and forms a U-shaped valley with steeply sloping sides and a wide bottom. Alluvial foundation deposits up to about 80 feet thick fill the valley floor. Many of the foundation strata extend over large areas. Four major units have been identified in the alluvial foundation. (See Figure 1.) The uppermost unit is shallow alluvium (Qalg2) consisting of non-plastic fines, sand, gravel, and cobbles. Below the Qalg2 is a thin, finer layer (Qals) consisting of plastic and non-plastic fines, sand, and some gravel; it was concern about this material that prompted construction of the auxiliary foundation trench. The Qals overlies the denser deep alluvium (Qalg1), which consists of non-plastic fines, sand, gravel, and cobbles. Within Qalg1 is a thin, less dense sub-layer (Qalg1s), which contains greater proportions of mostly silty fines and sand, and smaller amounts of gravel. Along the right abutment boundary of the valley are Quaternary terrace deposits (Qt) and alluvial fan deposits (Qaf). The spillway is founded on the latter. Table 1 below summarizes the foundation stratigraphy, including age, thickness and mean index properties for each unit. When developing the means in the table, samples with less than 50 percent recovery (about 6 percent of the full data set) were excluded, out of concern that they may not be representative of the sampled interval. The sampling method, which controls the sample size and the maximum particle size collected, could affect the statistics.

On average, samples from the Qalg1s contain roughly equal proportions of gravel, sand, and fines. Only 22 4 out of 18 samples displayed measurable plasticity. Of those that did, the mean LL was about 26 percent, and the mean PI was about 12 percent. Although the mean PI is 4 percentage points higher than that of the Qals, the difference probably is not statistically significant, as the set of Qalg1s samples with measurable plasticity is small. The mean clay content is approximately 10 percent. From these results, the finer fraction of the Qalg1s would be expected to be non-plastic more often than plastic. The Qalg2 (shallow alluvium) and Qalg1 (deep alluvium) have similar statistics. On average, the samples consisted 50 to 55 percent gravel, about 35 percent sand, and 10 to 15 percent fines. The finer fractions of these two units are non-plastic.

Table 1 – Stratigraphy of Deer Creek Dam's foundation

Geologic Unit	Age	Thickness and Properties (mean values)
Talus (Qt)	Quaternary	Up to 60 feet thick
Alluvial Fan (Qaf)	Quaternary	Up to 100 feet thick 40% gravel, 31% sand, 17% silt, 12% clay.
Shallow Alluvium (Qalg2)	Quaternary	From 15 to 25 feet thick 49% gravel, 38% sand, 9% silt, 4% clay
Clayey layer (Qals)	Quaternary	From 3 to 12 feet thick. 25% gravel, 34% sand, 25% silt, 16% clay, LL = 26, PI = 7%;
Deep Alluvium (Qalg1)	Quaternary	From 20 to 50 feet thick. 55% gravel, 33% sand, 8% silt, 4% clay.
Deep Alluvium Sand/Silt subunit (Qalg1s)	Quaternary	From 3 to 10 feet thick. 35% gravel, 35% sand, 20% silt, 10% clay, LL = 26, PI = 12
Oquirrh Formation and Bridal Veil Falls Member	Pennsylvanian – Permian	Up to 25,000 feet thick. Bedrock

The Need for Modification of Deer Creek Dam

In 1990, the U.S. Bureau of Reclamation recognized the potential for liquefaction to occur in the dam's foundation soils during a large magnitude earthquake, and result in instability or large dynamic deformation of the embankment. A field investigation program was conducted at the downstream toe in 1992, after which analyses indicated that the upper 20 feet of the foundation (mostly Qalg2) could become liquefied or at least lose significant amounts of strength due to the maximum credible earthquake (MCE).

Additional investigation programs were undertaken in 1998, 1999, 2000, and 2002. These included exploratory drilling, standard penetration testing (SPT), Becker penetration testing (BPT), cross-hole shear-wave velocity measurements, sampling and laboratory testing, and installation of piezometers. Liquefaction potential of the foundation and the dynamic stability of the dam were evaluated using the information acquired from these investigations. Engineering evaluations indicated four potentially liquefiable units in the foundation beneath and downstream of Deer Creek Dam. These units, described earlier in general terms, are the upper Qalg2, Qals, Qalg1s, and Qaf. It was determined that any one of the four could experience significant strength loss (to very low residual values) due to the design earthquake, resulting in instability of the dam.

In 1993, conceptual design work was begun on a new alignment of U.S. Highway 189 by the Utah Department of Transportation (UDOT). UDOT plans to widen the highway to four lanes and relocate it so that it will cross over the downstream slope of Deer Creek Dam, rather than along the crest as it does now. This will require construction of a large highway fill on the dam's downstream slope, and a bridge to span the spillway. Once construction is started, access to the foundation of Deer Creek Dam for additional testing or treatment will be lost, so it was necessary to proceed with any foundation work required without further delay. The highway construction work is now planned to begin shortly after completion of foundation treatment.

In late 2001, Reclamation conducted a probabilistic risk analysis to assess the danger that Deer Creek Dam poses to the public because of potential for earthquake-induced failure. The study focused on failure by embankment instability due to liquefaction of the lowest of the potentially liquefiable units (Qalg1s). The other layers considered potentially liquefiable were not explicitly included, because nearly any treatment required for Qalg1s would treat the overlying layers at little or no additional cost. Also, the auxiliary cutoff trench interrupts the other potentially liquefiable layers, except near the right side of the valley. The auxiliary trench did not extend down to the Qalg1s layer. The conclusion of the risk analysis was that the risk to the public downstream of the dam exceeded Reclamation's guideline levels requiring action to be taken to reduce the risk.

Following the 2001 risk analysis, the Bureau of Reclamation continued to investigate the foundation of Deer Creek Dam, collect and test soil samples, evaluate data, and perform post-liquefaction stability and deformation analyses. These investigations confirmed that Deer Creek Dam required corrective action to improve the embankment foundation and provide adequate stability under seismic loading.

ANALYSES AND EVALUATIONS

Seismic Setting

Deer Creek Dam and Reservoir are located about 15 miles east of the active Wasatch fault zone. The dam lies within the tectonically active Intermountain Seismic Belt (ISB). Based on previous investigations, a maximum magnitude of M 6.5 was assumed for the ISB areal source zone.

Seismotectonic studies for the Deer Creek Dam included development of hazard curves and ground-motion time histories. The probabilistic hazard study allows for explicit inclusion of ranges of possible interpretations for all the input components used to develop the hazard model; these include slip rates and magnitudes for fault sources, and attenuation of ground motions with distance from the fault. The results of the probabilistic seismic hazard analysis are presented as hazard curves showing ground motion parameters as a function of the annual probability of exceedance. Controlling earthquakes were estimated based on deaggregation of the hazard by magnitude and distance. Uniform hazard spectra (UHS) were prepared for annual probabilities of exceedence of $1x10^{-4}$ and $2x10^{-5}$ (recurrence intervals of 10,000 and 50,000 years). These target UHSs were used to select spectrum-compatible acceleration time histories for use in the dynamic analyses of the dam.

The closest segments of the Wasatch fault, the Round Valley fault, the Strawberry fault, and the ISB areal source zone control the hazard for peak horizontal acceleration (PHA), and have response spectral periods with energy at frequencies near the natural period of the dam/foundation system (~0.75 sec). The PHA corresponding to $1x10^{-4}$ annual probability of exceedence, was estimated as 0.4 g as measured on a bedrock outcrop. For $2x10^{-5}$ annual probability of exceedence, the estimate is 0.6 g. Controlling magnitudes and distances and response spectral values of interest are summarized in Table 2. For PHA, significant contributions to the hazard occur from moderate magnitude events associated with the ISB areal zone and Round Valley faults. However, for 0.75-second spectral acceleration (SA), the hazard is dominated by large magnitude events occurring on the Wasatch fault.

Table 2 – Controlling Earthquakes

Ground Motion Parameter	$1x10^{-4}$ Annual Frequency of Exceedence (10,000-Year Return Period)		$2x10^{-5}$ Annual Frequency of Exceedence (50,000-Year Return Period)	
	M	Distance, km	M	Distance, km
PHA –>	6.7	11.5	6.6	8.5
0.75-second SA –>	7.0	15	7.0	12.5

Based on the values in table 2, it was deemed appropriate to analyze the dam with time histories that are consistent with a magnitude of 7.0; the dam response is expected to be most sensitive to motions with periods on the order of 0.75 seconds. Two three-component acceleration-time histories were prepared for each annual frequency of exceedence.

Site Response

The dynamic response of the dam and foundation to seismic loading was modeled by inputting the time histories into the equivalent-linear computer program, SHAKE96, in order to estimate the peak cyclic shear stress within the layers of concern for liquefaction. In SHAKE96 the soil profile is modeled as a series of shear beams of varying stiffness. Analyses were performed using four different acceleration time histories (two for each of the annual probabilities exceedence discussed above). The shear stress values so obtained were used to estimate the cyclic stress ratio, CSR. The CSR is the ratio of the peak shear stress from the "average" cycle of earthquake loading to the effective stress existing immediately prior to the earthquake. With several empirical adjustments made for earthquake duration and other conditions, the CSR is used in the well-known Seed-Lee-Idriss simplified procedure for assessing liquefaction potential [Seed 1983], as well as in empirical models for estimating the probability of liquefaction for use in the probabilistic risk analysis.

Measured shear-wave velocities were used in the response analyses. The shear-wave velocity data generally indicated that the units are stiff, not loose or soft as was indicated by the SPT and BPT results. With the higher, measured shear-wave velocities, SHAKE96 *may have* overestimated the cyclic shear stresses and underestimated the shear strains. The calculated CSRs were, however, very high, and it did not appear that changes in the stiffness of certain layers would alter the general indication of severe loading under the larger earthquakes used in the analyses.

Following the Seed-Lee-Idriss procedure, the average cyclic shear stress for calculating the CSR was estimated as 65 percent of the peak value from the SHAKE96 analysis. The "raw" CSR so determined was then adjusted by empirical factors that account for the effect of earthquake magnitude (as a proxy for duration), the very high overburden stress under the embankment, and so on, as described by Seed [1983].

Liquefaction Potential

For assessing liquefaction potential of the foundation materials at Deer Creek Dam, standard penetration test (SPT) and Becker penetration test (BPT) blowcount data were used as an index of the soil's resistance to dynamic loading. Moderately low blow counts (on the order of 10 to 15) appeared in almost every BPT sounding at elevations corresponding to each potentially liquefiable layer, indicating continuity of loose material over a large area. Few useable SPT data were available in the potentially liquefiable units because of gravel content. There, it was necessary to rely primarily on the BPT data, using the SPT data to provide a general confirmation that the BPT results were providing credible information at the site. BPT data were converted to equivalent N_{1-60} values using the methods of Harder and Seed [1986], and/or Sy and Campanella [1994], the latter of which incorporates measurements of rod friction to adjust for its effect on blowcount. In spite of the dense overburden above and below the potentially liquefiable layers in question, there was reasonable agreement between the results of the two methods. Liquefaction potential of the identified low strength layers was concluded to be high for the 10,000-year earthquake.

Shear-wave velocity data were also used as a means to help determine location and continuity of the low density layers within the alluvium foundation. In general, the shear-wave velocities were above the "threshold" limits above which liquefaction could be triggered (except for shear-wave velocities of the Qalg2 layer, from testing conducted in 1992). This may have been due, at least in part, to the insensitivity of the measurements to thin layers of soft material. The measured velocities did, however, exhibit the same high-low trends as the blowcounts. These data helped to identify the presence and limits of the four potentially liquefiable layers

Liquefaction potential in Deer Creek Dam's foundation was determined using empirical curves relating liquefaction resistance to SPT blowcount. [Seed, 1983]

Embankment Stability Analyses

Post-liquefaction static stability analyses were performed to assess the performance of the dam embankment bearing on liquefied foundation layers. The entire thickness of the Qals and the Qalg1s, the top 10 to 20 feet of the Qalg2, and saturated Qaf were all assumed to liquefy. All other foundation soils and the embankment fill were assumed not to liquefy, based on *in situ* data and compaction records.

The stability analyses were performed using Spencer's method coded in the computer programs SLOPE/W and UTEXAS3. Spencer's method is a limit-equilibrium approach that considers force and moment equilibrium equations of planar statics. It assumes that the inclination of the interslice forces is the same for all slices, and the programs iterate to find the combination of force inclination and safety factor that satisfies static equilibrium.

Eight cross sections through the dam from the left to right abutments and along the spillway were analyzed. The critical failure surfaces (i.e., those with the lowest factors of safety) consisted of large active wedges that included the crest of the dam, a long "neutral block" sliding on a base of liquefied alluvium, and a small passive wedge at the downstream. The calculated factors of safety were below 1.0 with a variety of strength assumptions for the liquefied materials. Because the active wedges would include the crest and large amount of the upstream slope, it was judged that instability would cause large deformations that would allow the embankment to be overtopped by the reservoir.

The judgments based on post-liquefaction stability analyses were verified using the dynamic run-out method (DRUM) of Tan *et al* [2000]. This procedure is used to estimate the amount of movement and reduction in crest elevation resulting from instability. A series of stability calculations are made on deformed cross sections to estimate the net driving force on the slide mass, for use in calculating the expected acceleration and distance traveled. The DRUM analysis used UTEXAS3 for the stability calculations. DRUM indicated that if liquefaction of the Qalg1s unit were to occur, the unmodified dam would be unstable and very large deformations would occur, unless the residual undrained shear strength of liquefied Qalg1s were much higher than expected, on the order of 3000 lb/ft^2. This would only occur if the liquefied material were to become strongly dilatant at large shear strains.

DESIGN ALTERNATIVES

Alternatives Developed

Several remedial concepts were considered for the Deer Creek Dam modification, including the following:

1. Dynamic compaction: densification in-place
2. Jet grouting: solidification in-place or cementing in-place
3. Compaction grouting: densification in-place
4. Replacement: excavation and replacement with compacted fill material
5. Shear pins: control of deformation by reinforcement
6. Widening of the embankment crest: maintaining freeboard should instability occur
7. Dewatering with wells: lowering the degree of saturation and reducing the cyclic stress ratio from a given earthquake
8. Gravel drains: relief of earthquake-generated excess pore water pressure
9. Stone columns: densification in-place

All of the alternatives considered combine foundation treatment with the fill to be placed on the downstream slope as part of the proposed relocation of U.S. Highway 189, acting as a berm to buttress the downstream slope. Some of the alternatives were determined to be well suited for certain areas of the foundation, but not for others; others were rejected as completely impractical. In some cases two or more alternatives were combined to attain full coverage of the treatment area for the purpose of the evaluation. (The upstream slope was found to be stable already because the upstream cutoff trench acts as foundation treatment under it.)

Evaluation of the different treatment concepts involved developing conceptual designs including required material strengths, areal extent and depth, and a minimum acceptable safety factor in the limit-equilibrium analyses. Each alternative considered meets the basic requirement of treatment to the bottom of the Qalg1s, the lowest liquefiable layer recognized in the valley, although the level of confidence that could be placed in each varied considerably. Given the location of the dam upstream of the city of Provo, a very high degree of confidence is required. This favors methods that allow easy verification of successful treatment.

Treatment below the Qalg1s was considered, but was determined to be unnecessary because of the absence of evidence for liquefiable materials below the Qalg1s.

Selected Alternative

The alternative selected for modification construction consists of treating foundation soils in the downstream valley area by excavation through the potentially liquefiable material at the downstream toe of the dam, and replacement of it with rolled fill to form a shear key. There is the possibility of adding jet grouting adjacent to the upper part of the spillway where excavation is precluded by space limitations. The decision on jet grouting will be made on the basis of direct observation and testing of the foundation materials exposed in the foundation excavation. The embankment for the relocated highway, and additional fill outside the designed highway footprint will act as a berm, increasing the effective overburden stress within the key, and buttressing the downstream slope of the embankment. To maximize the area of treatment by excavation and replacement, two reinforced tie-back walls are needed to support excavations adjacent to the lower part of the spillway and downstream toe in order to permit a larger area for the invert of the excavation. Elsewhere, there will be unsupported side slopes of approximately 1.5:1. Extensive dewatering is required to maintain stability of the dam and cut slopes, and to permit excavation and replacement "in the dry." Figure 2 shows the approximate limits of the deep excavation for removing the loose alluvium and replacing it with compacted fill, and the location of the tie-back walls. Figure 3 shows the final configuration of the berm and highway embankment in plan view.

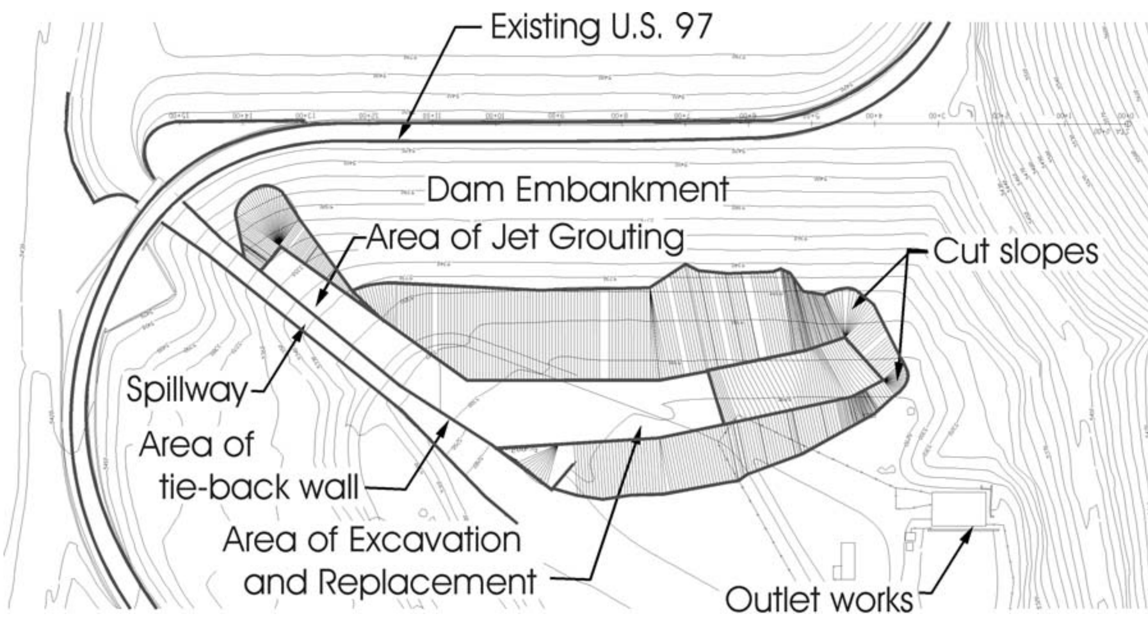

Figure 2. Excavation for Selected Design Alternative

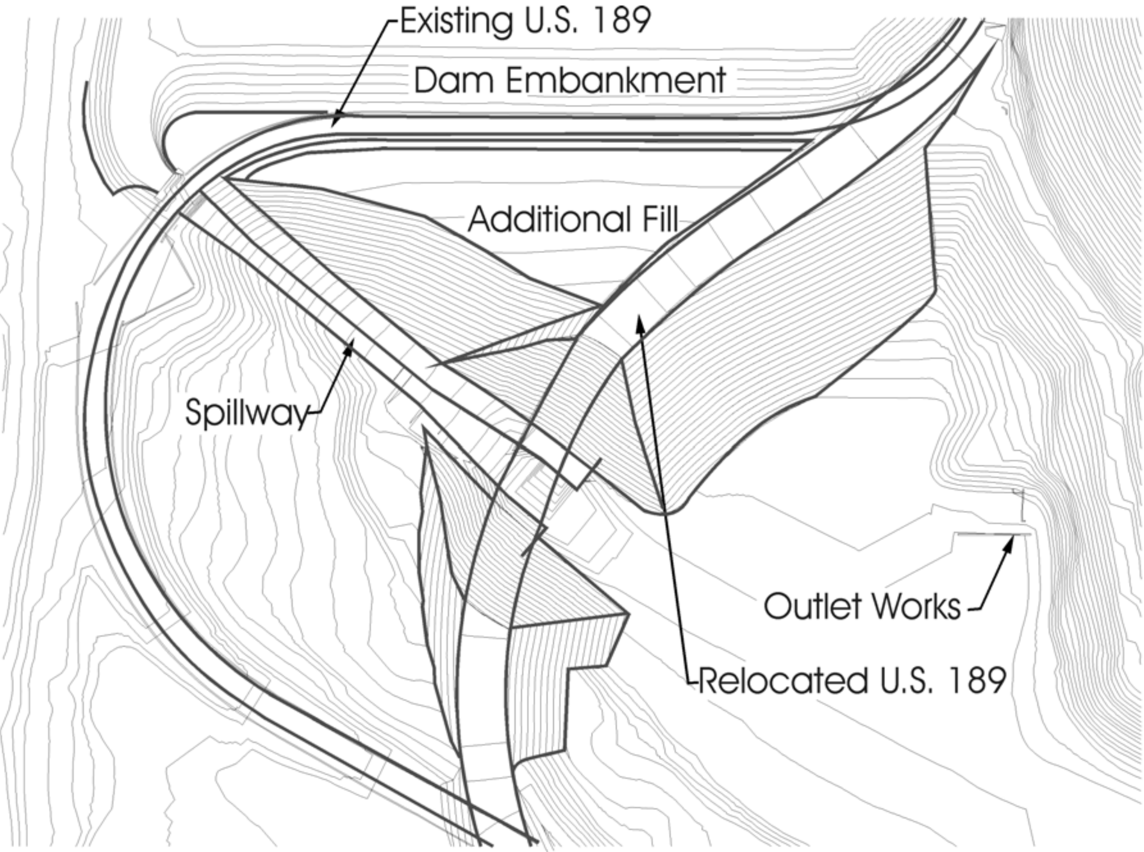

Figure 3. Plan view of Deer Creek Dam with proposed Highway 189 alignment

To ensure adequate embedment of the compacted fill into the firm foundation soils, the foundation treatment will extend approximately 5 feet below the Qalg1s layer. The potentially liquefiable units above the Qalg1s will be removed and replaced in the process of excavating to the Qalg1s. The compacted backfill will consist of select gravelly soils from required excavation and from nearby borrow pits.

For analysis, the treated zone was assigned a drained strength of 35 degrees and no cohesion. The required width and depth of treatment were determined by performing static slope-stability analyses using the program SLOPE/W to show that a minimum factor of safety of 1.3 had been attained, with Qalg1s remaining in the foundation assumed to be liquefied.

To prevent foundation or core material from piping into the shear key backfill in places where high seepage gradients could develop, the compacted backfill will include chimney filters and filter blankets. There are also drains to avoid having the key inhibit seepage and increase piezometric levels under the embankment. These might also provide some pore-pressure relief for loose foundation materials during earthquake loading.

Preliminary analyses indicate that excavation and replacement to create a key of strong material within the liquefied foundation, combined with the berm to provide confining stress on the key fill and buttress the embankment slope, would provide adequate post-earthquake stability for the dam embankment without the jet grouting. Only a fairly narrow untreated area would remain between the key and the right abutment. (Sections that actually have the key will have minimum factors of safety of 1.3.) Depending on the properties of materials exposed in the excavation (continuity of layering, density, etc.), a decision will be made about jet grouting along the upper part of the spillway to help stabilize the portion of the dam between the excavated key and the right abutment. If jet grouting is pursued after completion of this portion of the modification, the jet grout holes could be deep as 70 feet, through alluvium with about 30 percent gravel and 10 percent cobbles and boulders. Because of the potential for cobbles and boulders to block the jet of water and cement, spacing of the grout columns may need to be fairly close in order to allow the grout columns to overlap and form massive blocks that act as shear walls, rather than slender columns that would have to resist lateral loads by bending moment capacity. The treatment may have to be continuous through all of the foundation because of the difficulty in determining the elevation boundaries between units.

Although cost should be a consideration in the selection of the preferred concept, this approach was selected over some others with lower costs because of the need to provide extremely reliable performance. The annual probability of severe earthquake loading is relatively high at Deer Creek Dam, as is the number of people that would be affected by a breach. There must not be any planes of weakness to act as weak links in a chain, as might occur at depth with dynamic compaction from the surface, or with compaction grouting if there are fine-grained layers that do not give up their pore water easily. Verification of construction is much easier for conventional rolled fill than it is for *in situ* treatments at large depths. With the dewatering option, there were concerns about perched water, and fine-grained materials that simply would not drain, due to high air-entry pressure. Silty foundation materials are likely to be too impervious for dissipation of excess pore pressure to occur through gravel drains fast enough to prevent major losses of strength. Brittleness is a serious concern for cementing methods if they are subjected to very high peak accelerations and very high loads, unlike compacted soils that can deform plastically. (This will, of course, have to be considered in the design of jet grouting along the spillway, if it determined to be necessary.) Some of the concepts, such as shear pins and drains to dissipate excess pore pressure during the course of an earthquake, require very complex analyses for verification of a proposed design. The results of those analyses can be quite sensitive to input assumptions. In order provide the high level of confidence required in this situation, those concepts may require very conservative (and therefore expensive) designs.

The rolled earthfill key and berm selected for modification of Deer Creek Dam provide a ductile foundation and embankment that can undergo considerable deformation with little or no loss of strength. Conventional construction methods and construction verification permit a high degree of confidence that the desired results have been achieved at the end of the project. The analysis to verify adequacy of design is primarily based on limit equilibrium stability analyses which are well understood by the profession, as are the strength properties of compacted fill materials. The selected alternative for treatment of the foundation of Deer Creek Dam, being constructed as this paper is being written, is therefore the most appropriate method. The U.S. Bureau of Reclamation is confident that this design provides the highest degree of safety than can reasonably be attained.

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