# LESSONS FROM THE LIVES OF TWO DAMS

The Fourth Victor de Mello Lecture

by

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**Abstract:** Many embankment dams completed during the first six decades of the 20th century have been found deficient relative their ability to resist currently anticipated levels of seismic shaking and probable maximum flood. In this Fourth Victor de Mello Lecture, two recent case histories are described. One is a hydraulic fill structure completed in 1920 that is founded on alluvial material, some zones of which are susceptible to liquefaction. The other is a zoned earthfill dam completed in 1956 that is founded over a channel filled with loose, uncompacted, hydraulically placed tailings from gold mining operations. Each dam has been upgraded in phases over periods of several decades using different strategies and ground improvement technologies to improve stability and reduce failure risks. Several take away lessons from these experiences concerning current risk mitigation strategies, the importance of correct soil and site characterization, and implementation and effectiveness of different ground stabilization and improvement methods are presented.

**Keywords:** earthquakes, embankment dams, liquefaction, risk mitigation, site characterization, soil improvement

## 1. Introduction

Dr. Victor de Mello was one of the "Giants of Geotechnics" of the 20th century who leaves a legacy of exceptional contributions from his major accomplishments as a practicing, civil and geotechnical engineer, as a teacher both inside and outside the classroom, as a researcher, as a leader in his profession, and as a dynamic, yet philosophical and congenial colleague and friend. I am greatly honored by the invitation to deliver this lecture in celebration of his life and professional contributions, while at the same time daunted and humbled by the challenge of contributing something worthy of the honor.

Victor de Mello devoted his 1977 Rankine Lecture to considerations in embankment dam design (de Mello, 1977), with special focus on filters, drainage and seepage control, as well as stability issues. He discussed factors of safety and their meaning, introducing considerations of probability and variability that, while relatively new then, are now central to assessment of dam safety. I have also chosen to address embankment dams, but in this lecture, the focus is on dealing with problems that arise in existing dams that resulting from age and from risks caused by extreme events, especially earthquakes and floods that were incompletely understood and accounted for at the time of original construction many decades ago.

Many large embankment dams were constructed in the U.S.A. during the first six decades of the 20th Century. The M 9.2 Great Alaska Earthquake and the M 7.5 Niigata Earthquake in Japan, both in 1964, focused attention on soil liquefaction, and the near catastrophic failure of the Lower San Fernando Dam in southern California in the 1971 M 6.6 San Fernando Earthquake led to reevaluation of the seismic vulnerability of many other dams. The Maximum Credible Earthquakes, Maximum Probable Floods, and populations at risk have increased significantly at many sites. Risk analyses have led to unacceptably high potential consequences requiring implementation of mitigation measures at many dams. Two of these dams are described in this paper.

San Pablo Dam, near Oakland, California and completed in 1921, is a hydraulic fill structure founded on alluvial deposits. Mormon Island Auxiliary Dam (MIAD), near Sacramento, California, is a compacted fill embankment founded on hydraulically deposited dredger tailings resulting from gold mining operations; completed in 1956. Each dam was subsequently deemed unsafe under the anticipated seismic loading conditions. Several modifications have been made to each dam to improve resistance to anticipated earthquake loadings and updated flood risks. These modifications took place from 1967 to 2010 at San Pablo Dam and have extended from the late 1980's to a planned final completion in 2016 at the Mormon Island Auxiliary Dam. Some conclusions and lessons learned about the development of geotechnical earthquake engineering for dams, seismic remediation strategies, the importance of proper site and material characterization, and the advantages and limitations of some ground improvement methods can be derived from these two case histories.

## 2. San Pablo Dam

This dam is a 53.3m (170ft) high, 38.1m (125ft) crest width, 366m (1200ft) long hydraulic fill dam founded on alluvial sediments that contain some zones that are susceptible to liquefaction. The hydraulic fill material used for construction of the embankment consists of weathered sandstone and shale that was obtained from the East Bay Hills near Oakland, California. The site is located within a few kilometers of several major faults, and it is estimated that there is a 62 percent probability of one or more earthquakes of magnitude 6.7 or greater during the period 2003 to 2032.

Photos of the construction of the dam illustrating the excavation for the core trench and placement of the hydraulic fill embankment materials are shown in Fig. 1. The original hydraulic fill embankment construction was completed in 1921. Tests on a few sandy samples of the embankment materials in the 1960's and 1970's indicated potentially liquefiable behavior. Evidently this led to the assumption of a liquefiable embankment because it was a hydraulic fill, and hydraulic fills of cohesionless materials are invariably of low relative density and high liquefaction potential unless densified following deposition.

A small downstream buttress fill was constructed in 1967 to improve seismic stability. Then, following the 1971 San Fernando Earthquake in southern California, a much larger compacted fill buttress extending to bedrock was completed in 1979. Construction of this buttress required draining the reservoir, with its attendant depletion of the area's water supply capacity and disruption of recreational use of the reservoir. A cross section of the maximum embankment section with both the 1967 downstream stabilizing berm and the much larger 1979 upstream berm in place is shown in Fig. 2.

A new seismic stability evaluation was completed in 2004 assuming a liquefiable embankment and a M7.25 earthquake on the Hayward Fault which passes just 3 km southwest of the dam. The results indicated the potential for vertical slumping of up to 10.7 m and overtopping of the dam by the impounded reservoir. To provide additional freeboard for the short term, the reservoir level was lowered by 6 m. Consideration was given to completely rebuilding the dam; however, this would have required again draining the reservoir, an action that had considerable opposition. Instead, an in-place alternative was chosen that consisted of Cement Deep Soil Mixing (CDSM) to bedrock at depths of up to 36.5m in the downstream foundation alluvial soils and construction of a large buttress fill on the downstream embankment slope, shown as Initial Remedial Concept in Fig. 3, from Yiadom and Roussel (2012).

An extensive new field investigation program was then completed that included many cone penetration tests (CPT) and borings into both the embankment and foundation materials. The results of these field investigations and laboratory tests on representative samples are described in detail by Moriwaki, et al (2008). They described the sampled hydraulically placed material as consistently very clayey, very “lumpy”, and over-consolidated. Of special interest is Fig. 4, taken from that reference, which shows CPT and plasticity data for the hydraulic fill embankment shell materials, zones (1) and (2) in Fig. 2. This data shows clearly that this material is not susceptible to liquefaction, as had been assumed for the previous evaluations of San Pablo Dam and for the Initial Remedial Concept in Fig. 3. In retrospect this finding is not surprising given that the material used for the hydraulic fill came from the colluvial slopes of the surrounding hills where the soil type is known to be largely silty and clayey.

This re-characterization of the hydraulic fill embankment material from liquefiable to non-liquefiable, fine-grained soil enabled significant reductions in the required sizes of the 2010 buttress and CDSM block, as may be seen in Fig. 3 by comparing the Final Remediation Design with the Initial Remedial Concept. Seismic deformation analyses of the maximum composite dam cross-section were done using computer program FLAC with input ground motions that had a peak acceleration of 0.98g and associated spectral content consistent with the occurrence of a M7 earthquake on the nearby Hayward Fault. The results indicated maximum seismically induced permanent displacements in the range of 0.3 to 0.6m (1 to 2ft) (Kirby, et al, 2010), which were considered well within the range for acceptable performance.

The reduced volumes of required CDSM and the new downstream buttress fill realized a cost saving of about US $40 million, a very significant amount for a project with a construction cost of about $60 million. Kirby, et al, (2010) describe the design of the CDSM foundation block, the construction process, and the methods used for quality control of the material. Another cost saving feature was that the spoils from the deep cement mixing could be incorporated into the new downstream buttress. An aerial photo (Google, 2012) of the project after completion is shown in Fig. 5. This project is an excellent illustration of the importance of correct soil identification and classification prior to analysis and design of risk mitigation strategies for existing dams.

## 3. Mormon Island Auxiliary Dam

The Mormon Island Auxiliary Dam (MIAD) forms a part of the Folsom Project located on the American River about 32 km (20 miles) northeast of Sacramento, California. This project provides water supply, hydroelectric power, and flood protection for a large metropolitan area. It consists of a concrete main dam, right and left wing dams, the zoned and rolled earthfill MIAD, and eight earthfill dikes that are needed to contain Folsom Lake. A plan showing these features is given in Fig. 6. Fig. 7 is an aerial photo showing MIAD as it appears at present (Google Earth, 2013).

MIAD was constructed by the U.S. Army Corps of Engineers and completed in 1953, after which operation and maintenance activities have been the responsibility of the U.S. Bureau of Reclamation. The dam is comprised of a thin central impervious core bounded by fine and coarse filter transition zones extending to weathered metamorphic bedrock and compacted earthfill shells both upstream and downstream, as shown in Fig. 8. The embankment shells are composed of alluvium dredged from the site. The design width of the dam crest is 7.77m (25.5 ft), the upstream slope varies from 3H:1V to 4.5H:1V, the downstream slope varies from 2.5H:1V to 3.5H:1V. The structural height of the embankment is 50.3m (165 ft), and the dam is 1470 m (4820 ft) long.

The alluvial foundation materials, consisting of varying amounts of gravels, sands, silts and clays, were dredged and re-dredged to depths of very near the bedrock along an approximately 300 m (900 ft) long strip adjacent to the present location of the left abutment of the dam as a part of gold mining operations during the latter part of the 19th Century. As a result of this dredging and re-deposition, these materials were left in a loose state. The upstream and downstream toes of the embankment are underlain by an approximately 20 m (60 ft) thick layer of loose tailings for this 300 m length of the dam. Evaluation of the seismic safety of the dam during the 1980s indicated that liquefaction of these tailings was likely during the design earthquake. A series of dam and foundation modifications for risk mitigation have been undertaken since then, and these activities are described below.

The results of field and laboratory tests were used to develop the distributions of normalized SPT (Standard Penetration Test) values of $(N_1)_{60}$ in the foundation that are shown in Fig. 9. These values were used to evaluate factors of safety against liquefaction and for the establishment of parameters needed for dynamic deformation analyses. Ground improvement was implemented beneath both the upstream and downstream embankment toe areas to mitigate the liquefaction risk and to limit dynamic displacements.

Owing to severe drought conditions in California in the late 1980s the reservoir level was low, and this made it possible to undertake deep dynamic compaction (DDC) in the dry from the upstream embankment. As shown in Fig. 10, an access excavation was made to provide level ground for carrying out the work. A block of densified soil was formed by repeated dropping of a 2.0 m (6.5 ft) diameter steel drop weight of 31.75 tonnes (35 tons) over a 244 by 46 m (800 by 150 ft) treatment area. The drop height of 32.9 m (108 ft) corresponded to a free fall distance of 30 m (98.4 ft). Three coverages of the area were made, with 30 drops at 15.2 m (50 ft) center to center drop point spacing for the first, or primary, coverage, 30 drops at points splitting the primary coverage spacing for the second coverage, and 15 drops at points splitting the secondary coverage for the third coverage. Finally surface "ironing" was accomplished using 2 weight drops from a height of 10m (30 ft) at adjacent points to cover the entire area. Finally, the access excavation was re-filled, and a post treatment berm was placed on the upstream slope, as shown schematically in Fig. 10. The photograph in Fig. 7 shows this berm extending from the upstream slope into the reservoir following subsequent increase in the reservoir water level.

The effectiveness of the DDC can be seen in Fig. 11, which shows values of Becker Penetration Test (BPT) resistance, converted to equivalent SPT values of $(N_1)_{60}$, as a function of elevation. The BPT measures the number of blows to drive a 168 mm (6.6 in) outside diameter double-walled casing a distance of 0.3 m using a double-acting diesel pile driving hammer. This test is useful in soils containing gravel and cobbles, where difficulties are often encountered when using the SPT. The decrease in penetration resistance with depth in Fig. 11 is characteristic of the DDC method for soil improvement, with an effective treatment depth of about 10 m (35 ft) being about the maximum attainable when heavy weights are used in soils of no to low plasticity. As shown in Fig. 11, the DDC was unable to densify the soil within the treatment zone over the full depth to the underlying bedrock.

Following an extensive testing program for determination of the most suitable methods for in-place improvement of the downstream foundation material, the system shown schematically in Fig. 12 was designed, with construction completed during 1993-1994. The improvement zone was a strip along the downstream toe of the dam that is 900 ft (275m) long by 200 ft (61m) wide in plan, in the area labeled "Downstream improvement" in Fig. 7. Excavation into the existing downstream embankment was required in order to develop a level working platform, Fig. 13, for installation of 1.2 m (4 ft) diameter wet, bottom feed, vibro-replacement stone columns and the upstream and downstream drainage zones that are composed of 250 mm diameter gravel columns on 1.0 m centers installed using a vibro-pipe. It is important to note that excavation into the downstream slopes of embankment dams is not without risk owing to reduced seepage paths and lower factors of safety against stability failure while the excavation is open, both of which must be accounted for in assessing risk during construction.

The primary purpose of the stone columns was to densify the loose liquefiable dredged alluvium foundation material so that it would not liquefy under the design earthquake. The upstream and downstream drainage zones are intended to intercept pore pressure plumes migrating towards the stone column treated zone in the event liquefaction develops in the adjacent untreated dredged alluvium during an earthquake. Prevention of pore pressure increases in the stone column zone is important for maintenance of shear strength during and after shaking.

The profile through the dredge spoils in the stone column area indicated that the upper 4.5 to 6 m contains coarse sand to cobble size material, and the lower 3 to 6 m is silty sand to silty clay with 10 to 77 percent fines, with an average fines content of 30 percent (Allen, et al., 1995). The pre- and post-treatment penetration resistance values shown in Fig. 14 are consistent with these conditions. A cross-section of the embankment showing all the modifications and ground improvement work through 1994 is shown in Fig. 15.

Subsequent investigations and risk analyses were initiated in 2001. These studies took into account that a potentially liquefiable zone remains beneath the upstream block of material that had been densified by deep dynamic compaction. After extensive re-analyses of available data and further in-situ testing, it was concluded also that the necessary downstream foundation densification was not achieved in the lower part of the soil profile during installation of the downstream stone columns, owing primarily to the fine-grained nature of the material. Furthermore, contamination of the stone columns by the fines resulted in lower column strength than had been anticipated and impeded the drainage capability as well. These re-analyses led to the conclusion that the most critical seismic failure mode would result from liquefaction of the upstream and downstream foundation materials leading to significant deformations of the dam in the downstream direction, with vertical displacements sufficient to result in overtopping for high reservoir levels.

A risk analysis done in 2007 showed that both the Annual Failure Probability and the Annualized Life Loss were above the Bureau of Reclamation guideline values. Corrective Action Studies for Seismic and Static Risk Reduction completed in 2010 led to a design that included a concrete key block with compacted soil above in the downstream stone column area and a compacted soil buttress fill over the downstream embankment slope that includes underlying filters and drains. No further remedial work is planned for mitigation of the upstream liquefaction risk; i.e., the key block and buttress design is considered sufficiently robust that if any deformations develop in the upstream direction, the remaining downstream core, filters, and berm will still be adequate to maintain stability and the necessary freeboard.

An additional point of interest is that jet grouting was proposed initially for construction of the key block. However, the results of an extensive test program indicated that the required continuity and strength of soilcrete could not be obtained owing to the presence of gravel, cobbles, and stone columns in the treatment zone. Jet grouting was ultimately judged to be technically and economically unfeasible for this purpose at this site.

As a result, construction of the key block was accomplished within a series of contiguous cells, each being formed within an open, braced excavation. A photograph of this work in progress is shown in Fig. 16.

The final configuration of MIAD will be as shown schematically in Fig. 17. The key block construction was completed in February 2013, and the buttress fill is scheduled for completion in 2016.

## 4. Take-away lessons from these two projects

Several lessons and conclusions can be drawn from the San Pablo Dam and Mormon Island Auxiliary Dam seismic remediation work, and other projects within the author's experience, that relate to geotechnical engineering of existing embankment dams, strategies for mitigation of risk, site and material characterization, and ground improvement methods and their applicability.

<u>Geotechnical Engineering and Failure Risk Mitigation for Existing Embankment Dams</u>

Most existing large embankment dams in the U.S. were constructed prior to the 1960s. Little attention was paid to seismic issues in their design. However, large earthquakes in Alaska and Japan in 1964, and the failure of the Lower San Fernando Dam in California in 1971 triggered assessment of the seismic resistance of major dams. Many dams have required modifications to protect against cracking and excessive deformations in the event of future earthquakes. In addition, seismicity reevaluations and redefinitions of maximum credible earthquake at a site, along with increases in the probable maximum flood and the populations at risk have resulted in increased demands.

Potential failure mode analyses and formal risk analyses are now widely used for determining the urgency of undertaking risk mitigation activities, evaluation of the effectiveness of various types of remediation to be employed, and for prioritizing projects within available time and budget. Risk management guidelines and details of the risk assessment and evaluation process have been developed by several water resources and regulatory agencies. Publications by the U.S. Bureau of Reclamation and by the U.S. Army Corps of Engineers, many of which are available on-line, provide extensive information on these procedures and interpretation of the results.

A reasonable goal is to bring the safety of the dam, as measured by global stability, resistance to deformation sufficient to prevent overtopping resulting from excessive crest settlement, filter protection, safety against cracking, and drainage provisions downstream of the seepage barrier to states that are as safe as would be attainable if the dam were being designed and constructed today.

Remediation strategies for achieving the needed levels of stability, mitigation of liquefaction. controlling deformations under seismic loading, and prevention of overtopping seem to have followed a path over the past 50 years or so from adding a simple buttress fill, to incorporating different types of in-situ ground improvement such as deep dynamic compaction, vibro-compaction, vibro-replacement, and/or compaction piles both upstream and downstream, to a focus on downstream work only.

The downstream only option avoids the need for working over and through water, unless the reservoir level can be drawn down. An upstream embankment failure can be allowed provided the downstream embankment is buttressed sufficiently to prevent excessive loss of freeboard and the upstream failure zone does not encroach on the dam core at a point beneath the reservoir level. Satisfying these conditions must be demonstrated by suitable analyses. A downstream buttress fill overlay above a block of foundation soil treated in-situ, as done for the San Pablo Dam, is simple and reliable. The same is true for the buttress fill over a key block, now under construction at the Mormon Island Auxiliary Dam.

It is imperative to keep in mind, however, that whenever a project involves excavation into the downstream slope; e.g., during the stone column and drain installation, Fig. 13, or into the foundation, as was necessary for the key block construction at MIAD, Fig.16, there may be increased seepage as well as a temporary decrease in factor of safety against stability failure while the excavation is open that must be evaluated and assessed in terms of increased failure risk.

Limit equilibrium and deformation analyses under both static and dynamic loading conditions are necessary. These analyses, if properly carried out, provide critical information about the current condition of the embankment, the potential impacts of different seismic or other new loading conditions, the locations of potential failure surfaces and large deformation zones and patterns, whether the deformations may have detrimental effects on the dam core, filters, and seepage control components, and the effectiveness of different remediation methods and designs in assuring that adequate stability and deformation limits can be achieved.

A number of readily available computer programs is available for the limit equilibrium evaluations; the deformation analyses are usually done using finite difference; e.g., FLAC and/or finite element; e.g. PLAXIS programs. The most critical input parameter for any of these analyses is the shear strength (Duncan, 2013). Determination of the appropriate strength value is often challenging, especially in situations involving liquefaction where knowledge of the post-triggering strength of the liquefied soil is required in order to assess the potential consequences.

## Site and Material Characterization

The validity and reliability of all the analyses, selection of risk mitigation methods, and predictions of future behavior hinge on proper knowledge of the subsurface materials, their boundaries, the groundwater conditions, and how the relevant properties are measured and assigned. Review of available, original geological and geotechnical reports and construction records is essential. Information about past modifications to the dam must be carefully assessed. At the same time, the information about the actual present characteristics of the embankment and foundation soils may not be available or totally correct, as was found to be the case at San Pablo Dam.

Incorrect identification and characterization of materials can lead to significant overestimates or underestimates of both the dam safety and the needed extent, time, and cost of ground improvement.

## Ground Improvement Methods and Their Applications in Embankment Dams

Ground improvement is now a major sub-discipline within geotechnical engineering and geo-construction. There are many methods and materials that can be used to meet a variety of ground improvement and reinforcement applications in embankment dams. A comprehensive description and classification of methods was developed by the ISSMGE Technical Committee on Ground Improvement (Chu, et al. 2009). An open access web-site, GeoTechTools.org, described by Schaefer, et al (2012) is now available that provides information and interactive selection guidance on 46 technologies useful for soil stabilization, ground improvement and reinforcement, and geo-construction. The information provided for each technology includes a technology fact sheet, photos, case histories, design guidance, quality control and quality assurance information, cost information, specifications, and a bibliography.

The choice of the most appropriate ground improvement method or methods for mitigation of failure risks to dams is critical. In retrospect, the use of deep dynamic compaction for upstream foundation improvement at MIAD was later deemed not effective because of the unimproved zone that remained beneath the densified block, Fig. 10 and Zone 8 in Fig. 15. The presence of the high fines content in the lower portion of the downstream foundation soil at MIAD, Fig. 14 and Zone 12 in Fig. 15, was later deemed responsible for the stone columns to provide the improvement needed.

A few general observations concerning trends in the use of different ground improvement methods for mitigation of liquefaction risk and excessive deformations in dam foundations are:

- The use of vibro-compaction and vibro-replacement is decreasing
- The use of deep soil mixing is increasing.
- Buttress fills and downstream overlay fills are among the most cost-effective treatment methods provided suitable fill material and the necessary space are available.
- The promise of jet grouting for use in dam foundations is yet to be realized.
- What you can see, measure, and test is invariably a better and more reliable option than what you can't see, provided cost and construction risks are acceptable.

## 5. Some Continuing and Unresolved Problems

A number of unknowns, uncertainties, and problems can be identified that, if resolved, could lead to better risk evaluations, more optimized selection of mitigation strategies and ground treatment methods, and improved predictions of future behavior. A desirable goal is to "get it right the first time" so that subsequent mitigation measures, as were needed at both San Pablo Dam and Mormon Island Auxiliary Dam, will not be necessary. Among these unknowns, uncertainties and problems are:

- Anticipating (predicting) future increases in demand on the facility; e.g., greater seismic loading, larger probable maximum flood, adverse consequences of climate change, increased population at risk
- Interpreting and communicating the results of a risk analysis
- Deciding the acceptable level of risk
- Assessing the liquefaction potential of soils containing gravel and cobbles
- Assessing the liquefaction potential of silty soils
- Assessing the post-earthquake residual strength of liquefied soil
- Selecting and implementing the appropriate soil constitutive model for use in liquefaction and dynamic deformation analyses
- Assessing the reliability and accuracy of dynamic deformation analyses. A widely accepted "rule of thumb" has been that actual deformations may be within about +/-

100 percent of the computed or estimated value; however, it is not really known how valid this estimate is.

- Accounting for time (aging) effects following densification and/or admixture stabilization of foundation and embankment soils
- Writing enforceable specifications that will produce the needed end results, but also allow for inherent variability in materials and other site-specific conditions
- Assessing compliance with the ground improvement specifications for uniformity and post-treatment strength and stiffness requirements

## 6. Concluding Comments

Getting it right the first time can be very difficult given the unknowns and uncertainties at the time of initial design and construction of an embankment dam. The two case histories described in this paper illustrate that getting the remediation of a deficient existing dam right the first time can also be very difficult. The potential consequences of climate change, increasing numbers and magnitudes of extreme events (floods, storms, earthquakes, fires, etc.) must be considered from the outset of a project. Simple, observable, and measurable methods for dam strengthening and risk mitigation should be used wherever possible. A reasonable overall goal should be to make an existing, deficient embankment dam as safe as if you were starting a new project today. Resolution of the issues listed in the previous section should be instrumental in helping to reach this goal by enabling better selection and optimization of methods for mitigation of risks to existing dams in the future.

## References


Allen, M.G., Jones, R. and Gularte, F.B. (1995). Bottom-feed stone columns, wet replacement construction method: Mormon Island Auxiliary Dam modifications. Soil Improvement for Earthquake Hazard Mitigation, Geotechnical Special Publication 49, ASCE, pp.82-95.

Chu, S., Varaksin, S., Klotz, U., and Menge, P. (2009). Construction Processes, Proc. Int. Conf. on Soil Mechanics and Geotechnical Engineering, Alexandria, Egypt, IOS Press.

de Mello, V.F.B. (1977). Reflections on design decisions of practical significance to embankment dams, The Seventeenth Rankine Lecture, Geotechnique, Vol. 27, No. 3, pp. 281-355.

Duncan, J.M. (2013). Slope stability then and now, Proceedings of the GeoCongress 2013: Stability and Performance of Slopes and Embankments III, p 2191-210, 2013.

Kirby, R.C., Roussel, G.L., Barneich, J.A., Yiadom, A.B. and Todaro, S.M. (2010). Design and Construction of Seismic Upgrades at San Pablo Dam Using CDSM, Proc. U.S. Society of Dams Conference, pp. 137-152.

Mitchell, J.K. (2008). Recent developments in ground improvement for mitigation of seismic risk to existing embankment dams, Geotechnical Earthquake Engineering and Soil Dynamics IV, ASCE Geotechnical Special Publication 181, pp. 1-20.

Moriwaki, Y., Dinsick, A., Barneich, J., Roussel, G., Yiadom, A., Starr, F., and Tan, P. (2008). Seismic Characterization and Its Limited Implication for San Pablo Dam. Geotechnical Earthquake Engineering and Soil Dynamics IV: pp. 1-10. doi: 10.1061/40975(318)179

Schaefer, V.R., Mitchell, J.K., Berg, R.R., Filz, G. M and Douglas, S. C. (2012). Ground improvement in the 21st century: a comprehensive web-based information system, Geotechnical Engineering State of the Art and Practice, K. Rollins and D. Zekkos, eds., ASCE Geotechnical Special Pub. No. 226, pp. 272-293.

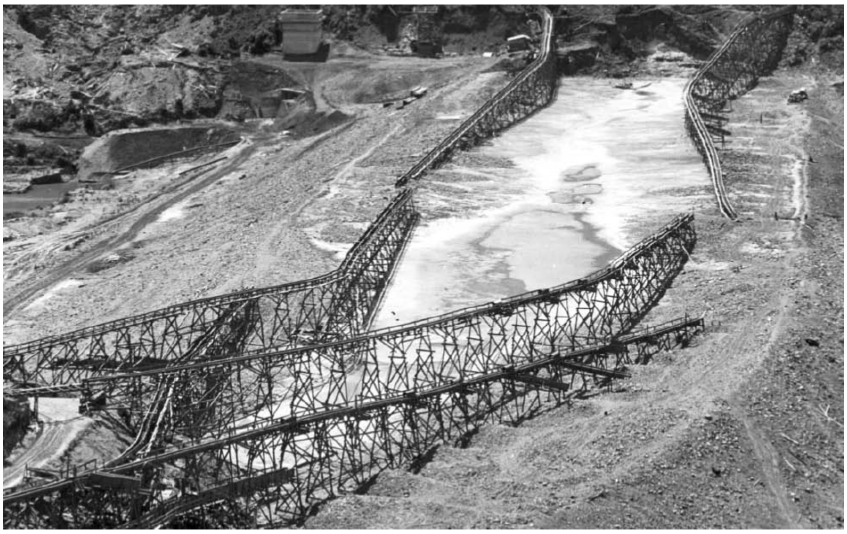
TNM Terra Engineers Ninyo and Moore (2007). Site Characterization of Dam Area, Report, Design Services for San Pablo Dam Seismic Upgrade, April 2007. 347 p.

U.S. Bureau of Reclamation (2010). Mormon Island Auxiliary Dam Modification Project, Final Supplemental Environmental Impact Statement/Environmental Impact Report. Folsom California, Mid-Pacific Region, State Clearing House #2009042077. May 2010.

Yiadom. A.B. and Roussell, G.L. (2012). Seismic Upgrades of San Pablo Dam, Geo-Strata, May/June 2012, pp34-39.

(a)Core trench to bedrock with shoring in 1917

(b) Hydraulic fill construction site in 1919



(c)Hydraulic fill transportation and deposition in 1920

Fig. 1. Stages in the construction of San Pablo Dam. (East Bay Municipal District construction photos Reproduced and reported by TNM Terra Engineers, Inc., Ninyo and Moore, 2007)

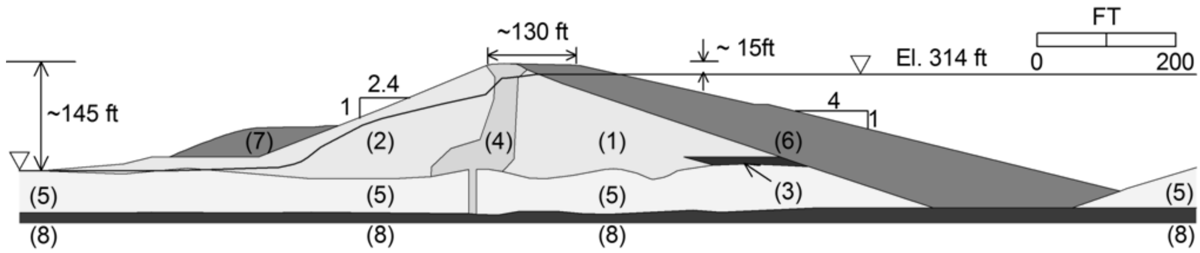



Fig. 2. Composite maximum section of San Pablo dam after buttress additions. (adapted from Moriwaki, et al. 2008). (1) upstream shell (hydraulic fill); (2) downstream shell (hydraulic fill); (3) ponded clay/silt; (4) core and key trench (hydraulic fill); (5) alluvial foundation soils; (6) upstream buttress (well-compacted) completed in 1979; (7) downstream buttress (less-compacted) added in 1967; and (8) bedrock.

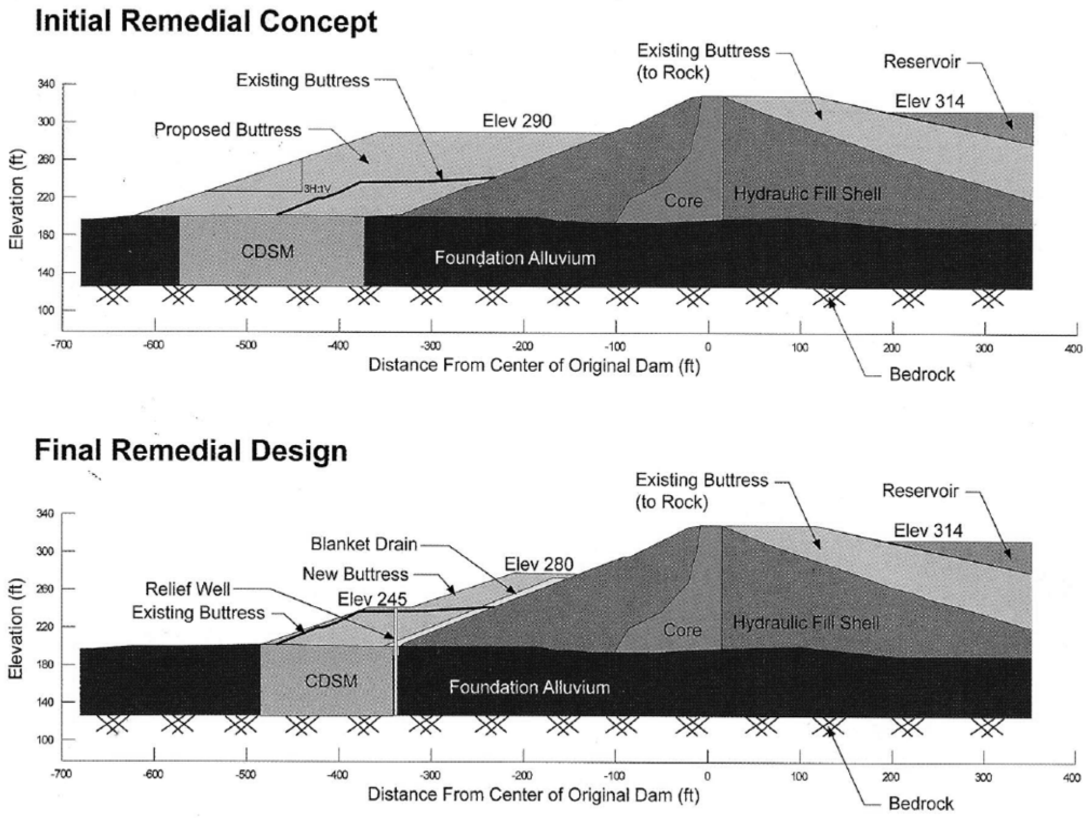



Fig. 3. Remedial designs for mitigation of seismic risk to San Pablo Dam. (from Yiadom and Roussel, 2012).

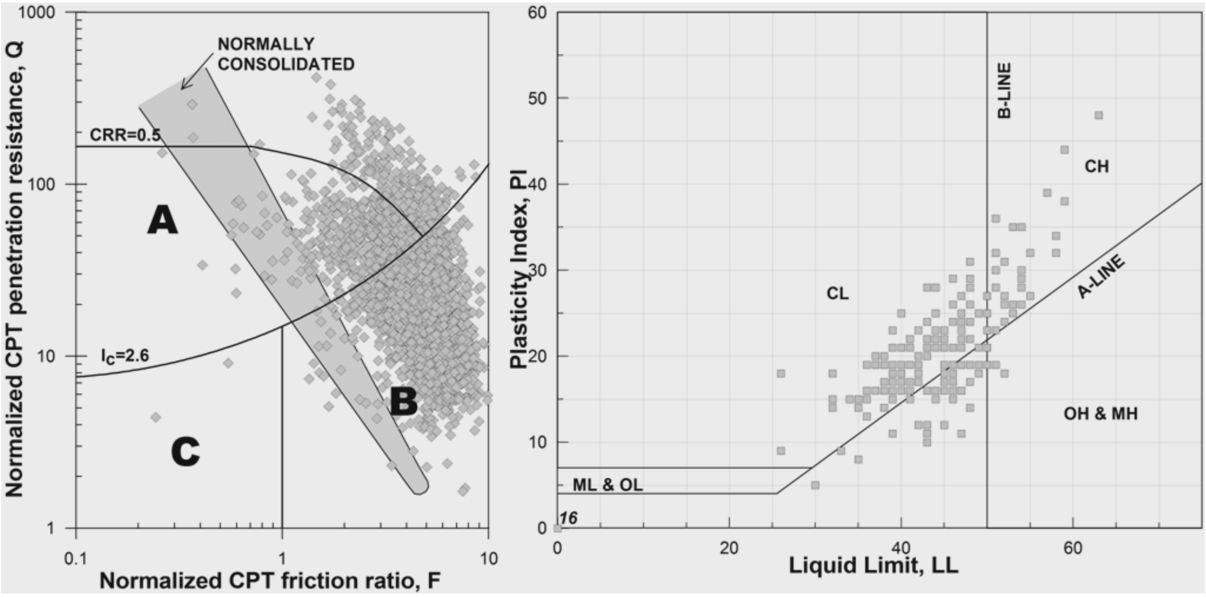



Fig. 4. CPT and Classification data for the San Pablo Dam hydraulic fill shell material. Zone A: Cyclic liquefaction possible; Zone B: Cyclic liquefaction unlikely; Zone C: Flow/cyclic liquefaction possible. (from Moriwaki, et al, 2008).


Fig. 5. San Pablo Dam in 2010 after seismic remediation using cement deep soil mix in the downstream foundation and a downstream stability berm. (Google Earth, Imagery Date 10/2/2009).

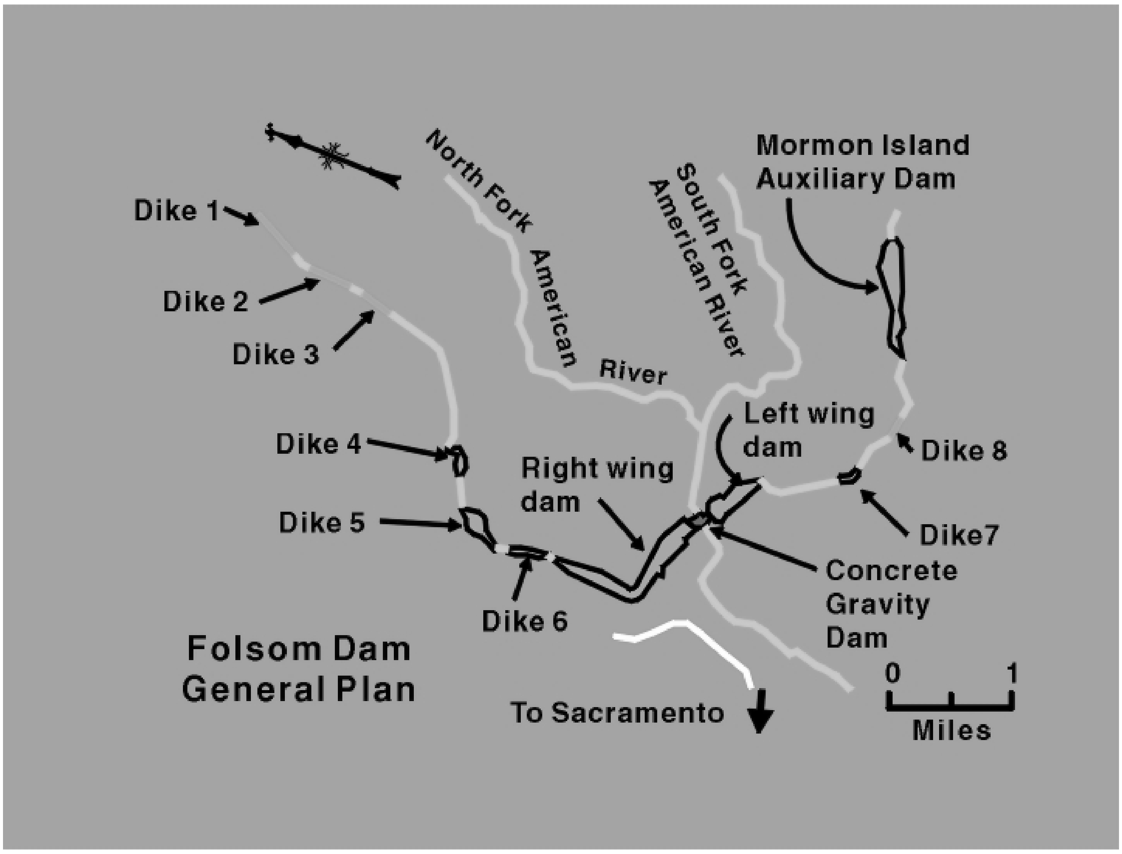



Fig. 6. Components of the Folsom Project near Sacramento, California


Fig. 7. Arial view of Mormon Island Auxiliary Dam as it appeared in August 2013. Critical section over potentially liquefiable foundation material extends about 300 m from the left abutment. (Google Earth, Imagery Date 8/13/2013)

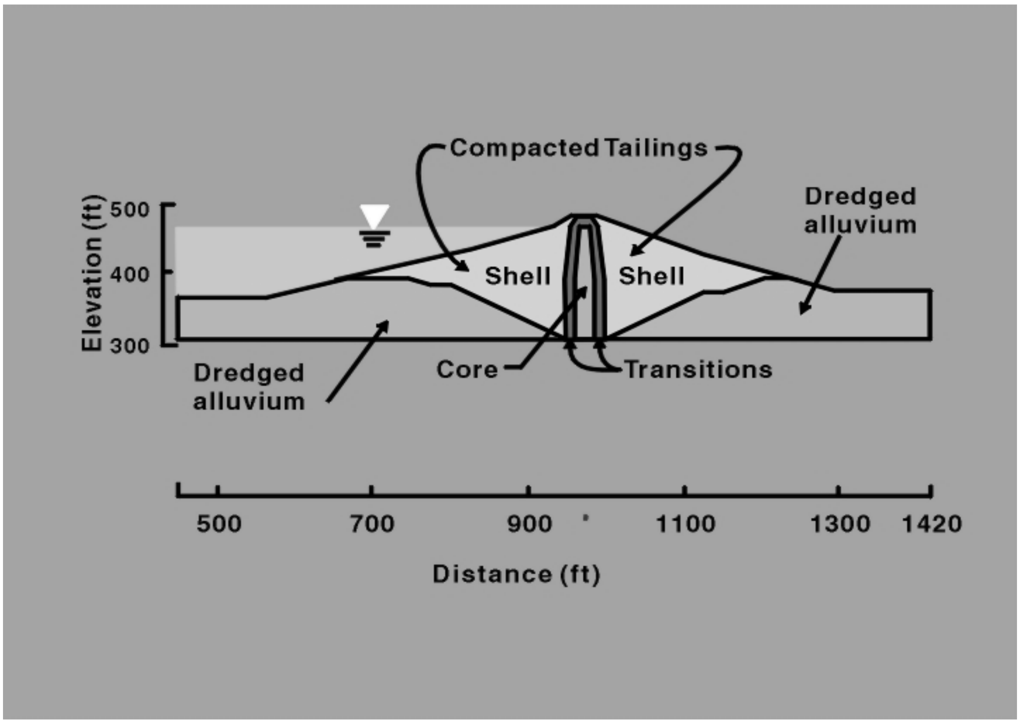



Fig. 8. Cross section of the Mormon Island Auxiliary Dam as constructed in 1956.

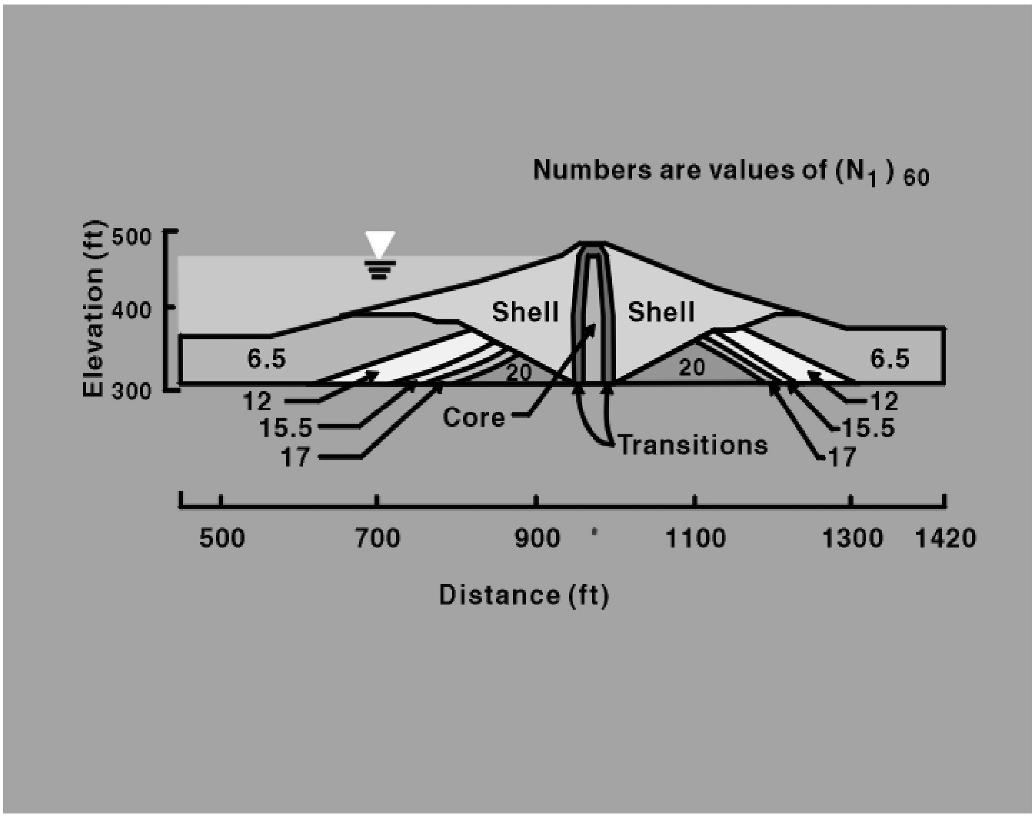



Fig. 9. Characterization of the MIAD foundation material in terms of SPT values of $(N_1)_{60}$ for use in liquefaction potential assessments.

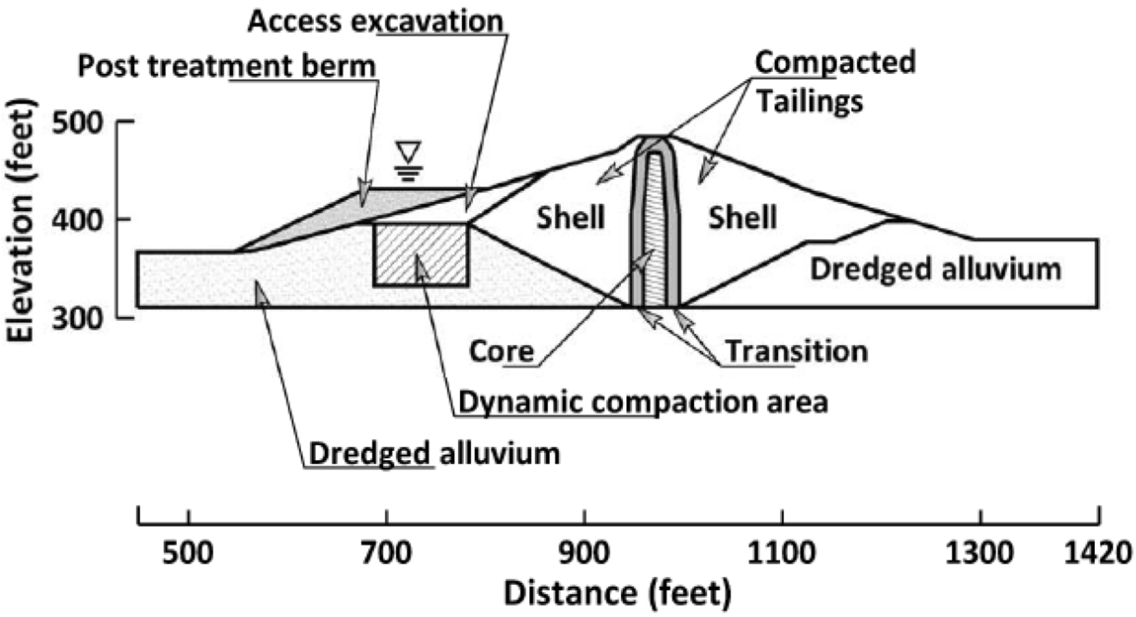



Fig. 10. Ground improvement design intended for mitigation of liquefaction risk beneath upstream embankment of Mormon Island Auxiliary Dam. DDC completed in 1990.

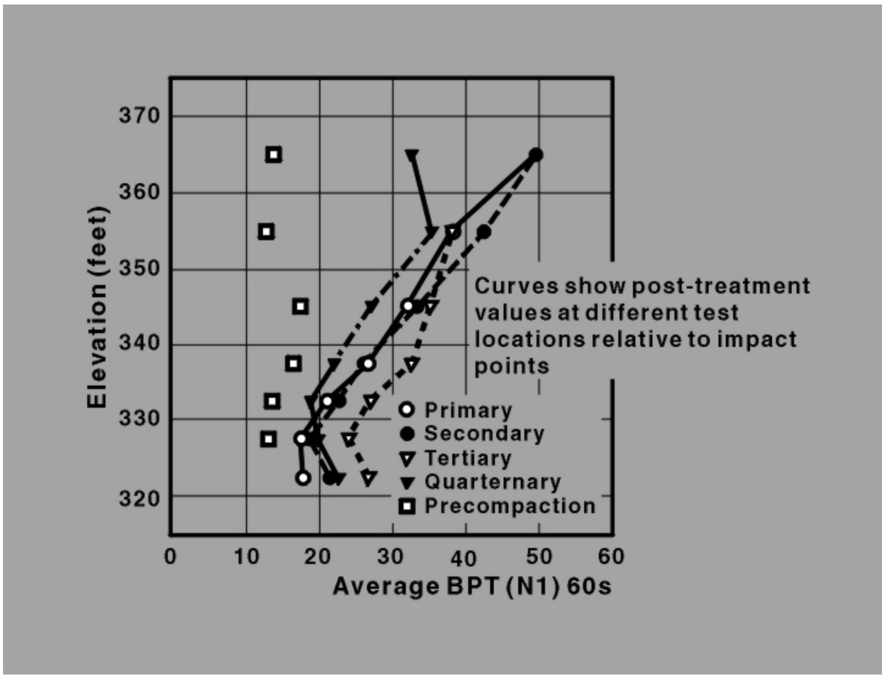



Fig. 11. Pre- and post-deep dynamic compaction Becker Penetration Test equivalent $(N_1)_{60}$ values beneath the upstream slope of Mormon Island Auxiliary Dam.

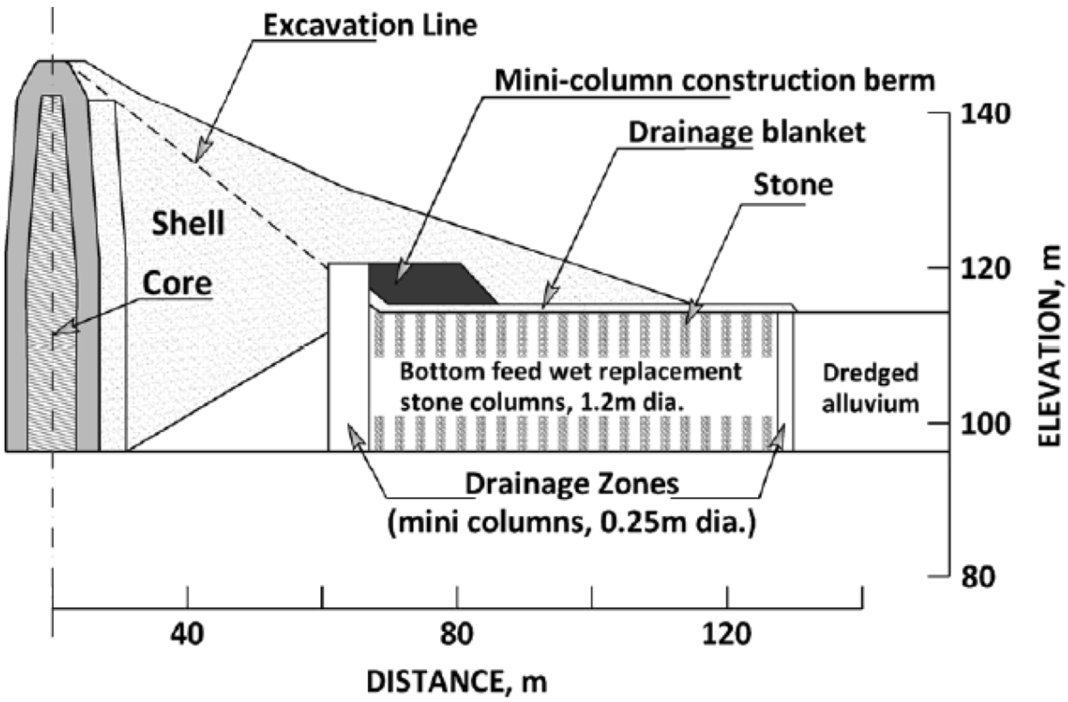



Fig. 12. Downstream foundation improvement for reduction of seismic failure risk at Mormon Island Auxiliary Dam. Construction was completed in 1994.

Fig. 13. Installation of stone columns and gravel mini-column drains beneath the downstream embankment of Mormon Island Auxiliary Dam in 1994. Note the excavation and steepened slope needed for development of a working platform and for optimization of the improvement zone location for its effectiveness as a key block.

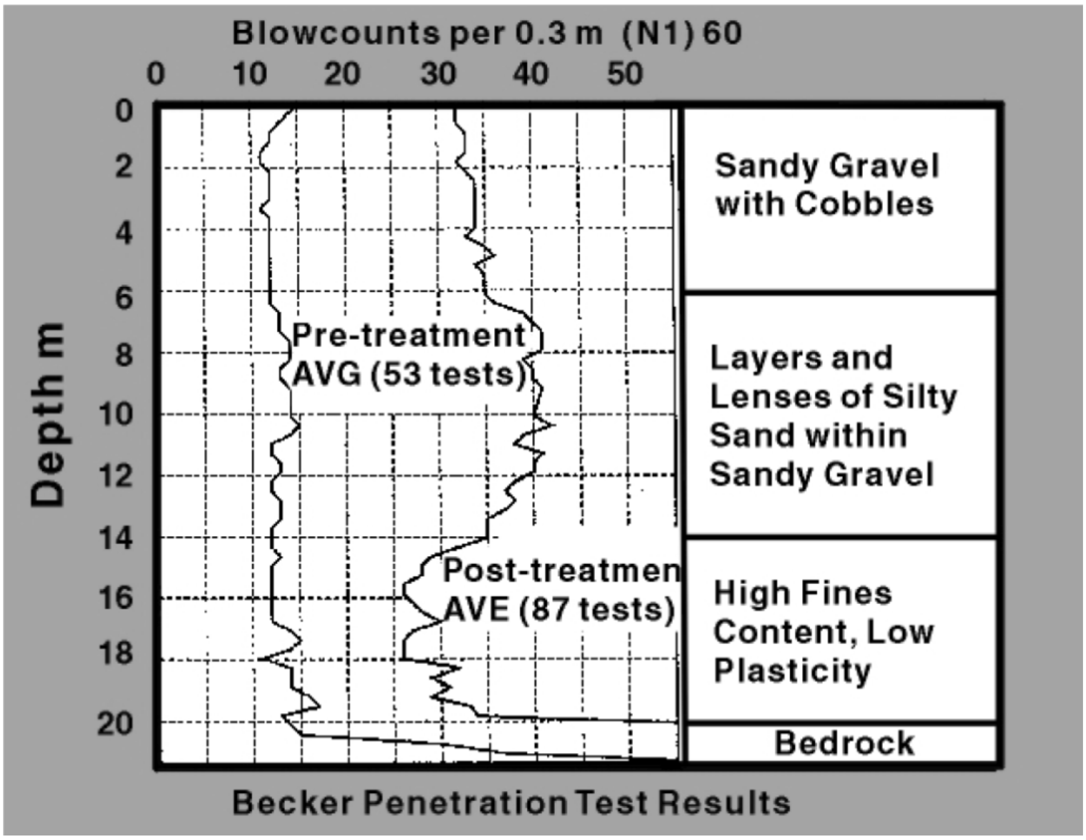



Fig. 14. Penetration resistance profiles in the downstream stone column treatment area, Mormon Island Auxiliary Dam.

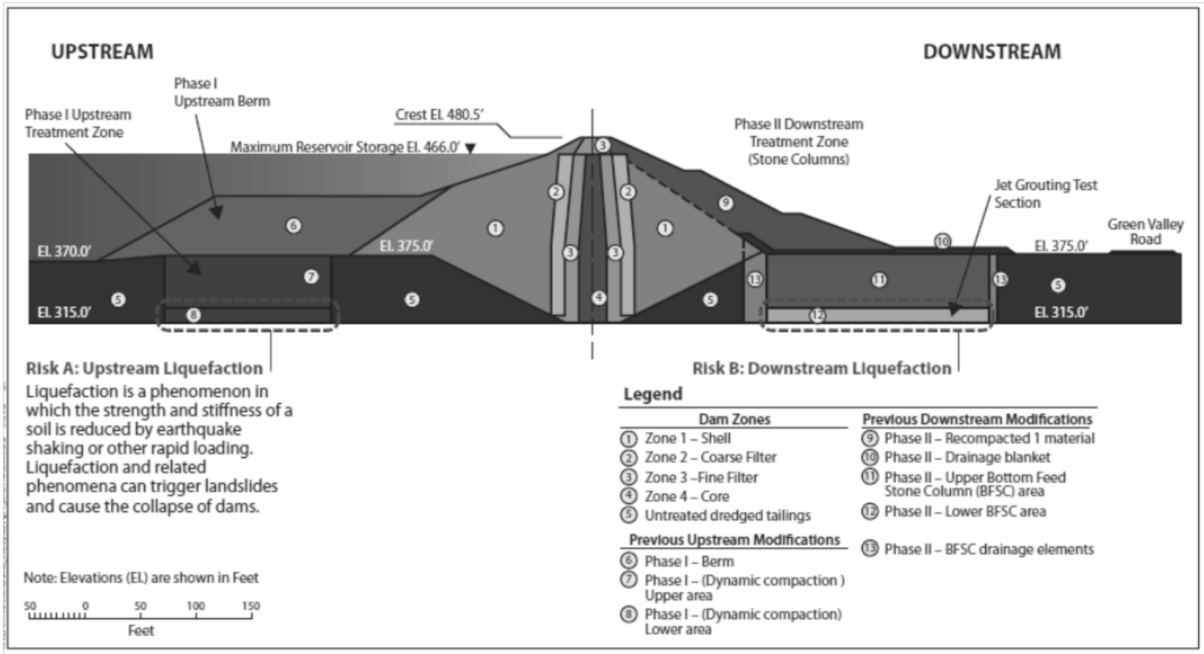



Fig. 15. Cross-section of Mormon Island Auxiliary Dam showing conditions after modifications completed in 1994. (adapted from U.S. Bureau of Reclamation, 2010)

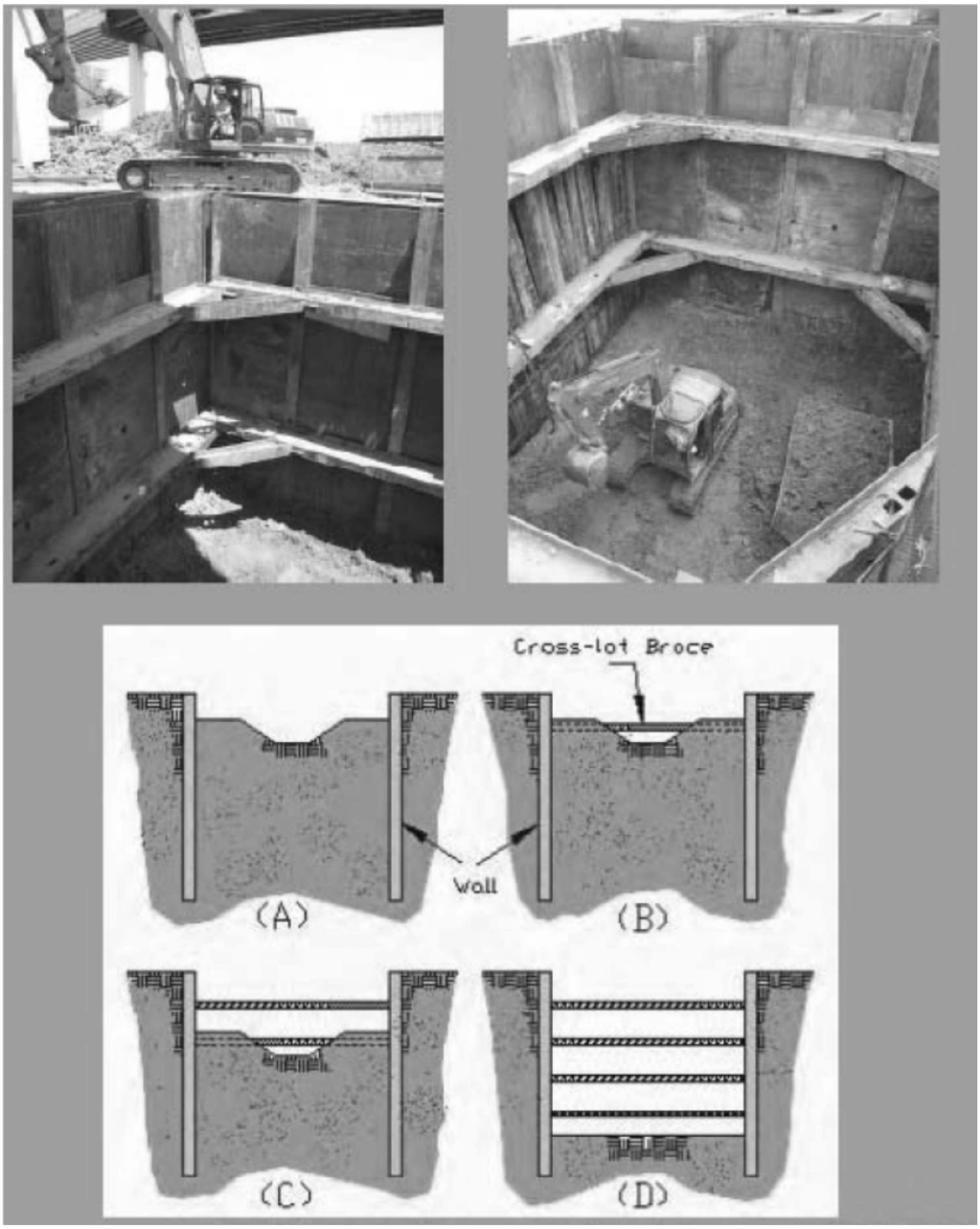



Fig. 16. Schematic diagram and photos of the excavation and bracing system for construction of the concrete key block in the downstream toe area of Mormon Island Auxiliary Dam (adapted from U.S. Bureau of Reclamation)

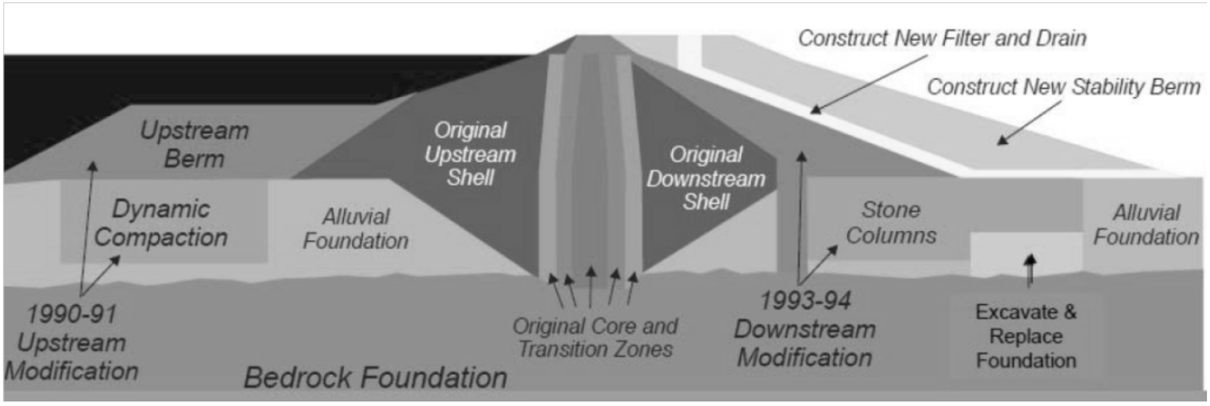



Fig. 17. Mormon Island Auxiliary Dam after completion (est. 2016) of modifications to assure seismic safety. (adapted from U.S. Bureau of Reclamation)