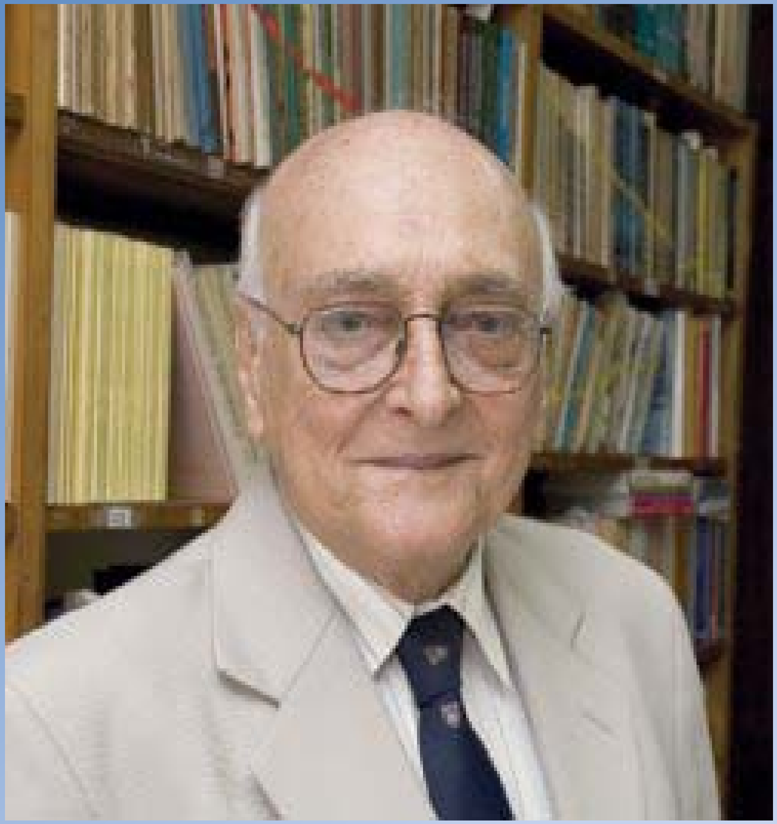
# LESSONS FROM THE LIVES OF TWO DAMS

*The Fourth Victor De Mello Lecture*

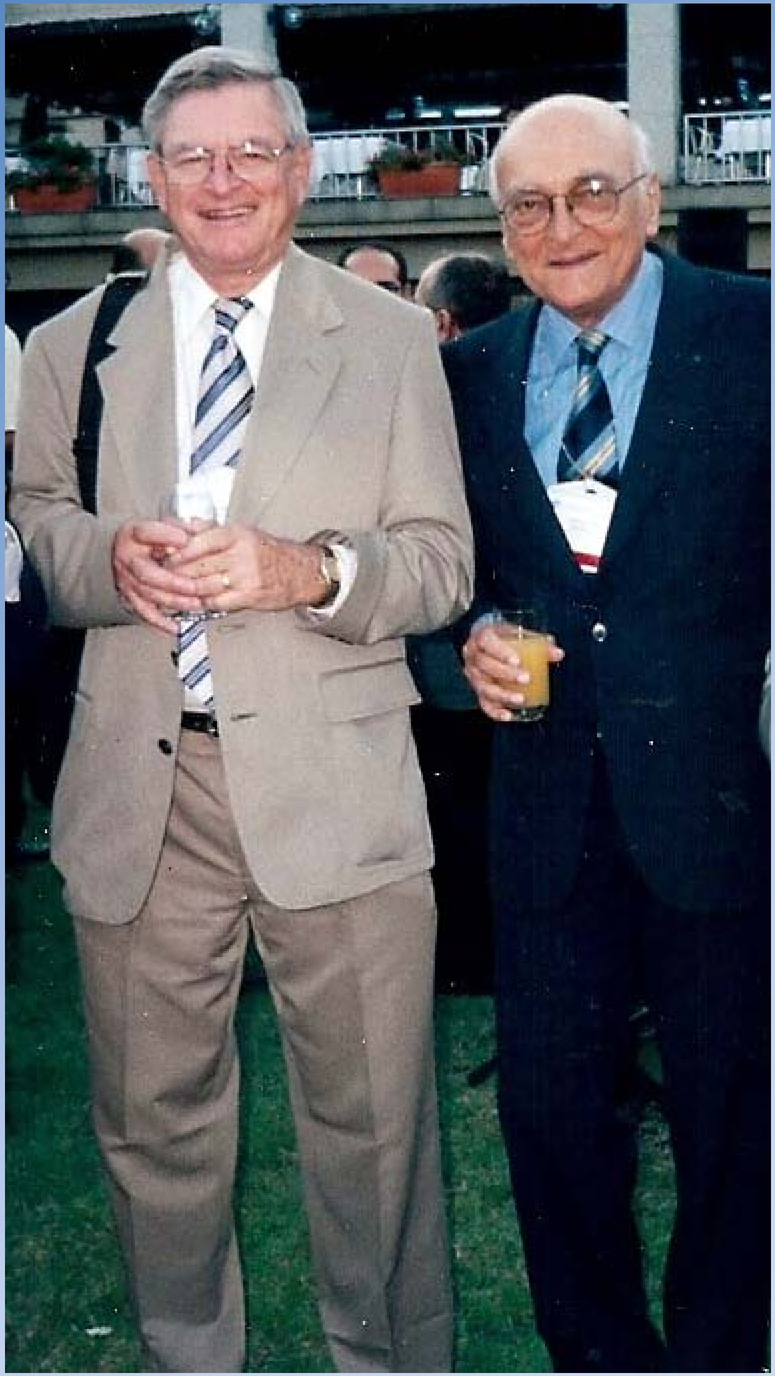
Goiania, Brazil September 10, 2014

James K. Mitchell, ScD, P.E.
Distinguished Professor, Emeritus
Virginia Tech, Blacksburg, VA, U.S.A.



Victor F.B. de Mello
A 20th Century
Giant of Geotechnics -

**AND SO MUCH MORE:**
- Colleague
- Friend
- Dedicated leader of his profession
- Wise Teacher, Researcher, Engineer
- Philosopher
- Innovator
- Motivator



At the International Conference on Soil Mechanics and Geotechnical Engineering in Istanbul, 2001.

In a note to me, Aug-Sept. 2001:

*I suddenly feel back at start, bereaved of my zests of the 13/Jun/56 Graduation into the Brave New CIVIL World, and the 13/Jun/81 incumbency of worldwide service. But all the more, I nurture, we* ***must nurture,*** *the only timeless pervading truth of WELL-WISHING, DOING, and ENJOYING. … as best we may be permitted.*

Victor de Mello at his graduation from M.I.T. in 1946.

# Background to this lecture

- Victor de Mello devoted his 1977 Rankine Lecture to several very important aspects of embankment dam design: filters, seepage and drainage, stability.
- This de Mello Lecture is also concerned with embankment dams.
- Many large embankment dams constructed during the first six decades of the 20$^{th}$ Century.
- The Great Alaska Earthquake (M 9.2) in 1964 and the Niigata, Japan Earthquake (M 7.5), also in 1964 focused attention on soil liquefaction and ground deformation.
- The near catastrophic failure of the Lower San Fernando Dam in the M 6.6 EQ in 1971 in Southern California led to reevaluation of the vulnerability of many other dams.
- Maximum Credible Earthquakes, Maximum Probable Flood estimates, and populations at risk have increased at many sites.
- Risk analyses have indicated unacceptably high consequences unless mitigation measures are implemented at many dams.
- This presentation describes what has been done at two of these dams.

# Overview of Presentation

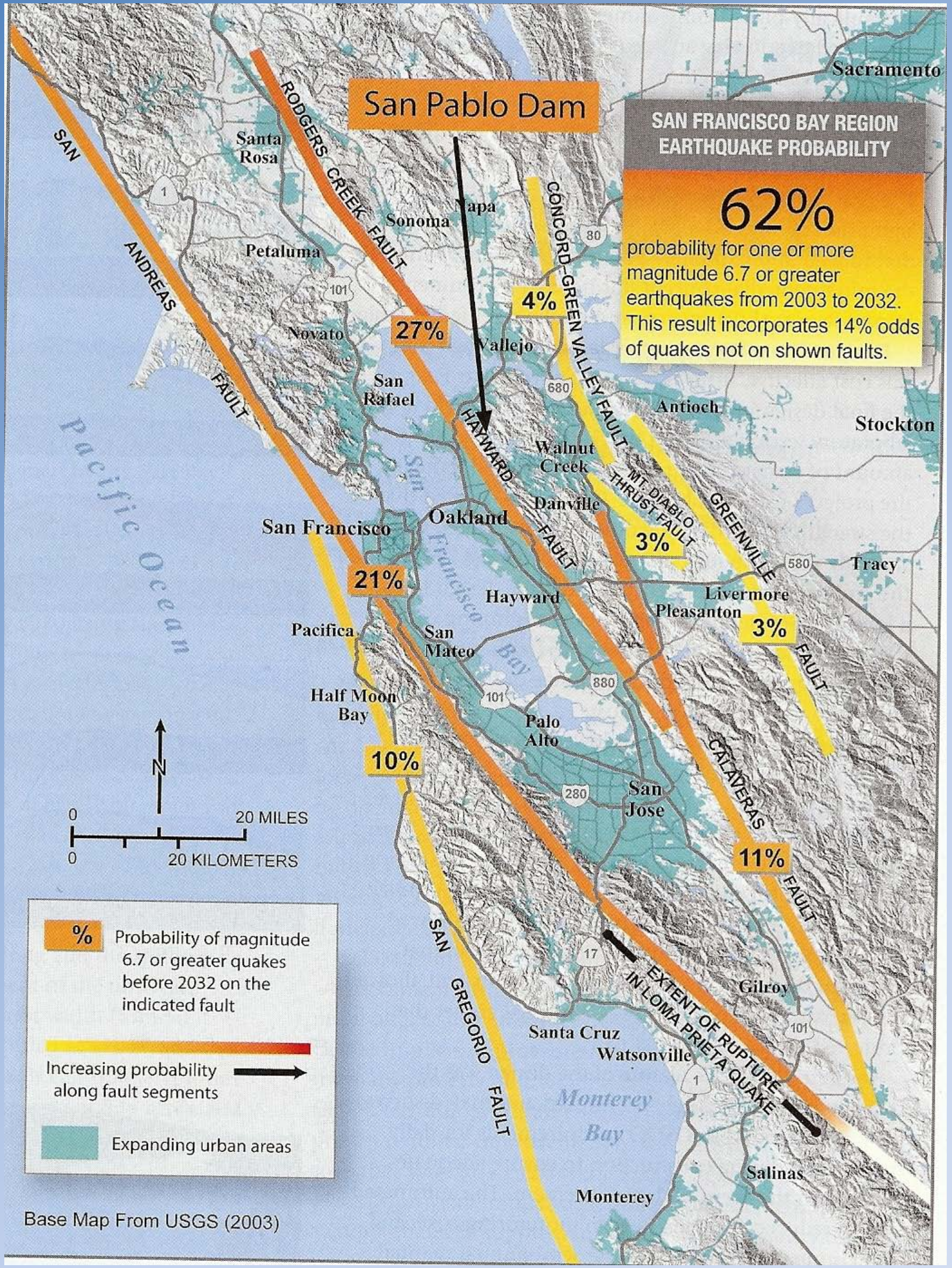
- **San Pablo Dam**, near Oakland, California, is a hydraulic fill structure founded on alluvial deposits; completed in 1921.
- **Mormon Island Auxiliary Dam (MIAD)**, near Sacramento, California, is a compacted fill embankment founded on hydraulically deposited dredger tailings from gold mining operations; completed in 1956.
- Each dam was subsequently deemed unsafe under anticipated seismic loading conditions.
- Several modifications have been made to each dam to improve resistance under anticipated earthquake loadings, extending over the period 1967 to 2010 at San Pablo Dam and from the late 1980's to 2016(?) at Mormon Island Dam.
- These modifications will be described and illustrated.
- Some conclusions and lessons learned about 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 will be offered.

# SAN PABLO DAM

Image Source: Wikipedia, Aug. 17, 2014

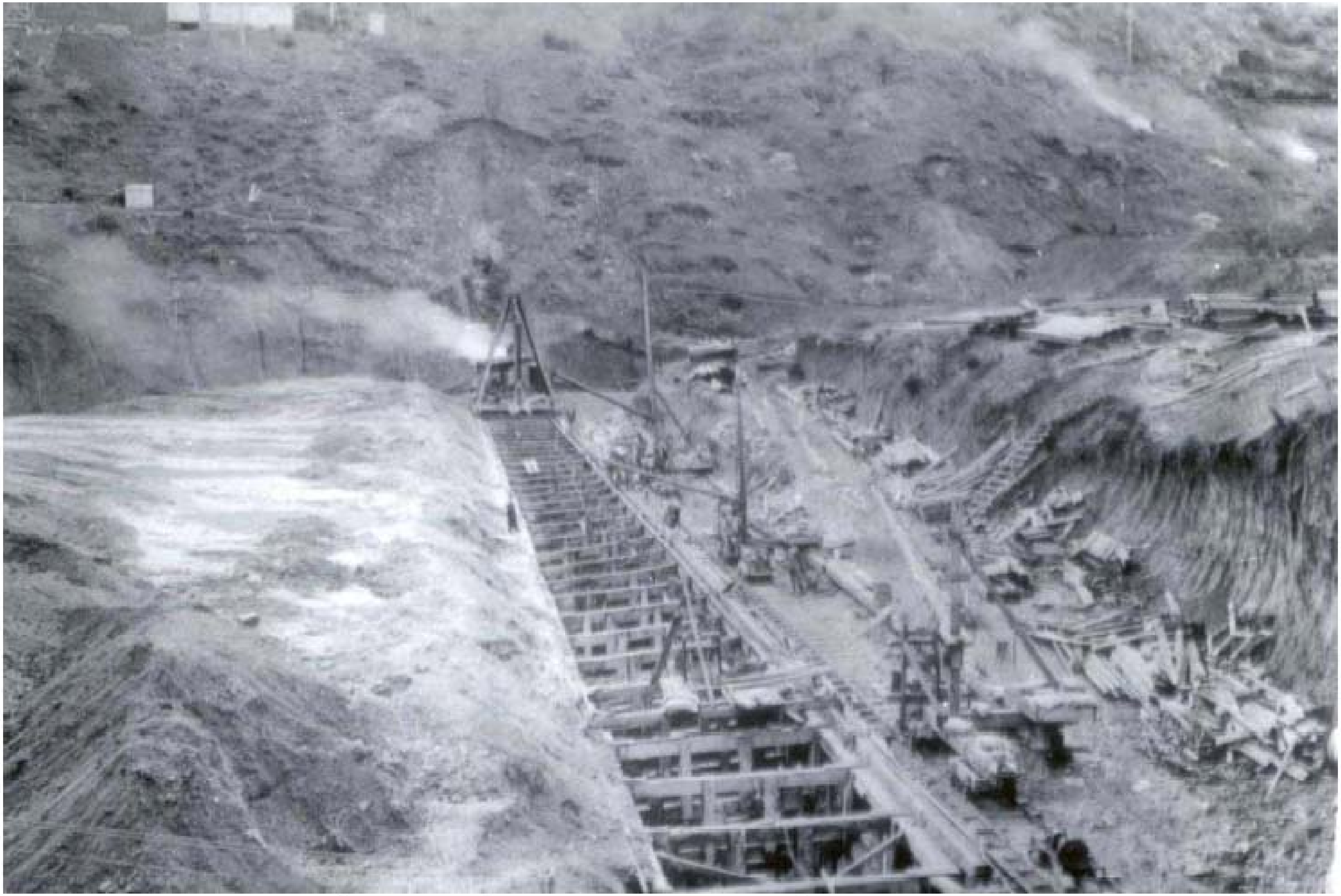
# Features of the Original Dam

- 170ft (53.3m) high, 1200ft (366m) long hydraulic fill dam completed in 1921.
- Founded on alluvial sediments containing some zones susceptible to liquefaction.
- Embankment of hydraulically placed fill material consisting of weathered sandstone and shale obtained from hills near Oakland, California.
- Located within a few km of major active faults.



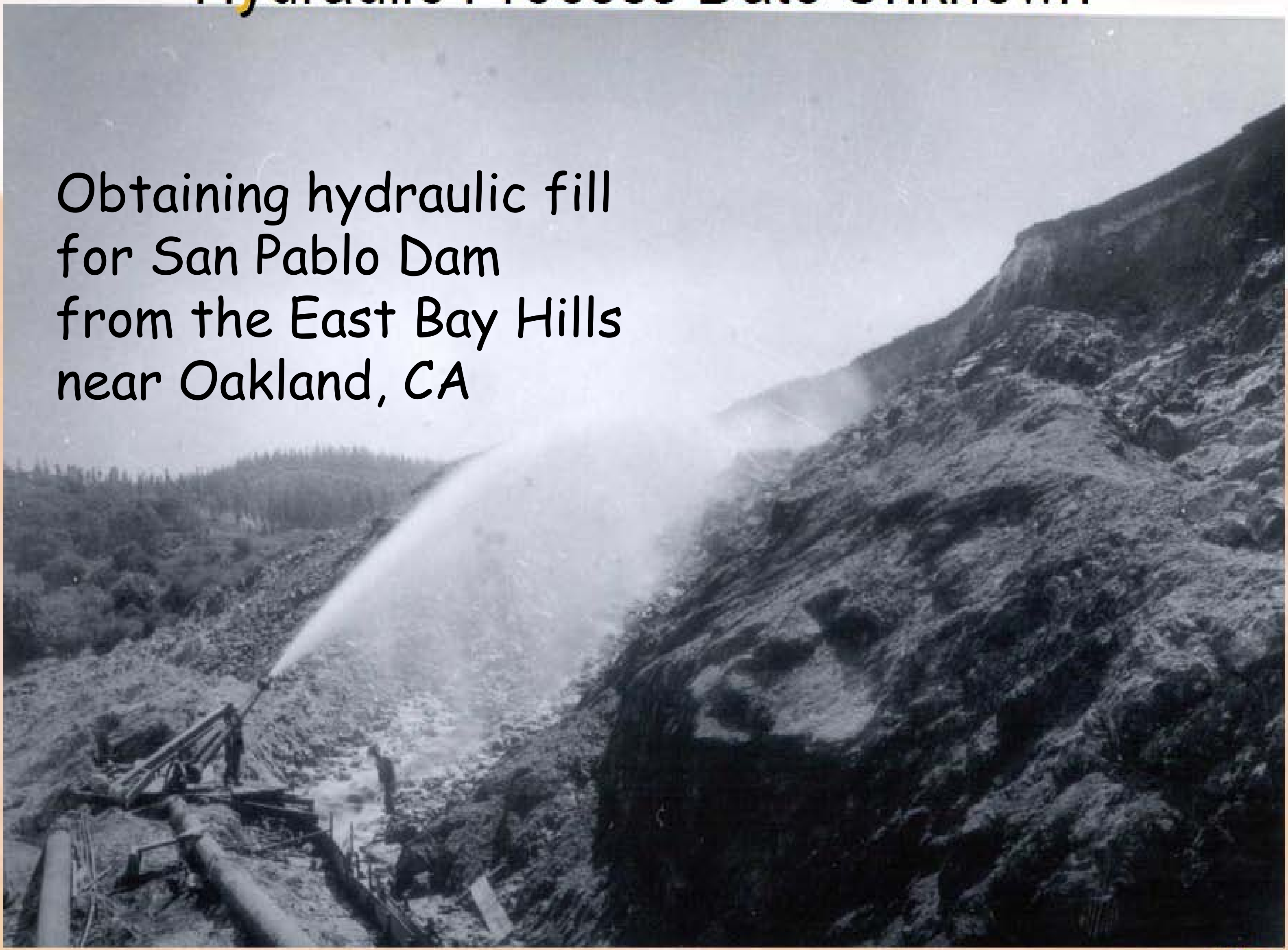

# San Pablo Dam – Amidst the faults

From Yiadom and Roussel GeoStrata, May/June, 2012



| Date: 03/02/07 | DESIGN OF SAN PABLO DAM SEISMIC UPGRADES | CORE TRENCH WITH SHORING (1917) | Figure 5B |
|---|---|---|---|
| TNM | | | |

# Hydraulic Process Date Unknown



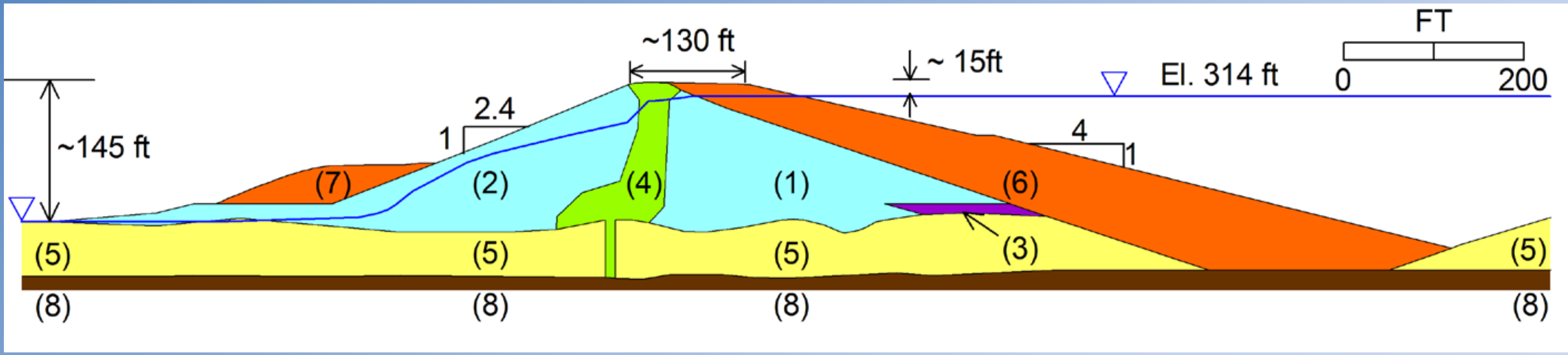
| Date: 03/02/07 TNM | DESIGN OF SAN PABLO DAM SEISMIC UPGRADES | HYDRAULIC FILL CONSTRUCTION (1919) | Figure 5A |
|---|---|---|---|

| Date: 03/13/07 <br> TNM | DESIGN OF SAN PABLO DAM SEISMIC UPGRADES | MATERIAL TRANSPORTATION AND DEPOSITION - SAN PABLO DAM APRIL 17, 1920 | Figure C-12 |
|---|---|---|---|

**Placing Rock Hydraulically in San Pablo Dam, fragments up to 1 ft$^3$.** (from Moriwaki, et al, 2008)

From El Sobrante Historical Society
(https://sites.google.com/site/elsobrantehistoricalsociety/historic-sites)

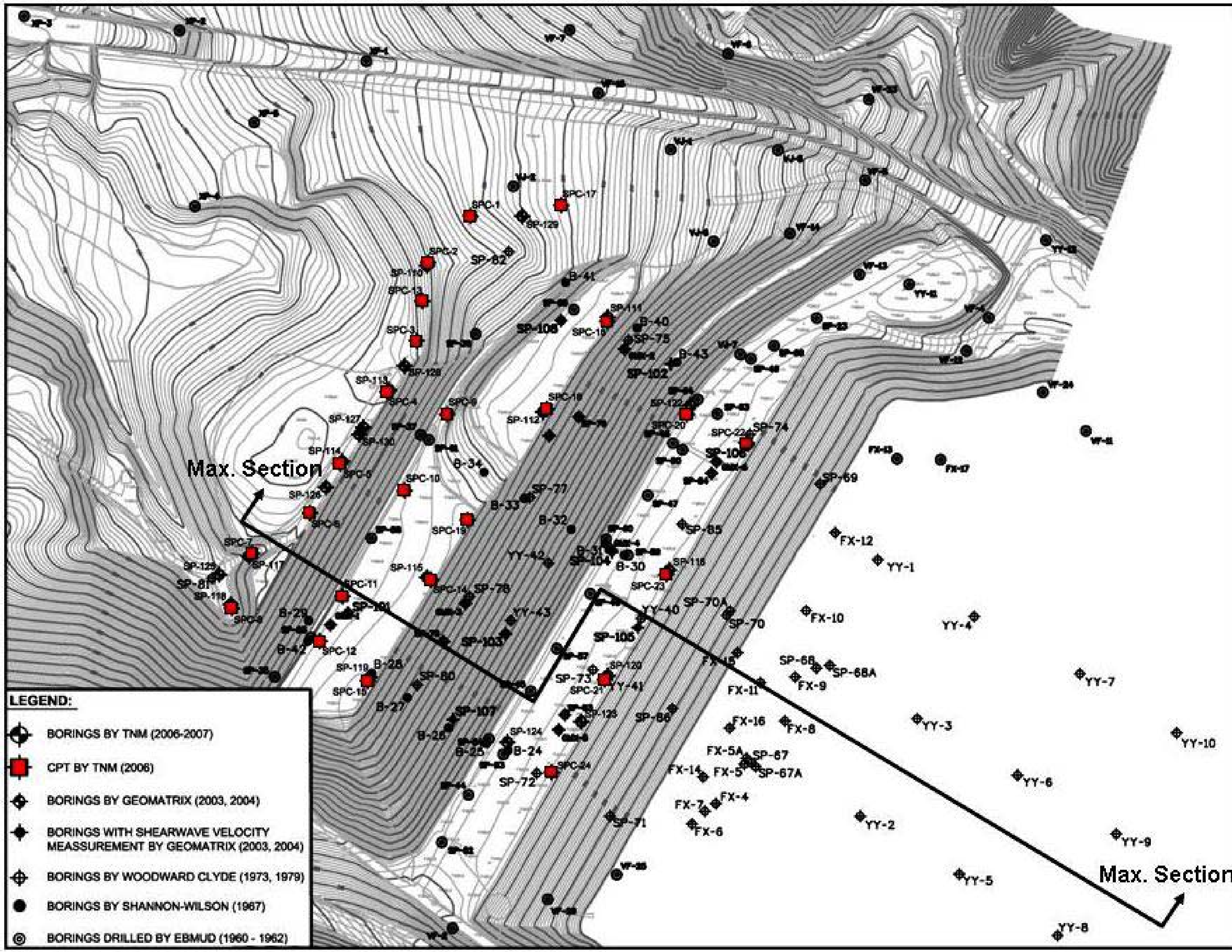
- Evaluations in 1960's and again in 1970's *assumed a liquefiable embankment*, evidently because it was a hydraulic fill.
- A few tests on sandy embankment samples indicated liquefaction potential.
- a small downstream buttress was constructed in 1967.
- A larger upstream buttress to bedrock was completed in 1979.



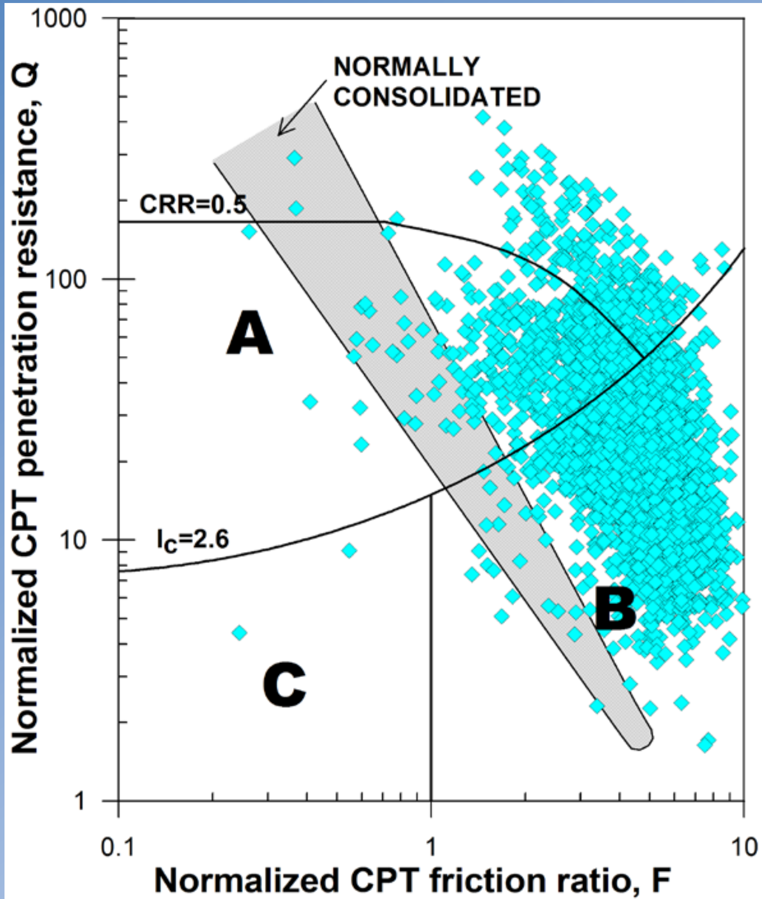

Composite maximum section of the dam after buttress additions. Note a very wide crest of about 130 feet (39.6m) for the dam. The various zones of the embankment and foundation: (1) upstream shell (hydraulic); (2) downstream shell (hydraulic); (3) ponded clay/silt; (4) core and key trench (hydraulic); (5) foundation soils; (6) upstream buttress (well-compacted) (1979); (7) downstream buttress (less-compacted) (1967); and (8) bedrock.

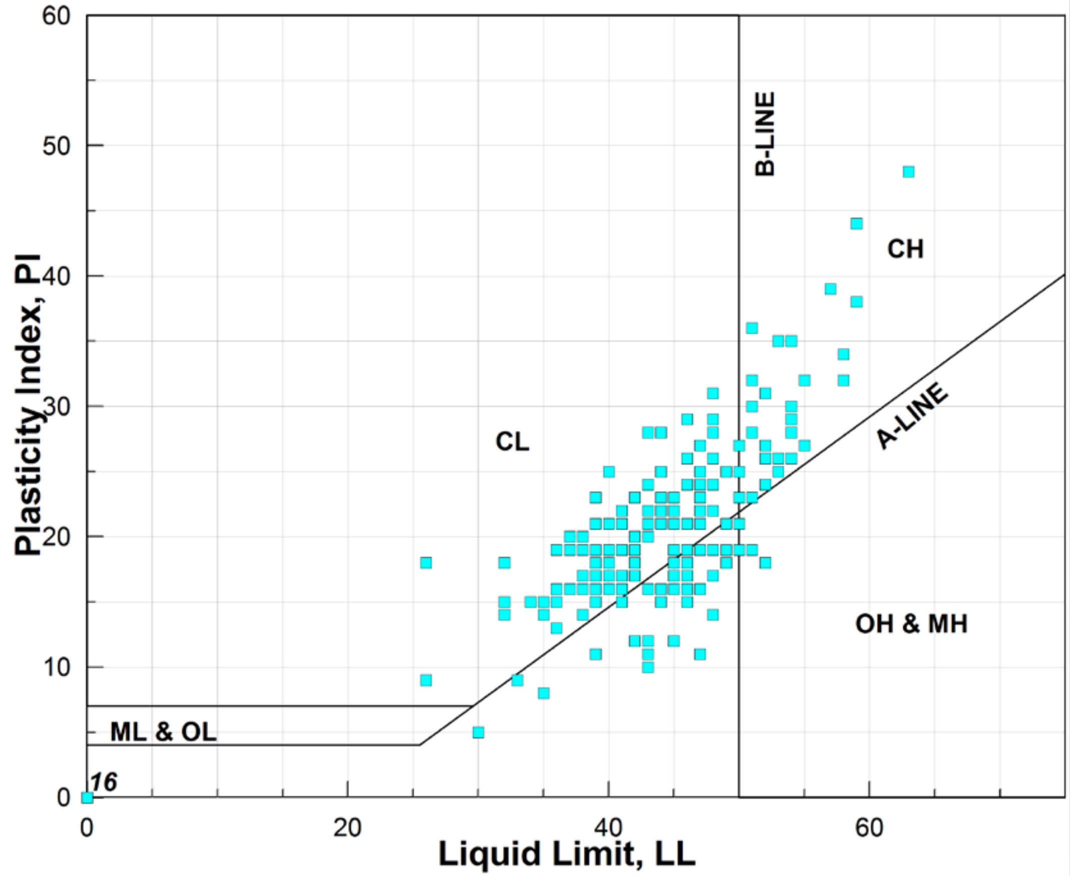
Moriwaki, Y., Dinsick, A., Barneich, J., Roussel, G., Yiadom, A., Starr, F., and Tan, P. (2008) Seismic Characterization and Its Limited Implication for San Pablo Dam. Geotechnical Earthquake Engineering and Soil Dynamics IV: pp. 1-10.
doi: 10.1061/40975(318)179

- Seismic re-evaluation in 2004 assumed liquefiable embankment and indicated excessive slumping (35 ft, 10.7m) and overtopping in a M7.25 EQ on Hayward Fault.
- To provide additional freeboard for the short term the reservoir level was lowered by 20 feet (6m).
- Completely rebuilding the dam would require draining the reservoir.
- Chose an in-place alternative instead, with Cement Deep Soil Mixing (CDSM) to depths up to 120ft (36.5m) through the alluvial foundation soils to rock and a large downstream buttress fill.
- Extensive new field investigations, including many CPTs, were completed.




Field Investigation Locations (from Moriwaki, et al, 2008)



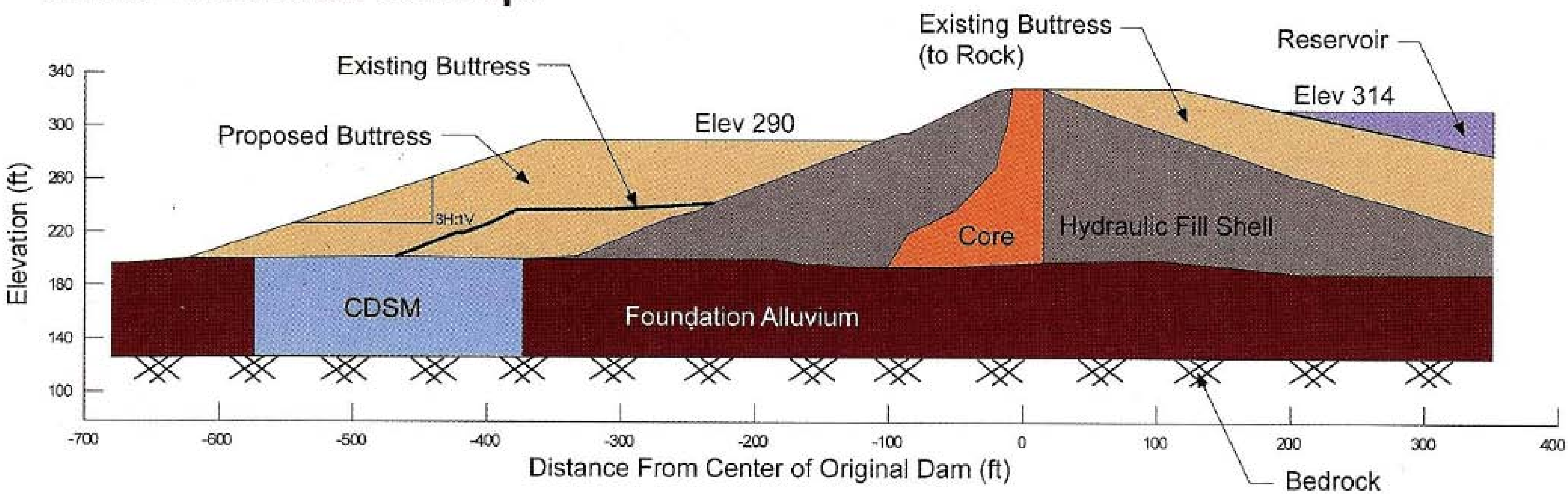




Zone A: Cyclic liquefaction possible;
Zone B: Cyclic liquefaction unlikely;
Zone C: Flow/Cyclic liquefaction possible.

**CPT and Plasticity Data for Shell Materials** (from Moriwaki et al, 2008)

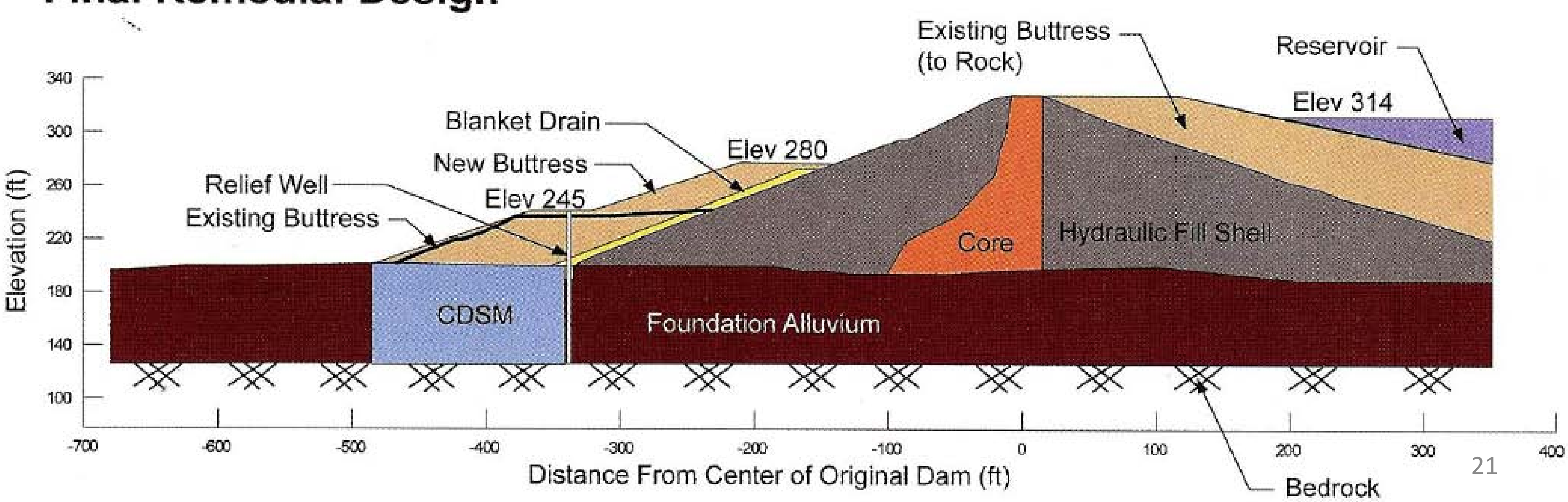
SHELL MATERIAL IS NON-LIQUEFIABLE

## Initial Remedial Concept

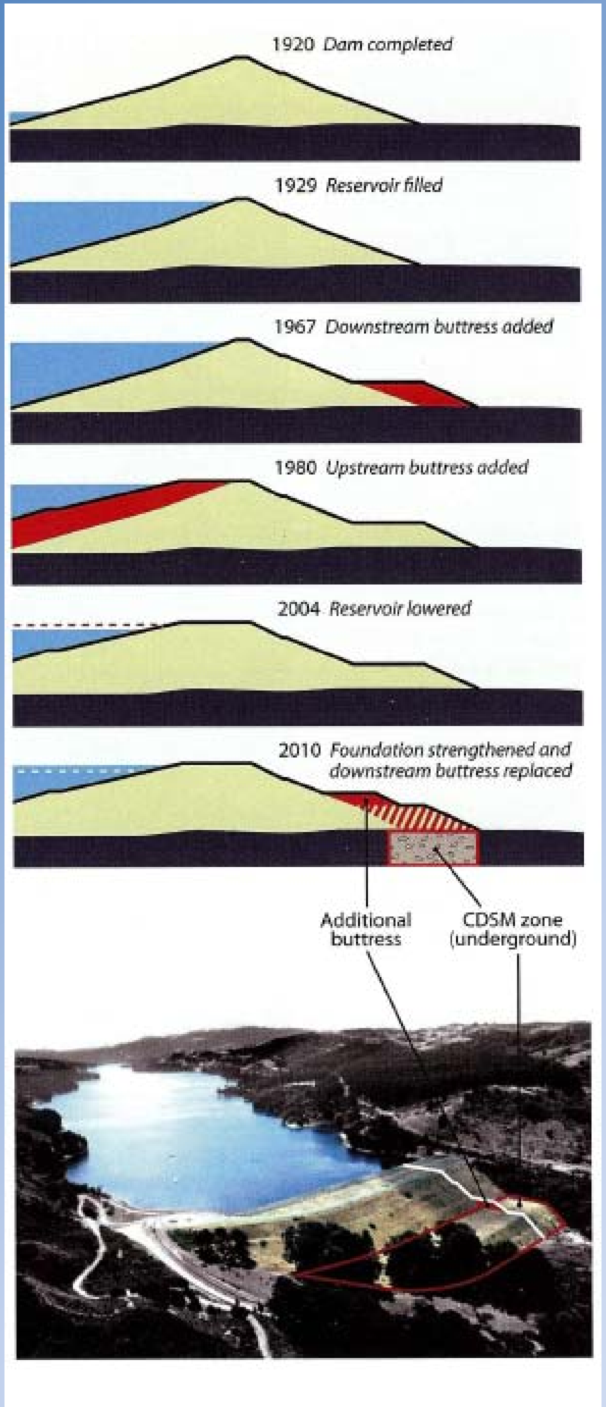



(From Yiadom and Roussel, *Geo-Strata*, May/June 2012)

## Final Remedial Design



## Stages in the seismic remediation of San Pablo Dam




Re-characterization of the hydraulic fill embankment material from liquefiable to non-liquefiable, fine-grained soil enabled significant reduction in the required sizes of the 2010 buttress and CDSM block, with a cost saving of ~U.S.$40 million.

From Yiadom and Roussel, *Geo-Strata,* May/June 2012


SAN PABLO DAM: Remediated (2010) using Cement Deep Soil Mix Block and Downstream Berm

# MORMON ISLAND AUXILIARY DAM


**General**

- Dam type ...................Earthfill
- Watercourse....... Blue Ravine
- Reservoir ...........Folsom Lake
- Construction Date ......... 1956

**Dimensions**

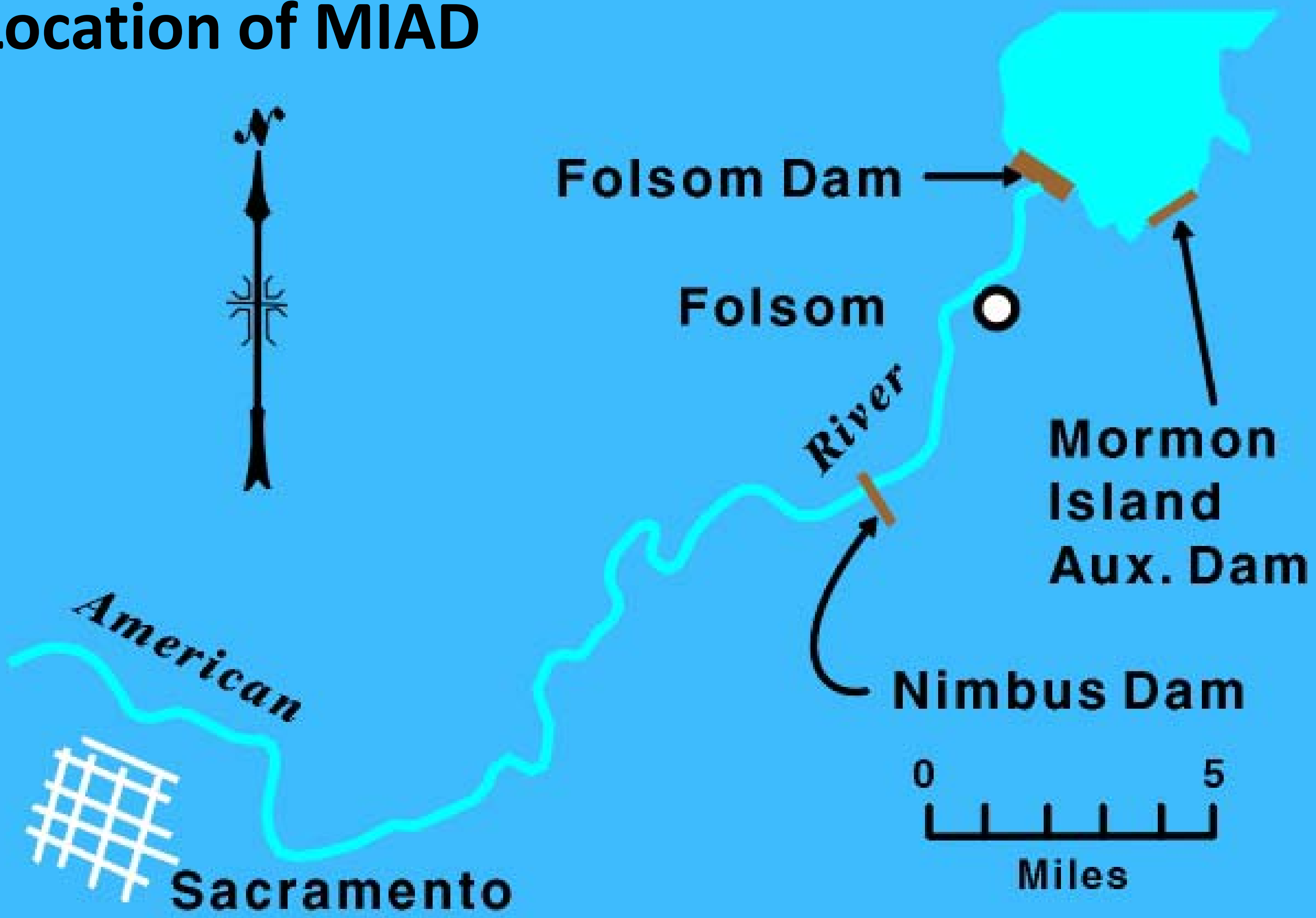
- Crest Elevation:....... 480.0 ft
- Structural Height:........ 110.0 ft
- Crest Length: ..........4,820.0 ft

**Hydrology**

- Drainage Area: .....1,875 sq mi

RECLAMATION

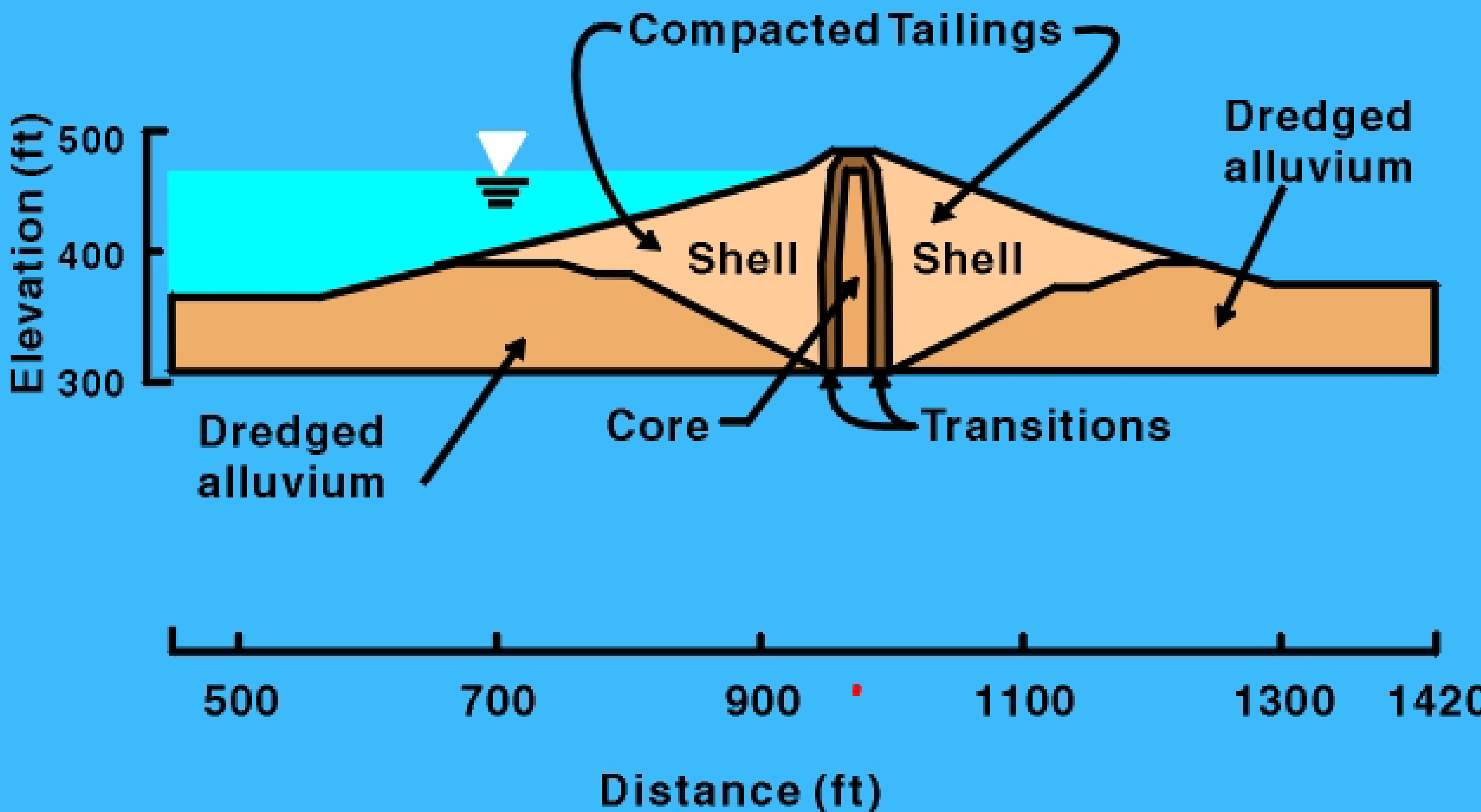
# Location of MIAD



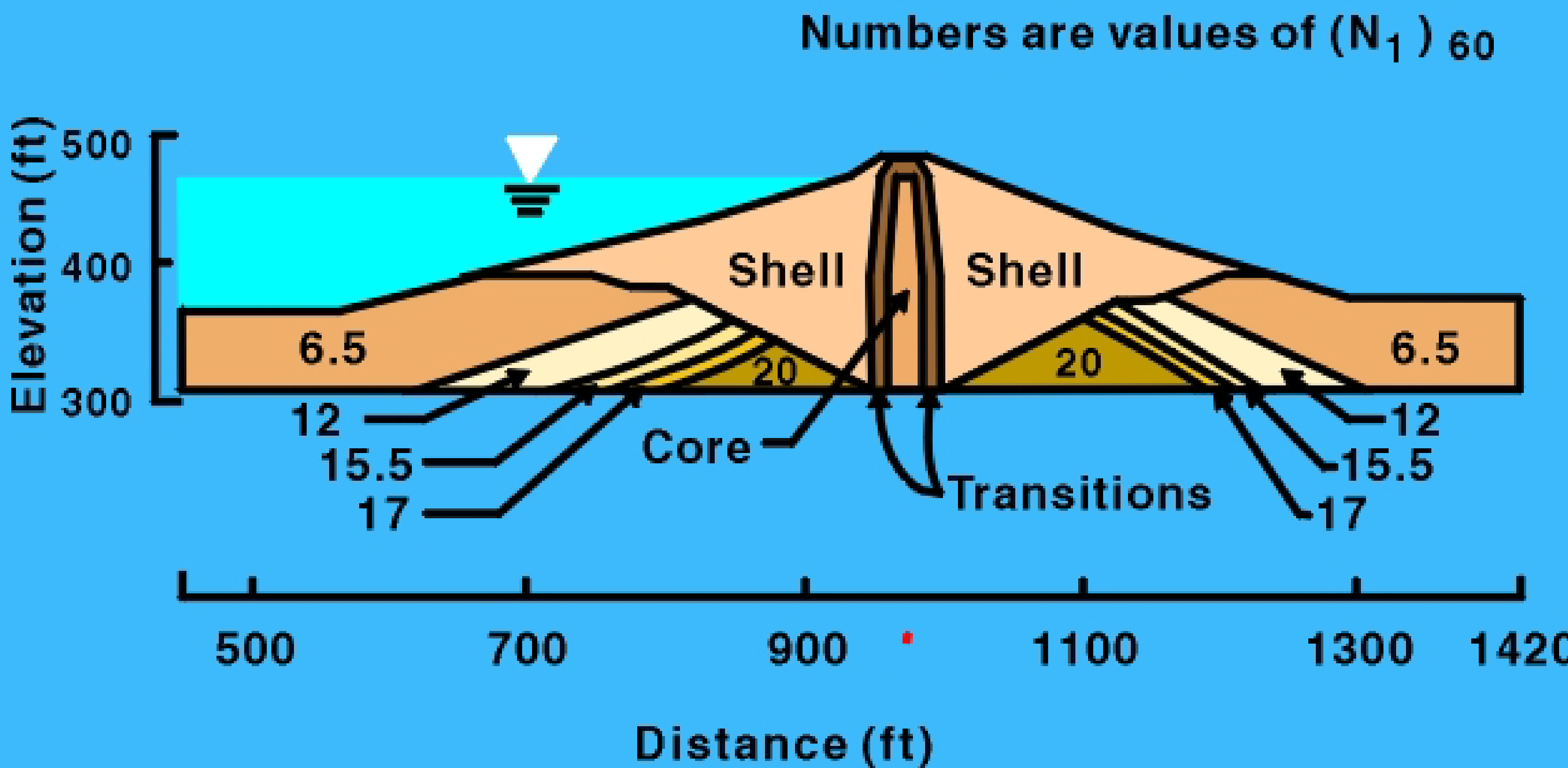

Folsom Dam General Plan


# FOLSOM PROJECT


# FOLSOM MAIN DAM AND WING DAMS




# Cross section of Mormon Island Dam




Penetration Resistance of Foundation –re-deposited dredged alluvium

# Mormon Island Auxiliary Dam

Upstream and downstream improvements done from late 1980s to 1994

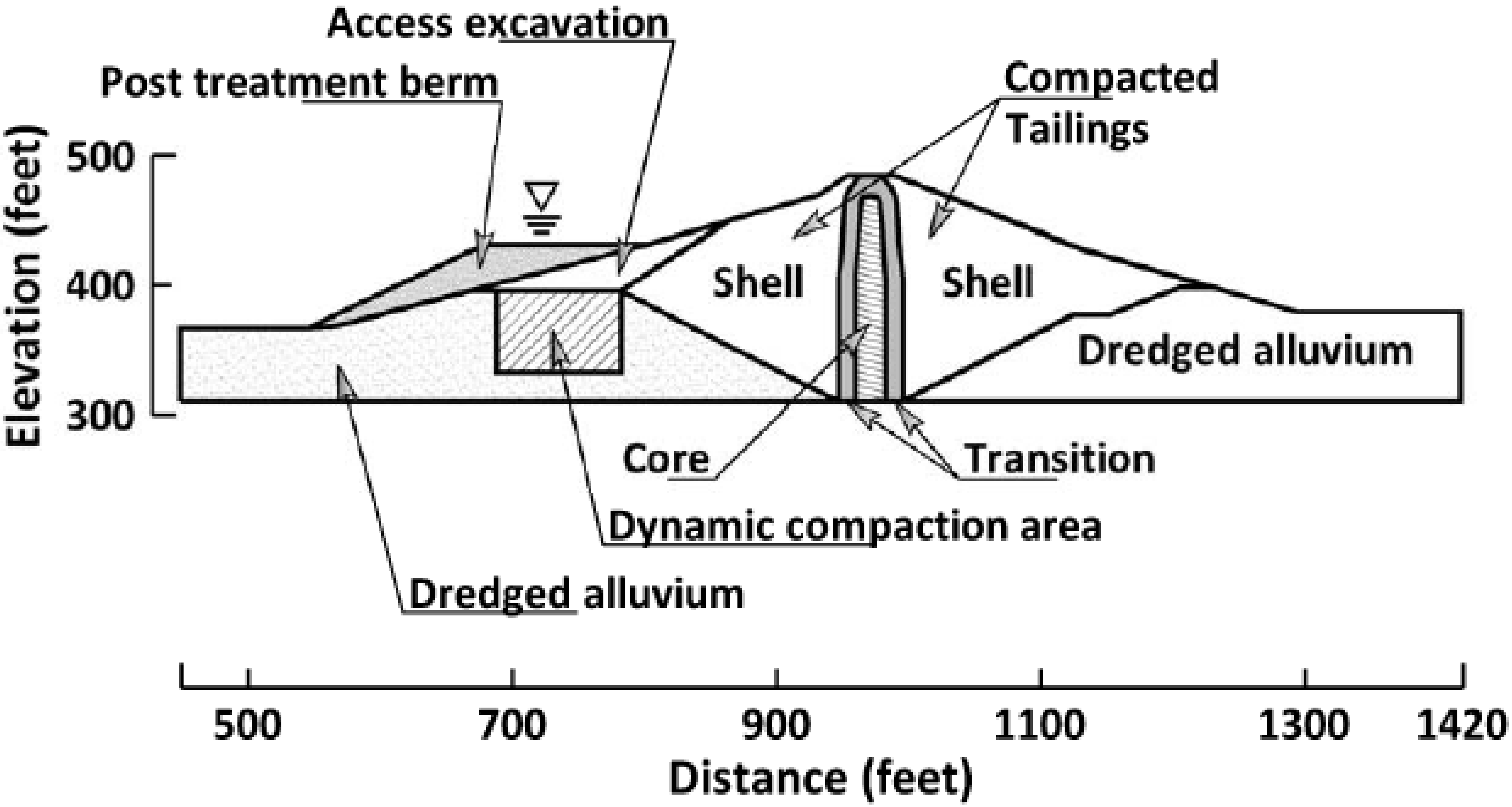
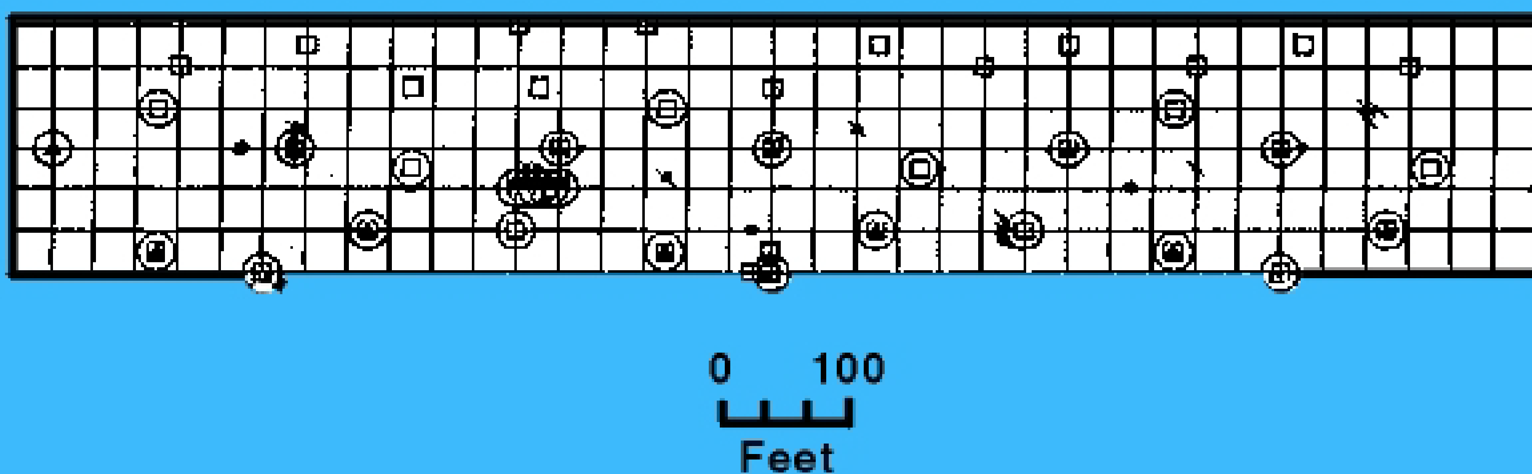



Fig. 6. Ground improvement design for stabilization of the upstream embankment of Mormon Island Auxiliary Dam done in 1990.

Upstream Ground Improvement in Progress (1990)

## Mormon Island Auxiliary Dam

# Dynamic Compaction Grid




- Precompaction BPTs
- Intermediate BPTs (Following phase 2 dynamic compaction)
- Post compaction BPTs (Oct. 1990)
- Post BPTs (Mar. 1991)
- Cased and uncased BPTs (Sept. 1991)
- SPT Locations all

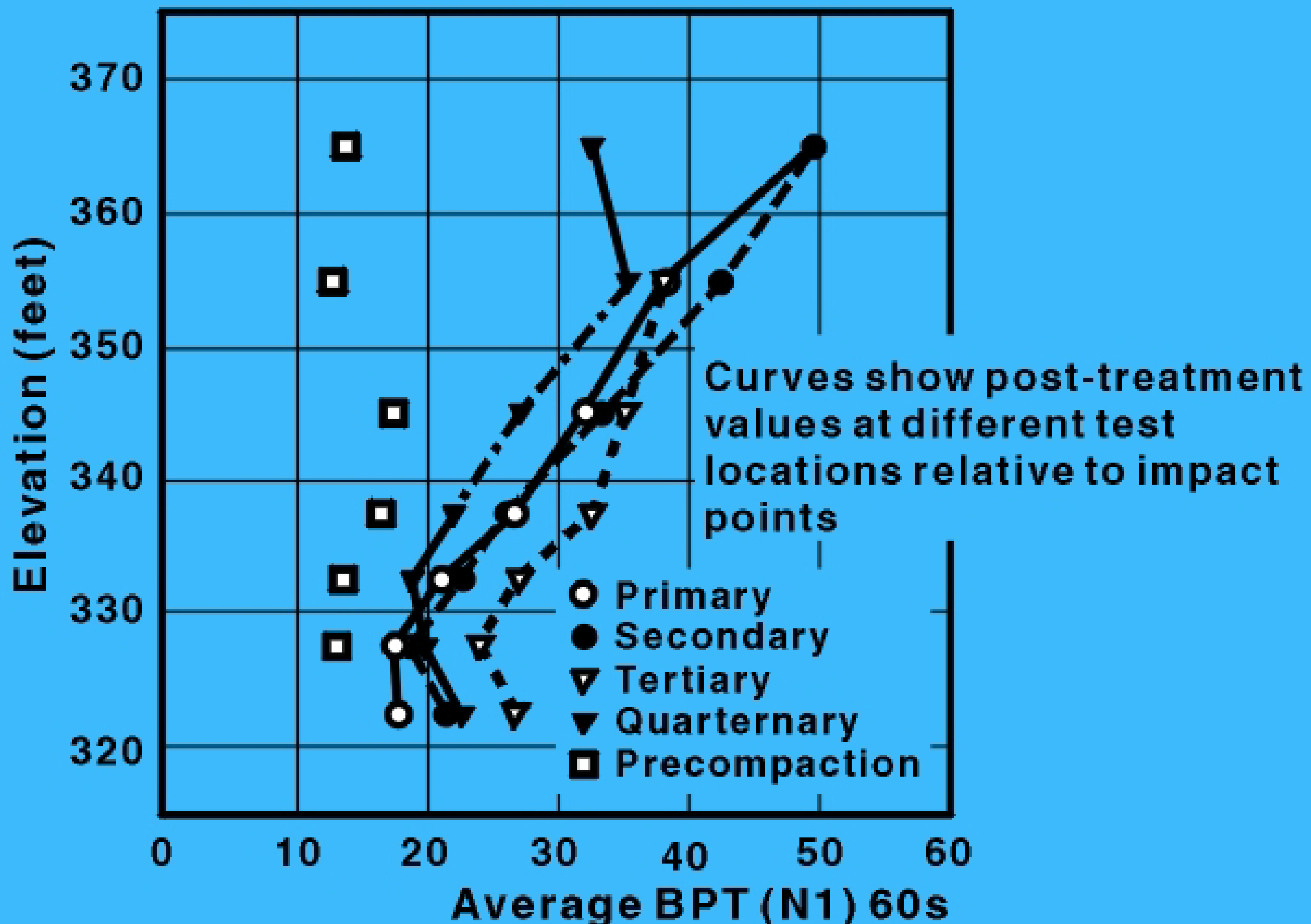
BPT = Becker Penetration Test: 168 mm double–wall casing driven using diesel pile driving hammer. (used in soils containing gravel and cobbles)

# Deep Dynamic Compaction

| | |
|---|---|
| Treatment zone | 800 x 150 ft |
| Steel drop weight | 35 tons (6.5 ft Ø) |
| Drop height | 108 ft ⟹ 98.4 ft free fall |

# Dynamic Compaction Program

| Coverage | Spacing | Number of Drops |
|---|---|---|
| Primary | 50 ft c-c | 30 |
| Secondary | Split primary | 30 |
| Tertiary | Split secondary | 15 |
| Ironing | Edge to edge | 2 @ 30 ft drop |




# Effect of Location on BPT Resistance

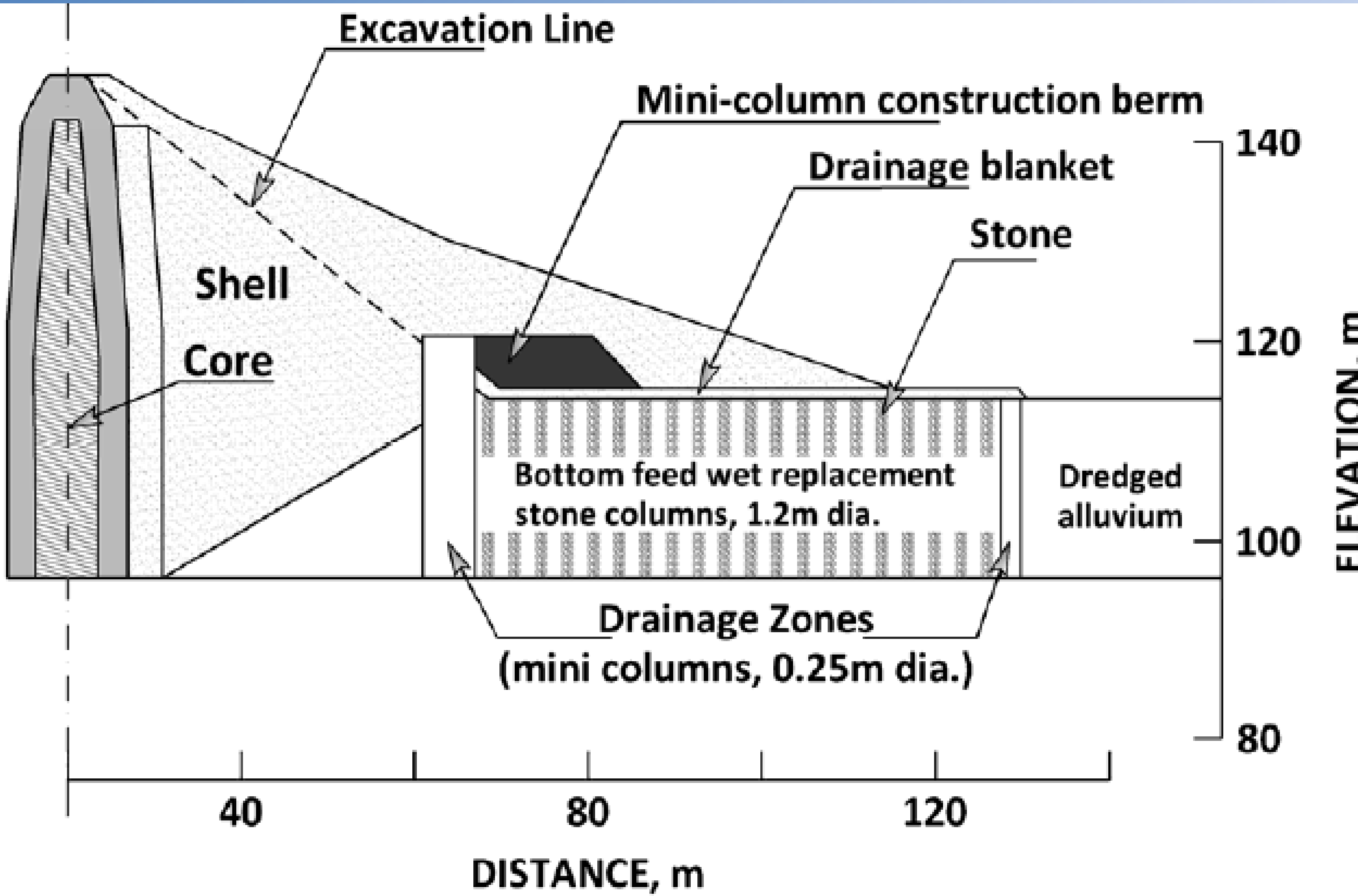
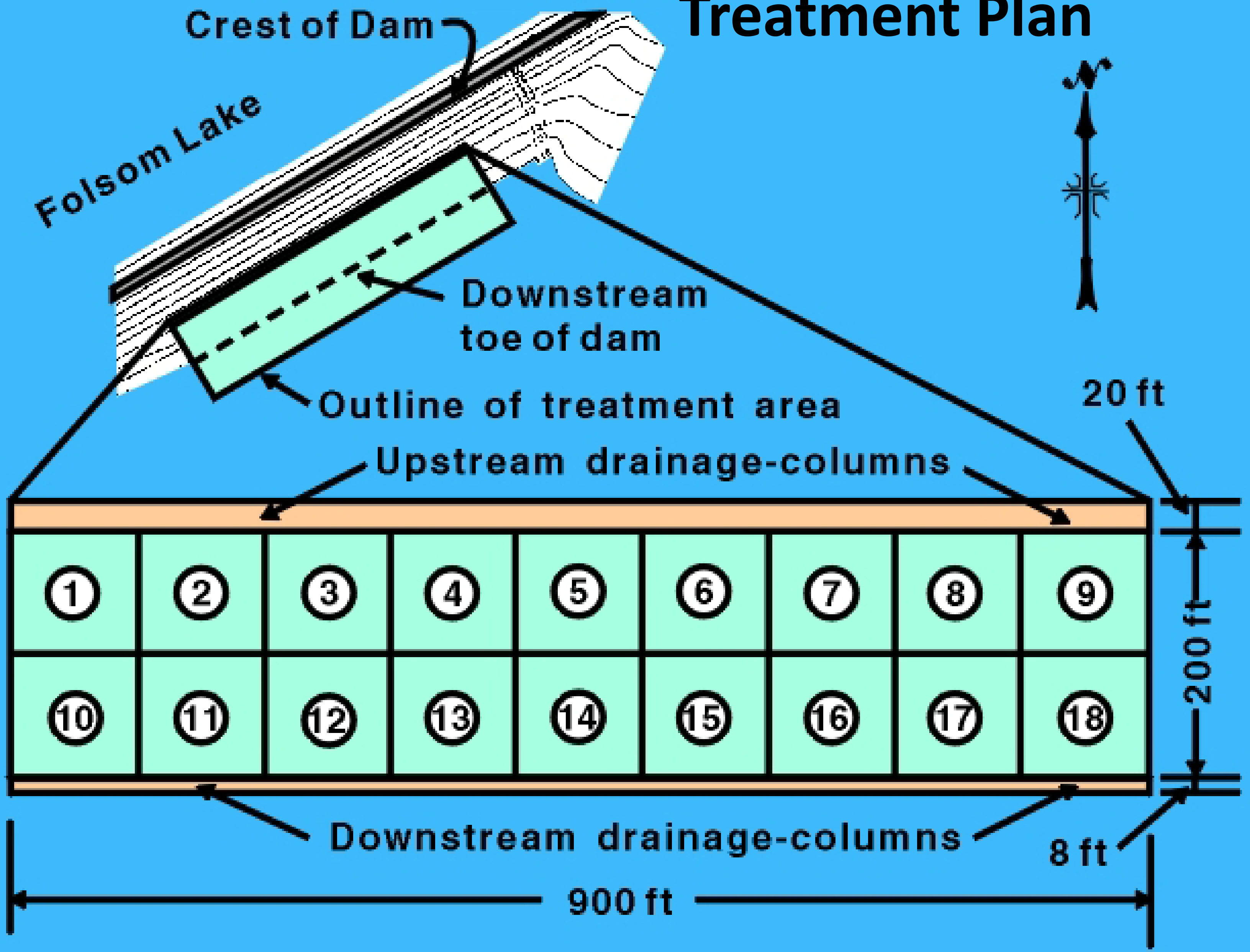



**Fig. 7. Ground improvement design for stabilization of the downstream embankment of Mormon Island Auxiliary Dam done in 1993-94.**

# Treatment Plan

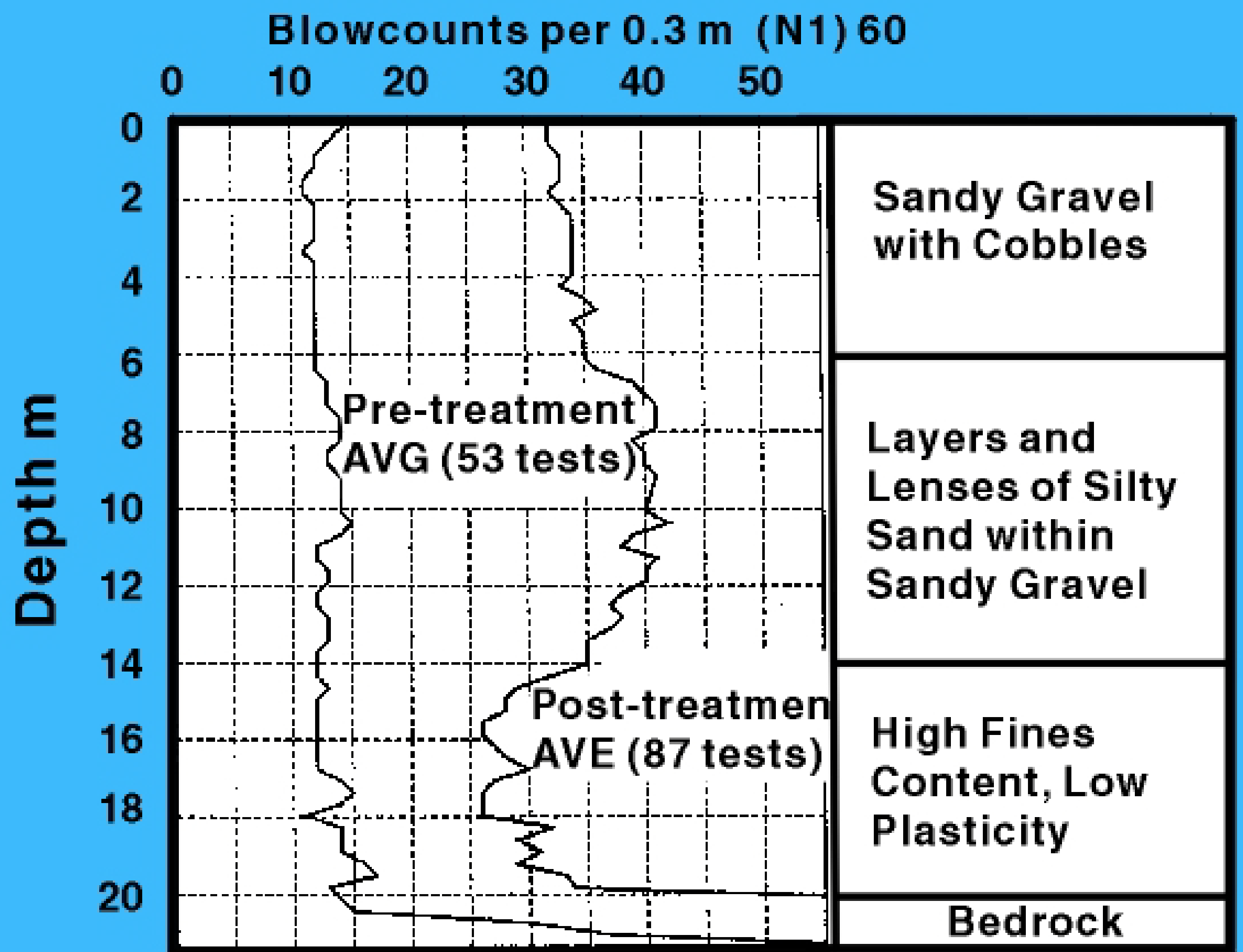


# Installation of bottom-feed wet replacement method stone columns at Mormon Island Auxiliary Dam (1994)

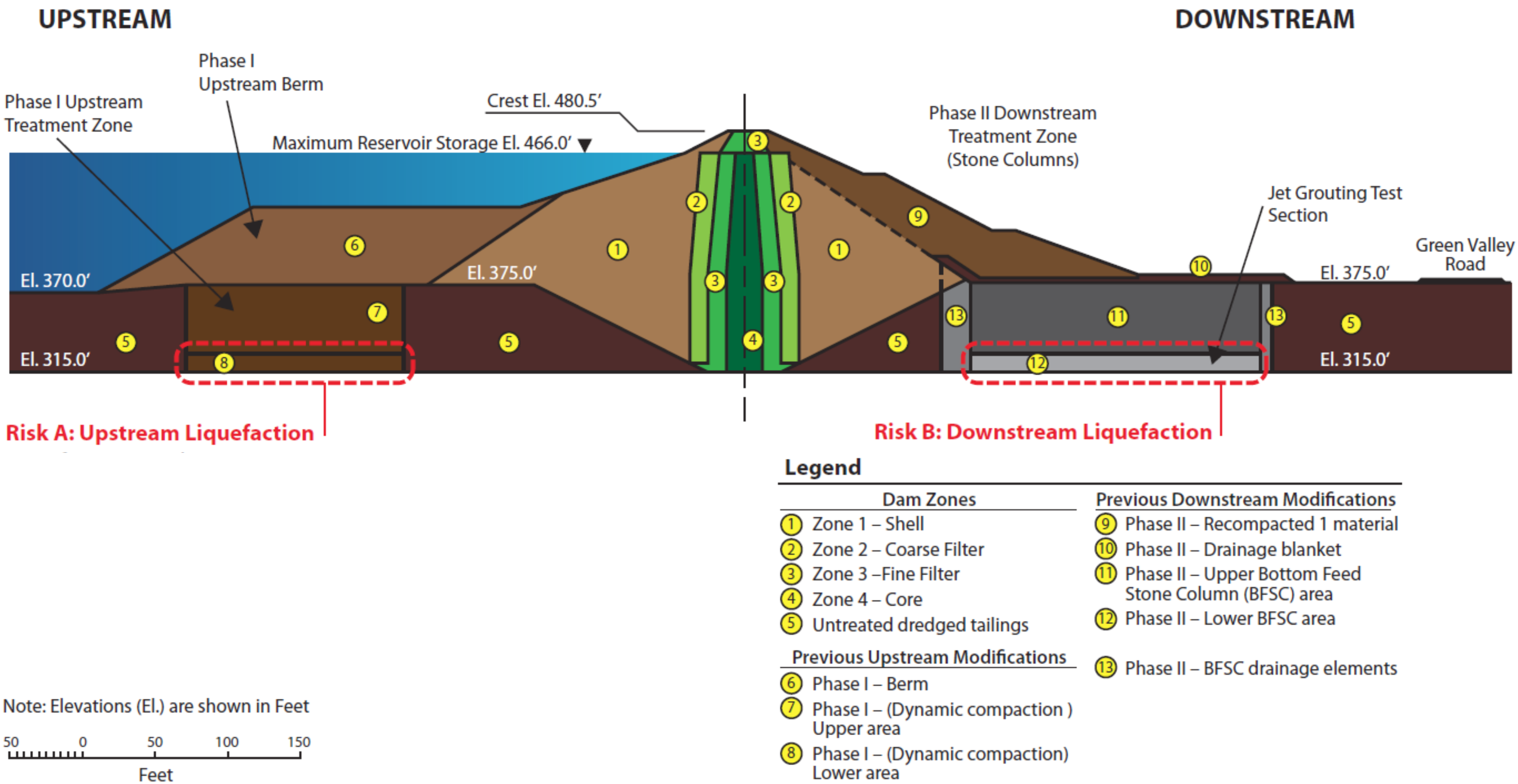
(Note temporary steepening of downstream embankment slope

to provide level working platform)


Stone column
construction at MIAD




Becker Penetration Test Results




# MIAD after modifications completed in 1994

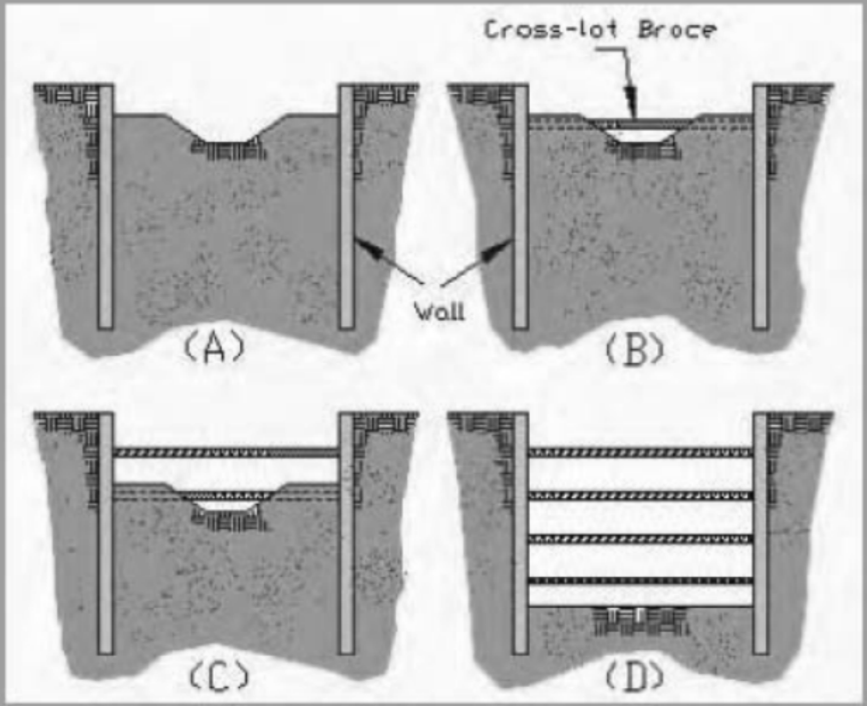
(adapted from U.S. Bureau of Reclamation, 2010)

## MIAD – The Story Continued Starting in 2004

- Reevaluations – greater seismic and hydrologic risk, larger population at risk (increased demand).
- Residual liquefaction risk beneath upstream DDC zone (decreased resistance to stability failure).
- Adequacy of lower portion of downstream vibro-replacement treated zone could not be demonstrated.
- Concluded that liquefaction still possible within lower part of the stone column treatment zone.
- Risk analysis indicated Annual Failure Probability and Annualized Life Loss above guideline values.

# SELECTED CORRECTIVE ACTIONS

- No further upstream treatment planned. Therefore needed downstream modifications to prevent loss of freeboard and overtopping or global stability failure if the upstream embankment fails.
- Analyses showed that a high strength foundation keyblock along the stone column treatment zone and a properly filtered blanket stability berm over the downstream embankment slope could provide the required resistance.
- Jet Grouting was initially proposed for construction of the keyblock.
- Test program indicated that JG treatment could not provide the needed strength and continuity.
- Decided on a cellular excavation with replacement by a concrete shear block along the downstream toe and a blanket stability berm over the downstream embankment slope.




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)

Photo 4. Rock surface at the bottom of Cell C.

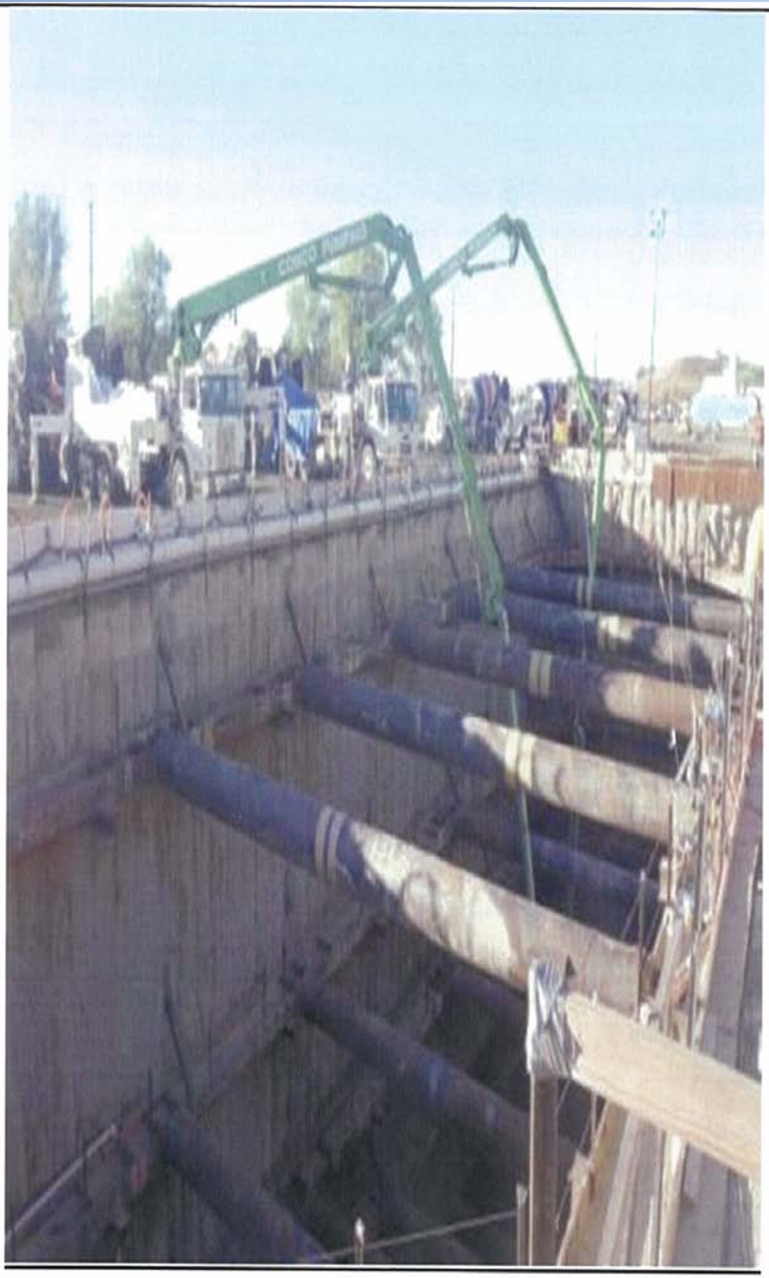


Photo 11. Concrete pump trucks delivering concrete to Cell C.

Sliding resistance along the block – rock interface is a major consideration.

# Construction of Downstream Filter, Drain and Overlay Fill, July 2014

# CURRENT STATUS

**Phase 1** (Completed February 2013)

- The concrete keyblock is 56.6 feet (17.3m) wide (including 1.0m thick secant pile containment walls along upstream and downstream sides) , 900ft (274m) long and 40-ft (12.2m) thick extending from a depth of 35 ft (10.7m) to bedrock at 75ft (22.9m) depth.
- Cost of $25,000,000.

**Phase 2**

- Downstream overlay buttress fill of sand, gravel, and crushed rock. Transition and filter zones between existing downstream embankment slope and new fill.
- Contract award of $45,719,235
- Now (2014) under construction; estimated completion in 2016.

# WHAT ARE SOME TAKE AWAY LESSONS FROM THESE TWO PROJECTS?

# GEOTECHNICAL ENGINEERING FOR DAMS

- Seismic considerations were minimal for dams constructed before the 1960's.
- The demands have increased over time; i.e., seismicity and probable maximum flood estimates have gone up.
- Populations at risk have increased downstream of many existing dams.
- Potential Failure Mode Analyses and Risk Assessments provide major inputs for evaluating existing dams, for deciding to take corrective action, and for prioritizing projects.
- Potential consequences of climate change, increasing number and magnitude of extreme events (floods, storms, earthquakes, fires, etc.) should be considered.
- Getting it right the first time can be very difficult given the unknowns and uncertainties at the time of initial design and construction.

# REMEDIATION STRATEGIES

- Simpler is better
- Focus is downstream (upstream work requires reservoir drawdown and/or working over and through water)
- Can allow upstream failure if downstream is buttressed to prevent excess loss of freeboard (and can demonstrate this by suitable analysis)
- Excavate and replace plus a downstream overlay or buttress fill is simple and reliable – but may involve an elevated failure risk during construction
- Dynamic deformation analyses are now widely used
- 3-D analyses increasingly used
- A reasonable, but sometimes unattainable, goal: bring a dam to a state that is as safe as if it were being designed and built today.

# SITE AND MATERIAL CHARACTERIZATION

- The validity and reliability of all analyses and predictions hinge on proper knowledge of the subsurface materials and their boundaries and how the relevant properties are assigned.
- Review original information about the soils and their properties carefully and critically – not all the interpretations may have been correct.
- Incorrect identification and characterization of materials can lead to an overestimate of the needed extent of ground improvement and unnecessary extra cost. (Could be unconservative also)

# GROUND IMPROVEMENT

- Vibro-replacement use in dam foundations is decreasing.
- Use of deep soil mixing is increasing.
- The promise of Jet Grouting for use in dam foundations is yet to be realized.
- What you can see and measure is invariably a better and more reliable option than what you can't - provided cost and construction risks are acceptable.
- Consideration starting to be given to sustainability issues (energy consumption and greenhouse gas emissions).

# Some Continuing and Unresolved Problems

- Interpreting and understanding the results of a risk analysis.
- Deciding the acceptable level of risk.
- Assessing true liquefaction potential of soils containing gravel and cobbles.
- Assessing liquefaction potential of silty soils.
- Assessing post-earthquake residual strength of liquefied soil.
- Assessing compliance with specifications.
- Selecting and implementing the appropriate soil constitutive model for liquefaction and dynamic deformation analyses.
- Assessing the reliability and accuracy of dynamic deformation analyses – is the "factor of 2 rule" overly conservative?
- Anticipating future increases in demand.

# CONCLUDING COMMENTS

- Assuring the safety of existing embankments dams will continue as a major problem in geotechnical engineering given increased demands resulting from greater probable flood and earthquake loading, increasing populations at risk, the need to protect vital components of infrastructure, the failure of many dams to meet current design criteria.
- Extreme events (earthquakes, floods) can dominate remedial designs – but traditional considerations; e.g, protection against uncontrolled seepage and piping, stability, maintenance of adequate freeboard, slope protection – must always be assured.
- Getting the corrective action right the first time can be difficult.
- Proper site and material characterization is essential.
- Try to make an existing deficient dam as safe as if you were starting a new project today.

# CONCLUDING COMMENTS (Cont.)

•Victor de Mello presented five embankment dam Design Principles in his 1977 Rankine Lecture, referred to by both John Burland and Harry Poulos in their De Mello Lectures (2008 and 2010).

They are essentially as follows:

DP1 – Aim to design out any risk from behavior triggered by local phenomena – *Robustness.*

DP2 – Use a dominant feature to cut across uncertainties – *Change the problem.*

DP3 – Aim at homogenization – *Redundancy.*

DP4 – Minimize rapid uncontrolled loading – *Observational control.*

DP5 – Question each design assumption and the consequences of departure from it – *Ask "what if" questions.*

•These same principles are generally applicable to the two cases described here, although DP4 should be modified to "Minimize the ***effects of*** rapid uncontrolled loading."

THANK YOU FOR YOUR ATTENTION AND FOR THE OPPORTUNITY TO BE HERE!