Embankment Seismic Performance

Best Practices in Dam and Levee Safety Risk Analysis

Part D – Embankments and Foundations

Chapter D-8

Last modified June 2017, presented July 2019

US Army Corps of Engineers®

Objective

- To develop an understanding of the general procedure for evaluating embankment performance during seismic loading.

Key Concepts

- Coincident seismic event and reservoir loading
- Liquefaction
- Deformation and breach
- Cracking causing erosion and breach

Outline

- Important case histories
- Steps for risk assessment
- Seismic potential failure modes
- Loading considerations
- Likelihood of liquefaction or cyclic softening/strength loss
- Embankment deformation
- Internal erosion through cracks

Outline

- Important case histories
- Steps for risk assessment
- Seismic potential failure modes
- Loading considerations
- Likelihood of liquefaction or cyclic softening/strength loss
- Embankment deformation
- Internal erosion through cracks

Failure due to Earthquakes

- Only about 1.5 percent of historical failures of embankment dams have been attributed to earthquakes.

Foster et. al (2000)

Mode of failure	No. of cases: All failures	No. of cases: Failures in operation	% failures (where known): All failures	% failures (where known): Failures in operation	Average frequency of failure ($\times 10^{-3}$): All failures	Average frequency of failure ($\times 10^{-3}$): Failures in operation
Earthquake–liquefaction	2	2	1.6	1.7	0.18	0.18
Unknown mode	8	7				
Total no. of failures	136	124			12.2 (1.2%)	11.1 (1.1%)
Total no. of failures where mode of failure known	128	117				
No. of embankment dams	11 192	11 192				

Sheffield Dam 1925

Lower Sand Fernando Dam 1971

(Photos Courtesy of National Information Service for Earthquake Engineering, University of California, Berkeley, Karl Steinbrugge Collection)

Figure 10: View of Breach in Fujinuma Main Dam from Left Abutment Looking Upstream (N37.3021°, E140.1952°, April 23, 2011)

GEER Association Report No. GEER-25e - Preliminary

Figure 14: View of Fujinuma Dam Embankment Fill Layers On Top of Black Organic Foundation Residual Soil Exposed within Downstream Slope at Left Abutment (N37.3024°, E140.1951°, April 23, 2011)

GEER Association Report No. GEER-25e - Preliminary

Fujinuma Dam 2011

Figure 2: Photograph of Major Slumping of Landside Slope of Naruse River Right Levee and Approach Road at River Kilometer 30.0 (N38.5307, E 141.0064, April 20, 2011) – see Figure 8 for additional views and aerial image

Figure 27: View of Slumped Hinuma River Left Levee Induced by Liquefaction (N36.2861, E140.5238., April 24, 2011)

GEER Association Report No. GEER-025b

Naruse and Hinuma Levees 2011

Outline

- Important case histories
- <u>Steps for risk assessment</u>
- Seismic potential failure modes
- Loading considerations
- Likelihood of liquefaction or cyclic softening/strength loss
- Embankment deformation
- Internal erosion through cracks

General Steps for Evaluation of Seismic Risks for Embankments

- Evaluate site conditions and develop representative characterization of the embankment and foundation materials
- Develop detailed site-specific failure modes
- Develop event trees to assess failure modes
- Establish loading conditions for earthquake ground motions and associated magnitudes, as well the coincident reservoir level
- Perform a screening by evaluating the load combinations and site characteristics to determine if seismic potential failure modes will be significant risk contributors
- If seismic potential failure modes are significant risk contributors, conduct more rigorous evaluations of liquefaction, deformation, cracking, etc.…
- Evaluate consequences

Outline

- Important case histories
- Steps for risk assessment
- Seismic potential failure modes
- Loading considerations
- Likelihood of liquefaction or cyclic softening/strength loss
- Embankment deformation
- Internal erosion through cracks

Most Important Issue - Liquefaction

- Liquefaction occurs when earthquake shaking causes water pressure to increase in soils thus greatly reducing the shear strength of the soil.
- Saturated, clean, loose cohesionless or uncompacted materials are most susceptible.
- Liquefiable soils are common to alluvial valleys, where earth embankments are typically built.

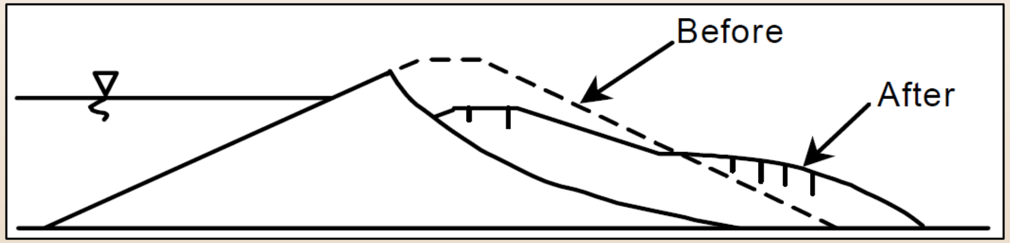
Photos from National Information Service for Earthquake Engineering, University of California, Berkeley

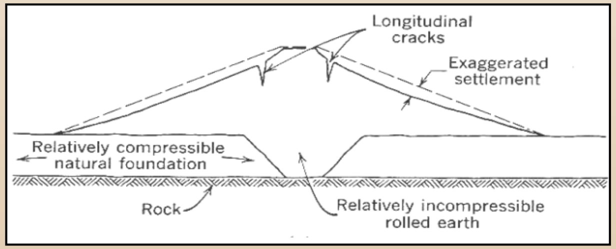
Other Important Issue: Cyclic Softening/Strength loss

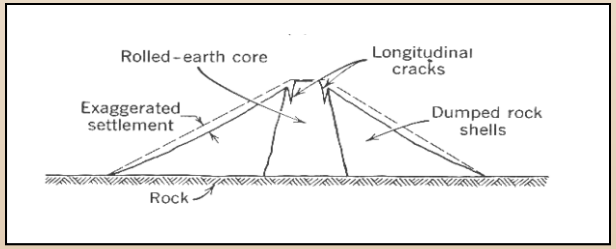
- Strength loss can occur in soft or sensitive clays
- These type of soils are common in lacustrine or glacial-lacustrine environments.
- Normally consolidated to slightly over consolidated clays can have low seismic shear strengths and can lose significant strength if the earthquake-induced strains are large enough

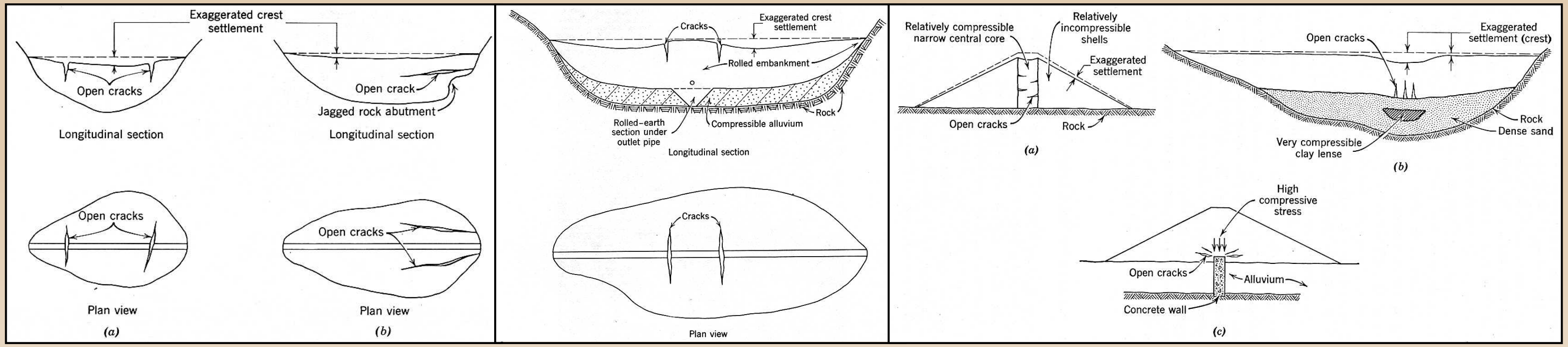
Typical Potential Failure Modes

- Overtopping erosion due to deformation exceeding the available freeboard
  - Liquefaction and non-liquefaction
- Internal erosion due to transverse cracking
  - See characteristics susceptible to cracking described in D-6 (Internal Erosion)
  - Liquefaction or cyclic softening and sliding









Cracking due to Fault Rupture

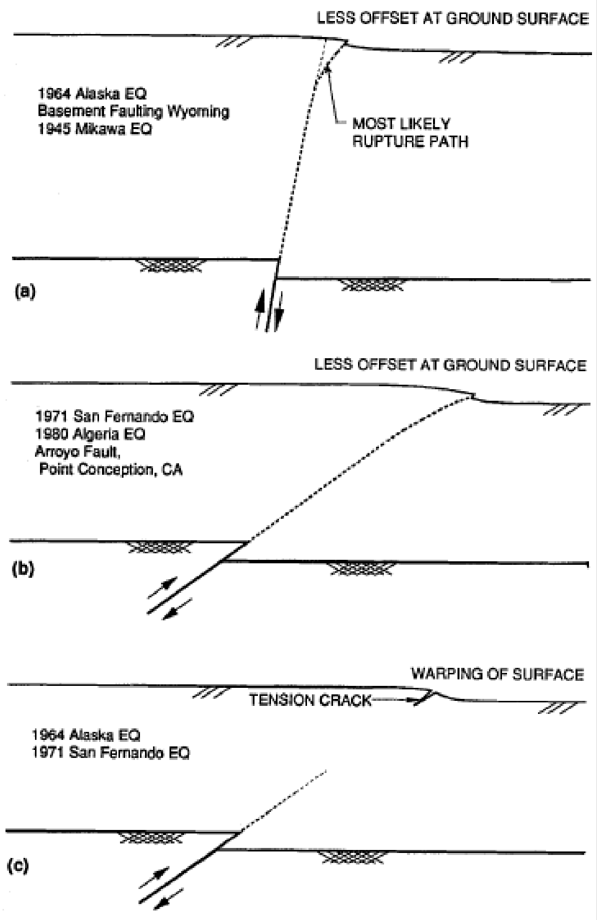


FIG. 6. Path of Reverse Fault Rupture through Soil: (*a*) Stiff Earth Materials, Stee Dip; (*b*) Stiff Earth Materials, Shallow Dip; (*c*) Ductile Earth Materials

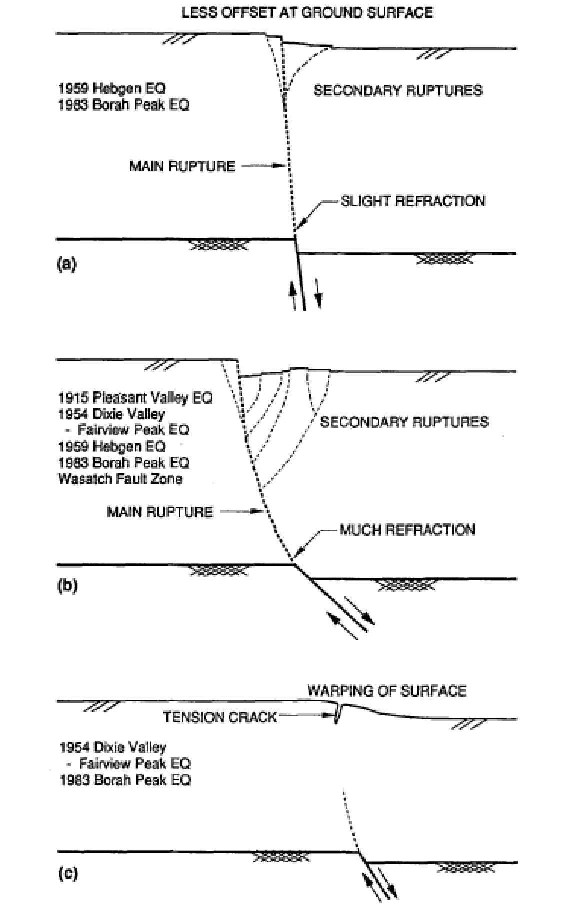


FIG. 7. Path of Normal Fault Rupture through Soil: (*a*) Stiff Earth Materials, Steep Dip; (*b*) Stiff Earth Materials, Shallow Dip; (*c*) Ductile Earth Materials

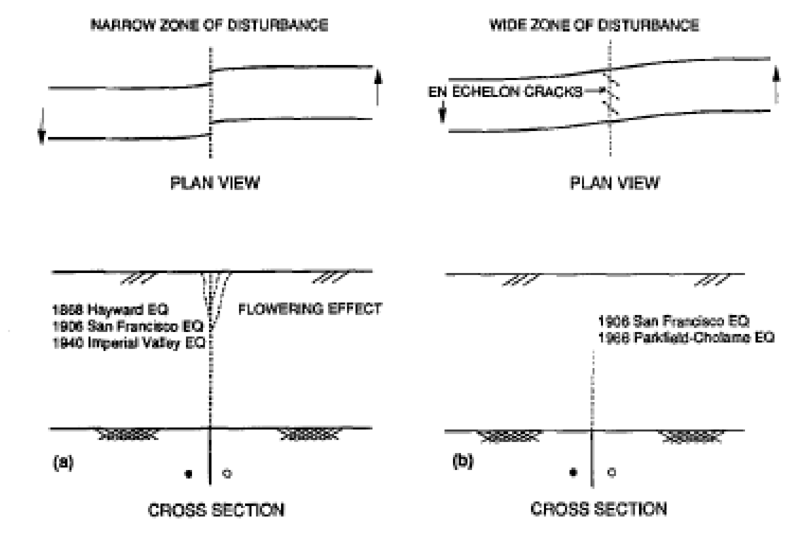


FIG. 8. Path of Strike-Slip Fault Rupture through Soil: (*a*) Stiff Earth Materials; (*b*) Ductile Earth Materials

Source: Bray et al. (1994)

Cracking due to Embedded or Adjacent Structures

- At conduit contacts
  - Typically, located deep in the embankment and thus cracks may close due to confining pressures
- At spillway wall contacts
  - Separation in these areas observed in case histories
  - Typically, transverse orientation
- At concrete/embankment wrap-around sections
  - Similar behavior to the other structure contacts
  - Typically, more circuitous seepage path

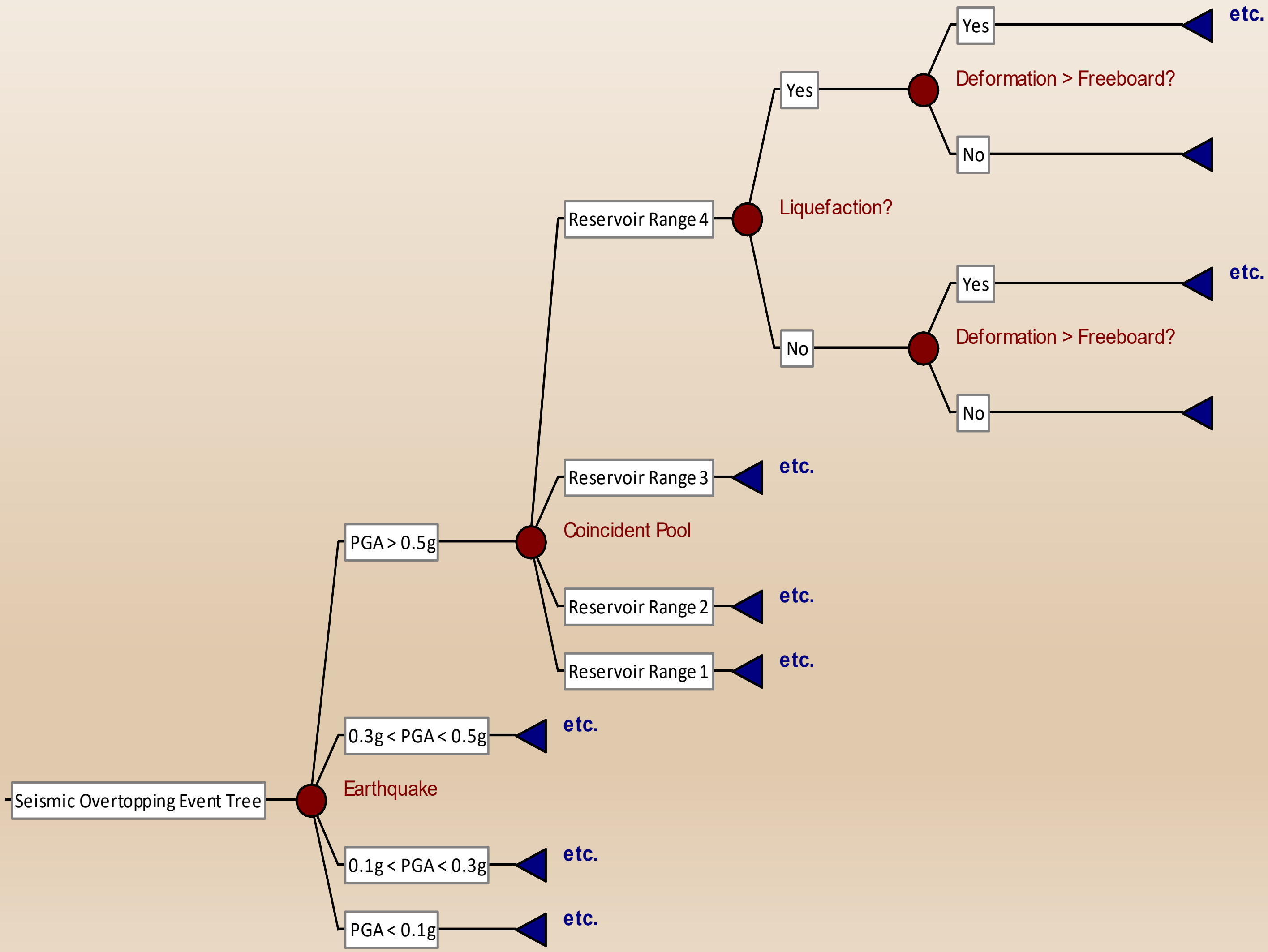
Steps for Evaluation of Seismic Potential Failure Modes

- Estimate the likelihood of liquefaction or cyclic softening and sliding of any foundation or embankment materials;
- Estimate the residual shear strength of the materials that may liquefy or may experience strength loss;
- Estimate the deformation of the embankment.

Complete the Event Tree Similar to Static Potential Failure Modes

- For overtopping erosion, assess the likelihood of failure due to overtopping erosion for various depths of overtopping, as described in Chapter D-3.
- For internal erosion, assess the potential cracking characteristics (location and size) and estimate the likelihood of failure due to concentrated leak erosion as a function of the earthquake and coincident water level, as described in Chapter D-8.

Sample Event Tree for Seismic Crest Deformation

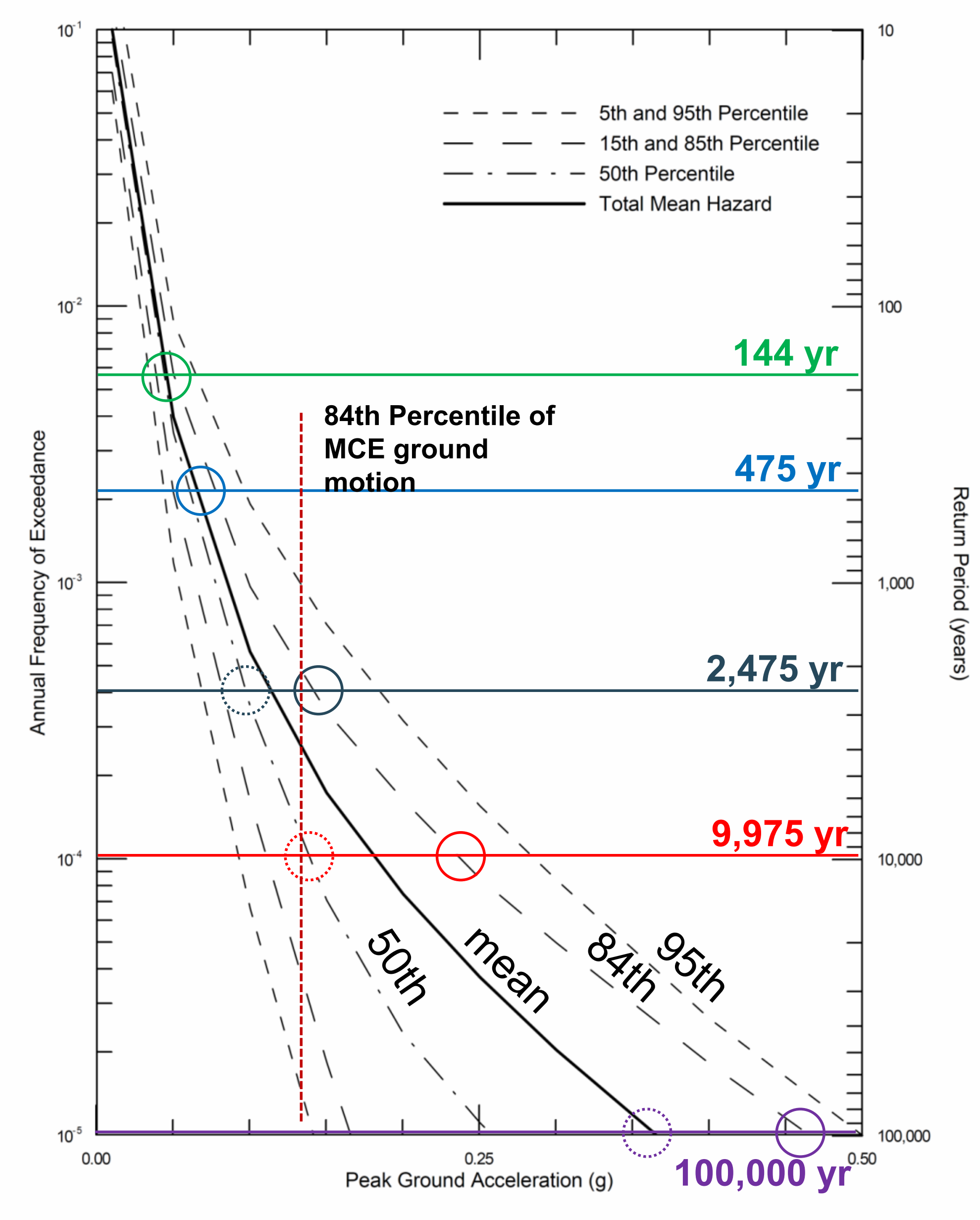


Outline

- Important case histories
- Steps for risk assessment
- Seismic potential failure modes
- Loading considerations
- Likelihood of liquefaction or cyclic softening/strength loss
- Embankment deformation
- Internal erosion through cracks

Seismic Loading Considerations

- Annual exceedance plots to develop load partitions for input to event tree
- Acceleration time histories also used
- Deaggregations needed for magnitude with greatest contribution to the loading partition.
  - USGS has a number of useful tools



Reservoir / River / Coastal Loading Considerations

- For water supply embankments use of the percentage of time exceedance plot to develop partitions (or as input into event tree), consider the normal operating level or percentage of time above a threshold level.
- For flood control embankments and levees with large fluctuations in reservoir, river or coastal water levels, failure modes are also a function of the coincident water level
  - Use stage-duration relationship for frequencies
  - SRP = f(PGA or M_w, Coincident Pool)

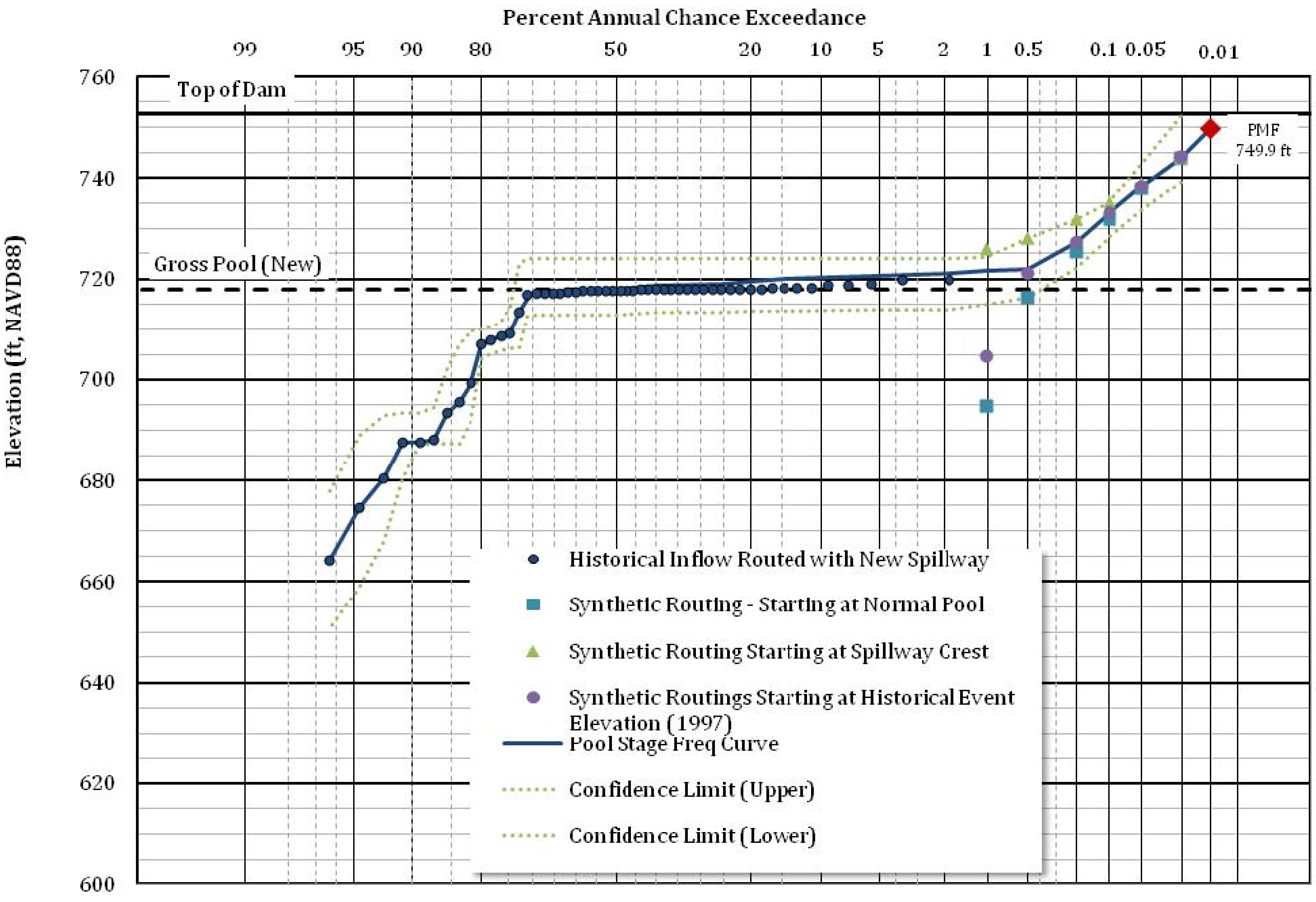


Figure 5-2. Best case, worst case, and best estimate pool frequency relationships

Example of Failure Mode Screening based on Joint Loading Probability

APF ≈ Σ (Annual frequency of an earthquake within PGA partition) (Fraction of a year for reservoir within partition) (Assumed SRP = 1)

					184.0	189.0	197.5	200.0	205.0	216.5	228.5	228.6	234.0	$Pool_A$ (g)
					186.0	197.5	200.0	205.0	216.5	228.5	228.6	234.0	239.0	$Pool_B$ (g)
					1.00E+00	7.75E-02	1.91E-02	6.37E-03	2.40E-04	2.01E-05	1.51E-06	1.48E-06	4.63E-07	P_A
					2.11E-01	1.91E-02	6.37E-03	2.40E-04	2.01E-05	1.51E-06	1.48E-06	4.63E-07	1.57E-07	P_B
PGA_A (g)	PGA_B (g)	AEP_A	AEP_B	P_{AB}	7.89E-01	5.84E-02	1.27E-02	6.13E-03	2.20E-04	1.86E-05	3.23E-08	1.02E-06	3.05E-07	P_{AB}
0.000	0.050	9.26E-01	7.37E-02	8.53E-01	6.72E-01	4.98E-02	1.09E-02	5.23E-03	1.88E-04	1.59E-05	2.75E-08	8.69E-07	2.60E-07	
0.050	0.100	7.37E-02	3.04E-02	4.33E-02	3.41E-02	2.53E-03	5.52E-04	2.65E-04	9.53E-06	8.06E-07	1.40E-09	4.41E-08	1.32E-08	
0.100	0.150	3.04E-02	1.62E-02	1.41E-02	1.12E-02	8.27E-04	1.80E-04	8.67E-05	3.12E-06	2.64E-07	4.57E-10	1.44E-08	4.32E-09	
0.150	0.200	1.62E-02	9.98E-03	6.24E-03	4.92E-03	3.65E-04	7.96E-05	3.83E-05	1.38E-06	1.16E-07	2.02E-10	6.37E-09	1.91E-09	
0.200	0.250	9.98E-03	6.90E-03	3.08E-03	2.43E-03	1.80E-04	3.93E-05	1.89E-05	6.79E-07	5.74E-08	9.96E-11	3.14E-09	9.41E-10	
0.250	0.300	6.90E-03	4.95E-03	1.96E-03	1.54E-03	1.14E-04	2.49E-05	1.20E-05	4.31E-07	3.64E-08	6.32E-11	1.99E-09	5.97E-10	
0.300	0.400	4.95E-03	2.88E-03	2.07E-03	1.63E-03	1.21E-04	2.64E-05	1.27E-05	4.56E-07	3.85E-08	6.69E-11	2.11E-09	6.32E-10	
0.400	0.500	2.88E-03	1.86E-03	1.02E-03	8.02E-04	5.94E-05	1.30E-05	6.24E-06	2.24E-07	1.89E-08	3.29E-11	1.04E-09	3.10E-10	
0.500	0.600	1.86E-03	1.24E-03	6.24E-04	4.92E-04	3.64E-05	7.95E-06	3.82E-06	1.37E-07	1.16E-08	2.02E-11	6.36E-10	1.90E-10	
0.600	0.700	1.24E-03	8.52E-04	3.85E-04	3.04E-04	2.25E-05	4.91E-06	2.36E-06	8.48E-08	7.17E-09	1.24E-11	3.93E-10	1.18E-10	
0.700	0.800	8.52E-04	5.92E-04	2.60E-04	2.05E-04	1.52E-05	3.31E-06	1.59E-06	5.72E-08	4.84E-09	8.39E-12	2.65E-10	7.93E-11	
0.800	0.900	5.92E-04	4.26E-04	1.67E-04	1.32E-04	9.74E-06	2.13E-06	1.02E-06	3.67E-08	3.11E-09	5.39E-12	1.70E-10	5.09E-11	
0.900	1.000	4.26E-04	3.06E-04	1.20E-04	9.45E-05	7.00E-06	1.53E-06	7.35E-07	2.64E-08	2.23E-09	3.87E-12	1.22E-10	3.66E-11	
1.000	1.250	3.06E-04	1.41E-04	1.65E-04	1.30E-04	9.62E-06	2.10E-06	1.01E-06	3.63E-08	3.07E-09	5.32E-12	1.68E-10	5.03E-11	
1.250	1.500	1.41E-04	6.69E-05	7.40E-05	5.84E-05	4.32E-06	9.43E-07	4.54E-07	1.63E-08	1.38E-09	2.39E-12	7.55E-11	2.26E-11	
1.500	1.750	6.69E-05	3.34E-05	3.35E-05	2.64E-05	1.96E-06	4.27E-07	2.05E-07	7.37E-09	6.23E-10	1.08E-12	3.41E-11	1.02E-11	
1.750	2.000	3.34E-05	1.68E-05	1.67E-05	1.31E-05	9.73E-07	2.12E-07	1.02E-07	3.67E-09	3.10E-10	5.38E-13	1.70E-11	5.08E-12	
2.000	2.250	1.68E-05		#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	
2.250	2.500			#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	**Total**
2.500	3.000			#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	**APF**
					7.31E-01	5.41E-02	1.18E-02	5.68E-03	2.04E-04	1.73E-05	2.99E-08	9.45E-07	2.83E-07	**8.02E-01**

Example of Failure Mode Screening based on Joint Loading Probability

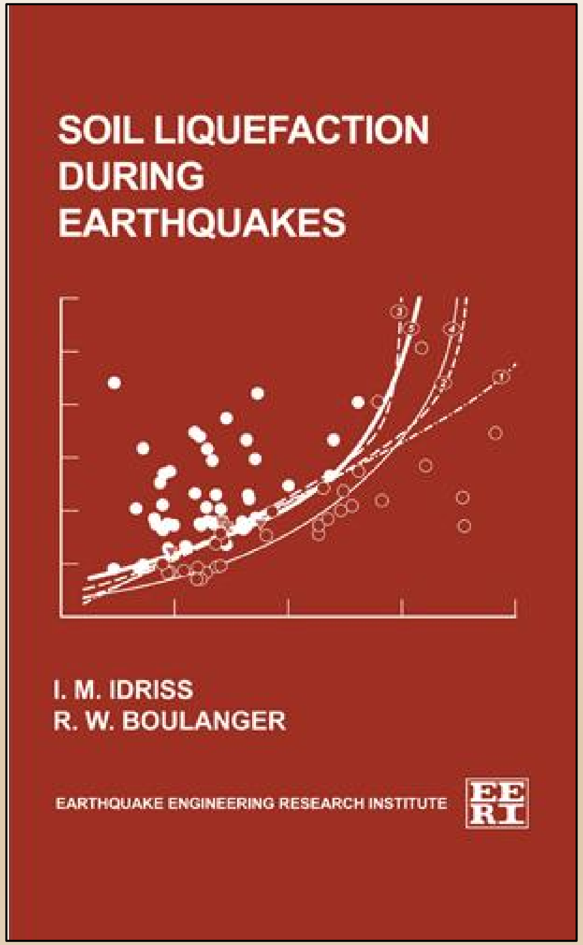
APF ≈ Σ (Annual frequency of an earthquake within PGA partition) (Fraction of a year for reservoir within partition) (Assumed SRP = 1)

					184.0	189.0	197.5	200.0	205.0	216.5	228.5	228.6	234.0	$Pool_A$ (g)
					186.0	197.5	200.0	205.0	216.5	228.5	228.6	234.0	239.0	$Pool_B$ (g)
					1.00E+00	7.75E-02	1.91E-02	6.37E-03	2.40E-04	2.01E-05	1.51E-06	1.48E-06	4.63E-07	P_A
					2.11E-01	1.91E-02	6.37E-03	2.40E-04	2.01E-05	1.51E-06	1.48E-06	4.63E-07	1.57E-07	P_B
PGA_A (g)	PGA_B (g)	AEP_A	AEP_B	P_{AB}	7.89E-01	5.84E-02	1.27E-02	6.13E-03	2.20E-04	1.86E-05	3.23E-08	1.02E-06	3.05E-07	P_{AB}
0.000	0.050	9.26E-01	7.37E-02	8.53E-01										
0.050	0.100	7.37E-02	3.04E-02	4.33E-02										
0.100	0.150	3.04E-02	1.62E-02	1.41E-02										
0.150	0.200	1.62E-02	9.98E-03	6.24E-03										
0.200	0.250	9.98E-03	6.90E-03	3.08E-03										
0.250	0.300	6.90E-03	4.95E-03	1.96E-03										
0.300	0.400	4.95E-03	2.88E-03	2.07E-03					4.56E-07	3.85E-08	6.69E-11	2.11E-09	6.32E-10	
0.400	0.500	2.88E-03	1.86E-03	1.02E-03					2.24E-07	1.89E-08	3.29E-11	1.04E-09	3.10E-10	
0.500	0.600	1.86E-03	1.24E-03	6.24E-04					1.37E-07	1.16E-08	2.02E-11	6.36E-10	1.90E-10	
0.600	0.700	1.24E-03	8.52E-04	3.85E-04					8.48E-08	7.17E-09	1.24E-11	3.93E-10	1.18E-10	
0.700	0.800	8.52E-04	5.92E-04	2.60E-04					5.72E-08	4.84E-09	8.39E-12	2.65E-10	7.93E-11	
0.800	0.900	5.92E-04	4.26E-04	1.67E-04					3.67E-08	3.11E-09	5.39E-12	1.70E-10	5.09E-11	
0.900	1.000	4.26E-04	3.06E-04	1.20E-04					2.64E-08	2.23E-09	3.87E-12	1.22E-10	3.66E-11	
1.000	1.250	3.06E-04	1.41E-04	1.65E-04					3.63E-08	3.07E-09	5.32E-12	1.68E-10	5.03E-11	
1.250	1.500	1.41E-04	6.69E-05	7.40E-05					1.63E-08	1.38E-09	2.39E-12	7.55E-11	2.26E-11	
1.500	1.750	6.69E-05	3.34E-05	3.35E-05					7.37E-09	6.23E-10	1.08E-12	3.41E-11	1.02E-11	
1.750	2.000	3.34E-05	1.68E-05	1.67E-05					3.67E-09	3.10E-10	5.38E-13	1.70E-11	5.08E-12	
2.000	2.250	1.68E-05		#VALUE!										
2.250	2.500			#VALUE!										**Total**
2.500	3.000			#VALUE!										**APF**
					#N/A	#N/A	#N/A	#N/A	1.09E-06	9.18E-08	1.59E-10	5.03E-09	1.50E-09	**1.18E-06**

Outline

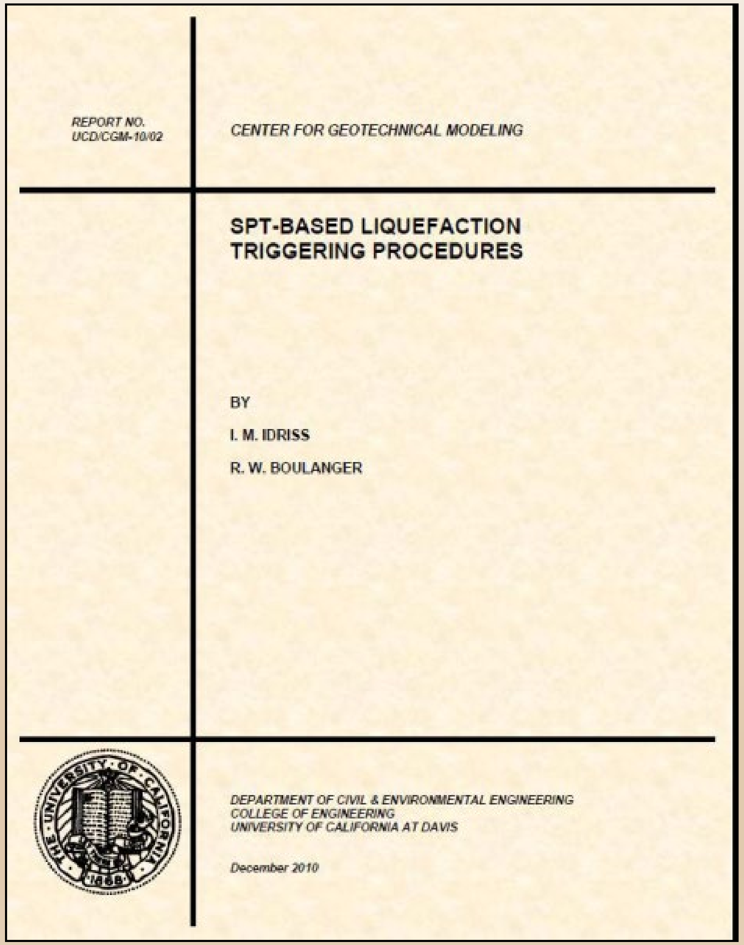
- Important case histories
- Steps for risk assessment
- Seismic potential failure modes
- Loading considerations
- Likelihood of liquefaction or cyclic softening/strength loss
- Embankment deformation
- Overtopping
- Internal erosion through cracks

Likelihood of Liquefaction


SOIL LIQUEFACTION DURING EARTHQUAKES

I. M. IDRISS
R. W. BOULANGER

EARTHQUAKE ENGINEERING RESEARCH INSTITUTE


REPORT NO. UCD/CGM-10/02

CENTER FOR GEOTECHNICAL MODELING

SPT-BASED LIQUEFACTION TRIGGERING PROCEDURES

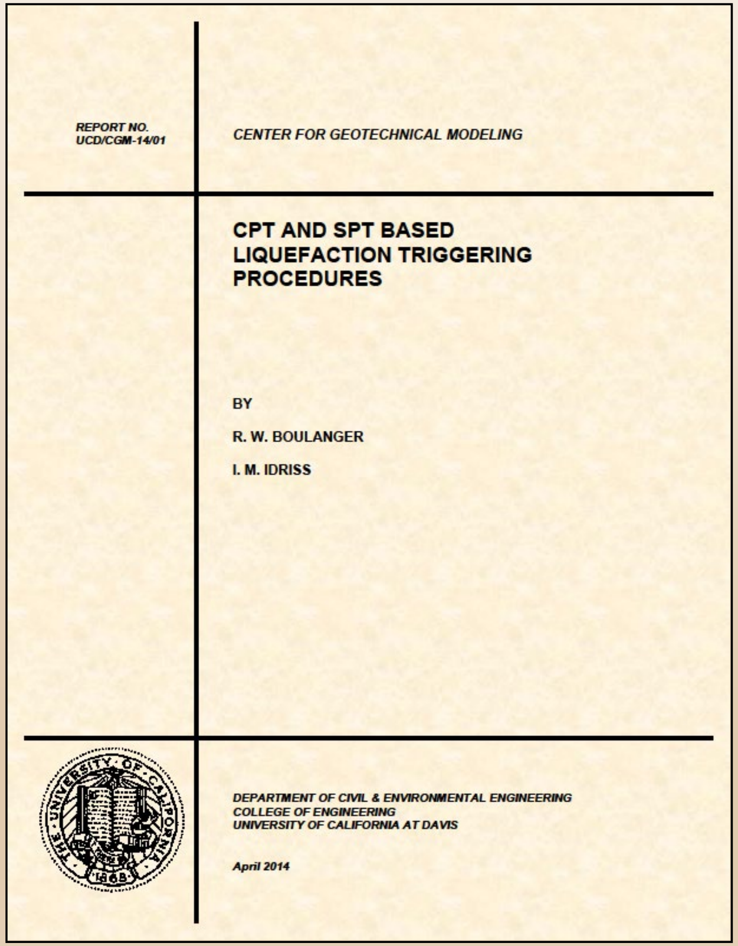
BY

I. M. IDRISS

R. W. BOULANGER

DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING
COLLEGE OF ENGINEERING
UNIVERSITY OF CALIFORNIA AT DAVIS

December 2010


REPORT NO. UCD/CGM-14/01

CENTER FOR GEOTECHNICAL MODELING

CPT AND SPT BASED LIQUEFACTION TRIGGERING PROCEDURES

BY

R. W. BOULANGER

I. M. IDRISS

DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING
COLLEGE OF ENGINEERING
UNIVERSITY OF CALIFORNIA AT DAVIS

April 2014

Probability of Liquefaction

Boulanger and Idriss (2010):

$$CSR_{M=7.5,\sigma'_v=1atm} = 0.65 \frac{\sigma_v}{\sigma'_v} \frac{a_{max}}{g} r_d \frac{1}{MSF} \frac{1}{K_\sigma}$$

$$P_L\left((N_1)_{60cs}, CSR_{M=7.5,\sigma'_v=1atm}\right) = \left[\frac{\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.67 - ln\left(CSR_{M=7.5,\sigma'_v=1atm}\right)}{0.13}\right]$$

Probability of no liquefaction: $P_{NL} = 1 - P_L$

Cyclic Softening of Clays

- Probability of cyclic softening can be calculated by comparing the CSR, which is based on differing earthquake return periods, to the cyclic resistance ratio (CRR) of clay.
- If CRR is less than CSR there is a high potential for strength loss
- CRR calculated as

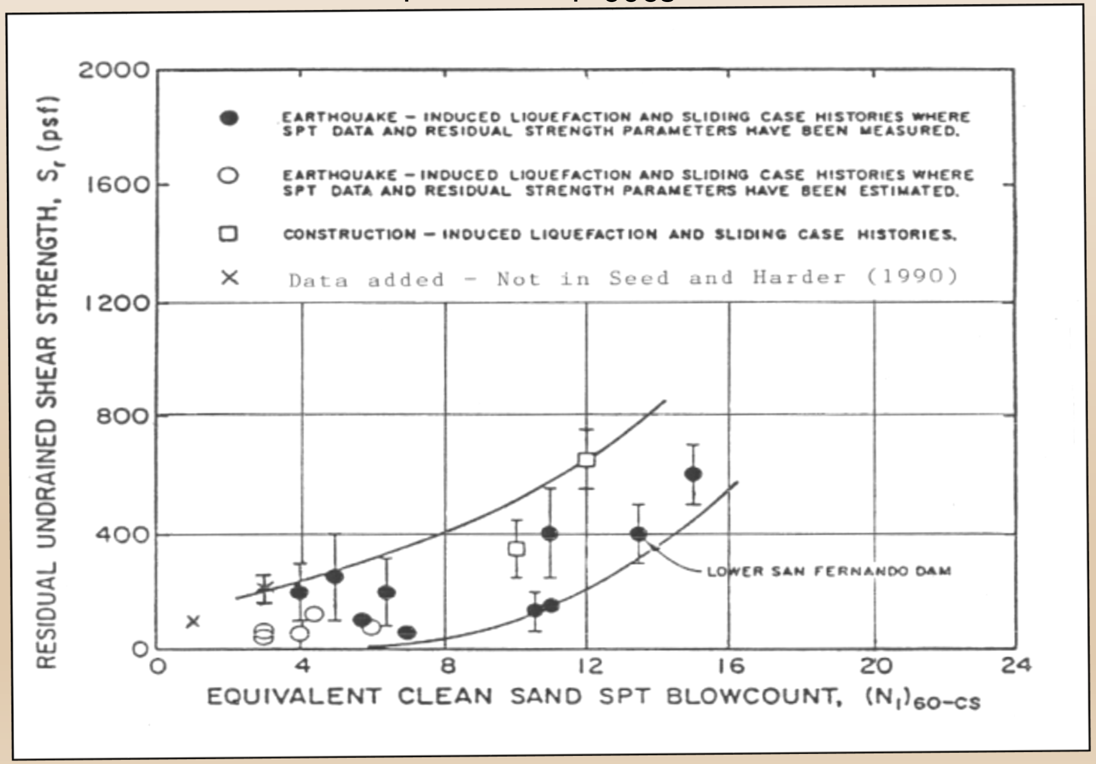
  CRR = 0.8*S*(OCR)^m

  (Idriss and Boulanger, 2008)
- Post seismic fully remolded strengths can be estimated using CPT, VST or by other empirical correlations such as using Liquidity Index

Residual Shear Strength of Liquefied Soil

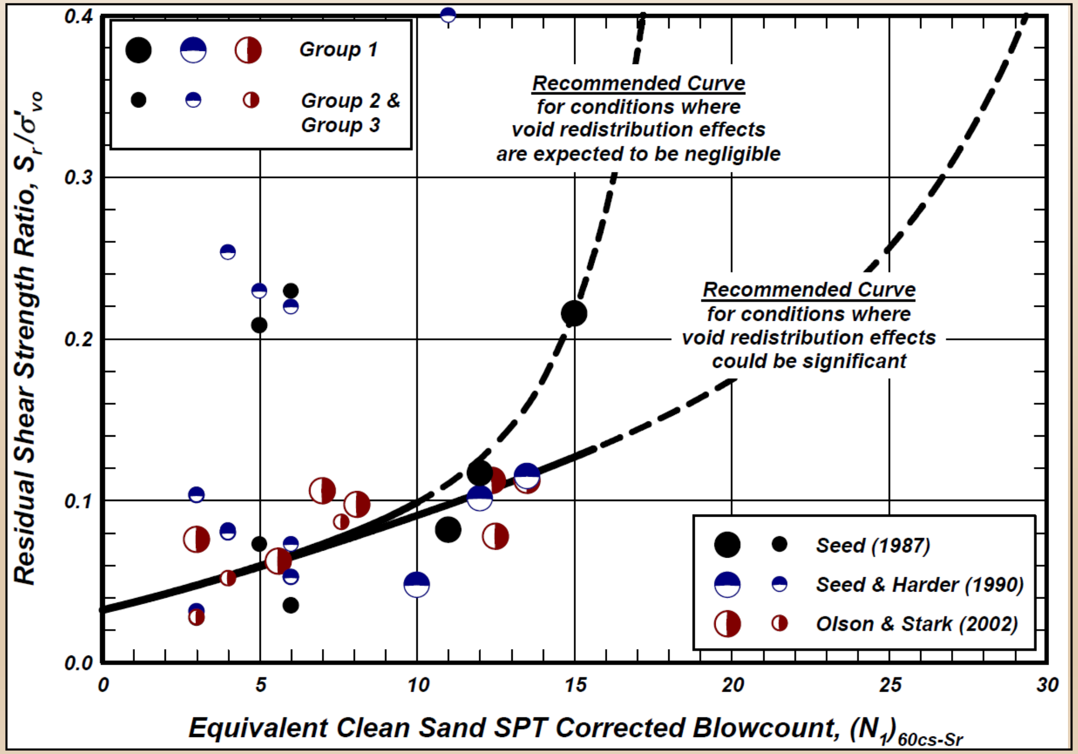
Seed and Harder (1990)

$S_r = f((N_1)_{60cs})$



Idriss and Boulanger (2008)

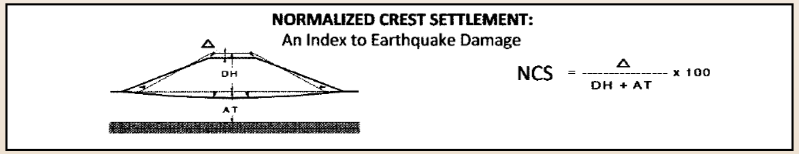
$S_r/\sigma'_{vc} = f((N_1)_{60\text{-}Sr})$

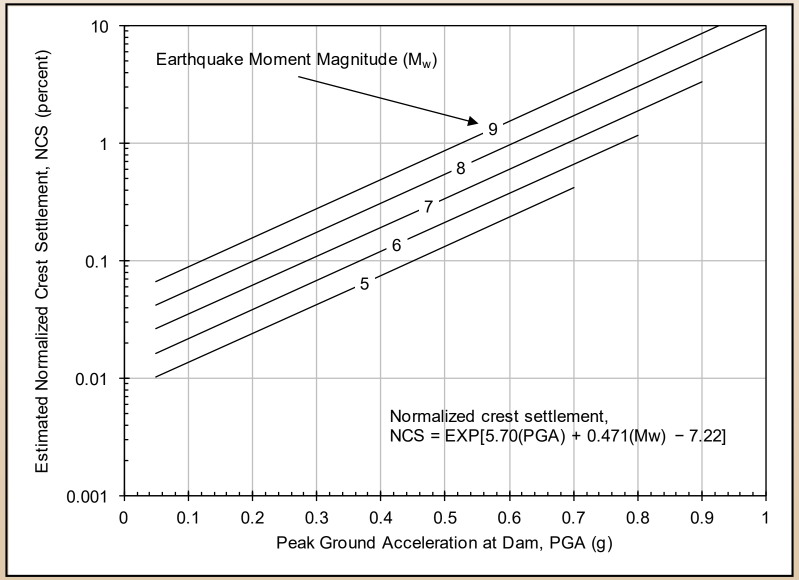


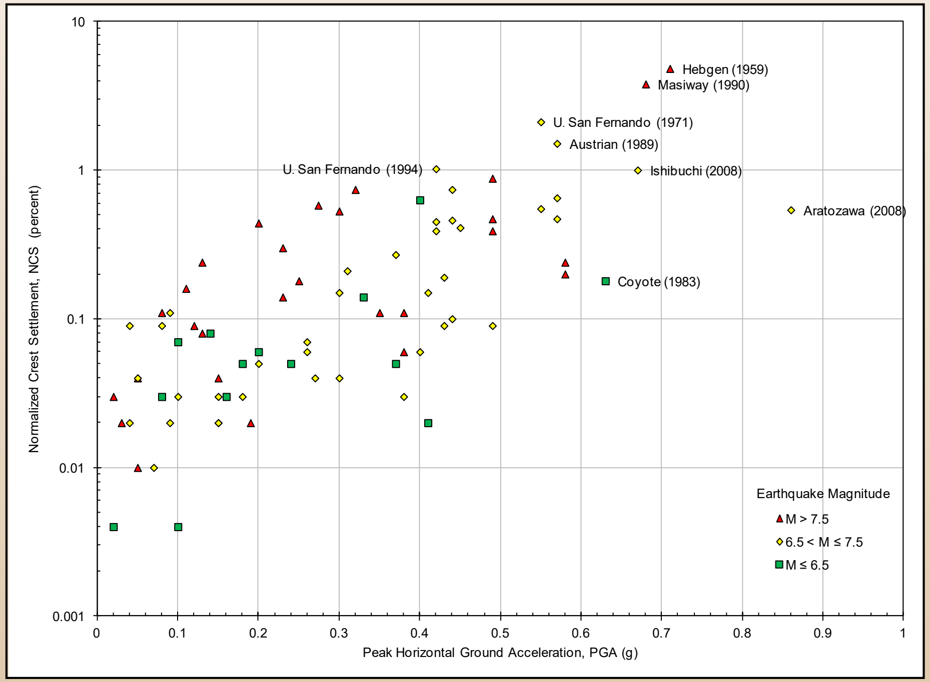
Outline

- Important case histories
- Steps for risk assessment
- Seismic potential failure modes
- Loading considerations
- Site characterization
- Likelihood of liquefaction and residual strength of liquefied soil
- Embankment deformation
- Internal erosion through cracks

Empirical Deformation (No Liquefaction occurs)



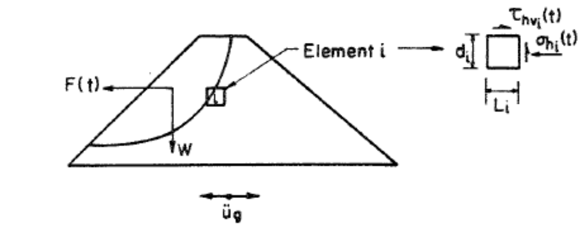




Swaisgood (1998, 2003, 2014)

Simplified Dynamic Sliding Mass Deformation

- Newmark (1965) and modified and updated by
  - Makdisi and Seed (1978)
  - Watson-Lamprey and Abrahamson (2006)
  - Bray and Travasarou (2007)



$$F(t) = \sum_{i=1}^{n} \tau_{hv_i}(t)\, L_i + \sigma_{h_i}(t)\, d_i$$

n = number of elements along the sliding surface

$$k_{av}(t) = F(t)/W$$

FIG. 3 CALCULATION OF AVERAGE ACCELERATION FROM FINITE ELEMENT RESPONSE ANALYSIS

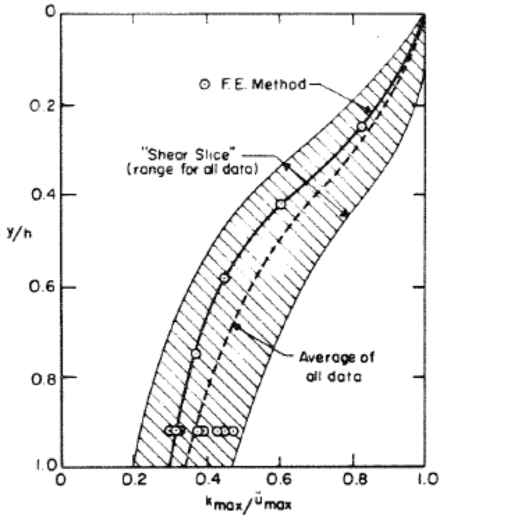


FIG. 9 VARIATION OF "MAXIMUM ACCELERATION RATIO" WITH DEPTH OF SLIDING MASS

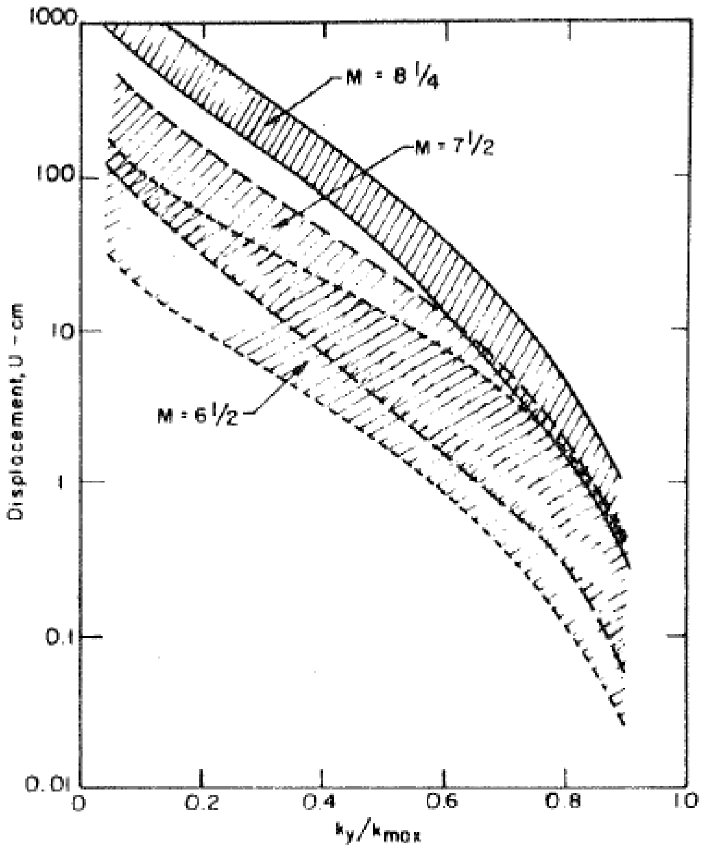
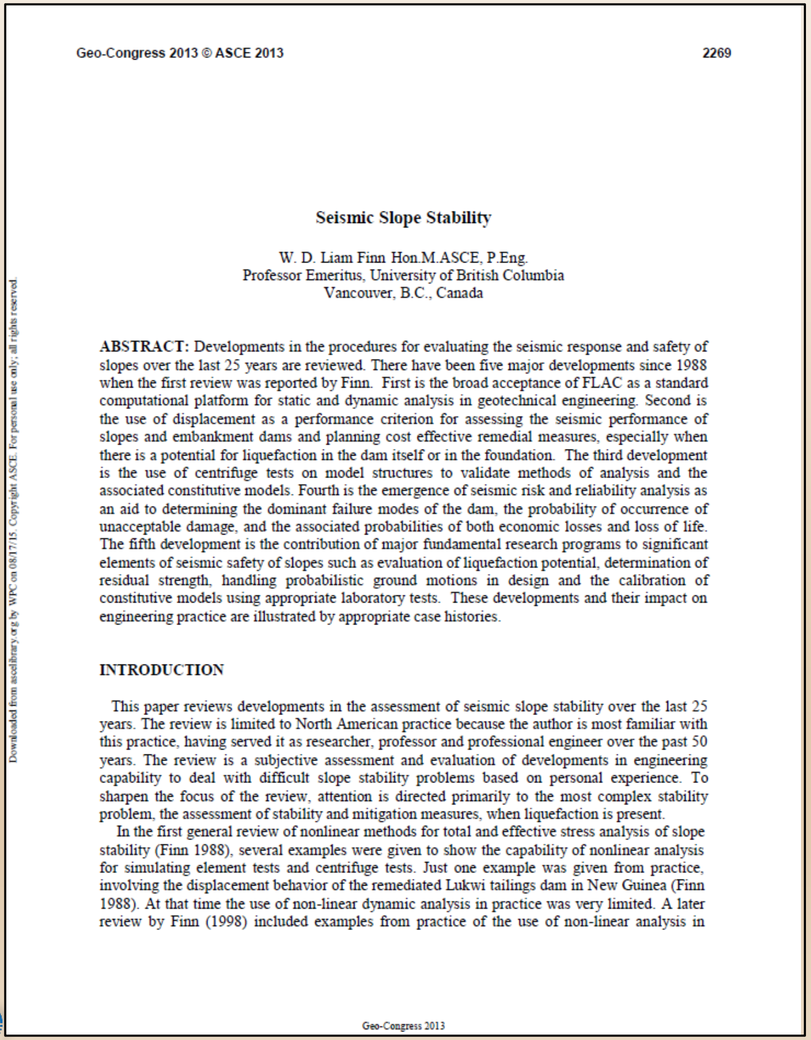


FIG. 14 VARIATION OF PERMANENT DISPLACEMENT WITH YIELD ACCELERATION - SUMMARY OF ALL DATA

Non-Linear Deformation Analysis



Geo-Congress 2013 © ASCE 2013 2269

Seismic Slope Stability

W. D. Liam Finn Hon.M.ASCE, P.Eng.
Professor Emeritus, University of British Columbia
Vancouver, B.C., Canada

ABSTRACT: Developments in the procedures for evaluating the seismic response and safety of slopes over the last 25 years are reviewed. There have been five major developments since 1988 when the first review was reported by Finn. First is the broad acceptance of FLAC as a standard computational platform for static and dynamic analysis in geotechnical engineering. Second is the use of displacement as a performance criterion for assessing the seismic performance of slopes and embankment dams and planning cost effective remedial measures, especially when there is a potential for liquefaction in the dam itself or in the foundation. The third development is the use of centrifuge tests on model structures to validate methods of analysis and the associated constitutive models. Fourth is the emergence of seismic risk and reliability analysis as an aid to determining the dominant failure modes of the dam, the probability of occurrence of unacceptable damage, and the associated probabilities of both economic losses and loss of life. The fifth development is the contribution of major fundamental research programs to significant elements of seismic safety of slopes such as evaluation of liquefaction potential, determination of residual strength, handling probabilistic ground motions in design and the calibration of constitutive models using appropriate laboratory tests. These developments and their impact on engineering practice are illustrated by appropriate case histories.

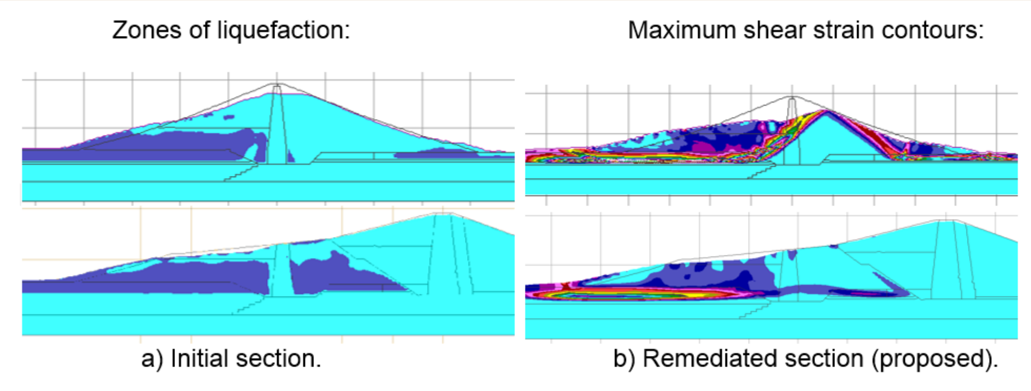
INTRODUCTION

This paper reviews developments in the assessment of seismic slope stability over the last 25 years. The review is limited to North American practice because the author is most familiar with this practice, having served it as researcher, professor and professional engineer over the past 50 years. The review is a subjective assessment and evaluation of developments in engineering capability to deal with difficult slope stability problems based on personal experience. To sharpen the focus of the review, attention is directed primarily to the most complex stability problem, the assessment of stability and mitigation measures, when liquefaction is present.

In the first general review of nonlinear methods for total and effective stress analysis of slope stability (Finn 1988), several examples were given to show the capability of nonlinear analysis for simulating element tests and centrifuge tests. Just one example was given from practice, involving the displacement behavior of the remediated Lukwi tailings dam in New Guinea (Finn 1988). At that time the use of non-linear dynamic analysis in practice was very limited. A later review by Finn (1998) included examples from practice of the use of non-linear analysis in

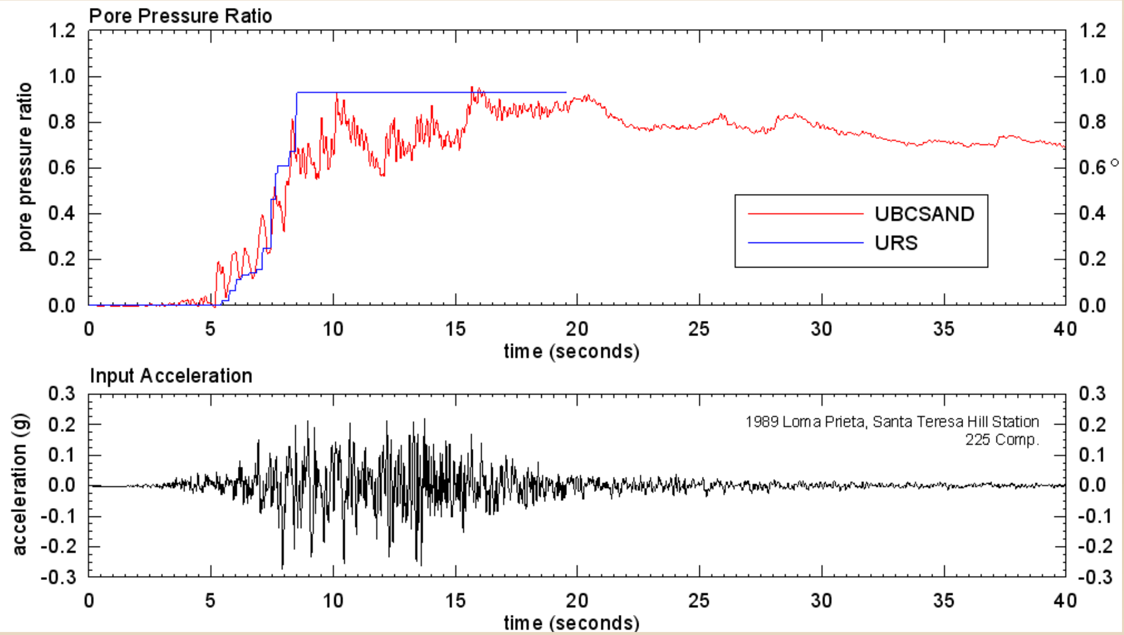
Downloaded from ascelibrary.org by WPC on 08/17/15. Copyright ASCE. For personal use only; all rights reserved.

Geo-Congress 2013



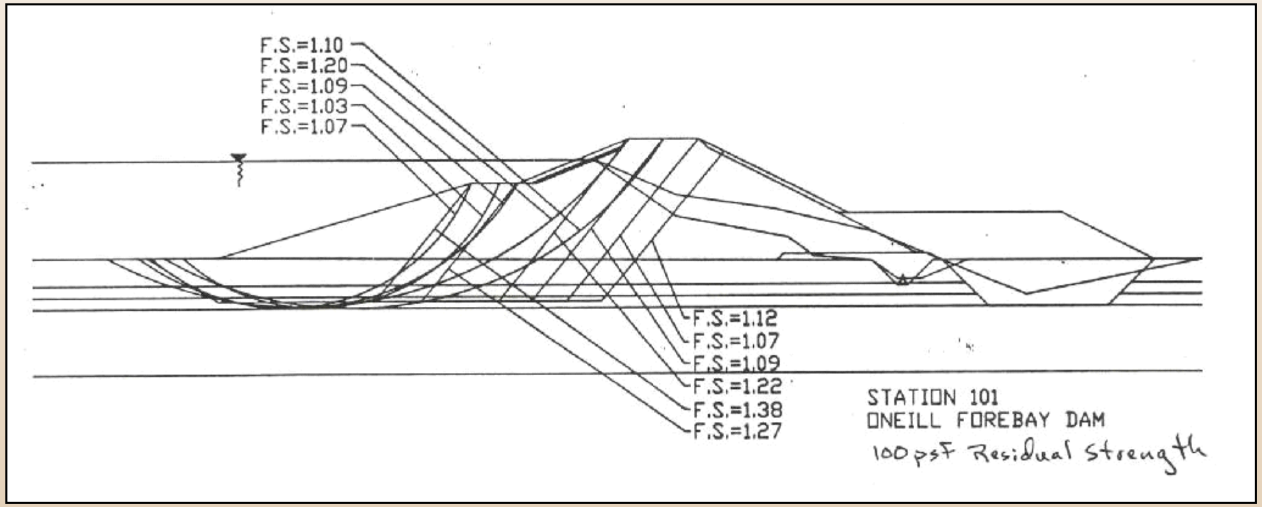
a) Initial section.

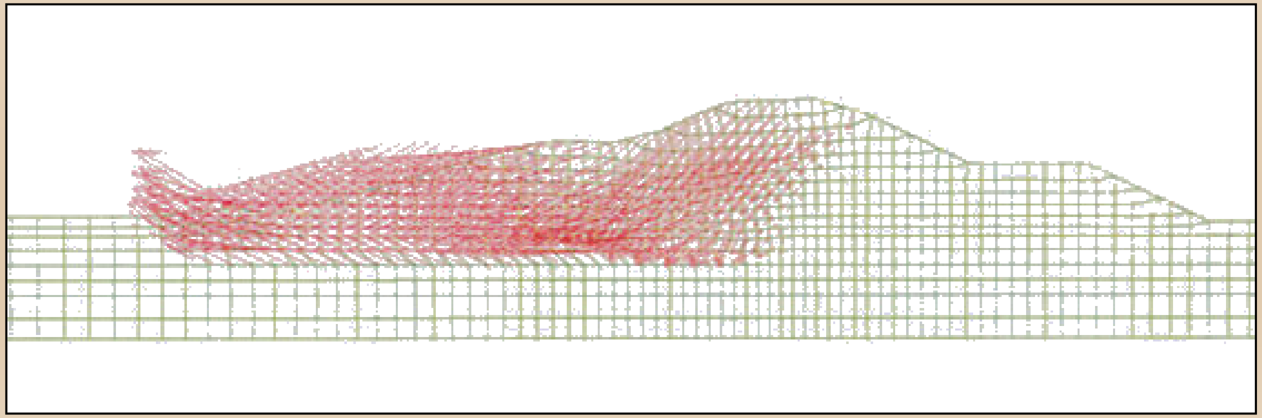
b) Remediated section (proposed).



Post-Earthquake Deformation

- Post EQ Slope Stability Analysis
- Deformation - use a computer program such as FLAC
  - Model potentially liquefiable materials using residual undrained shear strengths
  - Evaluate deformed shape and displacement magnitudes from applied gravity loading only
- Neglects dynamic deformations that could occur during shaking





Outline

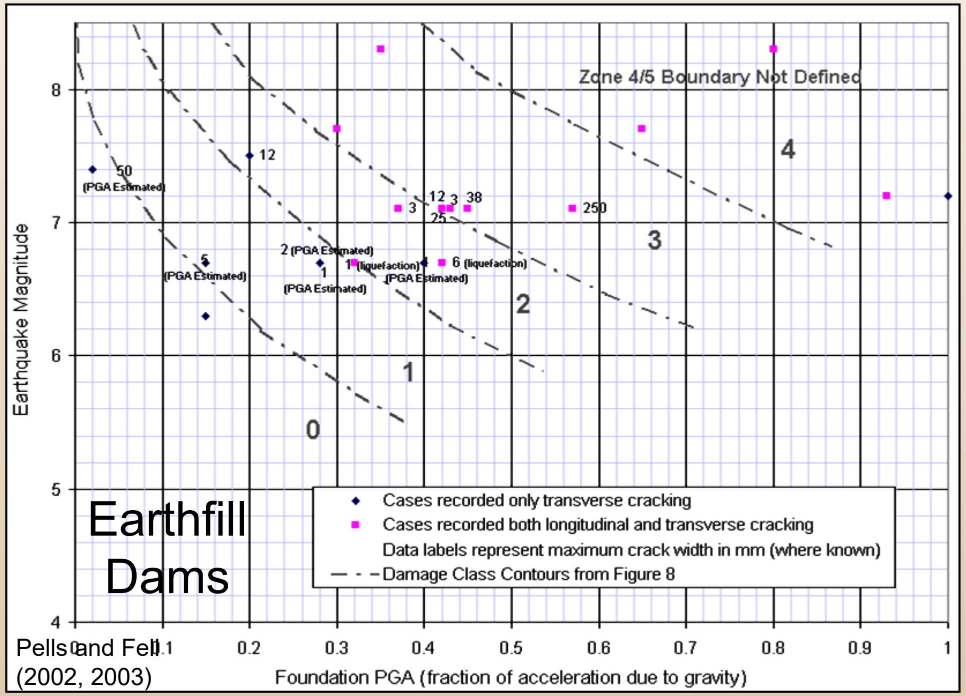
- Important case histories
- Steps for risk assessment
- Seismic potential failure modes
- Loading considerations
- Likelihood of liquefaction or cyclic softening/strength loss
- Embankment deformation
- Internal erosion through cracks

Internal Erosion through Cracks

after Pells and Fell (2002, 2003)

- Estimate damage class from deaggregation of seismic hazard for each seismic load partition
  - Assume Damage Class 3 or 4 if liquefaction occurs
- Estimate maximum likely crest settlement.
  - Cracking begins at settled crest elevation
- Estimate probability of transverse cracking
  - Use expert elicitation and Fell et al. (2008) as guide
- Estimate maximum likely crack width at the crest using Fell et al. (2008) as guide
- Estimate maximum likely crack depth.
- Estimate probability of initiation, continuation, and progression of concentrated leak erosion for reservoir partitions
  - See Chapter D-6 Internal Erosion

Damage Class = f (PGA, M_w)



PGA-M_w pairs can be obtained from seismic hazard deaggregation

Damage Class		Maximum Longitudinal Crack Width [(1)] (mm)	Maximum Relative Crest Settlement [(2)] (percent)
Number	Description		
0	No or Slight	< 10	< 0.03
1	Minor	10 to 30	0.03 to 0.2
2	Moderate	30 to 80	0.2 to 0.5
3	Major	80 to 150	0.5 to 1.5
4	Severe	150 to 500	1.5 to 5
5	Collapse	> 500	> 5

Notes: (1) Maximum likely crack width is taken as the maximum width of any longitudinal crack that occurs.
(2) Maximum relative crest settlement is expressed as a percentage of the structural height.

Damage Class		Probability of Transverse Cracking	Maximum Likely Crack Width at the Crest (mm)
Number	Description		
0	No or Slight	0.001 to 0.01	5 to 20
1	Minor	0.01 to 0.05	20 to 50
2	Moderate	0.05 to 0.10	50 to 75
3	Major	0.2 to 0.25	100 to 125
4	Severe	0.5 to 0.6	150 to 175

Main Issues to Consider

- Defensive measures of dam: filters to prevent or control internal erosion of the dam and its foundation; zones of good drainage capacity (e.g., free-draining rockfill)
- Embankment stability during and immediately after the earthquake
- Earthquake-induced deformations (i.e., settlement and cracking) and dam freeboard
- Liquefaction potential of saturated sandy and silty soils and some gravels with a sand and silt matrix in the foundation, and possibly in the embankment
- Cyclic softening potential of soft or sensitive clays in the foundation and possibly in the embankment

Adapted from Fell (2005) *Geotechnical Engineering of Dams*