Utah Chapter EERI Short Course, April 9th 2014

Maverik Center, West Valley, Utah

Evaluation and Mitigation of Liquefaction Hazard for Engineering Practice

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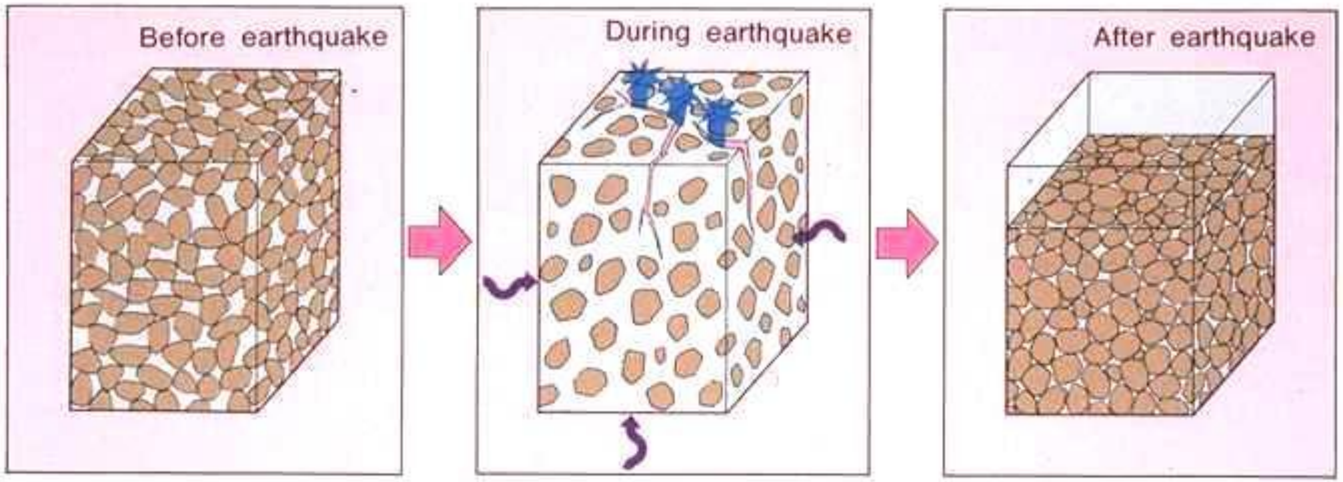
Show Canterbury Video

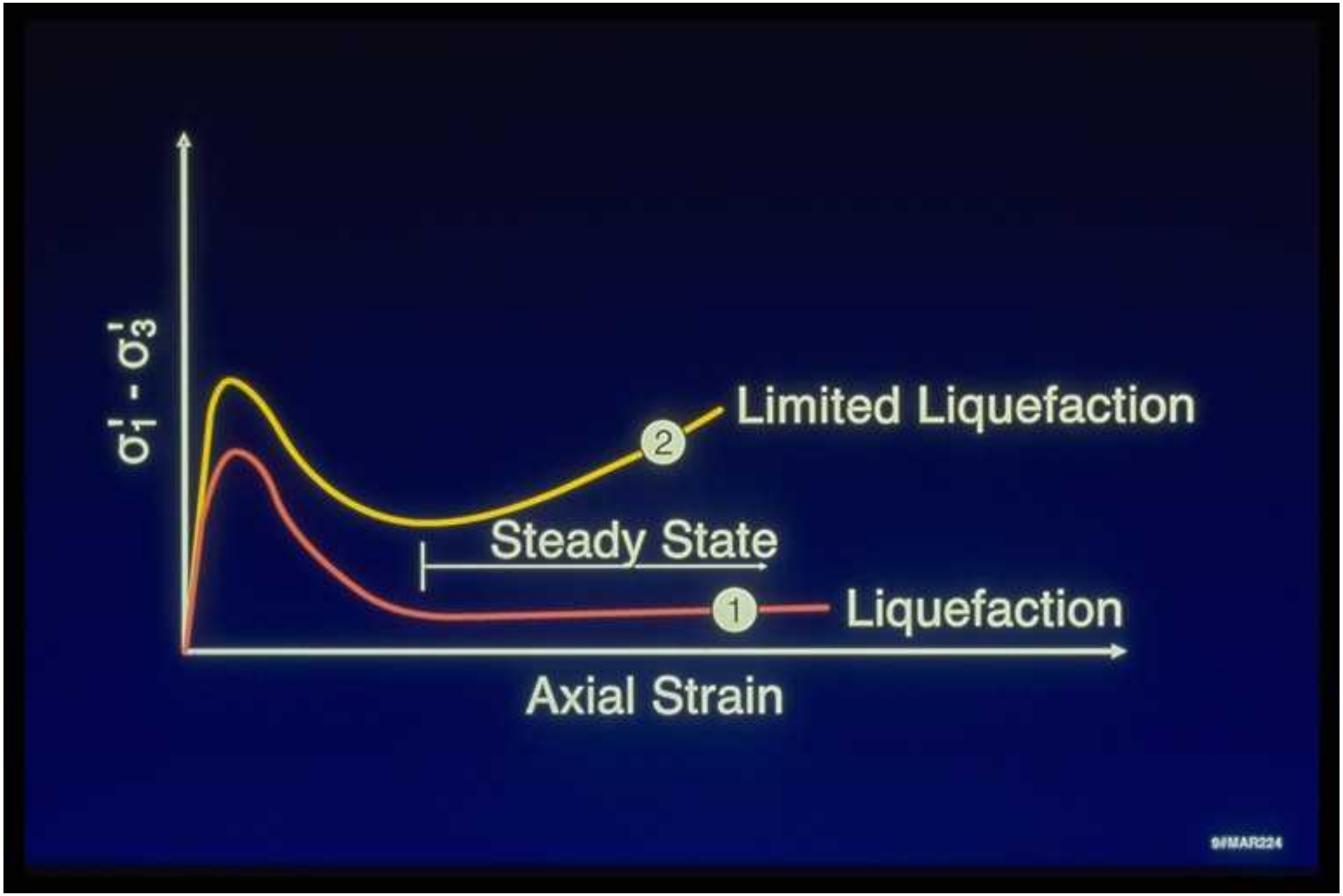
▼Mechanism of liquefaction.



1. **Loose sand before liquefaction:**
   Sand grains are interconnected with each other.

2. **Liquefied state:**
   Sand grains floating in ground water. Seismic force applied to the ground water causes water and sand to jet out on the ground surface.

3. **Completion of liquefaction:**
   Sand grains have been interconnected more compactly than they were before the earthquake, and the entire ground has subsided.



$\sigma_1' - \sigma_3'$
Limited Liquefaction
2
Steady State
1
Liquefaction
Axial Strain
9#MAR224

Flow (static) Liquefaction

- Strain softening response in undrained shear
- Trigger mechanism required
- Static shear stress greater than minimum (*liquefied*) undrained shear strength
- Kinematic mechanism required
  - Uncontained flow
  - Contained deformation

Flow (static) liquefaction steeply sloping ground

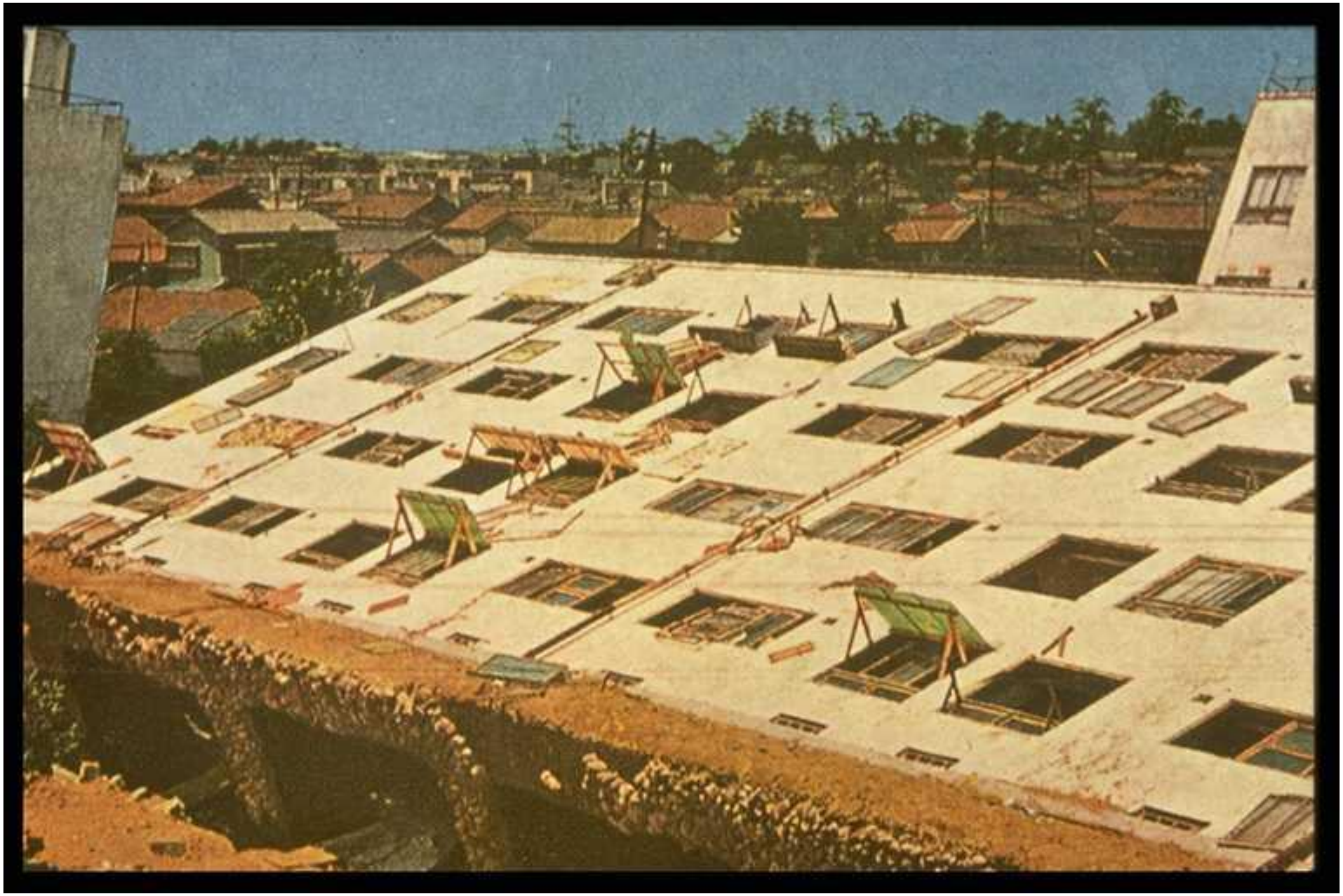
Sites defined as:

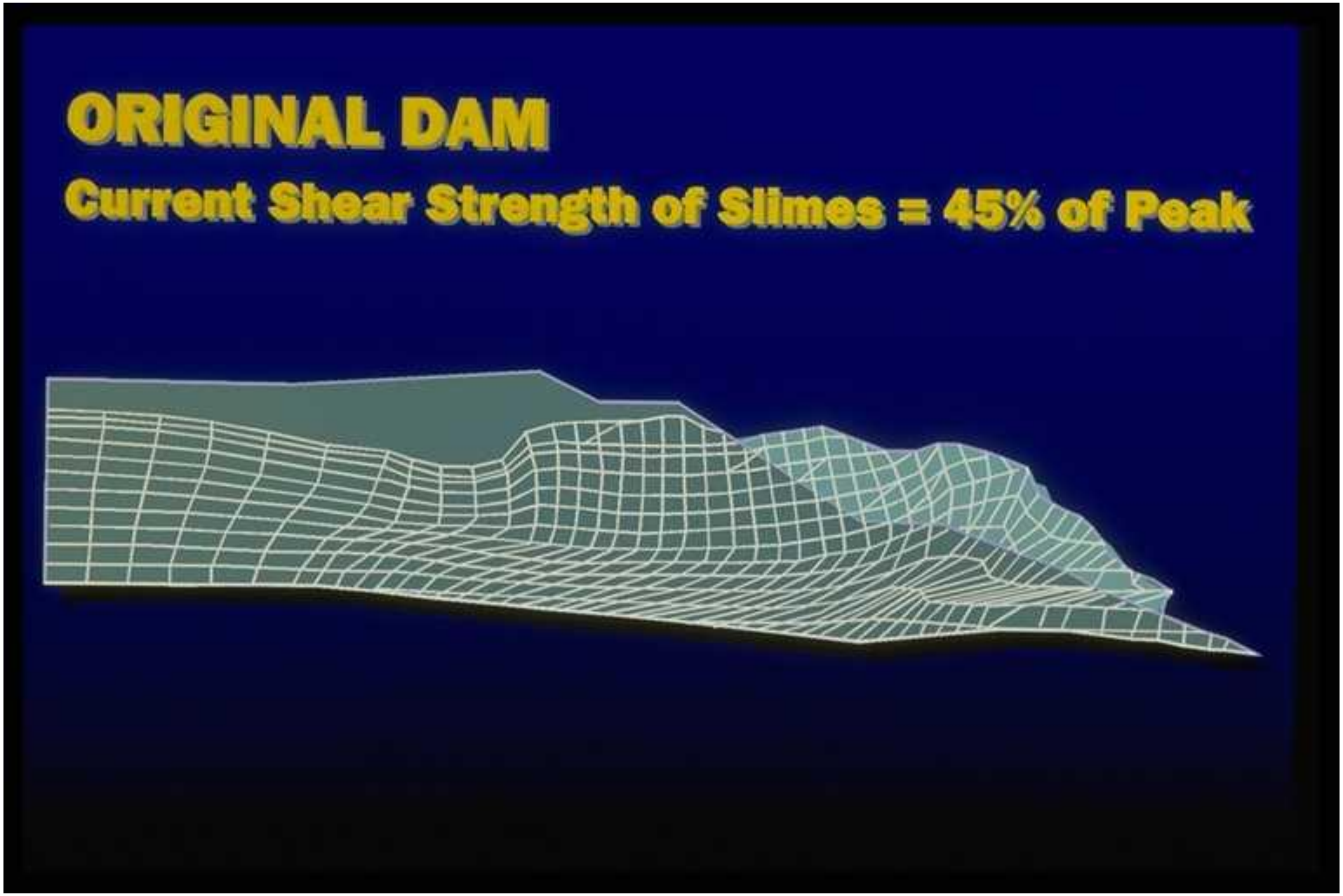
Steeply sloping (> 5 degrees) or earth embankments (e.g. dams)

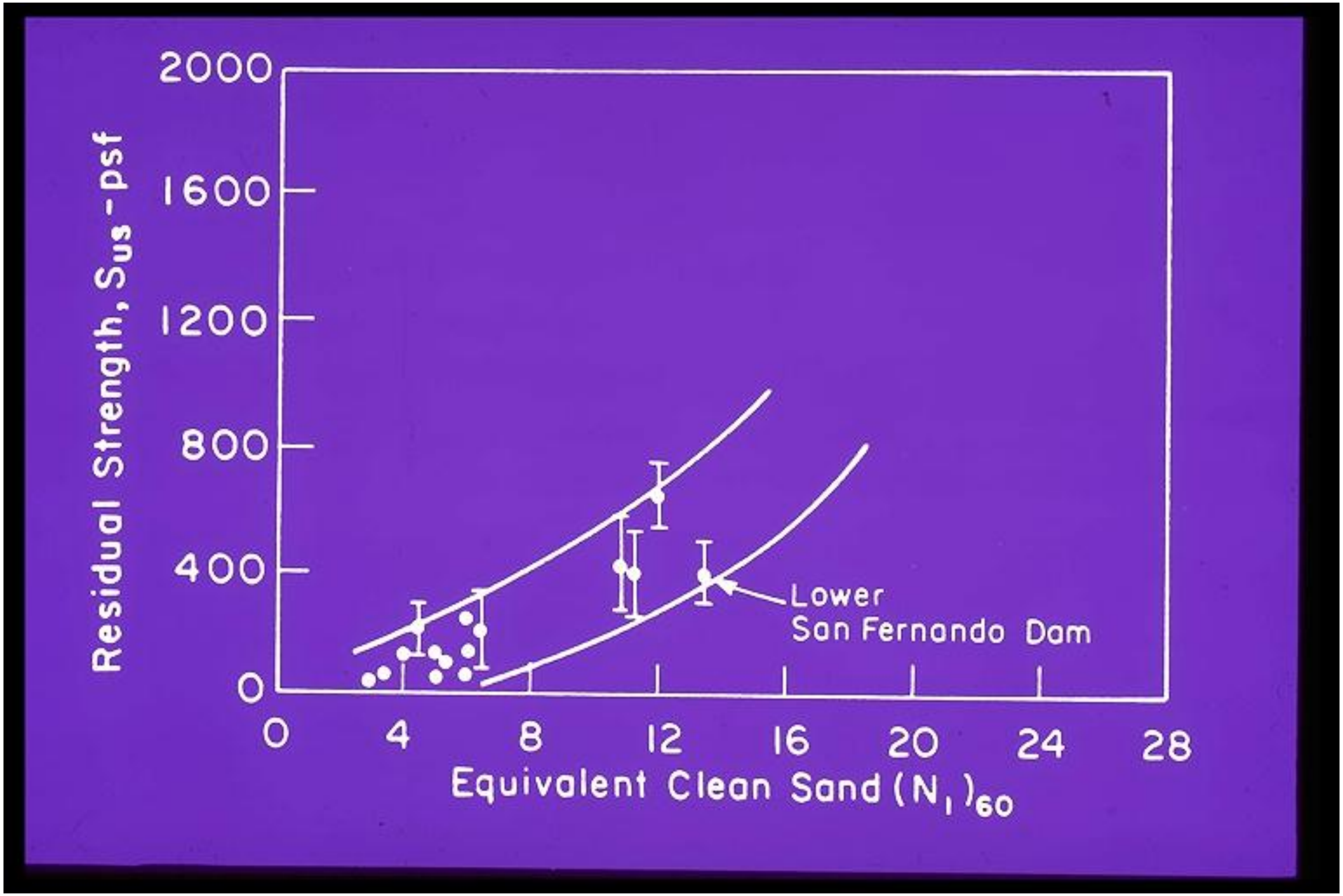
Sequence to evaluate flow liquefaction:

1. Evaluate susceptibility for strength loss
2. Evaluate stability using post-earthquake shear strengths
3. Evaluate trigger for strength loss

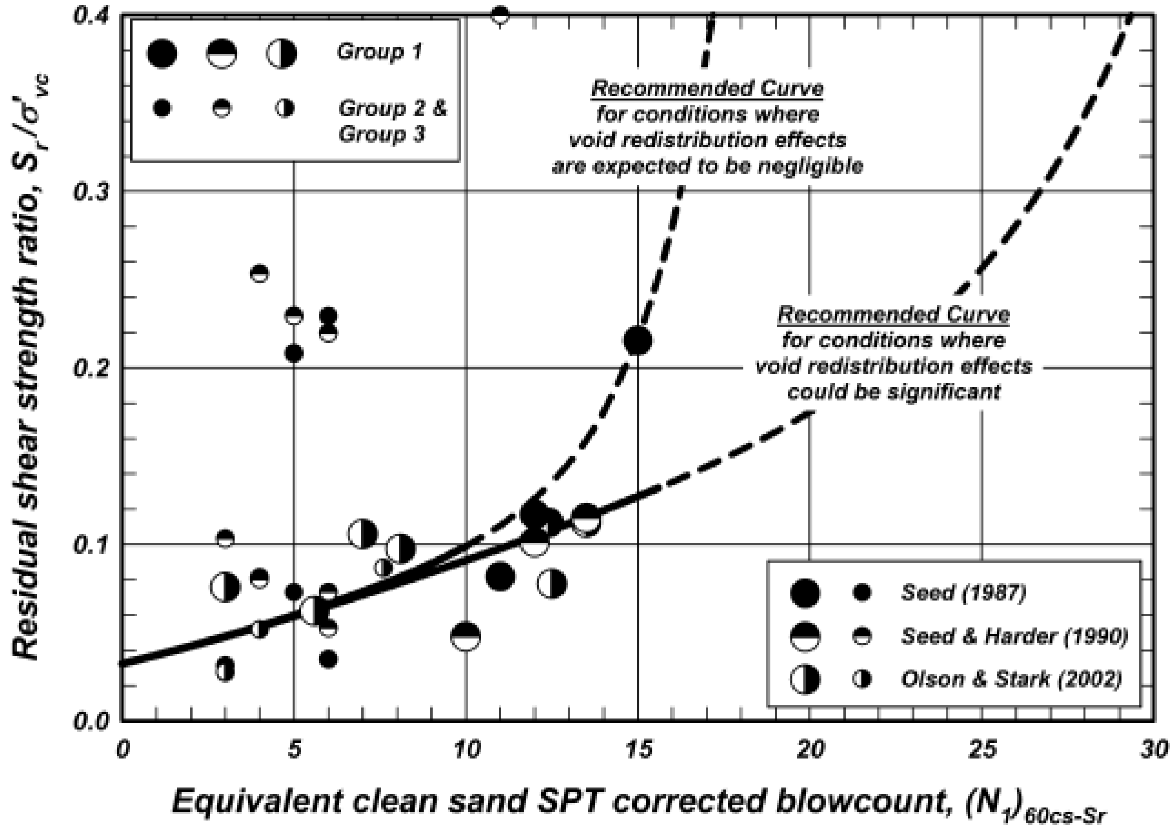
If soils are susceptible, and instability possible, it is often prudent to assume trigger will occur




ORIGINAL DAM
Current Shear Strength of Slimes = 45% of Peak

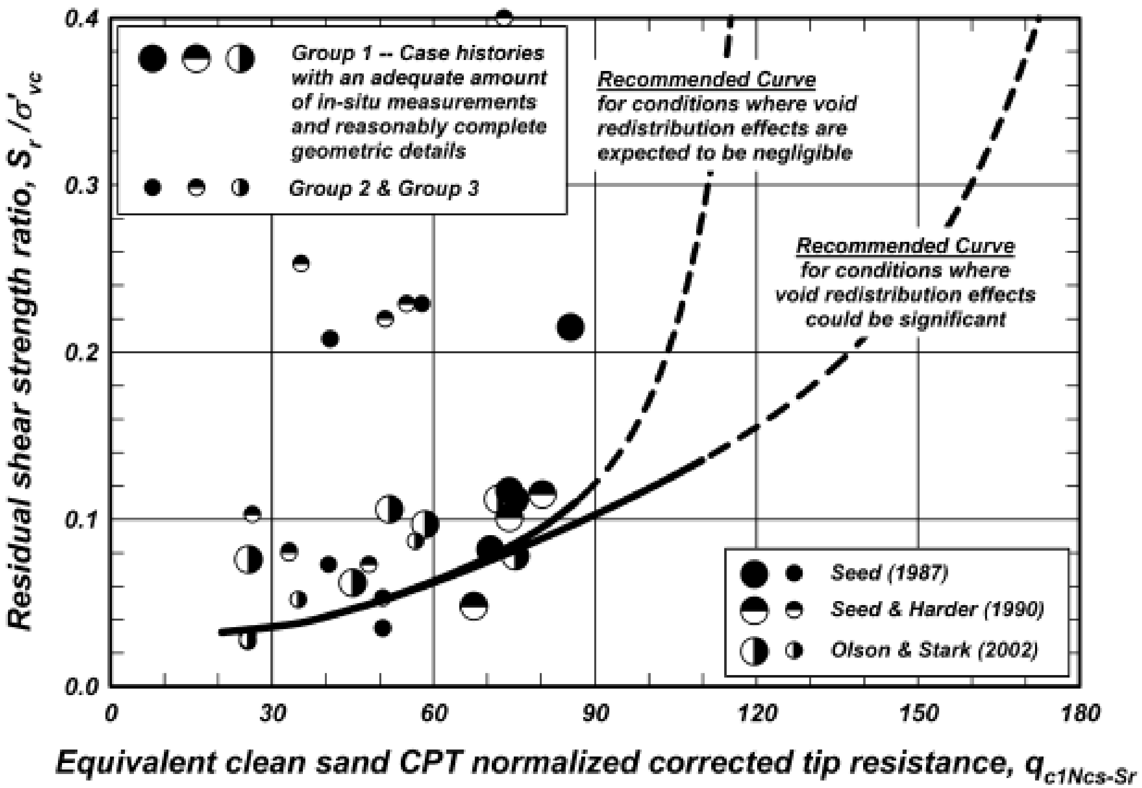

Residual Strength, Sus - psf
2000
1600
1200
800
400
0
0
4
8
12
16
20
24
28
Equivalent Clean Sand (N1)60
Lower
San Fernando Dam

Estimates of Residual Shear Strength from SPT-N Data



Idriss and Boulanger, 2008

Estimates of Residual Strength



Idriss and Boulanger, 2008

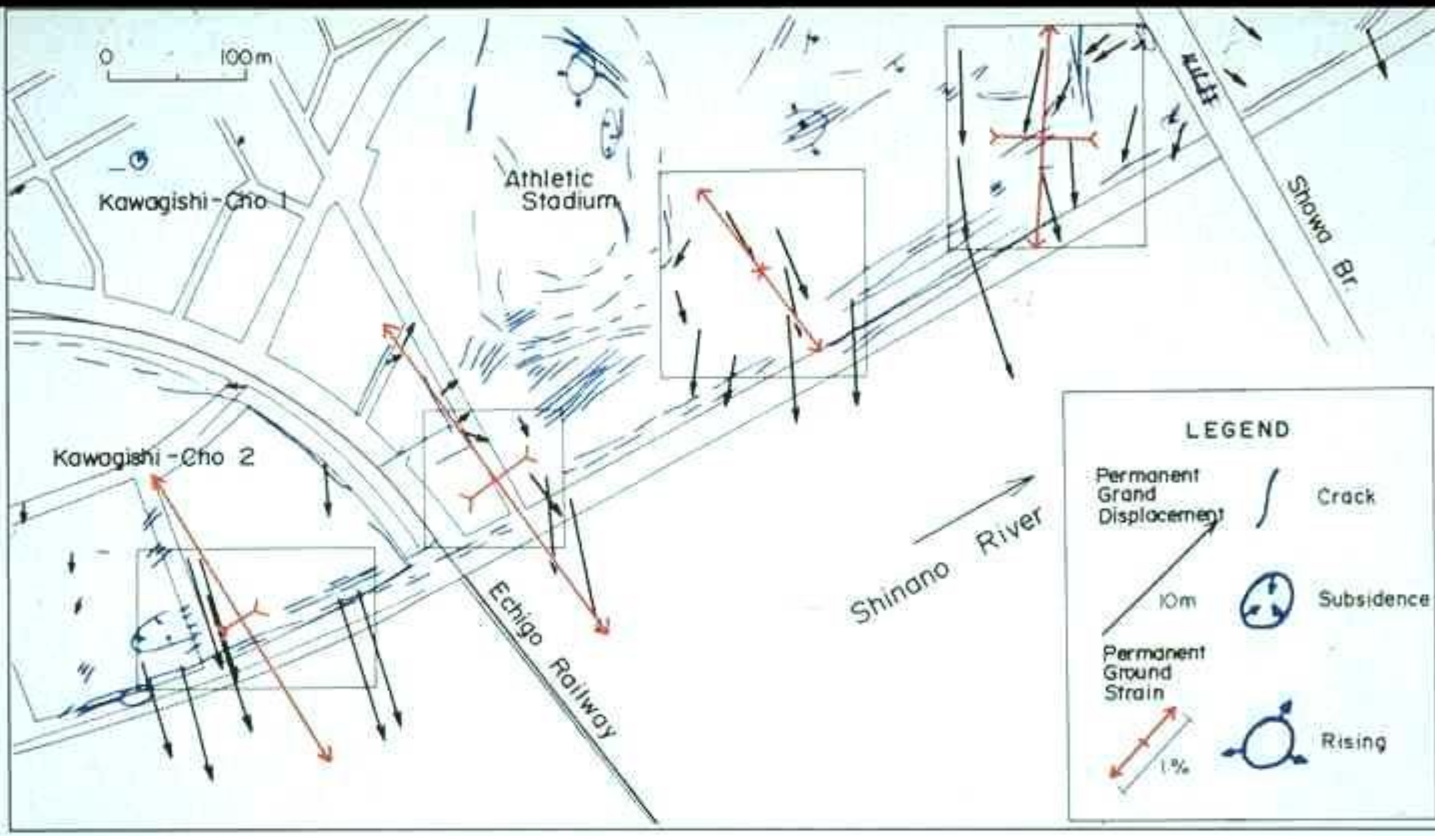
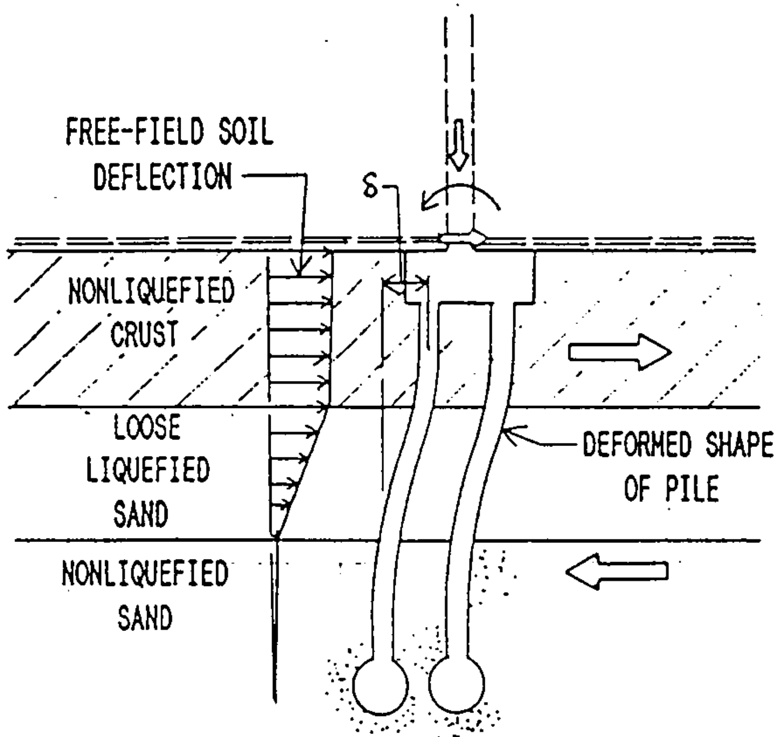


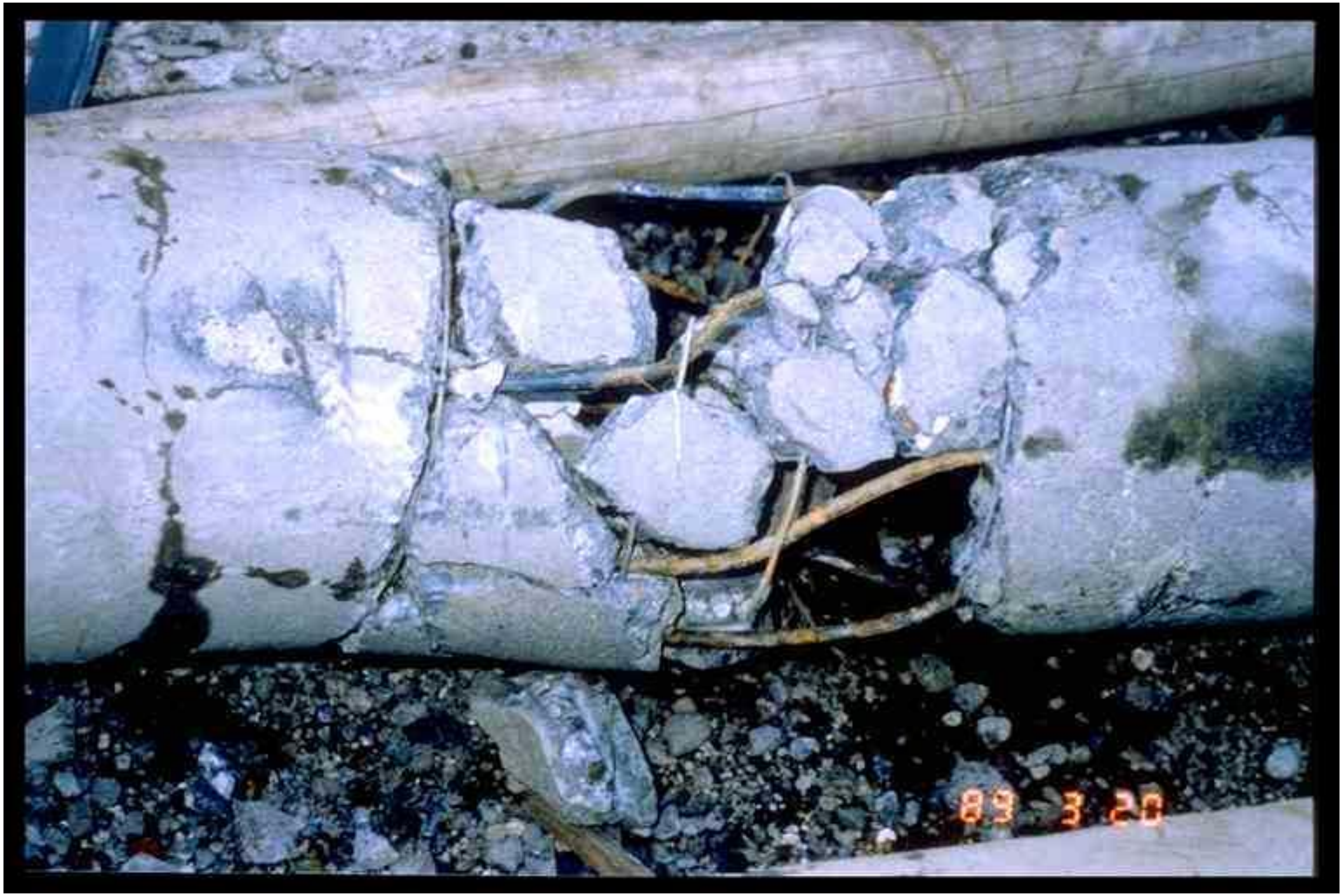
Fig. 5-8 Permanent ground strains, displacements, and ground failures in the area along the Shinano River


2T



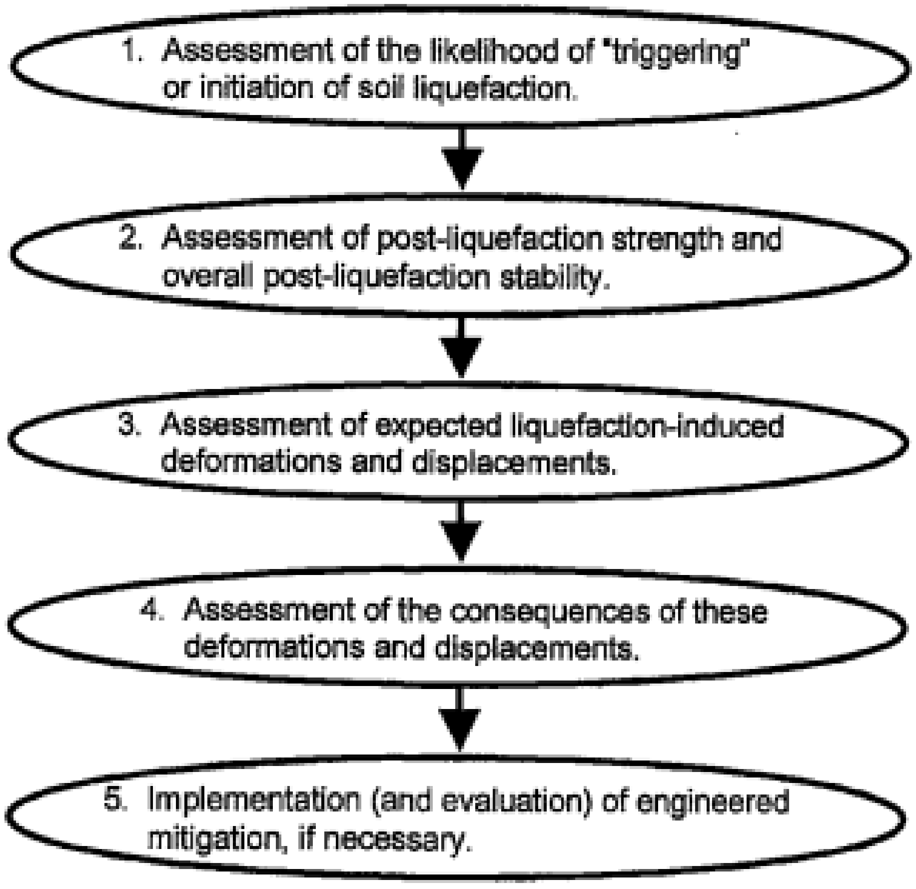
(a) PILE CONFIGURATION
DUE TO LIQUEFACTION

Show Pile Videos


89 3 20

危険
作業半径
立入禁止

Key Elements in Liquefaction Studies



After Seed et al. 2001

Demand vs Resistance Capacity For Sands

Demand – ***PGA and Seismic Shear Stresses***

Simplified Method and or Site Response Analysis

Resistance Capacity - - ***in situ tests – SPT, CPT, Vs***

Extreme Measures -*Test samples cored from frozen ground*

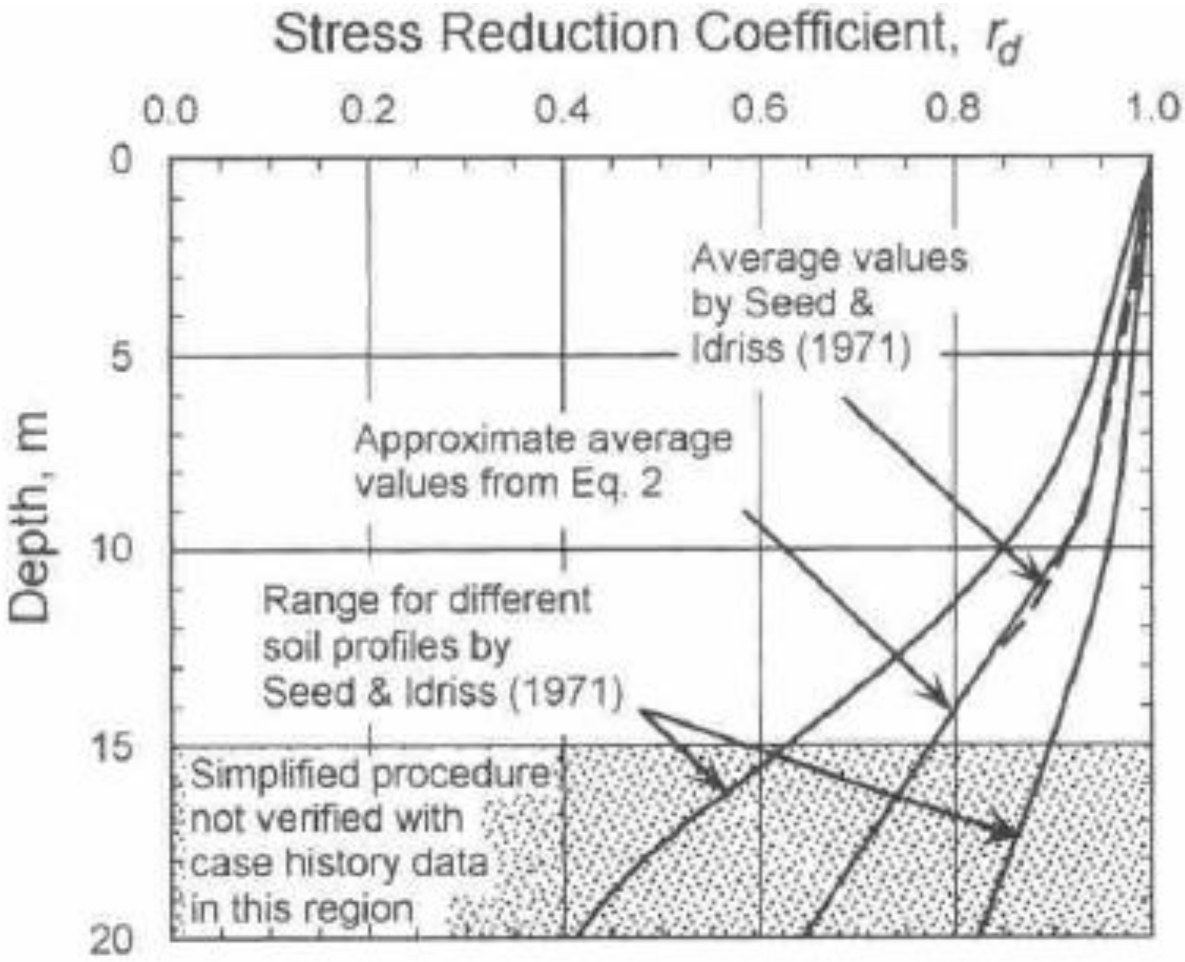
Cyclic Shear Stress Ratio

The Simplified Approach estimates average cyclic shear stress ratios (CSR) caused by earthquake shaking using:

$$CSR = 0.65 \frac{a_{max}}{g} \cdot \frac{\sigma_{v0}}{\sigma'_{v0}} \cdot \frac{r_d}{MSF}$$

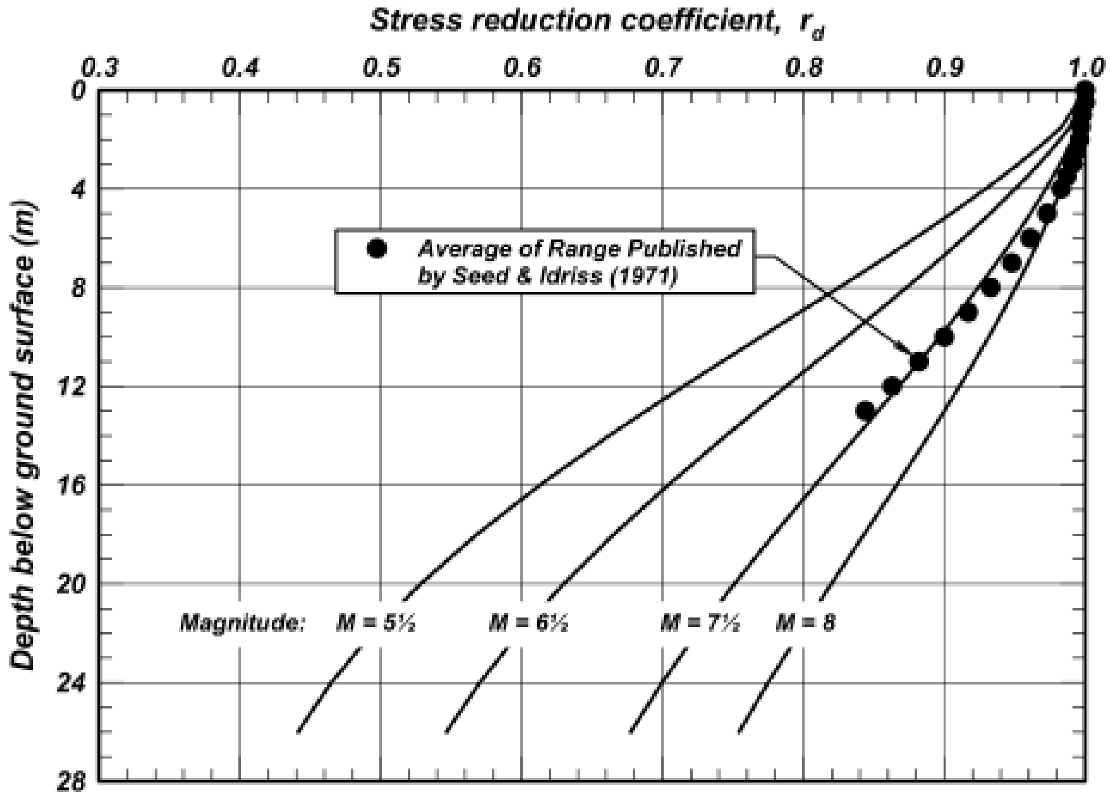
Alternative approach is to do site response analyses.

Stress Reduction Coefficient rd



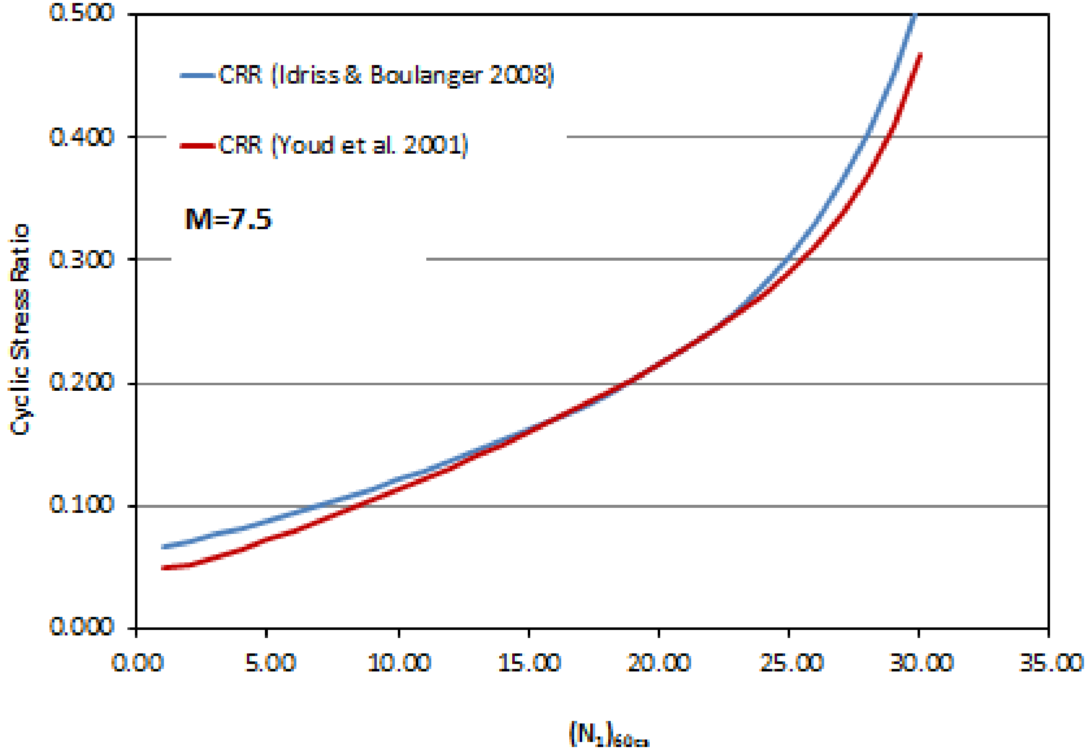
Youd et al. 2001

Magnitude Dependent Stress Reduction Coefficient



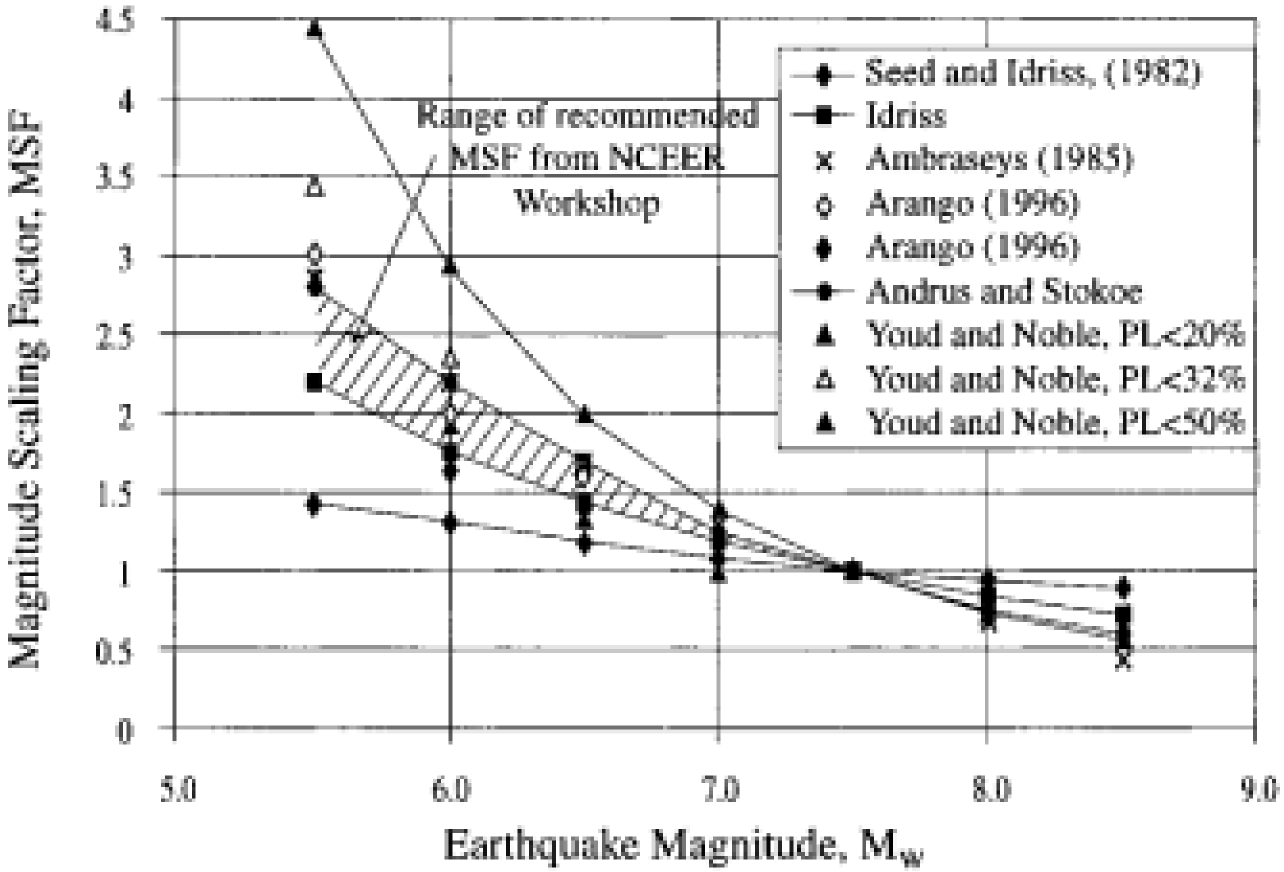
Idriss and Boulanger, 2008

Liquefaction resistance curves for M=7.5 by Youd and Idriss& Boulanger procedures



Note that the differences in rd makes little difference in results. Correlation with data takes care of it.

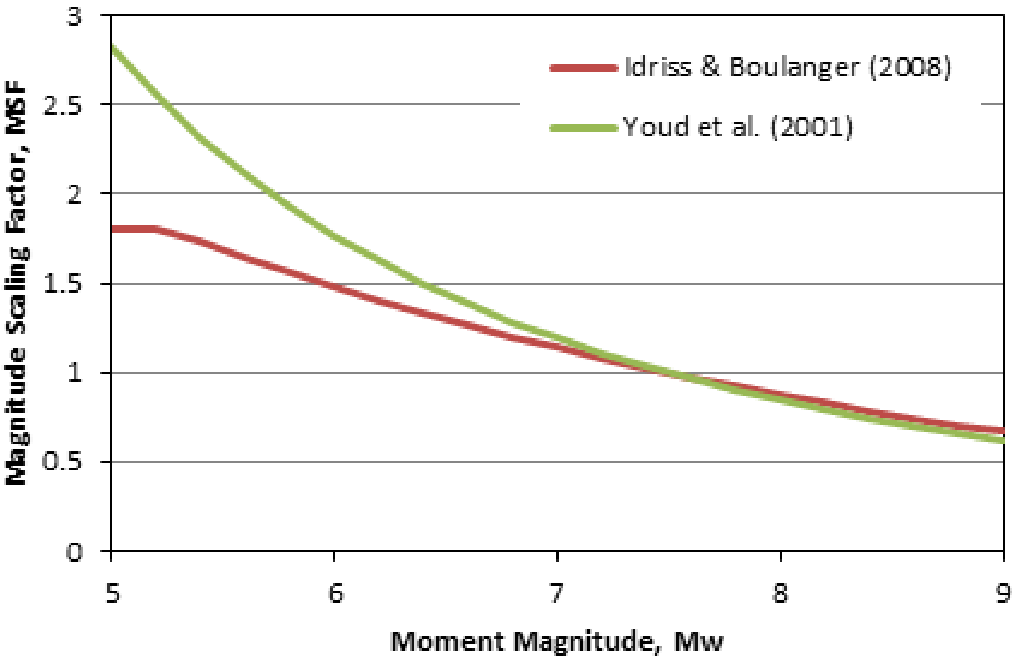
Magnitude Scaling Factors-MSF - 2001



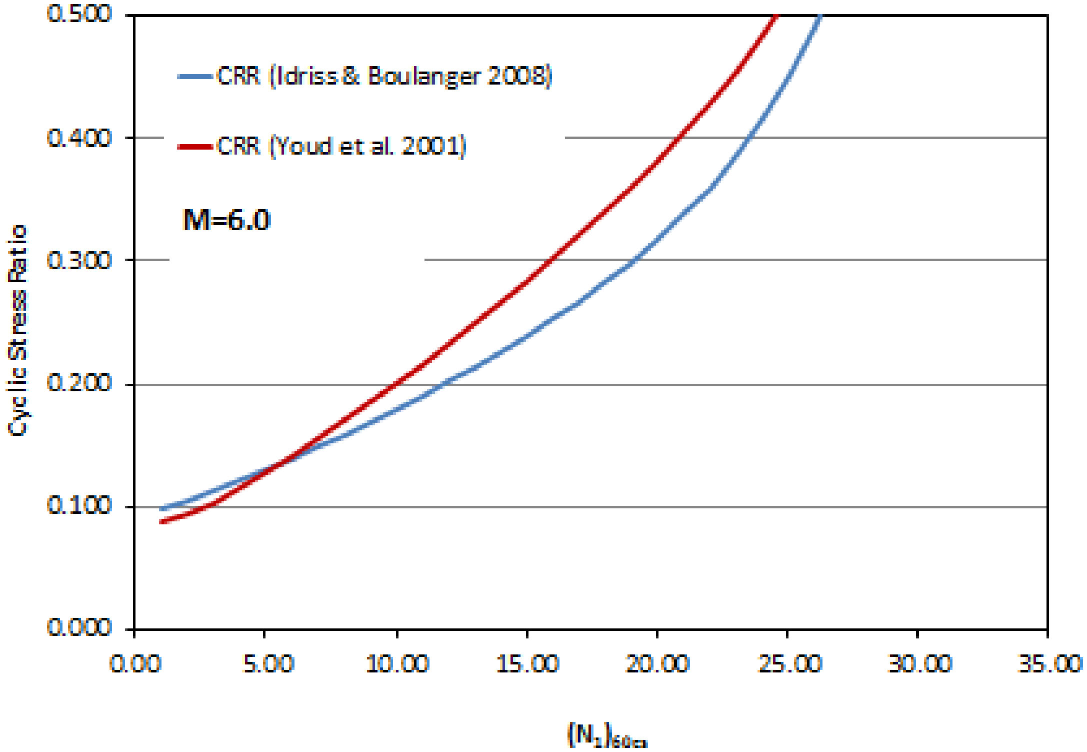
MSF from Idriss and Boulanger (2008)

$$MSF = \text{min of } \left[6.9\exp\left({}^{-M_w}\!/_{4}\right) - 0.058\right] \quad \text{or} \quad 1.8$$

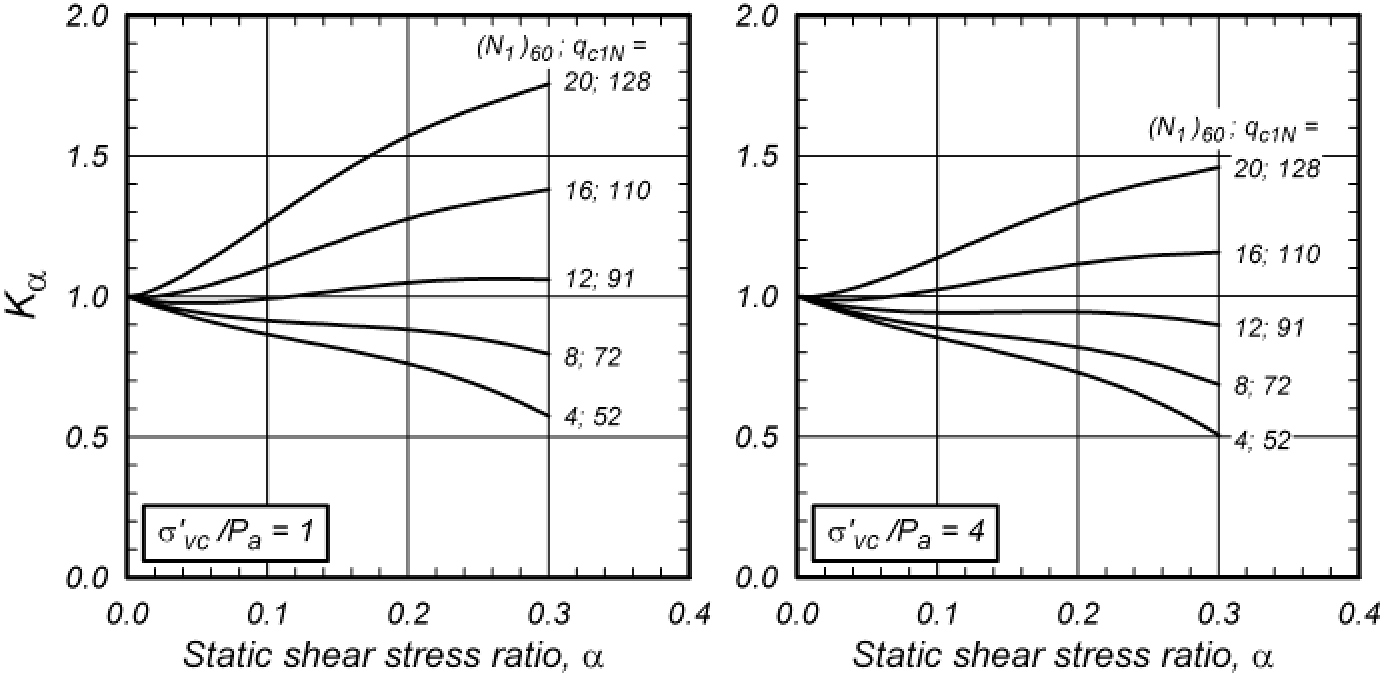
Comparison of Youd et al. (2001) and Idriss & Boulanger (2008) Magnitude Scaling Factors



Liquefaction resistance curves for M=6.0 by Youd and Idriss& Boulanger procedures



Variations of K_α with SPT & CPT Data



Site Response Analysis for

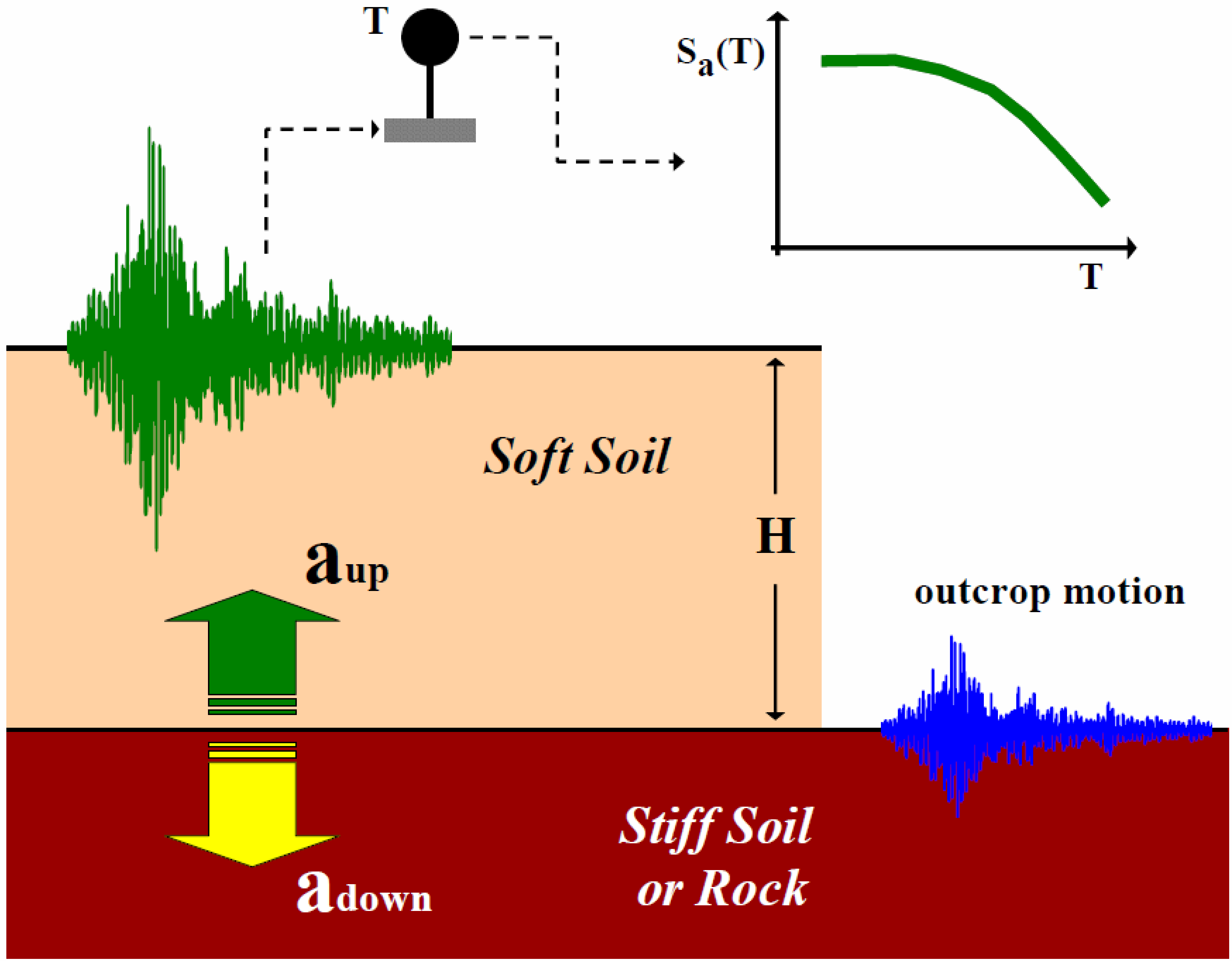
1. Determination of PGA

and

2. Cyclic shear stresses

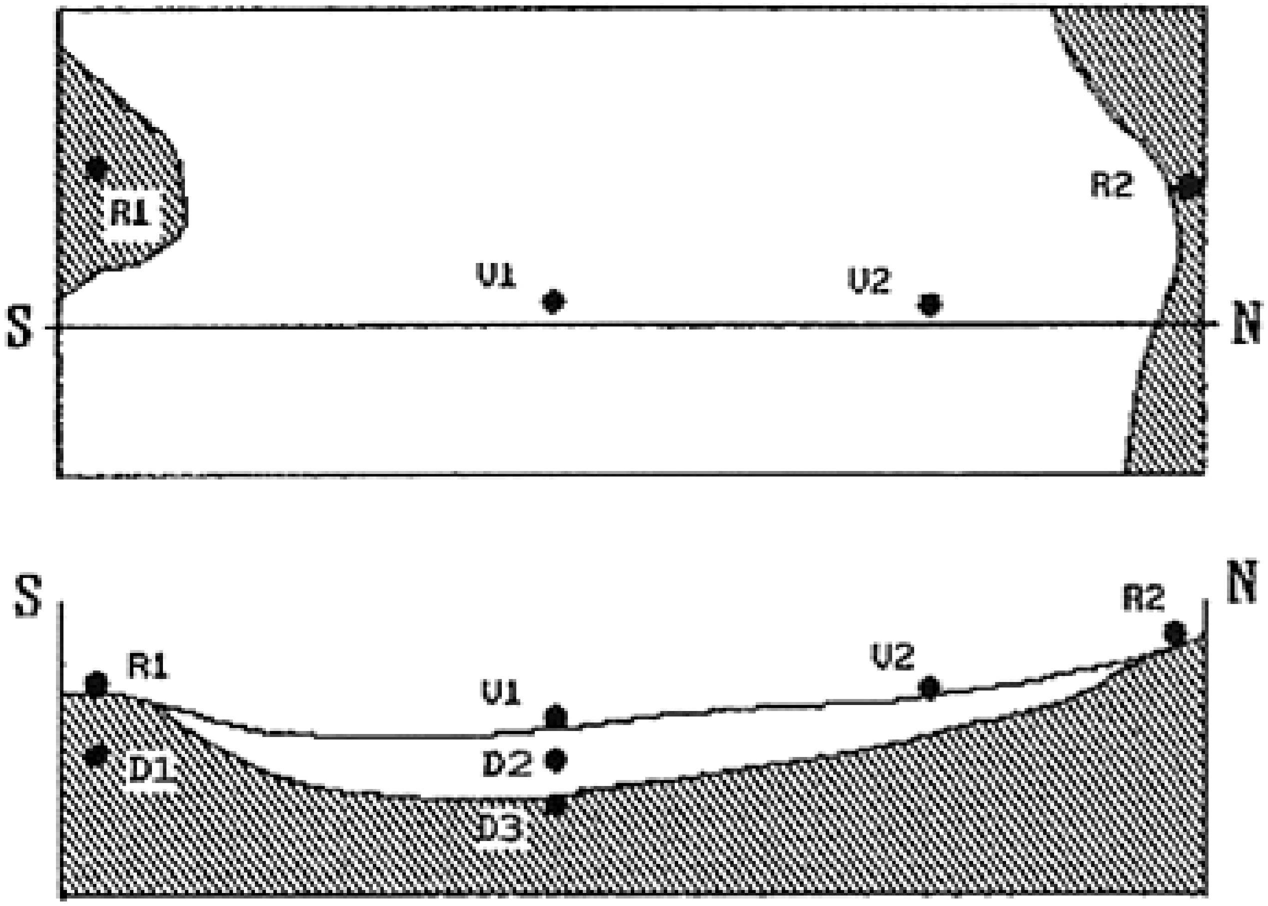
This is a developing trend. It is very prevalent in Vancouver.

Is this the best approach? Much better than the Simplified Method?

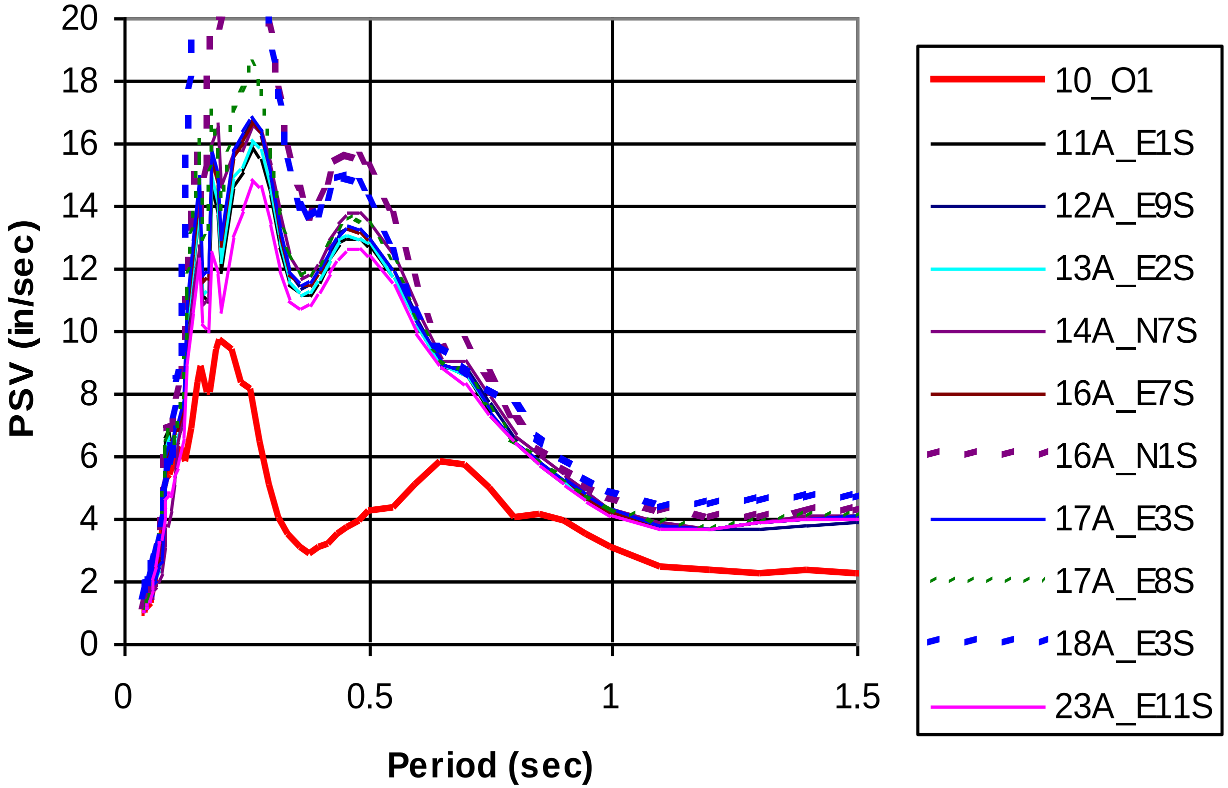


T
$S_a(T)$
T
Soft Soil
H
a_{up}
outcrop motion
Stiff Soil
or Rock
a_{down}

Turkey Flat Instrument Layout



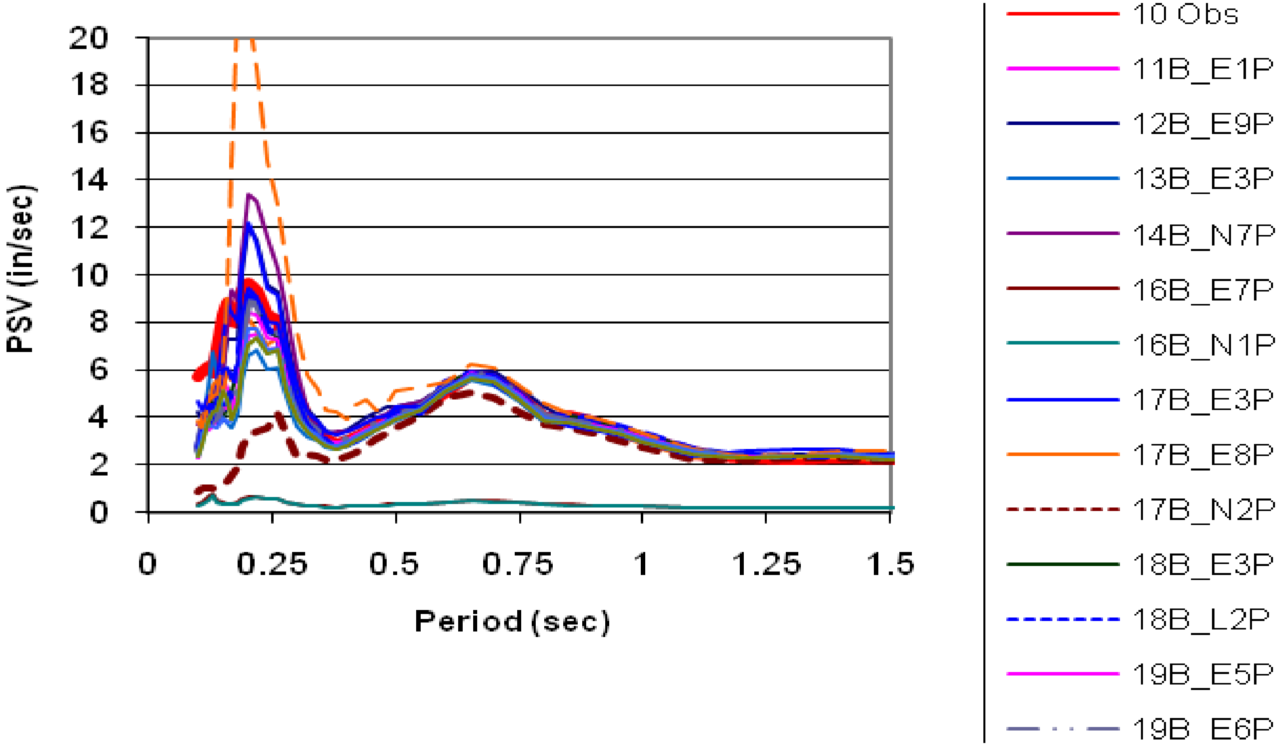
Site Response to Outcrop Input Motions



Red curve is recorded response - other colors are predictions very bad predictions.

Probability of liquefaction would be seriously over-estimated.

Predictions using recorded motions at D at base of soil column. VGood Results



Caution on using site response analysis to get acceleration (PGA) or shear stresses.

Remember that all the liquefaction assessment charts were developed using shear stresses computed using the simplified equation.

See comments on getting PGA in next slide

PGA for Simplified Method

"The formal assessment of liquefaction at a site using the simplified procedure should be based on the a_{max} that is estimated to develop in the absence of soil softening or liquefaction."

Boulanger and Idriss, 2014

Validation of analytical methods 1

Prediction Exercise 1; Element Tests

Saada and Bianchini (1988) prediction exercise demonstrated that ability of a model to simulate element tests is no guarantee of how it will perform in other element stress fields with different stress paths. Models need to be calibrated for the dominant stress paths expected in application as far as is possible with the conventional tests used in engineering practice.

Centrifuge Tests

The centrifuge test with artificial gravity 20g- 60 g can create stresses in a relatively small model that are representative of the stresses in the field.

Also by creating slopes or introducing structures into soil model we can create Inhomogeneous stress states. These pose greater challenges for soil models.

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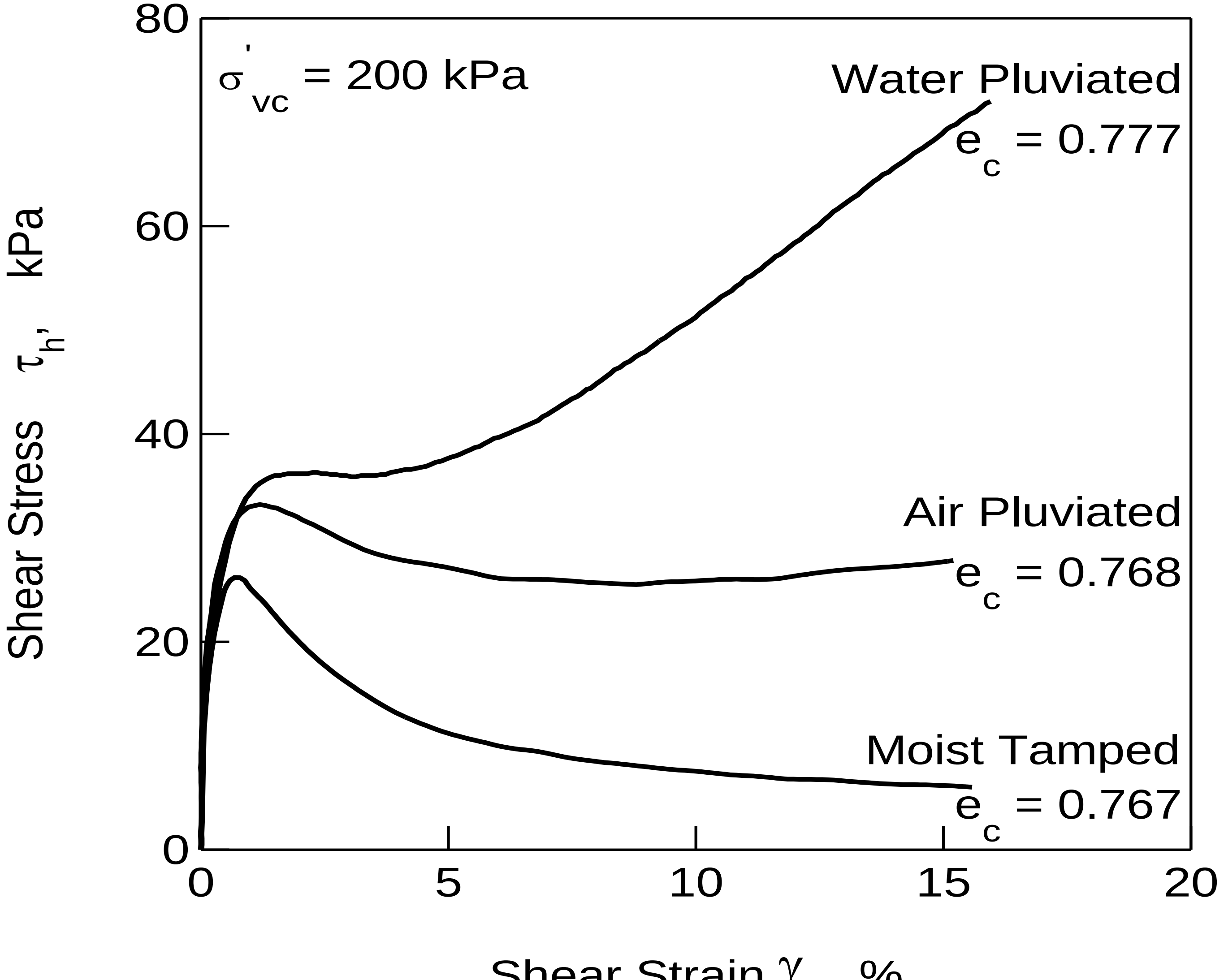
Validation of analytical methods 2

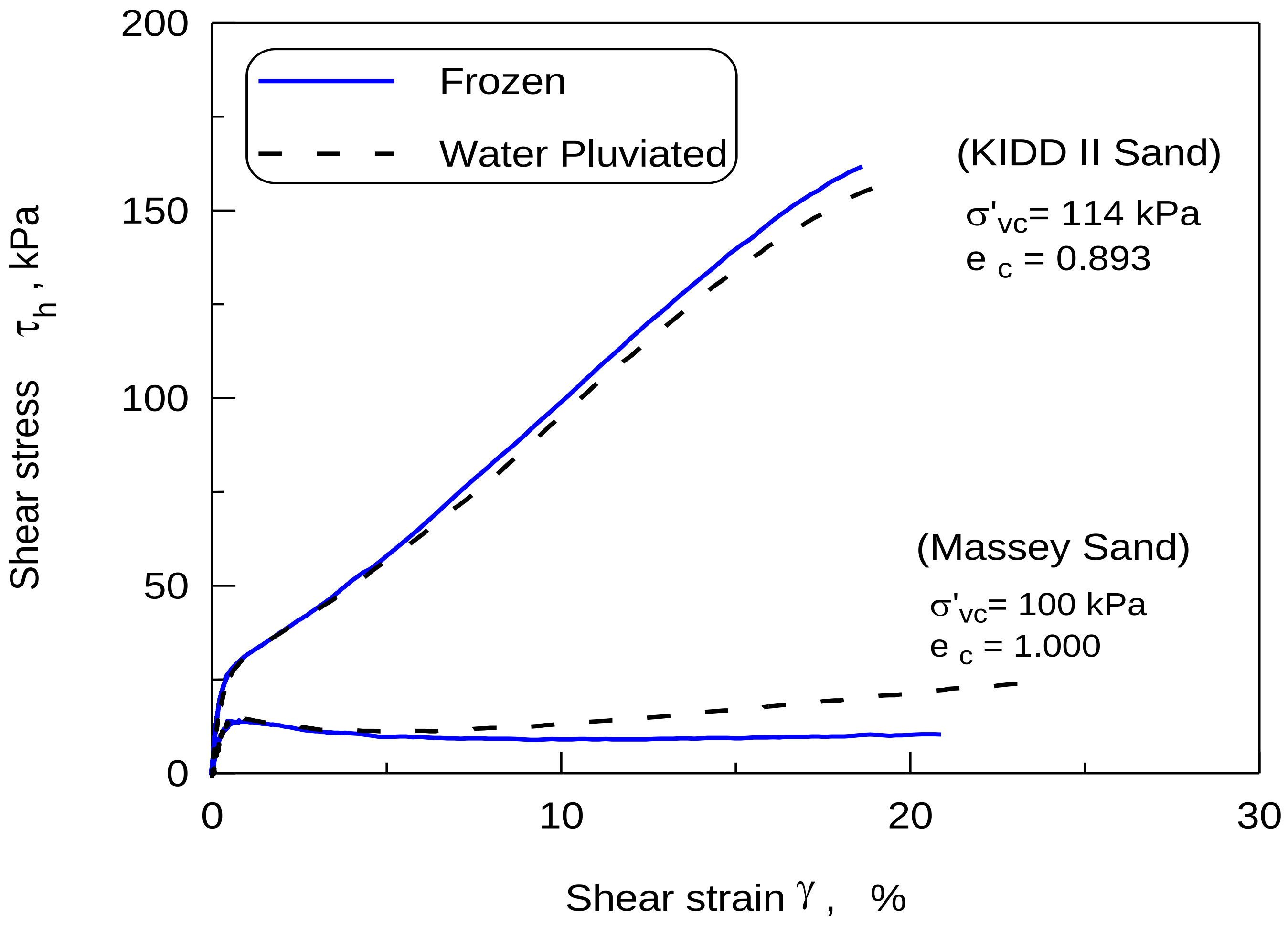
Prediction Exercise 2; Centrifuge Tests

Smith (1994) warned about this in his discussion of the VELACS project which evaluated how well different constitutive models predicted the results of centrifuge tests: “A particularly insidious feature of the calibration process is that a predictor could calibrate his/her model to fit the bulk of the (largely triaxial) data provided in the information package and still make poor predictions of seismically induced stress paths”

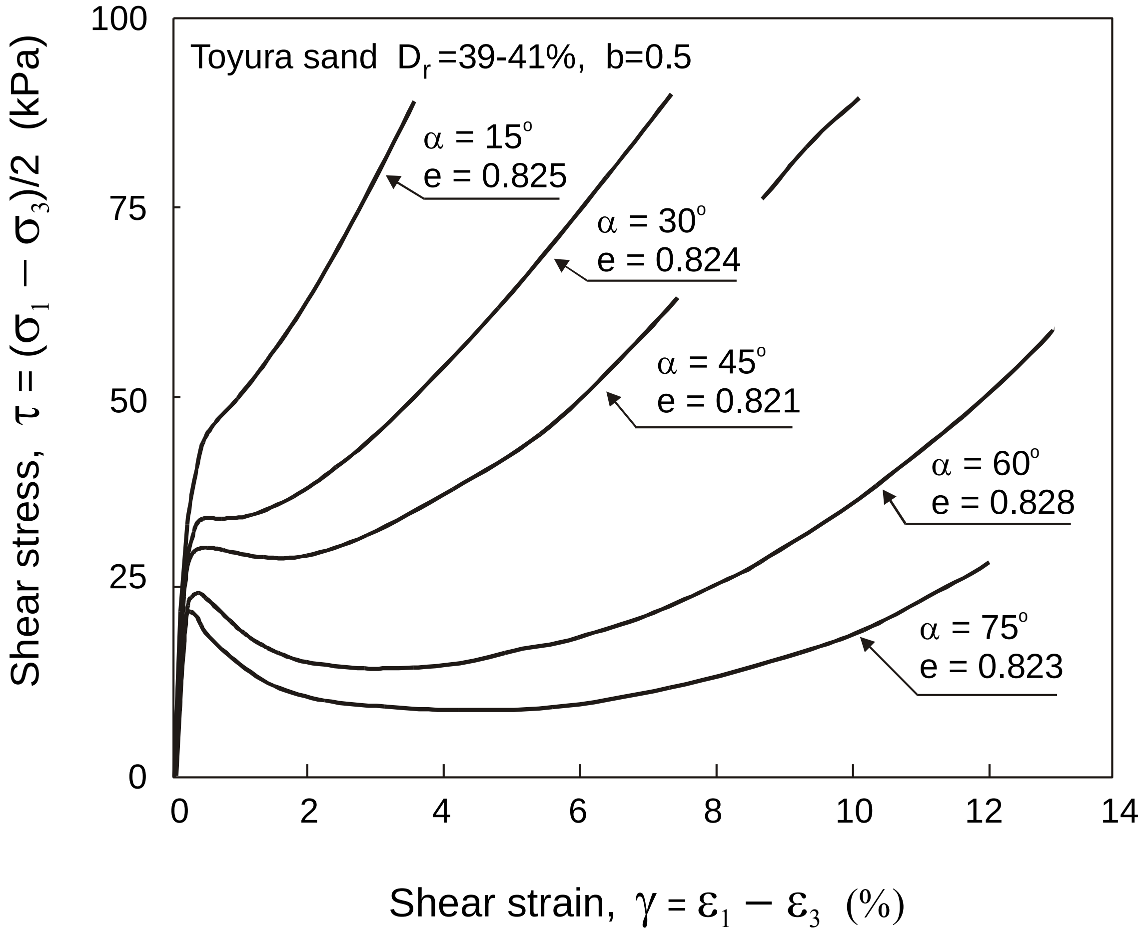
Effect of Sample Preparation



Case for Water Pluviation



Effect of Loading Path on Stress- Strain Response



For liquefaction field studies use cyclic simple shear test data to calibrate computational model in computer program, if possible.

Resistance Capacity

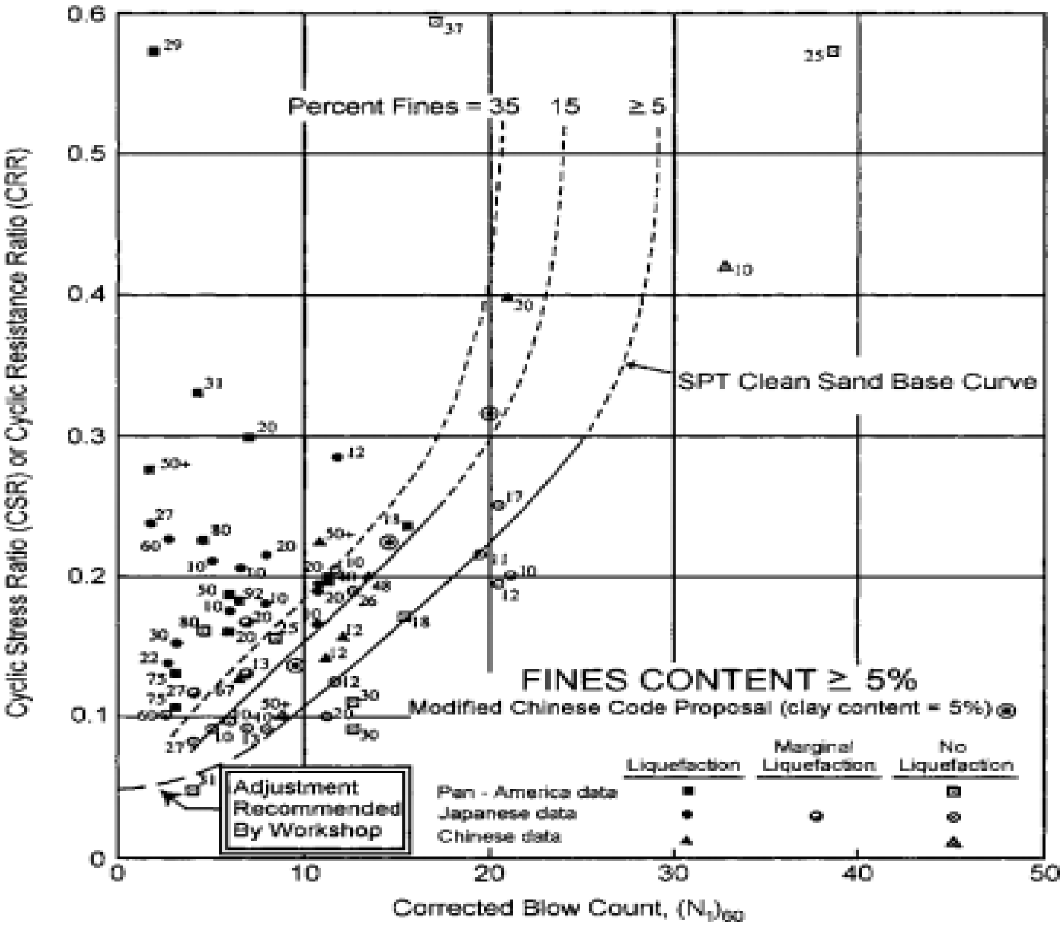
Development of SPT liquefaction triggering criterion CSR ≥ CRR

Ohsaki 1964 – Whitman1970 – Seed 1976
Seed et al 1985

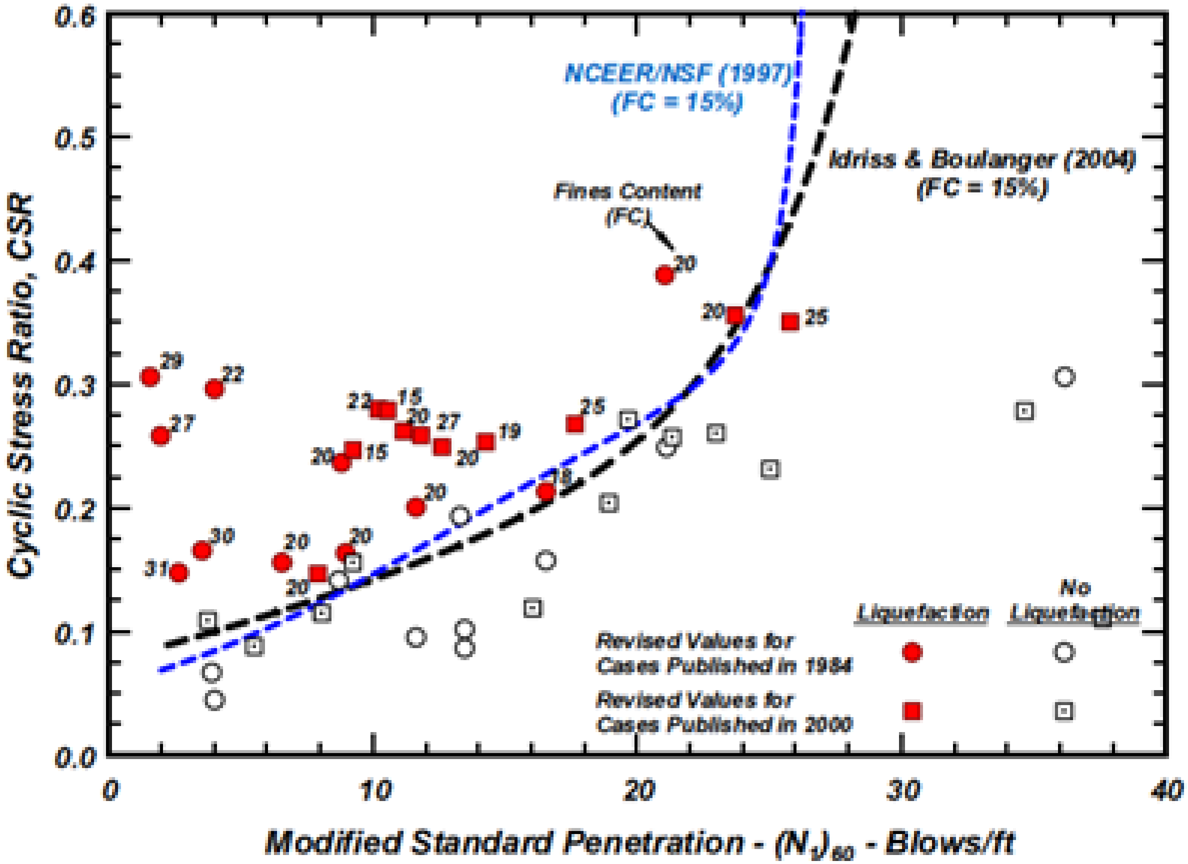
Youd et al 2001 State of Practice

Idriss and Boulanger,EERI Manual 2008 and associated seminar program leads to controversy and formation of NSF Committee to resolve issues by developing an acceptable new state of practice.

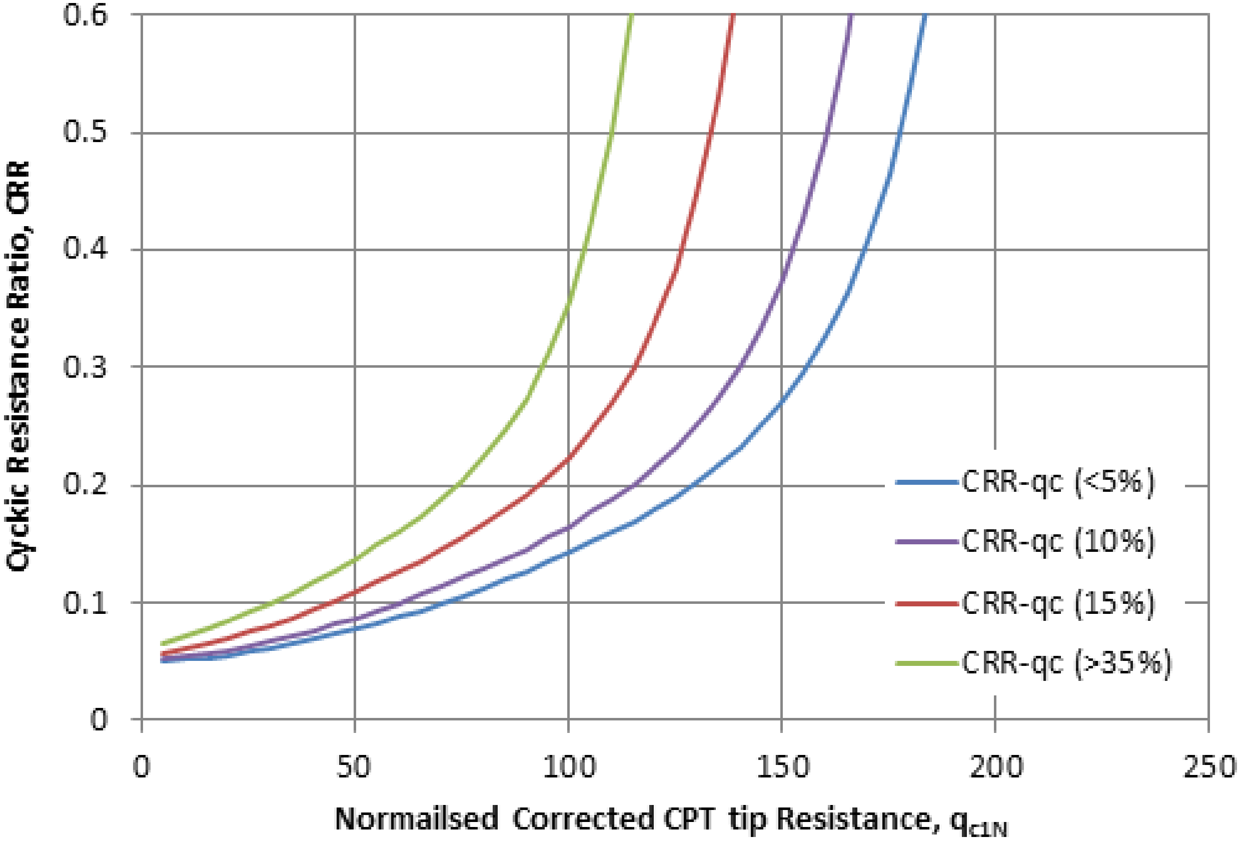
SPT- Liquefaction Assessment Chart



SPT case histories



CPT - Liquefaction Assessment Chart



Worked Examples
CLiq
(slick)

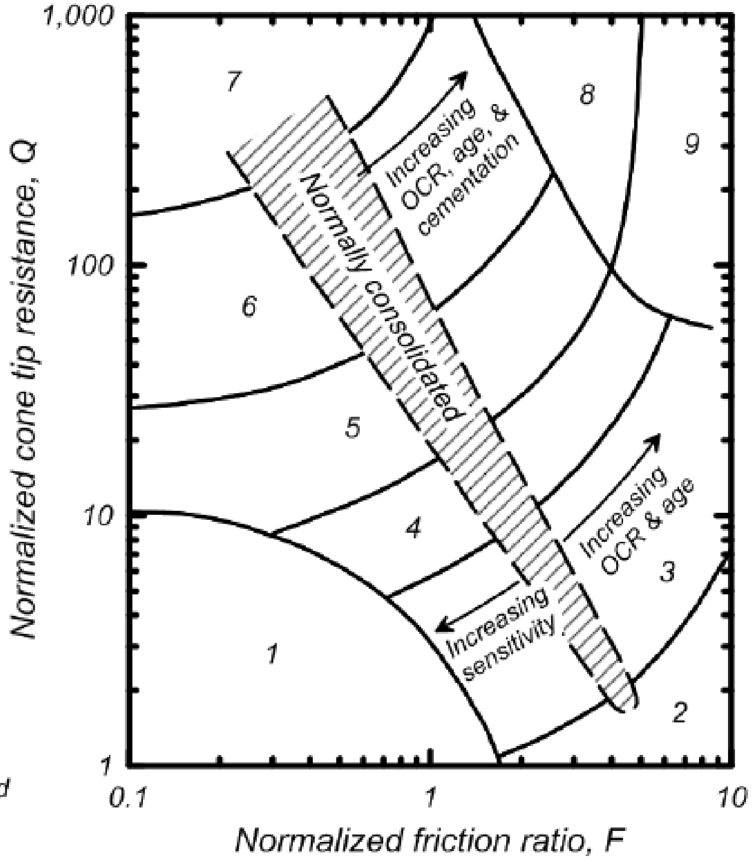
http://www.geologismiki.gr/

John Th. Ioannides

CLiq v.1.7

GeoLogismiki, in collaboration with Gregg Drilling Inc. and Prof. Peter Robertson, is pleased to announce the availability of our new Cone Penetration Test based soil liquefaction assessment software, *CLiq*.

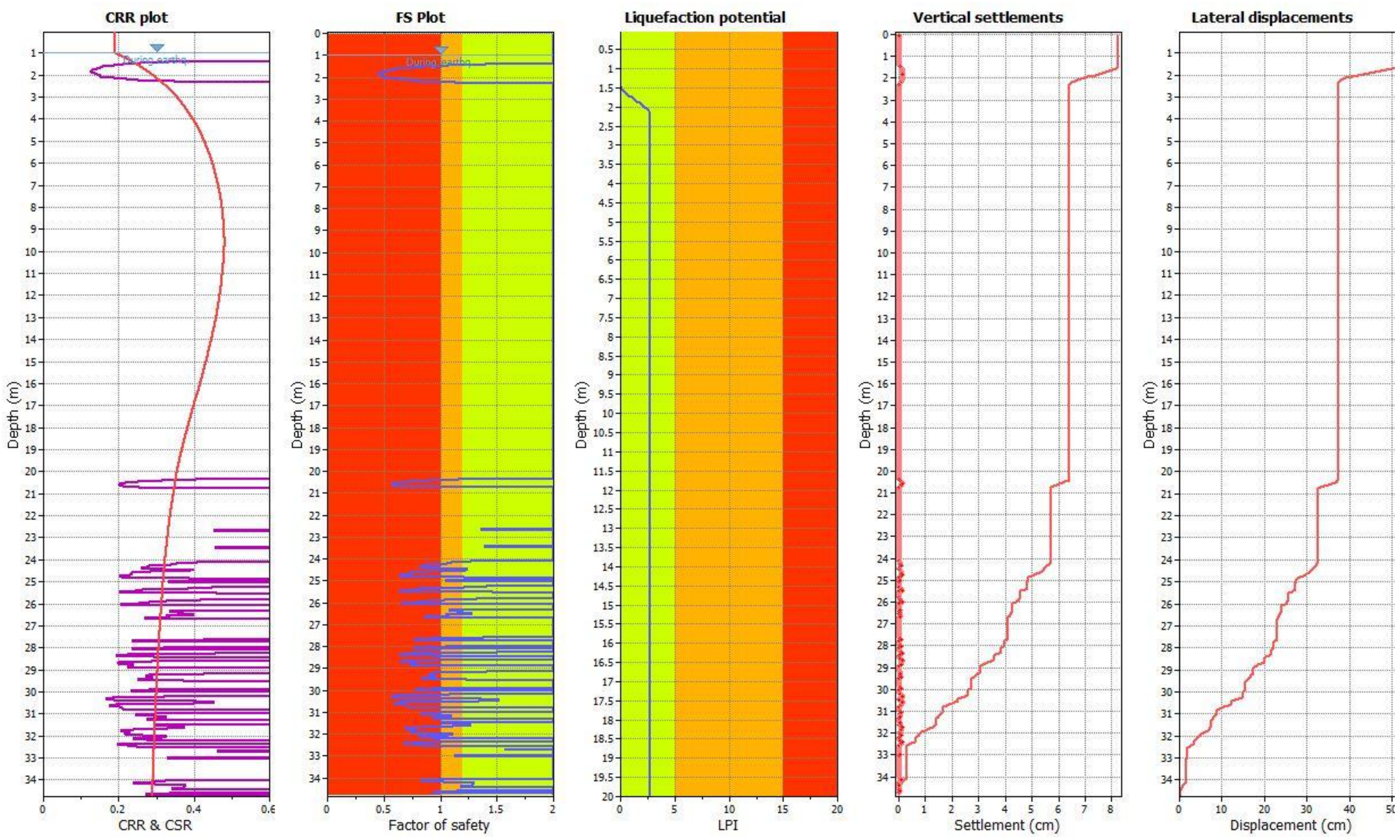
Soil Behavior Chart by Robertson



1. Sensitive, fine grained
2. Organic soils - peats
3. Clays - silty clay to clay
4. Silt mixtures - clayey silt to silty clay
5. Sand mixtures - silty sand to sandy silt
6. Sands - clean sand to silty sand
7. Gravelly sand to dense sand
8. Very stiff sand to clayey sand *
9. Very stiff, fine grained *

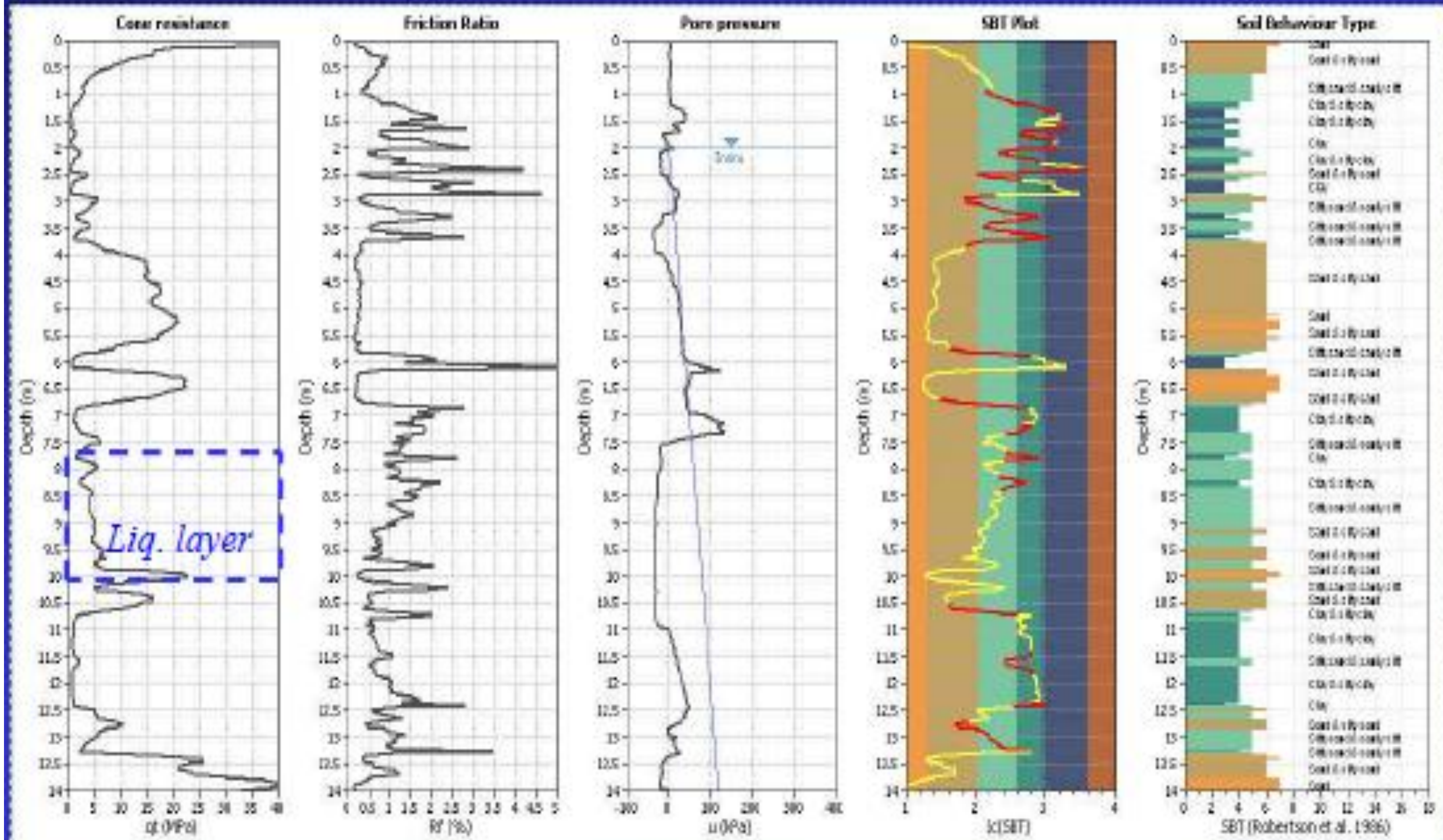
* Heavily overconsidated or cemented

Evaluation of Liquefaction Potential at a Site



Moss Landing - Lateral Spread

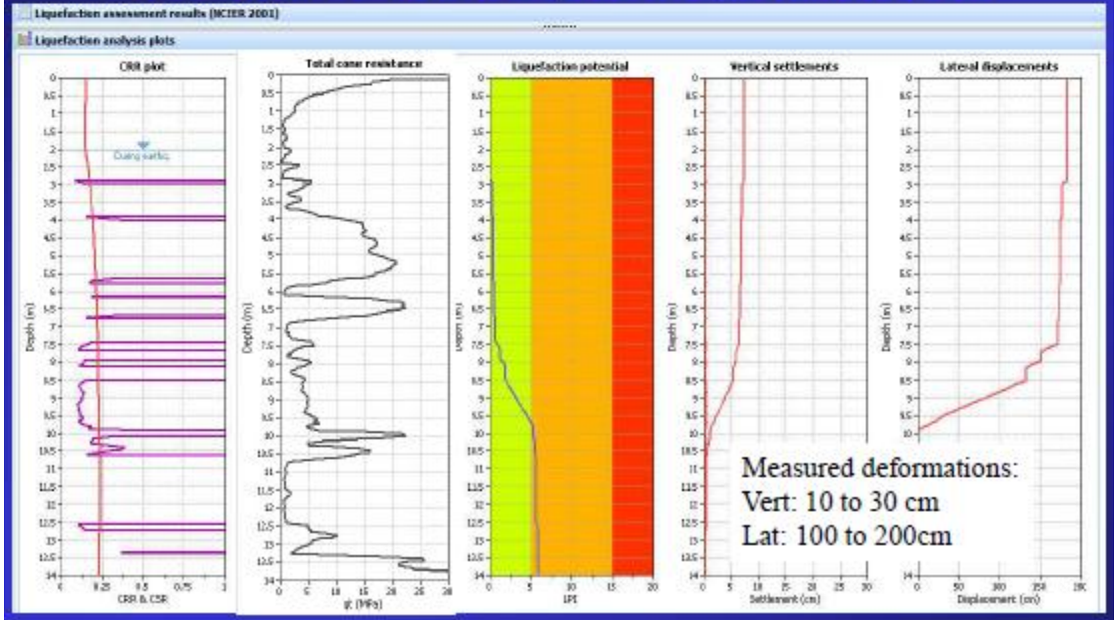
Moss Landing Marine Lab UC 8



$L = 20m$, $H = 5m$; $a_{(max)} = 0.28g$, $M = 6.9$

Moss Landing UC 8

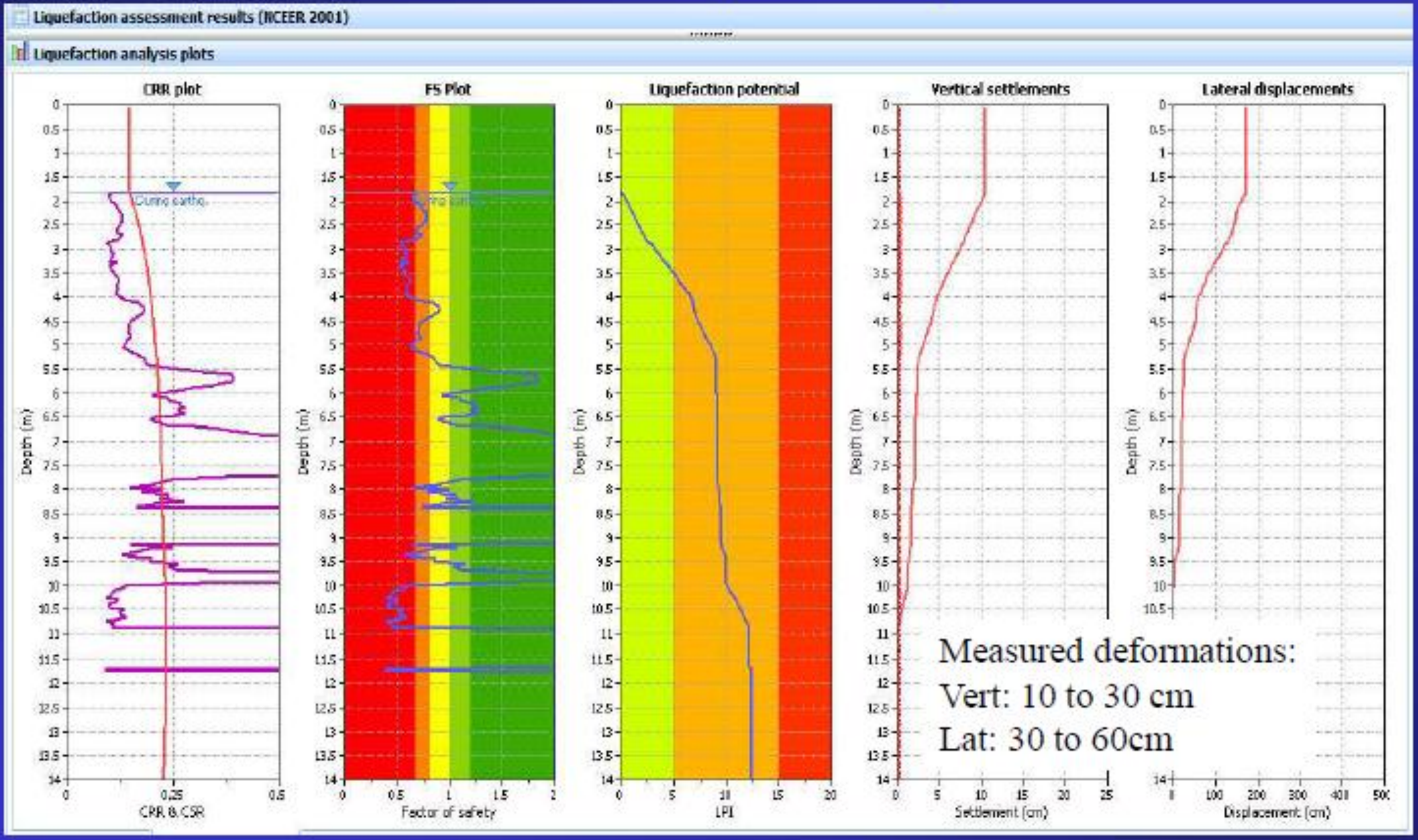
UC 8 NCEER (R&W) Method



L = 20m, H = 5m

Moss Landing UC 8

UC 14 NCEER (R&W) Method



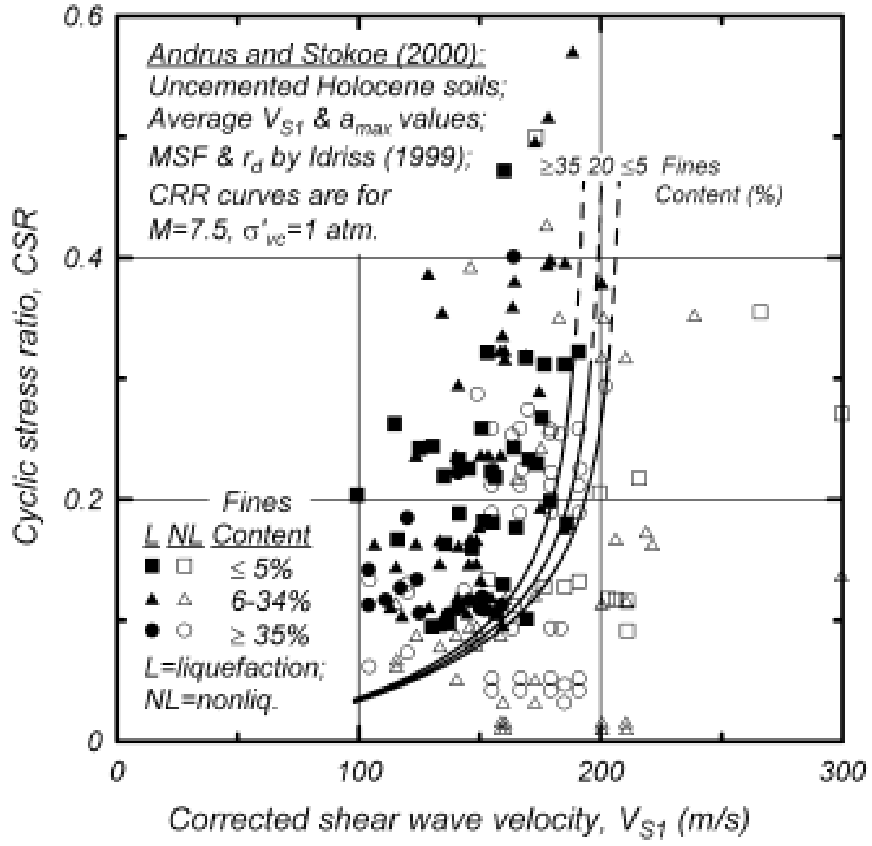
Example in I&B 2008 Appendix B: L = 20m, H = 5m

Moss Landing UC 14

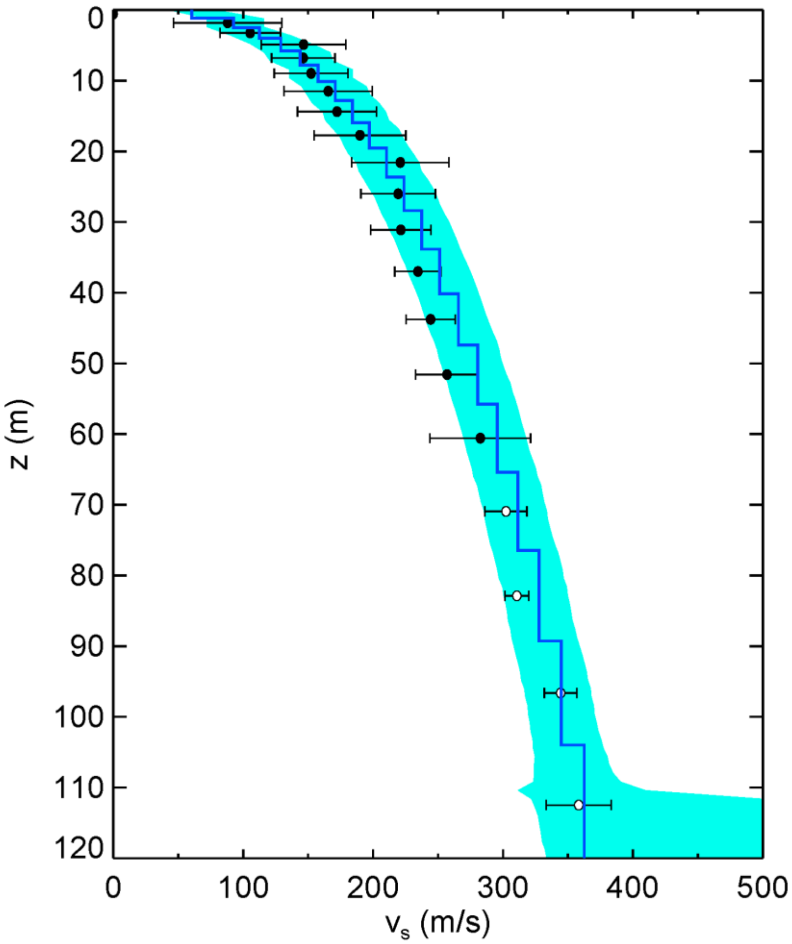
Locations of Liquefaction Testing

Contours of Liquefaction Index

Vs-based Liquefaction correlation for clean uncemented sands (after Andrus & Stokoe 2000)

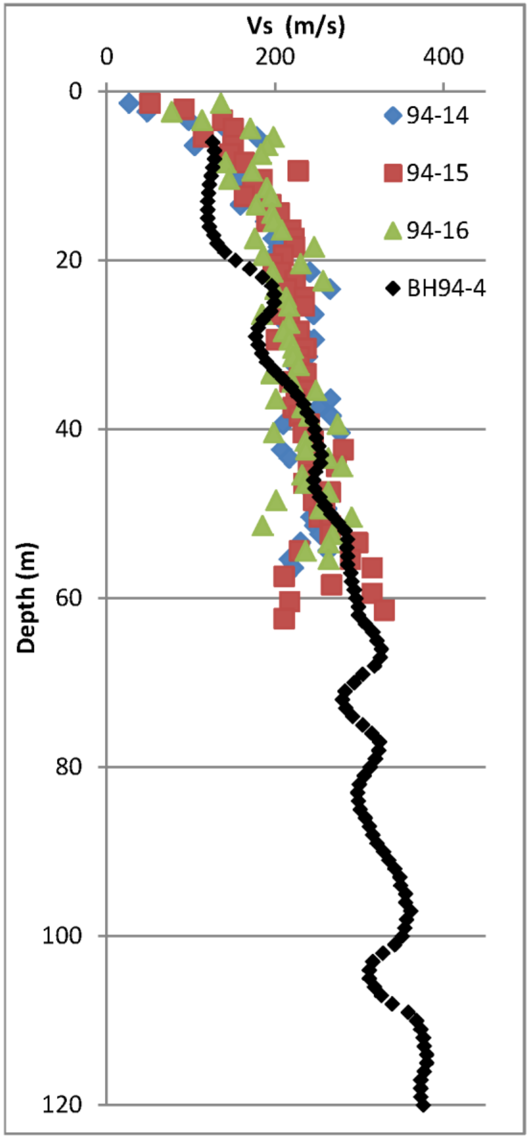


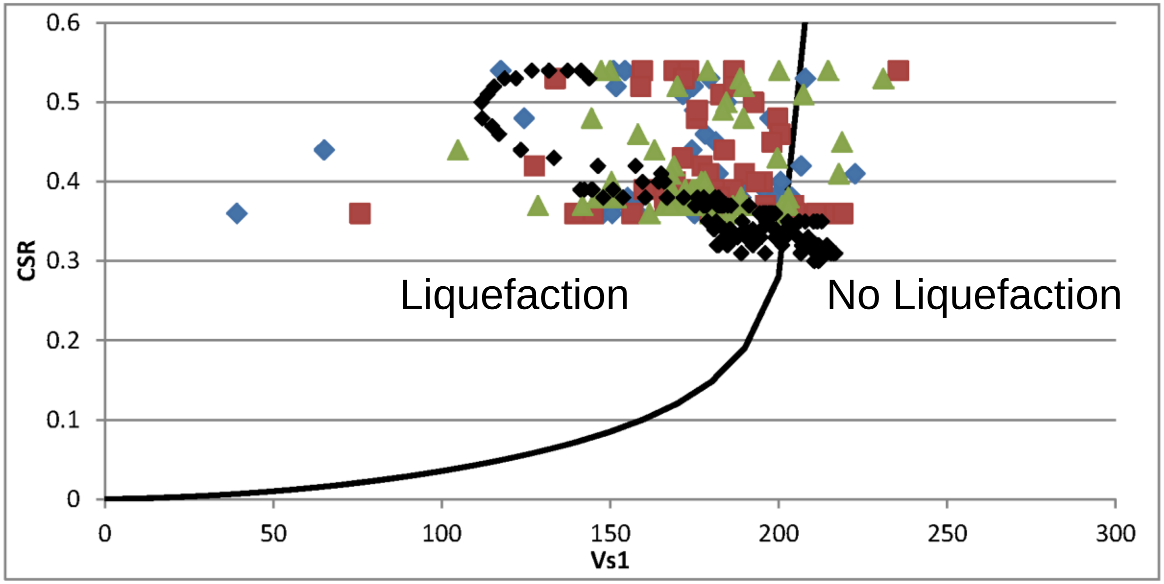
Velocity Vs by Inversion of Ambient Motions



- Filled circles depict averaged down-hole and SCPT measurements to 60 m depth.
- Open circles depict averaged down-hole only measurements.
- Average relative difference in V_S is **5%** between average geotechnical data and inversion result to 110-m depth.

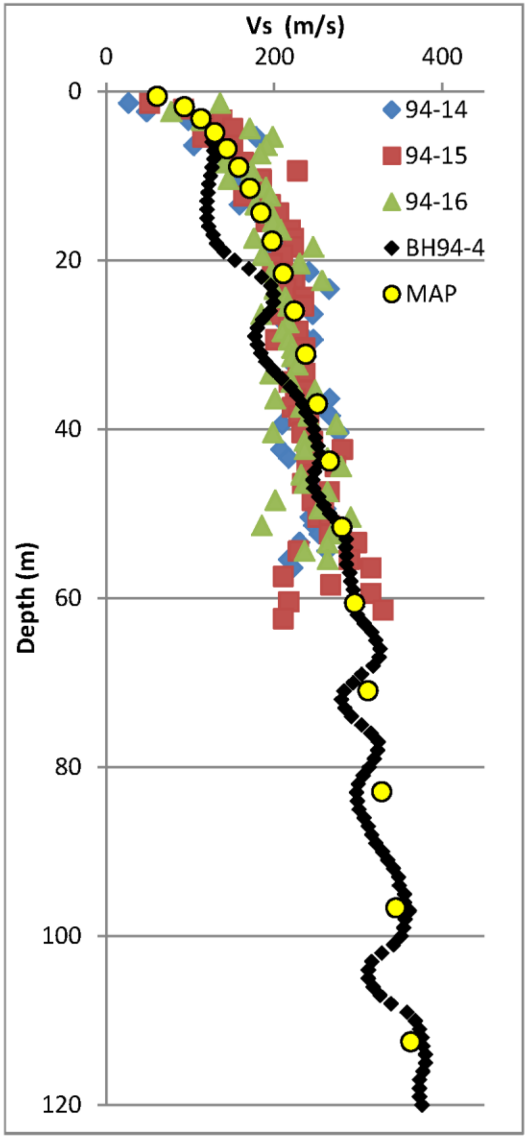
Liquefaction Triggering by Downhole V_s

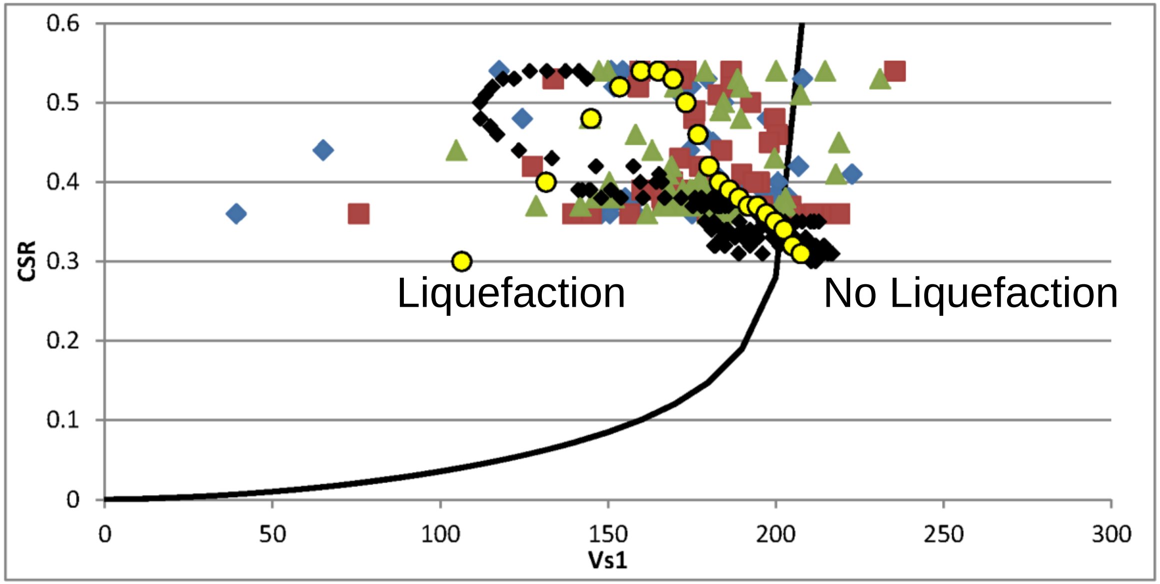




94-14: LPI = 81
94-15: LPI = 73
94-16: LPI = 63
Downhole: LPI = 77

Liquefaction Triggering using Ambient V_s





94-14: LPI = 81
94-15: LPI = 73
94-16: LPI = 63
Downhole: LPI = 77
Inversion: LPI = 69

CRR for Fine Grained Plastic Soils

1.The direct method using cyclic loading tests on high quality samples.

Otherwise:

2.Measure the monotonic undrained shear strength, S_u, in situ (Vane shear test or from CPT) or by test on high quality samples or

3.Estimate Su based on the stress history of the soil profile

Then estimate CRR from Su by empirical methods

Evaluating CRR for Fine Grained Plastic Soils

The cyclic shear stress τ_{cyc} is 65% of the peak shear stress as for sand but the ratio

$$CRR_{M = 7.5} = (\tau_{cyc}/S_u)_{N = 30} \, (S_u/\sigma^i_{vc})$$

$(\tau_{cyc}/S_u)_{N = 30}$ is evaluated from a substantial data base for N = 30 cycles when M = 7.5.

The value 0.83 was selected for clay-like soils subjected to direct simple shear loading conditions.

This value may change as more data as more data becomes available.

$CRR_{clay} = 0.8 \, (S_u/\sigma^i_{vc})$ (15)

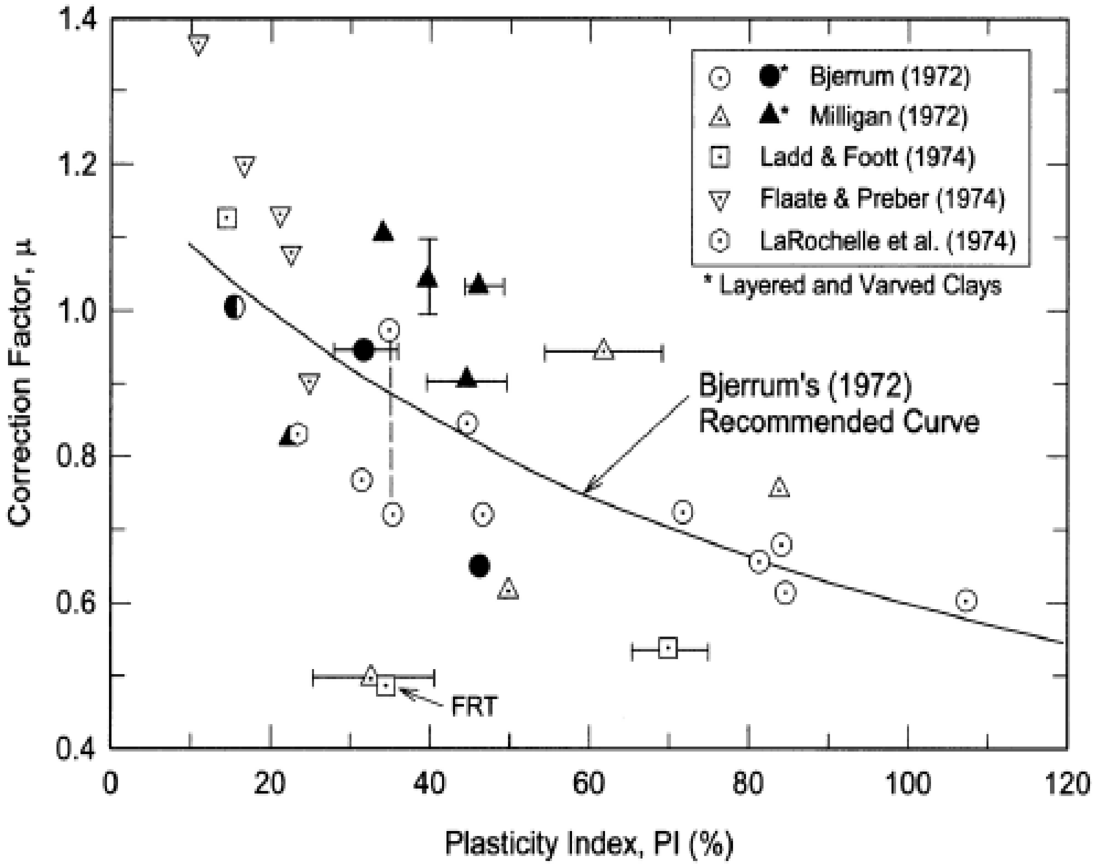
For the present, CRR is given by

$$\mathbf{CRR_{clay} = 0.83 \, (S_u/\sigma^i_{vc})}$$

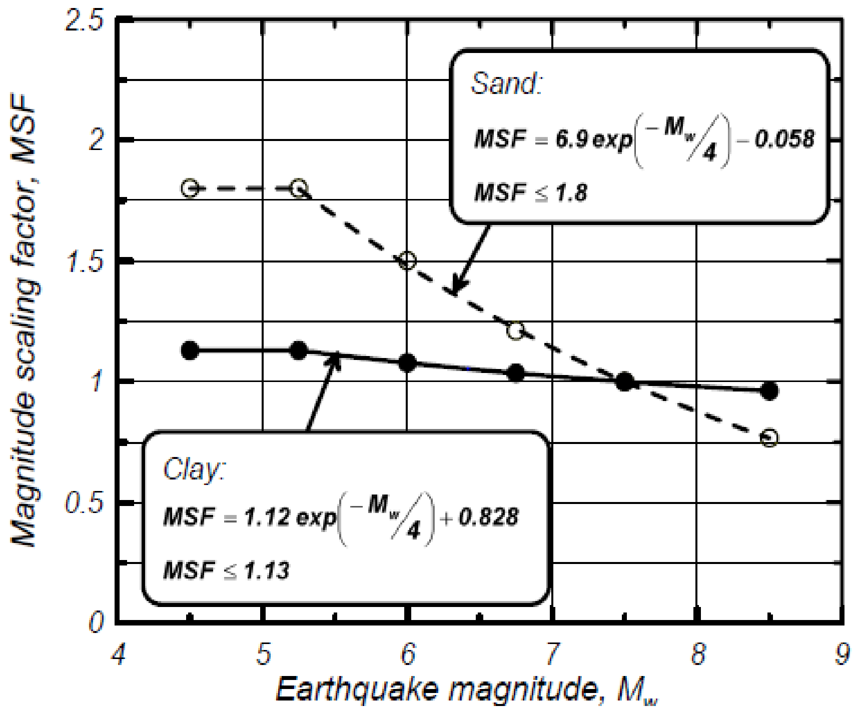
If a correction factor C_{2D} = 0.96 is included to represent the fact that motions occur in the field in two directions then:

$$\mathbf{CRR_{clay} = 0.8 \, (S_u/\sigma^i_{vc})}$$

Bjerrum Vane Shear Correction Factor

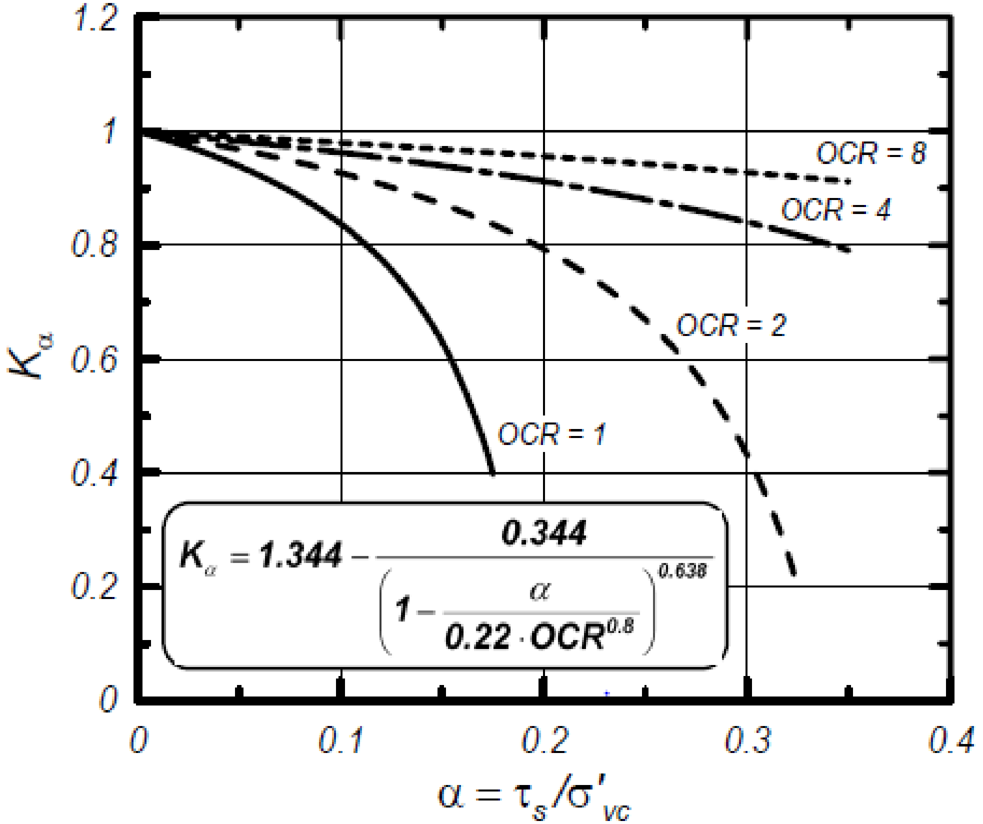


Magnitude Scaling Factors for Sands and Clays



Boulanger and Idriss 2004

Slope Correction Factor Kα for Plastic Soils



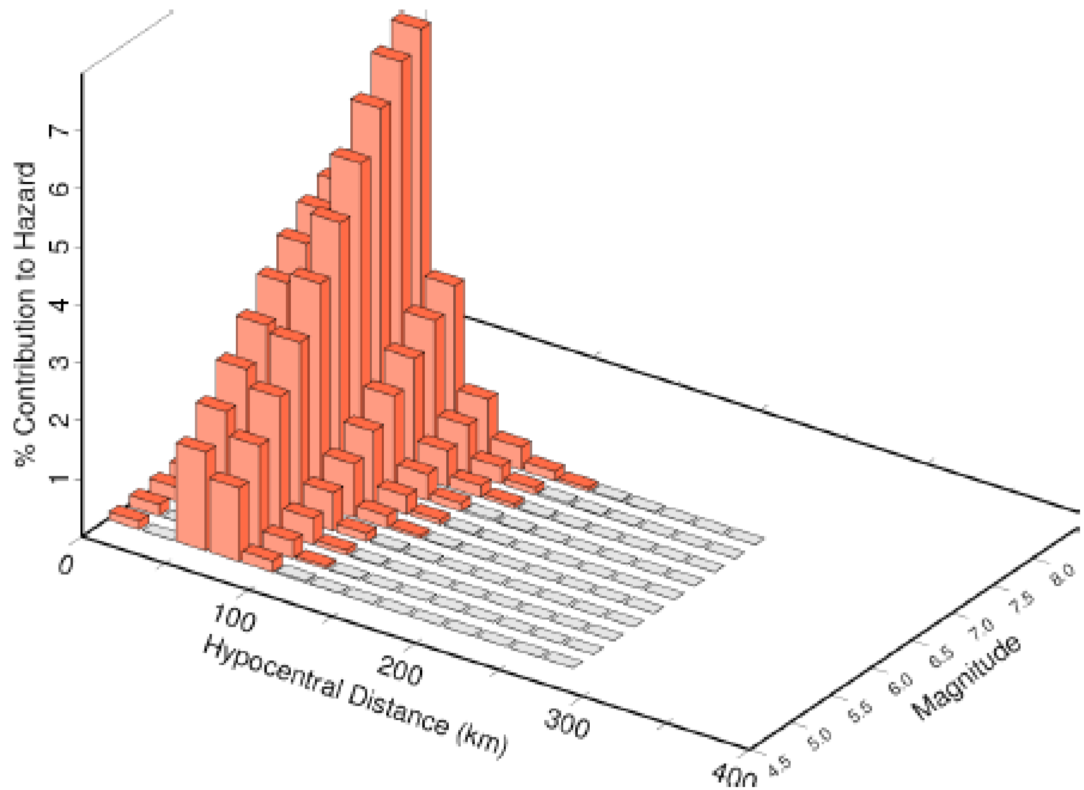
The challenge of probabilistic ground motions

The simplified method is based on an associated M and Acceleration

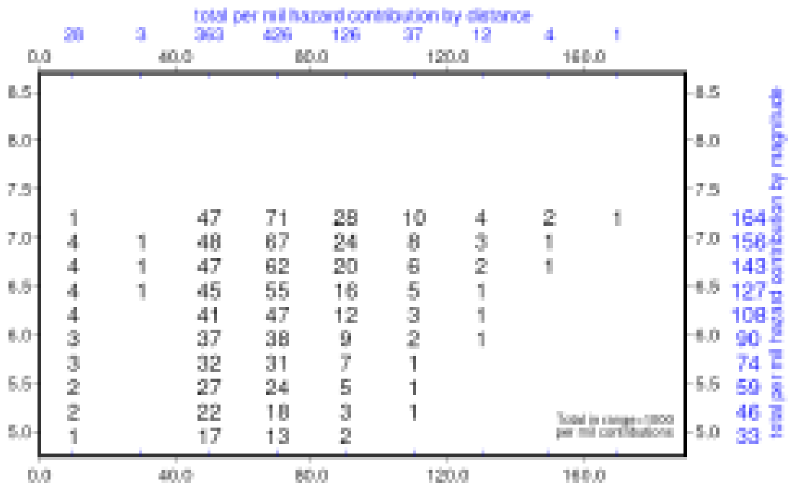
Probabiliistic accelerations result from contributions of all magnitudes between considered Mmin and Mmax. So how do you employ the simplified method?

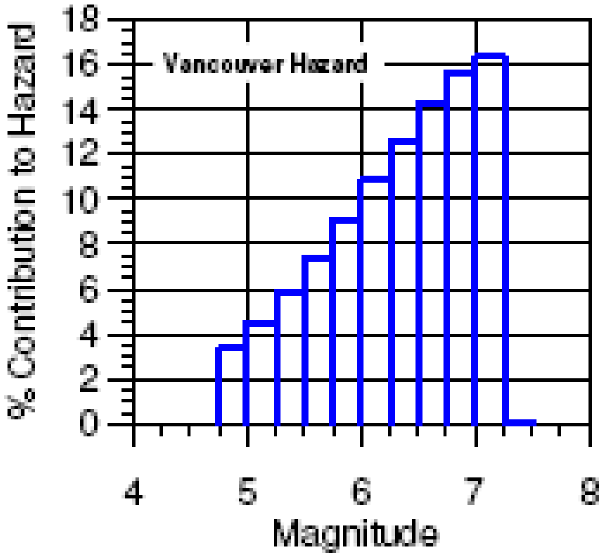
Serious implications also for lateral spreading and settlement. Discussed later.

Magnitude-distance deaggregation for NBCC 2005 PGA in Vancouver

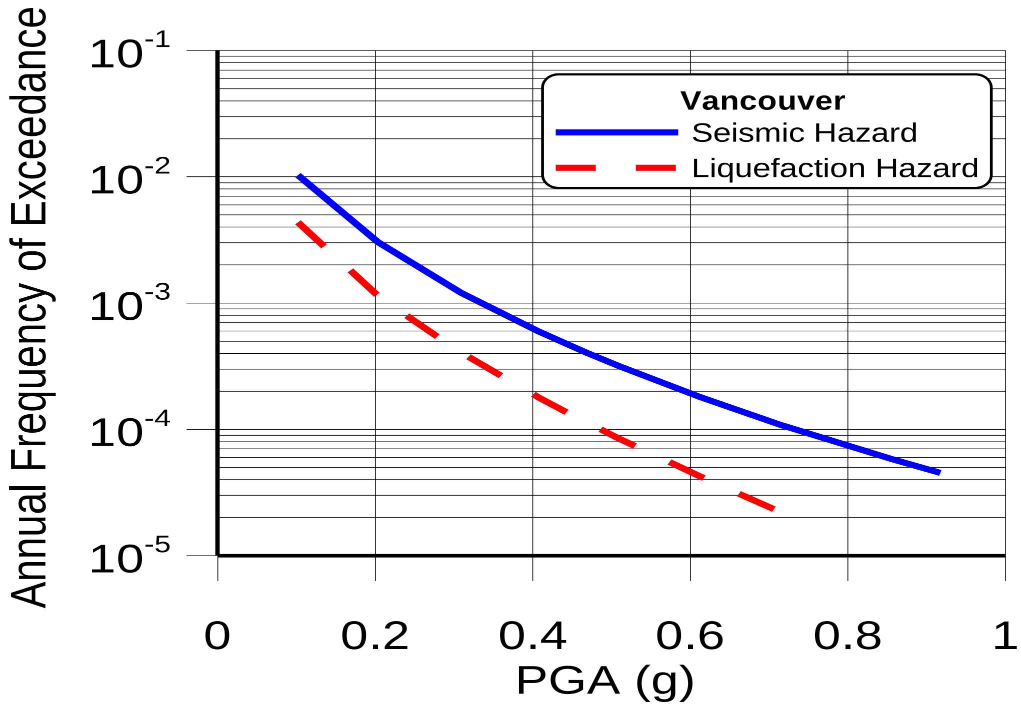


Contribution of each 'Bin Magnitude'

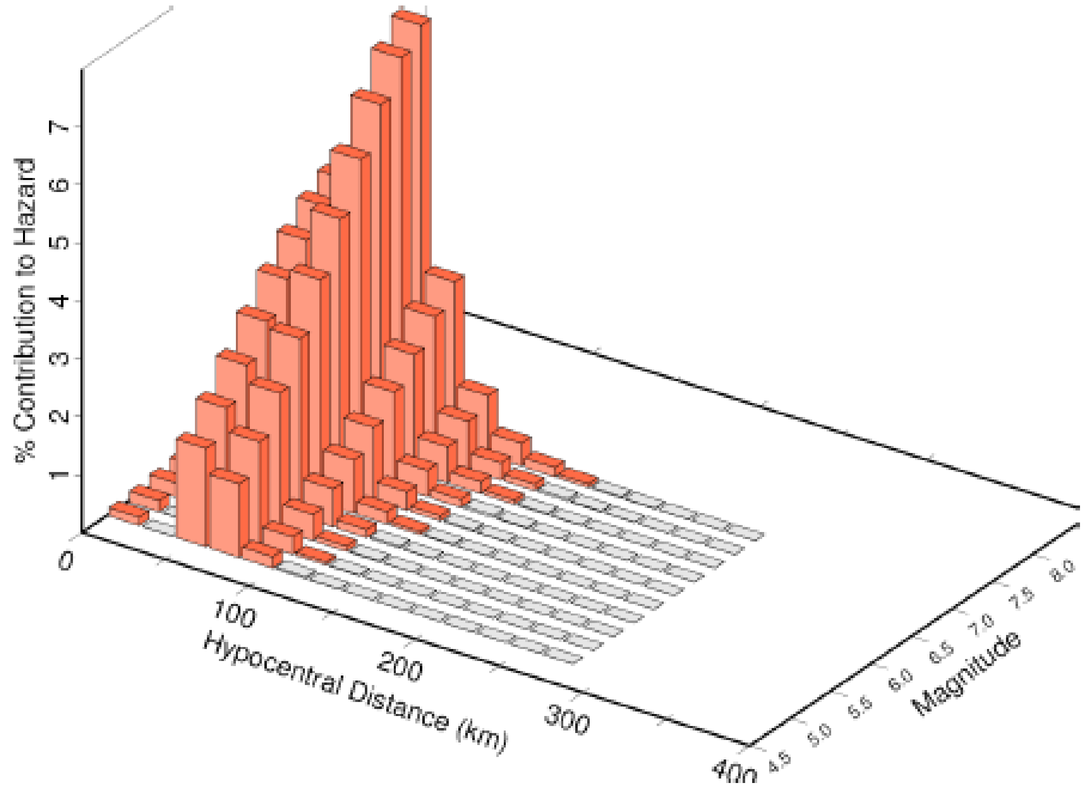




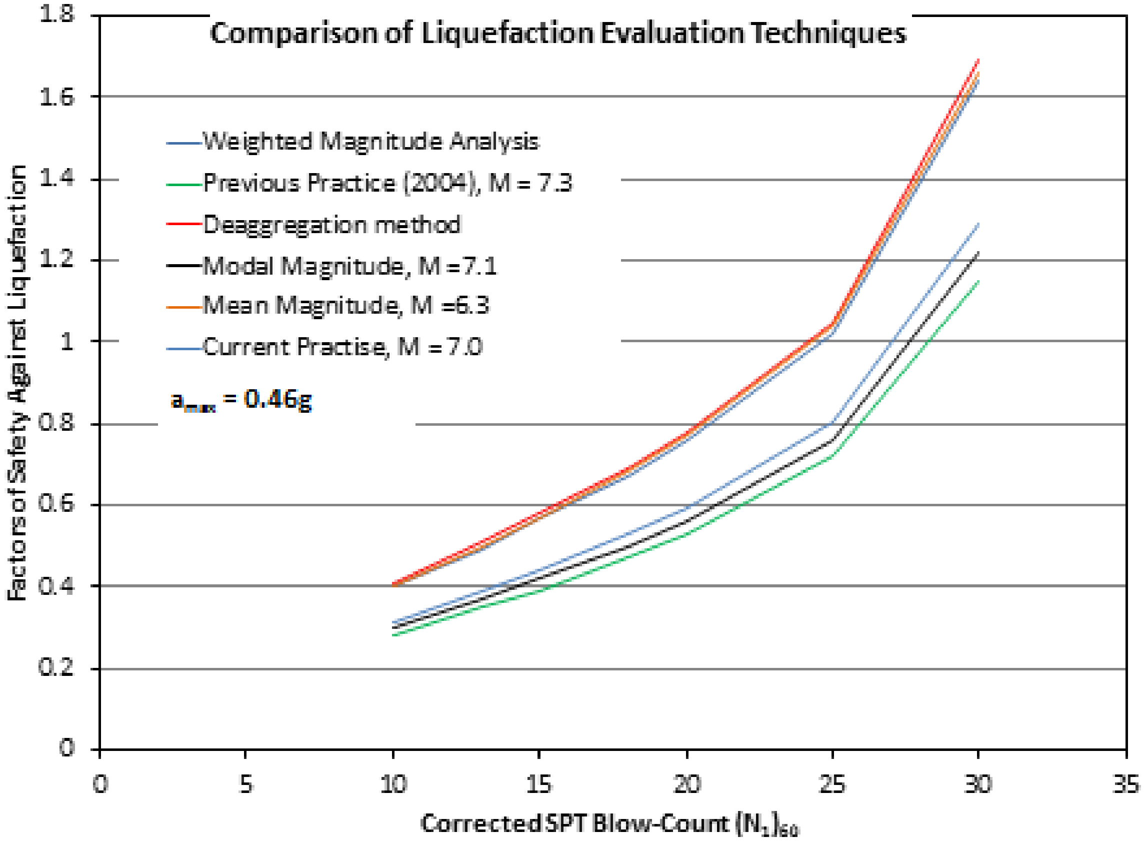
Liquefaction hazard curve weighted for magnitude M = 7.5
First proposed by Idriss, 1984.



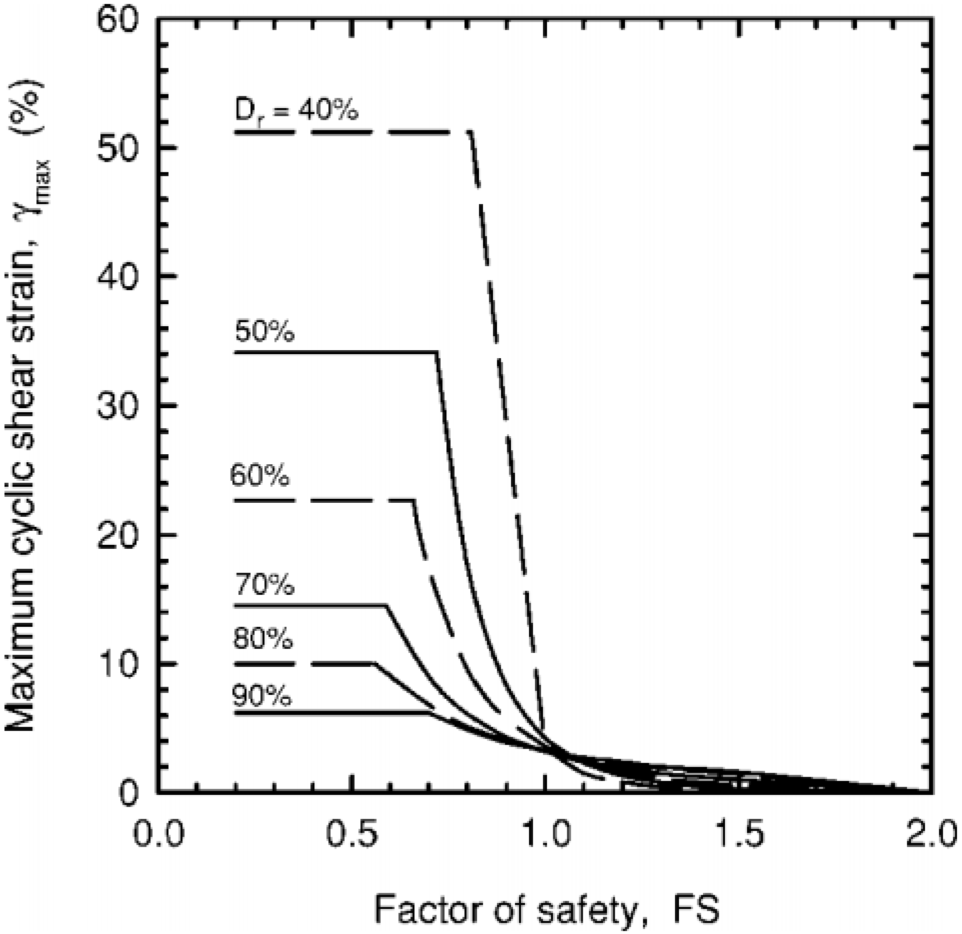
Magnitude-distance deaggregation for NBCC 2005 PGA in Vancouver



Liquefaction potential for various $(N_1)_{60,cs}$ values and seismic site conditions, using Youd et al. (2001)



Maximum cyclic shear strain V_s, FS and D_r

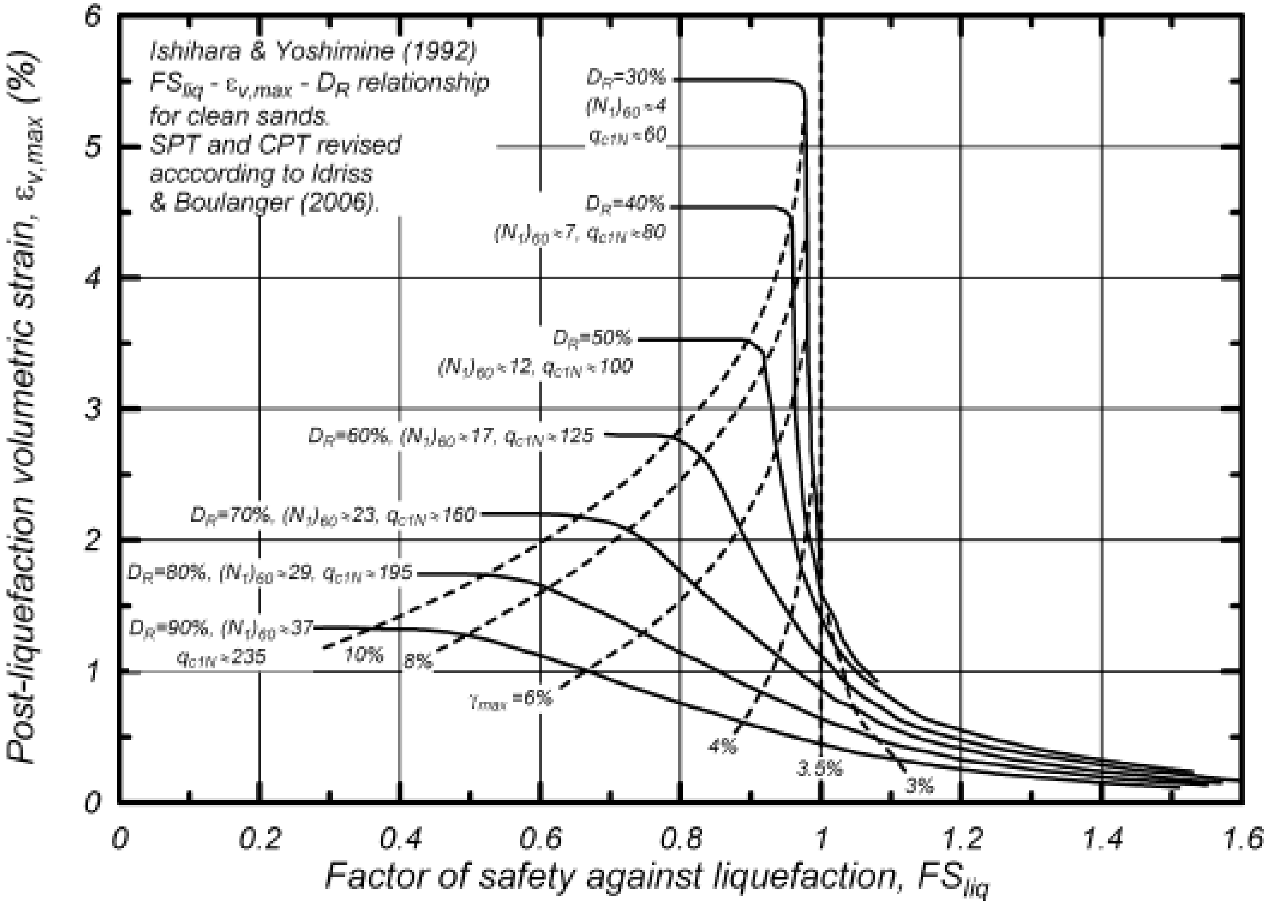


Expected lateral spreading displacement

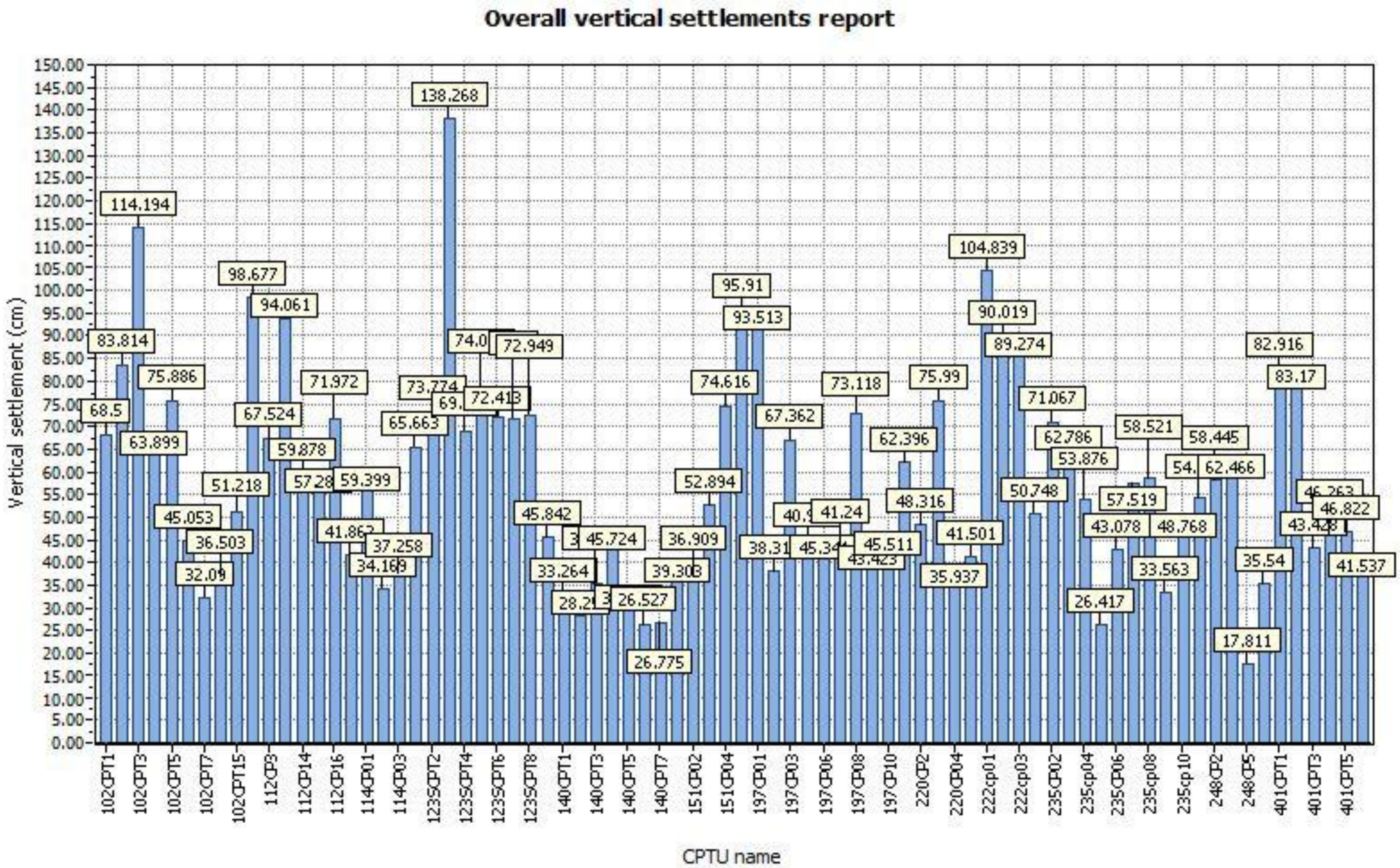
$$LDI = \int_0^{Z_{max}} \gamma_{max} dz$$

$$D_H = \begin{cases} (S+0.2)LDI, & \text{ground slope case} \\ 6W^{-0.8}LDI, & \text{free face case} \end{cases}$$

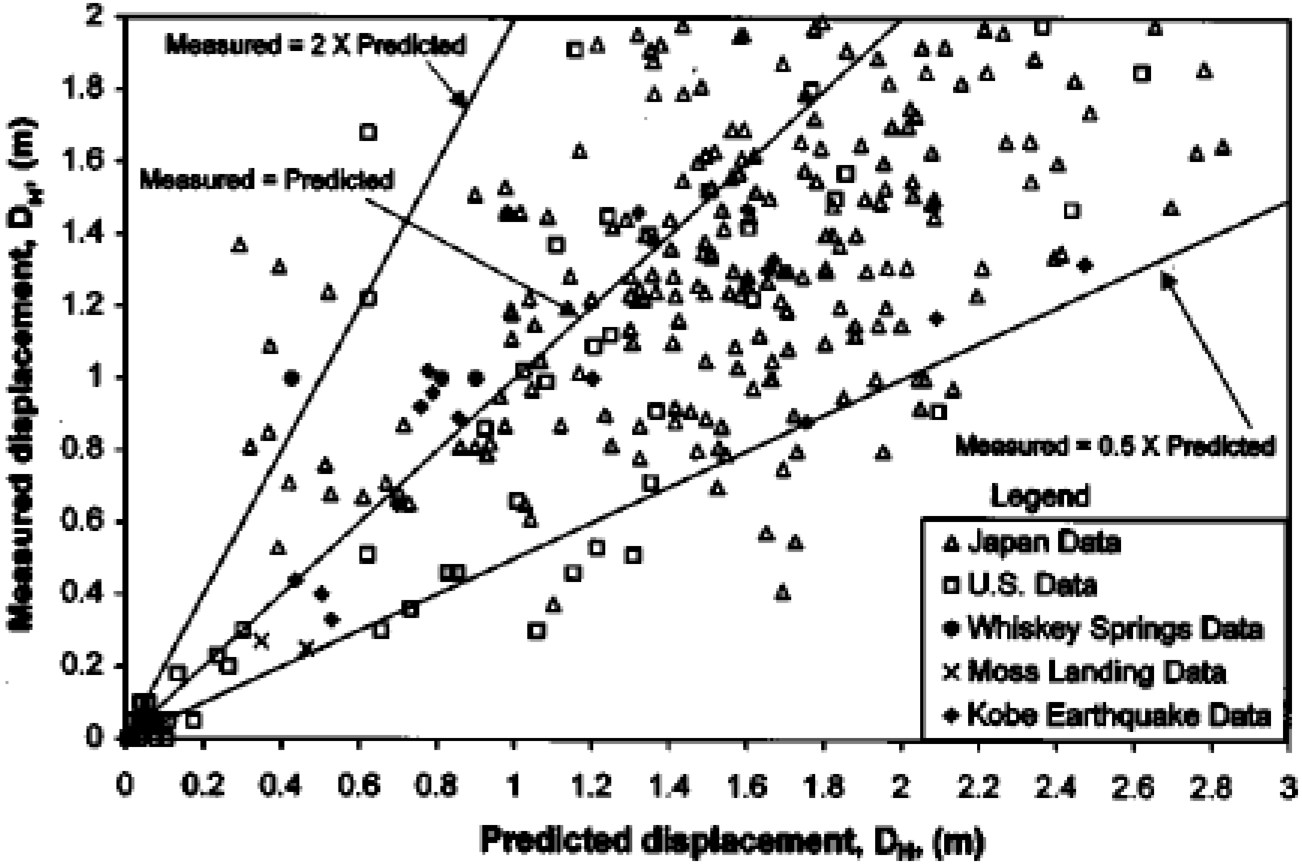
Variation of volumetric strain with relative density, SPT and CPT resistance, and FS against liquefaction



Overall Vertical Settlements



Measured versus predicted displacements



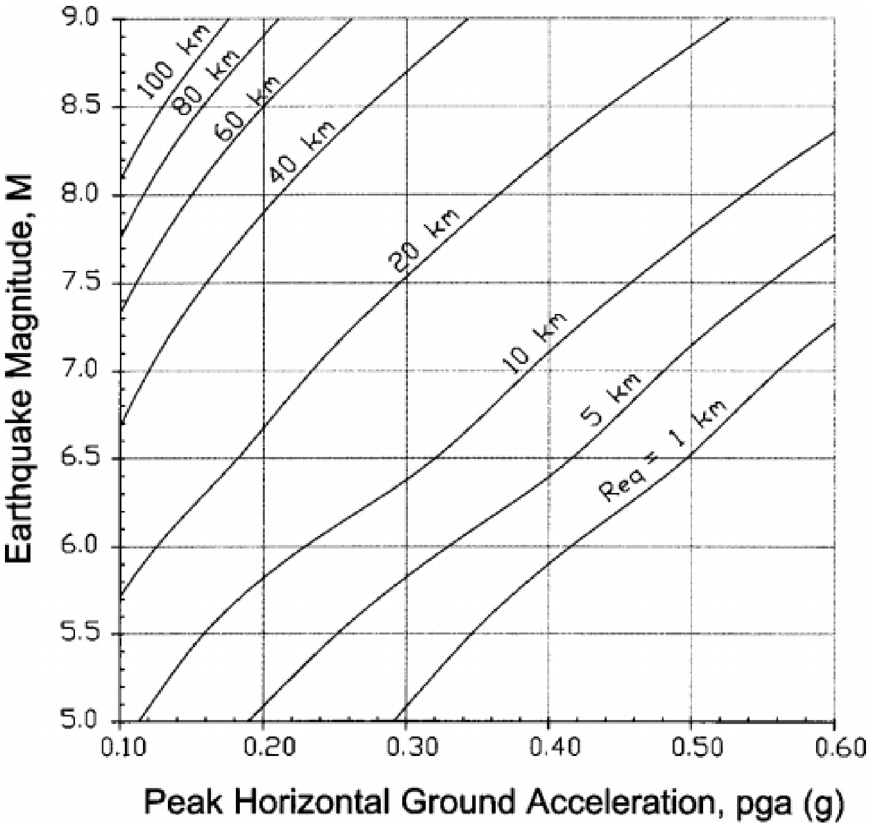
Lateral Displacement Equation

Youd's Equation:

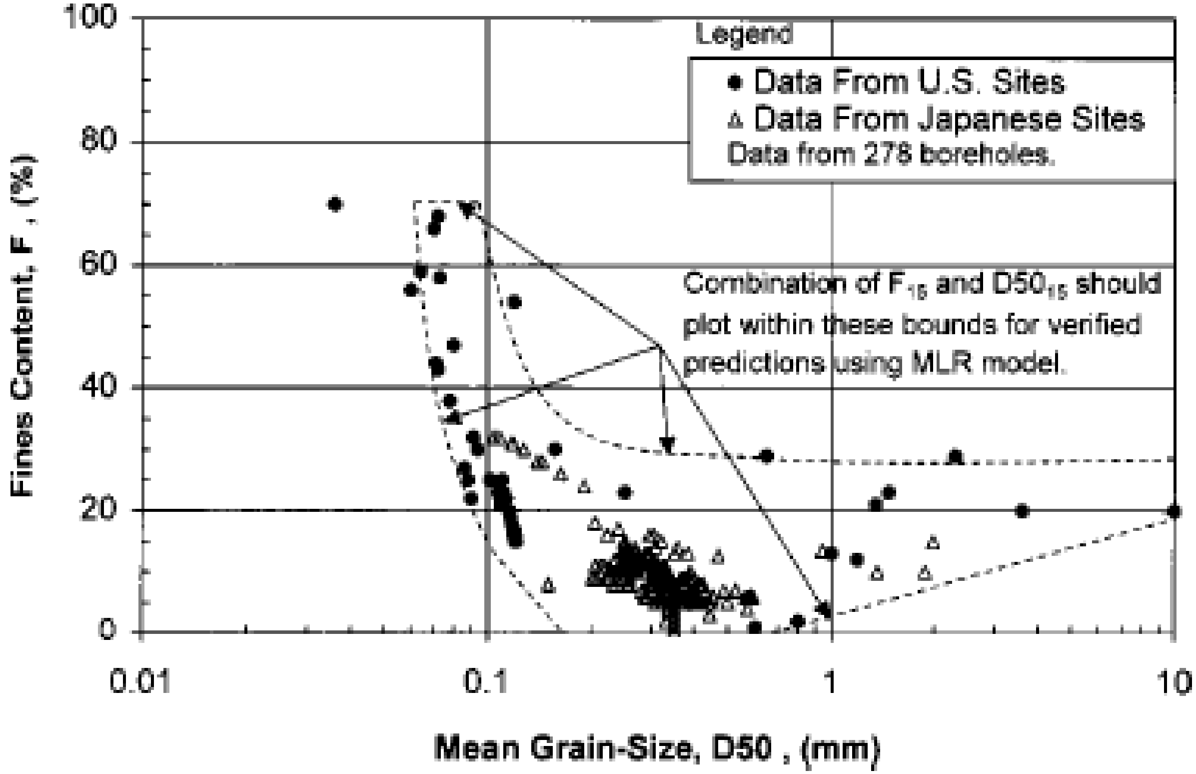
$$\log D_H = b_0 + b_1 M_w + b_2 \log R^* + b_3 R + b_4 \log W + b_5 \log S$$

$$+ b_6 \log T_{15} + b_7 \log(100 - F_{15}) + b_8 \log(D50_{15} + 0.1\ \text{mm})$$

Determining equivalent source distance



The PGA is the average of 3 GMPE for Site Class D


Legend
Data From U.S. Sites
Data From Japanese Sites
Data from 278 boreholes.
Combination of F15 and D5015 should plot within these bounds for verified predictions using MLR model.
Fines Content, F, (%)
Mean Grain-Size, D50 , (mm)
0
20
40
60
80
100
0.01
0.1
1
10

Input parameters for school project

Input Parameters	
$D50_{15}$ (mm)	0.25
F_{15} (%)	5
T_{15} (m)	3
S (%)	4
M	7

Accelerations for calculating D

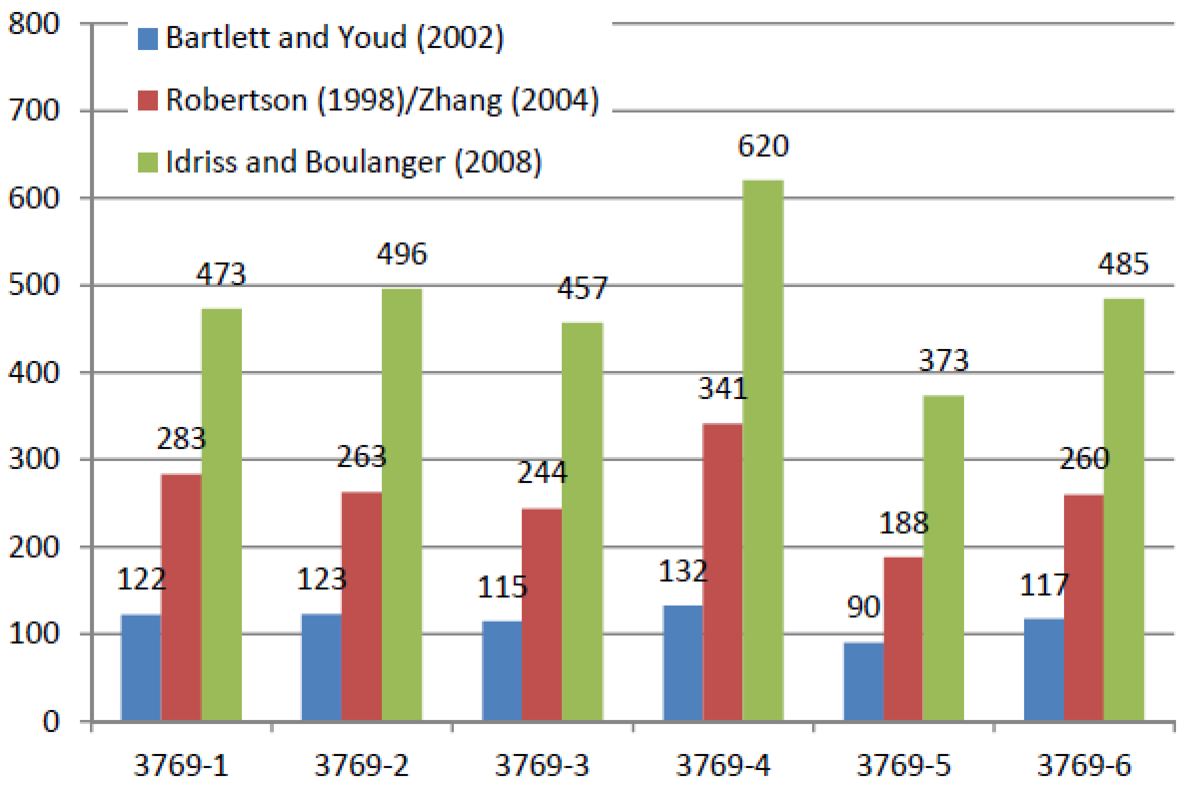
Acceleration for Calculating D	Distance, D (km)	Lateral Displacement (m)
0.20g Avg. Hazard Acceleration, ε = 0	D = 25	0.54
0.35g Site Response Analysis	D = 12	1.79
0.46g Code Acceleration (Site Class C)	D = 6	4.10

Sample calculation of lateral spreading displacements using the deaggregation method

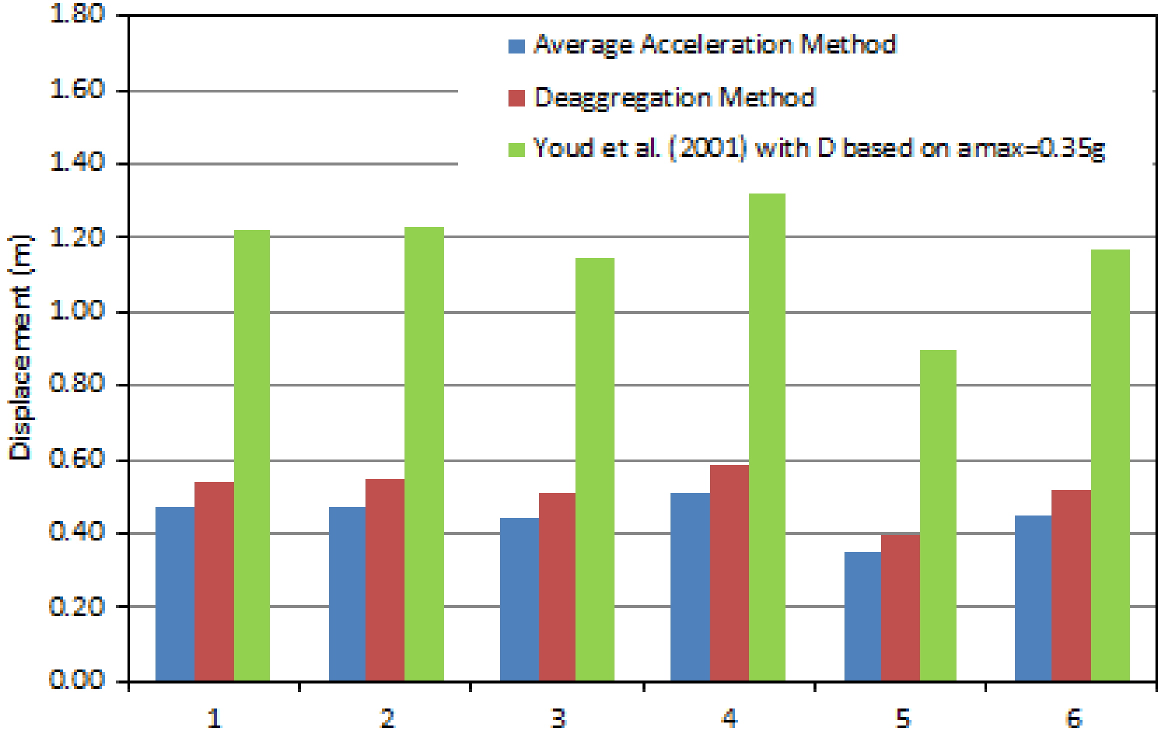
	Distance Bins																
Magnitude Bins	2.5	7.5	12.5	17.5	22.5	27.5	32.5	37.5	42.5	47.5	52.5	57.5	62.5	67.5		197.5	Row Totals:
5.1	0	0	0	0	0	0	0	0	0	0	0	0	0	0		0	0
5.3	0	0	0	0	0	0	0	0	0	0	0	0	0	0		0	0
5.5	0	0.0001	0	0	0	0	0	0	0	0	0	0	0	0		0	1E-04
5.7	0	0.0004	0	0	0	0	0	0	0	0	0	0	0	0		0	4E-04
5.9	0	0.0008	0.0001	0	0	0	0	0	0	0	0	0	0	0		0	9E-04
6.1	0	0.0018	0.0003	0	0	0	0	0	0	0	0	0	0	0		0	0.002
6.3	0	0.0038	0.0007	0.0002	0	0	0	0	0	0	0.0001	0	0	0		0	0.005
6.5	0	0.0076	0.0017	0.0004	0.0001	0	0	0	0	0	0.0002	0.0002	0.0001	0		0	0.01
6.7	0	0.0145	0.0039	0.0011	0.0003	0.0001	0	0	0	0	0.0005	0.0004	0.0003	0.0002		0	0.021
6.9	0	0.0256	0.0082	0.0026	0.0009	0.0004	0.0001	0	0	0	0.0009	0.0008	0.0005	0.0004		0	0.041
7.1	0	0.0421	0.016	0.0057	0.0021	0.001	0.0004	0.0002	0.0001	0	0.0009	0.0008	0.0005	0.0004		0	0.071
7.3	0	0.064	0.0291	0.0119	0.0049	0.0024	0.001	0.0005	0.0003	0.0001	0	0	0	0		0	0.114
7.5	0	0.0897	0.0487	0.0227	0.0102	0.0053	0.0024	0.0014	0.0008	0.0004	0.0002	0.0001	0	0		0	0.182
7.7	0	0.0594	0.0374	0.0194	0.0095	0.0053	0.0026	0.0015	0.0009	0.0004	0.0003	0.0002	0	0		0	0.137
7.9	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8.1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8.3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		0
8.7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		0
8.9	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		0

Total of the Totals: 0.584

Using Different Methods



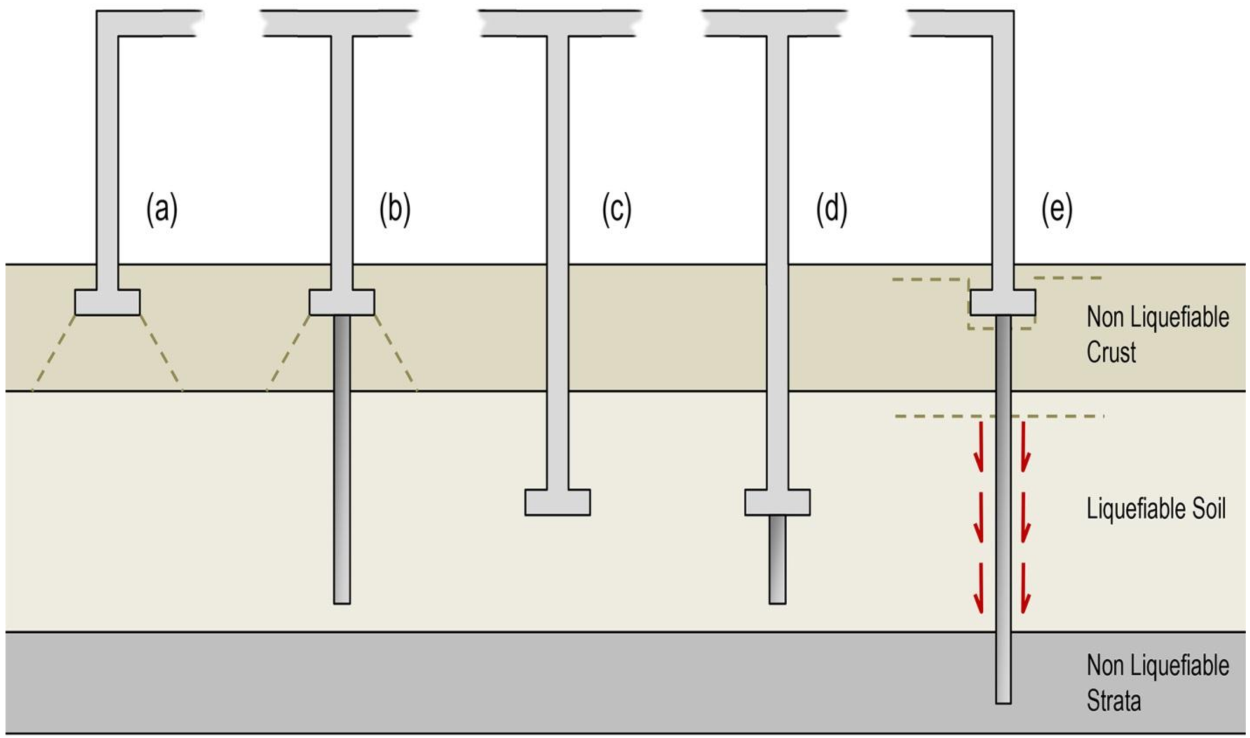
Calculated displacements for the school site in Delta



Closing the Loop

It is the usual procedure in the School Retrofit Program for the Geotechnical and Structural Engineers get together with at least two members of the Technical Review Board to decide how to deal with the consequences of Liquefaction in the most economical way.

An example of this cooperation follows for a particular school. The Structural Engineer involved is John Sherstobitoff, Ausenco Company, Vancouver, BC. The slides are abstracted from a recent presentation he made on the School Project.


(a)
(b)
(c)
(d)
(e)
Non Liquefiable Crust
Liquefiable Soil
Non Liquefiable Strata

Feb. 23rd, 2014

Seismic Retrofit Guidelines, 2nd Edition
November 4th, 2013

MANUAL VOLUME NO. 11

LIQUEFACTION GUIDELINES

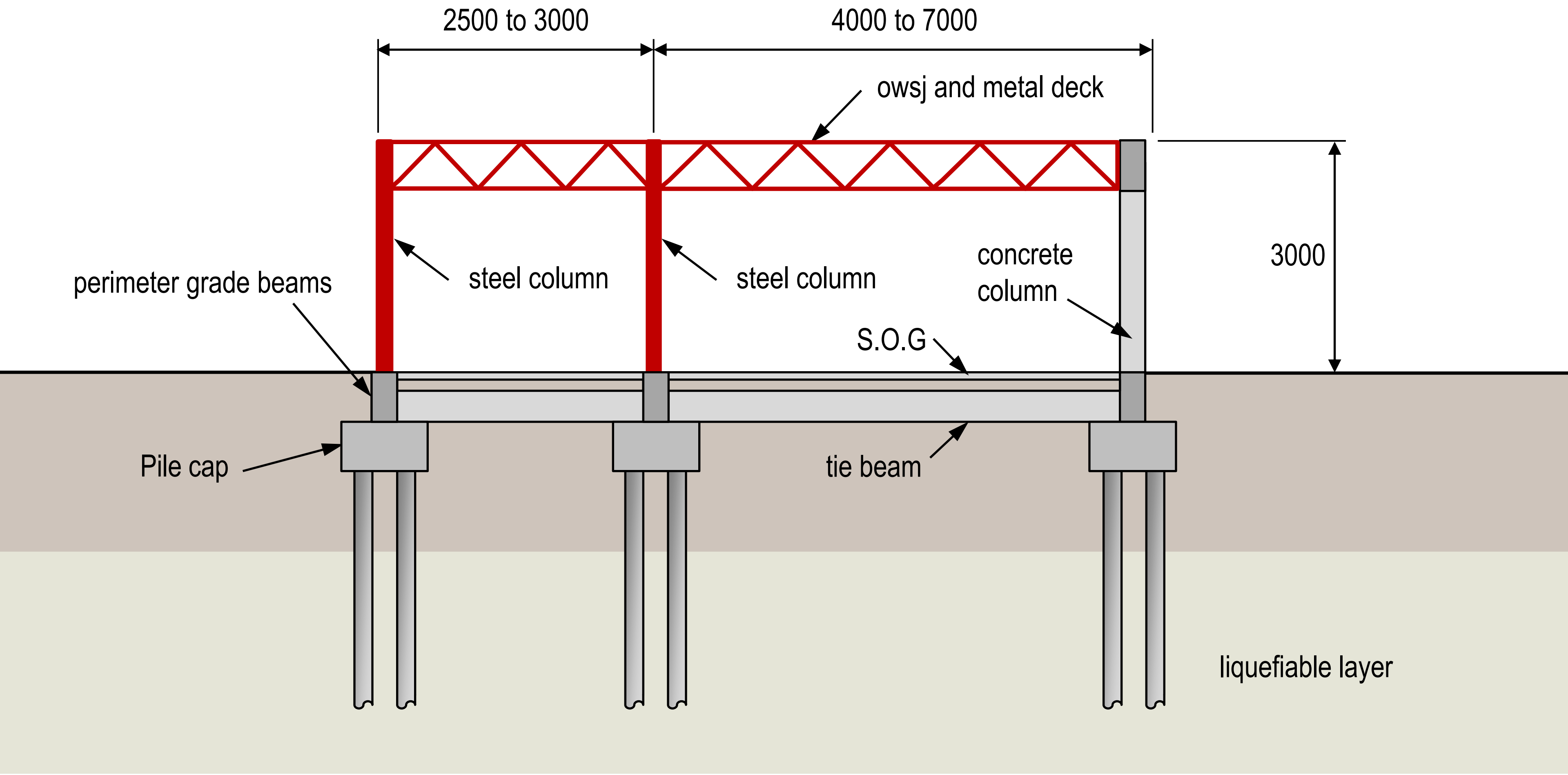
Structural Engineering Guidelines for the Performance-based Seismic Assessment and Retrofit of Low-rise and Mid-rise British Columbia Buildings

W.D. Liam Finn, A. Wightman, John Sherstobitoff and Jason Dowling

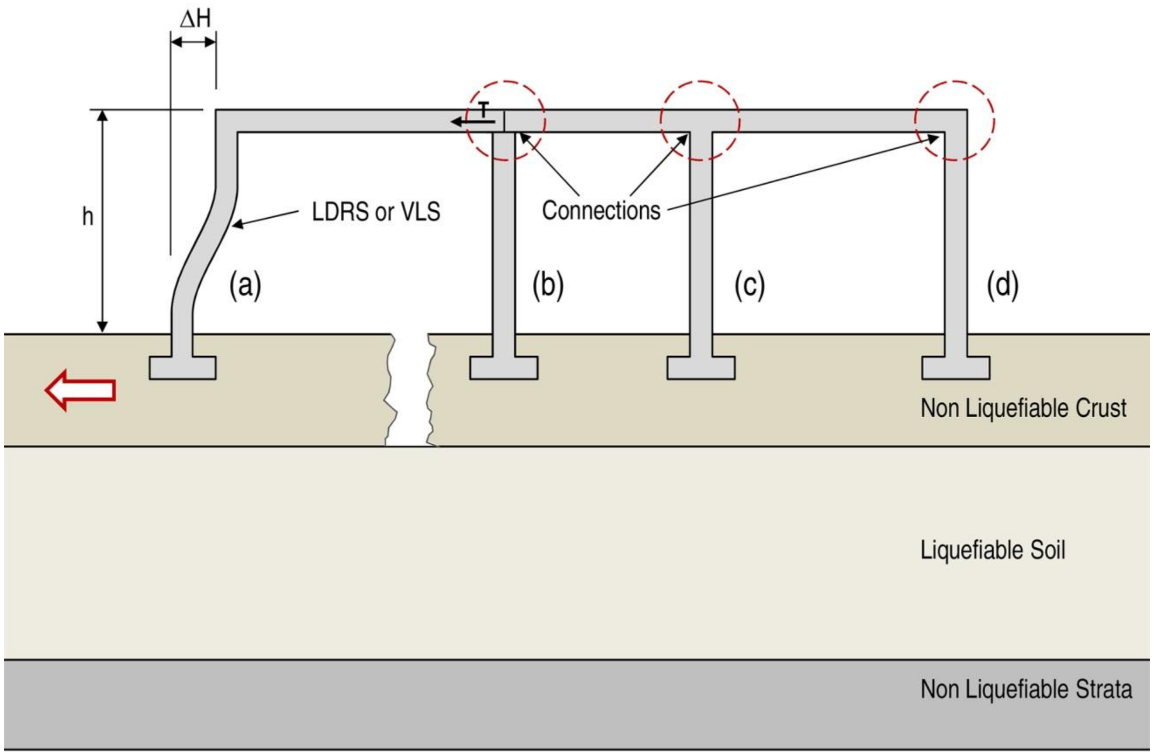
Seismic Retrofit Guidelines 2nd Edition   November, 2013   Manual Volume #11

Liquefaction Design Example with Performance Limits

- Building Description
  - One storey mixed concrete and steel framed structure
  - LDRS Concentric braced frame
    (Tension-Compression moderately ductile)
    - **DDL: 2.5%**
  - VLS Exterior: Non-ductile concrete columns
    - **DDL: 1.25%**
  - VLS Interior/Exterior: Steel columns
    - **DDL: 4%**
  - Liquefaction Drift Limit
    - **LDL: 4% or 2.5% * lesser of the two**
    - *** if liquefaction effects can cause such deformation**


2500 to 3000
4000 to 7000
owsj and metal deck
3000
steel column
steel column
concrete column
perimeter grade beams
S.O.G
Pile cap
tie beam
liquefiable layer

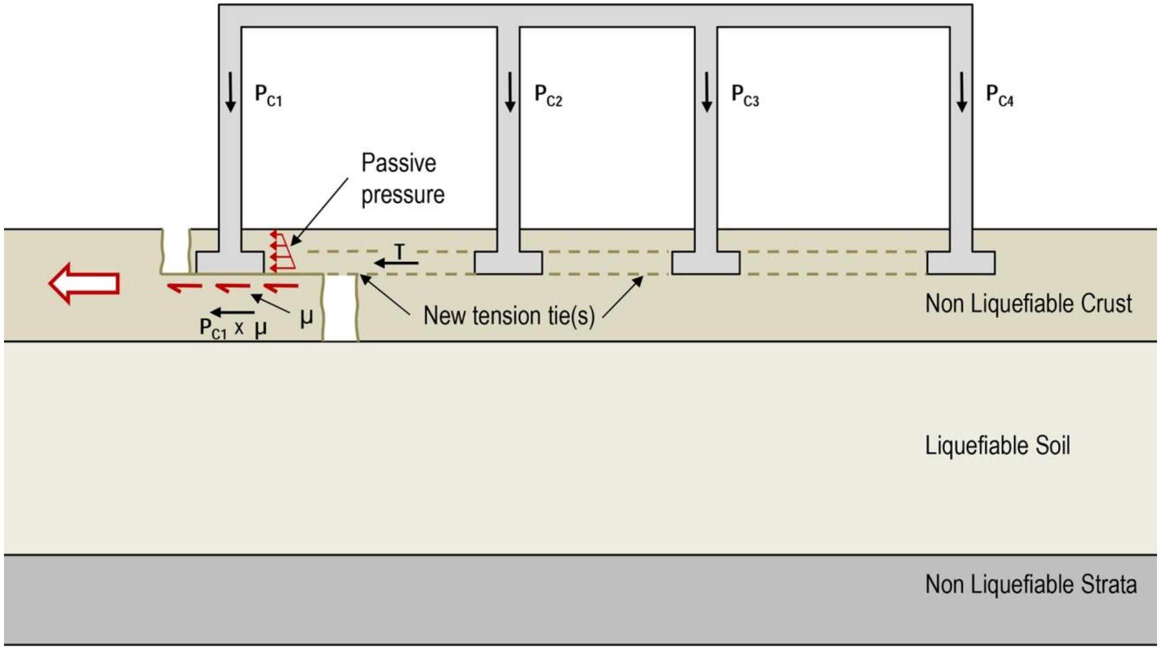
Effects of Lateral Spreading



School building on liquefied soil. Foundations not tied together.

What relative displacement to assign to crack?

Loads from Lateral Spreading on Retrofitted Foundation

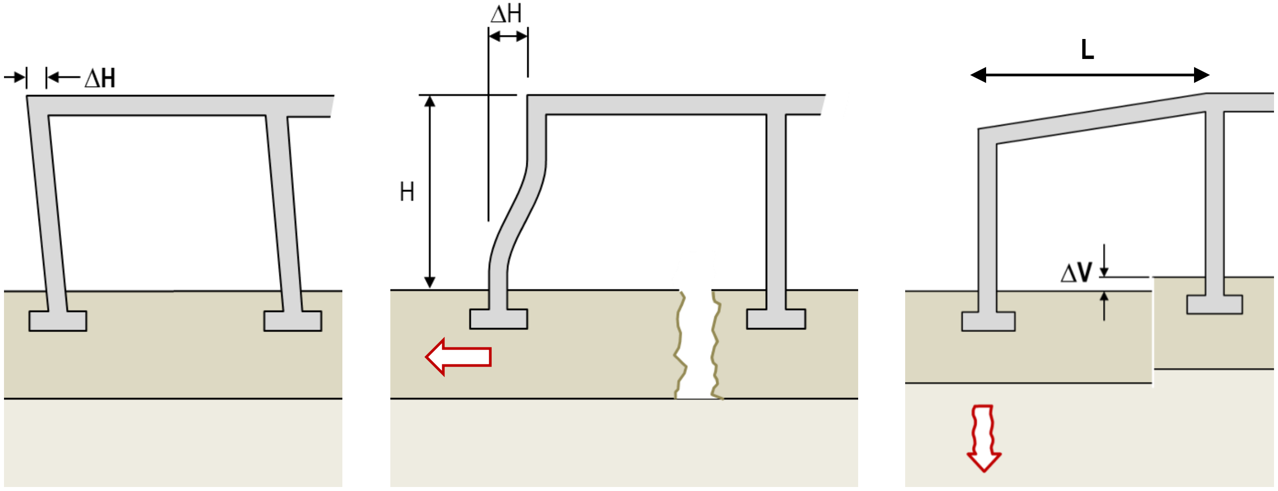


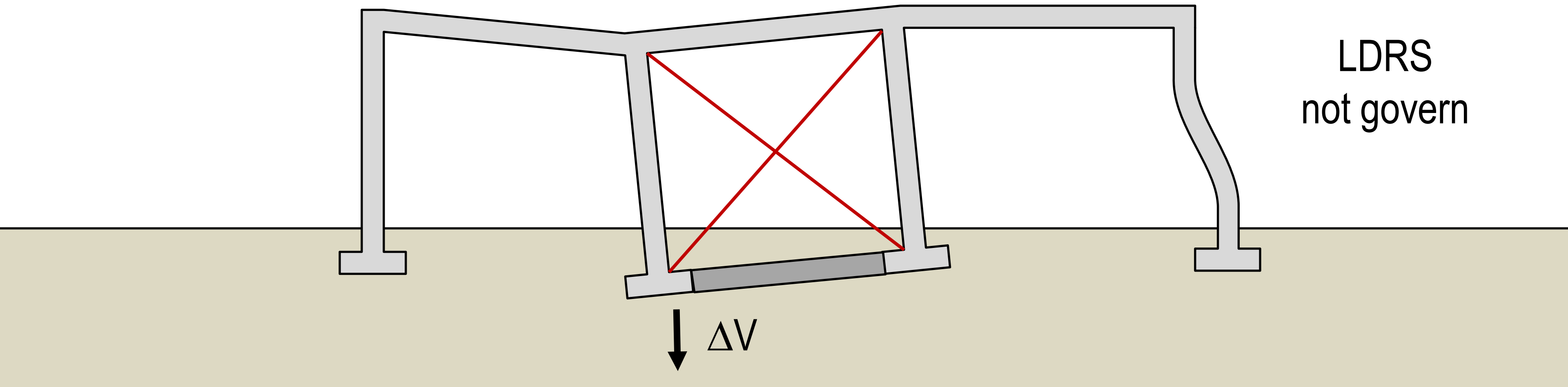
Investigate critical bay for crack location

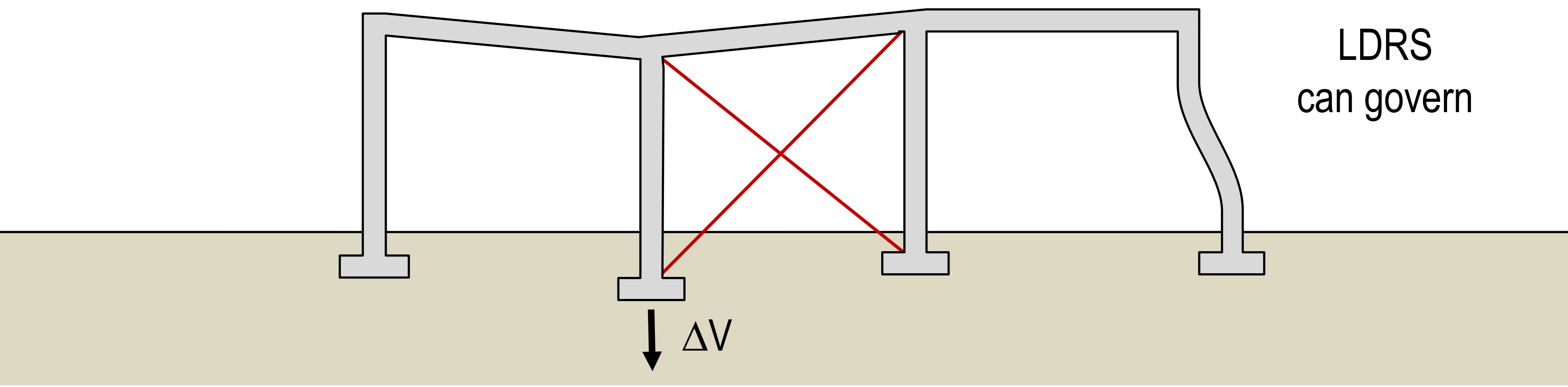
Liquefaction Drift Limit (LDL)

- Drift Components due to liquefaction
  - Residual drift (RD)
  - Effective drift demand due to lateral soil spreading (EDH)
  - Effective drift demand due to vertical soil settlement (EDV)

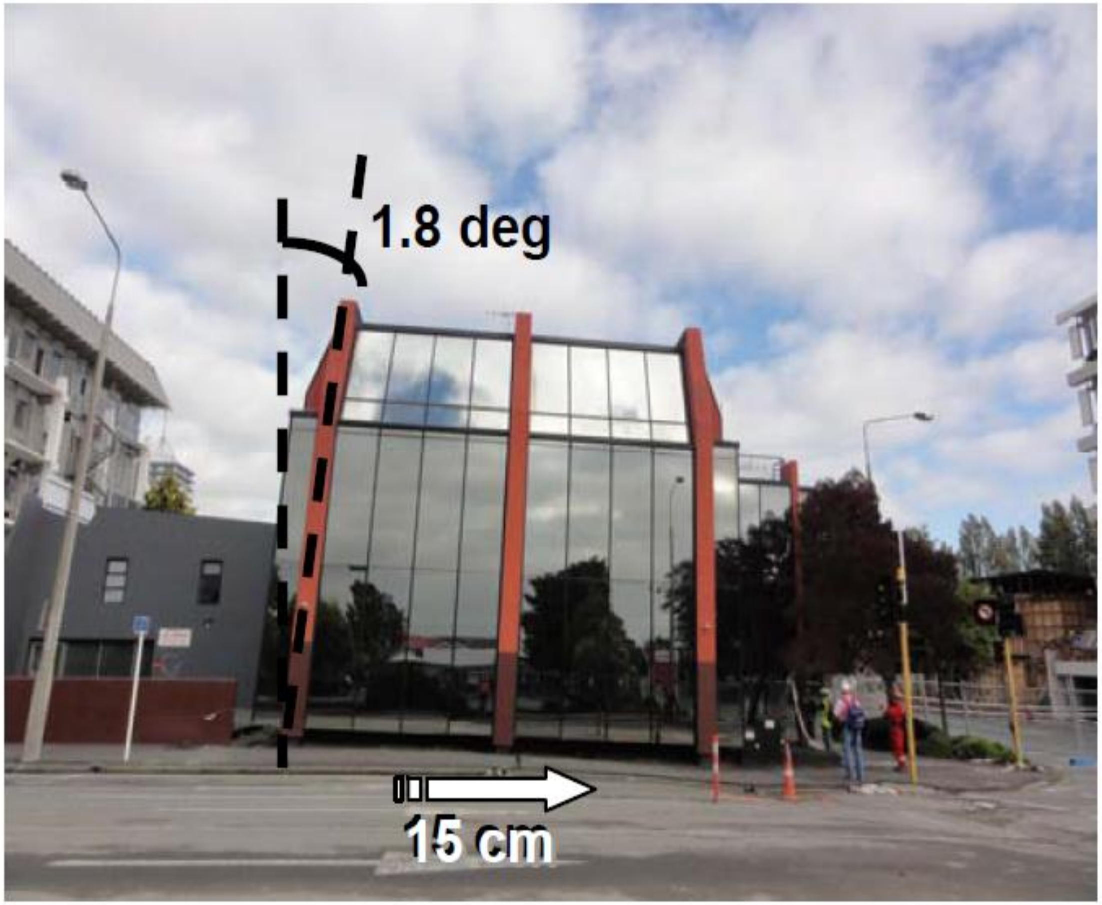
- RD + EDH + EDV < LDL

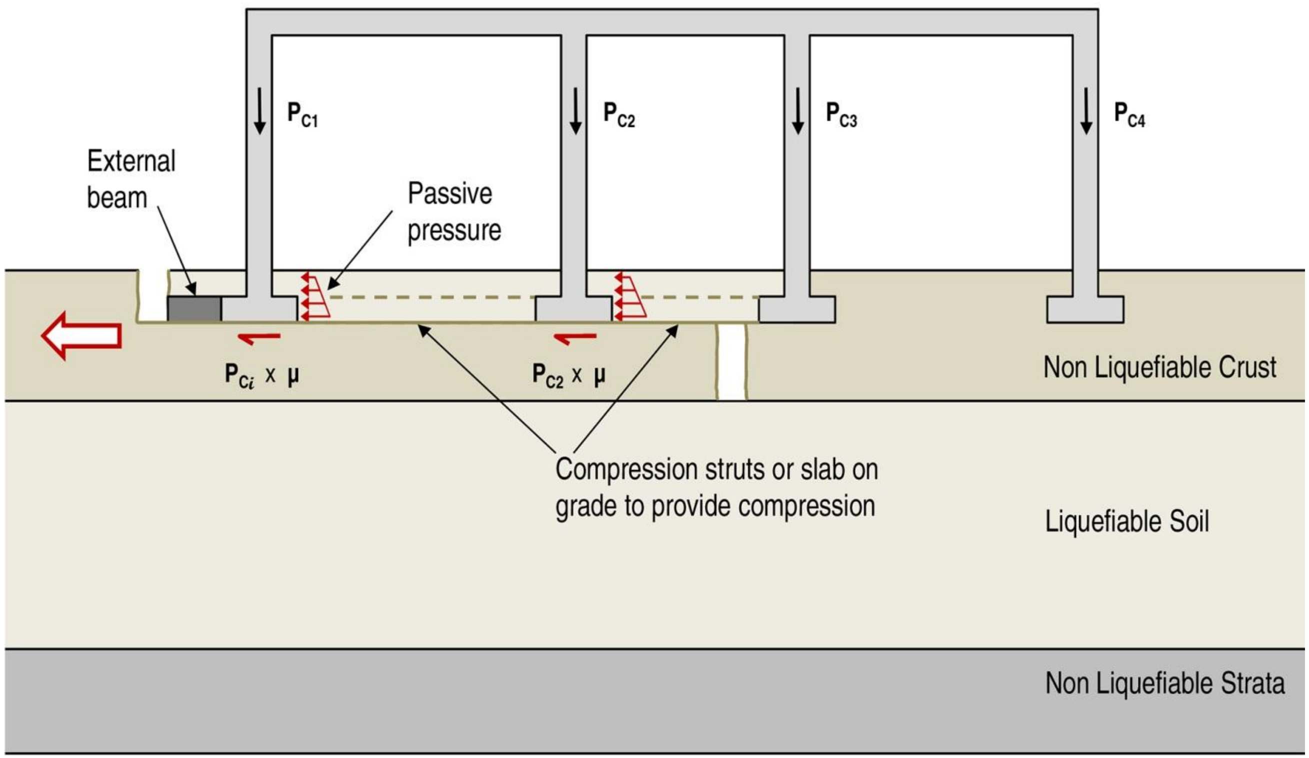



LDRS
not govern
ΔV

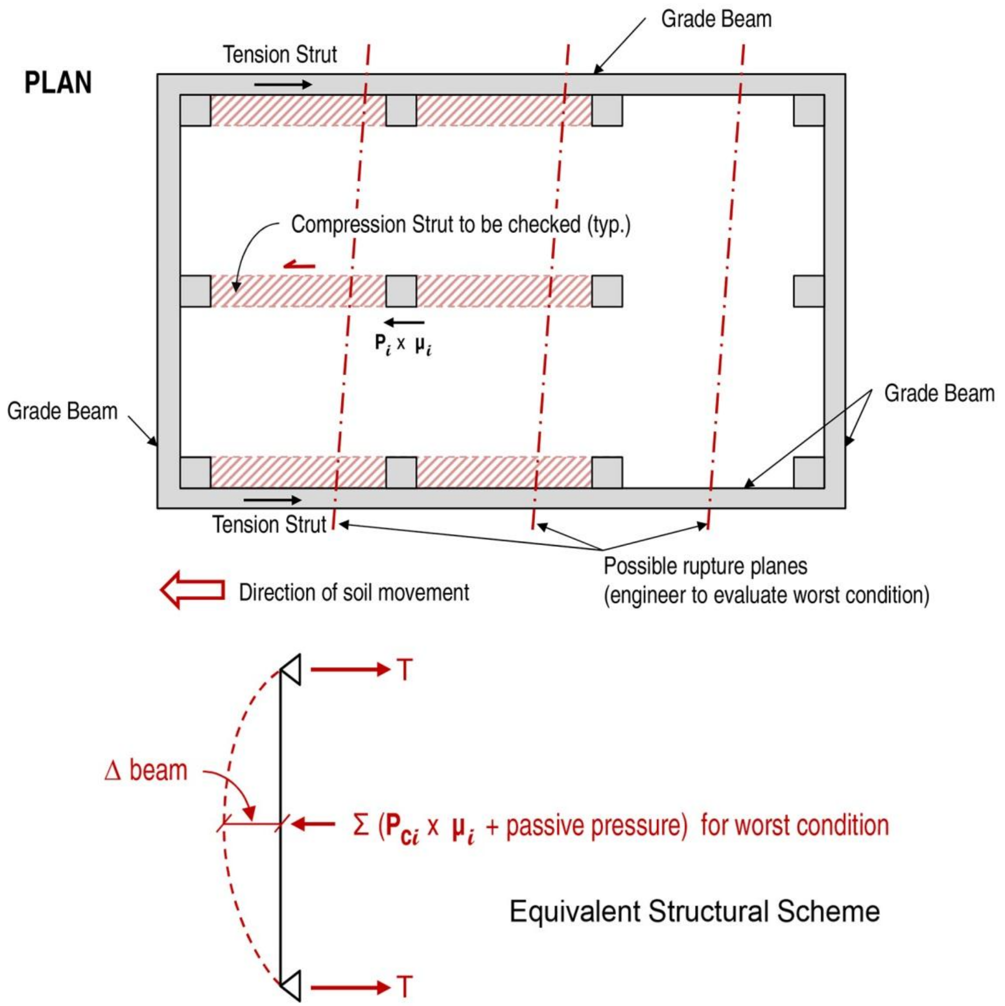

LDRS
can govern
ΔV

Example from Christchurch





P_{C1}
P_{C2}
P_{C3}
P_{C4}
External beam
Passive pressure
P_{Ci} x μ
P_{C2} x μ
Non Liquefiable Crust
Compression struts or slab on grade to provide compression
Liquefiable Soil
Non Liquefiable Strata



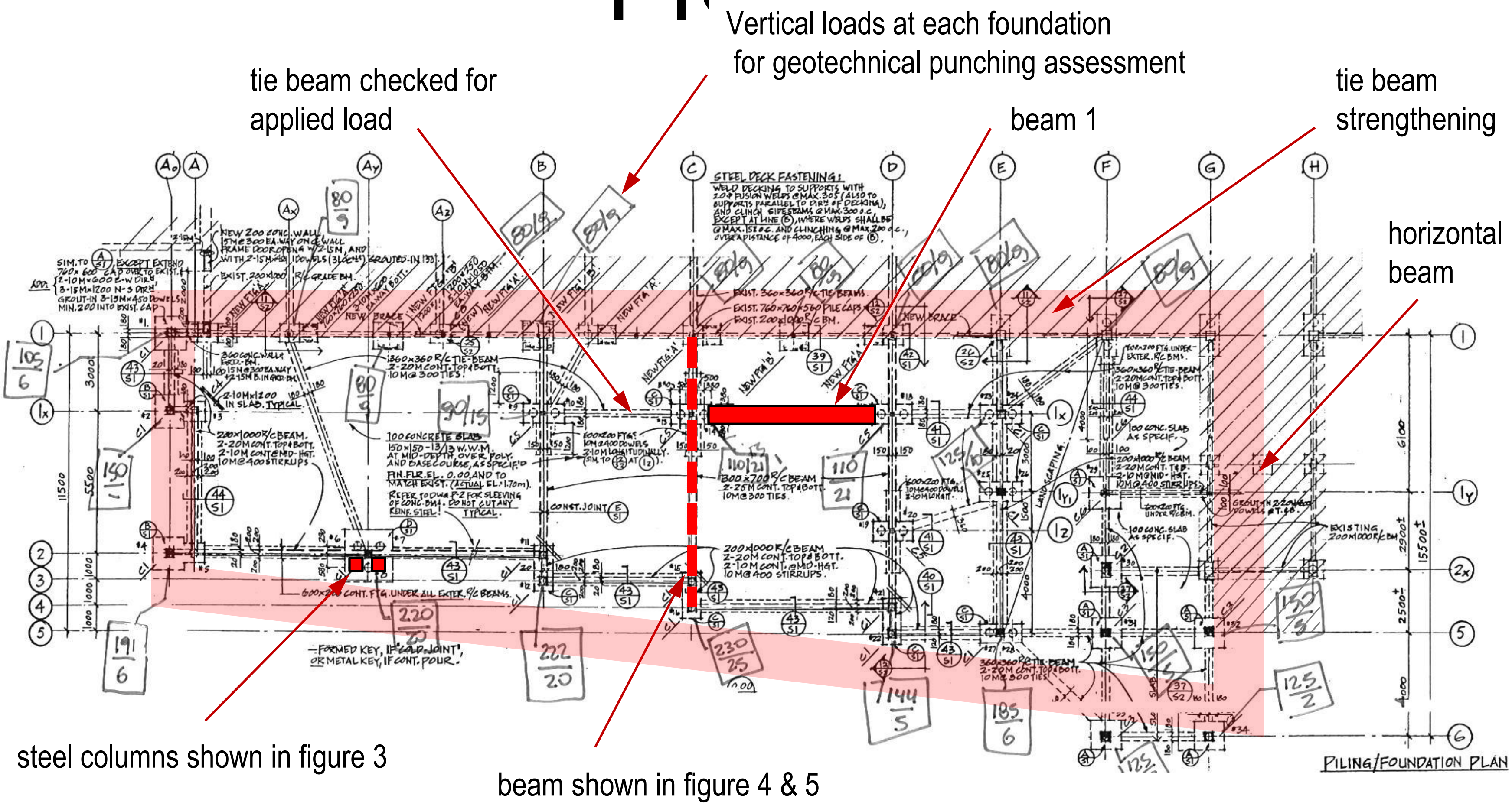
PLAN
Tension Strut
Grade Beam
Compression Strut to be checked (typ.)
P_i x μ_i
Grade Beam
Grade Beam
Tension Strut
Possible rupture planes
(engineer to evaluate worst condition)
Direction of soil movement
T
Δ beam
Σ (P_ci x μ_i + passive pressure) for worst condition
Equivalent Structural Scheme
T

Geotechnical Engineer Input

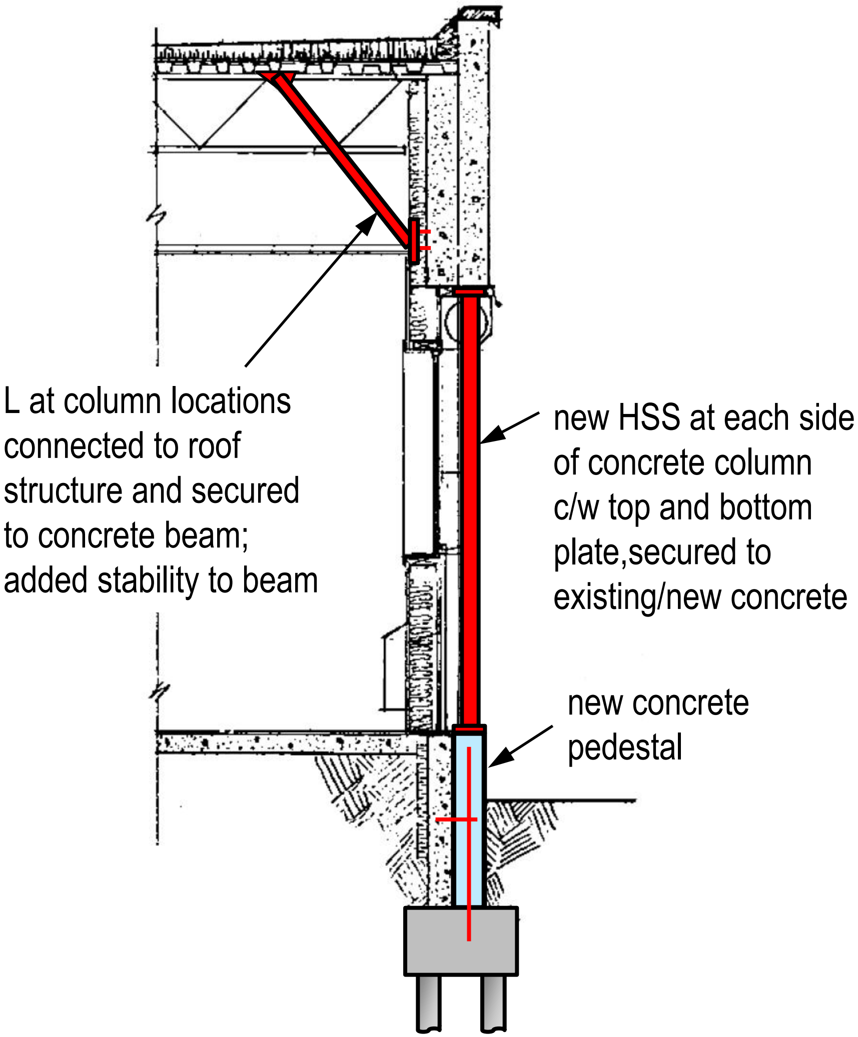
- Geotechnical Engineer to provide:
  - Differential free field vertical movement: **140 mm**
  - Differential free field horizontal movement: **600 mm**
  - Friction coefficient between soil and foundations: **0.4**
  - Bearing capacity on crust: **50 Kpa**
  - Passive pressure on grade beams: **10xH (m) : P (Kpa)**

- Based on vertical loads at each foundation provided by the Structural Engineer, the Geotechnical Engineer will determine if there is risk of punching for each foundation
  - Differential movement is too large, since the existing foundations are not adequately tied together in two directions

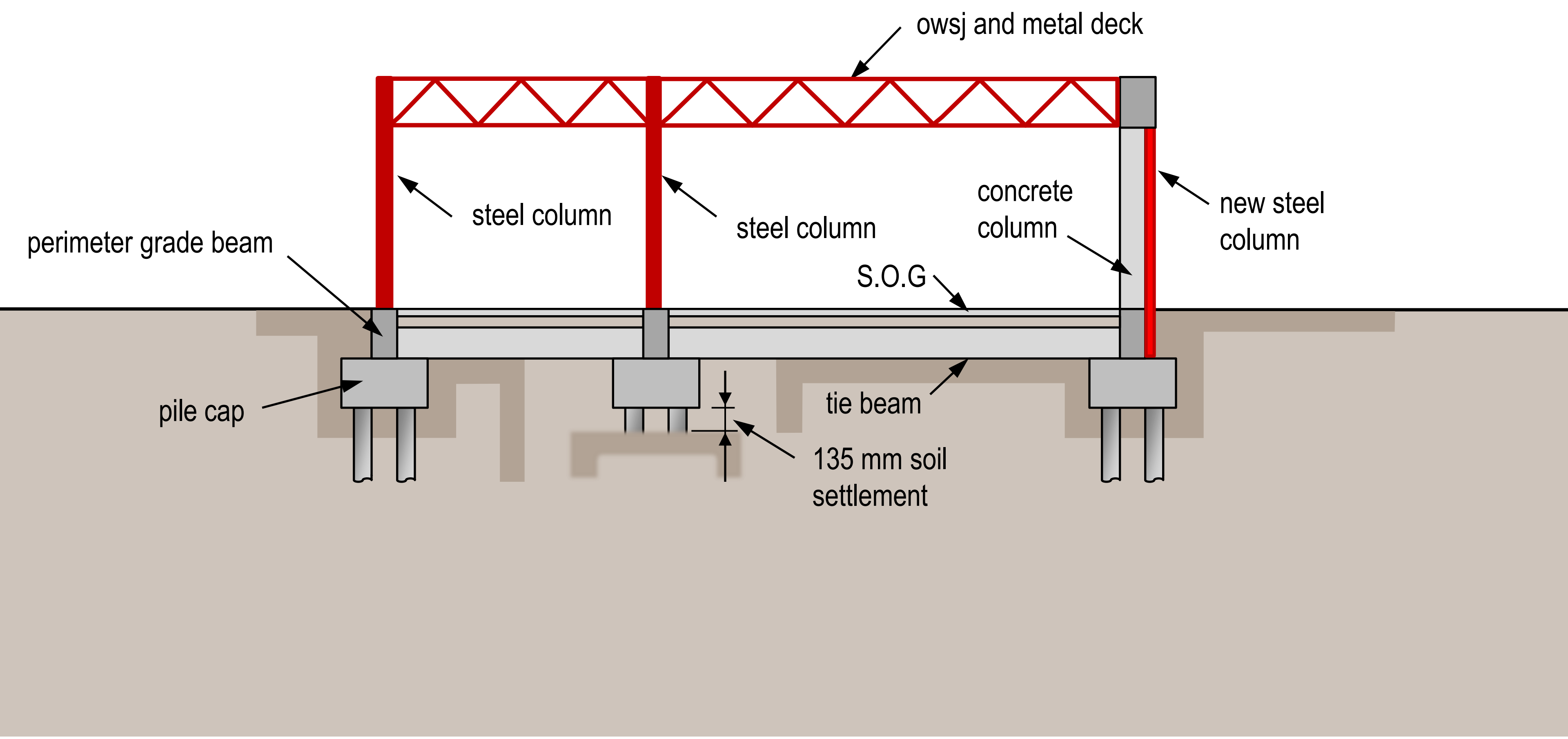
FIG 1


Vertical loads at each foundation for geotechnical punching assessment
tie beam checked for applied load
beam 1
tie beam strengthening
horizontal beam
steel columns shown in figure 3
beam shown in figure 4 & 5
PILING/FOUNDATION PLAN

Retrofit Details

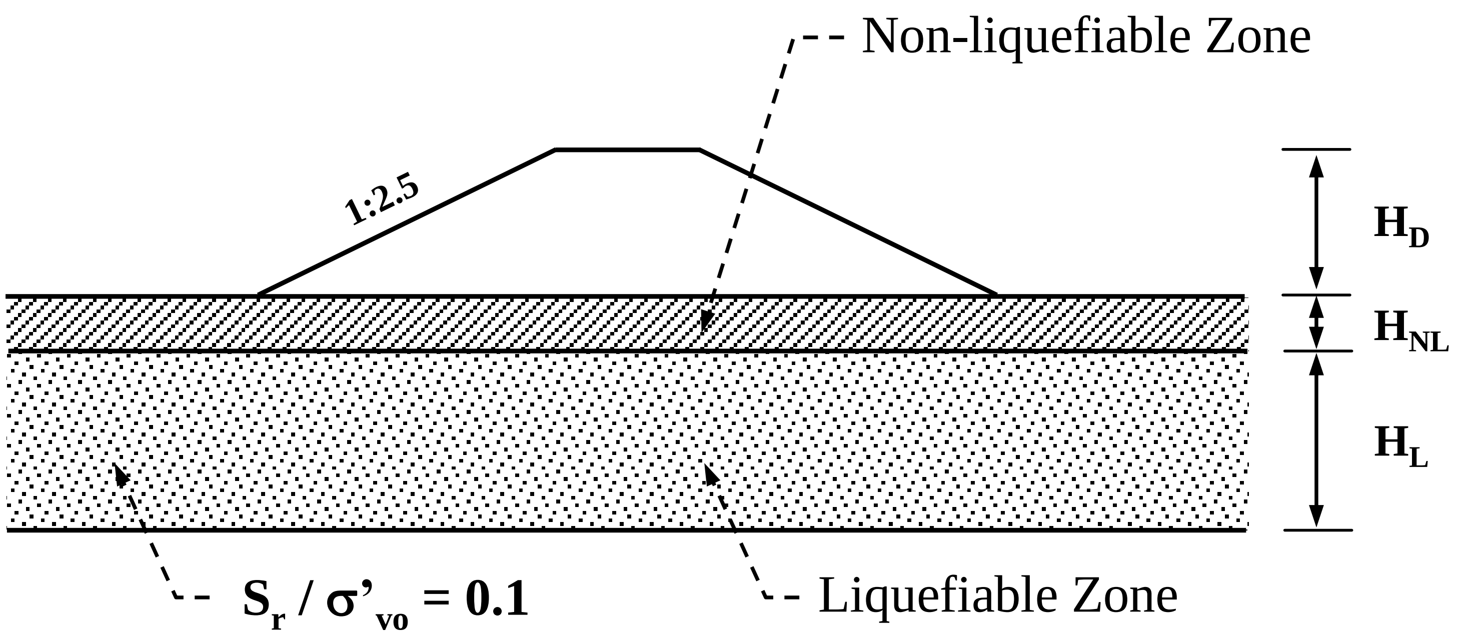


- Total drift due to liquefaction: 3.86 %
- Decided to add two exterior steel columns at each side of the existing concrete columns to minimize the impact to the inside of the building, allowing to increase the DDL of the VLS to 4%, the new steel columns have to be designed to carry all the loads carried by the existing concrete columns.


owsj and metal deck
steel column
steel column
concrete column
new steel column
perimeter grade beam
S.O.G
pile cap
tie beam
135 mm soil settlement

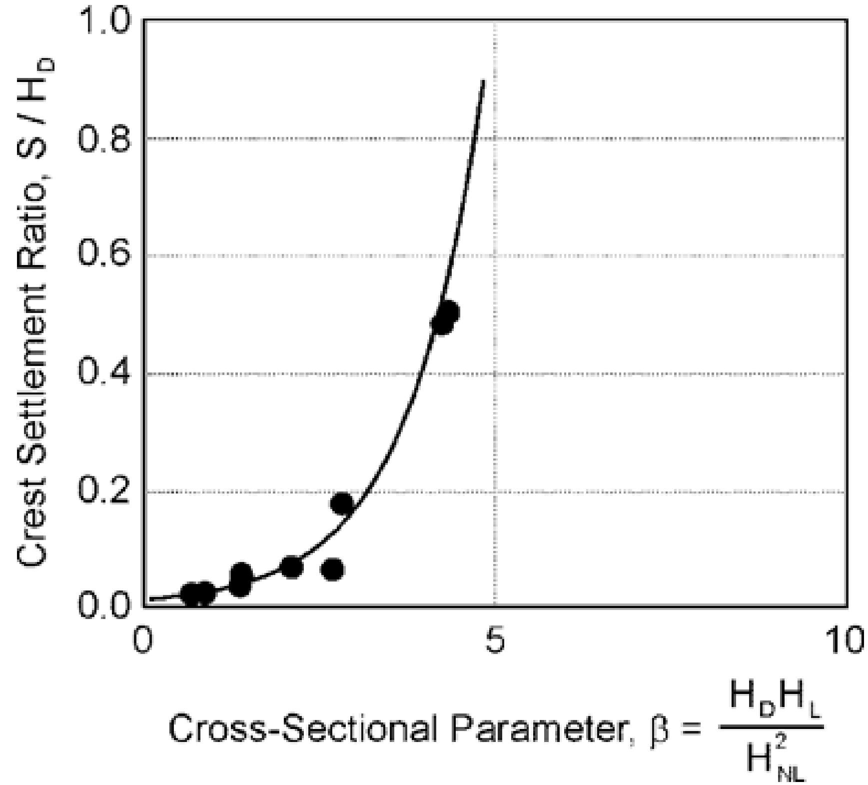
Stability of Japanese dykes

- **Analyses of dyke failures during Kushiro Earthquake.**
- **Soil Properties, input motions and failure data provided by Japanese.**
- **Analyses conducted in Vancouver at UBC.**
- ***No interactions during the analyses.***



Typical cross-section of Kushiro dike used in parametric studies

$$S = 0.01 \exp\left(0.922 \frac{H_D}{H_{NL}} \frac{H_L}{H_{NL}}\right)$$



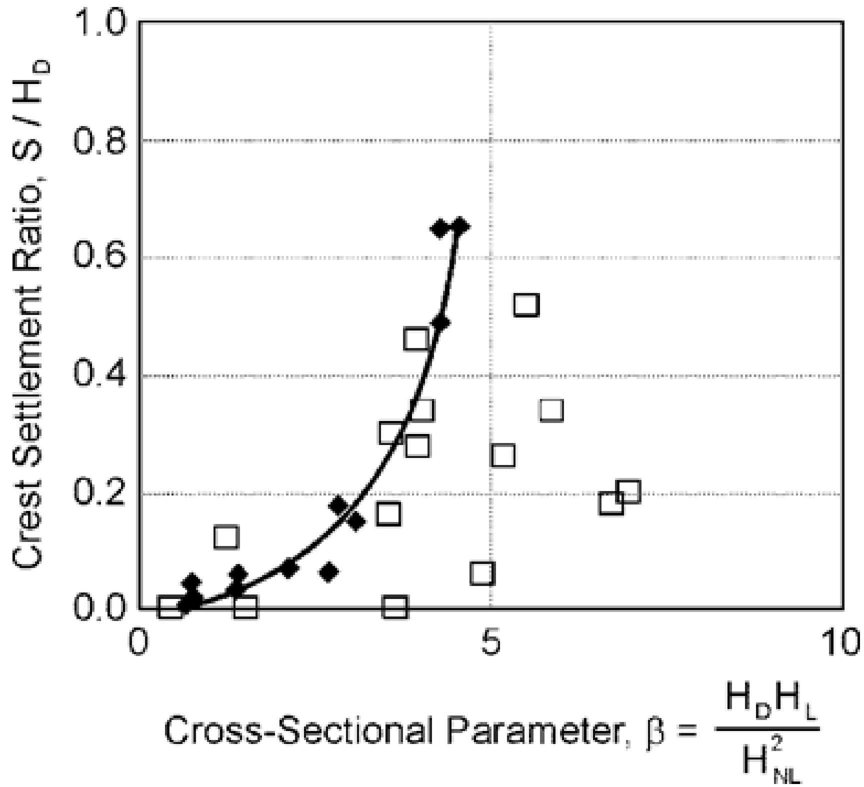
Comparison of observed settlements with the black prediction curve

Eastern Hokkaido Dykes

Western Hokkaido dykes

Subsequent to the Kushiro quake, an earthquake occurred off western Japan, which damaged many Western Dykes. I was invited to Sapporo to discuss the failures and how to prioritize remediation measures.

At the meeting the Japanese presented the results of applying my S-equation to the new set of failures. Fortunately the equation worked extremely well as shown in the next slide.



Comparison of observed settlements for all slopes against computed settlements for 1:2.5 slopes (solid curve).

Points not close to the curve are for slopes other than 1:2.5

Western Hokkaido Dykes

Slopes and Embankment Dams

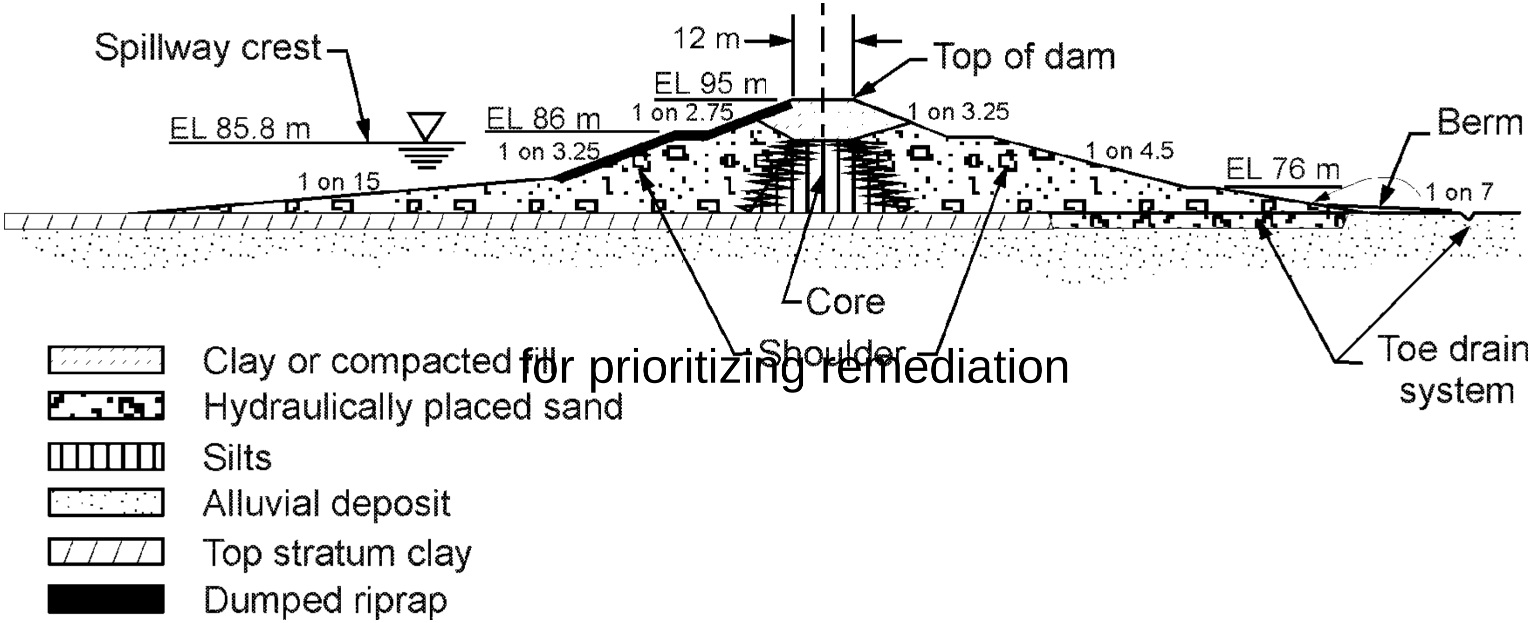
All examples had liquefaction problems and most pressing problem was the residual strength.

Empirical correlations for residual strength have been presented by Harder and Seed (1990) and Idriss and Boulanger (2008).

Sardis Dam Mississippi, 1988- 1994

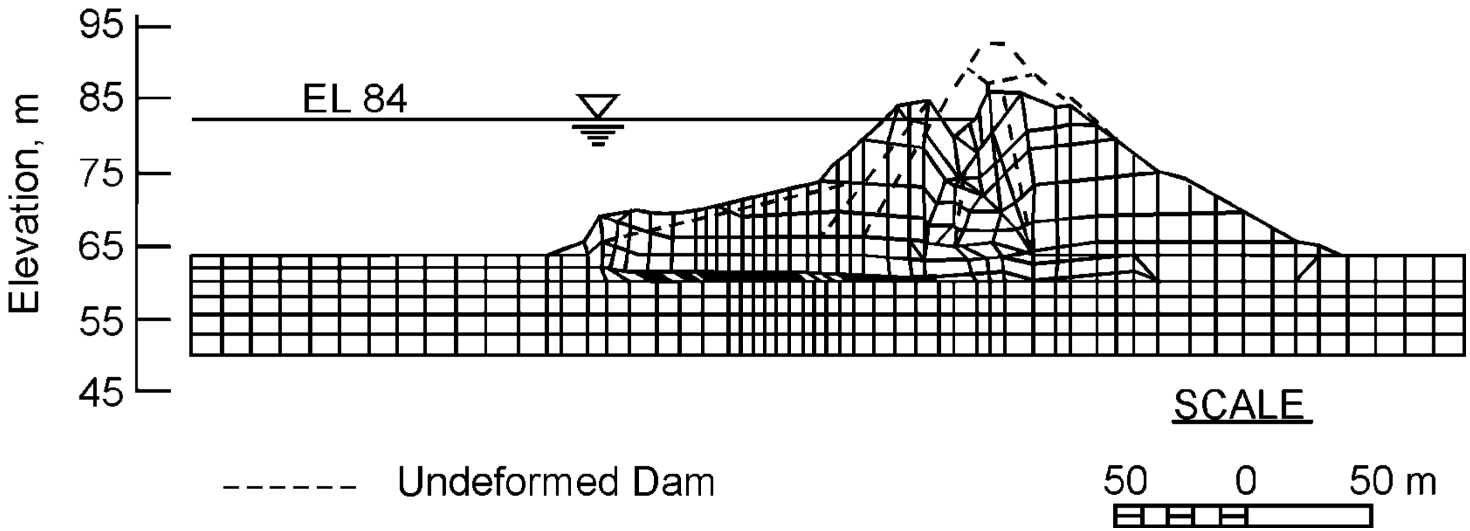
This is the first instance of performance based design of an embankment dam

First Example of Performance Based Design

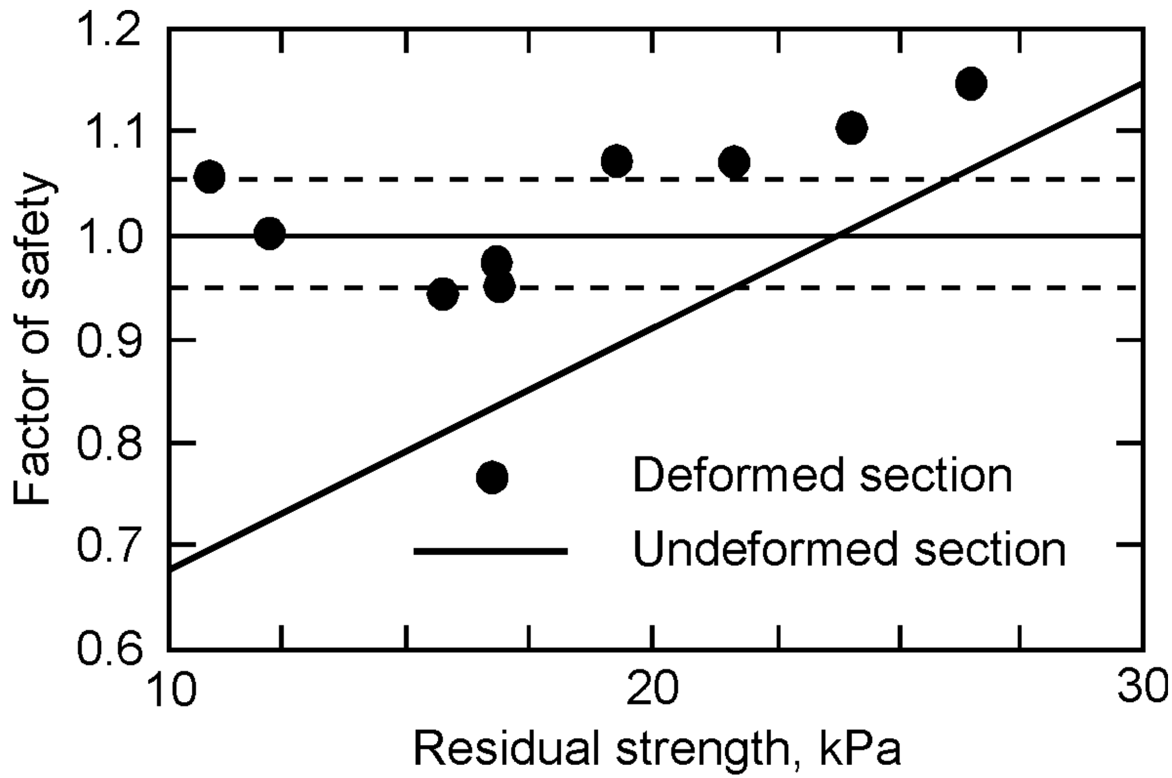


for prioritizing remediation

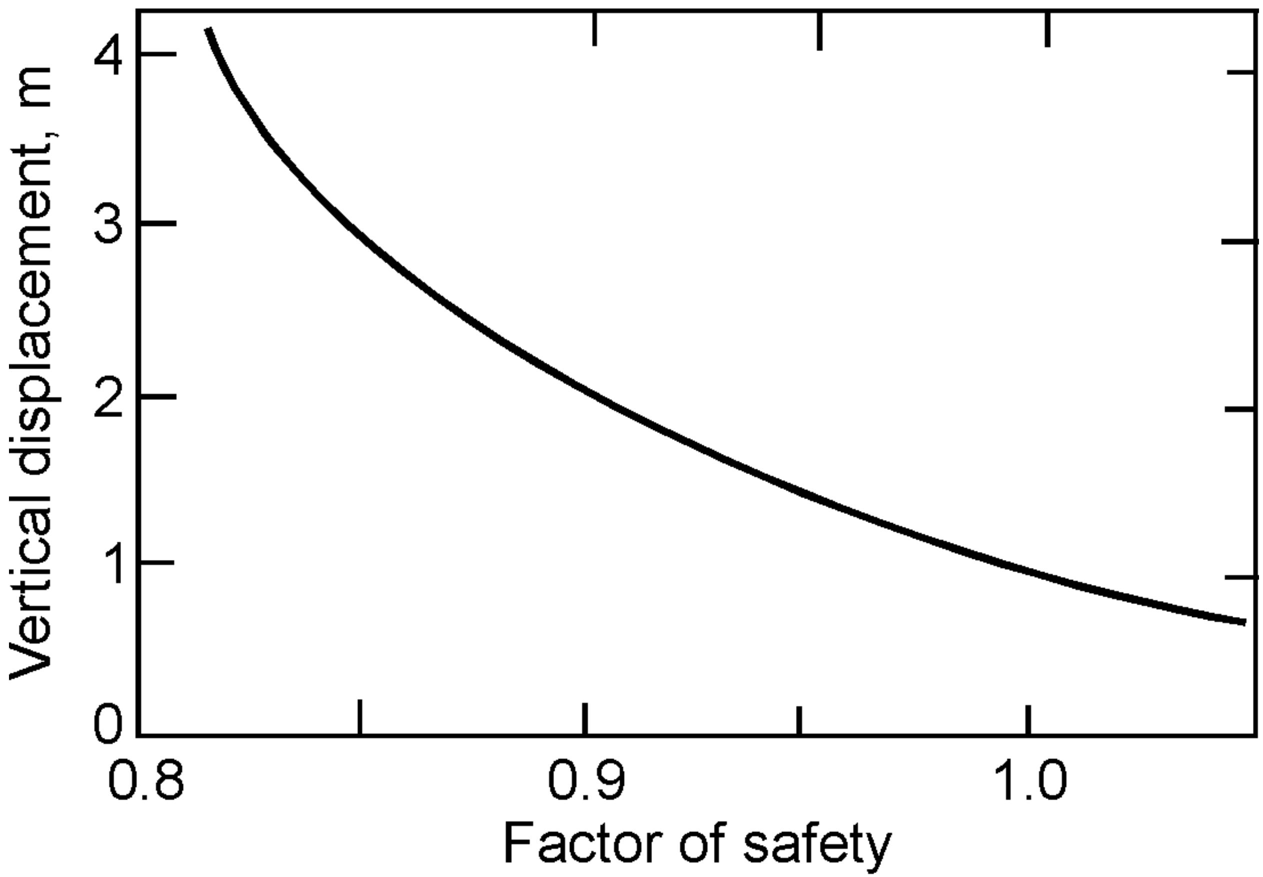
Cross-section of Sardis Dam



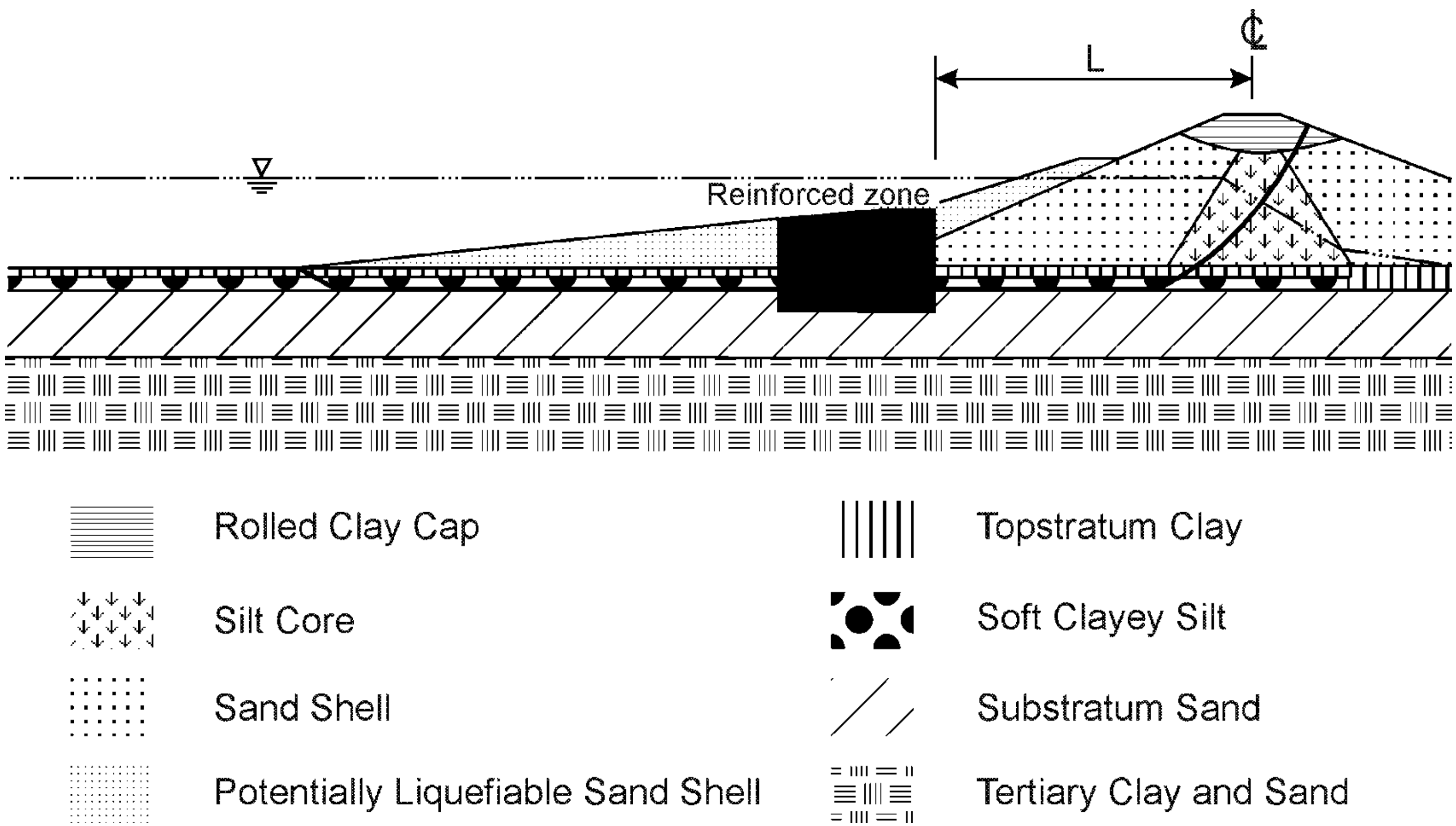
Post-liquefaction deformed shape of Sardis Dam: note different vertical and horizontal scales.



Factors of safety of Sardis dam as a function of residual strength in weak foundation layer



Variation of loss of freeboard with factor of safety of undeformed dam.



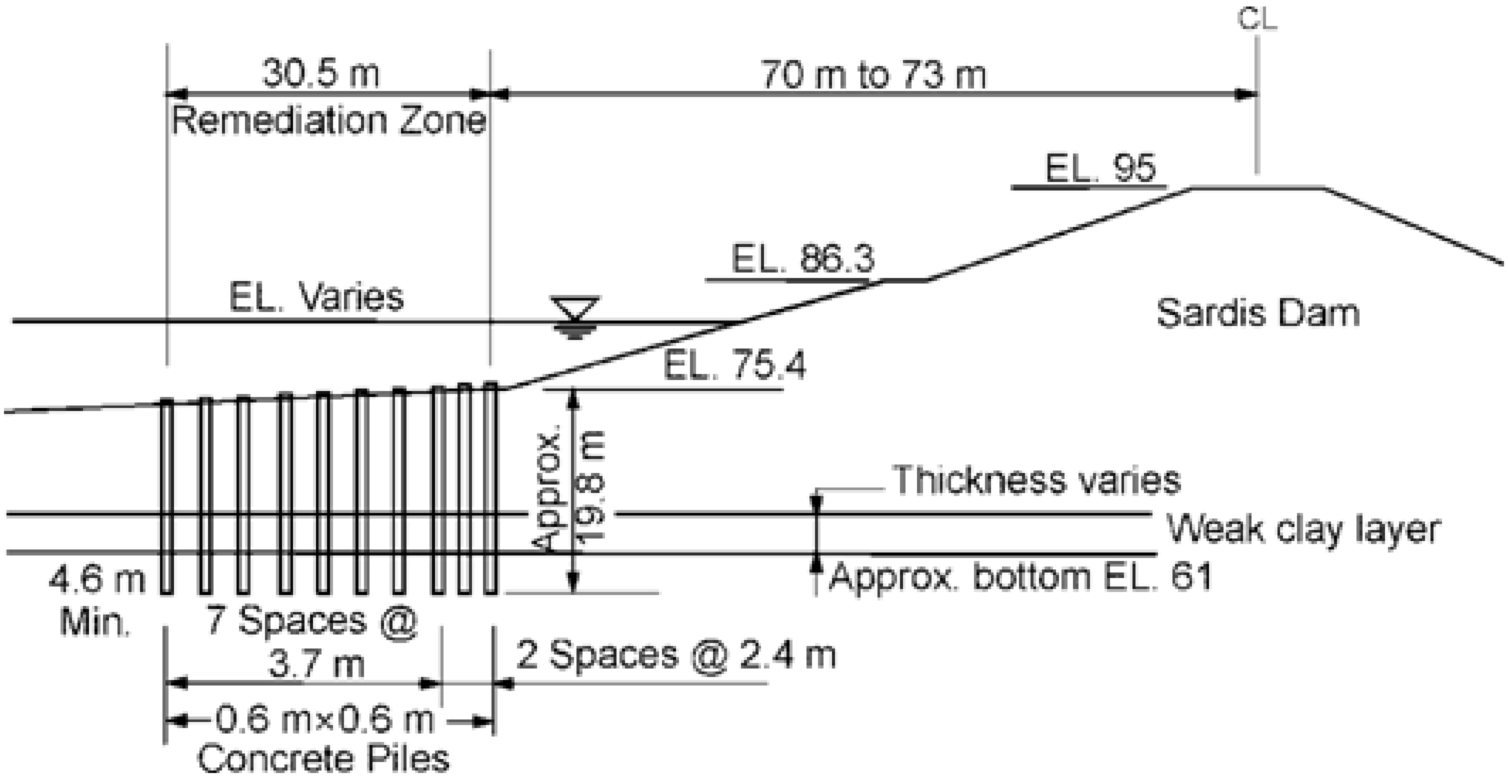
Location of remediation plug to stabilize upstream slope of Sardis dam

ALTERNATIVE REMEDIAL METHODS - SARDIS DAM

- Vibro-Grouted Stone Columns
- Deep Soil Mixing
- Cast-in-Place Piles
- Driven Piles

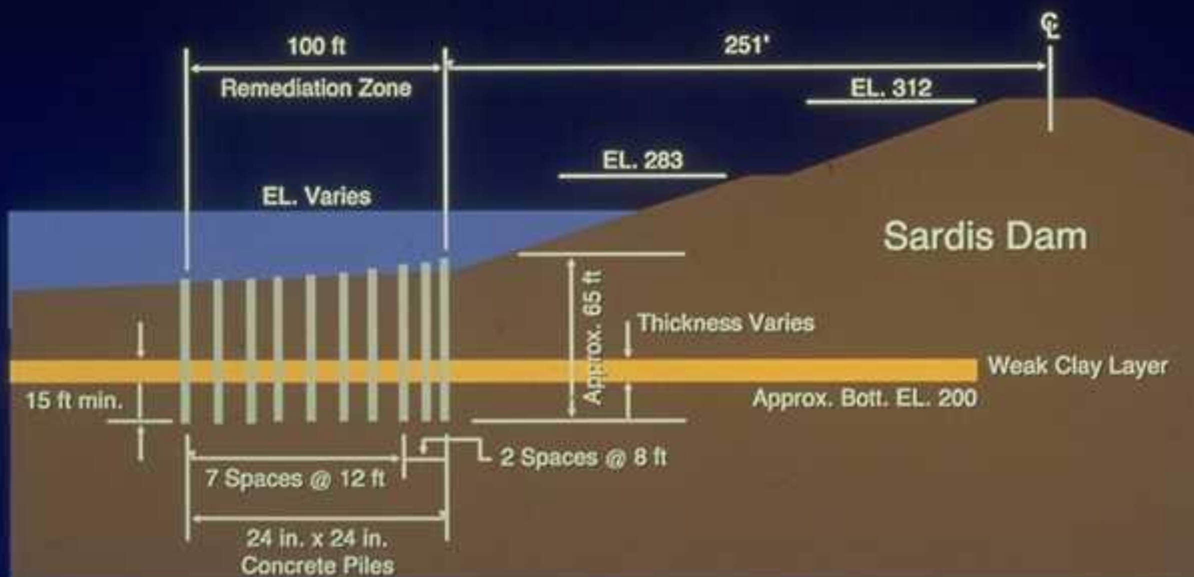
REMEDIAL MEASURES

- Engineering
- Cost
- Reliability
- Availability of Experienced Contractors
- Level of the Reservoir

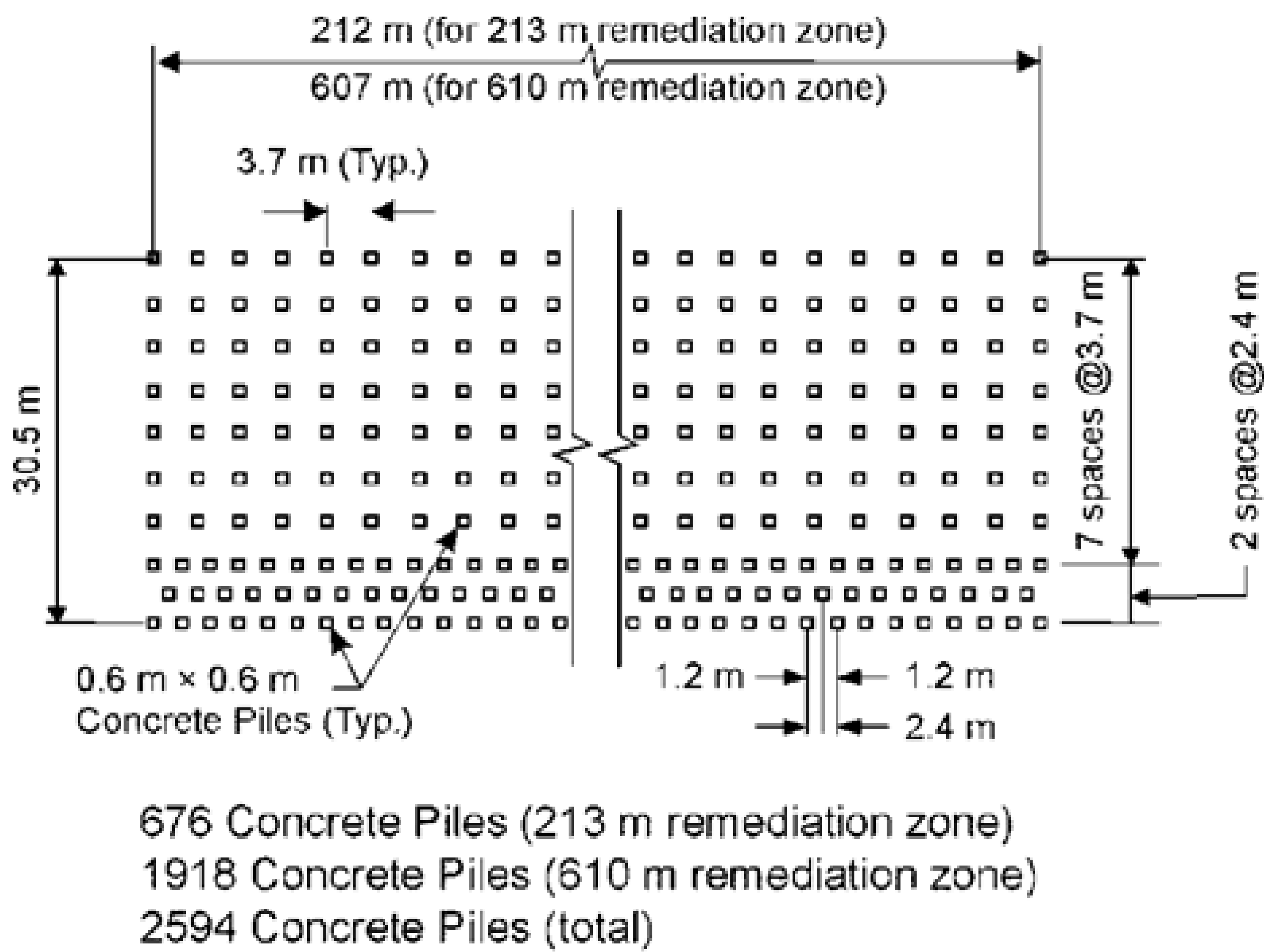


Elevation of pile remediation of Sardis dam (after Stacy et al., 1994)

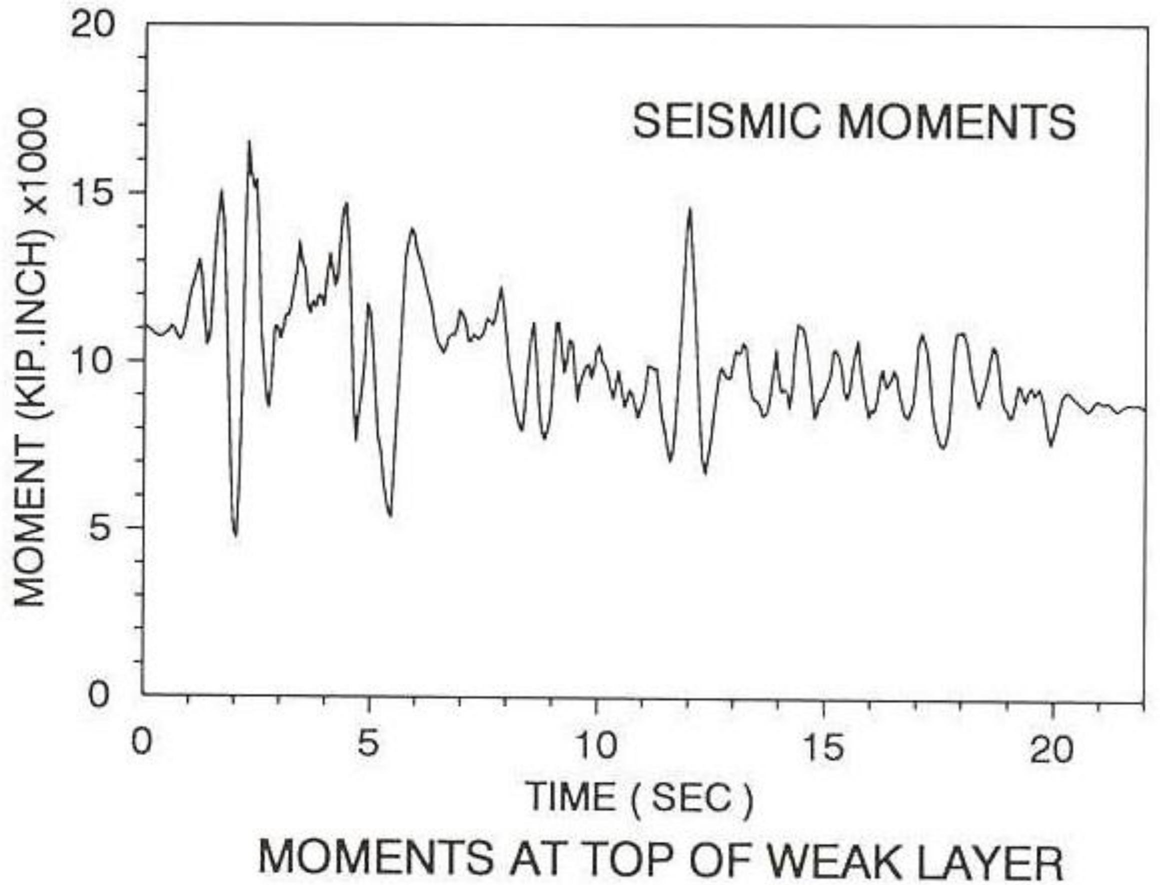
Cross-Section Of The Pile Reinforced Section Of Sardis Dam



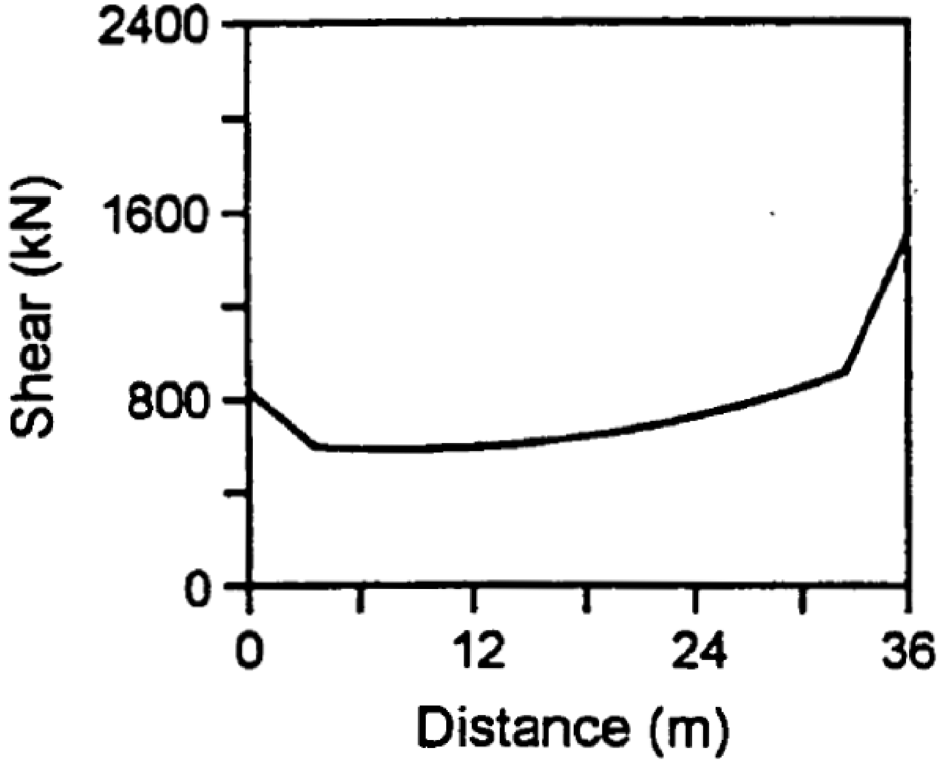
94MAR227



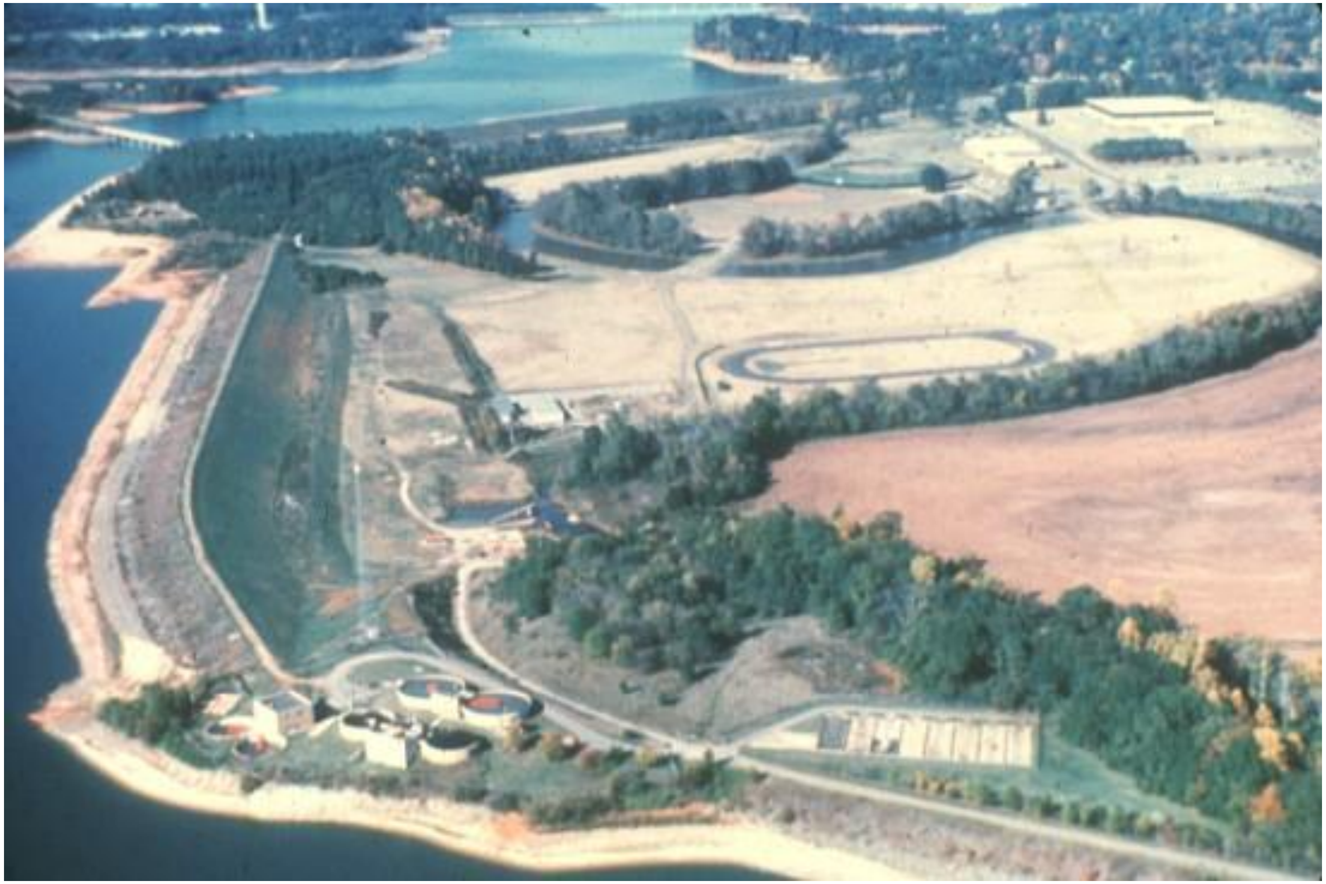
Plan view of pile remediation of Sardis dam (after Stacy et al., 1994)



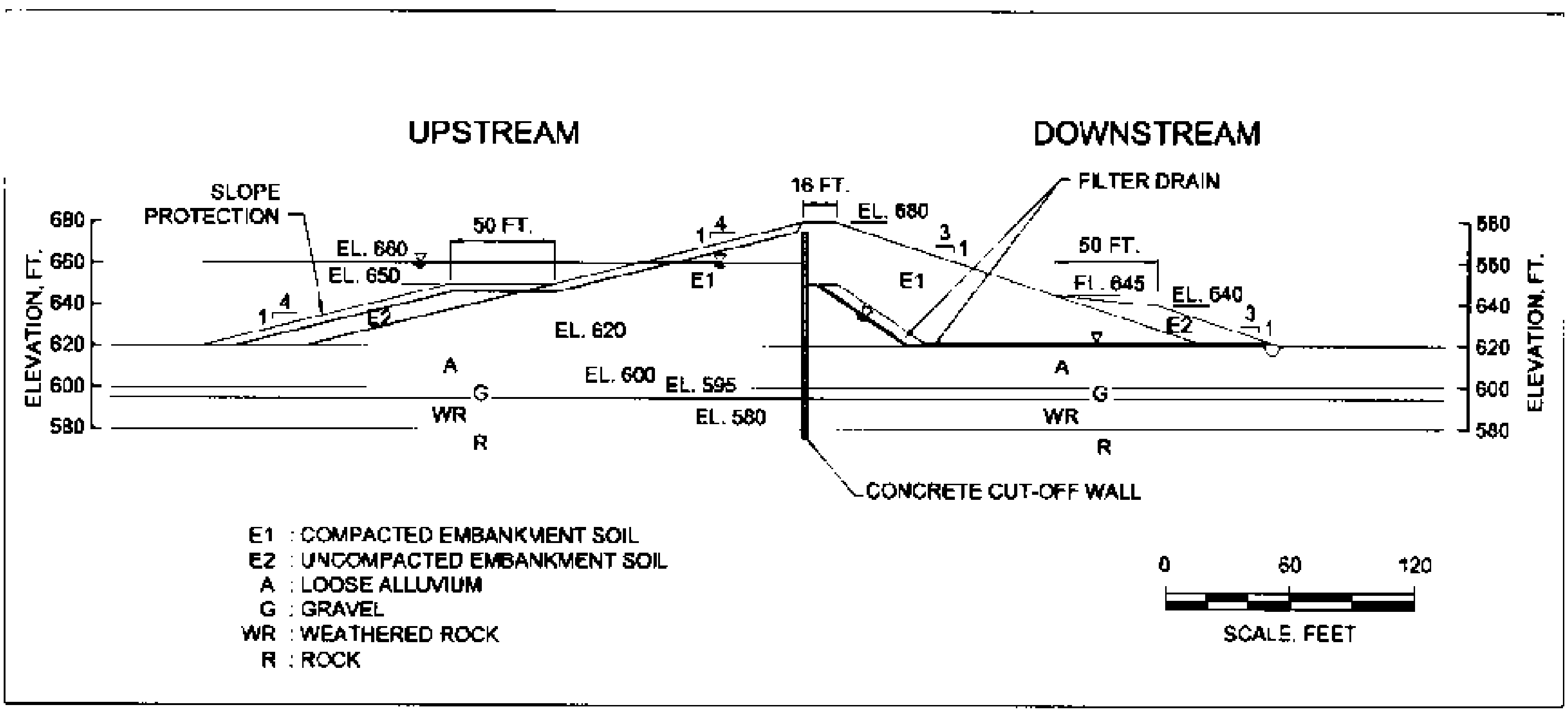
Dynamic bending moments in the leading upstream pile



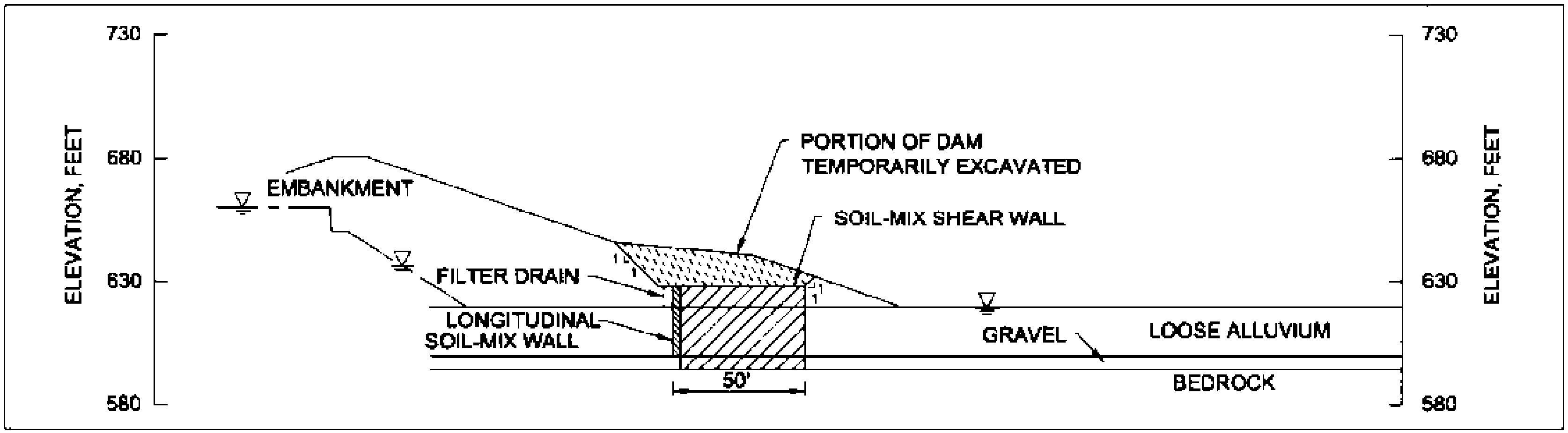
Distribution of pile shears between pile rows



Aerial View of Clemson Diversion Dams (Wooten et al, 2008)



Simplified cross-section Clemson Dams

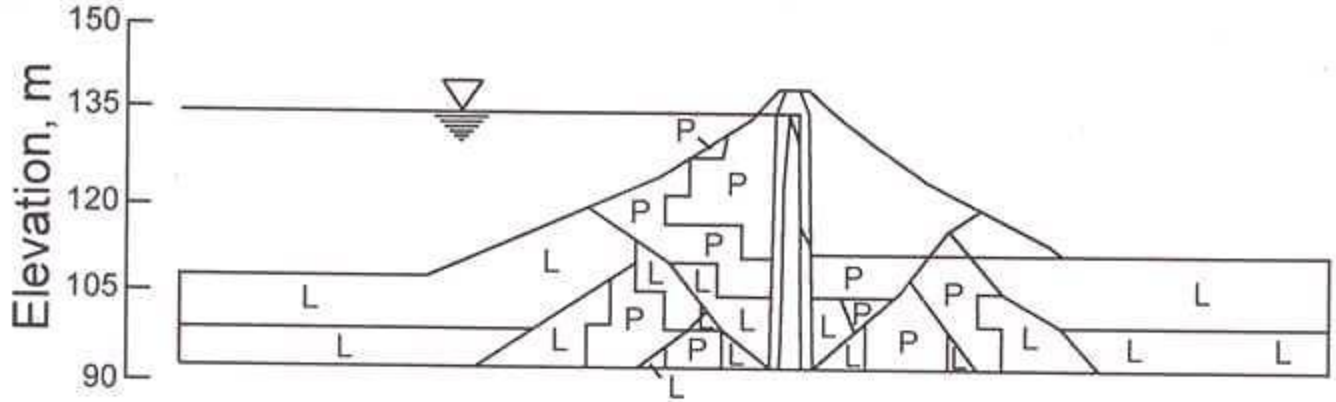


Cross Section of Remediated Downstream Slope (Wooten et al 2008)

Mormon Island auxiliary dam

Two performance criteria used:

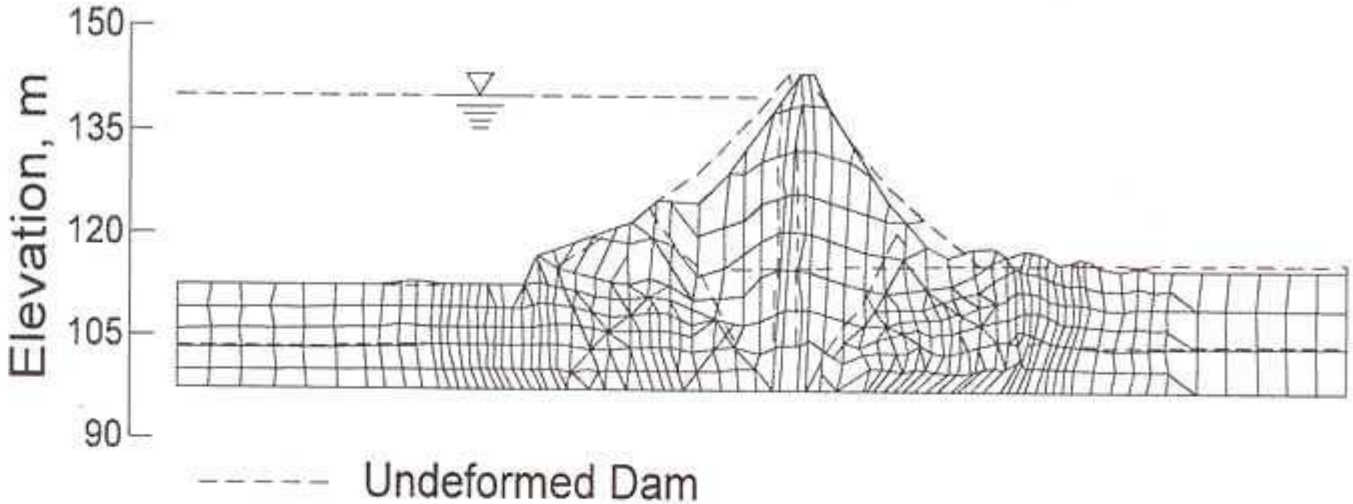
1. **Displacement criterion**
2. **Pore pressure level criterion of 20%**



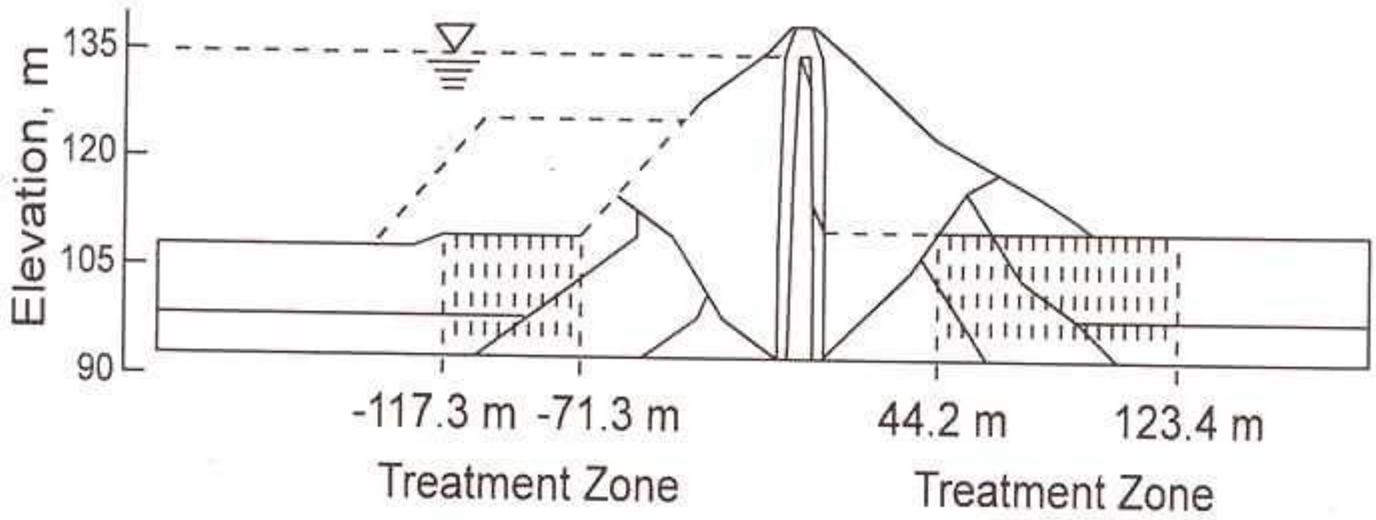
L = Liquefiable

P = Pore water pressure increase due to earthquake excitation

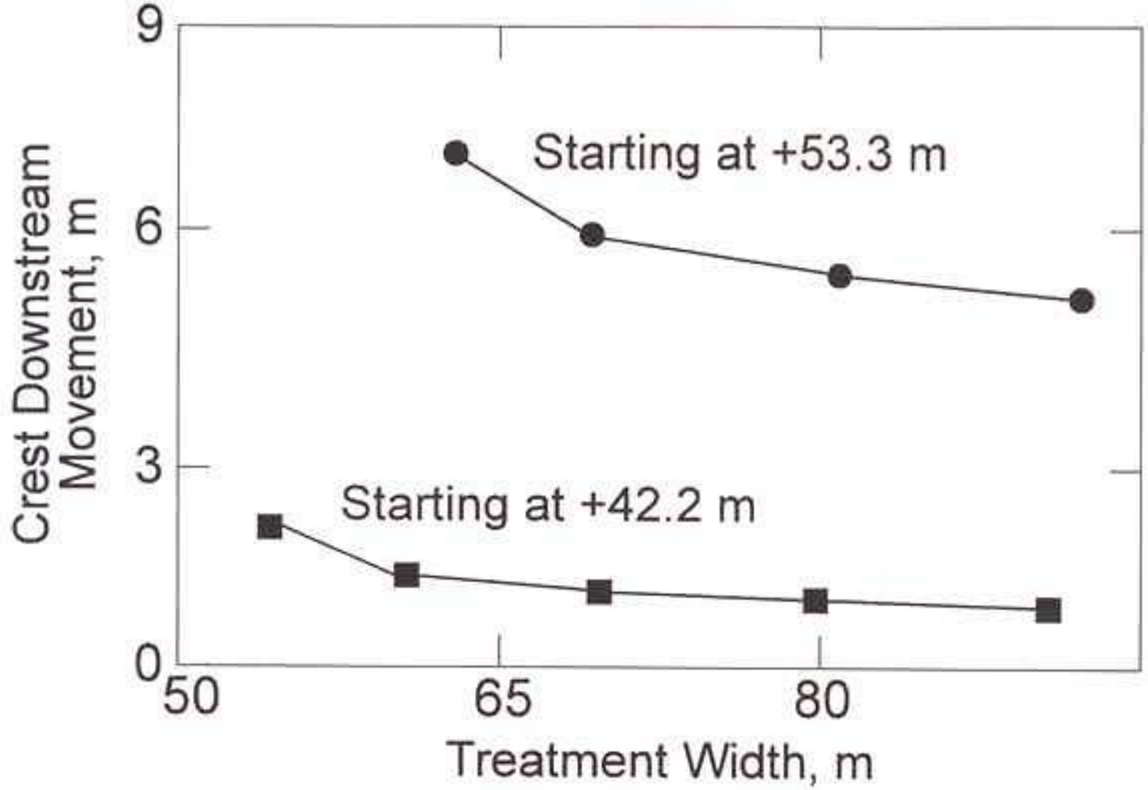
Mormon Island Auxiliary Dam: zones of liquefaction and excess pore pressure



Deformation pattern of dam at 50 percent strength reduction in liquefying materials



Cross section of Mormon Island Auxiliary Dam
showing zones to be treated



Typical effects from varying downstream treatment zone width and starting location

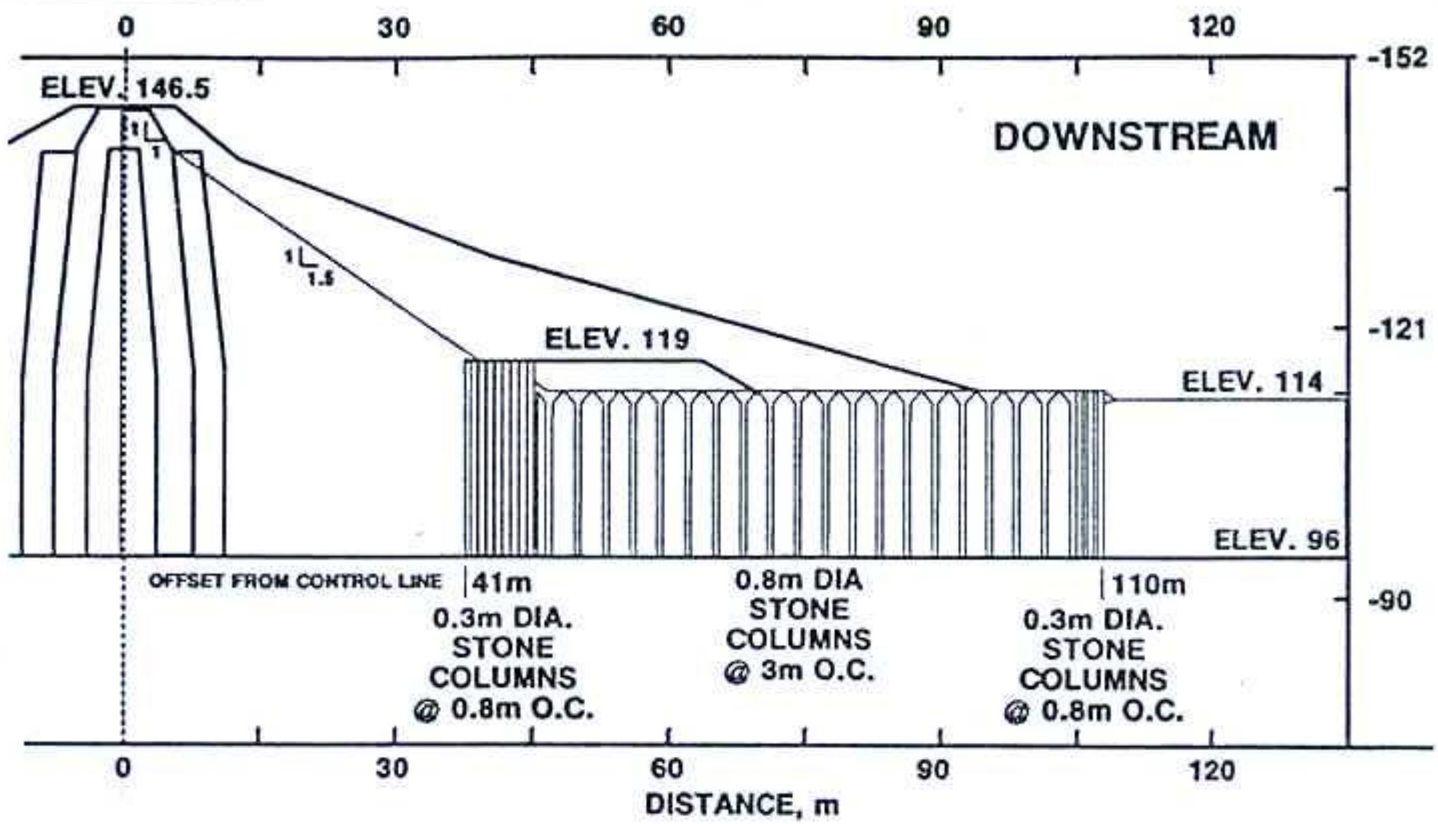
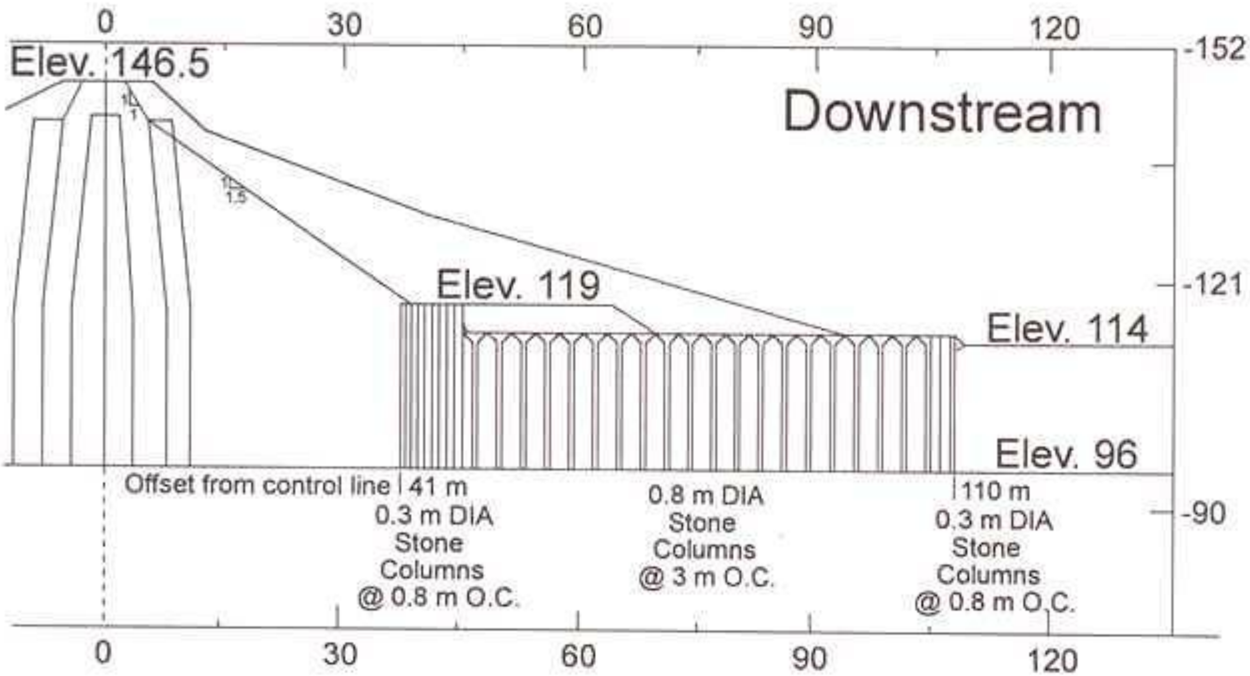
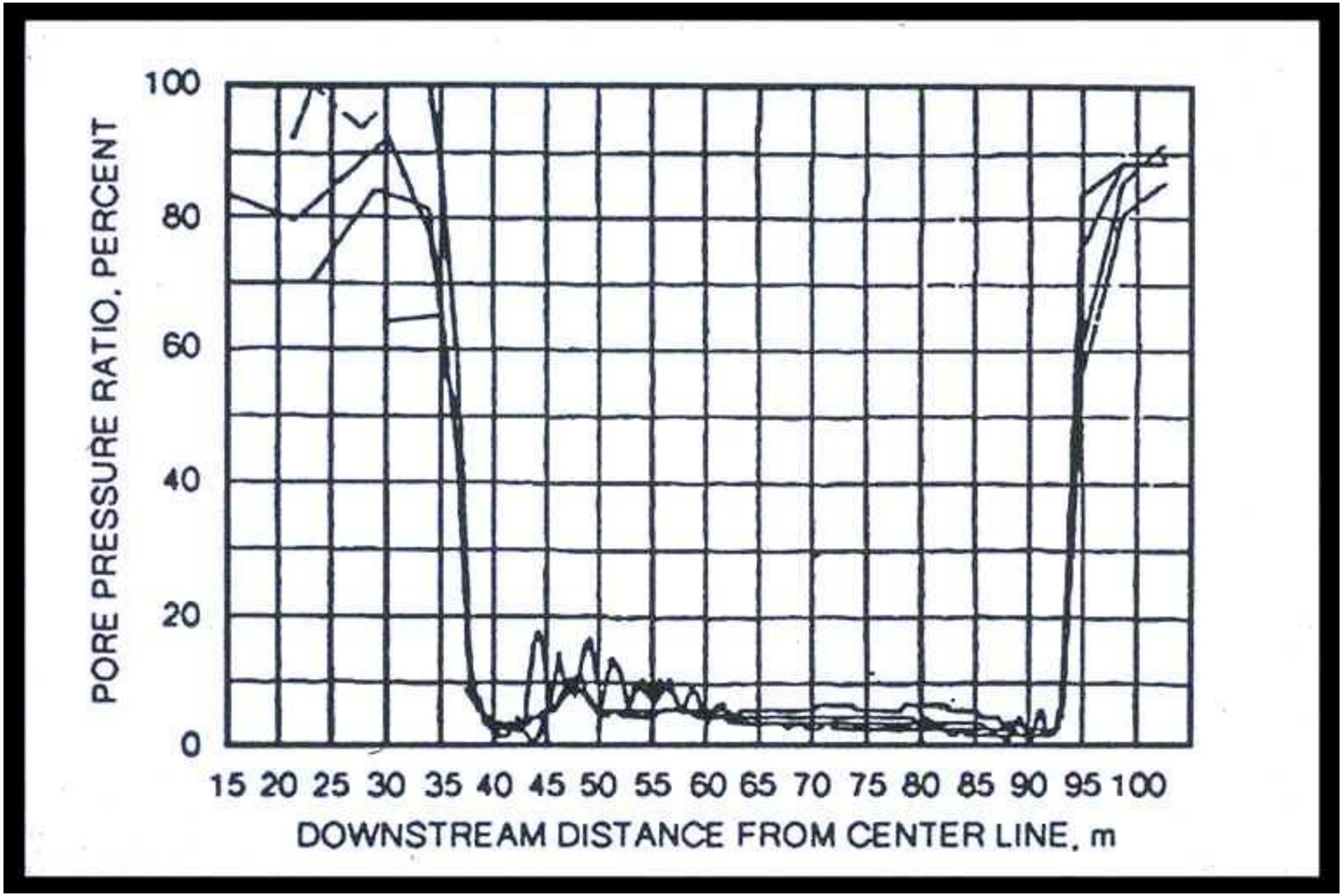


Fig. 32. Downstream remediation plan.



Downstream remediation plan


PORE PRESSURE RATIO, PERCENT
100
80
60
40
20
0
15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90 95 100
DOWNSTREAM DISTANCE FROM CENTER LINE, m