Geotechnical Aspects of Seismic Safety Evaluation and Remediation of Dams

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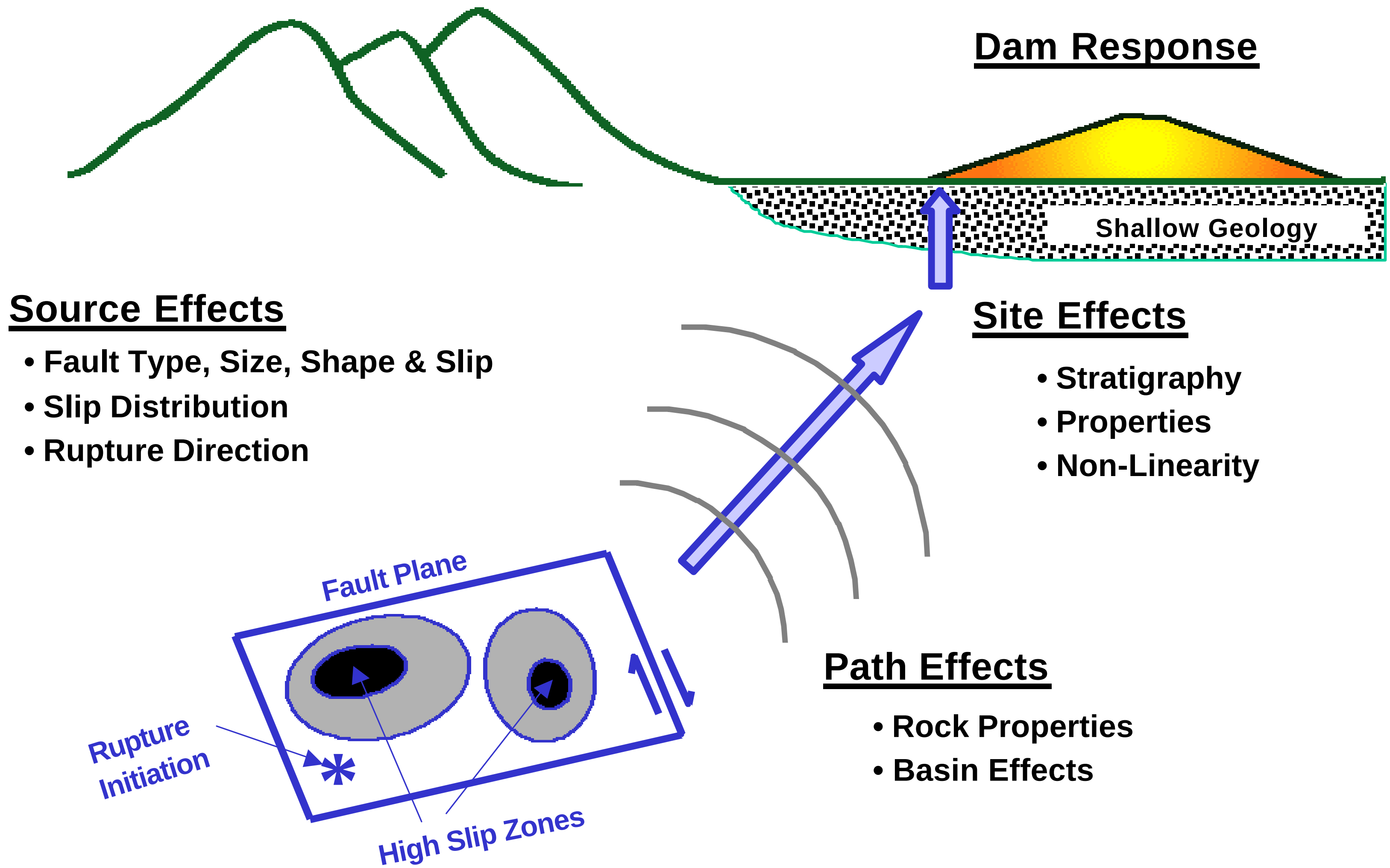
Purpose

- **Current State-of-the-Practice**
- **Skills we need to improve**
- **Knowledge gaps to reduce**
- **Future capabilities we need to develop**

Geotechnical Aspects

- **Ground motions & surface rupture**
- **Landslides**
- **Site characterization & material properties**
  - **In situ tests used as basis for characterization & property selection**
  - **Triggering of liquefaction**
  - **Residual shear strength of liquefied sand**
  - **Liquefaction behavior of silt & clay**
- **Site & dam response (observed & numerical predictions)**
- **Tolerable damage (very dam specific)**
- **Remedial measures for liquefaction**
  - **Treatment methods & important mechanisms**
  - **Role of physical modeling**

Ground Motions



Number of Recordings
M > 7.0 & R < 20 km

	SS	Rev
< 1999	3	5
> 1999	5	65

- Spectral values do not increase monotonically with earthquake magnitude.
- Directivity and permanent displacement effects are becoming better understood.

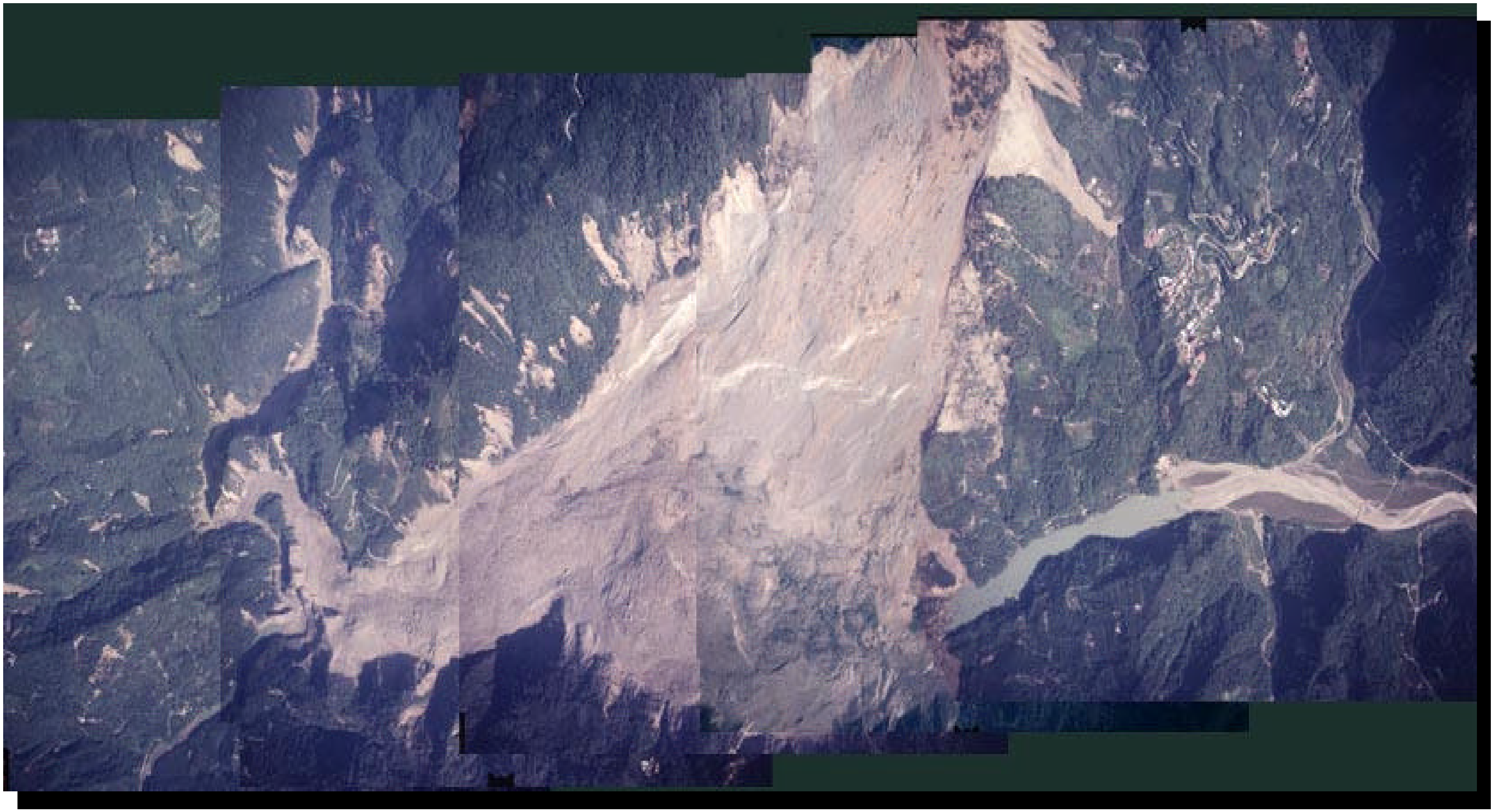
Surface rupture in rock


Surface rupture through alluvium

Earthquake Induced Landslides

- around reservoir
- at dam or appurtenant structures

Chao-lin Landslide in Taiwan - 120 million m^3

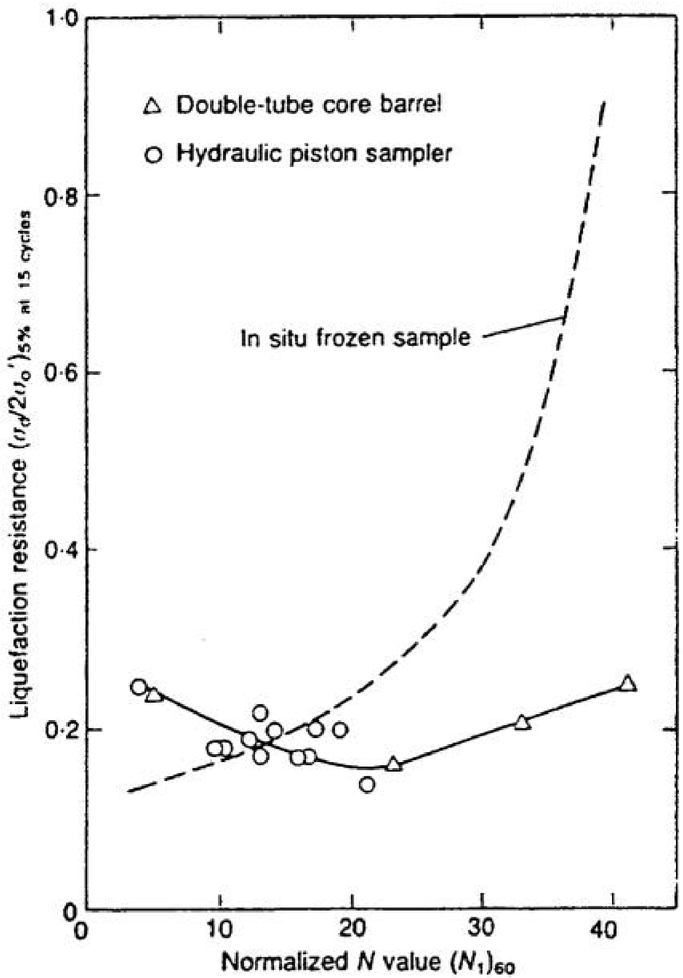


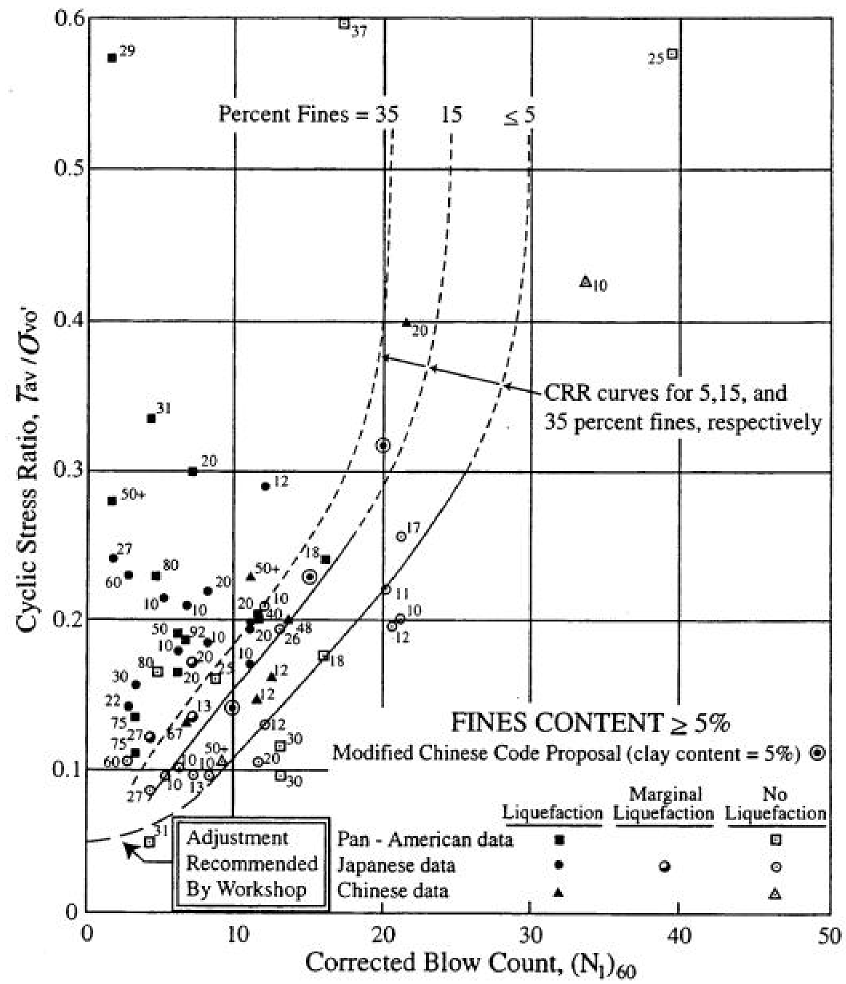
Site Characterization, Material Properties & Liquefaction Assessment

- SPT, CPT, V_S, BPT most common
- Semi-empirical correlations for liquefaction evaluations constitute state-of-practice.
  - Triggering
  - Residual shear strength
- Knowledge gaps & skills to improve:
  - Gravel & coarser particles (modernize SPT and BPT)
  - Silts and low plasticity clays
  - Probabilistic relations need refinement

Triggering of Liquefaction

- Semi-empirical approach avoids issues with sample disturbance.





NCEER (1998), modified after Seed et al. (1985)

Soils with Gravel & Coarser Particles

Harder & Seed (1986) correction

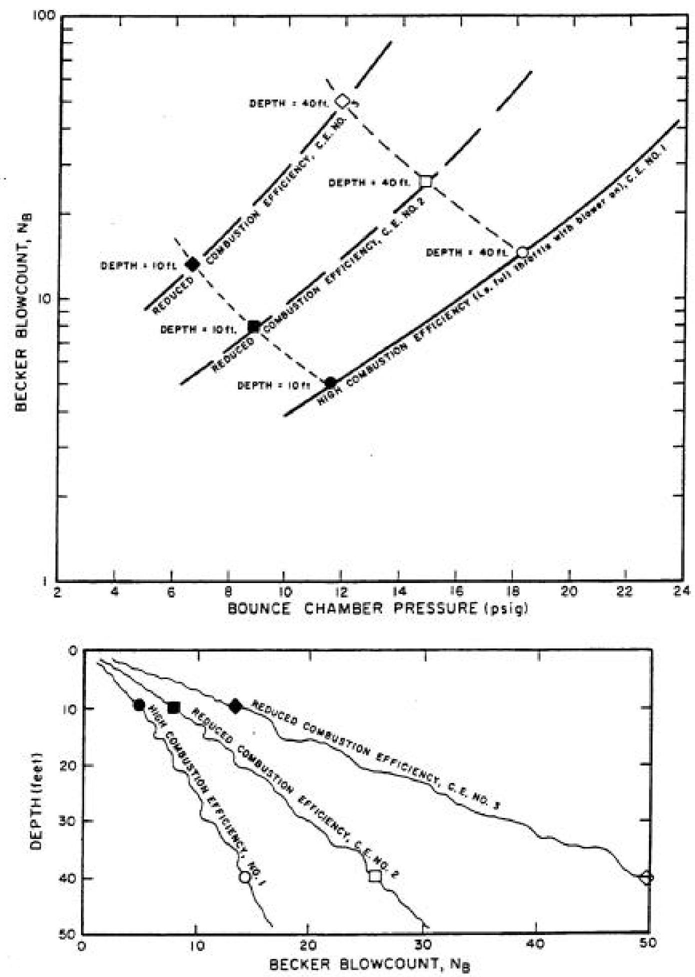


Figure 3: Effect of Diesel Hammer Combustion Efficiency on Becker Blowcount (after Harder and Seed, 1986)

Sy & Campanella (1994) correction

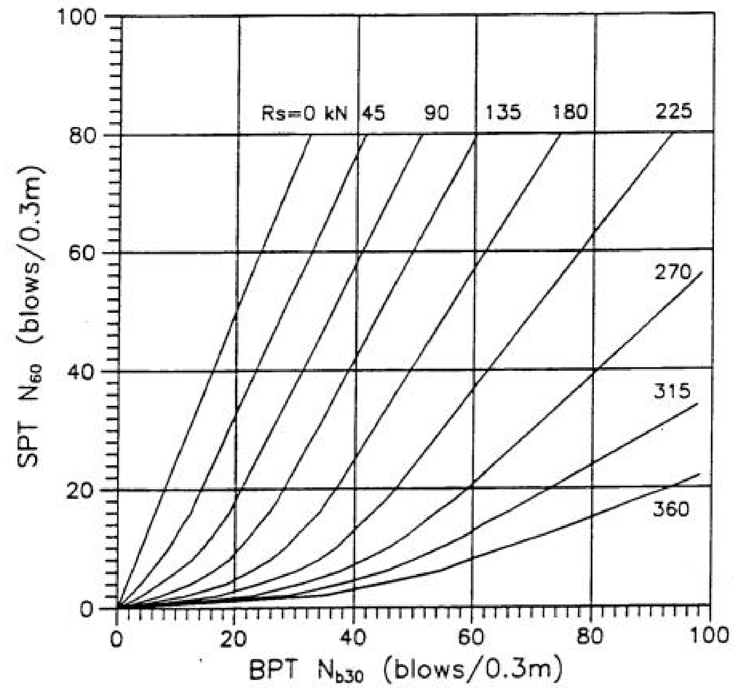


Figure 7: Computed BPT vs. SPT Correlation for Different BPT Casing Shaft Resistance (from Sy and Campanella, 1994)

- Becker Penetration Test (BPT) and SPT with continuous penetration measurements are SOP.
- BPT & SPT difficult to interpret, and corrections can be large.

Modernize Dynamic Penetration Tests

- Need to modernize BPT & SPT tests:
  - advanced instrumentation & electronics
  - automated recording of the delivered energy
  - energy & force measurements at the sampler
  - seek repeatability similar to CPT

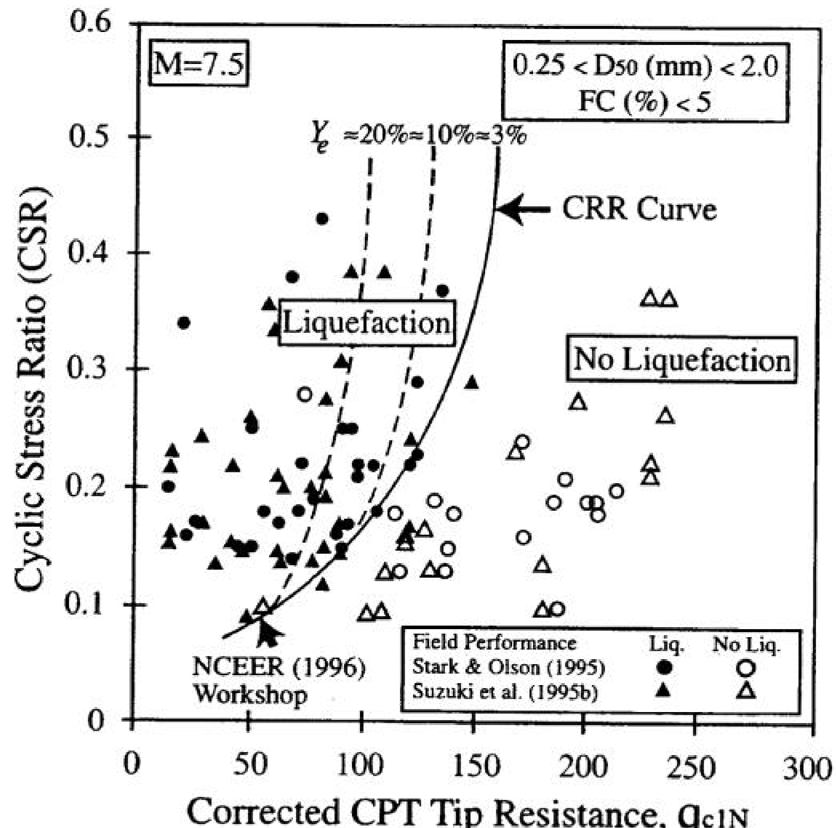
- Large Penetrometer Test (LPT) needs development in USA.

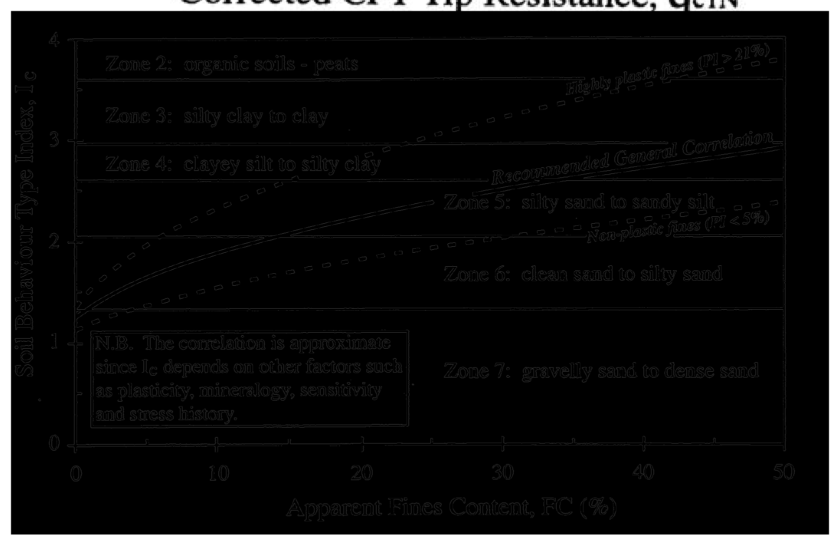
- Perhaps large-scale CPT, using sequential jacking like in some micro-tunneling setups.

Remember the CPT's history of development.

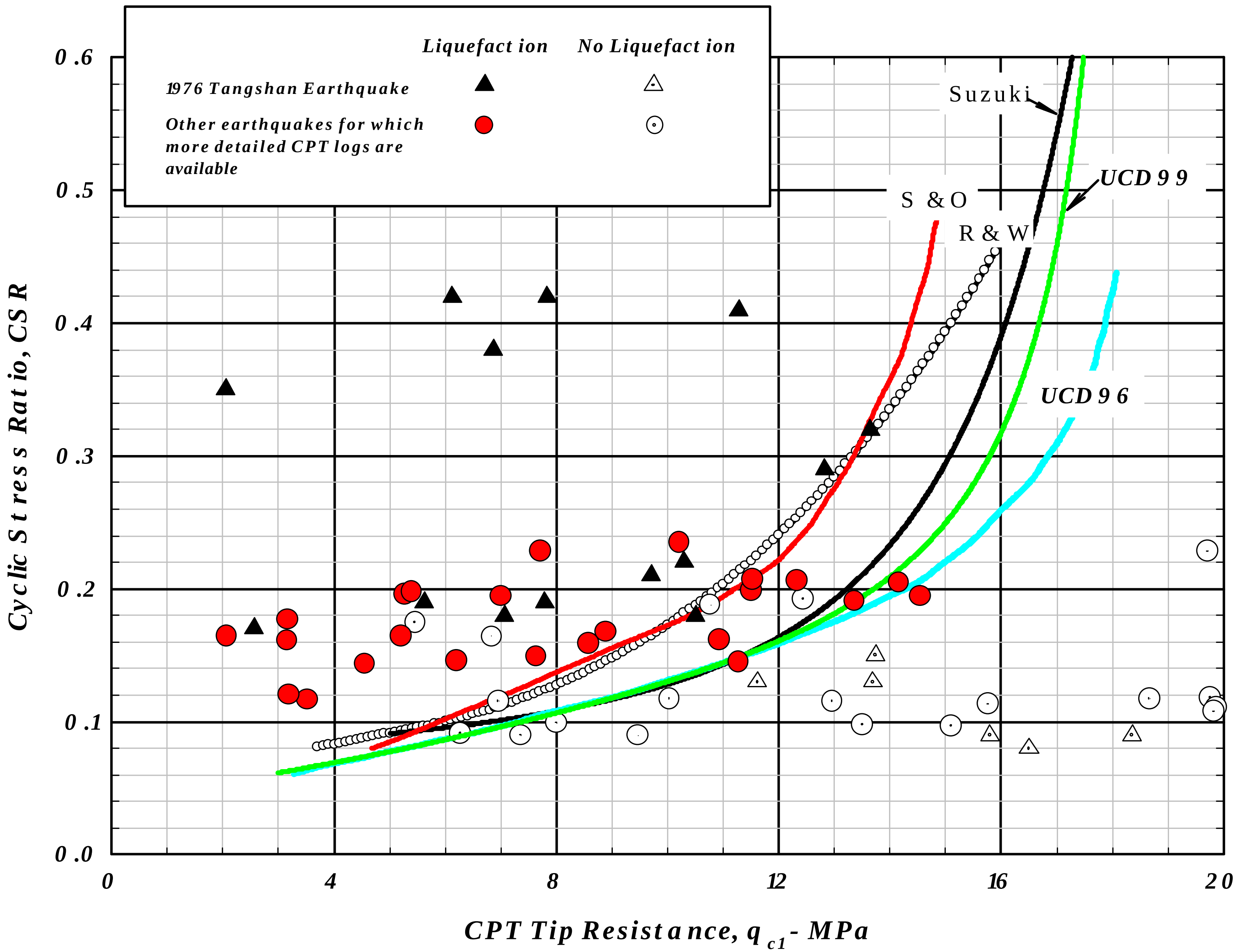
CPT Methods

- Well established approach -- excellent stratigraphic detail & repeatability.
- Several approaches proposed where the CPT data alone is used account for soil characteristics such as fines content.
- These CPT-only methods are <u>not reliable enough</u>, particularly in fine-grained soils, to warrant their use without sampling and laboratory testing. [e.g., Moss Landing data illustrate this well, Kulasingam et al. 1999].
- These CPT-only methods can predict a "coarsening" of the soil after deep densification work.
- Site-specific correlations between CPT data and soil characteristics should always be required, regardless of the project.





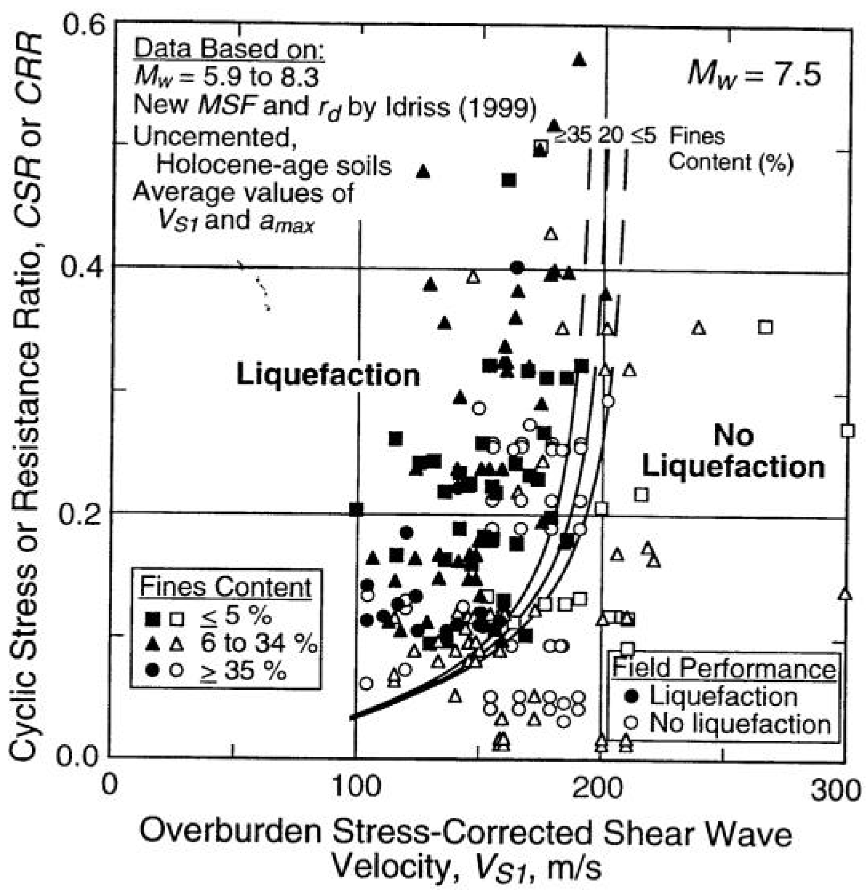
Robertson & Wride (1998)



I. M. Idriss (2000)

Shear Wave Velocity Methods

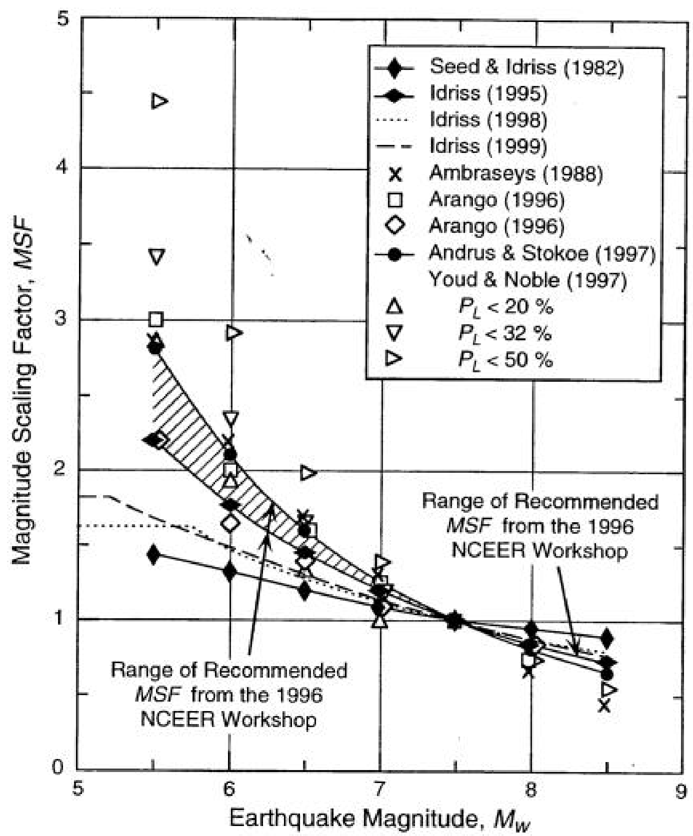
- Valuable in coarse cohesionless materials where penetrometers are obstructed.
- Need independent samples to determine soil characteristics.
- Case history database limited to relatively level ground sites with depths <10 m and uncemented soils of Holocene age.
- V_S more sensitive than penetration tests to weak inter-particle bonding.
- Can miss liquefiable layers than are thinner than the testing interval.
- Best situation is the use of several in situ tests (e.g., SPT, CPT and V_S) which complement each other & provide a check on consistency of the results.

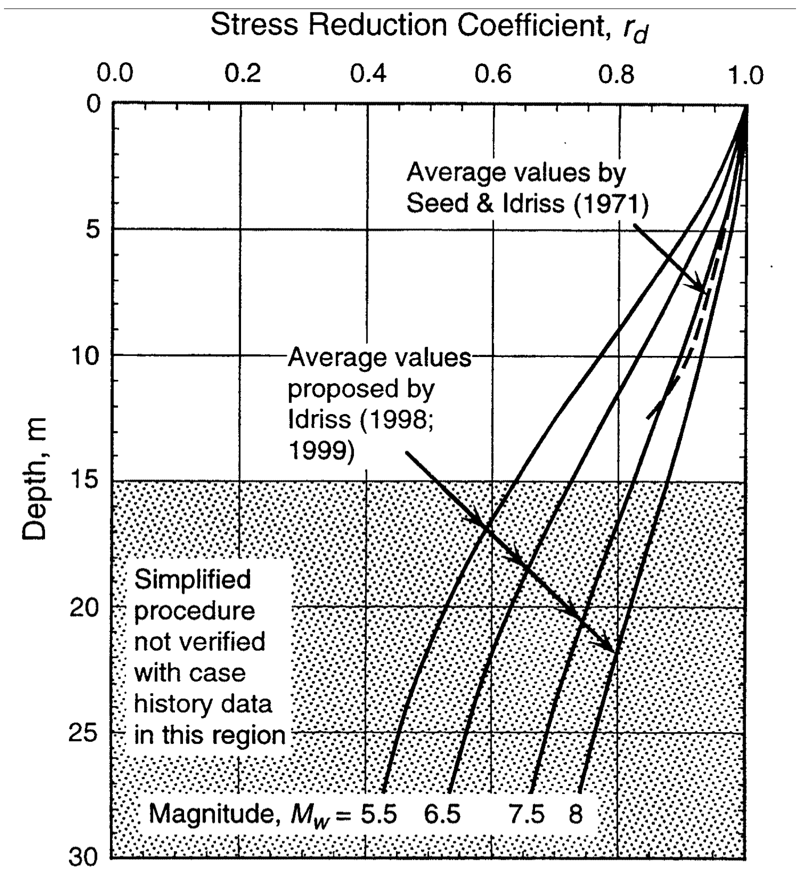


Andrus et al. (1999)

Magnitude Scaling Factor and r_d Values

MSF relations have been derived by empirical observations of liquefaction in the field and by combining lab tests with correlations between M_W and number of loading cycles. The latter approach (e.g., Idriss 1999) has a better physical basis and is recommended.

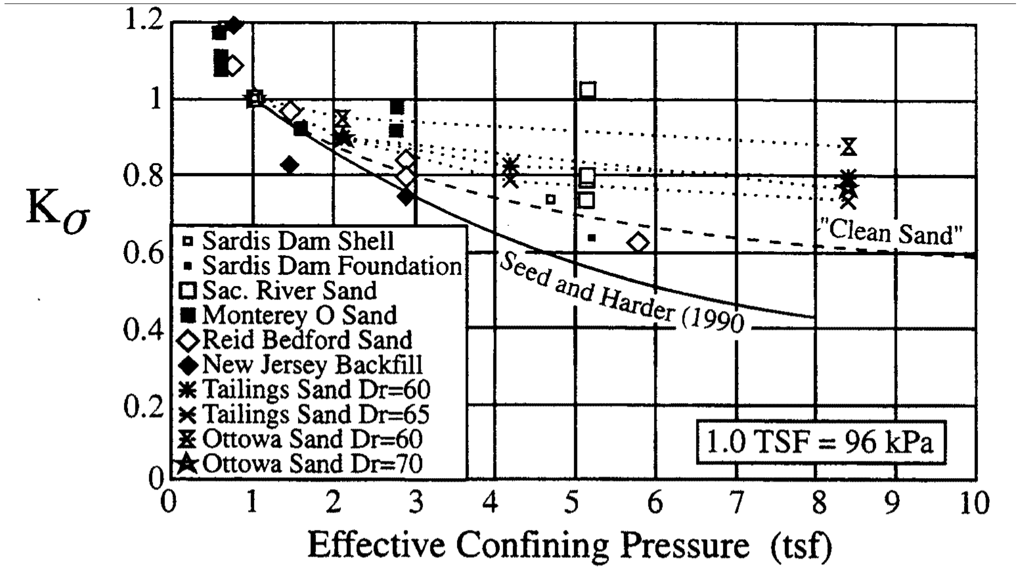


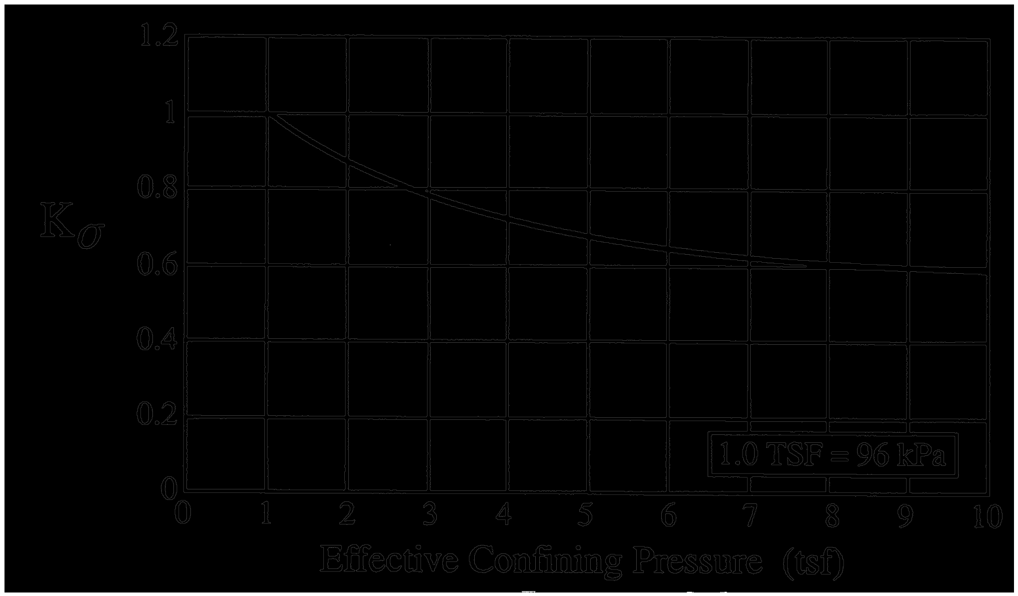


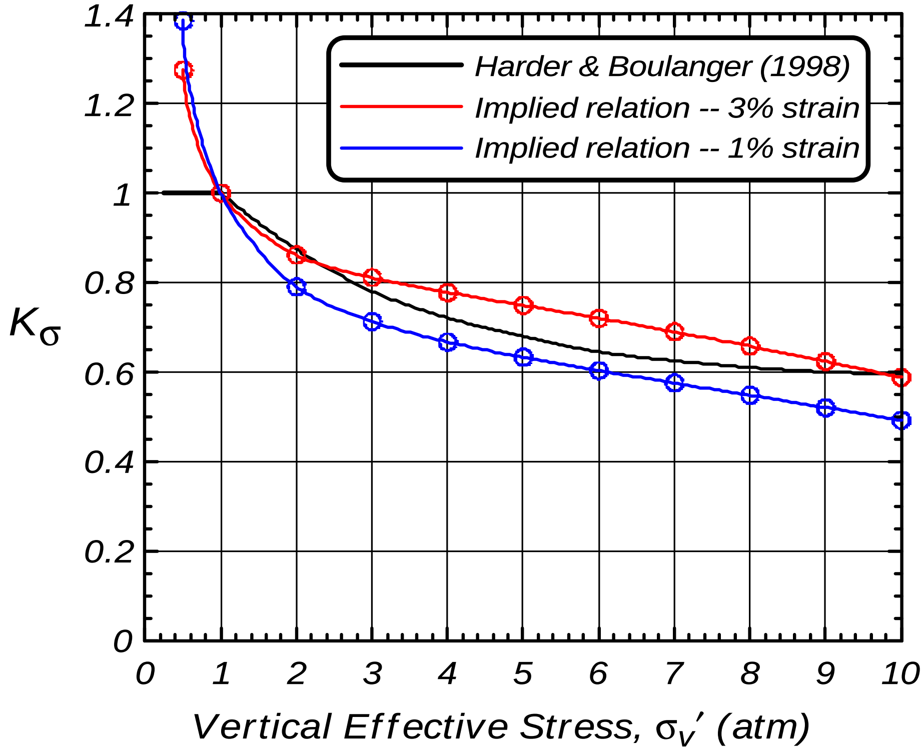
From Andrus et al. (1999)

Overburden Effects

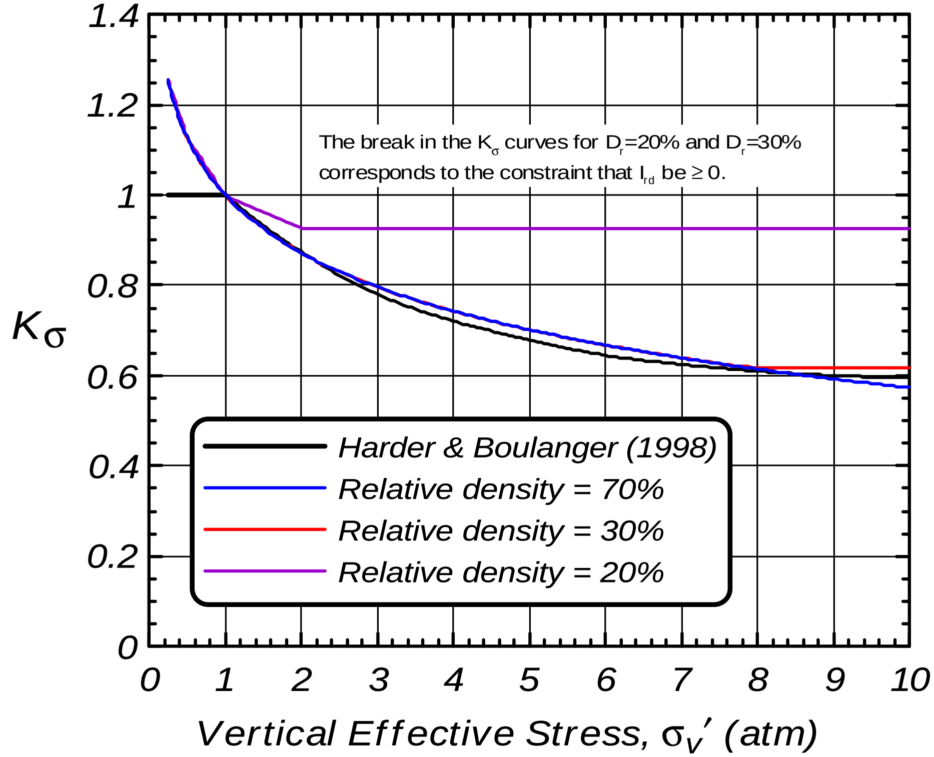
- Harder & Boulanger (1999) slightly less conservative than Seed & Harder (1990).
- WES suggested K_σ depends on soil density and compressibility.
- WES centrifuge study producing results that appear to contradict conventional K_σ concepts.







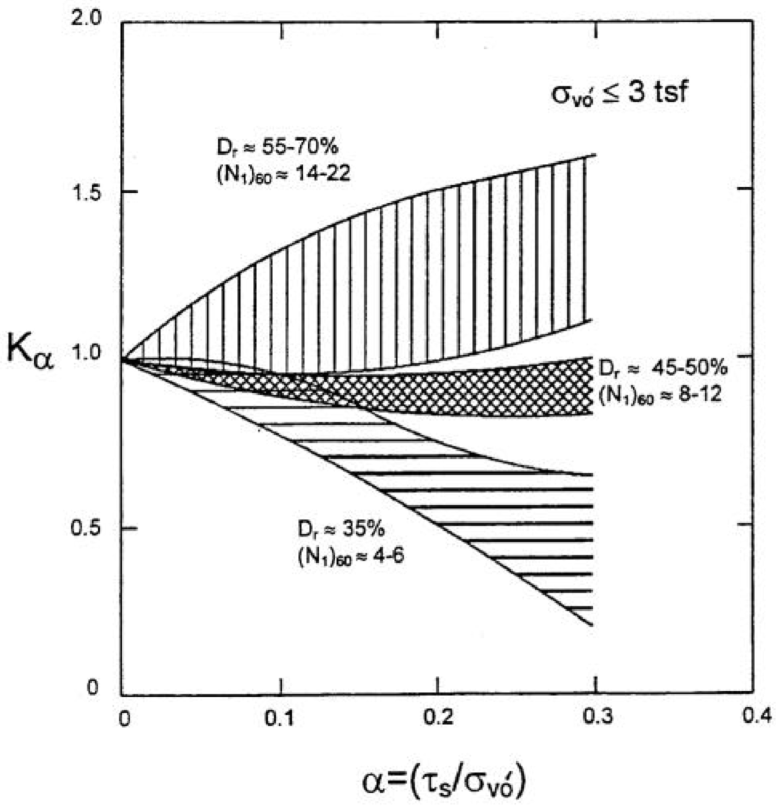
K_σ relations constructed using modulus reduction relations by Idriss (1999) and using two different shear strain criteria.

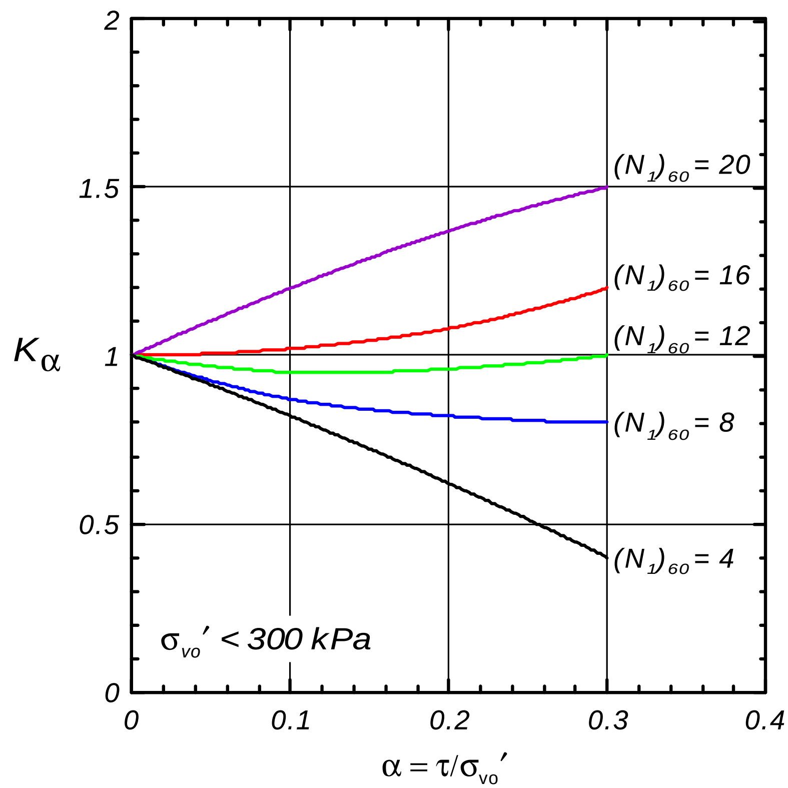


K_σ relations constructed assuming CSR_{liq} is uniquely related to the relative dilatancy index (I_{rd}) of Bolton (1990)and that the CSR_{liq} - D_r relation is linear for D_r = 20 to 70 % and σ_v' = 1 atm.

Effect of Sloping Ground

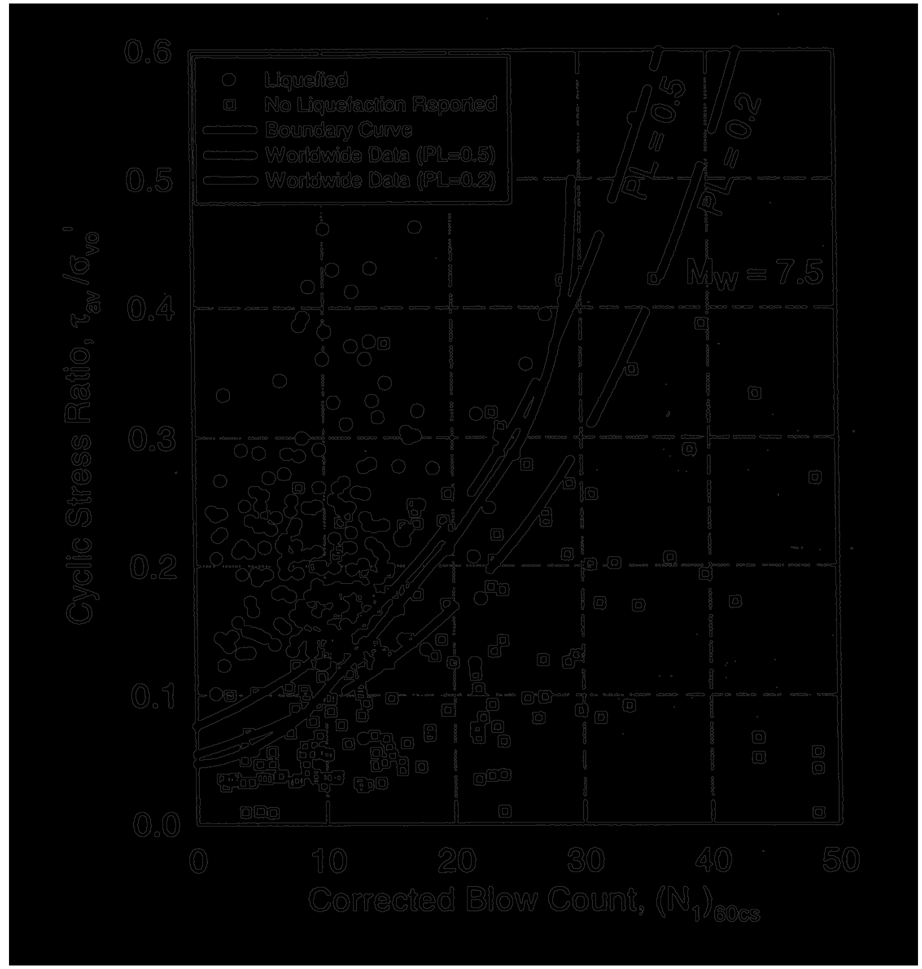
- Harder & Boulanger (1999) recommended revised K_α relations based primarily on simple shear and torsional hollow cylinder tests and on a shear strain of 3% (at which limiting pore pressure ratios are largely developed).
- K_α is a function of density and confining stress (or "state"). At much greater confining stresses, the values of K_α will be lower than shown.





Probabilistic Correlations

- Non-unique correlations between liquefaction resistance and any penetration test (SPT, BPT, CPT).
- Uncertainty in case history data.
- Judgement is used to develop the data points, & this represents some inherent probabilistic interpretation.
- Need to incorporate other physical information & fundamental understanding in developing these relations:
    - shaking table tests
    - centrifuge tests
    - laboratory tests
    - calibration chamber tests
- Probabilistic curves need further development.



Toprak et al. (1999) based on case history data points alone.

Residual Shear Strength (S_r) of Liquefied Sand

What is the residual shear strength of liquefied soil for locally undrained loading?

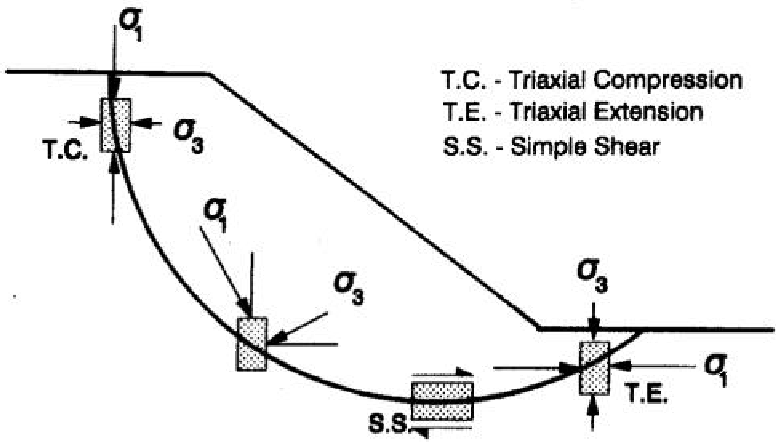
- Definition of strength?
- Stress path (including cyclic loading history)
- Fabric

What is the strength mobilized in the field under the influence of pore pressure or void redistribution (i.e., local drainage) and other factors?

- To what extent are back-calculated strengths from case histories affected by void redistribution and other factors?
- Under what field conditions are void redistribution and other factors important or unimportant?

Shear Strength under Locally Undrained Loading Conditions

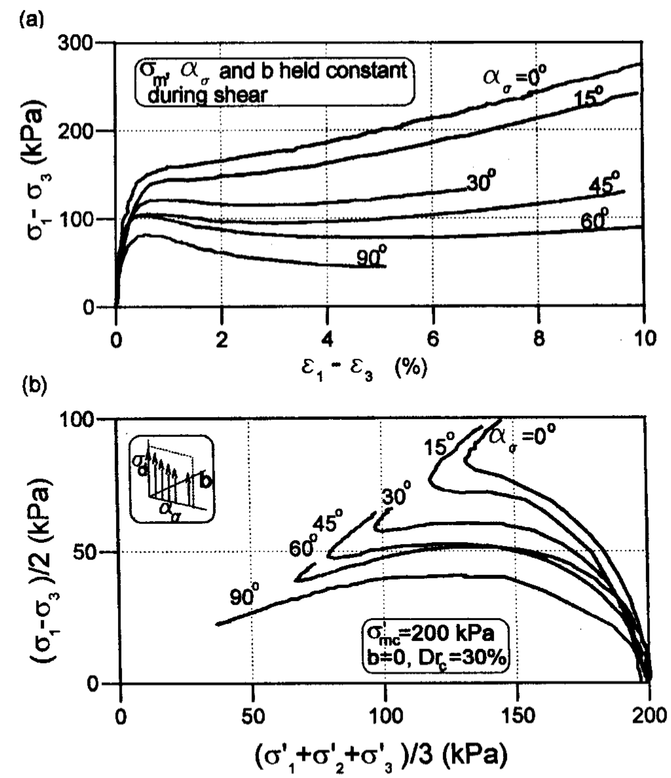
Fig. 1. Examples of principal stress direction variation along potential failure surfaces.



(a) Limiting equilibrium stress states beneath an embankment

Vaid & Sivathayalan (1998)

Fig. 5. Deviator stress – maximum shear strain and stress path response at $b = 0$ for Fraser River sand.



Vaid et al. (1998)

Defining shear strength:

- peak resistance?
- acceptable strain level?

The use of quasi-steady state (QSS) strengths (e.g, at phase transformation) implicitly adopts strain as the criteria defining strength.

Fig. 5. Static undrained simple shear behaviour at a fixed confining stress.

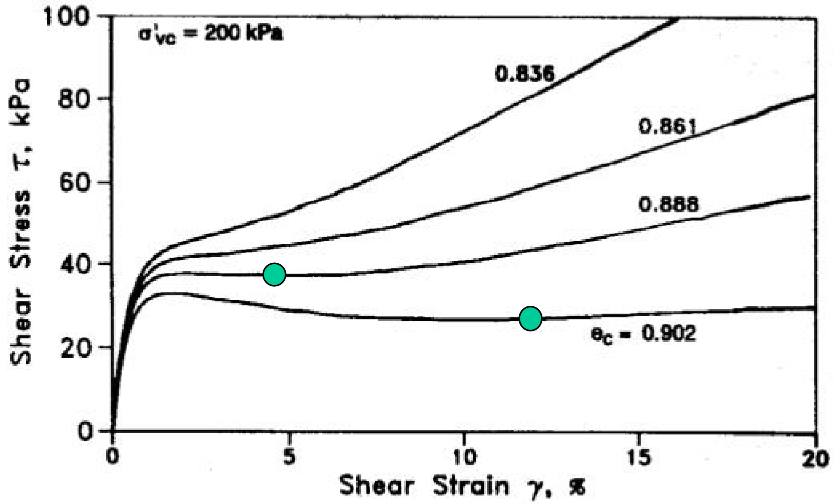
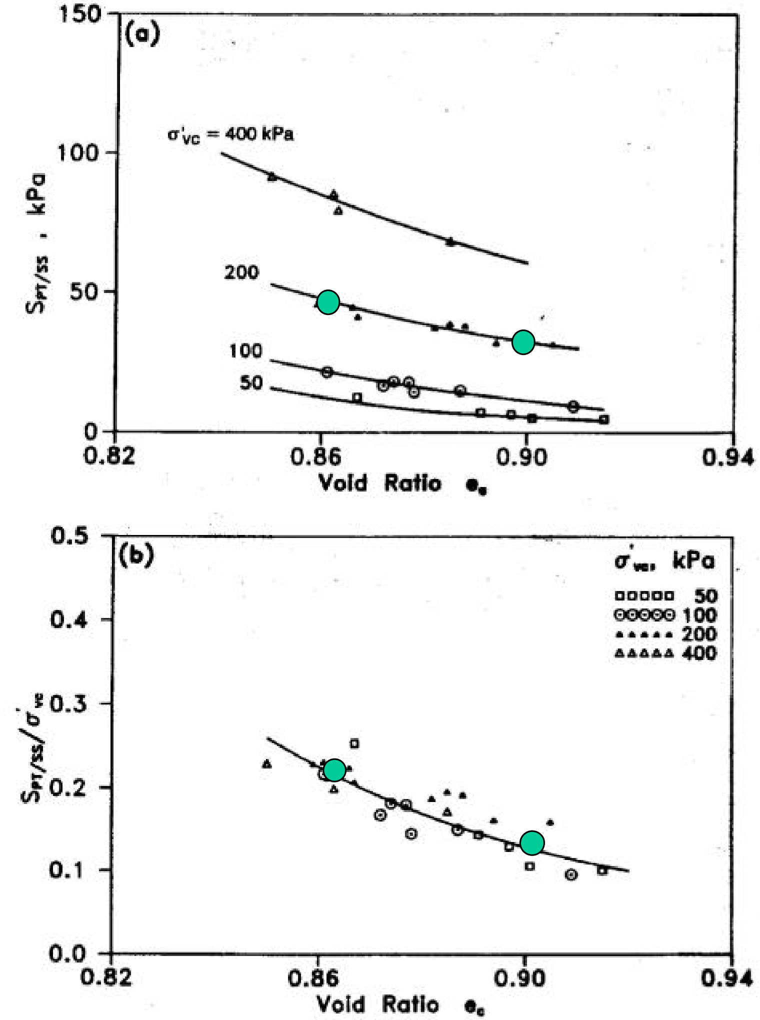


Fig. 10. Variation of undrained strength at PT/SS state with void ratio and confining stress in simple shear.



- S_r/σ_{vc}' ratios are reasonable for describing the QSS resistance, and are less sensitive to density than is the peak resistance.
- The strain at which a QSS resistance is mobilized is much smaller for a medium-dense sand than for a very loose sand.

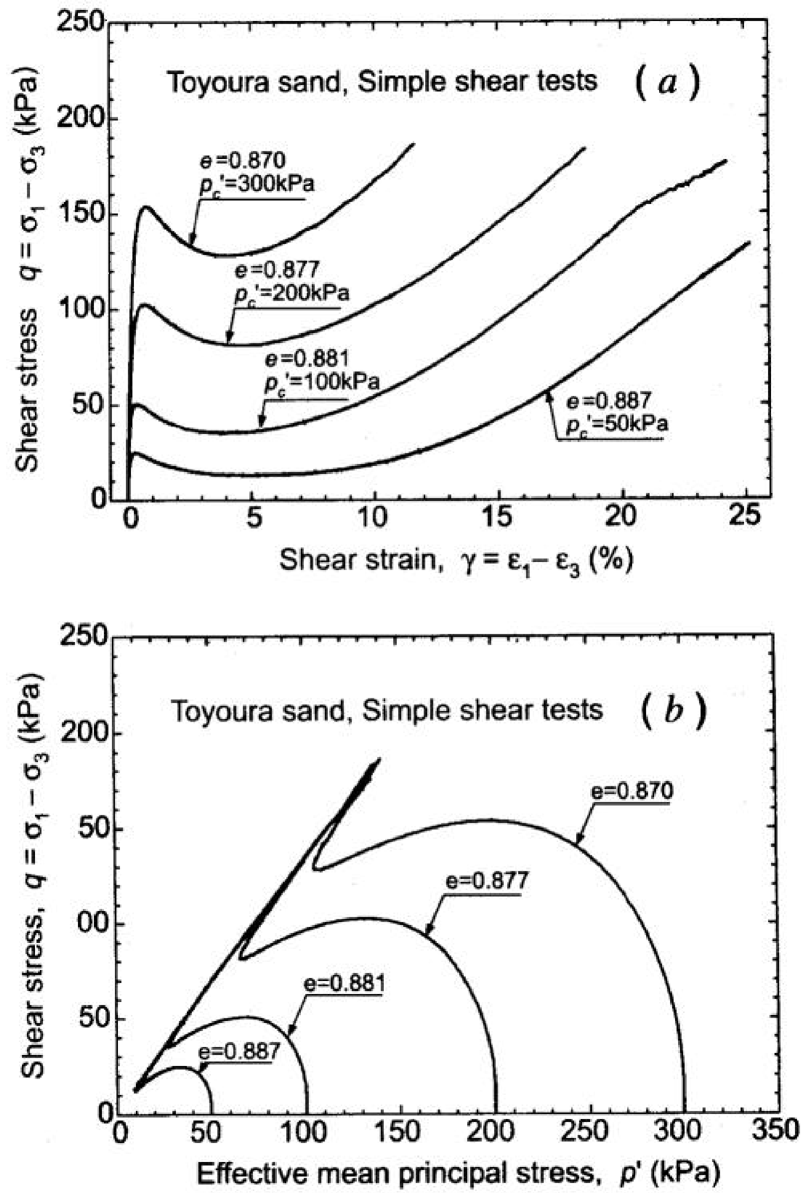
Vaid & Sivathayalan (1996)

- Stress paths from the start of undrained loading to the phase transformation point are very similar for a range of confining stresses. This is why S_r/σ_{vc}' ratios are a relatively unique descriptor of the QSS resistance for a given loading path and specimen and density.

- Strain-hardening after the QSS resistance has been mobilized can be very pronounced.

- The shear resistances do not converge at large strains, but it must be recognized that strains are highly non-uniform in most test specimens when the global strains reach 15 and 20%.

Fig. D1. Undrained simple shear behavior of Toyoura sand.



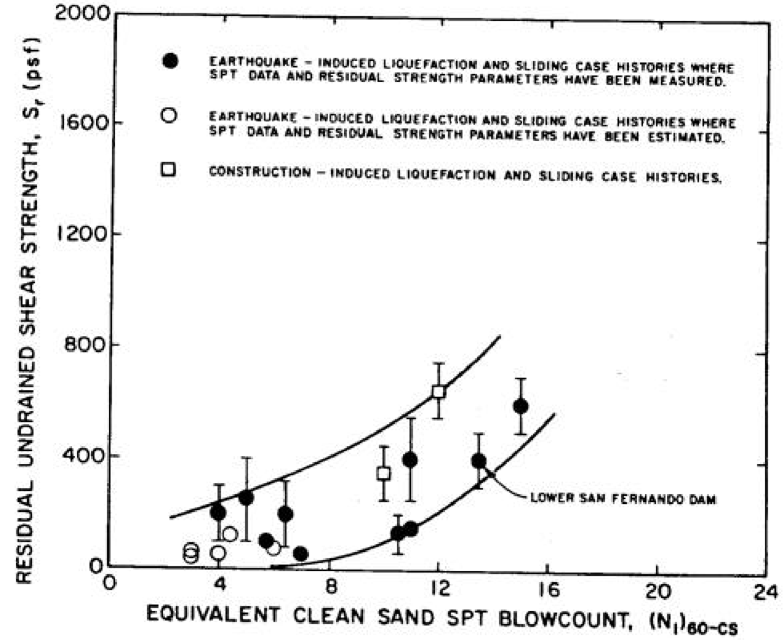
Yoshimine (1999)

Back-Calculated S_r Values

Seed (1986) back-calculated S_r values from case histories of flow failures.

He argued that any effects that void redistribution and/or other factors may or may not have had on S_r were implicitly accounted for.

For $N_{1\text{-}60\text{-}CS}$ values greater than 15, the problem is how to select an undrained strength or how to estimate induced shear strains.



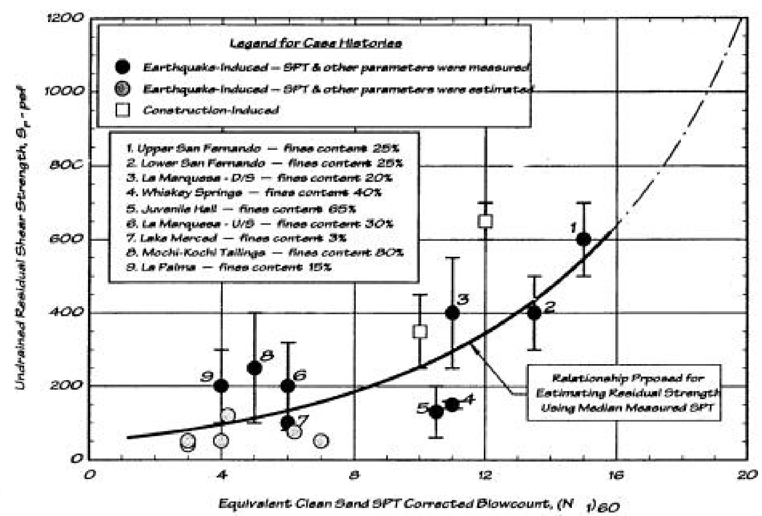


Fig. 14 Undrained Residual Strength, Sr, versus Equivalent Clean Sand SPT Corrected Blowcount Based on Field Case Studies Published by Seed (1987) and by Seed & Harder (1990)

Idriss (1999)

Shear Strength under the Influence of Void Redistribution & Other Factors

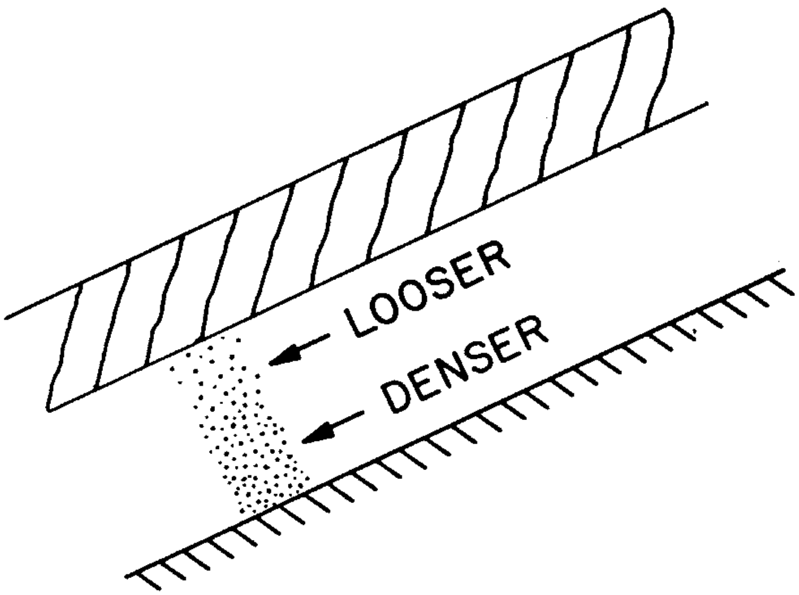


Figure 1. Mechanism B by NRC (1985) - Example of potential void redistribution within a globally undrained sand layer.

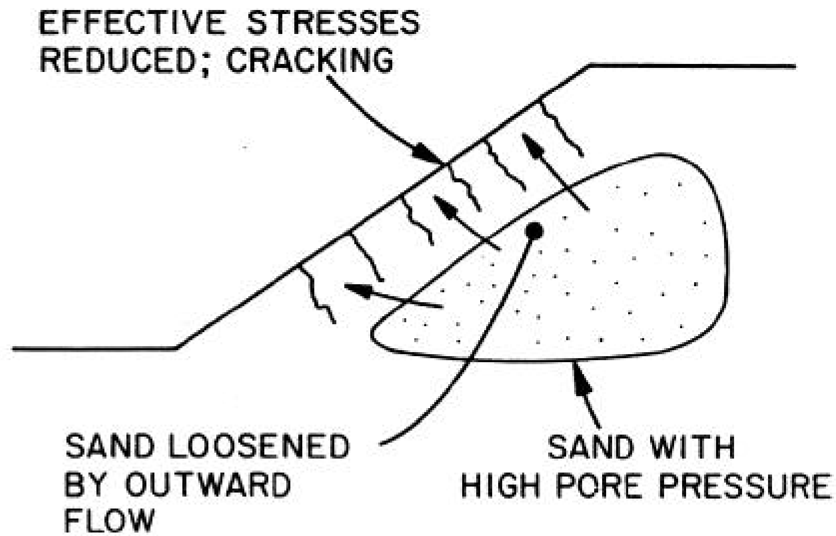


Figure 2. Mechanism C by NRC (1985) – Example of potential failure by spreading of excess pore pressures with global volume changes.

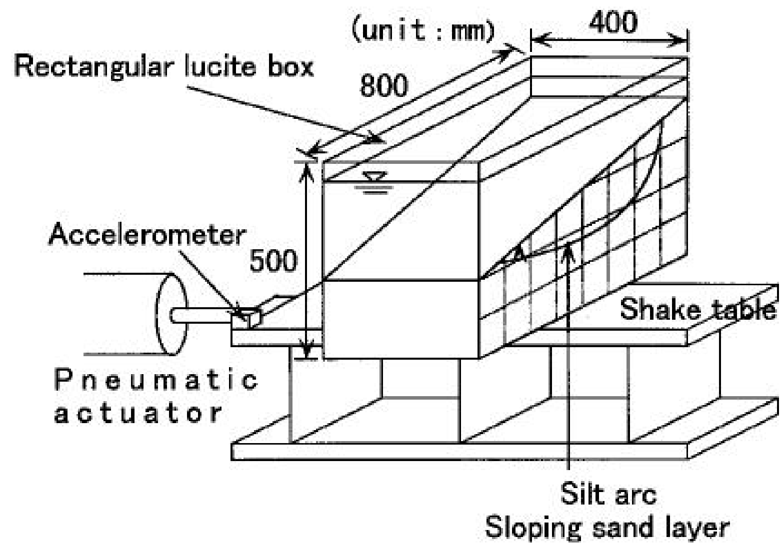


Figure 3. Shaking Table Test with a Silt Arc to Study Effect of Water Film (Kokusho 1999)

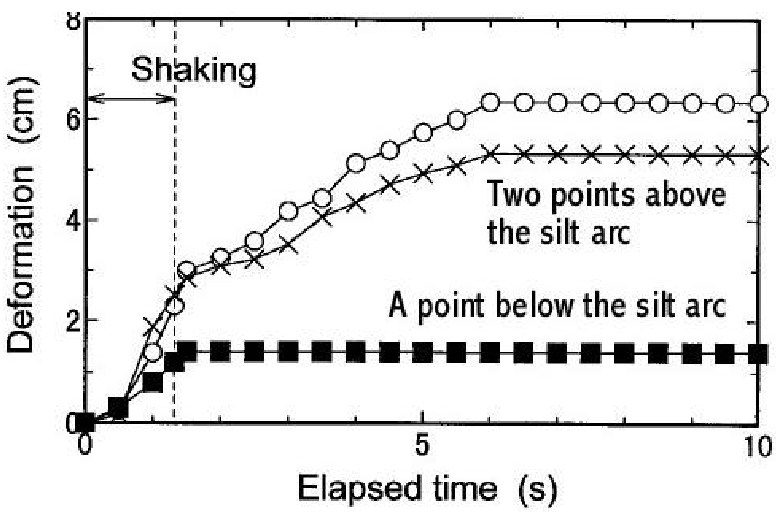


Figure 4. Deformation of Points Above Silt Arc Continued After Shaking

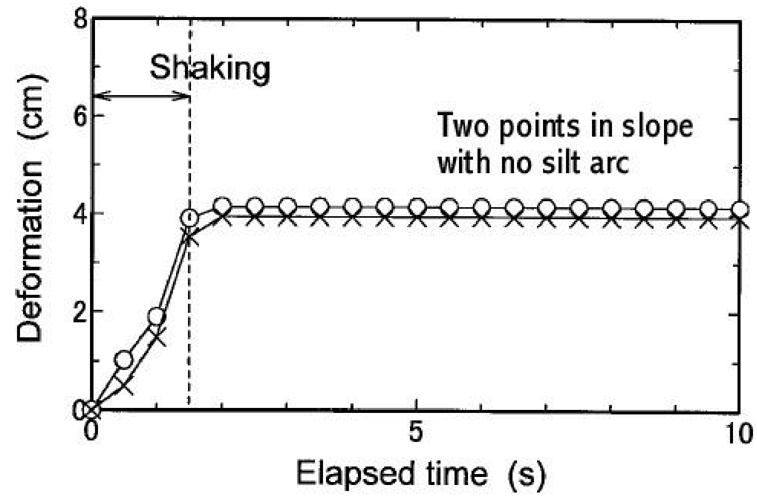
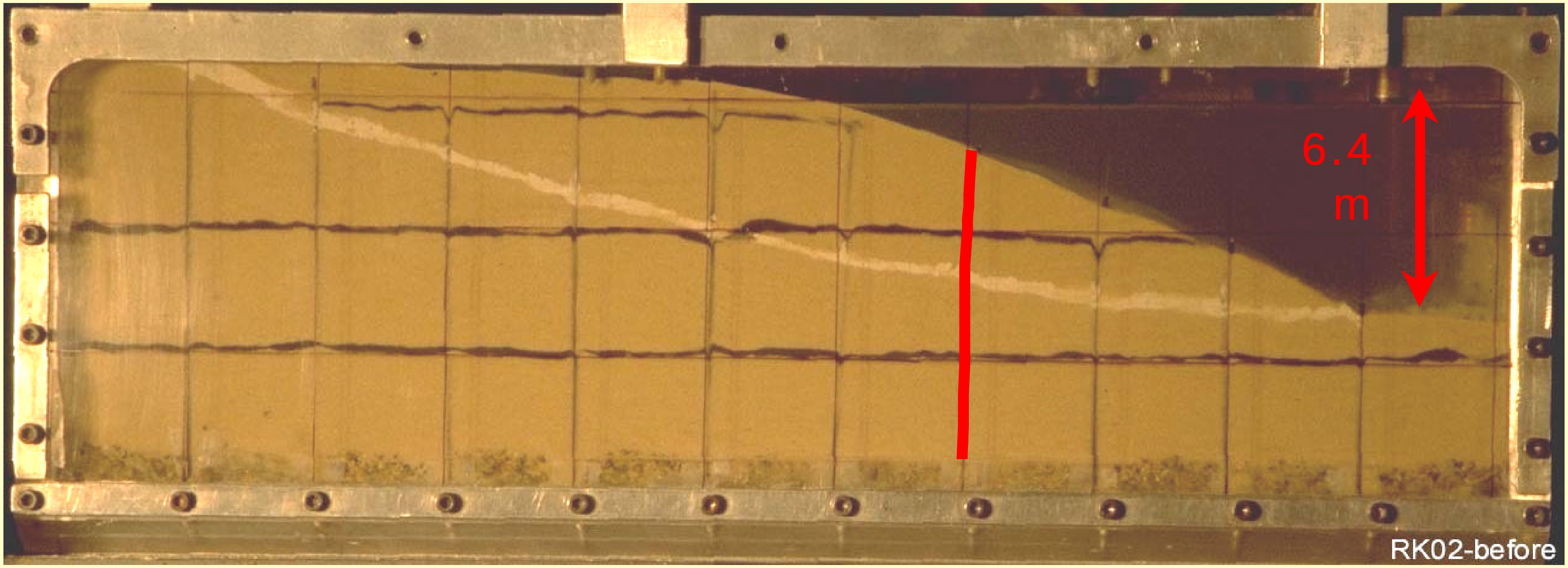
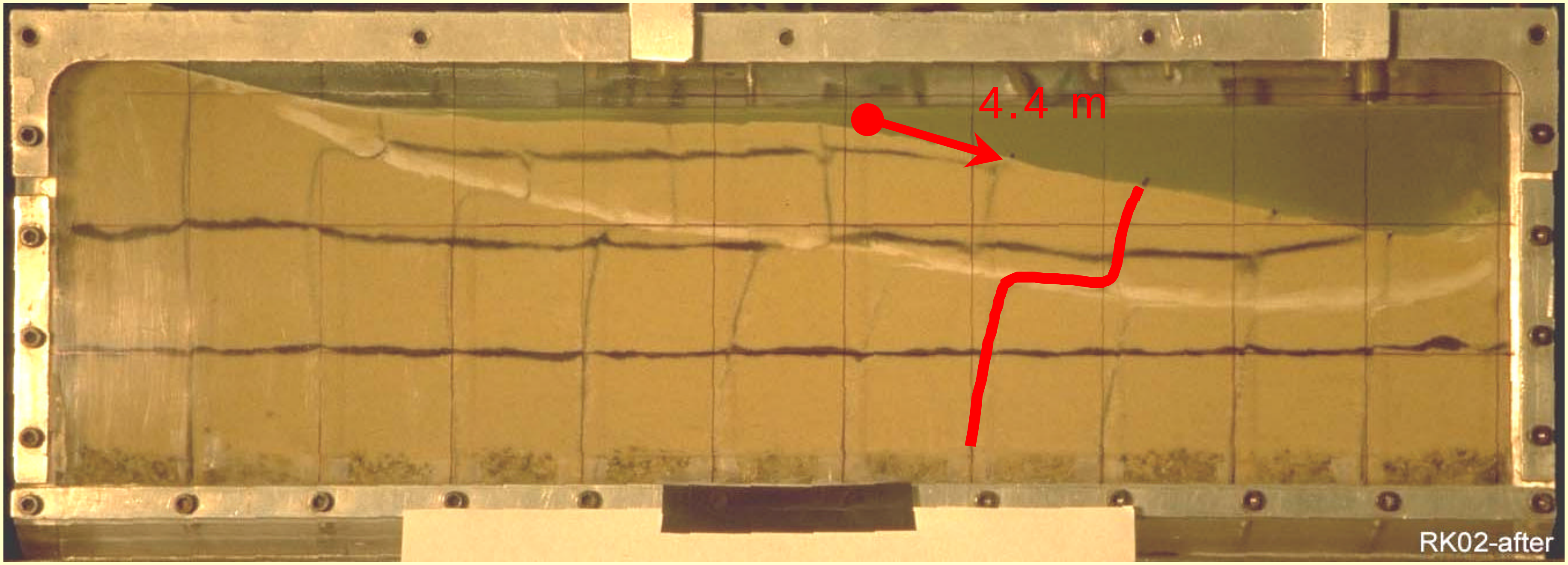


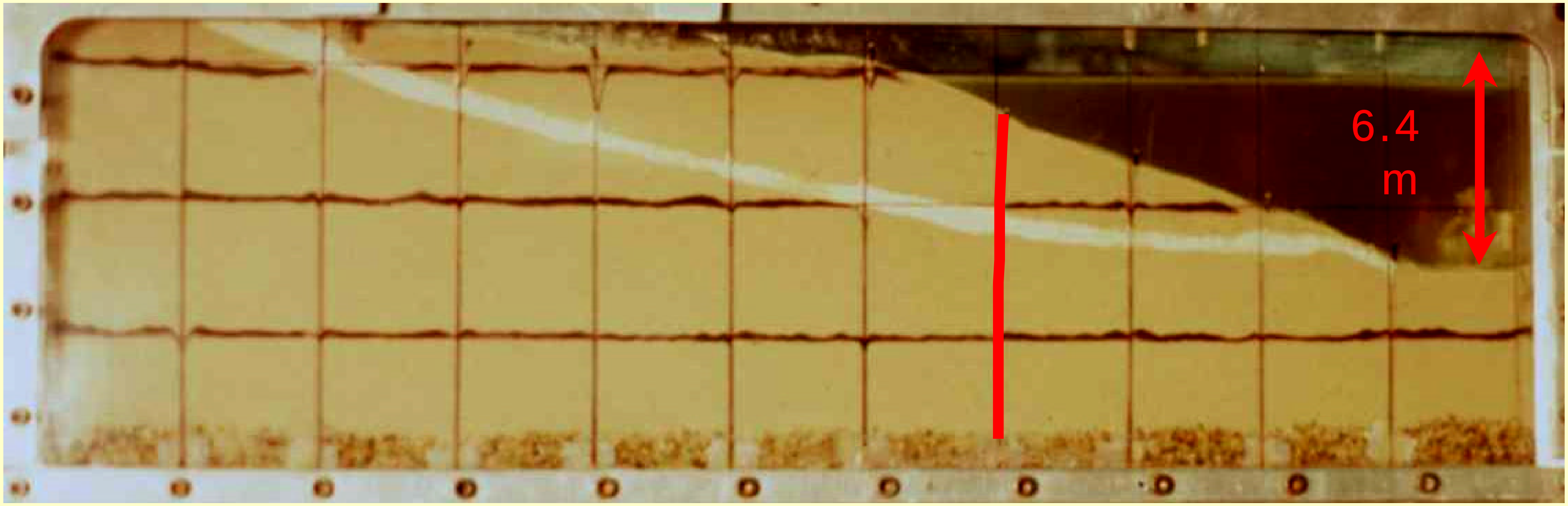
Figure 5. Deformation of Points in the Homogenous Slope Without a Silt Arc Stopped After Shaking (Kokusho 1999)

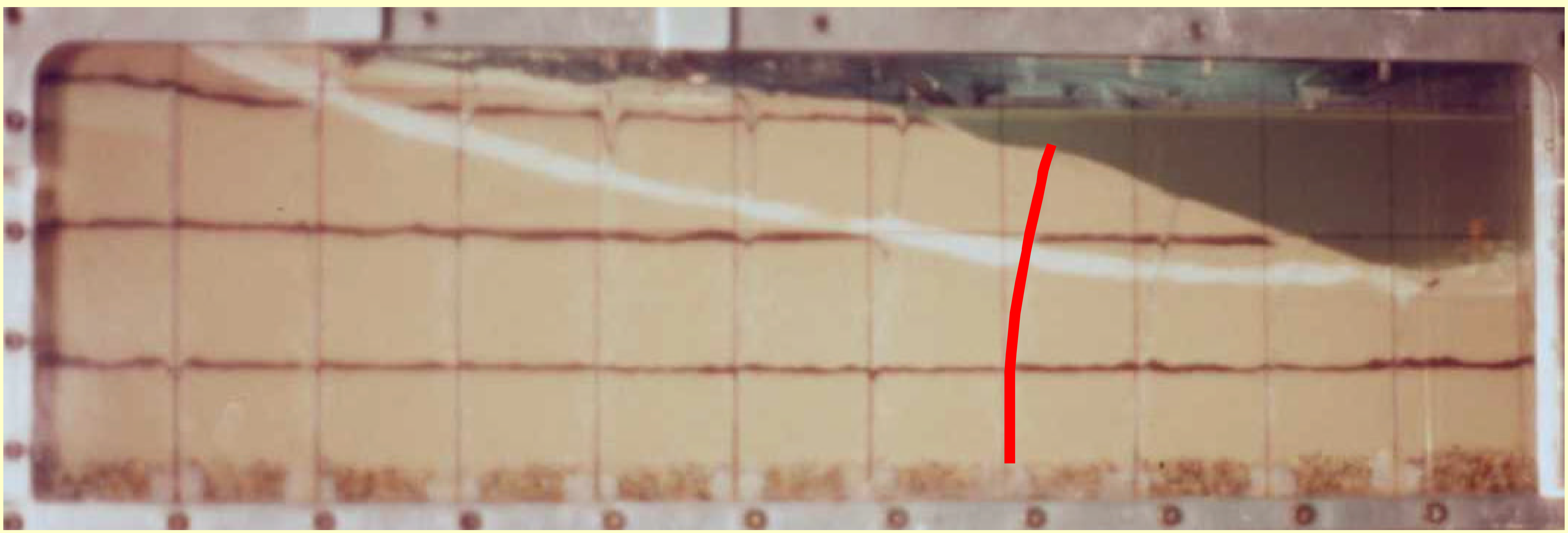
Test 5: D_r = 20%, Before shaking & after 2 shaking events



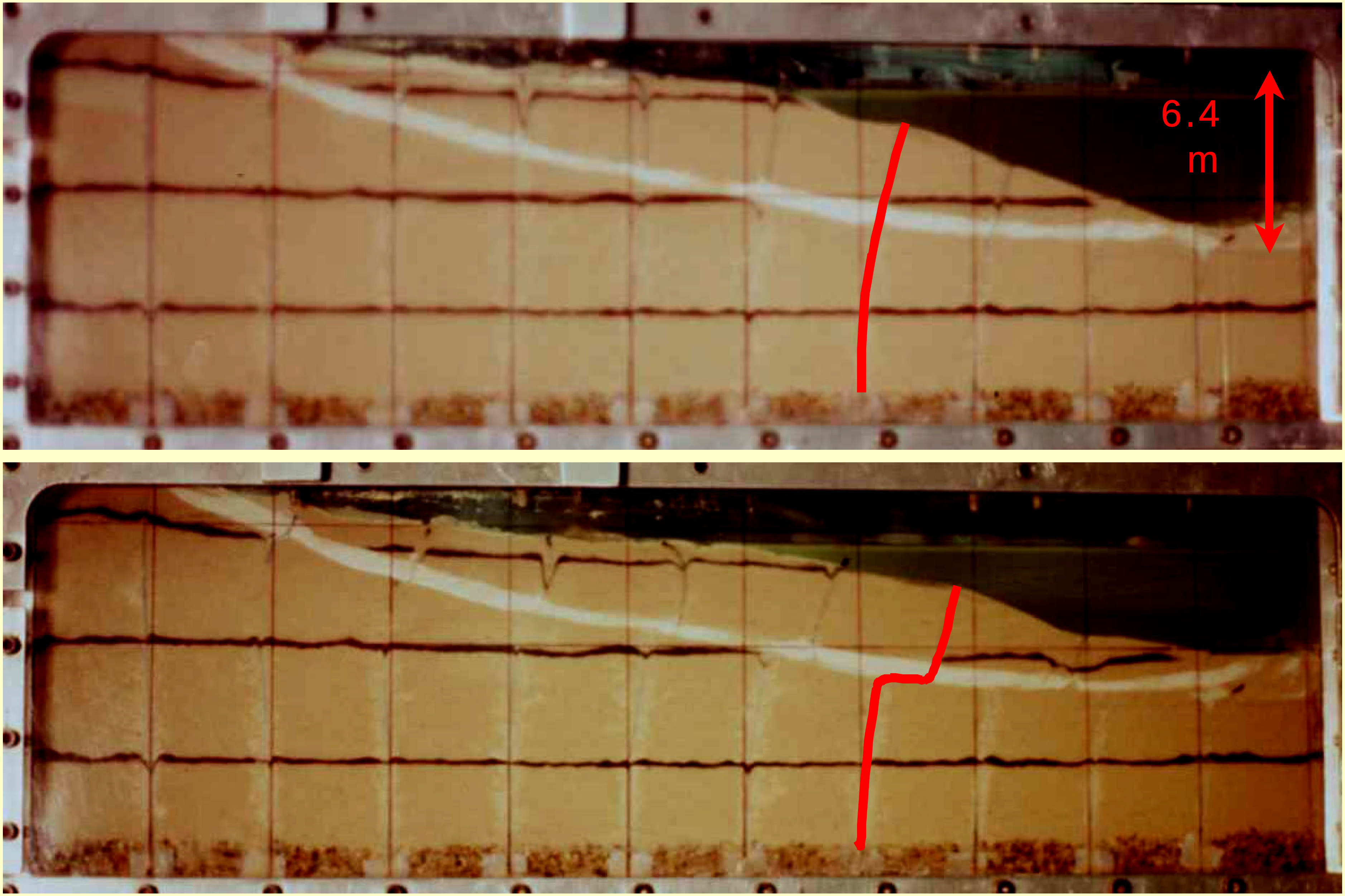


Test 8: D_r = 40%, Before shaking & after a short-duration event that triggered high r_u values





Test 8: D_r = 40%, Before & after a 2nd shaking event, which was the 1st motion plus long-duration aftershocks



Implications

- Void redistribution is more pronounced at the field scale than in laboratory test devices.

- The in situ S_r of liquefied soil depends on:
  - in situ boundary and loading conditions
    - stratigraphy
    - permeabilities
    - earthquake characteristics
    - stress path
  - pre-earthquake soil properties and state.

- Understanding this phenomena will improve our estimates of S_r and provide guidance on remediation strategies.

Silts & Clays

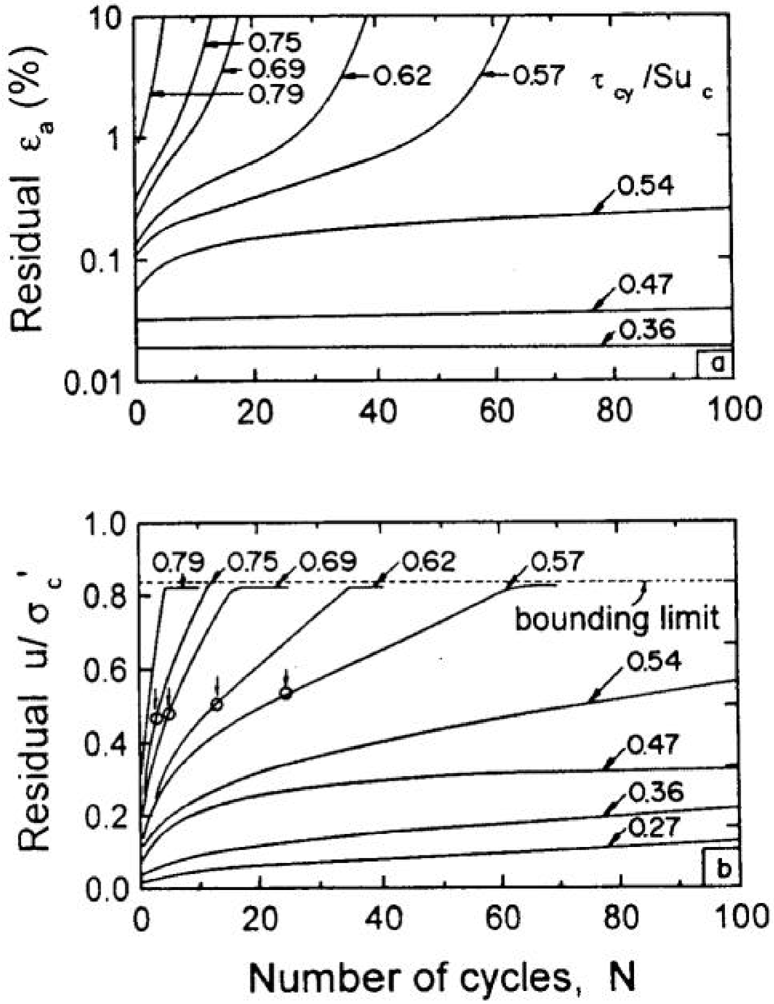


FIG. 8. Development of residual strain and pore pressure with cycles of loading.

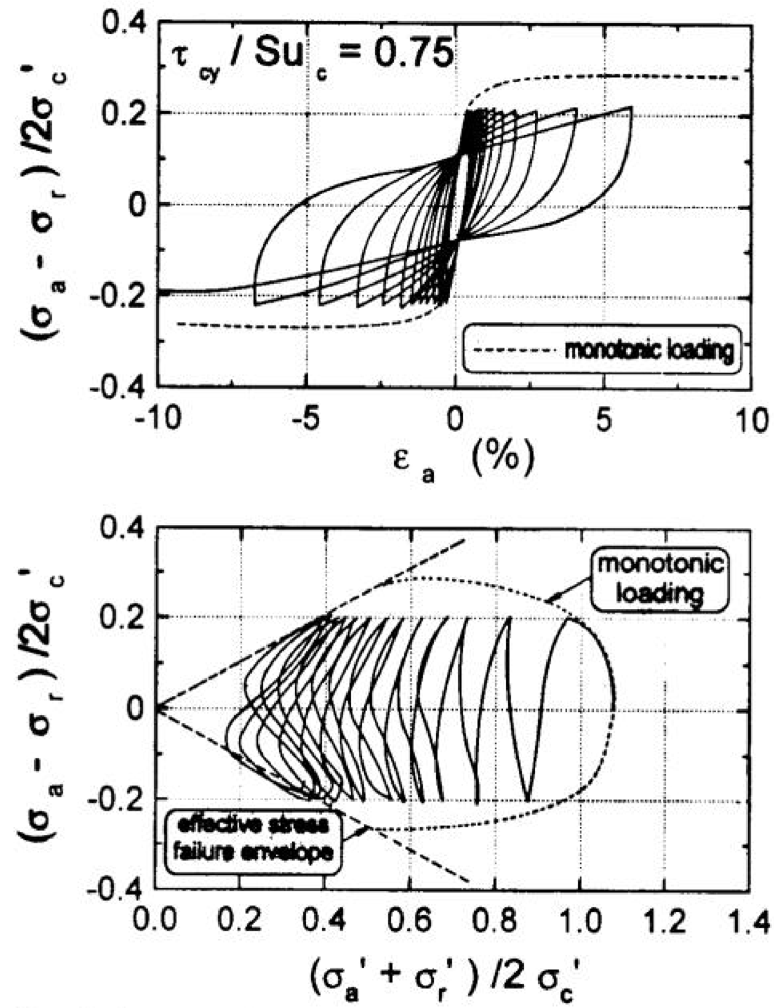


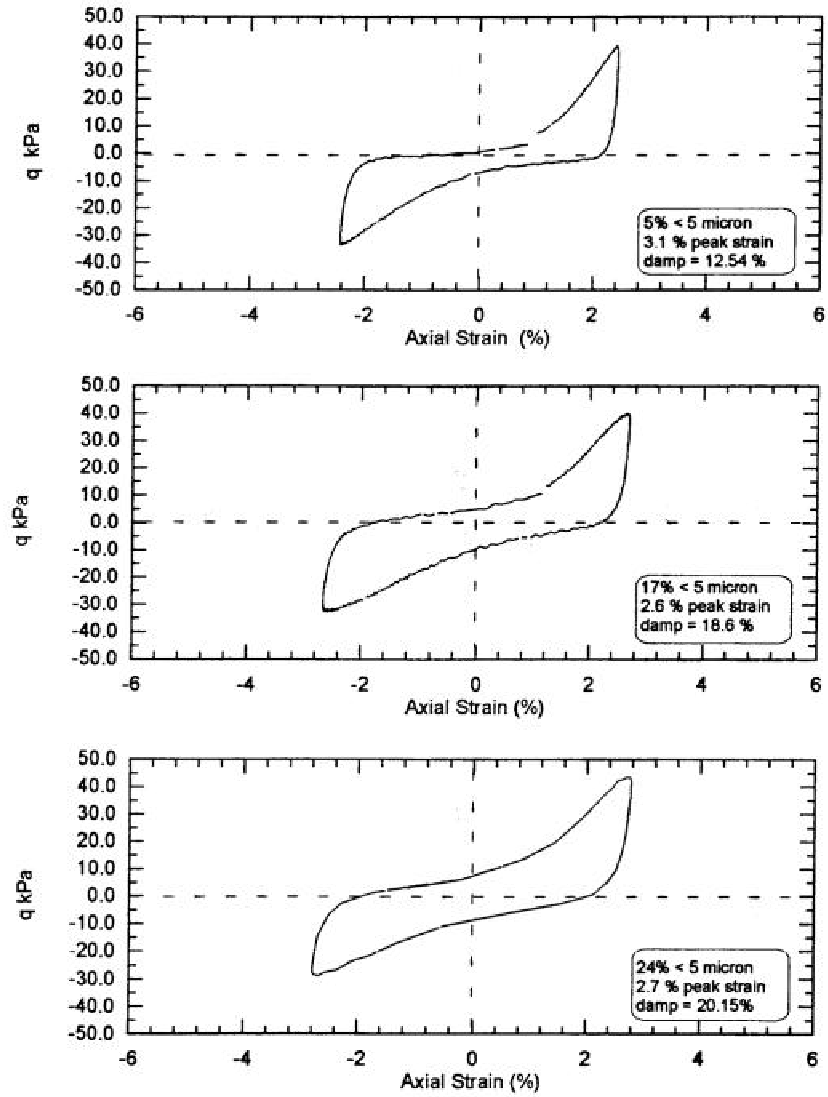
FIG. 9. Stress–strain response and effective stress paths during cycles of loading.

Triaxial tests on Cloverdale clay by Zergoun & Vaid (1994).

Romero (1996) prepared slurry-sedimented triaxial specimens of varying clay content. Cyclic and monotonic undrained tests were performed for a range of consolidation stresses and overconsolidation stresses.

Increasing clay content results in:

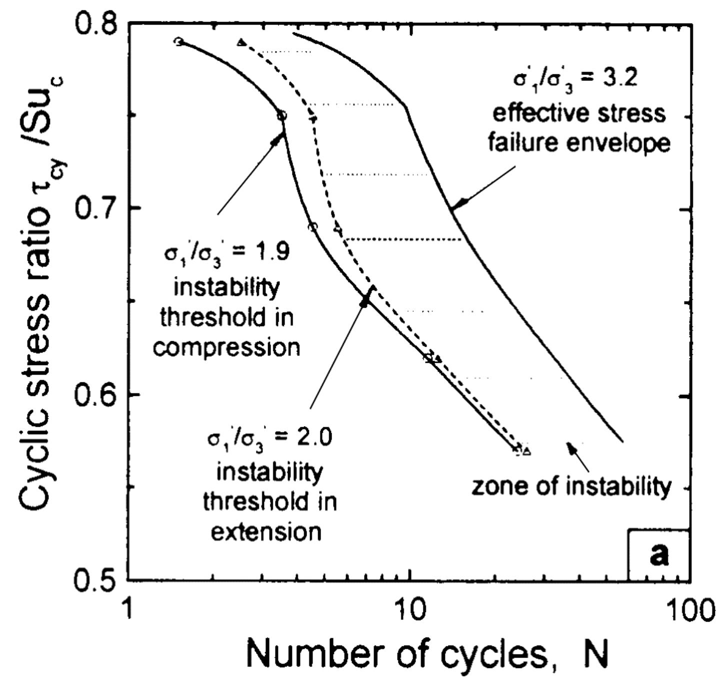
- increased compressibility
- more normalized stress-strain behavior
- suppression of phase-transformation
- increased cyclic shear resistance
- lower peak pore pressure ratios
- increased damping ratios at strains above about 2%.
- increased work dissipation to cause large strains
- increased effect of loading rate



Cyclic Strength of Fine-Grained Soils

Cyclic strength of clays can be normalized by their monotonic undrained shear strengths.

Chinese Criteria are a black/white descriptor of a continuous transition in behavior. These criteria should not be overly relied upon or used to avoid detailed in situ and laboratory testing.



Zergoun & Vaid (1994)

Does a soil behave more as a non-plastic silt or a plastic clay? Insight can be gained from a variety of data. Vane shear tests for undrained strengths and sensitivity (be careful for drainage rate). Consolidation tests to see if the soil has a well-defined preconsolidation stress. Undrained shear strength tests at different consolidation stresses to see if properties normalize and whether soils show phase-transformation behavior. Comparing V_s in the lab (bender elements or other) to V_s in the field can be a measure of sample disturbance.

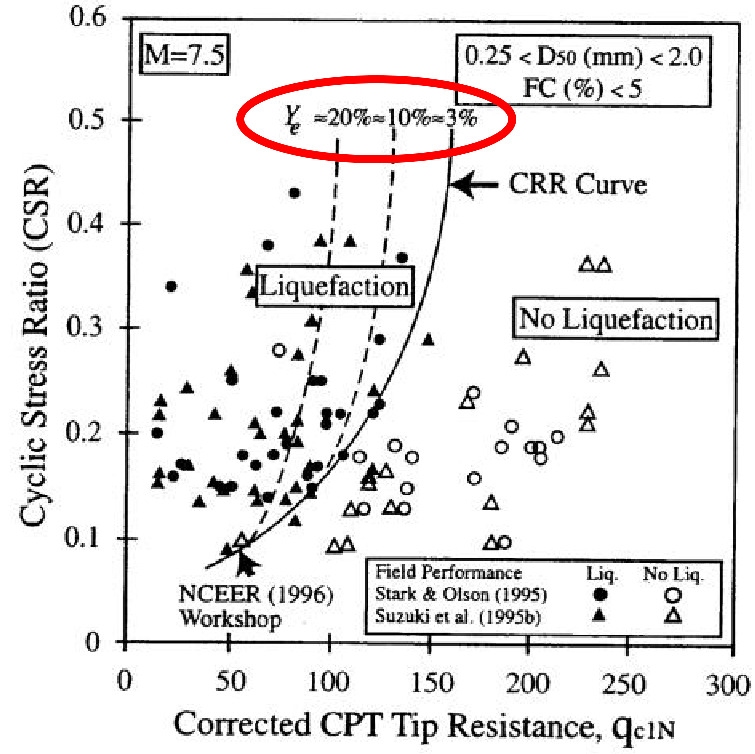
Research on the effects of fines and plasticity on cyclic loading behavior may benefit from:

- calibration chamber tests to get penetration resistances and shear wave velocities for different stress histories.
- cyclic laboratory tests to go with the calibration chamber tests
- centrifuge model studies with in-flight in situ testing prior to earthquake shaking events

Remediation

Treatment methods often produce a heterogeneous mass with zones of varying strength & stiffness.

- What is the composite behavior of such a treated mass?
- Reinforcing effects?
- Drainage effects during and after shaking?
- Time effects over years?



Treated zones are often within a larger body:

- How much to treat?
- What degree of treatment?
- Effect on deformations of entire dam?

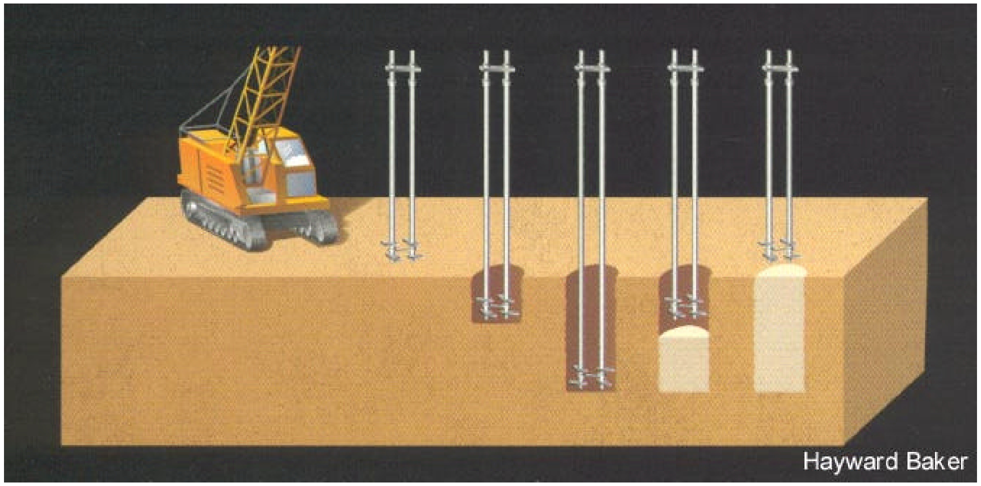
For very strong design motions, treatment may not be able to preclude triggering. If you achieve $N_{1\text{-}60}$ values of 25 or q_{C1N} values of 125, how will it behave if liquefaction (in terms of high excess pore pressures) is still triggered?

Pursue new treatment methods; e.g., passive cementation (Mitchell pursuing).
Improve understanding of existing options; e.g., DSM in-ground walls.

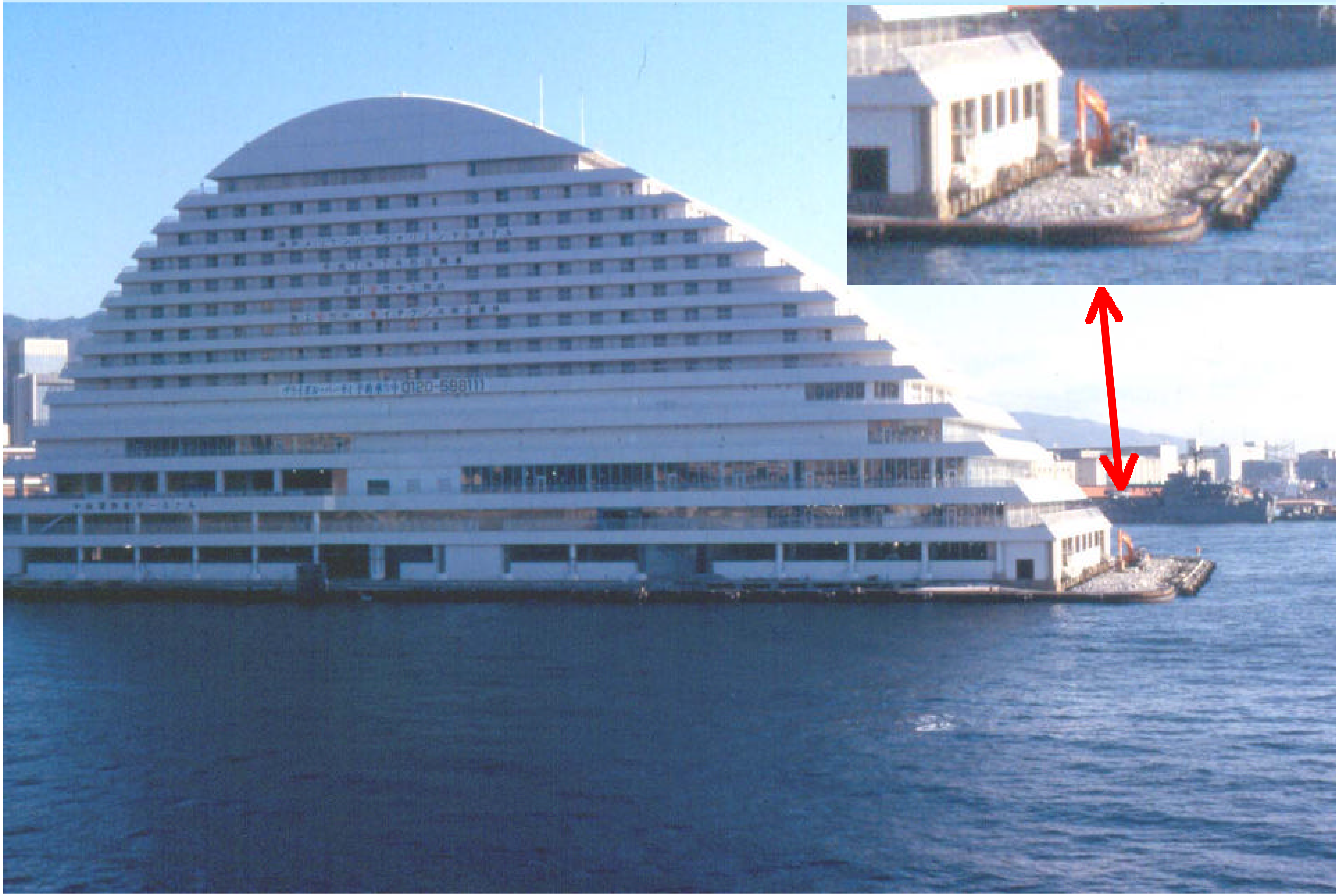
Deep Soil Mixing & In-ground Walls

In-ground walls improve liquefiable deposits by:

- Reducing earthquake-induced shear strains in the treatment zone, thereby limiting pore pressure development.
- Containing the enclosed soil should it liquefy, and thus contributing to the composite strength.
- Acting as a barrier to the migration of excess pore pressures from untreated to treated zones.



Oriental Hotel in Kobe, 1995



<u>Oriental Hotel:</u>

- Extensive liquefaction around the perimeter with deformations of 1 to 2 m in the quay wall & fill.
- No damage to foundation or evidence of liquefaction within the DSM walls (building footprint).

Other limited experiences in Kobe suggest that in-ground walls can be effective in mitigating liquefaction hazards (Hamada & Wakamatsu 1996).

Limited physical modeling studies provide some insight into earthquake behavior of in-ground walls (Babasaki et al. 1991, Suzuki et al. 1991).

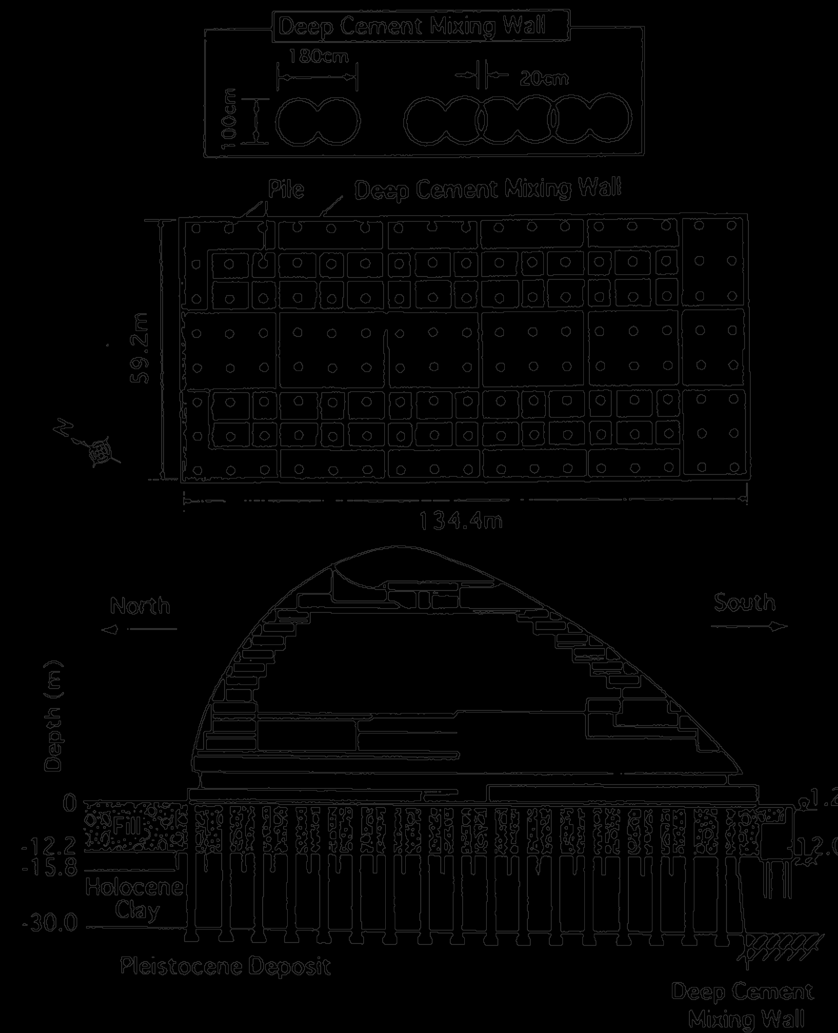
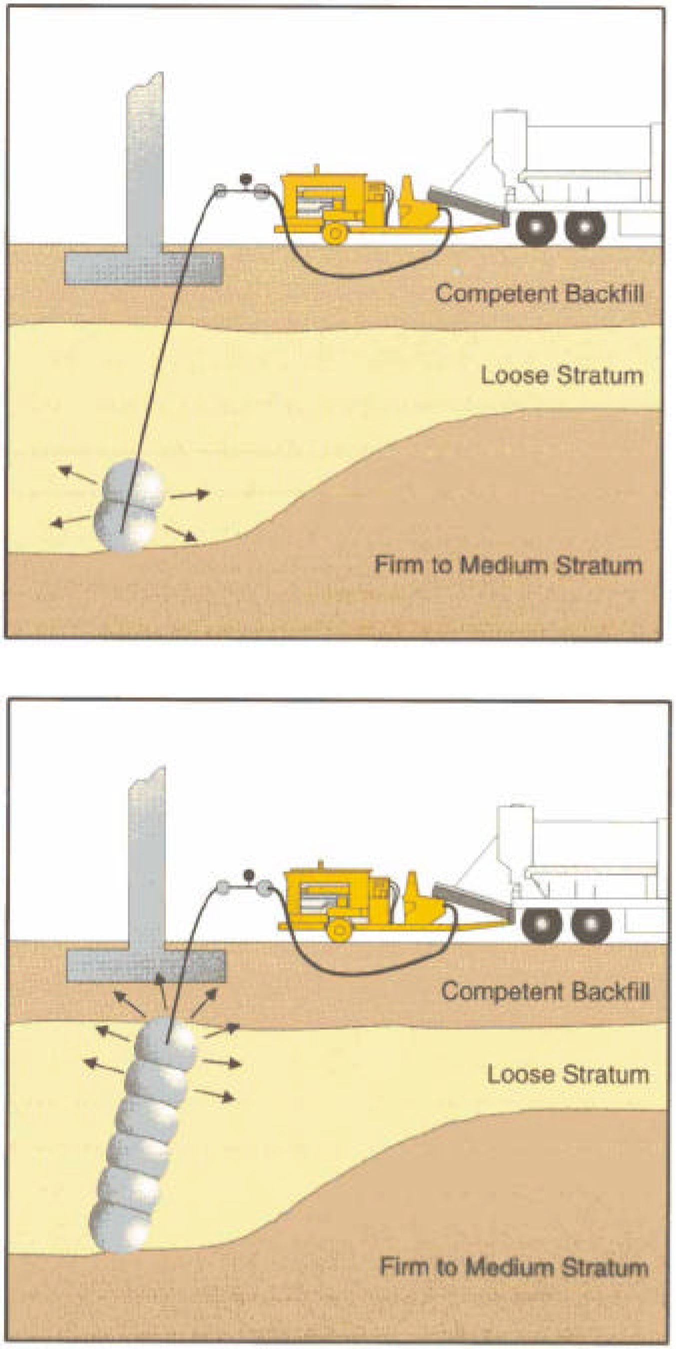


Fig. 3-5. Cross-Section and Plan of Oriental Hotel (Hamada and Wakamatsu 1996)

Compaction Grouting



Can target layers of limited thickness or extent, and work in areas with limited access.

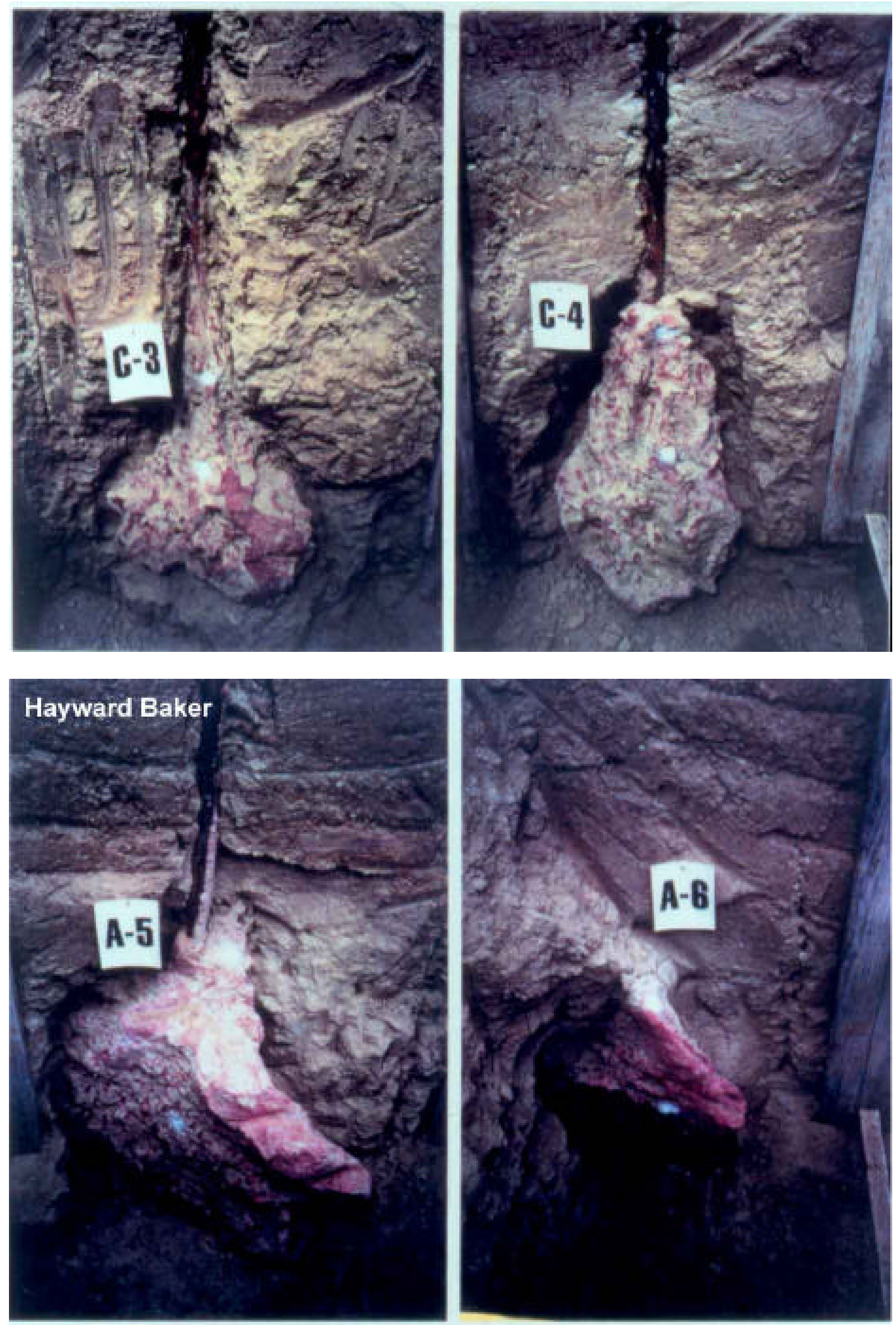
Recent theoretical work by Boulanger & Yu (1997), Mace & Martin (2000), & Jefferies et al. (2000) provide improved understanding of treatment mechanisms:

- relative roles of densification & lateral stress increases
- distribution of density changes

Grout characteristics affect bulb growth and shape, which may affect treatment effectiveness & post-treatment quality control testing.

Grout “takes” to achieve target penetration resistances are generally higher than expected. Main issue is that penetration tests are usually midway between treatment points, where densification is lowest.

Negative time effects observed by Mejia & Boulanger (1995), possibly due to relaxation of lateral stresses.



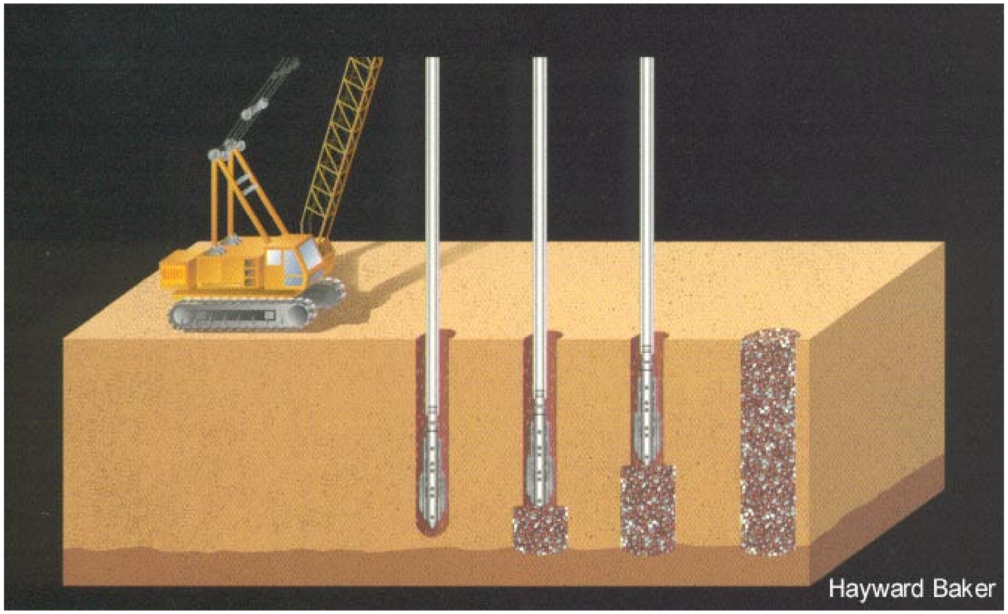
Stone Columns

Several techniques available for constructing vibro-replacement stone columns.

Treatment mechanisms include:

- densification
- lateral stress changes
- reinforcement of soil mass
- improved drainage

Potential benefits of reinforcement & drainage are usually not accounted for. They may or may not offer extra defense against earthquake damage depending on the situation.

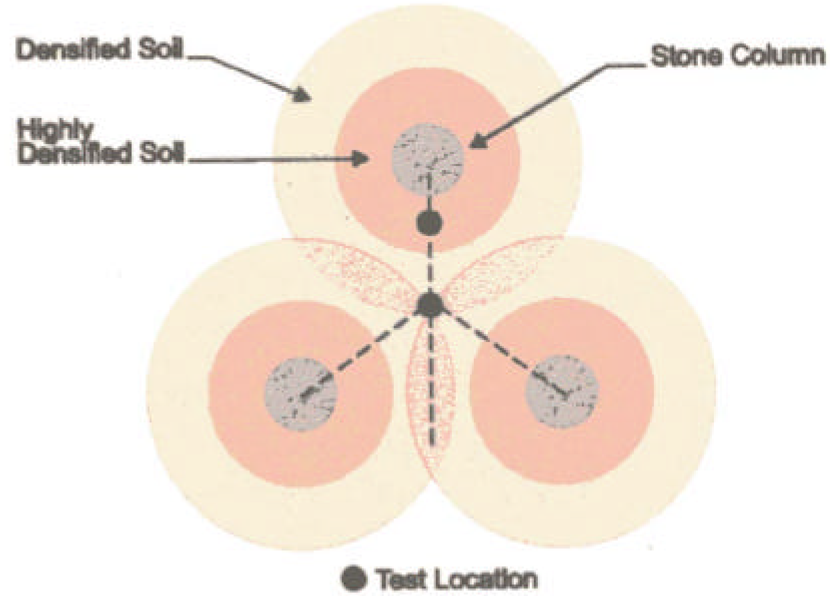


An increases in penetration resistance reflects changes to:

- lateral stress
- density
- fabric
- cementation
- age & stress-strain history

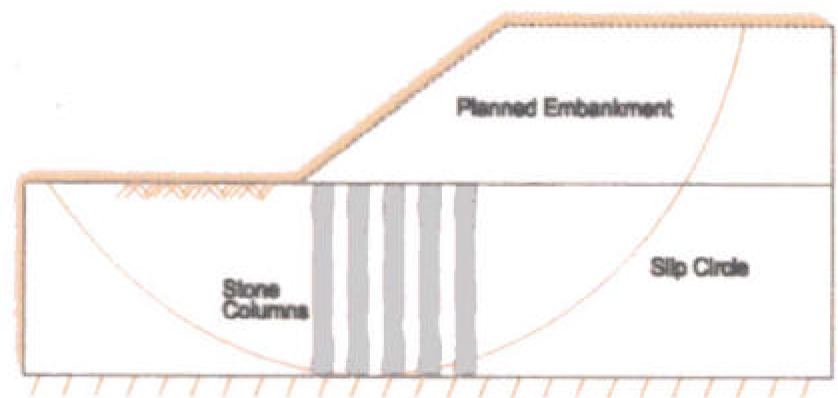
Potential shear strains due to liquefaction depend more on relative density, and so understanding the above contributions is important to the design process. E.g., Salgado et al. (1997) looked at K_o & D_r on q_c.

Understanding the composite behavior of a treated mass affects our choice of quality control procedures.



Recommended test locations for densification evaluation.

Hayward Baker

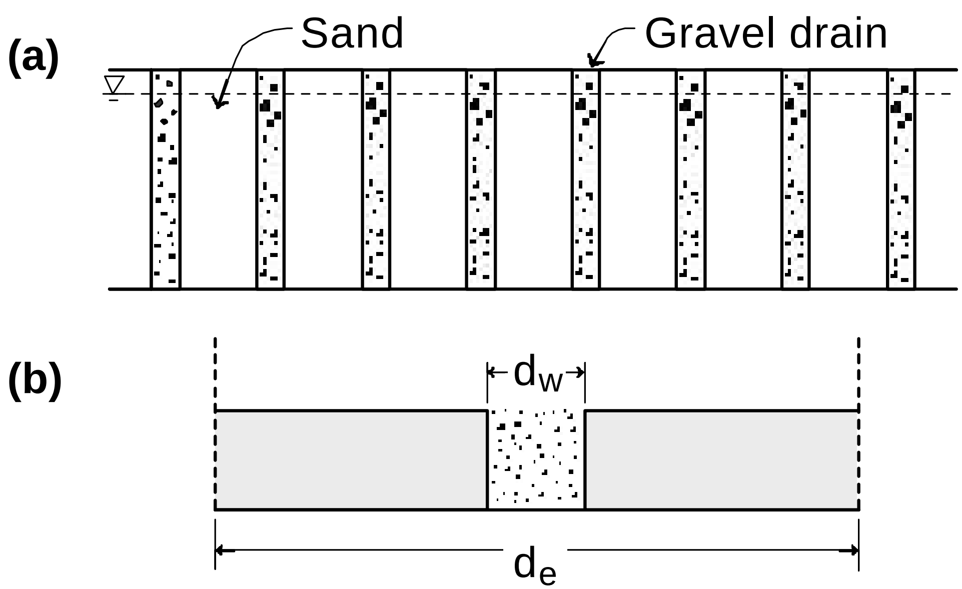


Slope cross section.

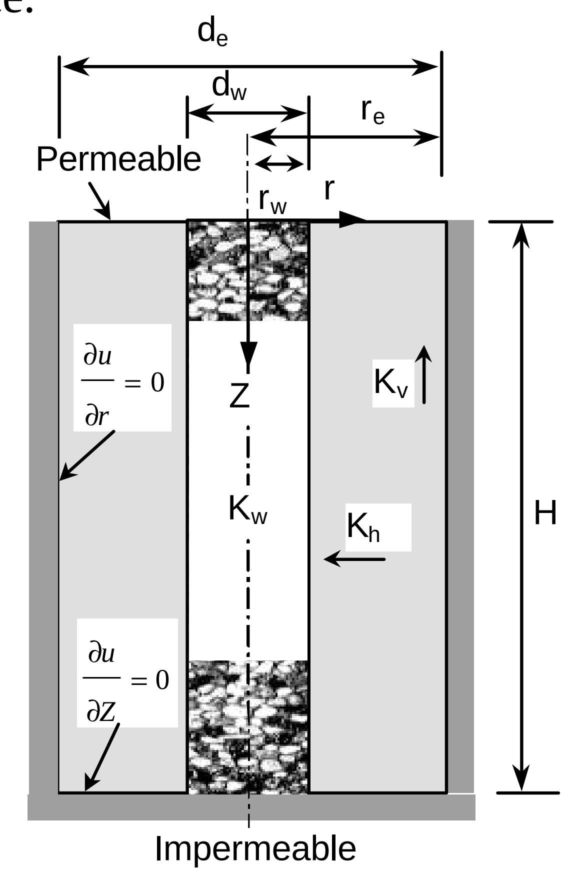
Hayward Baker

Drainage During Shaking

- The drainage capacity of stone columns or gravel drains during earthquake shaking is strongly affected by the drain resistance.
- Design diagrams by Onoue (1988) and Iai and Koizumi (1986) incorporate the effects of drain resistance, while the analyses of Seed and Booker (1977) greatly underestimate the range of drain permeability that adversely affects performance.



Seed & Booker (1977) assumed radial flow only, including flow through the drain to a infinitesimal sink at the drain center.



Onoue (1986) accounted for drain geometry & permeability.

- Intermixing of imported stone & native soil occurs during vibro-replacement for the purpose of densification.
- Intermixing with the native soil greatly reduces the permeability of the column. Low-permeability intervals due to construction defects also greatly increase the drain resistance.
- Densification has the advantage that potential deformations are reduced even if liquefaction is triggered.
- Drainage alone has the disadvantage that potential deformations are not reduced if liquefaction is triggered.
- Drainage may still serve as an additional line of defense against potential deformations in some cases. For example, drainage may reduce the potential for void redistribution and its detrimental effects on shear strength.

Naesgaard (1998)

Earthquake Performance of Liquefiable Sites Treated by Vibro- or Drain-Techniques (Boulanger et al. 1997)

Table 1. Summary of Case Histories

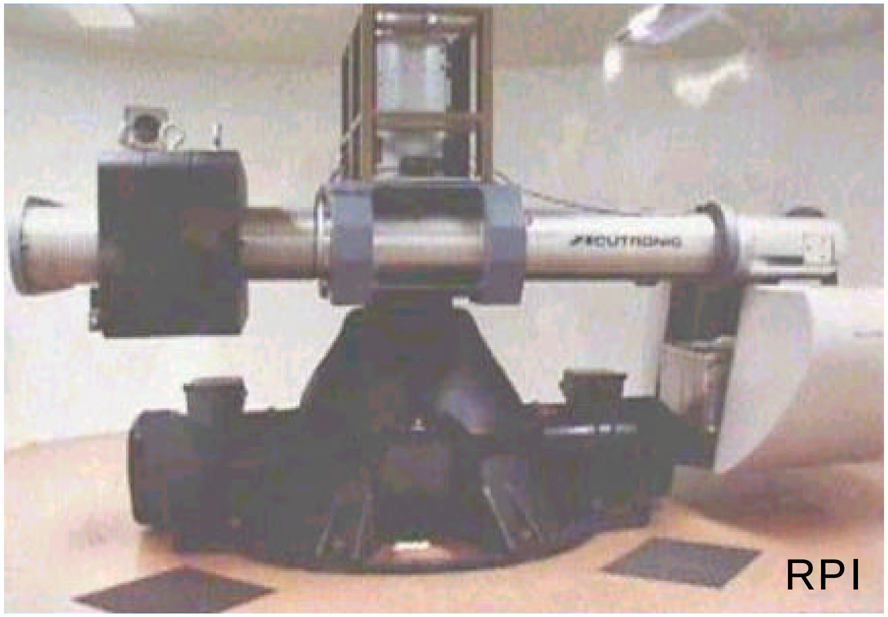
No. (1)	Site (2)	Location (3)	Method of treatment[b] (4)	Earthquake event (5)	Peak accel. (6)	Damage (7)
1	Nippon Oil Co.	Niigata	Vibroflotation	1964 Niigata	0.16 g	None; Minor
2	NTT building	Niigata	Vibroflotation	1964 Niigata	0.16 g	S_{max}≈0.5 m
3	Paper plant: (i) Group I (ii) Group II	Hachinohe	Vibroflotation	1968 Tokachioki	0.225 g	(i) None. (ii) S_{max}≈0.4 m
4	Group of oil tanks	Ishinomaki Port	Sand compaction piles	1978 Miyagiken-oki	0.18 g[a]	None
5	Med/Dental clinic	Treasure Island, CA	Vibroreplacement stone columns	1989 Loma Prieta	0.16 g	None
6	Building 450	Treasure Island, CA	Sand compaction piles	1989 Loma Prieta	0.16 g	None
7	Facilities 487-489	Treasure Island, CA	Vibrocompaction (vibroflotation)	1989 Loma Prieta	0.16 g	Minor cracking in floor of bldg. 487.
8	Approach to Pier 1	Treasure Island, CA	Vibroreplacement stone columns	1989 Loma Prieta	0.16 g	None
9	Wharves (6 locations)	Port of Kushiro	Gravel drains	1993 Kushiro-Oki	0.47 g	None, ranging to S_{max}≈20-40 mm
10	Jensen Filtration Plant	Northridge, CA	Sand drains	1994 Northridge	0.98 g	Cracks to 80 mm, offsets to 200 mm.
11	Warehouses (5 buildings)	Port Island, Kobe	Vibro-rod	1995 Hyogo-Ken Nanbu	0.34 g[a]	None, ranging to offsets of 100 mm.
12	Amusement park	Port Island, Kobe	Vibro-rod	1995 Hyogo-Ken Nanbu	0.34 g[a]	None; some cracks to 25 mm and ejecta along south side.
13	Small building	Port Island, Kobe	Vibro-rod	1995 Hyogo-Ken Nanbu	0.34 g[a]	S_{diff}≈150 mm beside building.
14	Rubble mound breakwater	Nishinomiya area	Sand compaction piles	1995 Hyogo-Ken Nanbu		S_{max}≈1-2 m.

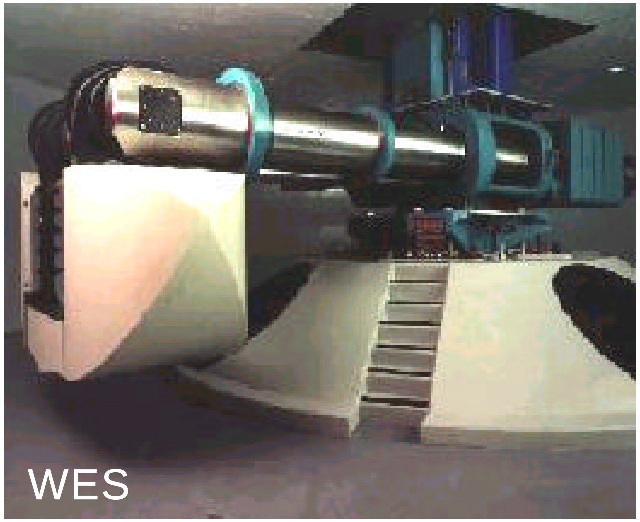
Physical Modeling

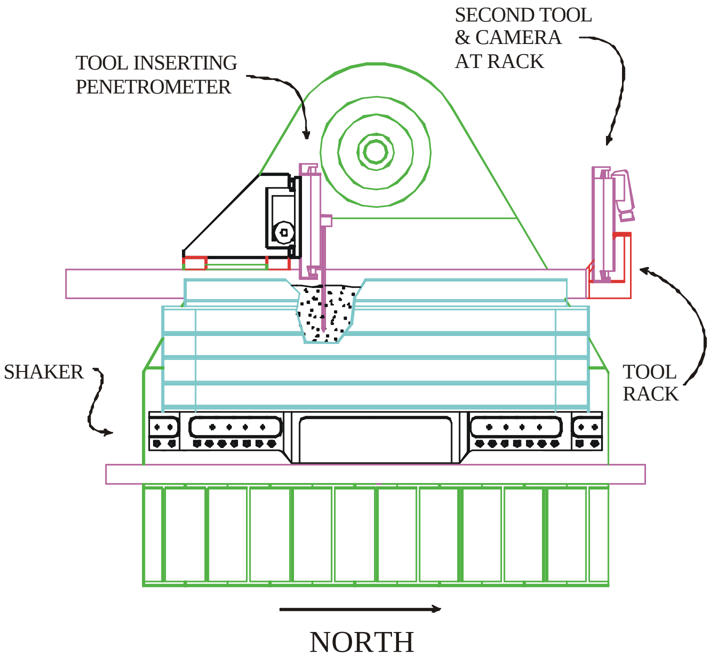
Improving representation of prototype processes:
- in situ testing (e.g., CPT, V_S, V_P)
- construction methods (e.g., vibroflotation)
- model preparation

Improving instrumentation: smaller, remote, more detailed.

NEES funding for RPI & UC Davis will produce major advances in capabilities.

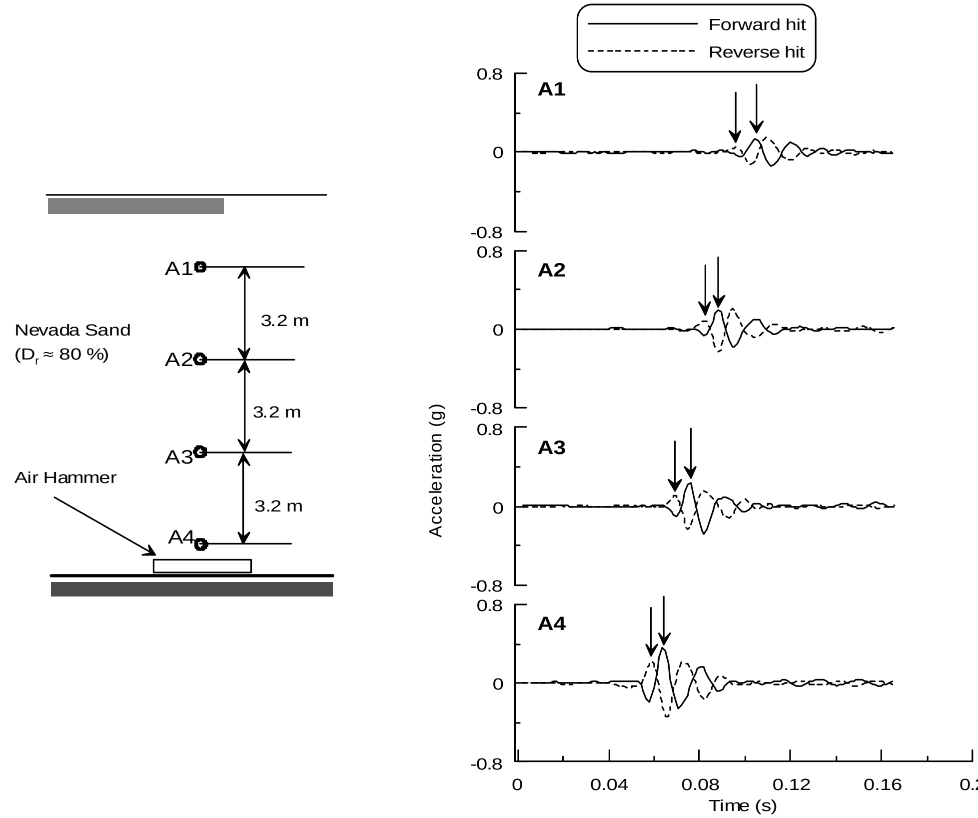






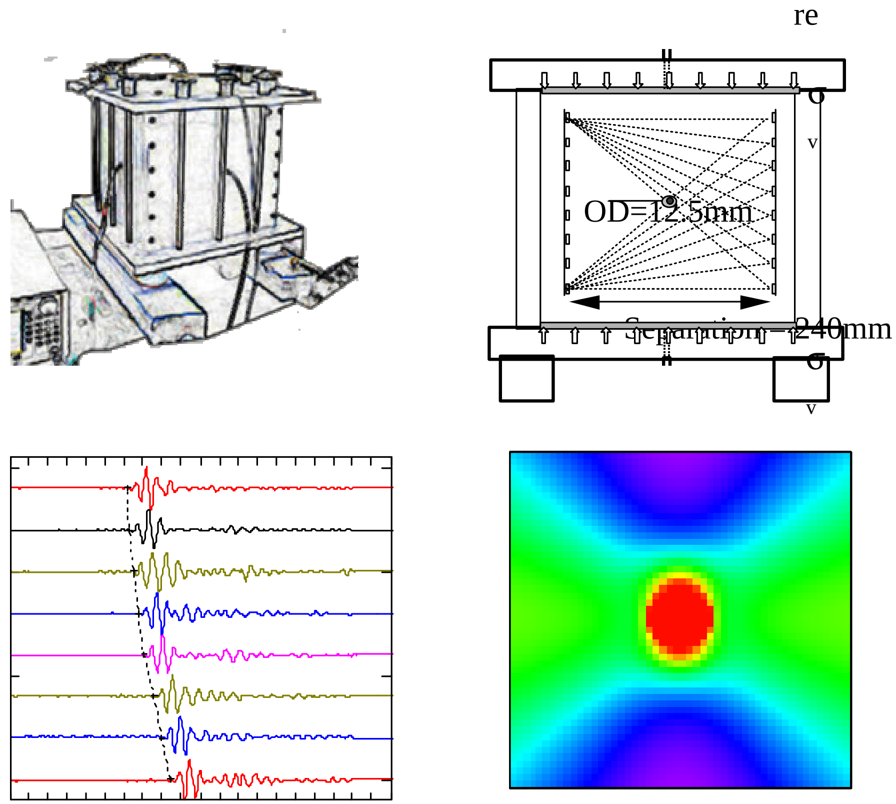
NEES funding for remote robot to perform in-flight:

- CPT testing
- pile driving
- vibroflotation
- deep soil mixing
- grouting
- dynamic compaction
- anchor installation



In-flight V_S Measurements on Nevada Sand ($D_r \approx 80\%$) at 80g (Arulnathan et al. 2000)

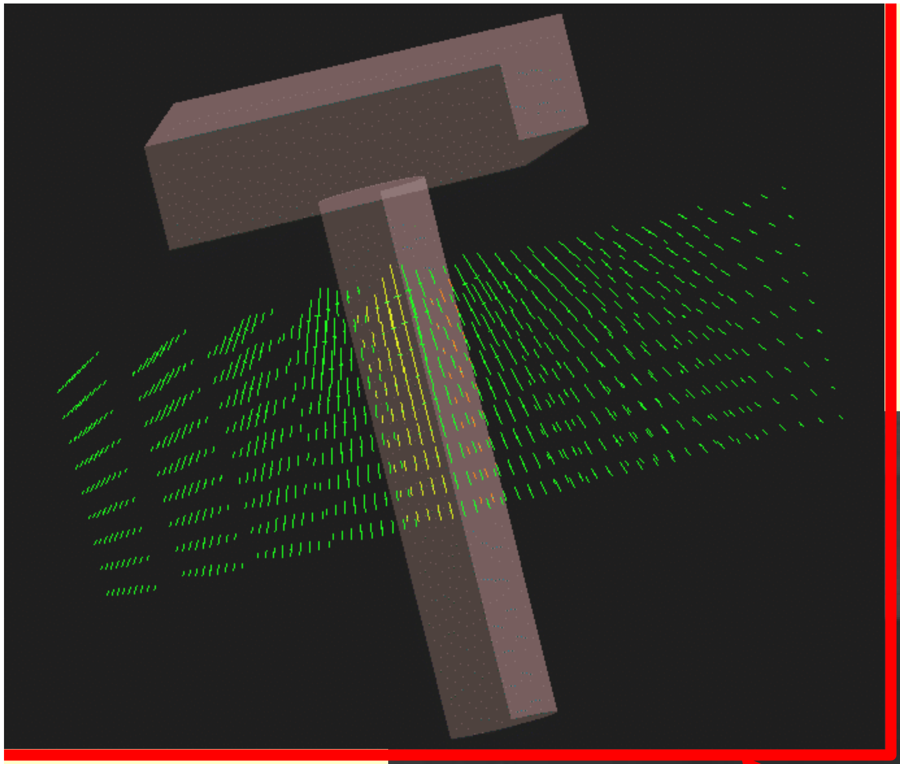
In-flight CPT from RPI



Subsurface Imaging (Santamarino et al. 2000)

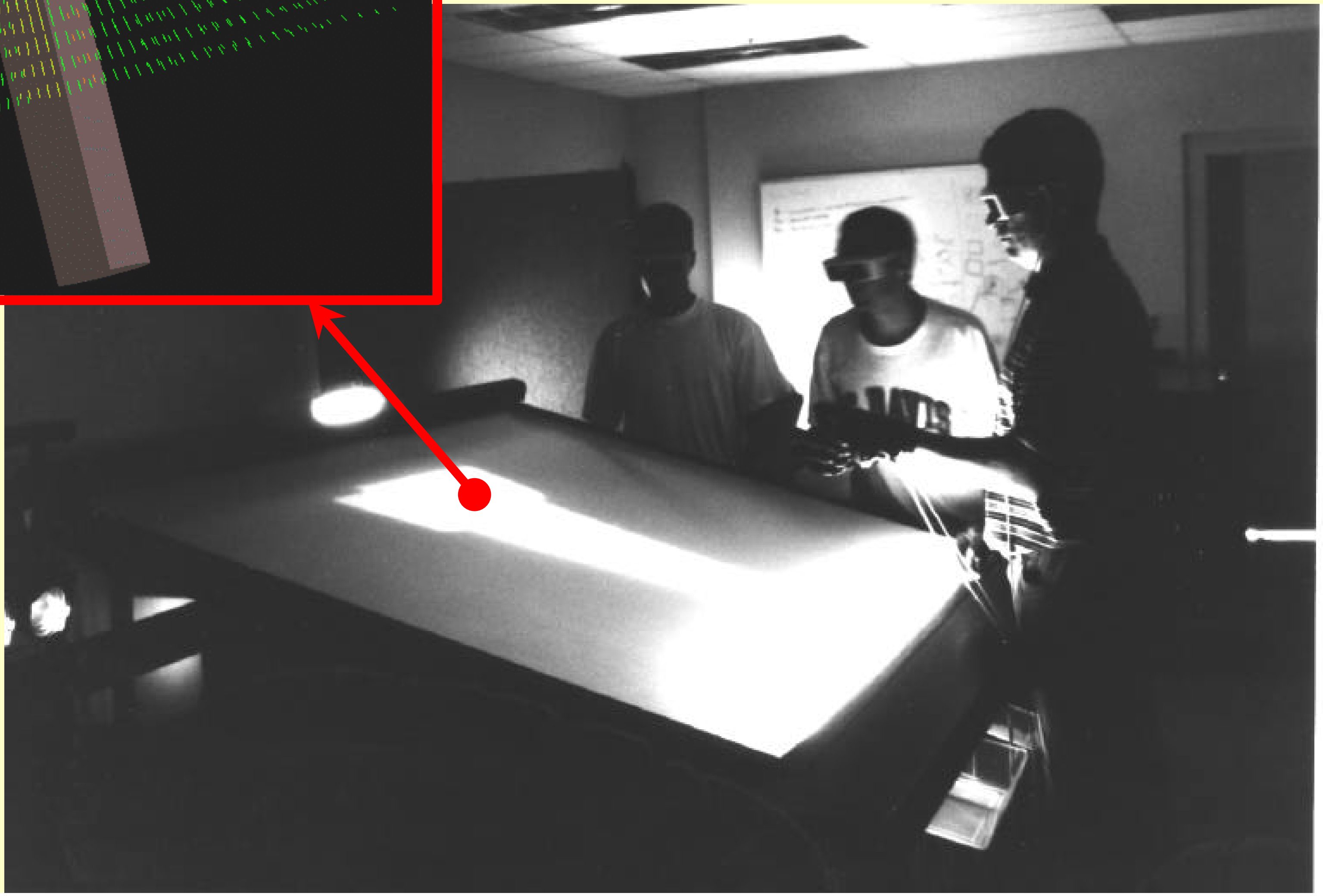
Numerical Modeling

- Nonlinear numerical modeling has become increasingly common, with most using relatively simple constitutive models. Can provide valuable insight into complex behavior.
- The analyses are only as good as the model parameter selections, which depend on the quality of the site characterization and in situ test data.
- Need careful checks on the numerical results, including checks against simpler analyses (e.g., limit equilibrium analyses) and examination of all measures of response (e.g., pore pressures, stresses, deformations, accelerations, spectra).
- Need improved constitutive models that are easily calibrated to design parameters and relations.



Visualization of Complex Systems:

- Physical model or FEM results
- Rendering tables can improve our comprehension of complex time-varying behavior.



Thank you.