Utah Liquefaction Advisory Group

February 4, 2013 Salt Lake City, Utah



The Utah Liquefaction Advisory Group is funded through a cooperative agreement between the Utah Geological Survey and U.S. Geological Survey, National Earthquake Hazards Reduction Program

Meeting agendas, summaries, and presentation files may be found on the Utah Geological Survey ULAG web page: http://geology.utah.gov/ghp/workgroups/ulag.htm

Products resulting from ULAG-related research may be found on the University of Utah ULAG web page: http://www.civil.utah.edu/~bartlett/ULAG/

Utah Liquefaction Advisory Group (ULAG)



Progress Report on Liquefaction Working Group

February 15, 2011 Salt Lake City, Utah

Steven F. Bartlett, Ph.D., P.E. Associate Professor University of Utah

ULAG Members and Participants





Members

Steve Bartlett, UU CE, Facilitator

Mike Hylland, UGS liaison

Richard Briggs, USGS

Les Youd, BYU CE

Kyle Rollins, BYU CE

Kevin Franke, BYU CE

Jim Bay, USU CEE

John Rice, USU CEE

Loren Anderson, USU CEE

David Simon, SBI

Grant Gummow, UDOT

Jim Higbee, UDOT

Travis Gerber, URS

Bill Turner, Earthtec

Ryan Cole, Gerhart-Cole

Objective 1

Develop Probabilistic Liquefaction Hazard Maps for Urban Counties in Utah

Salt Lake County

Utah County

Davis County

Weber County

Cache County

Objective 1 (cont.)

Types of Maps

- (1) Liquefaction Triggering Maps
- (2) Lateral Spread Displacement Hazard Maps
- (3) Liquefaction-Induced Ground Settlement Maps

Objective 2

Develop ARC GIS Programs for Implementing Probabilistic Mapping Procedures for Other Regions in U.S.

- Strong ground motion hazard estimates from PSHA and National Strong Motion Mapping Program
- User methods based on ArcGIS algorithms

Objective 3

Establish and Populate a Subsurface Geotechnical Database for Public Use

- Geotechnical Evaluations
- Land Use Planning
- Research
- Potential Partners
 - •UDOT
 - Salt Lake County and Cities

Objective 4

Education and Public Outreach

- User Friendly Maps
- Assist Counties in Implementation and Ordinances
- Outreach Seminars and Website

Status Previous Work

FY 2004

- Geotechnical Database (N. Salt Lake Co.)
- M7.0 lateral spread displacement hazard map (N. Salt Lake Co.) published in *Earthquake Spectra*.

FY 2005

• Geotechnical Database (S. Salt Lake Co.)

Status Previous Work

FY 2006

2.1.1	
Task 1: Development of CPT and SPT correlations (University of Utah)7	2.1.1 Do
2.1.2 Task 2: Correlation of Subsurface Geologic and Geotechnical ArcGIS [™] Database with Surficial	
Geologic Mapping (Utah Geological Survey)	2.1.2 Do
2.1.3 Task 3: Mapped mean annual probability of triggering liquefaction for southern Salt Lake County	213D
(University of Utah)	2.1.0 D
2.1.4 Task 4: Mapped probability of triggering liquefaction for a scenario earthquake for Salt Lake	2.1.4 D
County (University of Utah)	215D
2.1.5 Task 5: Mapped mean annual probability of lateral spread exceeding displacement thresholds of	2.1.5 D
0.1, 0.3 and 1.0 meters for northern Salt Lake County (University of Utah)	2.1.6 D
2.1.6 Task 6: Mapped lateral spread horizontal displacement for a scenario event for northern Salt Lake	
County (University of Utah)	2.1.7 D
2.1.7 Task 7: Synthesis report of seismically induced ground displacement in Salt Lake County	218D
(University of Utah, Simon-Bymaster, Inc., and Utah Geological Survey)	2.1.0 D
2.1.8 Task 8: CPT subsurface investigations in downtown Salt Lake City (University of Utah and	2.1.9 D
ConeTech)	
2.1.9 Task 9: Map production and report delivery (University of Utah and Utah Geological Survey)12	

Downtown Displacement Investigations



HORIZONTAL DISTANCE ALONG EXPLORATION LINE (M) (Measured From the Southeast Corner of 500W and 400S)

Possible Extension of the Warm
 Approximate CPT Sounding Locations

Status Previous Work

FY 2007

2.1.1 Done2.1.2 Done2.1.3 Done2.1.4 Done

FY 2008 (No Funding) FY 2009 (No Funding) FY 2010 (No Funding)

FY 2010 (Partial Funding from WBWCD) FY 2011 (UGS –Funding)

Probabilistic liquefaction potential map – (2002 Input)



Probabilistic liquefaction potential maps for 2500 and 500-year return periods



M 7.0 Lateral spread displacement map 15 percent change of exceedance



Probabilistic ground settlement maps for 2500 and 500-year return periods



M 7.0 ground settlement map 15 percent change of exceedance



Weber County Liquefaction Hazard Mapping



Figure 5.12. 50th percentile probabilities of lateral spread displacement exceeding 0.3 meters for a 2,500-year seismic event; Weber County, Utah

FEMA Project (U of U and UGS)

- Develop a new model ordinance for liquefaction hazards based on input and feedback from municipalities, technical advisory groups, and others.
- Educate various municipalities and their stake holders regarding risk-based decision making and hazard mitigation using the newly developed hazard ordinance that is coupled with the recently developed ULAG liquefaction hazard maps and support and encourage the implementation/adoption of the new liquefaction hazard ordinance in the various municipalities along the urban Wasatch Front.
- Develop methods to apply the liquefaction hazard maps to assess post-event traffic interruptions resulting from liquefaction-induced damage
- Educate the next generation of Utahans about earthquake hazards by focusing on a secondary education outreach curriculum and program delivered to Salt Lake and Weber Counties.

MAPPING THE PROBABILITY OF LIQUEFACTION-INDUCED GROUND FAILURE

by Dan Gillins, Ph.D., P.L.S.

"Simple calculations based on a range of variables are better than elaborate ones based on limited input."

-Ralph B. Peck

1. Research Project Background

- Liquefaction-induced ground failure can cause significant damage
- Best Defense:
 - 1. Identify areas at risk
 - 2. Establish planning, development, and engineering strategies





Liquefaction Potential Maps in Utah



(from Anderson et al., 1994)

Research Objectives

- A. Develop a hazard mapping method using:
 - 1. Probabilistic seismic hazard data
 - 2. Maps of surficial geology
 - 3. Topography
 - 4. Varying qualities of geotechnical data
- B. Method should output maps that:
 - Predict the quantity of ground displacement from lateral spread
 - 6. Estimate the uncertainty of these predictions

New Liquefaction Hazard Maps in Salt Lake County



Lateral spread displacement hazard based on magnitude 7 earthquake and 85% non-exceedance probability threshold



Location of compiled geotechnical investigations in Salt Lake County (from Hinckley, 2010)

Geology & Geotechnical Database



Available data in Weber County:

251 SPT boreholes
157 CPT soundings
21 Vs tests

Youd et al. (2002) Empirical Model

$Log D_{H} = \frac{b_{o} + b_{off} \alpha + b_{1}M + b_{2}Log R^{*} + b_{3}R + b_{4}Log W + b_{5}Log S + b_{6}Log T_{15} + b_{7}Log(100 - F_{15}) + b_{8}Log(D50_{15} + 0.1 \text{ mm})$

- Seismic Factors
 - *M*, *R*
- Topographic Factors *W*, S
- Geotechnical Factors
 - T₁₅, F₁₅, D50₁₅



Free-face ratio: W (%) = H / L * 100

New Empirical Model

$$Log D_{H} = \frac{b_{o} + b_{off} \alpha + b_{1}M + b_{2}Log R^{*} + b_{3}R + b_{4}Log W + b_{5}Log S + b_{6}Log T_{15} + a_{1}x_{1} + a_{2}x_{2} + a_{3}x_{3} + a_{4}x_{4} + a_{5}x_{5}$$

 x_i = the ratio of T_{15} in a borehole that has a soil index (SI) equal to *i*

Soil Index <i>(SI)</i>	Typical Soil Description in Case History Database	General USCS Symbol
1	Silty gravel, fine gravel	GM
2	Coarse sand, sand and gravel	GM-SP
3	Medium to fine sand, sand with some silt	SP-SM
4	Fine to very fine sand, silty sand	SM
5	Low plasticity silt, sandy silt	ML
6	Clay (not liquefiable)	CL-CH

Comparing the Models

Model	R² (%)	MSE	σ _{logDH}	P-Value
Full: Youd et al. (2002)	83.6	0.0388	0.1970	0.000
Reduced: no <i>F</i> ₁₅ or <i>D50</i> ₁₅	66.6	0.0785	0.2802	0.000
New: with soil type terms	80.0	0.0476	0.2182	0.000



 $\frac{1}{Log D_{H}} = \frac{b_{o} + b_{off} \alpha + b_{1}M + b_{2}LogR * + b_{3}R + b_{4}LogW + b_{5}LogS + b_{6}LogT_{15,cs} + a_{3}}{b_{6}LogT_{15,cs} + a_{3}}$

Estimating x_i Variables with CPT

Zone	Soil Behaviour Type (SBT)
1	Sensitive fine-grained
2	Clay - organic soil
3	Clays: clay to silty clay
4	Silt mixtures: clayey silt & silty clay
5	Sand mixtures: silty sand to sandy silt
6	Sands: clean sands to silty sands
7	Dense sand to gravelly sand
8	Stiff sand to clayey sand*
9	Stiff fine-grained*
	* Overconsolidated or cemented

$$I_{c} = [(3.47 - LogQ_{tn})^{2} + (LogF_{r} + 1.22)^{2}]^{0.5}$$

Robertson (1990) Soil Behavior Type Chart Boundaries of each zone estimated by circles with radius = I_c



Histograms of I_c for each SI



Charts to Estimate SI given I_c



Recommended normal probability density functions; Weber County



Point estimation chart; Weber County

Example 1

Find probability that: SI = 1 (i.e., fine gravel) given $I_c = 1.5$



 $P(SI = 1 | I_c = 1.5) = 1.92 / (1.92 + 0.82 + 0.27) = 0.63$

Example 1 (cont.)

$$P(SI = i | I_c = 1.5):$$

i	Р
1. Fine Gravels	0.63
3. Clean Sands	0.27
4. Silty Sands	0.09
5. Sandy Silts	0.00
6. Clays	0.00



Estimating $N_{1,60}$ from CPT Data


Critical Dataset Distributions



Histograms of Qal₁; Weber County, Utah

Critical Dataset Distributions



MANOVA of Geologic Units



Influence of Age



Time Since Initial Deposition or Last Critical Disturbance. t (years)

(from Hayati and Andrus 2009)

Spatial Dependence



Semivariogram of $Log(T_{15,cs})$

Example of Weighting Scheme for Interpolation

Topographic Variations



Contours Based on Digital Elevation Model (DEM) from USGS National Elevation Dataset



Percent ground slope: S (%)



Free-face ratio: *W* (%) = *H* / *L* * 100

Seismic Inputs

- Mean seismic variables from interactive deaggragation of the seismic hazard
- Seismic hazard based on 2008 source and attenuation models of the National Seismic Hazard Mapping Project (Peterson et al., 2008)



Solving a Grid Point

- 1. Extract cell values from raster data:
 - Geologic deposit, age
 - Site classification
 - Percent ground slope and elevation
- 2. Compute free-face ratio (W)
- 3. Input mean seismic variables
- 4. Compute weights to each SPT/CPT in corresponding geologic deposit
 - Based on semivariogram

Solving a Grid Point (cont.)

- 5. Randomly select data from an SPT/CPT
- 6. Model results from empirical models as normally distributed random variables
- 7. Solve probability chain $P[D_H > y]$
- 8. Repeat to define a distribution of outcomes

Monte Carlo Method

- Repeated random sampling to compute results
- Used when:
 - Unable to compute results deterministically
 - Systems have many degrees of freedom
 - Modeling phenomena with large uncertainty



The Normal Distribution

Liquefaction Triggering Maps



Median probabilities of P_L , 500year seismic event



Median probabilities of P_L , 2,500year seismic event

Lateral Spread Hazard Maps



Median probabilities of exceeding 0.3 m, 500-year event



84th percentile probabilities, of exceeding 0.3 m, 500-year event

Lateral Spread Hazard Maps



Median probabilities of exceeding 0.3 m, 2,500-year event



84th percentile probabilities, of exceeding 0.3 m, 2,500-year event

Lateral Spread Hazard Maps



Median probabilities of exceeding 0.6 m, 2,500-year event



84th percentile probabilities, of exceeding 0.6 m, 2,500-year event

Major Contributions

- 1. New method to map the probability of liquefaction-induced ground failure
 - Predicts the probability of horizontal displacement exceeding specified thresholds
 - Estimates the uncertainty of these predictions
 - Method uses:
 - A new empirical model for lateral spread analysis
 - State-of-the-art liquefaction triggering analyses
 - Strong ground motion estimates from the current NSHMP
 - Surficial geologic maps at the 7.5-minute
 - DEMs from the National Elevation Dataset
 - Available SPT, CPT, and V_s data

Major Contributions (cont.)

The new mapping method accounts for:

- Changes in topography
- Influence of the age of the geologic deposit
- Proximity to a geotechnical investigation
- Major sources of uncertainty
- 2. Development of a new empirical model for probabilistic lateral spread analysis
 - Uses data routinely collected in the field and reported on the borehole logs
 - Enables use of CPT data

Major Contributions (cont.)

- 3. New liquefaction hazard maps for Weber County, Utah
 - Identifies zones with high probability of large horizontal displacements from lateral spread
 - Can be used to establish planning, development, and engineering strategies
- Site classification map for Weber County, Utah
 - Classifies the site soil response according to definitions of NEHRP

Got Risk? Some Advantages of Performance-Based Design in Evaluating Liquefaction and Its

Effects

Contraction of the second seco

Kevin W. Franke, Ph.D., P.E. Assistant Professor Department of Civil and Environmental Engineering Brigham Young University, Provo, Utah

2013 Utah Liquefaction Advisory Group (ULAG) Meeting Salt Lake City, Utah, USA February 4, 2013



Performance-Based Design

Selectively looking at particular aspects of a problem (e.g. scenariobased design) may give us an idea of the hazards that we are dealing with...

But looking at the <u>whole</u> picture and <u>all</u> <u>the possibilities</u> (e.g. performance-based design) gives us a more complete and consistent perception of those hazards.



Traditional Approach to Liquefaction Assessment



Disadvantages of Traditional

Liquefaction Analysis Approach

- Often ignores uncertainty, which can be substantial
- Usually incorporates pseudo-probabilistic methods (i.e. probabilistic ground motions are used in a deterministic manner)
- Considers liquefaction initiation in a binary sense
- Typically quite subjective and inconsistent
- Considers *possibility* of hazard, but tells you nothing about *likelihood*



4

Performance-Based Liquefaction Assessment – A Uniform Hazard Approach

- Kramer and Mayfield (2007) introduced a performancebased approach
 - Uses probabilistic ground motions in a <u>probabilistic</u> manner
 - Accounts for uncertainty in soil parameters, seismic loading <u>AND</u> the liquefaction triggering model
 - Produces liquefaction hazard curves for each sublayer in the soil profile

$$\Lambda_{FS_{L}} = \sum_{j=1}^{N_{M}} \sum_{i=1}^{N_{a_{\max}}} P \Big[FS_{L} < FS_{L}^{*} \mid a_{\max_{i}}, m_{j} \Big] \Delta \lambda_{a_{\max_{i}}, m_{j}}$$
$$\lambda_{N_{req}^{*}} = \sum_{j=1}^{N_{M}} \sum_{i=1}^{N_{a_{\max}}} P \Big[N_{req} < N_{req}^{*} \mid a_{\max_{i}}, m_{j} \Big] \Delta \lambda_{a_{\max_{i}}, m_{j}}$$





Advantages of Performance-Based

Liquefaction Analysis Approach

- Accounts for all possible sources of uncertainty
- Evaluates liquefaction and its effects in terms of probability and uniform hazard
- Shown to be more consistent across different seismic environments
- Decisions are based on acceptable hazard and tend to be less subjective
- Results are typically compatible with higherorder risk-based analyses (e.g. evaluating losses conditional upon liquefaction)





6

Comparison Traditional Approach and Kramer & Mayfield (2007) PBD Model

- Using a generic soil profile, liquefaction potential evaluated in 10 different cities across the US
- Targeted hazard level from PBD model is 7% probability of exceedance in 75 years ($T_R = 1,033$ years)



Comparison Between Traditional Approach and Kramer & Mayfield (2007) PBD Model

• Results:



Conclusion: In terms of actual liquefaction hazard, the traditional approach tends to produce inconsistent results across the US.



Simplified Performance-Based Liquefaction Assessment

- Despite its greater consistency, the Kramer & Mayfield PBD procedure is difficult to perform.
- Mayfield et al. (2010) presented a simplified map-based procedure for liquefaction that targets a single hazard level of interest.
- The simplified procedure is used to develop very close approximations of fully probabilistic liquefaction results at a desired return period, <u>but with only a fraction of the effort</u>.





9



Simplified Performance-Based Liquefaction Assessment

Mayfield et al. (2010) showed that a full performance-based analysis could be performed on a <u>generic reference soil</u> profile to compute N_{req} at a single depth across a grid of multiple geographic locations, to develop a <u>Liquefaction Parameter Map</u> for N_{req}^{ref} .



Simplified Performance-Based Liquefaction Assessment

The mapped value of N_{req}^{ref} is then corrected for site-specific soil conditions and depths to produce the <u>site-specific</u> values of N_{req} , which correspond to the targeted hazard level.

$$N_{req}^{site} = N_{req}^{ref} + \Delta N_{req}$$

where

$$\Delta N_{req} = \Delta N_{\sigma} + \Delta N_F + \Delta N_{r_d}$$

 ΔN_{σ} = Stress Correction Factor ΔN_{F} = Soil Amplification Correction Factor $\Delta N_{r_{d}}$ = Depth Reduction Correction Factor



Comparison Between Full and Simplified PBD Procedures

Does the simplified PBD procedure work? <u>You bet!</u>! Below are the comparisons with the same generic soil profile and 10 US cities:



Probabilistic Liquefaction Hazard Maps vs. Liquefaction Parameter Maps

Liquefaction parameter maps (LPMs) ARE NOT the same as probabilistic liquefaction hazard maps:

- LPMs are developed from a fictional soil profile
- LPMs are intended <u>solely</u> for the site-specific, performance-based assessment of liquefaction
- When combined with site-specific soil boring information, LPMs result in probabilistic liquefaction triggering profiles
- Existing LPMs just focus on liquefaction initiation; research is under way to develop similar procedures for various liquefaction effects
- When LPMs are combined with hazard maps, an engineer has multiple tools to assess liquefaction hazard and to judge accordingly



Conclusions

- Performance-Based Design (PBD) methods account for all sources of uncertainty and produce results in terms of uniform hazard and likelihoods.
- Traditional "pseudo-probabilistic" methods for evaluating liquefaction are often subjective and inconsistent.
- Simplified, user-friendly PBD liquefaction methods that are compatible with current seismic codes have been created and are currently being expanded/improved.
- Liquefaction parameter maps are different than probabilistic liquefaction hazard maps, <u>but they compliment one another</u>.



Got Risk? Some Advantages of Performance-Based Design in Evaluating Liquefaction and Its

THANK YOU!!

Effects



2013 Utah Liquefaction Advisory Group (ULAG) Meeting Salt Lake City, Utah, USA February 4, 2013



Additional Things to Think About.....

 Mayfield et al. (2010) incorporates the Cetin et al. (2004) probability of liquefaction model. Similar full and simplified PBD procedures are currently being developed for other probabilistic liquefaction models including Boulanger & Idriss





US-PRC cooperative research in geotechnical engineering

T. Leslie Youd Professor Emeritus Department of Civil and Environmental Engineering Brigham Young University, Provo, Utah, USA

Part 1: Liquefaction during 2008 Wenchuan earthquake, consequent damage, and cooperative research

Cooperative Research on Liquefaction between Institute of Engineering Mechanics (IEM), Harbin, China and Brigham Young University (BYU), Provo, Utah, USA

➢In May 2010, Dr Cao Zhenzhong, IEM, arrived at BYU as a visiting scholar for joint study of liquefaction

➢After the 2008 Wenchuan earthquake, Dr Cao spent five months in the field locating, documenting, and field testing sites where surface effects of liquefaction were or were not observed

>Dr Cao wrote two papers in Chinese, then translated them into English

➢I then revised the first paper for publication in an English-language technical journal and am now revising the second

Paper No. 1 – Published in 2011 in Soil Dynamics and Earthquake Engineering (Elsevier)

Elsevier Editorial System(tm) for Soil Dynamics and Earthquake Engineering Manuscript Draft – Accepted for publication 7 April 2011 Manuscript Number: SOILDYN-D-10-00263R1 Title: Gravelly soils that liquefied during 2008 Wenchuan, China Earthquake, Ms=8.0 Article Type: Research Paper

Gravelly soils that liquefied during 2008 Wenchuan, China Earthquake, Ms=8.0

Zhenzhong Cao^a*, T. Leslie Youd^b, and Xiaoming Yuan^a

^aInstitute of Engineering Mechanics, China Earthquake Administration, Harbin 150080, China

^bCivil and Environmental Engineering Department, Brigham Young University, Provo 84602, U.S.

*Corresponding author. Tel: +86-451-86652643; Fax: +86-451-86664755, E-mail address: iemczz@163.com (Z. Cao)

Abstract: Field investigations following the 2008 Wenchuan earthquake (Ms = 8.0) identified 118 liquefaction sites nearly all of which are underlain by gravelly sediment in the Chengdu Plain and adjacent Mianyang area. Field studies, including core drilling, dynamic penetration tests (DPT), and multiple channel analysis of surface wave velocity tests (MASW) for measurement of shear wave velocities, reveal the following: (1) Sand boils and ground fissures, indicative of liquefaction, occurred across hundreds of square kilometers affecting 120 villages, 8 schools and 5 factories. (2) The Chengdu plain is underlain by sandy gravels ranging in thickness up to 540 m; loose upper layers within the gravels beds liquefied. (3) Mean grain sizes for gravelly layers that liquefied range from 1 mm to more than 30 mm. (4) Shear wave velocities in gravels that liquefied range up to 250 m/sec. (5) A 50% probability curve, developed from logistic procedures, correctly bounds all but four data points for the 47 compiled Vs data.


Map of liquefaction sites compiled by Dr. Cao and IEM team following 2008 Wenchuan earthquake



Building at Banqiao School, Mianzhu, pulled apart at foundation level by lateral spread toward an adjacent incised river; note fissures and sand boil ejecta in foreground. Differential settlements as great as 20 cm also occurred within the structure; due to earthquake damage, the building was demolished and replaced

2008/05/14

Typical fissure and sand boil at liquefaction site in a farmer's field; gravelly composition of ejecta indicates layer that liquefied was gravelly. Cavity was eroded by water and sediment eruption with velocities sufficient to carry gravel particles to ground surface

Paper No. 2 – Accepted for publication ASCE Journal of Geotechnical and Geoenvironmental Engineering

Dynamic penetration test (DPT) for evaluation of liquefaction resistance of gravels

Zhenzhong Cao, T. Leslie Youd and Xiaoming Yuan

➢ Few engineers in the western world are familiar with the Chinese dynamic penetration test (DPT)

➤The DPT, however, has been used in China for more than 40 years in field investigations of gravelly soils for foundation engineering (Code Provsion: DB51/T5026-2001)

➤The DPT was used for the first time to investigate gravelly soils that liquefied after the 2008 Wenchuan earthquake

➤This test could have important application for investigation of gravels in the USA and many other countries



74-mm diameter Chinese DPT tip



Fig. 11 Component sketch of dynamic penetration test (DPT) apparatus



Chinese DPT in operation



Conducting BPT at Echo Dam, Utah, USA with US Bureau of Reclamation drill rig and crew



Site 2 Songbai

Borehole, BPT and shear wave velocity logs from 2008 liquefaction site investigation on Chendu Plain

 $\ln(P_L/(1-P_L)) = -7.54 + 0.30N_{120} - 2.15\ln(CSR)$



Fig. 13. Probabilistic liquefaction triggering curves of gravelly soils

Correlation developed by Dr Cao between DPT penetration resistance and liquefaction resistance of gravelly soils

Advantages of Dynamic Penetration Test:

- (1) DPT equipment is simple and robust
- (2) The test is inexpensive and easy to conduct; about the same cost as SPT.
- (3) The equipment can be transported in a light-weight truck
- (4) The equipment can be set up and a test finished in one to two hours after arrival at a test site.
- (5) Verified correlations between DPT blows and design parameter for foundations have been developed from data collected in China [DB51/T5026-2001].
- (6) Continuous penetration data are obtained with depth; soil layers as thin as 10 cm can be defined.
- (7) The cone tip is easily driven through loose to dense gravelly to cobbly soils; refusal may occur in very dense layers or in layers with large cobbles and boulders.

There is need in USA for an economic, robust test to measure penetration resistance of gravels for liquefaction hazard evaluation:

➢For liquefaction resistance of sands and fine grained sediments, standard penetration tests (SPT) or cone penetration tests (CPT) are widely used

➢ Because of interference by large particles, SPT and CPT tests are not viable in gravelly soils

➢The Becker Penetration test (BPT) is sometimes used on critical or high-cost projects, such as large buildings, bridges or dams

 \geq Mobilization of BPT equipment to a site, however, is expensive (typically > \$10,000) and results are processed with uncertain correlations, making the results uncertain.

➤The DPT could provide more useful and economical measurements of penetration resistance in gravels

>DPT tests can be conducted with standard drill rigs at no greater cost than SPT or CPT



Becker Penetration test in operation The test is conducted with a pile-hammer to drive a 168mm diameter, closed-end, steel casing into gravelly sediment The penetration resistance is the number of hammer blows required to drive a casing 30 cm (1 ft) ➢ Many factors affect BPT resistance, such as hammer efficiency, bounce-chamber pressure, soil friction on casing, etc

Case Study: Deer Creek Dam, Utah, USA (constructed 1938-1942)



Cross section of Deer Creek Dam and foundation



Foundation materials generally sand, gravel and boulders



Cross section beneath longitudinal axis of dam showing locations of boreholes and BPT soundings used to develop stratigraphy and penetration resistance for assessment of liquefaction hazard



Liquefaction hazard analyses were uncertain, leading US Bureau of Reclamation to construct \$10 M remedial measures to assure safety of downstream population

COLLABORATIVE PROJECT BETWEEN

INSTITUTE OF ENGINEERING MECHANICS

AND

BRIGHAM YOUNG UNIVERSITY

Techniques for Prediction and Mitigation of Liquefaction Hazard (2011-2016)

April 26, 2011

Cooperative research project signed between IEM and BYU

OBJECTIVE OF COLLABORATIVE PROJECT

A collaborative project between Institute of Engineering Mechanics and Brigham Young University is proposed to: (1) conduct common research on liquefaction hazard prediction and mitigation in China and the US, (2) exchange liquefaction prediction techniques and methods for sandy, silty, and gravelly soils between China and US, and (3) facilitate collaboration through scholarly exchanges and inter-institutional enrollment of students subject to approval by the enrolling institution. Any provisions in this agreement may be amended or revised by the two universities after consultation and mutual agreement.

TOPICS FOR COLLABORATIVE RESEARCH

We will actively pursue opportunities for conducting research of mutual interest with funding from US and Chinese sources. Information obtained from joint research will be exchanged freely by both sides and researchers will work to publish results as co-authors on technical papers. Topics for collaborative research project are:

(1) Mapping of liquefaction hazard;

(2) Prediction of lateral spread displacement;

(3) Gravel liquefaction assessment methods (DPT and MASW); highest priority

(4) Behavior of piles in liquefied soil and lateral spreads;

(5) Liquefaction assessment comparison in China and USA and improvement for Chinese seismic code and US practice;

(6) Influence of soil nonlinearity and liquefaction on ground response spectra; Ground motion acceleration/displacement prediction on liquefied sites;

(7) Ground settlement assessment caused by liquefaction;

(8) Axial capacity of deep foundation in gravels.

(9) In-situ and lab geotechnical testing techniques.

US-PRC cooperative research in geotechnical engineering

T. Leslie Youd Professor Emeritus Department of Civil and Environmental Engineering Brigham Young University

Part 2: Sensors and instrumentation needs in geotechnical earthquake engineering



Regional map showing location of WLA

Wildlife Liquefaction Array (WLA)

- WLA was instrumented by US Geological Survey in 1982 (T.L. Youd, Principal Investigator)
- Recorded two earthquakes within 14 hours in November 1987: Elmore Ranch (M=6.2), which did not generate significant pore pressures, and Superstition Hills (M=6.6) which liquefied the site
- WLA was re-instrumented as a field equipment site for the NSF Network for Earthquake Engineering Simulation (NEES) program (T.L Youd, BYU; Jamie Steidl, UCSB; and Bob Nigbor USC; principal investigators



General setting and recent earthquakes that have shaken WLA (Holzer et al., 1989)

Liquefaction Occurrences Near Wildlife Site

Liquefaction Effects observed following six earthquakes in past 82 Years







1950 sand boil that erupted about 1.5 km northwest of WLA Site

1982 USGS WLA instrumentation

- •One accelerometer placed at ground surface
- •A second accelerometer was placed immediately below the liquefiable layer
- •Five piezometers placed in the liquefiable layer and one in a deeper layer



View of Wildlife site morning after 1987 Superstition Hills Earthquake with sand boils in foreground and instrument hut in background. (USGS photo)





Acceleration and Pore Pressure records generated during the 1987 Superstition Hills earthquake



WLA Site Response – 1987 Superstition Hills earthquake



Reason for continued rise of pore water pressure: Although strong ground accelerations ceased at about 23 sec, ground displacements continued to rise with maximum of 22 cm (peak to peak) at about 35 sec. Cyclic shear strain, as a consequence of ground displacement, generates increased pore water pressures.

Correlation of acceleration and pore water pressure spikes was due to dilatent arrest of ground movement producing a sudden drop of pore pressure and the acceleration spike. Movement then ensued in the opposite direction. These spikes are numbered on the upper plots (Zeghal and Elgamal, 1994)



Stress-Strain Analysis M Zeghal (RPI) A-W Elgamal (RPI) J. Geotech Eng, ASCE, 6/94 Instrumentation, Records & Interpretation T L Holzer (USGS) T L Youd (Brigham Young Univ) T C Hanks (USGS) Science, Vol. 244, pp 56-59, 1989

Research sponsored by: NCEER (USA), NSF (USA), and KAJIMA (Japan)

Analysis by Zeghal and Elgamal (1994)

Shear stresses were calculated from measured ground accelerations and mass of soil above liquefied layer Shear strains were calculated from ground displacements, determined from double integration of acceleration records, and dividing by distance between accelerometers Note initial near-vertical stress strain loops that flattened and develop banana-type loops with time



Predicted and actual response spectra for Superstition Hills earthquake - WLA site



Stream erosion is cutting into bank adjacent to new site generating a free face that should facilitate ground deformation and lateral spread (view looking northwest)



Instruments placed at WLA to develop NEES instrumented field site



WLA site is available for additional instrumentation, experiments, and subsurface investigation with new tools and procedures

Shape Accelerometer Array



- Triaxial measurement of x, v, and a at 30 cm depth intervals.
- 30 Hz sampling rate.
- Retrievable, reusable.
- Capable of measuring time histories of dynamic accelerations, velocities and displacements
Recovery of Liquefied Sand with Increasing Undrained Shear Strain

by Gary Norris, Mohamed Ashour, Tung Nguyen, Horng-Jyh Yang and Sherif Elfass

> Presented by Gary Norris, Ph.D., P.E. University of Nevada, Reno



Example from Seed Demonstraing Gain in Deviatoric Stress and Decrease in Excess Pwp, Hence Gain in Effective Confining Pressure

Effect of Cyclic Loading to Liquefaction upon Subsequent Undrained Stress-Strain Relationship for Sacramento River Sand (Dr = 40%) (Seed 1979)



Mohr Circles for Drained Triaxial Compression Tests



Drained Axial Compression Test Stress-Strain Curves



Normalized Vertical Axis (i.e. SL)



and Normalized Horizontal Axis (i.e. $\varepsilon_1/\varepsilon_{50}$) with Appropriate Fitting Equation



Approximating Axial Strain at 50% SL for Sand

Uniformity Coefficient, C_u





Creating SL vs ε_1 Curves with Equation (Given $\varepsilon_{50,ref}$)



From SL vs ε_1 , Convert SL to σ_d







Interrelationships Among (a) Drained and Undrained Stress-Strain Behavior (b) Isotropic Consolidation Rebound, and (c) Effective Stress Path

Hooke's Law in Terms of Effective Stress

$$\varepsilon_1 = \frac{\sigma'_1}{E} - \nu \frac{\sigma'_2}{E} - \nu \frac{\sigma'_3}{E}$$

$$\varepsilon_2 = -\nu \frac{\sigma'_1}{E} + \frac{\sigma'_2}{E} - \nu \frac{\sigma'_3}{E}$$

$$\varepsilon_3 = -\nu \frac{\sigma'_1}{E} - \nu \frac{\sigma'_2}{E} + \frac{\sigma'_3}{E}$$

$$\varepsilon_{\nu} = \frac{(1-2\nu)}{E} \left(\sigma'_1 + \sigma'_2 + \sigma'_3\right)$$



 $\sigma'_1 = \sigma'_3 + \sigma_{d1}$

 $\sigma'_2 = \sigma'_3 + \sigma_{d2}$

σ'₁=σ'₃+σ_{d1}

σ'3

Modified Hooke's Law

$$\varepsilon_1 = \frac{\sigma_{d_1}}{E} - \nu \frac{\sigma_{d_2}}{E} + (1 - 2\nu^*) \frac{\Delta \sigma_3}{E^*}$$

$$\varepsilon_2 = -\nu \frac{\sigma_{d_1}}{E} + \frac{\sigma_{d_2}}{E} + (1 - 2\nu^*) \frac{\Delta \sigma_3'}{E^*}$$

$$\varepsilon_3 = -\nu \frac{\sigma_{d1}}{E} - \nu \frac{\sigma_{d2}}{E} + (1 - 2\nu^*) \frac{\Delta \sigma_3}{E^*}$$

$$\varepsilon_{v} = (1 - 2v) \frac{\sigma_{d_{1}}}{E} + (1 - 2v) \frac{\sigma_{d_{2}}}{E} + 3(1 - 2v^{*}) \frac{\Delta \sigma_{3}'}{E^{*}}$$

 $\varepsilon_v = 0$ in undrained behavior

 $\sigma'_2 = \sigma'_3 + \sigma_{d2}$

Volume Change due to Shear and Isotropic Consolidation/Rebound

$$\varepsilon_{v} = (1 - 2v) \frac{\sigma_{d1}}{E} + (1 - 2v) \frac{\sigma_{d2}}{E} + 3(1 - 2v^{*}) \frac{\Delta \sigma_{3}}{E^{*}}$$

$$\varepsilon_{\text{v,shear}} = (1 - 2\nu) \frac{\sigma_{d1}}{E} + (1 - 2\nu) \frac{\sigma_{d2}}{E}$$

$$\mathcal{E}_{v,iso} = 3(1 - 2\nu^*) \frac{\Delta \sigma_3'}{E^*}$$

$$\varepsilon_{v} = \varepsilon_{v,shear} + \varepsilon_{v,isc}$$

$$\mathcal{E}_{v,shear} = -\mathcal{E}_{v,iso}$$

undrained



Interrelationships Among (a) Drained and Undrained Stress-Strain Behavior (b) Isotropic Consolidation Rebound, and (c) Effective Stress Path







Isotropic Consolidation and Rebound Response

Loose Nevada Sand

Drained Axial Compression Stress-Strain and Volume Change Response of Isotropically Rebounded Samples



Observed vs Constructed Stress-Strain and ESP (Triaxial Compression) Response of Loose Nevada Sand





Undrained Confining Pressure Change

$\Delta \sigma_{3} = \Delta \sigma_{d} = -\sigma_{d}$ + $\Delta \sigma_{d} = \sigma_{d}$

Undrained Lateral

Compression

 $\Delta u_b = \Delta \sigma_3$ $\Delta \sigma_3' = \Delta \sigma_3 - \Delta u_b = 0$ No change in ES of saturated soil

Axial Compression

Lateral Compression



Undrained Lateral Compression/ Axial Extension ($\sigma_{d2}=\sigma_{d1}$, $e_v=0$) from Drained Lateral Compression Tests ($\Delta\sigma_3$ '=0, $\sigma_{d2}=\sigma_{d1}$ each test)



Superposed Undrained Axial Extension/ Lateral Compression and Undrained Axial Compression ESP's



Undrained Cyclic Loading





Developing Liquefaction $(r_u < 1)$ Under Cyclic Loading

Full Liquefaction $(r_u = 1)$ Under Free-Field (ff) Cyclic Loading



Developing Liquefaction (r_u < 1) Under Cyclic Loading Fully Liquefied Soil ($r_u = 1$)

Example of Its Application



Undrained Soil Resistance at Different Depths



$$\sigma_{vo}' = \sigma_{3c}'$$
 originally, followed by $u_{xs,ff} = r_u \sigma_{vo}'$ from cyclic loading
 $\sigma_{v}' = \sigma_{3}' = \sigma_{3c}' - u_{xs,ff} = (1 - r_u) \sigma_{vo}'$ Effect of free-field PWP
 $\sigma_{3'} = (\sigma_{3c}' - u_{xs,ff}) - u_{xs,nf}$ Effect of free- and near-field PWP







Observed vs. Predicted Responses



Example of Its Application



Undrained Soil Resistance at Different Depths







Pile Deflection, y

LATERAL SPREAD

Amount of Lateral Soil Displacement

Mobilized Driving Lateral Forces Acting on Piles and Generated by Crust Layer(s)

Varying Strength of Liquefied Soil(s)



Soil Flow Around



Bartlett and Youd, 1995



Stress-Strain Behavior of Fully Liquefied Sand



Т



Developing Liquefaction ($r_u < 1$) Fully Liquefied Soil ($r_u = 1$) Under Cyclic Loading


Subsequent undrained stress-strain behavior of sand that has experienced developing or complete liquefaction







Example from Seed Demonstraing Gain in Deviatoric Stress and Decrease in Excess Pwp, Hence Gain in Effective Confining Pressure

Effect of Cyclic Loading to Liquefaction upon Subsequent Undrained Stress-Strain Relationship for Sacramento River Sand (Dr = 40%) (Seed 1979)



Post-cyclic undrained stress-strain behavior of completely liquefied Fraser sand



Compression Test after Isotropically Consolidated Undrained Cyclic Test on Loose Nevada Sand (ctest38)



Demonstraing Gain in Deviatoric Stress

Effect of Cyclic Loading to Liquefaction upon Subsequent Undrained Stress-Strain Relationship for Sacramento River Sand (Dr = 40%) (Seed 1979)





Demonstraing Decrease in Excess Pwp

Effect of Cyclic Loading to Liquefaction upon Subsequent Undrained Stress-Strain Relationship for Sacramento River Sand (Dr = 40%) (Seed 1979)









Figure 9-8: Isotropic Consolidation and Rebound Response with Extrapolation to Small Rebounded Pressure, Series 1, Loose Nevada Sand







Drained Stress - Strain Curves with Constructed Post Liquefaction Response, Loose Nevada Sand





Constructed Points from Drained Tests vs Recorded Stress-Strain Curves from Loose Nevada Sand

Simple Calculation of Response Based on Fundamental Understanding

 $\sigma_d = SL \sigma_{df}$ but SL=1 because movement up the failure line $\sigma_{df} = \sigma_3' [\tan^2(45 + \phi/2) - 1]$ $\varepsilon_1 = \varepsilon_{1f}$ but $\varepsilon_{1f} = x \varepsilon_{50}$ x = 19 for std triax = 30.5 for frictionless cap and base tests = 29.57 from theoretical σ_{3.ref}' $\varepsilon_{50} = \varepsilon_{50c} (\sigma_3' / \sigma_{3c})^n$ n = 0.6 in rebound 850c where $\varepsilon_{50c} = \varepsilon_{50,ref} (\sigma_{3c}/\sigma_{3,ref}')^n$ n = 0.2 in consol Assume increasing σ_3' calculate $\sigma_d \& \varepsilon_1$ from above σ_3 is limited by cavitation

Axial Strain at 50% SL for Sand



Uniformity Coefficient, C_u





Stress Path from Tests vs Constructed from Drained Tests vs Calculated



Calculated Response Added to Previous Stress-Strain Response



Conclusion

There is now a method for assessing the stress-strain and ESP of a sand recovering from complete liquefaction via its suppressed dilatantcy with imposed monotonic loading Thank You for Your Attention

Likewise, given $\varepsilon = SL e^{3.707SL} \varepsilon_{50} / \lambda$ $= (\sigma_d/\sigma_{df}) e^{3.707 \text{SL}} \varepsilon_{50}/\lambda$ $E = \sigma_d / \varepsilon = \sigma_{df} e^{-3.707 \text{SL}} \lambda / \varepsilon_{50}$ $= A\sigma_{3}' e^{-3.707 \text{SL}} \lambda / \varepsilon_{50}$ At SL = 0, E = $E_i = A\sigma_3' 3.19/\epsilon_{50}$ $E/E_i = e^{-3.707SL}(\lambda/3.19) : E_i = 6.38 E_{50}$ G = E/2 (1+v); v varies with SL $G_i = E_i/2 (1+v_i)$ $G/G_0 = e^{-3.707 \text{SL}} (\lambda/3.19) [(1+\nu)/(1+\nu_i)]$ $\gamma = \varepsilon (1+v)$

Equation Yields Young's and Shear Modulus Reduction Curves



New Relationship for Volume Change Curve

 ϕ_m is directly related to $d\epsilon_v/d\epsilon_1$

$$\frac{d\varepsilon_{v}}{d\varepsilon_{1}} = 1 - \frac{\phi_{m}}{\phi_{PT}}$$

$$\varepsilon_{v} = \int_{0}^{\varepsilon_{1}} \left(\frac{d\varepsilon_{v}}{d\varepsilon_{1}}\right)_{m} d\varepsilon_{1}$$

used to construct volume change curve



$$\varepsilon_{v} = \int \left(\frac{d\varepsilon_{v}}{d\varepsilon_{1}}\right) d\varepsilon_{1}$$
$$\frac{d\varepsilon_{v}}{d\varepsilon_{1}} = 1 - \frac{\phi_{m}}{\phi_{PT}}$$

$$\varepsilon_{v} = \int d\varepsilon_{1} - \frac{1}{\phi_{PT}} \int \phi_{m} d\varepsilon_{1}$$

$$\varepsilon_{v} = \varepsilon_{1} - \frac{1}{\phi_{PT}} \int \phi_{m} d\varepsilon_{1}$$









Implication of

$$\left(\frac{d\varepsilon_{v}}{d\varepsilon_{1}}\right) = 1 - \frac{\phi_{m}}{\phi_{PT}}$$

$$d\varepsilon_{v} = d\varepsilon_{1} + 2d\varepsilon_{3}$$

$$\frac{d\varepsilon_{v}}{d\varepsilon_{1}} = 1 + 2\frac{d\varepsilon_{3}}{d\varepsilon_{1}}$$

$$\left(\frac{d\varepsilon_{v}}{d\varepsilon_{1}}\right) = 1 + 2(-v_{tan})$$

$$1 - \frac{\phi_{m}}{\phi_{PT}} = 1 + 2(-v_{tan})$$

$$v_{tan} = \frac{1}{2}\frac{\phi_{m}}{\phi_{PT}}$$

$$v_{sec} = \frac{1}{2}\frac{\phi_{m,av}}{\phi_{PT}}$$

-Poisson Ratio



£1



Undrained Behavior from Drained SL vs ε_1 Response





Relationship of a) drained stress level (SL), b) decreasing confining pressure (σ'_3) that leads to c) the undrained peak resistance

Unload – Reload Relationships

 Comparison of Unload – reload relationship (Ramberg-Osgood) parameters to enable prediction of grouted shaft behavior

•
$$\Delta \rho_{u/r} = \rho_{ref} LL (1 + \beta_{u/r} LL)$$

Where $\rho_{ref} = Q_{tf} / M_i$; $M_i = 12,000$
 $LL = Q_t / Q_{tf}$

$$Q_t$$
 - Top reload force
 Q_{tf} – from model backbone curve
 $\beta_{u/r} = 22.7 \beta_{u/r} / \phi + 8.9394$ yields straight
line fit (R² = 0.98) of $\beta_{u/r}$ vs. $\beta_{u/r} / \phi$;

hence

$$\beta_{u/r} = 8.9394/(1-22.7/\phi)_{Super Pile 2012}$$
A Look Inside the Debate Over EERI Monograph MNO 12



T. LESLIE YOUD, PHD NAE, DIST M ASCE, HON M EERI PROFESSOR EMERITUS CIVIL AND ENVIRONMENTAL ENGINEERING BRIGHAM YOUNG UNIVERSITY PROVO, UTAH

PRESENTED TO:

CALIFORNIA GEOTECHNICAL ENGINEERS ASSOCIATION APRIL 15, 2011

> UTAH LIQUEFACTION ADVISORY GROUP FEBRUARY 4, 2013

Development of Procedures for Evaluation of Liquefaction Resistance

➢Following 1964 earthquakes in Alaska and Niigata, Japan ,the "simplified procedure" was developed by Seed and Idriss (1971) for evaluating seismic demand and liquefaction resistance of granular soils

The cyclic stress ratio, CSR, , the demand function, is calculated as: $CSR = 0.65 (amax/g)(\sigma_v/\sigma'_v)r_d$

>FS = (capacity/demand) = (CRR/CSR_{M=7.5})•MSF•K σ •K α

≻Initially, extraction of samples from liquefiable layers for laboratory cyclic shear testing appeared to be a viable technique for measurement of capacity of soils to resist liquefaction

≻Had laboratory testing been successful, a more orderly and theoretically rigorous procedure for evaluating liquefaction resistance would likely have developed

≻However, disturbance to granular soils during sampling, transport and sample preparation proved to be so severe that useful test results were not attainable

≻Thus, empirical procedures developed as the state-of-the art for liquefaction resistance estimation

➢ Professor H. Bolton Seed, his students and colleagues led the development of empirical procedures using standard penetration resistance (SPT N-values) as the key soil index property to which CRR was correlated

>There was some controversy over this approach, but Professor Seed had the stature and an unusual capacity to understand issues and select viable techniques so that development of empirical procedures based on SPT was generally orderly and proved to be viable.

➢ Professor Seed continued research on the empirical approach and carefully reviewed the work of others. He then wrote a paper about every five years to update the simplified procedure, the last being in 1985.

➢After Professor Seed's death in 1989, researchers and practitioners continued development of empirical techniques, including correlations with other index parameters such as cone penetration test (CPT) resistance, shear wave velocity, energy relations, etc

Seeing a need to update the state-of-the-art, I proposed to NCEER a workshop of 20 experts to review proposed changes and additions to the simplified procedure and to write a consensus update paper that practitioners and others could use with confidence (Youd and Idriss 1997;Youd et al 2001)

≻By the end of the workshop endeavor in 1997, however, Professor Idriss was proposing changes to the simplified procedure, including a revised r_d parameter; he pushed hard for the workshop to incorporate the revised r_d parameter into the workshop summary paper, but the participants resisted on the basis that the r_d he proposed had not been adequately vetted by the geotechnical profession.

Similarly, Professor Ray Seed was beginning a major overhaul of the simplified procedure by reevaluating case histories and analyzing the data statistically

≻Other researchers were also suggesting changes

≻Chaos was beginning to develop

Inherent Chaos In Empirical Procedure Development

➤The present state-of-the-art for liquefaction hazard assessment relies heavily upon empirical procedures, which are based on analyses of collected case histories and performance assessments

>Development of empirical procedures usually occurs through research and analyses by individual investigators; these investigators do not always (or seldom) agree; thus development of empirical procedures tends to be a messy and often chaotic process; disputes are common and to be expected

➢ With respect to liquefaction, many viable views and studied opinions about liquefaction have been proposed, including those by Professors Idriss and Boulanger and by those Cetin, Seed, et al.

➢ Formulations of alternative viewpoints are important to the development process in that they bring into consideration a breadth of interpretations of the available data and allow vetting and selection of the more viable alternatives

>When differing opinions become heated and divisive, however, anxiety develops within the practicing community who must select and apply a procedure, but are unsure which alternative is best or which is an acceptable standard of practice

≻This chaos is the present state of liquefaction affairs as the profession anxiously awaits a calming influence that will restore harmony and confidence in acceptable procedures

➢With that said, let us examine the differences between the post 2001 procedures proposed by Idriss and Boulanger (EERI MNO 12) and by Cetin, Seed et al (2004) and the consequences of those differences

≻In this presentation, I will primarily discuss SPT based procedures, but much of the dialogue is applicable to CPT based procedures as well

≻I will frequently refer to the following four publications:

SOIL LIQUEFACTION DURING EARTHQUAKES



SOIL LIQUEFACTION DURING EARTHQUAKES

by

I. M. IDRISS

Department of Civil and Environmental Engineering, University of California at Davis

and

R. W. BOULANGER

Department of Civil and Environmental Engineering, University of California at Davis

This monograph was sponsored by the Earthquake Engineering Research Institute with support from the Federal Emergency Management Agency



EARTHQUAKE ENGINEERING RESEARCH INSTITUTE MNO-12

I. M. IDRISS R. W. BOULANGER

EE RI

EARTHQUAKE ENGINEERING RESEARCH INSTITUTE

Technical Review and Comments:

2008 EERI Monograph

"SOIL LIQUEFACTION DURING EARTHQUAKES"

(by I.M. Idriss and R.W. Boulanger)

by

Raymond B. Seed

April 2010

Geotechnical Report No. UCB/GT - 2010/01

University of California at Berkeley

This publication (available from UC Berkeley) spells out Ray Seed's criticisms of EERI MNO 12 and provides additional supporting data and information for the Cetin, Seed et al procedure



This publication (available from UC Davis) is a follow up to EERI MNO 12 and provides updates and additional supporting information for the Idriss and Boulanger procedure Practical Recommendations for Evaluation and Mitigation of Soil Liquefaction in Arkansas

Principal Investigator and Graduate Student:

Dr. Brady R. Cox

Shawn C. Griffiths

Project Number: MBTC 3017

Date: February 2011

University of Arkansas Department of Civil Engineering MBTC

(Mack-Blackwel Transportation Center, University of Arkansas)

This publication was prepared to evaluate and compare results from application of: Idriss and Boulanger (2008) Cetin, Seed et al. (2004) Youd et al (2001)
Report preparation began before Ray Seed publicized his criticisms of EERI MNO 12

Liquefaction Case History Datasets

A major contribution of Seed, his students and colleagues, is an enlarged and critically examined liquefaction dataset of 201 case histories (95 from H.B. Seed 1984 dataset and 106 new histories (primarily from Japan). Cetin and Seed used this revised dataset in the development of their liquefaction evaluation procedure.

➢ Idriss and Boulanger re-evaluated the Cetin and Seed dataset, using about 160 case histories with little change, deleted about 40 case histories (primarily from Japan) that they felt were suspect, and added about 70 case histories (primarily from Japanese sources) making a total of 230 in their data set

Seed, 2010 statement

In the end, a total of 201 of the highest quality field performance cases were selected for use in the development of the new SPT-based correlation. Ninety-five of these cases had also been used by Seed et al (1984, 1985); the other 32 cases that had been used by Seed et al. were eliminated as we were able to set a higher standard due to the availability of a considerably larger body of candidate data. One hundred and six new cases were added. The 201 selected cases were then subjected to a more detailed back-analysis process than had previously been performed. The best-available method, on a site-specific basis, was used to evaluate CSR within the critical stratum at each site. For 53 of the cases, where a suitably local ground motion recording was available which could be used to generate an appropriate "input" motion, full site-specific (and event-specific) dynamic response analyses were performed. For the rest of the sites (where no local ground motion recording that could be used to create a suitable "input" motion was available), evaluation of CSR's was based on a new set of correlations (r_d values) for prediction of CSR on the basis of (1) ground conditions and (2) strong shaking characteristics.

Cetin, Seed et al probabilistic liquefaction triggering curves



SPT-based, probabilistic triggering curves of Cetin et al. (2004) with plotted field performance; Cetin, Seed at al. recommend use of the $P_L = 15\%$ curve for deterministic analyses (Seed 2010)

Idriss and Boulanger CRR curve



Figure 75. SPT case histories of liquefaction in cohesionless soils

with various fines contents plotted versus their equivalent clean sand $(N_1)_{60cs}$ values for M = 7.5 and $\sigma'_{vc} = 1$ atm.

CRR relationship proposed by Idriss and Boulanger in EERI MNO 12



Equivalent clean sand corrected standard penetration, (N1) 80cs

Figure 7.1. Liquefaction triggering correlations for M = 7.5 and σ'_v = 1 atm developed by: (1) Seed et al. (1984), as modified by the NCEER/NSF Workshops (1997) and published in Youd et al. (2001); (2) Idriss and Boulanger (2004, 2008); and (3) Cetin et al. (2004)

Comparison of three liquefaction triggering curves (CRR) (MNO 12, p.102)

Depth limitation of dataset (TLY)



Average depth to liquefiable layer versus number of case histories in Idriss and Boulanger (2010) dataset; below a depth of about 12 m (40 ft), there is insufficient case history data to constrain correlations, such as CRR or triggering curves; below 12 m there is significant uncertainty in extrapolated liquefaction curves.

TLY comment on the compiled dataset:

➢As I understand the process, the decision as to whether liquefaction did or did not occur at a particular site is based on observed or lack of observed surface liquefaction effects, such as sand boil deposits, lateral spread, ground fissures, ground settlement, etc

≻Could there be false negatives in the data set? that is sites where liquefaction occurred but did not generate surface effects. I believe there are and that they are more likely to occur above deterministic liquefaction curves than below

➢ For example, at the strong-motion instrument site on Treasure Island surface effects of liquefaction were not reported within 30 m following the 1989 Loma Prieta earthquake

≻Thus this site should be classified as non-liquefied?

≻However, there is a shift of frequency to longer period motions late in the acceleration record, a clear indication that the underlying soil softened due to increased pore-water pressure and that liquefaction likely occurred

≻Could false negatives in the dataset significantly influence the positioning of liquefaction curves?

Seed's re-evaluation of r_d

2.4.1 Simplified rd (Seed and Idriss, 1971)

Prior to 2003, a number of significant liquefaction triggering relationships were based on the use of the "simplified" r_d recommendations of Seed and Idriss (1971) as the basis for assessment of in situ cyclic stress ratios (CSR's) in back-analyses of the critical field performance case histories upon which such correlations are based. These original r_d recommendations of Seed and Idriss are shown in Figure 2-2. In situ CSR's were estimated based on the peak ground surface acceleration as

 $CSR_{eq} = 0.65 \times a_{max}/g \times \sigma_v/\sigma'_v \times r_d \qquad [Eq. 2-1]$

where r_d is a modal mass participation factor defined as per Figure 2-2. (Seed 2010)



▷ Based on more than 2,000 ground response analyses, Seed argues that r_d values taken from the 1971 r_d graph (at left) are too high and that use of those r_d values in the back analyses to calculate CSR led to plotted CSR values that are too high; thus predicted CSR or CRR based on past relationships, including that of Youd et al 2001, yield values that are too high

Seed also argues that rd variability is much greater than indicated on the 1971 rd graph



Figure 2-8: Comparison Between the Empirical r₄ Relationship of Cetin et al. (2004) and the Results of Site Response Analyses for a_{max} ≥ 0.23g



Figure 2-4: Comparison of the Simplified r_d Recommendations of Seed and Idriss (1971) with the Results of 2,153 Seismic Site Response Analyses to Evaluate r_d for a Broad Range of Site Conditions and a Broad Range of Shaking Levels and Shaking Characteristics (Cetin, 2000; Cetin et al.,2004)

Probabilistic r_d curves published by Cetin, Seed et al 2004

Cetin, Seed et al (2004) equation for r_d

$$r_{d}(z, M_{w}, a_{max}, V_{s,12}) = \frac{\left[1 + \frac{-23.013 - 2.949 * a_{max} + 0.999 * M_{w} + 0.0525 * V_{s,12}}{16.258 + 0.201 * e^{0.341(-z+0.0785 * V_{s,12}+7.586)}}\right]}{\left[1 + \frac{-23.013 - 2.494 * a_{max} + 0.999 * M_{w} - 0.0525 * V_{s,12}}{16.258 + 0.201 * e^{0.341(0.0785 * V_{s,12}+7.586)}}\right]}$$

Idriss and Boulanger r_d curves (from EERI MNO 12)



Figure 51. Variations of the stress reduction coefficient r_d with depth and earthquake magnitude (Idriss 1999).

Idriss and Boulanger r_d relationship

Idriss and Boulanger (2008) recommend Equations 22 - 24 for calculation of r_d to a limiting depth of 20 m (65 feet); because of increasing uncertainty with depth they suggest use of site specific response analyses for layers deeper than 20 m and/or critical r_d evaluations (EERI MNO 12, p. 68)

$$r_{d} = \exp(\alpha(z) + \beta(z)M)$$
(22)

$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right)$$
(23)

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right)$$
(24)

Proposed $K\sigma$ curves



Comparison of three proposed K_o correction factors as a function of vertical effective stress and approximate depth (Cox and Griffiths, 2011)

Comparisons between procedures



Bridge sites in Arkansas for which comparisons were made between procedures of (1) Idriss and Boulanger; (2) Cetin, Seed et al; and (3) Youd et al. Site 1 is used for comparisons in this presentation (Cox and Griffiths 2011)



Comparison of the r_d values at Site No. 1 for three procedures; $M_W = 7.6$, pga = 0.82 (Cox and Griffiths 2011)

Borehole log from SPT SP-7, Sapanca Hotel site, Turkey where liquefaction occurred during 1999 Kocaeli earthquake (M = 7.6)

	UCB-BYU-UCLA ZETAS-&aU-METU Joint Research Sponscored by: NSF, Caltrans CEC, PG&E				Project Name: Geotechnical Site Investigations at Lateral Spread Sites Location: Sapanca Hotel, Sapanca Dete: August 21, 2000 Field Log by: K. O. Çetin Operator: 2ETAS (2Emin Tethnoigist, A. S.) Difiling Method: Rotary wash with 9 cm-diameter tricone bit Water Table Elevation: 1.25 m Notes:								Tect ID: SPT-SH7 GP3 Coordinates: 40.69527N 30.255637E Elevation: Drilling Equipment: Custom made, equivalent to Crealus XCSOH Responsible Engineers: K. Ö. Çetin and M. T. Yilmaz, M.E.T.U. SPT System: Rope, pulley and cathead method. AWJ rods. Hammer Type: Safety Hammer (per Kovacs et al. 1983)										
Depth Scale (m)	Lideology	1903	Sample Type and No.	Becovery/ Longth (cm)	SPT Blow/15 cm	Casing Depth (m)	Rod Length (m)	Ecorgy Radio (79)	Description	Qu Pocket Pen (dPa)	Torsue (dPs)	Moistue Content (%)	Liquid Limit	Rasticity Index	$\%$ fines < 75 μm	< 5 µm (%)	< 2 µm (%)	D60 (mm)	D10 (mm)	Remarks			
F	Ø.								GW: Gray sandy gravel														
1	õ	GM	S-SH7-1	23/45	9-7-4	0.90	4.27	60					NP	NP	14	-	-	5.0	<0.07				
ŧ	0	GW	S-SH7-2	14/45	4-2-1	1.30	4.27	-					NP	NP	1			8.4	1.7				
2									SW: Gray gravely sity sand	1													
-3		sw	8-8H7-3	10/45	4-3-2	2.60	5.80	60					NP	NP	5	•	-	3.0	0.23				
Ē		sw	S-SH7-4	22/45	3-4-4	3.40	7.32	63					NP	NP	4		-	3.9	0.33				
4															-								
5			8-8H7-5	21/45	3-5-4	4.20	7.32						DAP-		°		-	•••	0.20				
ŧ			S-SH7-6	0/45	3-4-5	5.00	8.84	65															
6		SM	S-SH7-7	14/45	7-7-9	5.90	10.37	60					NP	NP	13	-	-	1.5	<0.07				
7		SM-SM	S-SH7-8	13/45	4-6-11	6.80	11.89	68					NP	NP	11	-	-	1.6	⊲0.07				
8		SW-SM	S-SH7-9	22/45	6-7-6	7.80	11.89	65					NP	NP	12	-	-	2.0	⊲0.07				
9		SM-SM	S-SH7-10	18/45	7 -6- 6	9.00	13.42	63					NP	NP	6	-	-	3.1	0.29				
10		SM-SM	S-SH7-11	20/45	1-4-6	10.30	13.42	63					NP	NP	9	-	-	2.0	0.12				
Ŀ		-			<u> </u>		1				<u> </u>	<u> </u>	1		1				1				
12		SP-SM	8-8H7-12	22/45	2-5-6	11.80	14.94	61					NP	NP	5	-	-	0.84	0.24				
13		SM	S-SH7-13	11/45	3-8-5	13.50	16.46	65					NP	NP	14	-	-	0.84	<0.07	,			

Borehole log SP-7 from Sapanca Hotel site, Turkey where liquefaction lateral spread occurred during 199 Kocaeli earthquake

Source: DeDen, D.W. (2004). "Analysis of lateral spread sites from the 1999 Kocaeli, Turkey, earthquake," MS Thesis, Dept. of Civil and Environmental Engineering, Brigham Young University, Provo, Utah



Comparison of the r_d values at $M_W = 7.0$, pga= 0.4 for three procedures (Shawn Griffiths)

Idriss and Boulanger modification of C_N EERI MNO 12, p. 84-87

$$C_N = \left(\frac{P_a}{\sigma'_{vc}}\right)^m \tag{33}$$

where *m* is a parameter that depends on the sand properties and relative density (Boulanger 2003b). For example, equation 33 was used to fit the SPT calibration chamber data in Figure 58, resulting in the *m* values that are summarized in Figure 59. Figure 59 also shows a relationship for *m* that fits CPT calibration chamber test results, illustrating how a single C_N relationship with

$$m = 0.784 - 0.521 \cdot D_R \tag{34}$$

$$D_R = \sqrt{\frac{(N_1)_{60}}{46}} \tag{37}$$

$$C_N = \left(\frac{P_a}{\sigma'_{vc}}\right)^{0.784 - 0.0768\sqrt{(N_1)_{60}}} \le 1.7$$
(39)



Figure 58. Variation of SPT *N* values with vertical effective stress for three different sands at three different relative densities (Idriss and Boulanger 2004; data from Marcuson and Bieganousky 1977a, 1977b).



Comparison of the C_N values at Site No. 1 for three procedures (Cox and Griffiths 2011)



Comparison of the C_N values at $M_W = 7.0$, pga= 0.4 for three procedures (Shawn Griffiths)



Comparison of the $(N_1)_{60cs}$ values at Site No. 1 for three procedures (Cox and Griffiths 2011)



Comparison of the $(N_1)_{60cs}$ values at Sapanca Hotel site for three procedures (Shawn Griffiths)



For depths less than 70 ft, mean of calculated FS within +/-10% of individual values with Idriss and **Boulanger FS** highest and Cetin, Seed et al FS lowest

Comparison of the factor of safety (FS) against liquefaction at Site No.1 for three procedures; $M_W = 7.6$, pga = 0.82 (Cox and Griffiths 2011)



Mean of calculated FS within +/- 30% of individual values with Idriss and Boulanger FS generally highest and Seed, Cetin et al FS lowest

Comparison of the factor of safety (FS) against liquefaction for Sapanca Hotel site, $M_W = 7.0$, pga= 0.40 for three procedures (Shawn Griffiths)



Figure 69. Comparison of liquefaction analysis procedures from Idriss and Boulanger (2006) with those from the NCEER/NSF workshop (Youd et al. 2001): (a) ratio of the CRR values and (b) ratio of FS_{liq} .



Figure 70. Comparison of liquefaction procedures from Cetin et al. (2004) with those from the NCEER/NSF workshop (Youd et al. 2001): (a) ratio of the CRR values and (b) ratio of FS_{liq} .

Comparisons of FS published in EERI MNO 12

Above 12 m (40 ft), the Idriss and Boulanger FS and the Cetin, Seed et al FS are generally within +/- 30% of Youd et al FS

TLY Summary Comments on Calculation of Liquefaction Resistance:

- 1. Both Idriss and Boulanger (EERI MNO 12) and Cetin, Seed et al (2004) procedures incorporate improvements relative to Youd et al 2001, including improved data sets, more refined liquefaction triggering or CRR curves, and more careful analyses of the variables r_d , C_N , and K_σ
- 2. Values of r_d , C_N , and K_σ must not be substituted from one procedure to another
- 3. Cetin, Seed et al applied rigorous Bayesian statistical analyses in developing their procedure
- 4. Idriss and Boulanger applied thoughtful and careful theoretical and empirical adjustments in developing their procedure
- 5. Even with improvements, there is still considerable uncertainty in FS due to data scatter, local variations in soil properties, estimates of seismicity, etc
- 6. The procedure of Cetin and Seed generally leads to lower calculated FS against triggering of liquefaction compared to FS of Idriss and Boulanger (up to 30% lower than the mean of the two procedures)
- 7. Which procedure is best will be determined by future use, testing and research
- 8. FS from the Youd et al procedure are also within 10% to 30% of the mean
- 9. Although a 10% to 30% difference may seem large, many calculations in geotechnical engineering contain uncertainties of that magnitude
- 10. As noted later, FS is not the most important factor in analyzing overall liquefaction hazard; hence differences in calculated FS may not be a major issue for deterministic analyses where a "yes" or "no" answer is sufficient

"Sand-like" and "clay-like" materials



Criteria of Idriss and Boulanger (EERI MNO 12) for classification of "sand-like" and "clay-like" materials
Comments of Ray Seed (2010)

"A large and still growing body of laboratory and field performance data appears to contradict the recommendations of Boulanger and Idriss with regard to differentiation between silty and clayey soils that will exhibit "clay-like" and "sand-like" behaviors. In addition, close inspection of their own laboratory test data also appears to refute their recommendations."

"Unfortunately, their recommendations too often lead to unconservative elimination of soils from consideration with regard to potential liquefaction hazard, when in fact some of those soils can be expected to be potentially vulnerable to liquefaction. That, in turn, poses a potentially significant hazard to public safety if the recommendations of Boulanger and Idriss on this issue are widely implemented in practice."

TLY: I present here a few slides from my 2007 CGEA lecture to explain why "sandlike" and "clay-like" classifications are useful. This analysis is now published in: *Youd, T.L., DeDen, D.W., Bray, J.D., Sancio, R., Cetin, K.O., and Gerber, T.M., 2009, "Zero-displacement lateral spreads. 1999 Kocaeli, Turkey earthquake," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, v. 135, no. 1, p. 46-61.* Why did lateral spread **not** occur at finegrained sites in Adapazari, Turkey?

- Low-plasticity fine-grained sediments are divided into two types by Boulanger and Idriss (2006): "sand-like" and "clay like," with a boundary between PI of 3 and 8; for general application, a single conservative bound of 7 is recommended
- Sand-like fine-grained sediments from Adapazari (and many other localities) are strongly dilative (Bray and Sancio, 2006), which inhibits shear deformation
- Softened clay-like fine-grained sediments have finite undrained strength or stiffness, which may be sufficient to prevent lateral spread



Stress-strain loops for "sandlike" behavior contain flat segments where little shear resistance or stiffness develops during deformation; however steep ends of the loops, due to dilatency-induced hardening, limit shear deformation.

Stress-strain loops for "claylike" behavior have sloping segments indicating significant resistance to shear deformation or stiffness, even in a softened condition.

Graphs from Boulanger and Idriss, 2004

Suggested procedure for analysis of lateral spread in clay-like sediment

The MLR procedure [of Youd et al 2002] is not valid for analysis of lateralspread displacement in clay-like sediment. No case histories from those types of sediments are included in the database from which the MLR model was regressed.

To analyze lateral displacement, Youd et al (2009) suggest calculating the dynamic factor of safety (FS) against incipient ground displacement using pseudo-static procedures. For FS > 1.0, displacement should not occur. If FS \leq 1.0, displacement could occur and analytical procedures, such as the Newmark sliding-block model could be used to estimate ground displacement.

The following example illustrates the suggested procedure for clay-like soil:

Boulanger and Idriss (2006) suggest that the undrained cyclic shear strength, S_u , of clay-like soil is:

Su $\approx (q_{cT} - \sigma_v)/N_k$

where $q_{cT} = CPT$ tip resistance corrected for the influence of pore water pressure, $\sigma_v =$ total overburden pressure, and $N_k =$ a constant typically set at 15 for soft sediment.

For the Cumhuriyet Avenue site the lowest average CPT tip resistance, q_{cT} , for a 0.5 m^{*} thick segment of CPT 4-22 is 700 kPa (3.1 to 3.6 m depth); for CPT 4-24 the lowest average is 730 kPa (4.2 to 4.7 m depth). Applying the above equation and an N_k of 15 yields estimated Su of 42 kPa and 43 kPa for CPT 4-22 and 4-24, respectively. *Updated to values applied in Youd et al 2009 Using an infinite slope analysis, the static gravitational driving stress, τ_{st} , along a layer parallel to the ground surface at depth h, is

 $\tau_{\rm st} = \gamma h(\cos\alpha)(\sin\alpha)$

where α = ground slope (0.3%), h = depth to the failure plane (3.5 m to 4.5 m) and γ = unit weight (estimated as 20 kN/m³).

This equation yields τ_{st} between 0.10 kPa and 0.13 kPa, relatively small values.



Cumhuriyet Avenue, Adaparazi, cross section

The peak inertial shear stress, due to earthquake shaking is approximately (Seed and Idriss, 1982):

 $\tau_{\rm pk} = (\gamma h/g) a_{\rm max} r_{\rm d}$

where h = depth to the liquefiable layer (3.4 m and 4.4 m for CPT 22 and 24 respectively), g = acceleration of gravity, a_{max} = peak horizontal surface acceleration generated by the earthquake (approximately 0.4 g), and r_d = stress reduction coefficient (conservatively assumed as 1.0 for this analysis); note also that the factor 0.65 was also conservatively replaced by 1.0

Applying these parameters yields: $\tau_{pk} = 27$ kPa and 36 kPa for CPT 22 and 24, respectively. The factor of safety against onset of ground displacement is:

$$FS = S_u / (\tau_{pk} + \tau_{st})$$

= 42 kPa/(27 kPa + 0.1 kPa) = 1.5 (CPT 4-22)
= 43 kPa/(36 kPa + 0.1 kPa) = 1.2 (CPT 4-24)

thus, ground displacement should not have occurred at the Cumhuryet Avenue site, if underlain by clay-like sediment, in agreement with the observed displacement.

Note: Boulanger and Idriss suggest that the softened or residual strength of clay-like sediment is $0.83 S_u$; use of this reduced strength would be overly conservative because of (1) a_{max} is used in the analysis rather than a reduced value, and (2) softened strength values are usually measured after tens of cycles of loading which is not appropriate for incipient failure analyses

General conclusion:

Low sensitivity "clay-like" soils are generally resistant to lateral spread
 Caution: The sensitivity of "clay-like" soils should be checked; highly sensitive soils (such as Bootlegger Cove clay) may weaken and fail during seismic shaking

Post Liquefaction Shear Strength of Sands

The following discussion applies a logic path for liquefaction hazard evaluation that I commonly apply as a consultant and suggest to others

Evaluation of Liquefaction Hazard

Fundamental Questions:

- Will liquefaction occur (based on FS)?
 No no liquefaction hazard, no mitigation required Yes – continue to question 2
- Will liquefaction lead to detrimental ground deformation, ground displacement, or ground failure?
 No accept liquefaction hazard; no mitigation required Yes continue to Question 3
- 3. What mitigation is required to reduce liquefaction hazard to an acceptable risk?

Question 1. Will liquefaction occur?

Apply a verified evaluation procedure:

Several verified procedures are in use worldwide for evaluating FS against triggering of liquefaction, including those of Cetin, Seed et al (2004) and Idriss and Boulanger (EERI MNO 12)

Although there are differences between the procedures as discussed herein, they generally yield comparable FS (within +/- 30% of mean value)

For those instances where FS is critical, more than one procedure should be applied, with conservative engineering judgment to select an appropriate FS for engineering design, evaluation of potential social disruption, financial loss, etc Question 2. Will liquefaction lead to detrimental ground deformation, ground displacement, or ground failure?

Types of liquefaction-induced ground failure
➢ Flow failure*
➢ Cyclic slope deformation
➢ Lateral spread*
➢ Ground oscillation
➢ Loss of bearing strength
➢ Ground settlement

*evaluation considered in this presentation





Crest and upstream embankment of Lower San Fernando Dam slipped upstream and into reservoir due to liquefaction-induced flow failure during 1971 San Fernando, California earthquake (photo by Les Youd) Analysis:

Conduct limit-equilibrium analysis with the strength parameter set at post-liquefaction shear strength for liquefiable layers

≻If FS > about 1.2, safe against flow failure, but check amount of cyclic slope deformation that could occur

 If deformation is tolerable; accept liquefaction hazard, no mitigation required
 If FS < about 1.2 or if seismic predicted deformations are not tolerable, strengthen liquefiable layer by drainage, compaction, grouting, etc

For $(N_1)_{60-cs}$ ≤ 16, reasonably constrained post-liquefaction strength values can be selected

Critical issue: What is S_r for $(N_1)_{60-cs} > 16$?



Seed (2010) suggested critical strength curve



Figure 4-14: Regression of the Full-Scale Field Failure Case History Data of Seed and Harder (1990) Plotted in the Critical State Context as S_{u,r} vs. N_{1,60,CS}.

To what blowcount should the dashed curve be extrapolated?

Summary comments by Seed (2010) on critical shear strength

My recommendations for any specific application are project specific and material specific, but by way of general interim guidance I would suggest:

- The heavy dashed line shown in Figure 4-14 represents the best current set of recommendations based on the Critical State (S_{u,r}-based) approach.
- (2) The heavy dashed lines of Figure 4-13 or 4-17 show the most valid interpretations of the available field performance data within the alternative S_{u,r}/P-based framework.
- (3) For relatively "clean" cohesionless soils (fines content $\leq 5\%$), the S_{u,r} -based approach provides a better estimate of S_{u,r} than does the S_{u,r} /P approach; for these soils I recommend that values of S_{u,r} be estimated at the locations of interest using both the S_{u,r} -based and S_{u,r} /P-based approaches, and that the final analysis and/or design values be developed by weighted averaging of these, with weighting factors of approximately 3:1 to 4:1 in favor of S_{u,r} -based values. This serves to make S_{u,r} based assessment the primary approach for these soils, but incorporates some partial dependence upon initial effective overburden stress.
- (4) For liquefiable silty/sandy soils with somewhat higher fines contents (soils with higher compressibility), the same type of approach can be employed, but with somewhat lower weighting ratios.

Finally, under no circumstances should the $S_{u,r}$ /P-based recommendations of Idriss and Boulanger be employed. These produce values of $S_{u,r}$ that are considerably higher than can currently be supported based on the available data, especially in the critical range of $15 \le N_{1,60,CS} \le 30$ blows/ft, and they are unsafe.

Idriss and Boulanger curves (EERI MNO 12)



proposed by Idriss and Boulanger



Figure 4-16: Figure 4-15 Repeated, Showing Erroneous Data Points A, B and C

Seed (2010) criticized the upper curve of Idriss and Boulanger and contends that Point A on that curve was not properly analyzed; that point when properly analyzed plots near the lower curve as shown above.
Idriss and Boulanger unequivocally state that the upper curve is based on laboratory test data and was not guided or based on Point A in any way
The error in Point A (plotted by H. Bolton Seed) is a major plank in the argument of Seed (2010) that the curves of Idriss and Boulanger should not be used



TLY: In my opinion, the curves of Idriss and Boulanger recognize two important facts:

- 1. A transformation occurs from contractive to dilative behavior at about a corrected blowcount of 15
- 2. Transition into dilative behavior should be accompanied by a major increase in shear strength (upper curve)

Idriss and Boulanger equation for lower curve (uninhibited void ratio redistribution)

The lower relationship in Figure 89 corresponds to conditions in which the effects of void redistribution can be significant. This would include sites with relatively thick layers of liquefiable soils that are overlain by lower-permeability soils that would impede the post-earthquake dissipation of earthquake-induced excess pore water pressures. In this case, the trapping of upwardly seeping pore water beneath the lower-permeability layer could lead to localized loosening, strength loss, and possibly even the formation of water films (Whitman 1985). This relationship can be represented by the following equation:

$$\frac{S_r}{\sigma'_{vo}} = \exp\left(\frac{(N_1)_{60cs-Sr}}{16} + \left(\frac{(N_1)_{60cs-Sr}-16}{21.2}\right)^3 - 3.0\right) \le \tan\phi'$$

TLY: For uninhibited flow of water into or out of the shear zone (leading to zero positive or negative pore-water pressure change), the strength ratio would be tan \emptyset' ; if any positive pore pressure is retained in a dilating soil, the strength ratio would be < tan \emptyset' , as indicated in the above equation

Idriss and Boulanger equation for upper curve (negligible void ratio distribution)

$$\frac{S_r}{\sigma_{vo}'} = \exp\left(\frac{(N_1)_{60cs-Sr}}{16} + \left(\frac{(N_1)_{60cs-Sr} - 16}{21.2}\right)^3 - 3.0\right) \\ \times \left(1 + \exp\left(\frac{(N_1)_{60cs-Sr}}{2.4} - 6.6\right)\right) \le \tan\phi'$$
(80)

TLY: Uninhibited water flow into a dilation zone would prevent negative pore water pressure from developing in the shear zone and the strength ratio would be tan \emptyset' (as was the case for the equation for lower curve of Idriss and Boulanger)

If negative pore water pressure were to develop during shear and dilation, the strength ratio would be > tan \emptyset' ; however for safe design, engineers do not rely on negative pore water pressure for stabiliby, so " \leq tan \emptyset' " in the equation is correct.

TLY opinions:

- 1. As a consultant, I have recommended a strength ratio of tan \emptyset' at shallow depths (< 10 m) for soil profiles with minimum (N₁) _{60 cs} > 16 on the basis that dilative soils would have at least that strength (neglecting negative pore water pressures)
- 2. I believe that the curves, equations and statements by Idriss and Boulanger are on the right track; the undrained shear strength of granular soil should increase sharply as soils become dilative

TLY opinions continued



3. Why are there no data points on the diagram for (N₁)_{60cs} > 16? (This diagram has been in existence for more than 20 years)
A. There have been insufficient earthquakes to generate failures?
B. There has been insufficient

- investigation of slope failures that have occurred?
- C. There have been no failures in materials with corrected blowcounts greater than 16?
- 4. That there are no data for corrected blowcounts greater than 16 plotted on the 1990 Seed and Harder plot is not an omission, but a significant finding; liquefaction induced slope or embankment failures have not occurred in granular materials with corrected blowcounts greater than 16
- 5. The Seed and Harder plot would be greatly strengthened if data from non failures were added; one potential candidate is the Hebgen Lake Dam that was very strongly shaken during the 1959 West Yellowstone earthquake (M = 7.3) but slope failure did not occur, although the crest of the embankment settled a few feet (Sherrard et al 1963)

LATERAL SPREAD

Before earthquake





Tall building supported on piles pulled apart at foundation level by lateral spread toward a nearby island edge; building is located on, Rokko Island and was damaged during 1995 Kobe, Japan earthquake (photo by Les Harder)

Analysis:

Apply multiple linear regression (MLR) procedure of Youd et al, 2002

For Free Face Conditions:

 $\label{eq:2.1} \begin{array}{l} \text{Log DH} = -16.713 + 1.532 \ \text{M} - 1.406 \ \text{log R}^* - 0.012 \ \text{R} + 0.540 \ \text{log T15} + 0.592 \ \text{log W} + 3.413 \ \text{log (100} - \text{F15)} - 0.795 \ \text{log (D5015} + 0.1 \ \text{mm)} \end{array}$

For Ground Slope Conditions:

 $\label{eq:2.13} \begin{array}{l} \mbox{Log DH} = -16.213 + 1.532 \mbox{ M} - 1.406 \mbox{ log R}^* - 0.012 \mbox{ R} + 0.540 \mbox{ log T15} + 0.338 \mbox{ log S} + 3.413 \mbox{ log (100 - F15)} - 0.795 \mbox{ log (D5015 + 0.1 \mbox{ mm})} \end{array}$

Where:

 $R^* = R + Ro$ and Ro = 10 (0.89M - 5.64)

Note: No known lateral spreads have occurred in granular soils with minimum blowcounts in soil profile greater than 15

Summary:

Liquefaction may cause any of the following types of ground failure, but significant deformation or displacement is not likely (at least at shallow depths (<10 m)) for (N₁)₆₀ > 16 because such granular soils are dilative:
 Flow failure
 Cyclic slope deformation
 Lateral Spread
 Ground Oscillation
 Loss of bearing strength

➤Ground Settlement

2. Amount of ground deformation or displacement should be analyzed. If deformation or displacement is tolerable, liquefaction hazard is acceptable; no mitigation required. If deformations are potentially damaging, mitigation is necessary

3. The following mitigation measures may be applied:

- ≻Avoid the hazard
- ≻Accept the hazard
- ≻Strengthen the structure
- ≻Stabilize the ground

Need for a future workshop on liquefaction:

Because of the present chaos, EERI formed an Ad Hoc Committee on Soil Liquefaction During Earthquakes to evaluate the issues creating the chaos and to suggest possible procedures to restore harmony. Two possible ways forward are suggested:

- 1. An open discussion in the literature, with encouragement to Cetin, Seed et al and to Idriss and Boulanger to publish additional papers in support of their respective procedures; this approach would allow airing out differences and permit other professionals and stake holders to enter into the discussion
- 2. A formal workshop where proponents of various procedures could make presentations to a panel of experts and stakeholders who would then develop consensus guidance for the profession. Issues in addition to those noted herein would be considered in the workshop to provide a more global assessment of liquefaction issues

I prepared this presentation with both approaches in mind:

- 1. I have evaluated and discussed various issues raised in the dispute with liberal injection of my views; hopefully this presentation will engender further discussion and deliberation of the issues
- 2. If asked to make a presentation to the proposed workshop, these are the comments , opinions, and issues that I would raise

Analysis of Laterally and Axially Loaded Group of Shafts or Piles

using Computer Program DFSAP Deep Foundation System Analysis Program based on the Strain Wedge Model

developed for Washington State Department of Transportation

> by M. Ashour, G. Norris and J.P. Singh

THIS PRESENTATION

•Comparison between Current Practice and the Strain Wedge Model Technique Used in Program DFSAP

•Soil Liquefaction, Lateral Spreading and their Effect on Pile/Shaft Response

•Shaft/Pile Length (Short/Intermediate/Long) in Soil Profiles (Liquefied or Nonliquefied) and Pile Cap Effect

•Piles in Sloping Ground

•Linear and Nonlinear Equivalent Stiffness Matrix for Bridge Foundations

DIFFERENCES BETWEEN THE TRADITIONAL P-Y CURVE PROGRAM AND PROGRAM DFSAP

Traditional p-y Curve Does Not Account for the Following:

- Pile Bending Stiffness (EI)
- Pile Head Conditions (Free/Fixed)
- Pile Cross-Section Shape (Square/Circular/H-Shape)
- Pile-Head Embedment Below Ground
- Soil Profile Continuity (no Winkler Springs Interaction)
- Short / Intermediate Piles (developed for Long Piles)
- Soil Stress-Strain Behavior (uses Empirical Parameters
- Soil Liquefaction and Lateral Soil Spread
- Pile Group Interferrenc
- Vertical Side Shear Resistances (for Large Diameter Shaft)













Computer Program DFSAP Deep Foundation System Analysis Program developed using Strain Wedge Method for Washington State Department of Transportation for Analysis of Laterally and Axially Loaded Group of Shafts and Piles






























































Current Available Procedures that Assess the Pile/Shaft Behavior in Liquefied Soils (Using the Traditional p-y Curve):

- 1. Construction of the p-y curve assuming <u>soft clay</u> type response but using (undrained) strength equal to the residual strength of liquefied sand presented by Seed and Harder (1990)
- 2. The use of a random $P_{mult} < 1$ to reduce the stiffness of the traditional p-y curve of sand
- 3. Reduce the unit weight of liquefied sand by the amount of r_u (Earthquake effect in the free-field) and then build the traditional p-y curve of sand based on the new value of the sand unit weight.

proposed by Brown based on Cooper River Test)





















p-y Curve Empirical Formula in Liquefied Sand by Rollins et al. 2005

 $p_{(d=324 \text{ mm})} = A(By)^{C} \text{ for } D_{r} = 50\%$ where: A = 3 x 10⁻⁷ (z+1)^{6.05}, B = 2.8 (z+1)^{0.11} C = 2.85(z+1)^{-0.41} z is depth in (m) y is lateral deflection (mm) $p_{multiplier} = 3.81 \ln d + 5.6$

 $\mathbf{p} = \mathbf{p}_{(d=324 \text{ mm})} \mathbf{x} \mathbf{p}_{\text{multiplier}}$

















































	Li	<u>near</u> S	Stiffnes	ss Mat	rix						
	F1	F2	F3	M1	M2	M3					
Δ_1	K 11	0	0	0	0	-K16					
Δ_2	0	K22	0	0	0	0					
Δ_3	0	0	K33	K34	0	0					
Θ_1	0	0	K43	K44	0	0					
Θ_2	0	0	0	0	K55	0					
Θ_3	-K61	0	0	0	0	K66					
•	Linear St	tiffness	Matrix i	is based	on						
• Linear p-y curve (Constant E _s), which is not the case											
• Linear elastic shaft material (Constant EI), which is not											
	the actual behavior										
	From wh	ich									
	$\Delta_{\mathbf{P},\mathbf{M}} = \Delta$	$\Delta_{\rm P} + \Delta_{\rm M}$	and		$\overline{\theta_{P,M}} = \theta$	$_{\rm P} + \theta_{\rm M}$					



F1	F2	F3	M1	M2	M3
K 11	0	0	0	0	0
0	K22	0	0	0	0
0	0	K 33	0	0	0
0	0	0	K44	0	0
0	0	0	0	K55	0
0	0	0	0	0	K66
Non Non Non A_{P, N}	linear St linear p- linear st $_{I} > \Delta_{P} +$	tiffness y curve aft mate Δ _M	Matrix i erial (Va K	s based o arying El 11 = P _{appli}	$\int_{ed}^{0} / \Delta_{P,N}$
























