PEER Lifelines Project 3A02

DOCUMENTATION OF SOIL CONDITIONS AT LIQUEFACTION SITES FROM 1999 CHI CHI (TAIWAN) EARTHQUAKE

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Executive Summary

The 1999 Chi Chi, Taiwan, earthquake provides case histories of ground failure and non-ground failure that are valuable to the ongoing development of liquefaction susceptibility, triggering, and effects models because the data occupy sparsely populated parameter spaces (i.e., high cyclic stress ratio and high fines content with low to moderate soil plasticity). In this research program, we have conducted site investigation programs in the highly impacted cities of Nantou and Wufeng, Taiwan, and prepared the results in a consistent format for web publishing. Through a memorandum of understanding with Taiwan's National Center for Research in Earthquake Engineering (NCREE), we have secured access to an additional large data set from a similar site exploration program that was conducted in Nantou, Wufeng, and Yuanlin. This NCREE data has been translated and compiled in the same format used for the present study. The results are published on the web site of the Pacific Earthquake Engineering Research center for broad dissemination to the research community.

Introduction and Objectives

The 1999 M_w = 7.6 Chi-Chi Taiwan earthquake triggered numerous significant incidents of liquefaction in inland alluvial areas and in several coastal hydraulic fills (Stewart, 2001). Significant occurrences of ground failure in the form of liquefaction, ground softening, and lateral spreading were documented by NSF-sponsored reconnaissance teams in several affected areas. The goal of this project was to characterize the subsurface conditions at sites where ground deformations and/or building movements (or the lack of such deformations and movements) were well documented.

The project involved two independent tasks. First, a site investigation program was undertaken by the PI, his students, and his collaborators that focused on the cities of Nantou and Wufeng. Work in these cities was emphasized because of the high levels of shaking in these regions, and the need for case histories involving strong shaking in empirical liquefaction databases. The second task involved securing access to data from an additional site exploration program funded by Taiwan's National Center for Research in Earthquake Engineering (NCREE). A memorandum of understanding (re-produced in Appendix A) was reached with NCREE, which enabled the transfer of data from that program to the PI. The data from both testing programs was translated and compiled in a uniform format for dissemination via the world-wide-web. The web site is re-produced on the attached cd-rom (Appendix B). Some of the specific engineering implications of the collected data are discussed in Stewart et al. (2003), which is attached as Appendix C.

Investigated Regions

The site investigation programs sponsored by PEER Lifelines and NCREE resulted in a total of 92 Cone Penetration Test (CPT) profiles (of which 63 were seismic CPTs) and 98 soil borings with Standard Penetration Testing (SPT) (typically at 1.0 m spacing). The majority of the NCREE work was performed in the city of Yuanlin, whereas the entirety of the PEER work and some of the NCREE work was performed in the cities of Nantou and Wufeng. The locations of these cities is provided in Figure 1.

Field Exploration Protocols

In the work sponsored by the PEER Lifelines Program, most of the borings/CPTs were limited to depths of 10-30 m. CPT profiling was performed using a seismic piezocone according to standard techniques (ASTM D 5778-95). For SPT sampling, the percentage of the total theoretical energy delivered to the split-spoon sampler, or energy ratio, was controlled by following procedures in ASTM D6066-98 and ASTM D1586. We used a safety hammer with a rope/cathead release mechanism, two turns of the rope around the cathead, standard AW rod, and a 12 cm borehole diameter. Hence, the energy transmitted to the sampler would be assumed to be 60% if no short-rod correction was applied. The actual delivered energy was measured for each blow of the hammer using a rod section instrumented with accelerometers and strain gages (Abou-Matar and Goble 1997). In addition, in-situ vane shear tests were performed at selected locations in accordance with ASTM D 2573-94. The results are presented on the boring logs (see logs for Wufeng, Site A).

All retrieved soil samples were subjected to a full suite of laboratory index tests per ASTM standards including sieve, hydrometer, liquid limit, plastic limit, density and water content. Results are presented on the boring logs.

Sites selected for subsurface exploration included lateral spread sites, locations of tilted and/or settled buildings, and locations of no apparent ground failure based on post-earthquake reconnaissance. After the completion of field work, the horizontal and vertical positions of all boring and CPT locations were established by a professional land surveyor. The borehole and CPT locations are shown in site plans on the web site.

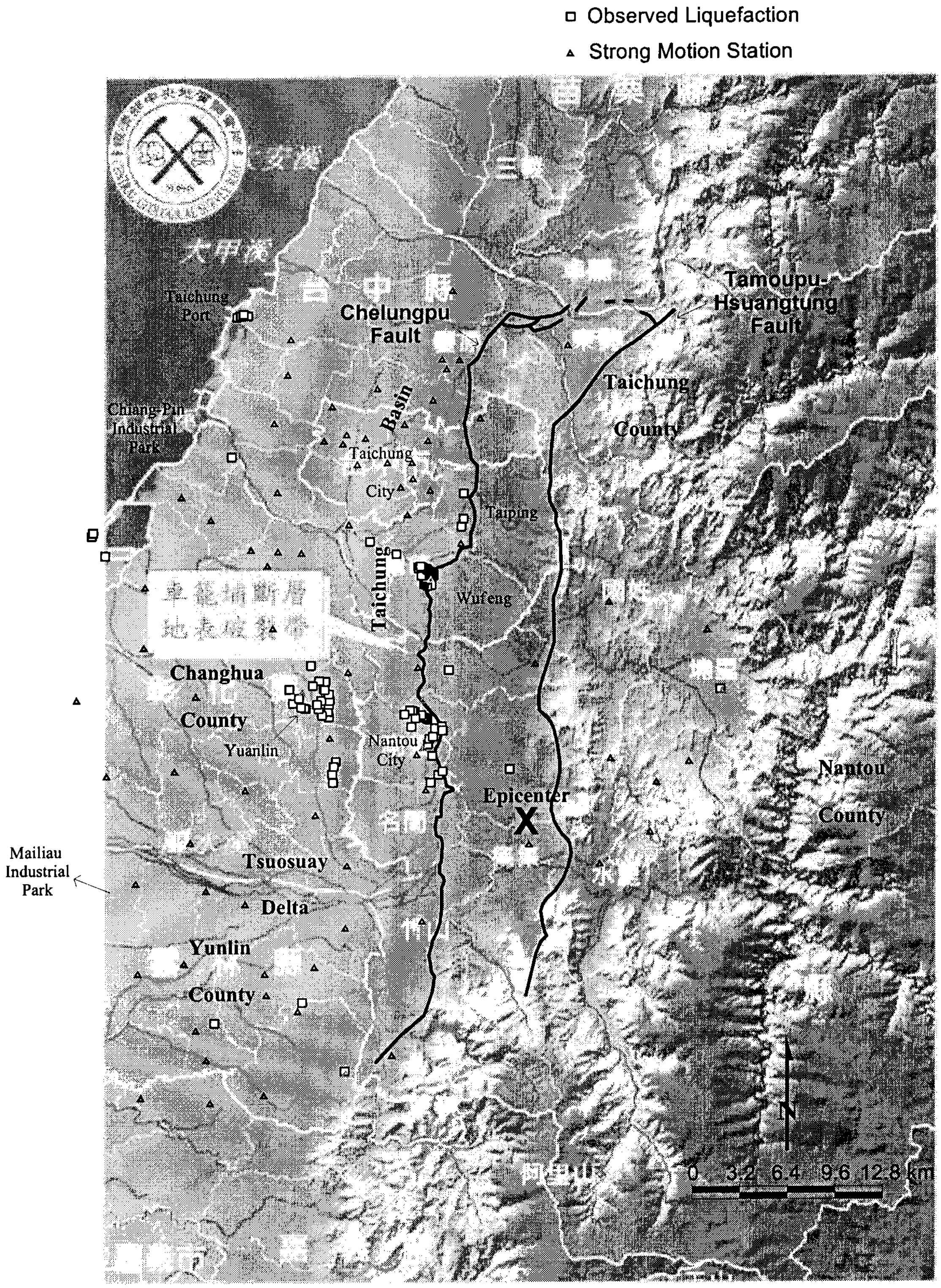

Fig. 1. Map of central Taiwan showing locations of Nantou, Wufeng, and Yuanlin

Examples of Collected Data

Example CPT and boring/SPT logs are presented in Figures 2 and 3, respectively. The CPT logs (e.g., Figure 2) show the measured tip resistance (q_c) as a total stress. Tip area effects associated with the piezocone configuration and the measured water pressure (Robertson and Campanella, 1988) are corrected for in the data. Also shown are side friction (f_s) and the friction ratio $R_f = f_s/q_c$. The file header provides metadata on site location, surveyed coordinates of the CPT, the name of the Excel data file with the data, the date of exploration, the water table depth, and the responsible engineer for the field work.

The boring logs (e.g., Figure 3) provide material descriptions, results of penetration testing (including incremental blow counts, measured energy ratio, and rod length), and results of various laboratory index tests. The header provides the same information as the CPT header, plus details on the drilling method, hammer type, and SPT hammer lift system.

Presentation of Data

All of the collected data is presented on the CDRom in Appendix B. The data is organized first by city, next by site title (A, B, C, etc.), and finally by hole location on the site.

Lessons Learned

Data collection studies such as the effort described herein will hopefully be undertaken following future significant earthquakes. Lessons learned from this study may be useful during such efforts. Some of these lessons include:

1. The most critical phase of the data collection effort is the post-earthquake reconnaissance during which locations of ground failure and non-ground failure are mapped, and the surface deformations are measured/characterized. It is absolutely essential that the post earthquake

reconnaissance provide quantitative descriptions of displacements, buildings tilts, etc. The preparation of maps drawn to-scale and showing the affected areas is also critical. In general, it is preferable to do a thorough job of damage documentation within a limited geographic region than to cover a broad region but without adequate detail. Finally, it should be emphasized that post-earthquake reconnaissance should document locations of non-ground failure as well as locations of ground failure.

2. The key to successful site characterization work in foreign countries is good local contacts. Coordination with university faculty and working engineers in the host country is absolutely vital. Ideally, these contacts should be established during or shortly after the reconnaissance work. In-person meetings with the collaborators should take place prior to the commencement of field work so that the role of all participants is well understood. Additional subjects that should be addressed in this initial meetings include availability of maps, site access issues, ability to transport equipment within urban areas, access to surveying crews, names and phone numbers of local contacts, and local customs (holidays, religious restrictions, etc.).
3. Drill rigs used to make SPT measurements should use standard equipment (hammers, rods, anvils, augers) whenever possible. For research-quality work, it is essential to measure the energy delivered to the rod.
4. A thorough suite of index tests should be performed on as many soil samples as possible. Even samples judged in the field to be "non-plastic" should be subject to liquid limit tests so that liquefaction susceptibility criteria can be checked.
5. The importance of documenting ground conditions in non-ground failure areas cannot be over-emphasized.

Acknowledgements

This project was sponsored by the Pacific Earthquake Engineering Research Center's Program of Applied Earthquake Engineering Research of Lifeline Systems supported by the State Energy Resources Conservation and Development Commission and the Pacific Gas and Electric Company. This work made use of Earthquake Engineering Research Centers Shared Facilities supported by the National Science Foundation under Award #EEC-9701568. In addition, the support of the California Department of Transportation's PEARL program is acknowledged.

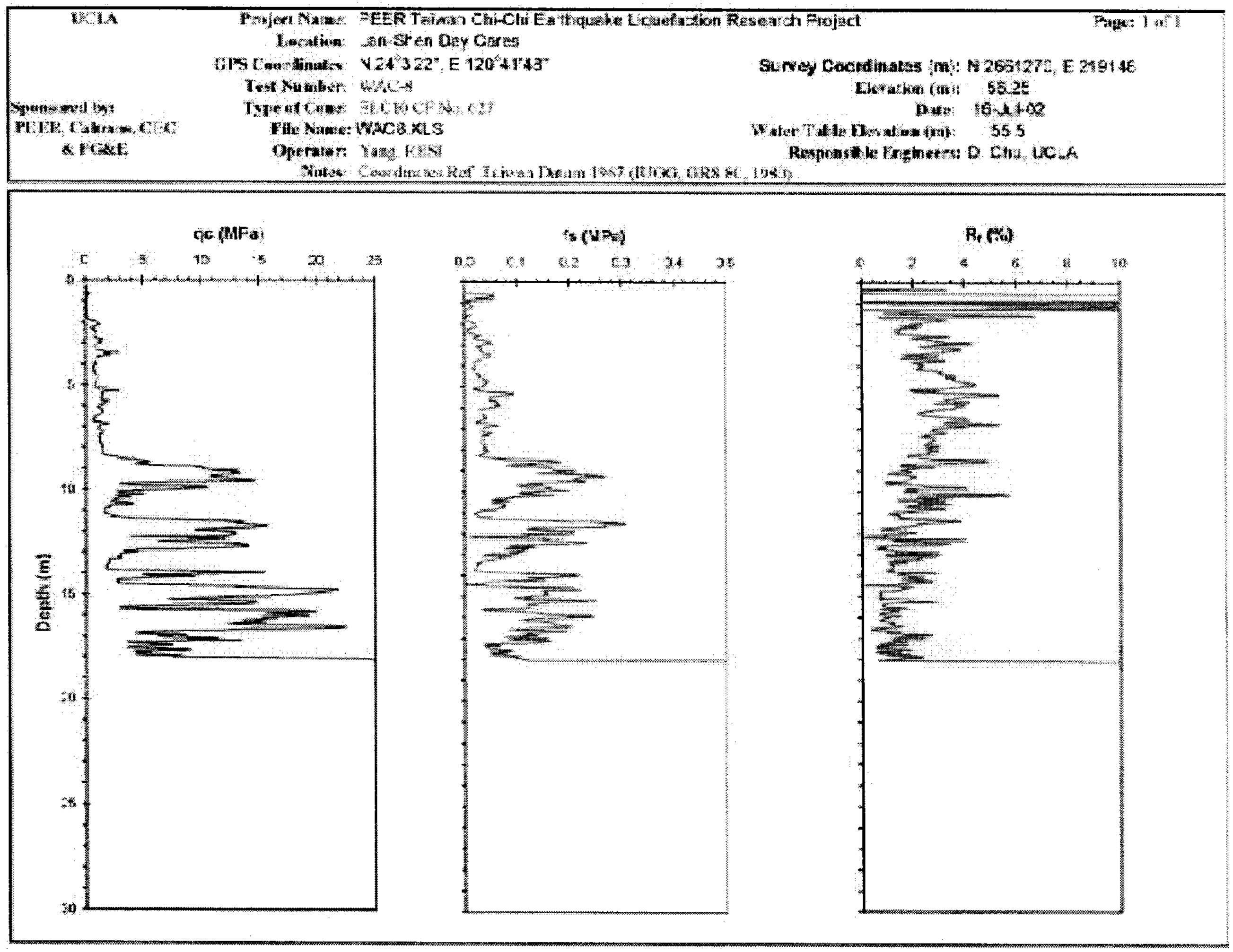

Fig. 2. Example of CPT log (WAC-8)

UCLA

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Project Name: PEER/Taiwan Chi-Chi Earthquake Liquefaction Research
Location: Lan-Shen Day Care site, 3.8 m from WAC-6, Wufeng, Taiwan
Date: 10/4-10/6/01
Field Log by: DC
Operator: Mr. Yang of Shin-Yang Drilling
Drilling Method: Rotary wash/percussion with 9-cm-diameter fishtail bit
Water Table Elevation: 54.5
Notes: Coordinates Ref: Taiwan Datum 1967 (IUGG, GRS 80, 1980)

Test ID: WAS-2
GPS Coordinates: N 24.056760, E120.696570
Elevation: 55.601
Drilling Equipment: Tripod Rig
Responsible Engineers: Daniel Chu
SPT System: Rope, pulley and cathead. AWJ rodss.
Hammer Type: Safety Hammer (per Kovacs et al., 11983)

USCS	Sample Type and No.	SPT Blows/15 cm	Casing Depth (m)	Rod Length (m)	Energy Ratio (%)	Description	Dry Unit Weight (kN/m^3)	Moisture Content (%)	Liquid Limit	Plasticity Index	% fines < 75 μm	< 5 μm (%)	< 2 μm (%)	D50 (mm)	D10 (mm)	Remarks
						FILL: Brown to orange brown, moist, medium dense, fine to medium grained silty sand with bricks										
	No Recovery	2-2-3		5.02	70	SILTY SAND: Gray to dark gray, wet, loose to medium dense, fine silty sand										
CL	S-1	2-2-4	3	5.48	69	SILTY CLAY: Light gray to greenish brown, wet, soft silty clay Changes to gray and orange brown mottled	16.5	20.0	28	9	85	21	18	0.03		
CL	SH-1	PUSH	3					25.6	40	21	93	41	34	0.009		
ML	S-2	3-3-5	3	7.94	70	SANDY SILT: Gray and orange brown to greenish brown mottled, wet, medium dense to stiff, little plastic, very fine sandy silt	16.2	25.9	30	5	88	20	16	0.04		
	SH, No Recovery	PUSH	3													
ML	SH-2	PUSH	6													
ML	S-3	5-7-12	6	10.99	80		17.3	18.9	22	3						
SM	S-4	8-9-10	6	12.03	80	SILTY SAND: Gray and orange brown to greenish brown mottled, wet, medium dense silty sand	16.9	20.7 21.3	19 22	7	18.4 75	4 13	2 9	0.16 0.07	0.03 0.0022	
CL-ML	S-5	5-6-7	6	12.97	73	CLAY AND SILT: Greenish brown, wet, stiff, sandy to silty clay to clayey silt Becomes dark gray	16.4	27.0 22.6 21.7	23 27	8 9	80 70	13 25	8 17	0.068 0.018	0.003	
SM	S-6	9-13-14	6	13.97	74		17.8	17.8 19.7 19.8	20 20	1	21 51	6 11	4 6	0.13 0.0066	0.018 0.0042	
SM	S-7	6-13-15	6	14.96	66	SILTY SAND: Light gray to greenish brown, wet, dense, very fine sandy silt to silty sand Becomes gray silty sand	16.7	20.2	24	1	20	4	3	0.11	0.02	
SM	S-8	6-12-17	6	15.89	77		18.2	16.8	20	2	11	3	2	0.14	0.06	
SM	S-9	6-17-22	6	16.97	68		16.6	22.7	18	1	17	4	3	0.14	0.025	
SM	S-10	7-17-20	6	17.89	75		17.5	20.4	17		14	3	2	0.15	0.03	
CL	S-11	3-6-9	6	18.99	76	SILTY CLAY: Gray, wet, medium stiff to stiff silty clay	16.8	20.8	25	8	99	36	30	0.01		
SM	S-12	7-8-9	6	19.91	72	SILTY SAND: Gray, wet, dense, fine silty sand	17.8	18.3	20	3	19	5	4	0.13	0.017	
CL-ML	S-13	8-9-13	6	20.93	65	SILTY CLAY: Gray, wet, medium stiff to stiff, plastic silty clay	16.6	23.6	23	7	90	24	18	0.022		
ML	S-14	8-12-15	6	21.93			16.5	24.1	30	4	98	28	19	0.014		
						SILTY SAND: Gray, wet, very dense, clayey sand to silty sand										
SC-SM	S-15	10-17-28	6	23.74	80		17.5	18.4	19	5	28	6	4	0.11	0.011	
SM	S-16	30-20/5	6	25.91	82		19.4	14.5	18		12	1	1	0.14	0.07	

Legend
S: Spit Spoon (SPT) SH: Shelby tube

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Fig. 3. Example boring log (WAS-2)

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Appendix A

PEER-NCREE
Memorandum of Understanding

UNIVERSITY OF CALIFORNIA, BERKELEY

BERKELEY DAVIS IRVINE LOS ANGELES RIVERSIDE SAN DIEGO SAN FRANCISCO SANTA BARBARA SANTA CRUZ

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

1301 S 46TH STREET
RICHMOND, CALIFORNIA 94804

October 1, 2002

Dr. Chin-Hsiung Loh, Director
National Center for Research in Earthquake Engineering
200, Section 3, Hsinhai Road
Taipei 106, Taiwan

RE: Permission to use and publish liquefaction data report

Dear Director Loh:

It is our understanding that you have been in correspondence with Mr. Daniel Chu, a graduate student working with Professor Jonathan Stewart of UCLA on a PEER-funded research project investigating soil liquefaction resulting from the 1999 Chi-Chi, Taiwan earthquake. As you may know, this research has involved extensive site exploration (20 borings with SPT and 33 CPT with > 500 laboratory index tests) in Nantou and Wufeng. We understand from Mr. Chu that shortly after the earthquake you sponsored site exploration work by Moh and Associates (MAA) in which site exploration work was performed in Yuanlin, Nantou, and Wufeng. This site exploration work consisted of borings with SPT, CPT profiling, and laboratory index testing.

We are writing to follow-up on your oral correspondence with Mr. Chu regarding the issue of data sharing. The PEER center and Professor Stewart will be happy to share with NCREE the complete results of Professor Stewart's ongoing research. You would be welcome to publish this data on your web site (with proper citation). What the PEER center would like from you is permission to use and publish on our web site the results of the NCREE site exploration work by MAA. Naturally, we would provide a citation to MAA and NCREE as the source of this data. Specifically, we are requesting that you provide us with hard copies of the data reports and digital data files from the CPT profiling.

We at PEER have enjoyed our interaction with the NCREE center, and look forward to future fruitful collaborations between U.S. and Taiwanese researchers. Thank you in advance for your consideration of this request.

Respectfully yours,

Jack P. Moehle
Director of PEER

Michael Riemer
PEER Lifelines Program Manager

Copy: Jonathan P. Stewart

Appendix B

CDROM with Web Page
(open by clicking on main.html)

Appendix C

TCLEE Manuscript Regarding Data Utilization

Citation: Stewart, J.P., Chu, D.B., Lee, S., Tsai, J.S., Lin, P.S., Chu, B.L., Moss, R.E.S., Seed, R.B., Hsu, S.C., Yu, M.S., and Wang, M.C.H. (2003). "Liquefaction and non-liquefaction from 1999 Chi-Chi, Taiwan, earthquake," in *Advancing Mitigation Technologies and Disaster Response for Lifeline Systems*, Technical Council on Lifeline Earthquake Engineering, Monograph No. 25, J.E. Beavers (ed.), 1021-1030.

LIQUEFACTION AND NON-LIQUEFACTION FROM 1999 CHI-CHI, TAIWAN, EARTHQUAKE

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Abstract:
The 1999 Chi Chi, Taiwan, earthquake provides case histories of ground failure and non-ground failure that are valuable to the ongoing development of liquefaction susceptibility and triggering models because the data occupy sparsely populated parameter spaces (i.e., high cyclic stress ratio and high fines content with low to moderate soil plasticity). In this paper, we synthesize results from several large site investigation programs conducted in Nantou and Wufeng, Taiwan, and compare the data to susceptibility and triggering models. With regard to liquefaction susceptibility, we find components of the well-known Chinese criteria associated with liquid limit (LL) and water content/LL to be reasonably well validated by the Taiwan data, but clay fraction criteria and CPT-based criteria to not be effective. Triggering models are generally validated for ground failure sites, but the data raise important questions regarding non-ground failure sites whose performance is not well predicted.

Introduction

The 1999 $M_w = 7.6$ Chi-Chi Taiwan earthquake triggered numerous significant incidents of liquefaction in inland alluvial areas and in several coastal hydraulic fills (Stewart, 2001). Due to significant interest in the available case histories of liquefaction and non-liquefaction, a series of site investigation programs were undertaken in 2000 by researchers with the National Center for Research in Earthquake Engineering (NCREE) in Taiwan and in 2001-2002 by the authors with funding from the Pacific Earthquake

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6 Project Engineer, Fugro Inc., Ventura, CA
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Engineering Research Center (PEER). Results of both site investigation programs are synthesized on the web page http://www.cee.ucla.edu/faculty/Taiwanwebpage/Main.htm.

The objectives of this paper are to (1) document a number of the particularly significant case histories to emerge from this earthquake, (2) compare these case histories to state-of-practice (i.e., Seed et al., 1985; Robertson and Wride, 1998; Youd et al., 2001) as well as state-of-the-art (Seed et al., 2001; Moss and Seed, 2002) liquefaction triggering procedures, and (3) compare the case histories to established liquefaction susceptibility criteria. As will be shown subsequently, the Taiwan liquefaction data has an important role to play in the ongoing development of empirical liquefaction assessment methodologies for two principal reasons:

- Many of the Taiwan case histories involve high cyclic stress ratios (CSR ≈ 0.4-0.6), where existing data is sparse.
- The Taiwan case histories involve primarily high fines content soils, where the existing data inventory is sparse [i.e., the Youd et al. (2001) triggering model is based on only 13 cases with ≥ 35% fines content, Seed et al. (1985)].

Site Investigation Program

The site investigation programs by NCREE and PEER resulted in a total of 47 Cone Penetration Test (CPT) profiles (of which 18 were seismic CPTs) and 48 soil borings with Standard Penetration Testing (SPT) (typically at 1.0 m spacing). The majority of the NCREE work was performed in the city of Yuanlin, whereas the entirety of the PEER work and some of the NCREE work was performed in the cities of Nantou and Wufeng. In this paper we focus on Nantou and Wufeng, where the CSRs were relatively high (CSR ≈ 0.4-0.6, as compared to CSR ≈ 0.2 in Yuanlin).

Most of the borings/CPTs were limited to depths of 10-15 m. CPT profiling was performed according to standard techniques (ASTM D 5778-95). For SPT sampling, the percentage of the total theoretical energy delivered to the split-spoon sampler, or energy ratio, was controlled by following procedures in ASTM D6066-98 and ASTM D1586. We used a safety hammer with a rope/cathead release mechanism, two turns of the rope around the cathead, standard AW rod, and a 12 cm borehole diameter. Hence, the energy transmitted to the sampler would be assumed to be 60% if no short-rod correction was applied. The actual delivered energy was measured for each blow of the hammer using a rod section instrumented with accelerometers and strain gages (Abou-Matar and Goble 1997). Using the average energy ratio (ER) for each test, we computed the blow-count normalized to 60% of the theoretical energy, N_{60}.

All retrieved soil samples were subjected to a full suite of laboratory index tests per ASTM standards including sieve, hydrometer, liquid limit, plastic limit, and water content. Results are presented on boring logs on the aforementioned web page. These test results were used for liquefaction susceptibility analysis, as discussed below.

Sites selected for subsurface exploration included lateral spread sites, locations of tilted and/or settled buildings, and locations of no apparent ground failure based on post-earthquake reconnaissance. A total of 22 sites in Wufeng and 27 in Nantou were investigated. Locations of the Wufeng sites are overlaid on damage locations in Figure 1. A similar map for Nantou is presented on the aforementioned web page.

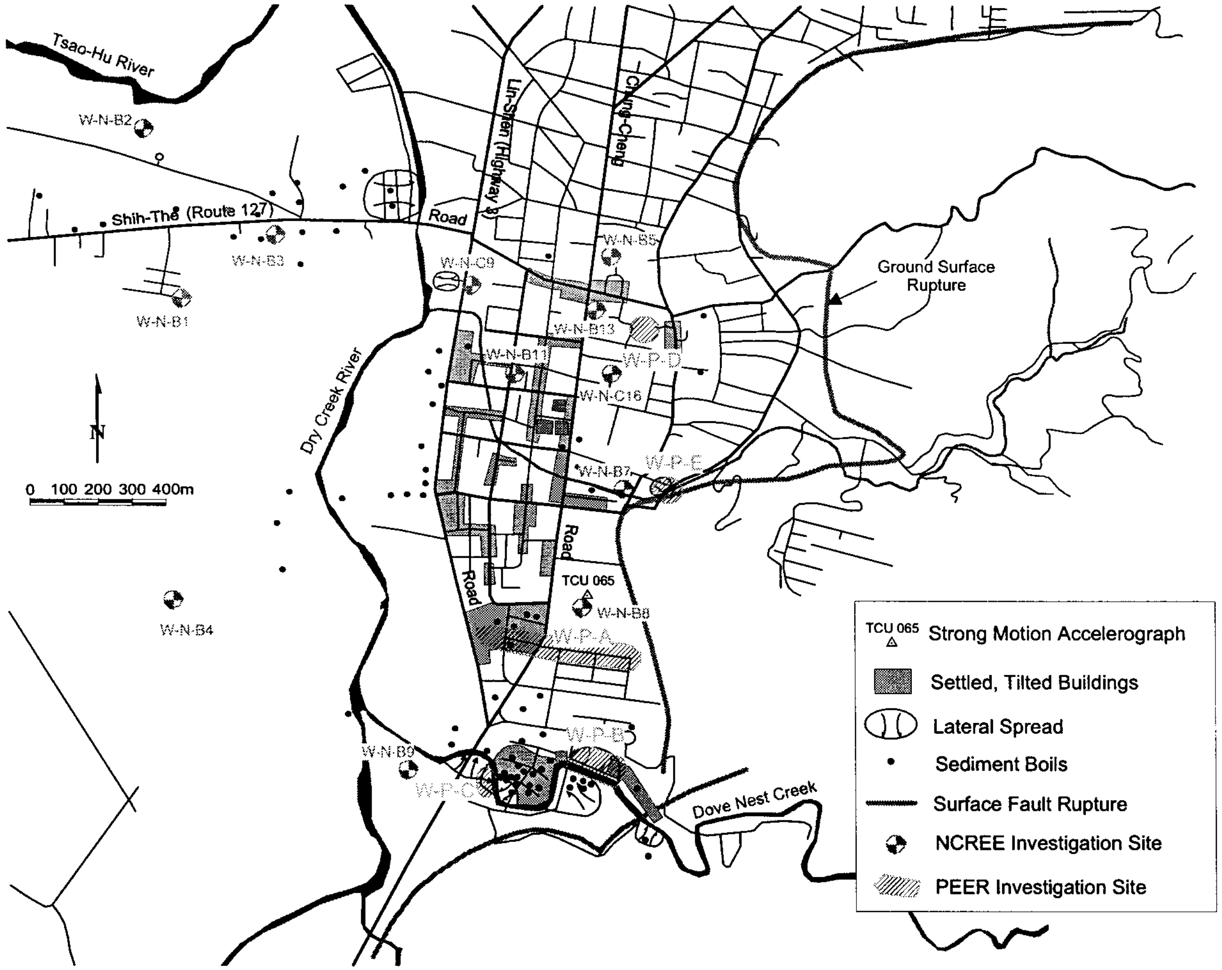

Fig. 1. Map of Wufeng showing ground failure zones and locations of investigated sites

Data Analysis for Liquefaction Susceptibility and Triggering

Seismic Demand in Nantou and Wufeng

Strong motion accelerographs (SMAs) are present in both Nantou (Station TCU076) and Wufeng (TCU065). In both cities, the SMAs are located within about 1 km of the liquefaction/non-liquefaction sites considered in this research, and are on generally similar site conditions (young alluvium). There was no evidence of ground failure in the immediate vicinity of either SMA. Both SMAs are on the footwall side of the ruptured Chelungpu fault, as are the subject liquefaction/non-liquefaction sites. The geometric mean peak horizontal accelerations of the two horizontal components of shaking were PHA = 0.38g in Nantou and 0.67g in Wufeng. These PHA values were used to estimate cyclic stress ratios at the liquefaction/non-liquefaction sites, as discussed below.

Liquefaction Susceptibility Analysis

Most of the soils at the investigated sites in Nantou and Wufeng contain significant fines (i.e. > 35% passing the #200 sieve). Fine-grained soils require analysis to evaluate their liquefaction susceptibility. A well known specification for checking liquefaction susceptibility is the "Chinese criteria," originally presented by Seed and Idriss (1982) and re-stated by Youd et al. (2001). The Chinese criteria specify that liquefaction can only occur if all three of the following conditions are met: (1) weight fraction smaller than 5μm (i.e., "clay fraction," CF) < 15%, (2) liquid limit (LL) < 35%, and (3) natural water content (w_n) > 0.9LL. More recently, Andrews and Martin (2000) stated that silty soils are susceptible to liquefaction if both LL < 32% and the amount finer than 2 μm < 10 %, whereas Sancio et al. (2002) found from Adapazari, Turkey, case histories that the Chinese criteria are effective provided the CF criterion is neglected. As described further below, in this study weight was given to the LL and w_n/LL components of the Chinese criteria during the identification of critical layers at the subject sites.

Liquefaction susceptibility criteria based on CPT test results are not well established, although Robertson and Wride (1998) have proposed that the I_c parameter can distinguish relatively granular soils from potentially plastic soils, with I_c = 2.6 being an approximate boundary between the two. Moss and Seed (2002) found from Bayesian analysis of case history data that I_c correlates poorly with the "clean sand" correction factors needed for CPT-based liquefaction triggering analysis, which suggests that a susceptibility threshold based on I_c = 2.6 may not be reliable. Their results indicate that for soils with an overburden-normalized tip resistance of $q_{c,1}$ > ~ 1 MPa, traditional liquefaction is unlikely to occur if friction ratio R_f> ~ 3%.

Liquefaction Triggering Analysis

The liquefaction triggering analysis procedures used here provide an estimate of cyclic resistance ratio (CRR) based on a measure of penetration resistance (SPT blow count or CPT tip resistance) normalized to 1.0 atm overburden pressure. The current standard of practice for CRR analysis consists of well-known SPT and CPT procedures summarized in Youd et al. (2001). The calculation of seismic demand in terms of a cyclic stress ratio (CSR) in these procedures is performed using ground surface PHA, effective and total stresses at the depth of interest, and stress reduction factors (r_d) by Seed and Idriss (1971) that are a function solely of depth.

New liquefaction triggering procedures for SPT and CPT have been presented by Seed et al. (2001) and Moss and Seed (2002). These procedures differ from the Youd et al. (2001) procedures in that they are based on different data sets (generally larger and more carefully screened) and are fully probabilistic. These procedures also use different r_d models, which are based on statistical interpretation of ground response analysis results. These r_d models are sensitive to depth, depth to groundwater, shear wave velocity in the upper 12 m, and earthquake magnitude.

In the back-analysis of liquefaction/non-liquefaction sites using SPT or CPT procedures, it is necessary to identify a critical layer having the minimum seismic resistance to liquefaction triggering. The identification of this "weakest strata" is ideally performed based on careful study of CPT tip resistance and friction ratio, in conjunction with a boring log with laboratory index testing to evaluate susceptibility. Shown in Figure 2 is a data set used to evaluate the location of the critical layer at an example site that did not show evidence of ground failure. Beginning with the CPT data on the left side of the figure, the critical layer is preliminarily identified as indicated by the dashed lines based on a combination of low q_c (indicating relatively low density) and low R_f (indicating low plasticity). The index properties from the layer (right side of figure) are compared to the LL and w_n/LL components of the Chinese criteria, which in this case suggest the layer is not susceptible to liquefaction. In such cases, additional layers are sought that might be susceptible, and these layers are used for subsequent analysis provided they are not at large depth. In the example, a marginally susceptible zone is identified at about 15 m, but this depth is too great for use with triggering models. Accordingly, since the critical layer is not susceptible, and potentially susceptible layers are deep, data from this site would not be included in data compilations for liquefaction triggering. Identified critical layers that are susceptible to liquefaction are used in the comparisons to liquefaction triggering models described below.

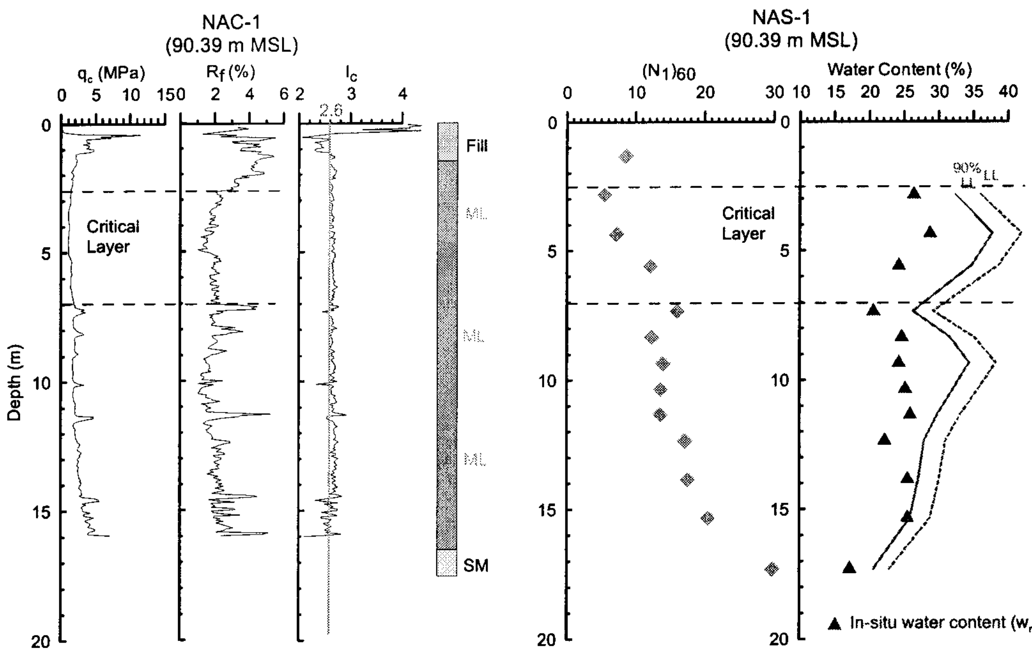

Fig. 2. Example data set from Nantou Site C showing identification of critical layer.

Data Synthesis and Comparison to Susceptibility and Triggering Models

Presented in Table 1 for each investigated site are critical layer depths, mean and standard deviation soil properties within the critical layers, and derived quantities utilized in the triggering analysis models. The statistics on soil properties are evaluated considering both vertical and lateral variability (lateral variability is considered when more than one boring/CPT are available for a site). Also shown are brief descriptions of site performance as observed during post-earthquake reconnaissance. In the following, sites are considered to have "ground failure" when some surface manifestation of ground failure was observed. It should be noted that sites without such surface manifestations of ground failure may have had localized liquefaction that was of little consequence due to the presence of a non-liquefied crust (Ishihara, 1985).

Plotted in Figure 3 are the average soil index properties within the critical layers for the sites in Table 1. The data are plotted as dots at sites with evidence of ground failure, and as open circles for non-ground failure sites. Figure 3(a) shows the data in LL-w_n space along with the boundary curves associated with the Chinese criteria. The results generally support the Chinese criteria (dots in susceptible space, circles in not-susceptible space). Important exceptions are two non-ground failure sites in the susceptible space, which are discussed further below. Figure 3(b) shows the data in LL-CF space. The results shows that some ground failure sites have CF > 15%. Thus, our findings in this regard are consistent with those of Sancio et al. (2002); namely, the CF component of the Chinese criteria appears to be unreliable, whereas the LL and w_n/LL components are generally validated by the Taiwan liquefaction data.

Another important observation relates to the use of CPT-based indices for liquefaction susceptibility evaluations. As shown for example by Site W-N-B8 in Table 1, some soils with I_c < 2.6 and R_f < 3% (indicating relatively granular soils and potentially high liquefaction susceptibility) are *not* susceptible based on the Chinese criteria, and in fact did not show evidence of ground failure. Conversely, Sites W-P-A-W with I_c > 2.6 and R_f > 3% pass the non-CF components of the Chinese criteria and demonstrated evidence of ground failure, although in each case it should be noted that the ground failure involved settlement of rather tall structures (> 4 stories). Accordingly, existing CPT-based indices do not appear to be reliable for evaluating liquefaction susceptibility for such conditions.

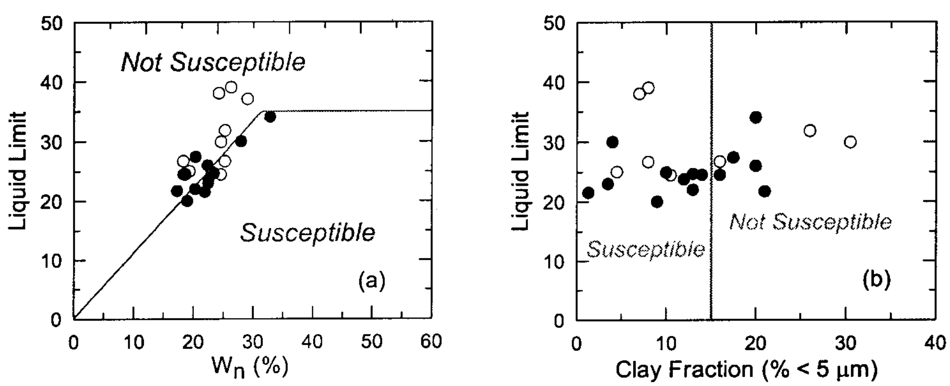

Fig. 3. Ground failure and non-ground failure sites plotted relative to mean soil index properties (dots = ground failure; circles = no ground failure observed)

Table 1. Inventory of data at selected sites in Nantou and Wufeng (mean ± standard deviation values given for data fields)

											Youd et al. (2001) procedures				Seed et al. (2001)/Moss and Seed (2002)					
Boring/CPT No	No.	No. CPT	Z_C (m)	q_C (Mpa)	R_f (%)	$(N_1)_{60}$	FC (%)	LL	ω_n/LL	< 5 μm (%)	CSR	$(N_1)_{60,CS}$	q_{C1N}	I_C	CSR	$(N_1)_{60,CS}$	q_{C1} (Mpa)	Δq_C (Mpa)	$q_{C1,mod}$ (Mpa)	Field Observations
W-P-A-E	1	3	3-4.5	0.8 ± 0.2	0.7 ± 0.6	9 ± 4	39.0 ± 9.9	24.5 ± 8.0	1.0 ± 0.1	10.5 ± 0.7	0.65 ± 0.03	16 ± 5	12.8 ± 2.3	2.6 ± 0.1	0.61 ± 0.01	13 ± 5	1.4 ± 0.2	0.2 ± 0.7	1.7 ± 0.7	No ground failure
W-P-A-W	3	6	5.5-9	2.1 ± 2.1	3.0 ± 0.8	18 ± 10	88.0	26.0 ± 5.7	0.9 ± 0.0	20.0	0.69 ± 0.01	15 ± 7	22.7 ± 19.8	2.8 ± 0.2	0.60 ± 0.02	15 ± 7	2.2 ± 2.0	2.8 ± 0.9	5.1 ± 1.8	Building settlement
W-P-B	1	5	2 - 5.3	2.2 ± 1.3	1.2 ± 5.3	9 ± 5	28.5 ± 8.2			3.5 ± 2.1	0.57 ± 0.03	15 ± 4	29.8 ± 17.3	2.4 ± 0.4	0.52 ± 0.11	12 ± 5	3.3 ± 2.9	0.8 ± 5.6	4.1 ± 8.0	Lateral spread
W-P-C	2	13	0.5 - 6	2.6 ± 2.1	2.4 ± 2.0	5 ± 2	17.6 ± 8.5	24.5 ± 4.9	1.0 ± 0.3	14.0 ± 12.7	0.59 ± 0.10	8 ± 3	39.2 ± 28.5	2.5 ± 0.4	0.62 ± 0.12	7 ± 3	3.7 ± 2.6	2.1 ± 1.9	5.8 ± 2.5	Lateral spread
W-P-D	1	1	1-3.3			13 ± 4	35.3 ± 0.6			12.7 ± 0.6	0.43 ± 0.00	20 ± 5				16 ± 5				No ground failure
W-P-E	1	1																		Building settlement
N-P-A1	1	1	1.5 - 7	1.3 ± 0.2	1.9 ± 0.3	8 ± 3	66.7 ± 12.6	39.0 ± 3.0	0.7 ± 0.1	8.0 ± 0.0	0.39 ± 0.02	15 ± 4	19.6 ± 2.3	2.7 ± 0.0	0.40 ± 0.01	14 ± 5	1.8 ± 0.2	1.4 ± 0.3	3.2 ± 0.5	No ground failure
N-P-A2	1		3.8-5.5			15 ± 2	10.0 ± 9.9	23.0 ± 1.4	1.0 ± 0.1	3.5 ± 3.5	0.39 ± 0.01	17 ± 1								Building settlement
N-P-A3	1		1 - 2			10 ±	14.0	30.0	0.9	4.0	0.28	12				11				Building settlement
N-P-A4	1		1.5-2.5			5 ±	35.0	20.0	1.0	9.0	0.33	11				8				Building settlement
N-P-B1	1		1.6-7.3			11 ± 6	37.4 ± 8.2	26.7 ± 3.2	1.0 ± 0.2	8.0 ± 6.7	0.38 ± 0.04	18 ± 7				15 ± 7				No ground failure
N-P-B2	1		1.7-3			8 ±	50.0	22.0	0.9	13.0	0.34	14				12				Building settlement
N-P-B3	1		3-5.5			29 ± 10	4.0 ± 1.0	21.5 ± 3.3	1.0 ± 0.2	1.3 ± 0.6	0.38 ± 0.01	29 ± 10				30 ± 10				Building settlement
N-P-B4	1		2-6.3			15 ± 1	21.3 ± 9.1	38.0	0.8	7.0 ± 4.7	0.37 ± 0.04	19 ± 4				17 ± 2				No ground failure
N-P-B5	1		3.2-6.5			19 ± 9	19.0 ± 0.6	25.0 ± 1.4	0.8 ± 0.1	4.5 ± 2.1	0.37 ± 0.03	23 ± 11				21 ± 11				No ground failure
N-P-C	2	3	1.5-3.4	4.8 ± 2.9	1.2 ± 1.1	13 ± 0	10.0 ± 0.5			3.8 ± 2.2	0.38 ± 0.02	13 ± 3	73.3 ± 62.8	2.1 ± 0.5	0.36 ± 0.10	11	7.3 ± 6.1	0.6 ± 1.0	7.9 ± 5.7	Lateral spread
W-N-B1	1		1.5-2.5			32	10.0			4.0	0.55				0.52 ± 0.11					Building settlement
W-N-B3	1		0.6-3.1			3	22.0			6.0	0.57	7				4 ±				Sand boils
W-N-B4	1		1-2.5			6	76.0	31.8	0.8	26.0	0.43	12				11 ±				No ground failure
W-N-B5	1		1.5-4.5			13 ± 6	67.5 ± 31.8	27.4	0.9	17.5 ± 4.9	0.62 ± 0.03	20 ± 7				19 ± 5				Building settlement
W-N-B6	1		0.2-4			11 ± 7	65.3 ± 30.5	29.9 ± 10.7	0.9 ± 0.2	30.5 ± 6.4	0.45 ± 0.03	18 ± 8				16 ± 6				No ground failure
W-N-B7	1	1	0.5-5	4.1 ± 3.1	1.6 ± 0.6	11 ± 4	34.5 ± 20.5			7.0 ± 4.2	0.44 ± 0.01	17 ± 2	54.8 ± 35.8	2.3 ± 0.3	0.43 ± 0.02	14 ± 2	5.2 ± 3.3	1.1 ± 0.6	6.4 ± 3.4	Sand boils
W-N-B8	1	1	5-9.5	5.7 ± 2.4	1.3 ± 0.7	16 ± 7	54.7 ± 17.6	26.7	0.6	16.0 ± 7.0	0.75 ± 0.01	24 ± 9	64.4 ± 26.4	2.2 ± 0.3	0.66 ± 0.03	21 ± 7	6.2 ± 2.5	0.9 ± 0.8	7.1 ± 2.3	No ground failure
W-N-B9	1		0 - 2.3			7	38.0 ±			9.0	0.43	13				10				Sand boils
W-N-B10	1	1	2 - 6	1.4 ± 0.9	1.8 ± 1.0	13 ± 1	45.5 ± 33.2	23.8 ± 0.4	0.9 ± 0.3	12.0 ± 11.3	0.65 ± 0.04	19 ± 0	19.3 ± 12.0	2.7 ± 0.2	0.56 ± 0.01	17 ± 2	1.9 ± 1.1	1.4 ± 1.1	3.3 ± 1.7	Sand boils
W-N-B11	1		1.6-3.6			10	45.0			9.0	0.64	17				14				Sand boils
W-N-B12	1		8 - 14			17 ± 4	40.5 ± 1.7	24.5 ± 0.8	0.8 ± 0.0	16.0 ± 1.6	0.54 ± 0.01	22 ± 5				26 ± 5				Building settlement
W-N-B13	1		1 - 3.5			4 ± 1	34.0 ± 5.7			9.0 ± 2.8	0.57 ± 0.11	9 ± 2				6 ± 2				Building settlement
W-N-B14	1		1 - 2			15	21.0			7.0	0.43	20				18				Sand boils
W-N-C9		1	1 - 2.8	0.8 ± 0.3	0.5 ± 0.5						0.55 ± 0.06		13.1 ± 5.7	2.6 ± 0.2	0.56 ± 0.06		2.1 ± 0.9	0.0 ± 0.5	2.1 ± 0.8	Sand boils
W-N-C15		1	2.2-2.8	0.7 ± 0.2	1.9 ± 0.9						0.44 ± 0.01		10.5 ± 3.6	2.9 ± 0.2	0.42 ± 0.01		1.0 ± 0.3	1.4 ± 0.9	2.4 ± 0.9	Sand boils
W-N-C16		1	2.5-5.5	2.2 ± 1.6	2.8 ± 1.1						0.56 ± 0.04		30.1 ± 21.7	2.7 ± 0.2	0.54 ± 0.03		2.7 ± 2.0	2.5 ± 1.2	5.2 ± 2.1	No ground failure
N-N-B1	1	1	1 - 6	1.4 ± 0.4	5.1 ± 0.7	7 ± 1	94.7 ± 1.5	37 ± 8	0.8 ± 0.1	57.7 ± 5.1	0.25 ± 0.00	13 ± 2	18.2 ± 7.7	3.0 ± 0.2	0.24 ± 0.01	14 ± 2	1.7 ± 0.6	3.7 ± 0.6	5.4 ± 1.0	No ground failure
N-N-B2	1		1 - 2.5			10	26.0			6.0	0.25	15			0.25	12				Sand boils
N-N-B3	1		3 - 9.4			16 ± 11	17.8 ± 1.7			8.8 ± 1.5	0.30 ± 0.02	21 ± 11			0.29 ± 0.02	18 ± 12				Sand boils
N-N-B4	1	1	2.4 - 8	3.9 ± 2.5	0.7 ± 0.5	7 ± 0	33.5 ± 12.0			9.0 ± 2.8	0.25 ± 0.01	13 ± 1	38.4 ± 22.1	2.2 ± 0.3	0.24 ± 0.00	10 ± 1	9.0 ± 2.1	0.2 ± 0.4	3.9 ± 2.1	Sand boils
N-N-B5	1		0.5-7.5			8 ± 4	65.0 ± 39.0			15.0 ± 14.7	0.31 ± 0.05	14 ± 4			0.29 ± 0.03	14 ± 6				Sand boils
N-N-B6	1		1.5-11			20 ± 4	25.5 ± 11.7			6.7 ± 2.3	0.37 ± 0.01	26 ± 5			0.32 ± 0.02	23 ± 4				Lateral spread
N-N-B7	1	1	0.8-11.5	7.1 ± 3.3	1.2 ± 0.9	17 ± 6	40.3 ± 30.5			13.5 ± 19.9	0.43 ± 0.01	24 ± 7	73.3 ± 31.0	2.1 ± 0.3	0.42 ± 0.01	22 ± 7	7.3 ± 3.1	0.7 ± 0.9	7.9 ± 3.1	Lateral spread
N-N-B8	1		0 - 2			4	40.0	24.9	0.9	10.0	0.27	10			0.27	7				Sand boils
N-N-B9	1		7 - 11			33 ± 35	27.7 ± 16.2			10.3 ± 7.6	0.23 ± 0.00	41 ± 39			0.21 ± 0.01	38 ± 38				No ground failure
N-N-B10	1		0 - 5.5			10 ± 7	24.7 ± 1.2			6.3 ± 0.6	0.28 ± 0.03	15 ± 8			0.27 ± 0.03	12 ± 8				Sand boils
N-N-B11	1		1.8 - 5.8			5 ± 1	92.5 ± 9.2	34.1 ± 3	1.0 ± 0.0	20.0 ± 7.1	0.30 ± 0.02	11 ± 1			0.29 ± 0.02	12 ± 1				Building settlement
N-N-B12	1		1.5- 4			8 ± 6	87.0 ± 4.2	24.7 ± 2	1.0 ± 0.1	13.0 ± 8.5	0.26 ± 0.02	15 ± 8			0.26 ± 0.02	15 ± 9				Sand boils
N-N-B13	1	1	1.8 - 5			26	76.0	21.7	0.8	21.0	0.30	36			0.29	38				Building settlement
N-N-B14	1	1	1 - 8.5			16 ± 4	19.8 ± 12.2			5.0 ± 1.8	0.35 ± 0.07	21 ± 2			0.32 ± 0.05	19 ± 3				Sand boils
N-N-C2		1	3 - 5	4.8 ± 1.3	0.6 ± 0.6						0.24 ± 0.00		53.9 ± 12.0	2.0 ± 0.2			5.4 ± 1.2	0.1 ± 0.5	5.5 ± 0.9	Sand boils
N-N-C3		1	1.5 - 3.5	2.4 ± 0.9	2.7 ± 0.8						0.30 ± 0.03		39.7 ± 13.7	2.5 ± 0.2			3.5 ± 1.2	1.9 ± 0.7	5.4 ± 1.0	Sand boils
N-N-C13		1	10-11.5	3.8 ± 0.4	1.6 ± 0.7						0.37 ± 0.00		34 ± 4.2	2.4 ± 0.1			8.8 ± 0.4	8.6 ± 0.6	7.9 ± 0.6	No ground failure

Plotted in Figure 4 are average CSR-penetration resistance data within the critical layers for selected sites in Table 1. Also shown are the CRR models discussed previously. In the case of the probabilistic models, the CRR curves shown apply for a 20% probability of liquefaction. Only site data that passes the LL and w_n/LL components of the Chinese criteria are plotted in Figure 4. Generally, the upper band of results at CSR ≈ 0.6 are Wufeng sites, whereas the lower band at CSR ≈ 0.4 are Nantou sites. Note that most of the data plotted in Figure 4 are dots (indicating ground failure sites). We actively sought non-ground failure sites during our field work, but many such sites are not susceptible to liquefaction based on soil plasticity.

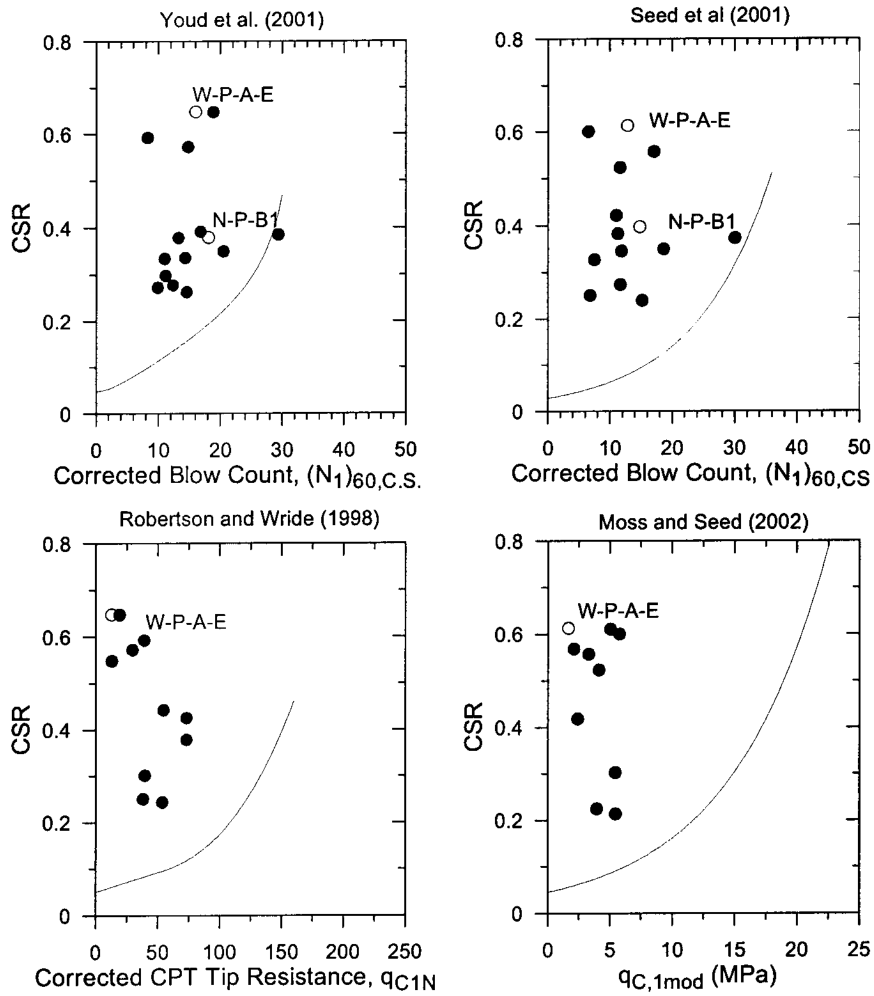

Fig. 4. Ground failure and non-ground failure sites plotted relative to existing liquefaction triggering models (dots = ground failure; circles = no ground failure observed)

The ground failure sites (dots) are generally encompassed by the CRR models, and it is not possible to judge the relative accuracy of these models based on the Taiwan data processed to date. The non-ground failure sites (open circles) that plot to the left of the CRR lines in Figure 4 merit additional discussion (i.e., sites W-P-AE and N-P-B1). These sites also correspond to the open circles within the "susceptible" space in Figure

3. Site W-P-AE consists of a ~2-3 m clay bed overlying a thick deposit of relatively sandy soil. The critical layer in this case was taken as the upper section of sand. Site N-P-B1 consists of ~1.5 m of unsaturated soil overlying ~2 m of silty sand, which was taken as the critical layer. Both sites have essentially "free-field" conditions – namely, the absence of tall structures (local structures at these sites are light, single story buildings). Both sites were observed within 2 weeks of the earthquake by reconnaissance team members, who report no evidence of ground failure in the area.

An appropriate question to ask is whether these sites may have in fact liquefied. Both of the SPT sites would be expected to have surface manifestation of liquefaction based on the relative thicknesses of the surface layer and liquefiable layer per the criteria of Ishihara (1985). Nonetheless, we speculate that the absence of ground failure does not necessarily mean the absence of liquefaction within the critical layers, especially since these layers are overlain by non-liquefiable strata, the sites lacked driving static shear stresses that could mobilize significant ground failure effects through the overlying strata, and the fines contents of the liquefiable soils at depth may be higher than those considered by Ishihara (1985). The driving static shear stresses appear to be particularly important, as most of the ground failure sites in our database *did* have significant static shear stresses. Accordingly, this situation raises fundamental questions about how a site is classified within the "ground failure" or "non-ground failure" categories as a result of observations from post-earthquake reconnaissance.

While there is no question about assigning "dots" to sites with ground failure, a "circle" denoting non-ground failure is strictly only applicable for the site-specific stress conditions, and caution must be exercised in assuming that other sites with similar combinations of CSR and penetration resistance would not liquefy if the static stress conditions were different. Moreover, it is noted that the CRR models presumably apply for a zero static shear stress condition, but for the reasons discussed above, the development of triggering models based on such protocols is likely unconservative for the high fines content materials commonly encountered in Nantou and Wufeng. This issue warrants further research in the development of next-generation triggering models.

Summary of Findings

Case histories of ground failure and non-ground failure from the Chi-Chi earthquake are important for the ongoing development of liquefaction susceptibility and triggering models because the effected soils have large fines contents and marginal plasticity levels, and because the earthquake induced large CSRs in these soils. Prior data sets had a paucity of data for such conditions. Comparisons of the data to models indicate that:

1. The CF component of the Chinese criteria appears to be unreliable, whereas the LL and w_n/LL components are generally validated by the Taiwan data. This is similar to the findings of Sancio et al. (2002) for soils in Adapazari, Turkey.
2. CPT-based indices appear unreliable for evaluating liquefaction susceptibility. It is noted that CPT indices for susceptibility are not formally proposed by Robertson and Wride (1998) or Moss and Seed (2002). Nonetheless, I_c has been applied in practice for this purpose, and this practice is not recommended.
3. Ground failure sites from Wufeng and Nantou are generally encompassed by available CRR models (i.e., the CSR values generally plot above the CRR line).

4. Some non-ground failure sites plot above the CRR lines, which raises questions regarding the nature of the site performance (did the site liquefy in a manner that was not manifest at the surface?) and the degree to which the apparently good site performance may be an adequate predictor of future performance at other sites. These questions remain unanswered, but are important for the ongoing development of robust empirical liquefaction triggering models.

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