Apr 29th - May 4th

Liquefaction Resistance of Gravelly Soils

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Commemorate the Legacy of Ralph B. Peck
April 29 - May 4, 2013 CHICAGO
Seventh International Conference on Case Histories in Geotechnical Engineering
and Symposium in Honor of Clyde Baker

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ABSTRACT

Liquefaction assessments of gravels and soils that contain a large gravel fraction are difficult. Undisturbed (intact) sampling of these soils is problematic and laboratory testing carried out on reconstituted samples or on frozen samples obtained from the field is time consuming, expensive, and interpretation of the results requires considerable judgment. Because of these and other issues, for a remote site in British Columbia, Canada (aka "Study Site"), it was decided to carry out the liquefaction potential assessment using existing published relationships and case history data on similar soils.

This case history describes the approach utilized, including material mechanical properties, measured shear wave velocities and insitu density data obtained from shallow test pits excavated across the study site. Comparisons to published data on similar soils are discussed. To assess the liquefaction potential of the gravels, normalized shear wave velocity data were related to void ratio. The void ratio was then related to the CRR using published relationships on a similar gravelly soil tested in the laboratory. The liquefaction potential was assessed in the conventional manner comparing the cyclic resistance ratio (after appropriate consideration of correction factors used in laboratory cyclic testing) to the seismic demand (CSR). The approach described in the case history generalizes the methodology for application to other gravel deposits at other sites.

INTRODUCTION

The existing industrial site has been active since the early 1950s. In terms of seismic performance, the site has essentially operated without incident. Over the years additions, retrofits, and various expansion upgrades have taken place, with the designs following local and regional building codes. In the last ten years, Canada, as well as many other regulatory agencies and countries, have gone through significant revision to their seismic design criteria. The details are not discussed here, but the end result for this site is a substantial increase in the seismic hazard.

To investigate the impact that the revised seismic design criteria had on the seismic liquefaction potential of the gravel deposits at the site for a proposed new structure, a limited-scope investigation was carried out to estimate the cyclic resistance ratio (CRR) of the insitu deposits with specific mechanical properties as measured in shallow test pits and in the laboratory with representative samples. Specifically, the procedure used to characterize the cyclic resistance ratio (CRR) for the upper sand and gravel deposits will be described. No sophisticated laboratory dynamic testing was carried out for this evaluation.

SITE AND SUBSURFACE CONDITIONS

The site is located approximately 30 km inland from the Pacific coast line, about 650 km north of Vancouver, British Columbia, Canada. The site lies in a fjord-like valley that most likely has been shaped by glaciers. The geology of the site consists of alluvial, post-glacial deposits overlying glacial till and bedrock. Depth to bedrock varies from less than 30 m to over 200 m, with rock dipping eastward toward the natural river channel. Figure 1 summarizes the subsurface conditions along a generalized profile depicting the subsurface strata beneath the structure under evaluation. Standard penetration test (SPT) N-values in the gravelly deposits were interpreted following the methodology derived by Vallee and Skryness (1979) as described further by Andrus (1994) and Mejia (2007, as reported by Idriss and Boulanger [2008]).

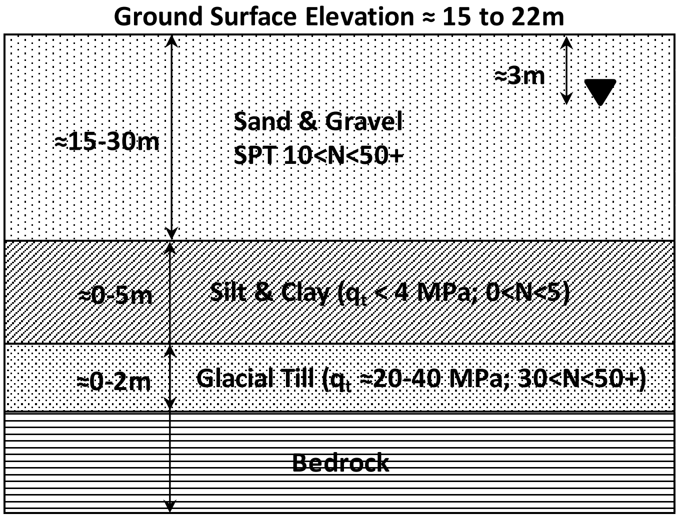

Fig. 1. Generalized Subsurface Profile

The sands and gravels are medium dense to dense, with occasional cobbles. Although SPT N-values are not necessarily indicative of dense material, due to gravel and cobbles, they can be indicative of loose sands if present. In this case, zones of loose and very loose materials (N < 10) were present. The CPT could not be advanced in the sand and gravel layer. Pre-drilled holes through this layer allowed CPT data to be acquired at depth.

Classification tests showed that the vast majority of the sands and gravels classify as well-graded clean sands and gravels (SW and GW) to silty sands (SM) and gravels (GM and GP). The silts and clays, and glacial till were not continuous and they were generally encountered just above bedrock at the structure location. Site-wide, the silts and clays encountered were much more extensive, and slightly overconsolidated near the surface, perhaps due to desiccation, decreasing to normally consolidated at depth. The corrected seismic piezocone penetration testing ($SCPT_u$) tip resistance (q_t) of the silts and clays was about 3.6 MPa, with a standard deviation (σ) of about 1.4 MPa, and an average undrained shear strength (S_u) of about 140 kPa. However, areas of very soft to soft silts and clays were also encountered. Classification tests indicated that about half of the samples were clays (CL) and the remainders were silts (ML) and silt-clay mixes (CL-ML).

The ground surface ranges from about elevation (el.) 15 to 22 m, and the depth to the groundwater table is approximately 3 m. It is noted that although the generalized conditions depicted in Figure 1 appear uniform, the site itself is far from uniform. Rock dips from west to east (toward the river channel) and the alluvial deposits vary in a similar manner. This situation was confirmed by adjacent boreholes and cone soundings across the entire site, which often showed dramatic differences in results over short distances. Lateral variability was particularly true for the upper sands and gravels, but applied throughout the entire profile.

MATERIAL PROPERITES

To assess the insitu material properties of the shallow gravels, a series of eight test pits and 14 surface seismic lines (spectral analysis of surface waves [SASW]) were carried out across the entire site (not just the structure under evaluation). The test pits were excavated up to 3 m deep (depth to groundwater) and insitu density tests were done at various depths. Twenty-six samples were taken for grain size analysis (generally 3 samples/test pit, however two test pits had four samples). Laboratory density testing (ASTM D1557) was carried out on two composite samples from two test pits. Shear wave velocity (V_s) testing was done using the SASW method at 14 locations across the entire site. The locations were chosen to assess site variability and to be near the test pit locations. In addition, exiting V_s measurements at downhole locations (at borings and $SCPT_u$ (where they could be advanced) were also utilized.

Figure 2 shows the grain size distribution of sandy gravel samples obtained from the 8 test pits. It is observed that the sand content (particle size equal to 4.75 mm and less) of the samples varies between about 60% and 50% (mean ≈ 57%). The mean uniformity coefficient (D_{60}/D_{10}) of all the data is about 30. Based on Siddiqi et al., (1987) Evans and Zhou (1995), and Lin and Chang (2002), these sand concentrations suggest that the cyclic strength of the composites is primarily matrix-controlled (the sand-sized fraction).

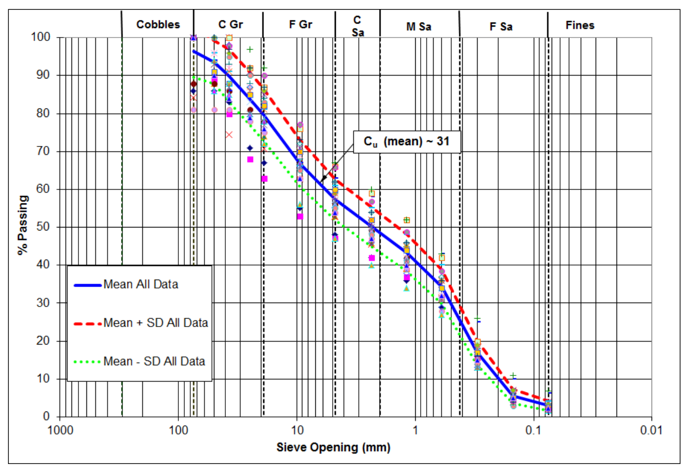

Fig. 2. Grain Size Distribution of Test Pit Samples

The gradation results show good consistency over all depths and test pit locations, giving us confidence that at least in the upper 3m or so of the profile, the sands and gravels across the site (and in the area of the structure under evaluation) had similar gradation characteristics. Limited SPT sampling in the gravels between depths of 3 and 20m confirms similar grain size characteristics, however consideration of the sampler size tempers the results somewhat.

Table 1 shows the mean results from the laboratory testing on samples obtained from the eight test pits. Field density testing was done at depths of 1, 2, and 3 m. The field dry density shown in Table 1 is the mean of 24 field tests. The measured dry density from the 24 tests ranged from 1928 kg.m^3 to 2313 kg/m^3. The average insitu dry density and moisture content of the samples taken were 2189 kg/m^3 and 4.6%, respectively, resulting in an average total unit weight (γ_{total}) of about 2290 kg/m^3. The standard deviation (σ) for the field dry density measurements was 104.5 kg/m^3. Maximum and minimum densities were also determined using ASTM D4254 for two samples, with the average maximum density being 2089 kg/m^3 and the average minimum density being 1780 kg/m^3. For the purposes of analysis, we used the average maximum density obtained from ASTM D1557 (2189 kg/m^3) and the average minimum obtained from ASTM D4254 (1780 kg/m^3) going forward.

Table 1. Laboratory Results From Test Pit Samples

Test Pit (TP)	Field Dry Density (kg/m^3)	Max Dry Density (kg/m^3)	Min Dry Density (kg/m^3)	C_u	D_{50} (mm)	Void Ratio e
All (8 TP at 3 depths each; 1, 2, & 3 m)	2189 σ = 104.5	2170	1780	31	2.4	0.26

Void ratios within the test pits varied between 0.19 and 0.43 with an average of about 0.26. The void ratio was computed assuming a specific gravity of 2.75. It is noted that the average density values reported in Table 1 indicate the field dry density is higher than the laboratory-determined dry density. This most likely due to the gravel corrections made for both the field and laboratory tests, and the fact that there were 24 field tests and only two laboratory-determined maximum density tests carried out.

Figure 3 shows the shear wave velocity measurements taken within the footprint of the proposed structure under evaluation. The solid lines are data from two surface SASW locations and the dashed lines represent data from the two downhole locations. The high shear wave velocities between elevation -10 and -30 m is the top of rock, which does vary beneath the proposed structure. It is noted that the results appear reasonably consistent except for downhole location B-2, which shows a lower shear wave velocity at depths below el. 8 m.

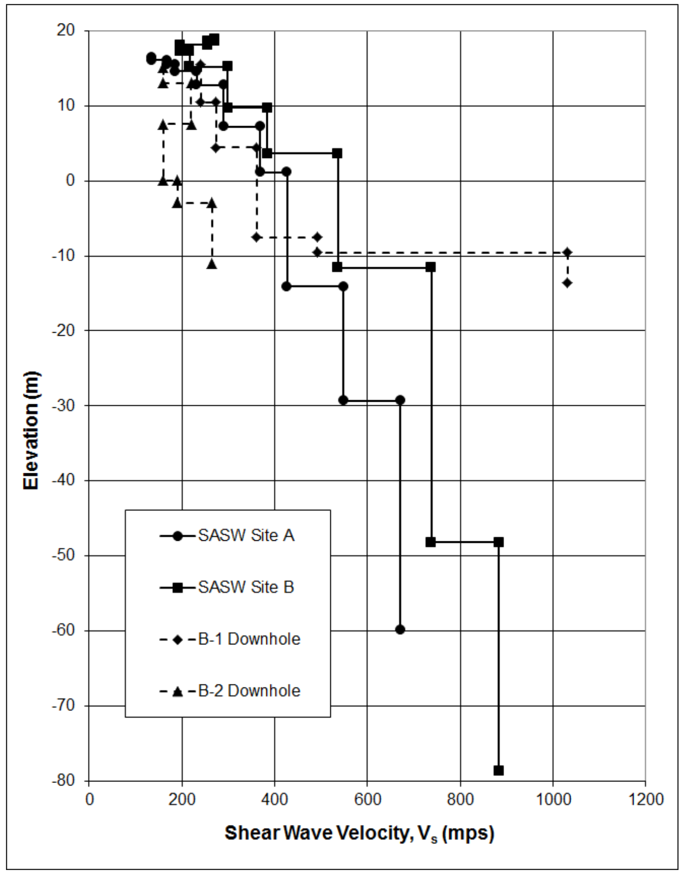

Fig. 3. Measured Shear Wave Velocity Results; Proposed Structure

Table 2 shows the measured (field) shear wave velocities (V_s) at the four locations (two SASW and 2 downhole) beneath the footprint of the structure under evaluation at the study site. The velocities shown are the ranges (interpreted low and high) and mean results in the sand and gravel deposits, at a depth of approximately 3 m. The data, designated with a B- or S-, are from downhole measurements and SASW measurements, respectively. The overall mean value of 213 m/sec is believed to be reasonable for this structure location and not unduly conservative (low).

Table 2. Field Shear Wave Velocity (V_s) Results

Hole/SASW Location	Low m/sec	High m/sec	Mean m/sec
BK07-05	160	219	190
B10-04	219	268	244
SASW10	134	232	183
SASW12	195	271	233
Mean	177	248	213

To normalize the V_s results in Table 3 to an overburden pressure of 1 atm, the following relationship was used (proposed by Andrus and Stokoe, 2000 for sands);

$$V_{s1} = V_s \left(\frac{P_a}{\sigma'_v} \right)^{0.25} \quad (1)$$

where, P_a is the atmospheric pressure and σ'_v is the insitu effective overburden pressure at a depth of 3m, both in the same units. The average field shear wave velocity (213 m/sec) corresponds to a normalized (V_{s1}) velocity of about 264 m/sec.

ANALYTICAL METHODOLOGY

Hardin and Richart (1963) and Hardin and Drnevich (1972) carried out extensive laboratory tests on granular materials to find a correlation between small-strain shear modulus, G_{max}, (and ultimately V_s, aka "analytical" V_s in this paper) and void ratio. To evaluate the shear wave velocities, the investigators performed a series of 46 resonant column tests on numerous specimens prepared from crushed quartz, Ottawa sand, and crushed quartz silt. The important conclusion derived from their studies was that, for a given confining pressure, the void ratio was the most significant variable, i.e., the wave velocities varied almost linearly with void ratio (e). The effects of relative density, grain size and gradation entered only through their effects on void ratio. Thus, for example, the velocities at 100% relative density may be quite different for two sands; however, their velocities are essentially the same when they are tested at the same void ratio.

Menq (2003) carried out similar studies and determined that the shear modulus was also correlated to void ratio, among other parameters. Menq concluded that confining pressure, void ratio, grain size, and uniformity coefficient (C_u) were all important parameters that affected G_{max}.

Thus, if G_{max} could be reliably determined, V_s can be calculated and then correlated to the void ratio determined from computations based on the results from the eight shallow test pits. With the void ratio-V_s correlation established, this opens up the possibility for a correlation to the cyclic strength ratio (CRR) via the void ratio determined on similar gravel deposits. Both the Hardin and Drnevich and Menq procedures are described below.

Hardin and Drnevich (1972)

As reported by Seed, et al., (1984), the following relationship was utilized to compute G_{max} (modified from Hardin and Drnevich [1972] to account for change in units):

$$G_{\max} = 14{,}760 \cdot \frac{(2.973 - e)^2}{1 + e} \cdot OCR^a \cdot \sqrt{\sigma'_m} \quad (2)$$

where G_{max} is the low-strain shear modulus in pounds/ft^2, e is the void ratio, *OCR* is the overconsolidation ratio (taken as 1 in this study), a is a parameter that depends on the plasticity of the soil (in this case = 0), and σ'_m is the mean principal effective confining stress in pounds/ft^2, computed as follows:

$$\sigma'_m = \left(\frac{1 + 2k_0}{3} \right) \sigma'_v \quad (3)$$

where k_0 is the at-rest earth pressure coefficient (taken as 0.5), and σ'_v is the vertical effective stress, taken as 1 atm (100 kPa ≈1 ton/ft^2). The resulting shear wave velocity at 1 atm overburden pressure (assumed to be normalized at 1 atm [V_{s1}]) was calculated using:

$$V_{s1} = \sqrt{\frac{G}{\rho}} \quad (4)$$

where ρ is the mass density of the soil. The resulting "analytical V_s at 1 atm vertical effective stress and a void ratio of 0.26 (average for the upper gravel deposit) is 263 m/sec.

Menq (2003)

As reported by Menq (2003), G_{max} is predominately a function of confining pressure, void ratio, and the D_{50} size of the deposit, as follows:

$$G_{\max} = C_{G3} \cdot C_u^{b1} \cdot e^x \cdot \left(\sigma'_0 / P_a \right)^{n_G} \quad (5)$$

where C_{G3}=67.1 MPa (1400 kips/ft^2), $b1$=-0.20, x=-1-$(D_{50}/20)^{0.75}$, and n_G=0.48$(C_u)^{0.09}$. Utilizing the mean properties from the test pit testing given in Table 2, G_{max} and V_s can be computed. (3)

The resulting "analytical" V_s at 1 atm vertical effective stress, void ratio of 0.26, C_u (uniformity coefficient) of 31, and D_{50} of 2.4 is 278 m/sec.

Upon comparing the mean normalized shear wave velocity from the measured V_s data to the "analytical" (computed velocities) using the mean properties from the eight test pits, one finds that the ratio of the "analytical" (computed) to the measured velocities (m/sec) is 263/264 ~ 1 using the Hardin and Drnevich relationship and 278/264 ~ 1.05 for Menq's relationship. The results indicate that, using the mean test pit properties and a void ratio of 0.26, the two approaches are quite comparable.

The relation of V_{s1} with void ratio is shown in Figure 4 for these two relationships. The results are shown for a vertical effective stress (σ'_v) of 1 atm (≈ 1 ton/ft^2) [a mean effective confining confining stress, σ'_m, of ⅔ atm (67.6 kPa, ~ 1410 pounds/ft^2)]. Thus, the shear wave velocity is interpreted as normalized (V_{s1}). This relationship can now be used to estimate the insitu void ratio of the gravel given a measured V_{s1} in the field. Although the variation between the two relationships is shown in Figure 4 as the void ratio changes, both are used going forward for analysis. The next step is then to relate void ratio with CRR, which is presented next.

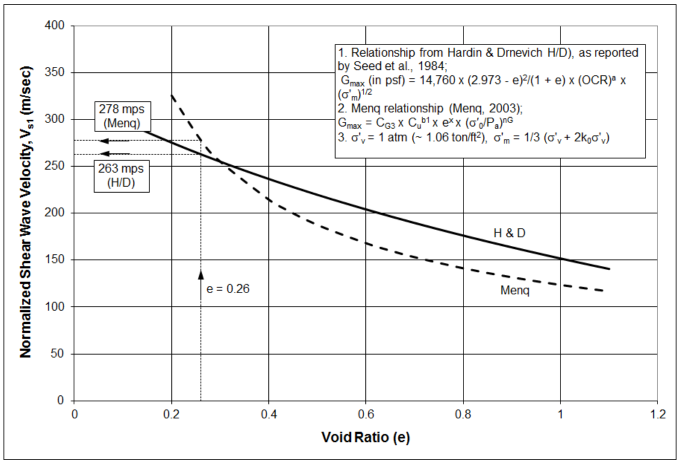

Fig. 4. Predicted Variation in V_{s1} with Void Ratio

EVALUATION OF LABORATORY CYCLIC RESISTANCE RATIO (CRR)

Previous investigations on the cyclic resistance of granular materials have shown several important findings. Mulilis et al., in 1975 investigated the effects of several methods of sample preparation on the cyclic stress-strain and liquefaction resistance of sands. Several methods of sample preparation were investigated including; pluviation through air, pluvial compaction through water, high and low frequency vibrations applied horizontally and vertically to dry and moist specimens, compaction by dry and moist tamping and rodding. The researchers concluded that the dynamic strength of saturated sands, remolded by different compaction procedures to the same relative density may be significantly different. For example, the researcher's results showed that the cyclic strength of samples prepared by dry or moist rodding or tamping was up to 33% higher than samples prepared using pluviation through water (likely the deposition process of the deposits at the current site site).

Evans and Zhou in 1995 concluded from their research that: (1) for the soils investigated, and gravel contents between 0% and 60%, it appeared that gravel particles floated in the sand matrix and that there was little contact between them; (2) there was no direct relationship between the cyclic resistance and relative density of the composite or matrix for various gravel contents (a finding that corroborates the conclusion derived by Hardin and Richart, 1963); and, (3) the researchers concluded that the cyclic loading resistance of the sand-gravel tested in their study could be estimated by testing the matrix sand alone at an "equivalent field density" that accounts for the presence of the gravel.

A comprehensive search and evaluation was carried out of technical literature that reported results of cyclic triaxial tests and of the laboratory-obtained cyclic resistance ratio of gravel and gravel/sand mixtures. The data were screened and only the materials that exhibited matrix-controlled behavior under cyclic loading were selected. Table 3 identifies the sources and relevant geotechnical and engineering properties. Columns 1 through 12 in the table summarize the physical properties of the granular materials (including in ascending order): material identification, unified soil classification system (USC) designation, maximum particle size, maximum and minimum void ratios, percent material passing No.4 ASTM sieve size, mean diameter (D_{50}), uniformity coefficient (C_u), specific gravity (G_s), void ratio of test sample (e_{test}), percent of gravel-sized particles, and test relative density (D_r). Column 13 presents the test effective confining pressure, and columns 14 and 15 present the cyclic stress ratio obtained after 10 and 20 loading cycles, respectively. Column 16 documents the respective references. It is noted that the maximum, the minimum and the as-tested void ratios were calculated based on the reported values of the maximum and minimum relative densities and the specific gravity of the materials.

A summary evaluation of the data given in Table 3 is presented below.

<u>Dynamic Strength of the Tehran Alluvium (Haeri et al., 2008)</u>

The northern and central parts of the City are underlain by sandy gravels and gravelly sands. Briefly, the material contains 45% and 49% gravel-sized and sand-sized particles, respectively, and has a C_u of 28. To investigate the dynamic strength properties of the granular material, 34 cyclic triaxial tests were carried out on samples prepared in the laboratory at a relative density, D_r, equal to 65%. Specimens were isotropically consolidated at three different confining pressures and sheared at various amplitudes of cyclic deviator stresses, at a loading frequency equal to 1 Hz. Tests results were properly corrected for membrane compliance effects (Evans and Seed [1987] and Nicholson et al., [1989]).

Table 3. Case Histories of Cyclic Triaxial Tests on Soils with Gravel

Material (1)	USCS (2)	Max Size- (in.) (3)	e_{max} (4)	e_{min} (5)	No. 4 (%) (6)	D_{50} (mm) (7)	C_u (8)	G_s (9)	e_{test} (10)	Gravel (%) (11)	D_r % (12)	σ_3c (kg/cm^2) (13)	CRR 10 Cyc. (14)	CRR 20 Cyc. (15)	Ref. (16)
Tehran Alluvial	SW-SM	0.5	0.56	0.34	55	4.00	28	2.57	0.43	45.0	65	1	0.30	0.28	Haeri et al 2008
	SW-SM	0.5	0.56	0.34	55	4.00	28	2.57	0.43	45.0	65	3	0.24	0.22	
	SW-SM	0.5	0.56	0.34	55	4.00	28	2.57	0.43	45.0	65	5	0.24	0.23	
Tokyo Gravel	(GW-GP)	3-1/8	0.43	0.22	~40	8.80	70	2.69	0.33	~60	-	3	0.39	0.36	Hatanaka, et al 1988
	(GW-GP)	3-1/2	0.43	0.22	~30	14.60	64	2.69	0.31	~70	55	3	0.18	0.16	
	(GW-GP)	3-1/8	0.46	0.26	~50	5.60	122	2.69	0.35	~50	-	3	-	-	
	(GW-GP)	-	0.44	0.23	-	-	-	-	0.27	-	80	3	0.34	0.28	
	(GW-GP)	3-1/8	0.43	0.22	-	9-15	60-70	2.69	0.26	-	54	3	0.39	0.36	
Taiwan Gravel	SP	1	0.51	0.33	40	7.00	33	2.71	0.43	60.0	40	1	0.48	0.43	Lin et al 2004 and Lin & Chang 2002
	SP	1	0.53	0.34	60	0.63	5	2.69	0.47	40.0	40	1	0.42	0.36	
	GP	1	0.68	0.44	80	0.55	4	2.68	0.57	20.0	40	1	0.35	0.29	
	SP	1/8	0.82	0.54	100	0.49	14	2.66	0.68	0.0	50	1	0.33	0.28	
	SP	1/8	0.82	0.54	100	0.49	14	2.66	0.62	0.0	70	1	0.40	0.37	
	SP	1/8	0.82	0.54	100	0.49	14	2.66	0.70	0.0	40	1	0.29	0.23	
	SP	1/8	0.82	0.54	100	0.49	14	2.66	0.68	0.0	60	1	0.37	0.32	
	GP	1	-	-	47	8.00	99	2.71	0.54-0.46	53.0	31	1	0.41	0.28	
	SP	1	0.53	0.34	-	-	-	-	0.63	40	20	1	0.32	0.29	
	SP	1	0.53	0.34	60	0.63	5	2.69	0.47	40	40	1	0.41	0.36	
	SP	1	0.53	0.34	-	-	-	-	0.40	40	60	1	0.56	0.49	
Aswan Riverbed	SP	1/8	0.80	0.24	100	0.25-0.35	2	2.65	0.53	0.0	48.5	1	0.30	0.26	WCC 1985
	SP	1/8	0.80	0.24	100	0.25-0.35	2	2.65	0.38	0.0	70.0	1	0.62	0.52	
Aswan Dune Sand	SP	1/8	0.77	0.45	100	0.34	2.3	2.66	0.61	0.0	50.0	1	0.34	0.29	WCC 1985
	-	-	-	-	-	-	-	-	-	-	50.0	3	0.28	0.23	
Lake Valley	SW-SM	0.5	0.69	0.34	75	1.40	29.3	2.84	0.55	25.0	40.0	2	0.23	0.20	Siddiqi 1987
	SW-SM	0.5	0.69	0.34	75	1.40	29.3	2.84	0.48	25.0	60.0	2	0.33	0.30	
Commercially-supplied Sand & Gravel	Mix of GP and SP	3/8	0.51	0.33	40	-	-	-	0.44	60.0	40.0	1	0.34	0.27	Evans & Zhou 1995
		3/8	0.60	0.40	60	-	-	-	0.52	40.0	40.0	1	0.26	0.22	
		3/8	0.80	0.52	80	-	-	-	0.69	20.0	40.0	1	0.18	0.16	
		3/8	0.99	0.57	100	-	-	-	0.82	0.0	40.0	1	0.16	0.13	
Monterey Sand	SP	0.02	0.85	0.56	100	0.36	1.4	2.65	0.60	0.0	87.0	0.54	0.43	0.38	De Alba et al 1975
	SP	0.02	-	-	-	0.36	1.4	2.65	0.65	0.0	70.0	0.54	0.30	0.27	
	-	-	-	-	-	-	-	-	0.70	0.0	50.0	0.54	0.22	0.20	

Each test specimen was compacted in four layers using the wet tamping method at a water content equal to 8.5%. Under-compaction of lower layers was also considered during specimen preparation.

Cyclic Undrained Shear Properties of Tokyo Gravel (Hatanaka et al., 1988)

The test site is located about 3.5 km to the northwest of Tokyo Station at a depth of about 20 m below the ground surface. The gravel is a diluvial (glacial and fluvio-glacial deposit) soil deposit locally known as "Tokyo Gravel." Extensive field investigations consisted of sampling with a large diameter spoon, undisturbed sampling utilizing ground freezing techniques, and downhole seismic surveys. Maximum particle size of the gravel is about 90 mm; sand contents range between 30% and 50% and C_u ranges between 64 and 122.

Cyclic triaxial tests were carried out on both undisturbed and reconstituted specimens. The effective confining pressure used in testing was 3 kilograms/cm^2 (ksc), which is equivalent to the effective overburden pressure at the sample depth. All cyclic tests were done at a loading frequency equal to 0.01 Hz. For the purpose of comparison, the undisturbed specimen was reconstituted following completion of the initial test keeping the same granular composition and relative density. To investigate the effects of density on the liquefaction strength for reconstituted samples, specimens were also prepared having a relative density equal to 80%. The higher density specimens were prepared by shaking the sample mold using a small vibrator after each layer of gravelly material had been placed. Fine soil grains were packed in the area near the cylindrical surface of the reconstituted specimens to minimize the effect of membrane penetration.

Determination of the maximum and minimum dry densities was done using the Japanese Society of Soil Mechanics and Foundation Engineering (JSSMFE) Standard method T 26-81T: minimum density was achieved by placing gravel very gently and slowly within the mold. The maximum density was obtained by placing the gravel inside the mold in seven layers and vibrating each layer with a small vibrator applied to the walls of the mold.

Liquefaction Potential of a Gravelly Soil in the Wufeng Area of Taiwan Observed after the Magnitude M_w 7.6 Chi Chi Earthquake (Lin and Chang, 2002and Lin et al., 2004)

The liquefied area is located near the Da Li River, about 2 km from the ruptured geologic fault. The material consists of gravel (53%) containing sand at a loose relative density (44%) and 3% silt sizes. A laboratory program of triaxial tests (nine sets of specimens) was carried out with reconstituted samples prepared with material extracted from exploratory pits including: specimens prepared at 40% relative density and 20%, 40% and 60% gravel content; specimens prepared at 40% gravel content and relative densities equal to 20%, 40% and 60%; and specimens prepared to the in-situ relative density ($D_{r} \approx$ 31%) and gravel content (53%) conditions. Note that the authors gave no indication as to how the field insitu dry density or D_r (31%) was determined. However, it does indicate the standards used to determine the max-min densities (see below).

The results were compared with the corresponding results obtained from sand (C_u = 13.5, max size = 10 mm, and D_{50} = 0.49 mm) at relative densities of 40%, 50%, 60% and 70%. To minimize the effect of membrane compliance, the effective triaxial cell confining pressure was kept at 1 ksc. Further decrease of membrane compliance was implemented by wrapping the specimens with filter paper as suggested by Evans et al., (1992).

Maximum and minimum densities of the gravelly composites were obtained following ASTM D4254-83 and ASTM D4253-83 standards. Specimens were prepared by splitting in five equal portions the required amount of material for a given relative density, placing each within the mold and then compacted at the designated relative density (no further details provided).

Nile River Bed Sands, Aswan Dam, Egypt: Cyclic Strength (Woodward-Clyde Consultants [WCC], 1985)

The sands were extracted from the main axis of the dam at depths between 2 and 7 m below the crest. The sands are uniformly graded and classified as SP, sub-angular to sub-round with less than 4% passing No. 200 sieve. Minimum density was obtained by wet-pluviation method, a density that agreed with that obtained with the "dry-tipping" procedure. The maximum density was obtained by using a Modified Japanese method in which the material is placed into a cylindrical mold in layers and each layer is densified by striking the mold with horizontal hammer blows from varying orientations.

Test specimens were prepared following a moist tamping procedure at two relative densities; 48.5% (4 tests) and 75% (5 tests). Tests were done at an effective cell confining pressure equal to 1 ksc.

Dune Sand, Aswan Dam, Egypt: Cyclic Strength (WCC, 1985)

The dune sand used in the construction of the Aswan dam was obtained from two nearby borrow areas. The sands are of Aeolian origin, primarily quartz, with a C_u of about 2. The maximum dry density was obtained by two methods. In Method 1, the dry sand was compacted to ASTM D1557 procedures; in a 1/30 ft^3 (0.00094 cubic meter) mold in five layers, each layer subjected to 25 blows of a 10-pound (4.5 kg) hammer falling freely from a height of 18 inches (45.7 cm).

In the second method, the dry sand was loosely placed in 1-inch (2.54 cm) thick layers in the same mold used in Method 1. Each layer was vibrated by 100 blows of a 1.3 pound (0.6 kg) rubber mallet on the outside of the mold wall.

Test specimens were prepared following a moist tamping procedure that provides little segregation in uniform soils and can be easily reproduced. However, previous research showed that the method provides cyclic strength significantly higher than results obtained from specimens prepared following pluviation techniques.

Eleven tests were performed on specimens prepared at a relative density around 50%. All specimens were isotropically consolidated; 6 specimens under an effective confining pressure equal to 1 ksc and 5 specimens to 3 ksc.

Cyclic Strength of Gravelly Soils, Lake Valley Dam, Sierra Nevada, California (Siddiqi et al., 1987)

The gravel material was obtained from the borrow area used during the buttressing operations of the dam located approximately 80 km northwest of Lake Tahoe. The material is comprised of gravel, sand and a small amount of silt and some cobbles. Gravel particles are rounded to sub-round.

Maximum and minimum densities were obtained following ASTM D2049-69, a standard that was discontinued in 1983 and replaced by ASTM test methods D4253, for maximum density using a vibratory table, and D4254 for minimum density.

The laboratory test program included two sets of tests: In the first set, 5 tests on 50.8 mm maximum particle size specimens and 5 tests on 12.7 mm maximum particle size were conducted. Specimens were isotropically consolidated to 2 ksc and compacted to 40% relative density.

The second set was similar to the first set but specimens were compacted to 60% relative density. Results that were corrected for membrane compliance and penetration effects showed no cyclic strength difference between the 50.8 mm and the 12.7 mm maximum particle sized specimens. Specimen preparation methodology was not provided in the reference.

Liquefaction Behavior of Sand-Gravel Composites (Evans and Zhou, 1995)

The gravel and sands used by the researchers were obtained from commercial sources. Gravel particles were sub-angular and granitic, with maximum particle size equal to 9.5 mm, D_{50} equal to 6.5 mm and C_u equal to 1.4. The sand had a maximum particle size equal to 1 mm, D_{50} equal to 0.40 mm and C_u equal to 2.0.

To determine the relative density of the sand/gravel composites, maximum/minimum dry densities were obtained for 0% to 100% gravel contents. The maximum density was obtained following ASTM-D2049 a standard that was discontinued in 1983 and replaced by ASTM test methods D4253 for maximum density using a vibratory table, and D4254 for minimum density. Minimum densities were obtained by placing the desired amount of material in a capped cylinder and then *"upset and up-righted"* carefully several times to achieve a very loose condition.

Laboratory test specimens were prepared by pluviation through air and then were isotropically consolidated in the cell chamber under an effective confining pressure equal to 100 kPa.

The undrained cyclic test program included sand/gravel composite specimens containing 0%, 20%, 40% and 60% gravel, all compacted at 40% relative density.

Cyclic Triaxial Tests on Monterey No. 0 Sand (Mulilis et al., 1975; De Alba et al., 1975)

The sand is a uniform, medium-sized sand composed primarily of quartz and feldspar, D_{50} = 0.36 mm, C_u equal to 1.44, and specific gravity equal (G_s) to 2.65. Maximum and minimum dry densities were obtained by ASTM D2049-69 and Kolbuszeswski's (1948) methods, respectively, from which the values e_{min} = 0.564, e_{max} = 0.852 were calculated. Triaxial test specimens were prepared by pluviation through air: 14 of them at D_r = 50%, 8 at D_r = 70% and 13 at D_r = 87%. Tests were carried out under an effective confining pressure equal to 0.56 ksc.

In summary, the relationship between void ratio and cyclic resistance ratio should be done by combining the results from all sources listed in Table 3 and discussed above. However, this is not possible for several reasons: (1) the method of sample preparation varied from research effort to another; and, (2) the methods to find the maximum and minimum densities of the materials (which were the basis for calculating the samples void ratio) also varied from one researcher to another. Therefore, results cannot be compared based on a given test void ratio or based on a given relative density. A review of the test conditions described in the references and summarized in Table 4 shows that only the testing performed on the Taiwan gravel was performed on samples prepared in conditions similar to those followed for the study site. Thus, going forward, these results are used for further evaluation.

COMPARISON BETWEEN TAIWAN AND STUDY SITE GRAVELS

Figure 5 is a repeat of the grain size distribution data of the study site gravels shown on Figure 2 combined with the grain size distribution data of the Taiwan Gravel containing 40% to

60% gravel from Lin and Chang (2002). Upon comparison, it is seen that the grain size results show a similar grain size distribution of for both materials up to the coarse sand/fine gravel size (about 10 mm). The C_u of the Study Site gravels ranges between 17 and 45 (mean about 31), and between 17 and 40 for the Taiwan gravels.

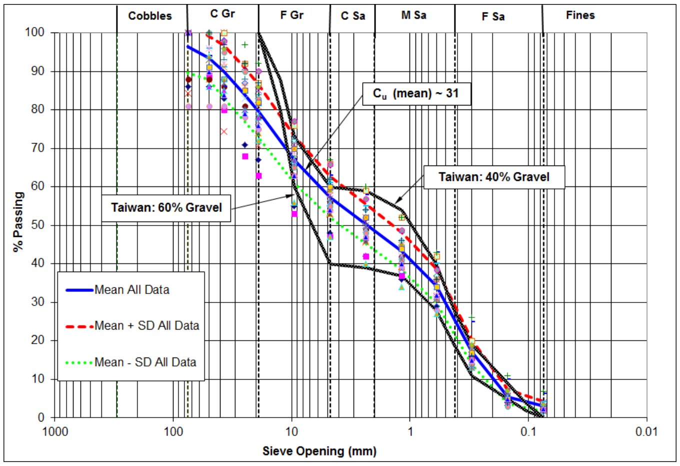

Fig. 5. Study Site and Taiwan Gravel Grain Size Distributions

The study site gravels contain larger size particles than the Taiwanese gravels. However, test data show that particle size is not a factor affecting the cyclic strength of the material. For example, based on data from Oroville Dam, Banerjee et al., (1979) concluded that the influence of specimen size (300 mm and 50 mm) and the maximum particle size virtually disappear when the cyclic strength data are corrected for system compliance effects. Evans and Seed (1987) performed cyclic tests on Watsonville gravel, ranging in maximum particle size from 0.8 mm to 50 mm, concluding that there is no apparent difference between the cyclic loading resistances of two gravels composed of very different grain size distributions provided that the relative density and structure of the specimens are similar and that the membrane compliance effects are accounted for. Siddiqi et al., (1987) conducted cyclic tests on 12 mm and 50 mm maximum gravel size from Lake Valley Dam, and they show that the cyclic strength from the two gravels follow the same relationship.

Maximum and minimum dry densities of the two different gravels were obtained following the same procedures: ASTM test methods D4253 for maximum density using a vibratory table and D4254 for minimum density. Consequently, the relative densities listed for the two materials are comparable with each other.

Based on laboratory relative density determinations for materials from study site test pits 2 and 8, maximum and minimum void ratios available for the study site materials from TP-2 (depth 1-2 m) with 47% gravel are 0.56 and 0.39, respectively. Maximum and minimum void ratios available for materials from TP-8 (depth 2-3 m) with 40% gravel are 0.53 and 0.33, respectively. Equivalent values for Taiwan gravels containing 40% gravel are 0.53 and 0.34, respectively. Equivalent values for Taiwan gravels containing 60% gravel are 0.51 and 0.33, respectively. All of these limiting void ratio values are within narrow ranges. This close comparison suggests that both materials at the two sites are nearly the same.

The similarity of properties (grain size and void ratio) between the two materials suggests that the laboratory CRR of the study site gravels could be estimated from the strength data of the Taiwan gravels listed in Table 3. The variation in CRR with void ratio for 10 and 20 loading cycles is presented in Figure 5. Cyclic stress ratios obtained in laboratory triaxial tests require some adjustments before they can be extrapolated to the field conditions, as discussed below.

CORRECTIONS TO THE LABORATORY-OBTAINED CYCLIC RESISTANCE RATIO (CRR)

It is well known that the cyclic triaxial test stress boundary conditions are quite different from those acting in the field during cyclic loading. For this reason, a conversion factor must be applied to the triaxial test results to reflect the simple shear stress and the three-directional shaking in the ground. De Alba et al., (1975) found that the conversion factor is a function of both the relative density of the material and the number of stress applications (cycles). Based on their findings, a conversion factor equal to 0.60 is determined to be applicable for the conditions at the study site.

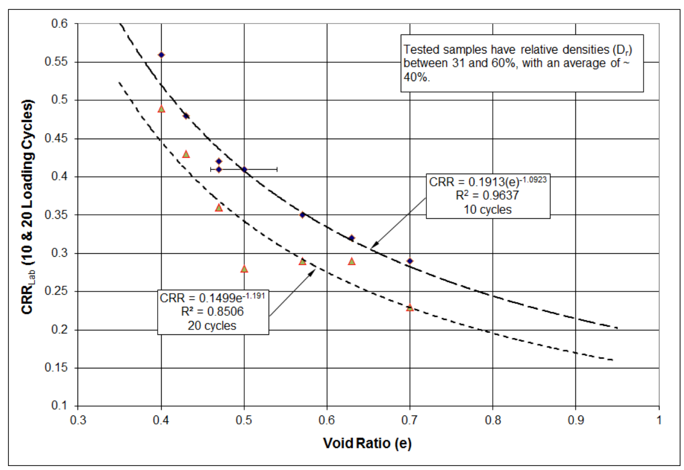

Fig. 6. CRR_{Lab} vs Void Ratio of Taiwan Gravels (based on the data from Lin & Chang (2002) and Lin et al., (2004)

The CRRs derived from Figure 6 correspond to strengths of freshly prepared specimens. Extensive research was carried out by Banerjee et al., (1979) for Oroville Dam. The effect of sustained pressure on the cyclic strength of dense gravel was investigated with the results of a series of cyclic tests performed after varying days of "curing time." Based on the results obtained, and the addition of several other field records, it was shown, for example, that 1000 years after deposition the strength gain due to "ageing" is at least 1.5 of the strength of freshly deposited gravel.

Lin and Chang (2002) do not provide details on the type of compaction used in the preparation of the triaxial specimens other than explaining *"specimens were prepared by splitting in five equal portions the required amount of material for a given relative density and placing each within the mold and then compacted at the designated relative density."* Since there is no indication that vibratory compaction was used, it is concluded that the five equal portions of material were compacted by either rodding or tamping. If this is the case, one can conclude that the measured stress ratios in the laboratory tests may be as much as 33% higher than the stress ratios that would have been measured from samples formed by pluvial deposition. This would indicate that the actual field stress ratio should be about 75% of the laboratory-measured value.

In summary, the laboratory CRR (dynamic strength) obtained from Figure 6 must be corrected to reflect the combined effects of stress boundary and multidirectional shaking (C_r = 0.6); age after deposition (C_A = 1.50) and method of specimen preparation (C_{SP} = 0.75) for a combined total correction factor equal to; 0.60 x 1.50 x 0.75 = 0.68.

STUDY SITE LIQUEFACTION ASSESSMENT

The results presented in Figure 6, once corrected as outlined above, can then be used to assess the liquefaction potential of the site gravels once the insitu shear wave velocity is determined. The procedure is as follows;

1. Normalize the field V_s obtained at any depth using Equation 1.
2. With the normalized shear wave velocity enter Figure 4 to obtain the corresponding void ratio, *e*. Use engineering judgment in combining the two relationships shown in Figure 4. Note, the use of Figure 4 implies that the ratio between the field and the analytical shear wave velocities is constant and equal to ≈ 1 along the depth of the gravel deposit.
3. After obtaining the void ratio (e), go to Figure 6, as appropriate, to obtain the laboratory CRR.
4. Convert the laboratory CRR by multiplying the result from Step 3 by 0.68 to obtain the "field" CRR.

The resulting liquefaction analysis for the proposed structure, utilizing the CRR values determined above, resulted in high factors of safety, indicating the gravel deposit was not susceptible to liquefaction given the revised seismic hazard for the study site. The analysis was conducted for each of the four locations where field shear wave velocity data was acquired, including downhole location B-2, which had the lowest measured V_s.

Additional liquefaction susceptibility assessments were carried out for the gravel deposit using "gravel-corrected" standard penetration test (SPT) N-value results. The gravel corrections were made using the procedure by Vallee and Skryness (1979), Andrus (1994) and Mejia (2007, as described by Idriss and Boulanger, 2008), where the cumulative SPT hammer blows/inch are plotted and evaluated. The results showed that the CRR derived from the corrected SPT N-values (normalized and corrected $[N_1]_{60}$) resulted in CRR values less than those developed based on the gravel evaluation discussed above. However, the results still showed the site was not susceptible to liquefaction, given the revised seismic hazard for the site.

OVERALL CONCLUSIONS

Case history literature can be valuable to the practicing engineer. In this case, several case histories were reviewed for application to a particular study site under evaluation. The case histories provided valuable information that, when coupled with site-specific data, resulted in a simple methodology to assess the CRR of the gravel deposits. With the case history information, a method was developed to correlate properties from one site to another for corroborative evidence to demonstrate site acceptability.

In this case, two methods (Hardin and Drnevich, and Menq) were used to estimate G_{max} and eventually V_s. Straight forward field and laboratory tests were used to correlate shear wave velocity with void ratio; and case histories were used to correlate void ratio with the cyclic resistance ratio of the gravel deposits. Liquefaction susceptibility assessments based on "gravel-corrected" SPT N-value results appear to have merit and are recommended for evaluation of sites where gravel and/or cobbles are present.

ACKNOWLEDGMENTS

The authors wish to express their appreciation to the Bechtel Corporation and the University of Texas at Austin for the opportunity to share this information and for encouraging the authors to publish case history information for the benefit of the industry.

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