Liquefaction subsurface investigation for Milford Dam

Timothy D. Stark, Justin R. Lewis, Gonzalo Castro, Francke C. Walberg, and David L. Mathews

Abstract: The US Army Corps of Engineers (USACE) completed a liquefaction potential analysis as part of the seismic evaluation of Milford Dam in 1986. This paper uses data from the 1986 study to compare fines content data from in situ frozen and standard penetration test (SPT) samples that suggest fines content can be overestimated by 1–10% by SPT samples in stratified sand deposits. This result may have implications for liquefaction assessments because split-spoon samples may overestimate the actual fines content, resulting in a liquefiable deposit being classified as nonliquefiable. In addition, the paper evaluates the effectiveness of ground freezing on maintaining in situ soil structure and aging of the foundation sands at Milford Dam.

Key words: soil mechanics, liquefaction, shear strength, cone penetration test, standard penetration test, seismic stability.

Résumé : Le Corps du génie de l'armée américaine (USACE) a réalisé une analyse du potentiel de liquéfaction dans le cadre d'une évaluation sismique du barrage de Milford en 1986. La présente étude utilise les données recueillies lors de l'étude de 1986 afin de comparer le contenu en particules fines d'échantillons in situ gelés ainsi que d'échantillons provenant d'essais de pénétration standard (EPS). Ces données suggèrent que le contenu en particules fines peut être surestimé de 1 à 10 % pour les échantillons d'EPS prélevés dans des dépôts de sable stratifié. Ce résultat peut avoir des implications dans l'évaluation de la liquéfaction puisque les échantillons de cuillère fendue peuvent surestimer le contenu en particules fines et ainsi le dépôt liquéfiable est classé comme non-liquéfiable. De plus, cet article évalue l'efficacité de conserver le sol gelé pour maintenir la structure in situ du sol et pour le vieillissement des sables de fondation au barrage de Milford.

Mots-clés : mécanique des sols, liquéfaction, résistance au cisaillement, essai de pénétration du cône, essai de pénétration standard, stabilité sismique.

[Traduit par la Rédaction]

Introduction

Milford Dam is a US Army Corps of Engineers (USACE), Kansas City District (KCD) project, located on the Republican River in east central Kansas. The site is 6.5 km (4 miles) northwest of Junction City, which is about 220 km (135 miles) directly west of Kansas City, Missouri. The dam is part of a general comprehensive plan for flood control and other purposes in the Missouri River basin. Construction of the dam began in 1962 and was completed in 1964. The reservoir was filled to multipurpose pool level (normal pool) of elevation 349 m (1144.4 ft) in 1967. At its highest point, Milford Dam rises 45 m (147 ft) to elevation 370 m (1213 ft). It spans 1920 m (6300 ft) across the river valley. At the multipurpose pool level, the reservoir retains approximately 512 million m^3 (415 000 acre-ft) of water. Figures 1 and 2 show a plan view and a typical cross section, respectively, of Milford Dam.

The dam is underlain by an alluvial foundation (Pleistocene and recent age) that consists of a natural fine-grained soil blanket that is underlain by sand deposits that become coarser with depth. At the midpoint of the dam (centerline of the river channel), the bedrock is about elevation 315.5 m (1035 ft), while the top of the alluvial foundation is approximately elevation 331 m (1085 ft). Along the length of the dam, the alluvial deposit has a thickness that ranges between 14 and 17 m (45–55 ft). The natural fine-grained blanket materials are typically lean clays, silts, and sandy silts. The foundation sands are generally classified as silty sands to poorly graded clean sands. Well-graded sands and low-plasticity silt can also be found within the foundation. A 3 m (10 ft) thick basal layer of gravels and cobbles is typically present just above bedrock. The dam has no positive cutoff to bedrock through the alluvium, so underseepage is controlled by pressure relief wells located along the downstream toe. As a result, the foundation alluvium is saturated beneath the dam, and the groundwater surface near the downstream toe is about 1.2 m (4 ft) below the ground surface.

The area where Milford Dam is located has experienced historic seismic events, with modified Mercalli (MM) inten-

Received 19 April 2011. Accepted 31 May 2011. Published at www.nrcresearchpress.com/cgj on 29 September 2011.

T.D. Stark. University of Illinois at Urbana-Champaign, 205 N. Mathews Avenue, Urbana, IL 61801, USA.
J.R. Lewis. Hayward-Baker, 1350 West Lake Street, Roselle, IL 60172, USA.
G. Castro. GEI Consultants, Inc., 400 Unicorn Park Drive, Woburn, MA 01801, USA.
F.C. Walberg. URS Corporation, 8300 College Boulevard, Suite 200, Overland Park, KS 66210, USA.
D.L. Mathews. US Army Corps of Engineers, Geotechnical Branch, Kansas City District, 601 E. 12th Street, Kansas City, MO 64106, USA.

Corresponding author: T.D. Stark (e-mail: tstark@illinois.edu).

Fig. 1. Plan view of Milford Dam showing boring and in situ test locations.

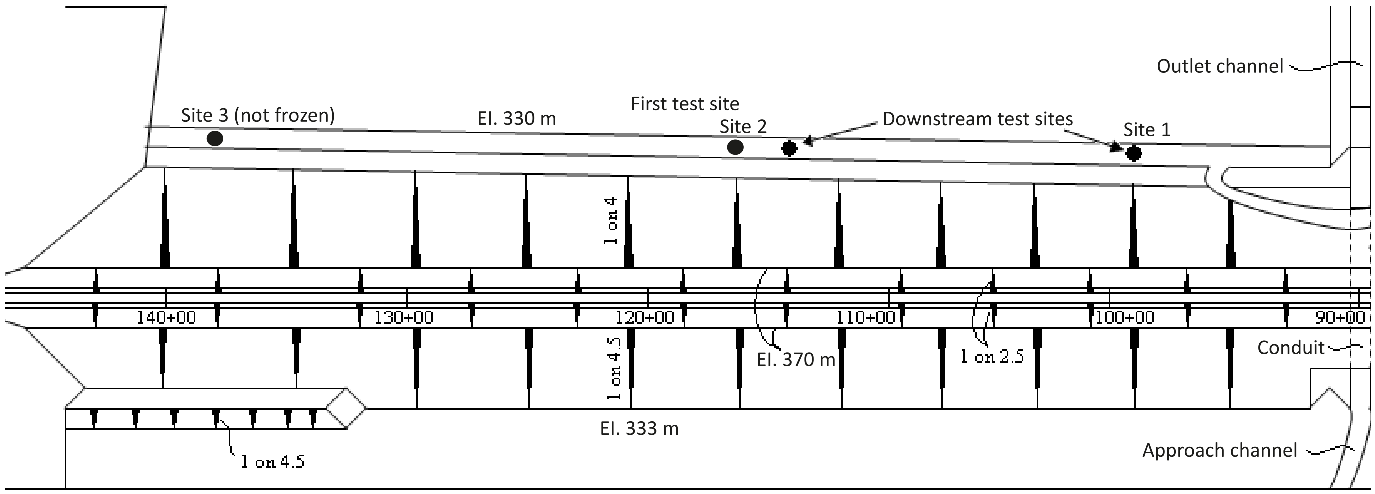
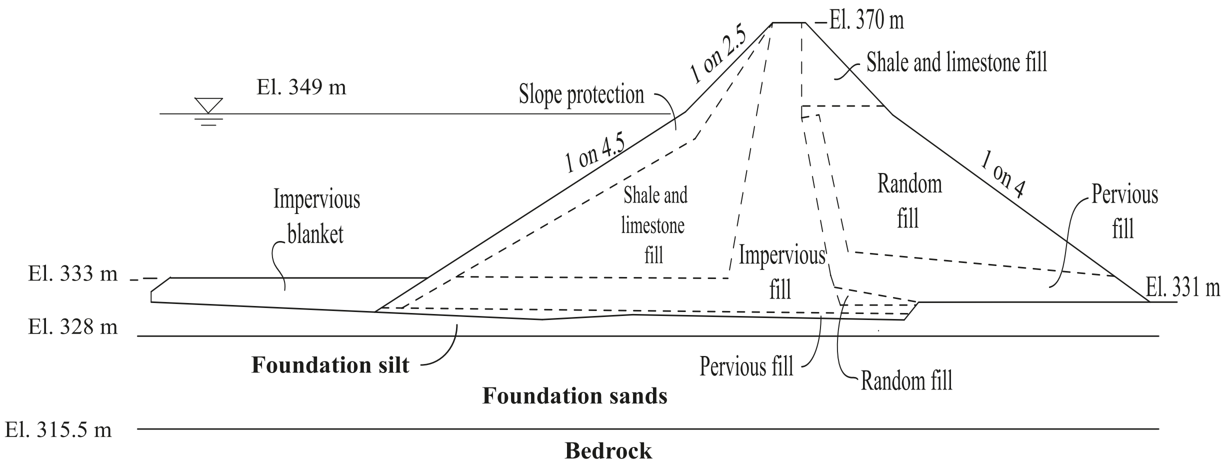


Fig. 2. Cross section of Milford Dam at station 115+00.



sities as high as VII and VIII. Another dam, Tuttle Creek, located just to the east of Milford Dam near Manhattan, Kansas, recently underwent seismic retrofit to reduce liquefaction-induced permanent deformations of the downstream slope that could lead to loss of the reservoir. The Tuttle Creek Dam seismic retrofit involves shear walls transverse to the dam centerline to stabilize the downstream slope.

As part of a program to review the seismic hazard of all Kansas City District dams, the District completed a liquefaction analysis of the foundation alluvium at Milford Dam in 1986 (USACE KCD 1986*c*, 1986*d*). For the liquefaction potential evaluation at Milford Dam, the District performed static and cyclic triaxial compression tests (USACE 1986*a*, 1986*b*; USACE KCD 1986*c*, 1986*d*), standard penetration tests (SPTs), and cone penetration tests (CPTs) on the alluvial foundation materials. The Milford Dam investigation was initiated after an initial screening with SPT results (USACE KCD 1986*c*) indicated that the foundation soils may be susceptible to liquefaction.

The USACE Missouri River Division office, which provided oversight to the seismic investigation efforts within the Division, initiated a program to develop the best available undisturbed sampling techniques for subsequent use at other sites. In the late 1970's and 1980's, with a rapidly changing state-of-the-art for earthquake liquefaction potential investigations, a program of drilling and sampling as well as sample handling and laboratory testing techniques and procedures for obtaining undisturbed samples of saturated sand below the groundwater surface was developed. This 1980's investigation resulted in a unique set of data that is used herein to investigate the shear behavior of the foundation sands and validate liquefaction triggering relationships, e.g., Stark and Mesri (1992) and Idriss and Boulanger (2004). The data is also used to evaluate the effectiveness of ground freezing on maintaining in situ soil structure and aging of the foundation sands at Milford Dam by comparing the results of monotonic and cyclic triaxial tests on samples cored from frozen ground with those obtained using fixed-piston samplers in unfrozen ground. This comparison also revealed that fines content data from in situ frozen samples can be overestimated by 1%–10% by SPT samples in a stratified sand deposit, which may result in a liquefiable deposit being classified as nonliquefiable.

Sampling of foundation sands

Figure 1 also shows the location of the two downstream test sites used to obtain the undisturbed samples and perform the in situ tests at Milford Dam. Initially, the USACE used

SPT blow count data and grain-size analyses of split-spoon samples to evaluate liquefaction susceptible deposits. Preliminary assessment using simplified analysis procedures showed marginal safety against liquefaction of the foundation sands at Milford Dam. To further investigate the stability of the dam, additional data for use in a more refined analysis was required. Accordingly, high-quality samples of the foundation sands were required for laboratory triaxial compression testing to evaluate liquefaction potential using the procedure developed by Seed (1981, 1983) after the near failure of the Lower San Fernando Dam, Los Angeles, California, and subsequently Poulos et al. (1985). Earlier, the USACE had conducted investigations (Walberg 1978) to evaluate techniques to reduce handling disturbance of thin-wall tube samples obtained by a fixed-piston sampler. High-quality thin-wall 75 mm (3 in.) diameter tube samples of saturated sand obtained from below groundwater were drained and then frozen after the sample reached the ground surface. This process is referred to herein as “aboveground freezing”, and the resulting samples are called “aboveground frozen samples”. To obtain even better quality samples, and to better reflect the in situ strength of the soil deposit, the USACE also used in situ ground freezing prior to sampling (coring) of the foundation sands. This process is referred to herein as “ground freezing”, and the resulting samples are called “frozen ground samples”.

Aboveground freezing

The first attempt to obtain better quality samples for laboratory testing was to freeze traditional thin-wall tube samples after the sample reached the ground surface. Freezing of thin-wall tube samples above the ground reduces the disturbance that can occur during transportation, storage, handling, sample extrusion, specimen trimming, and test setup in the laboratory. In addition, the disturbance during this sampling was minimized with the use of a Hvorslev fixed-piston sampler, appropriate sample tube area, and clearance ratios, and use of drilling fluid and drilling techniques that removed cuttings from the boring to completely clean out the borehole, minimize stress changes, and enhance sample recovery. For comparison and control purposes, some of these samples were not frozen. These samples are referred to herein as “unfrozen thin-wall tube” samples and are compared to aboveground frozen thin-wall tube samples.

After the tube samples were extracted from the ground, they were drained for 12 h or until 250 cm^3 of water was removed. It was thought that removal of 250 cm^3 of water before freezing would provide enough air voids to allow for expansion during phase change from water to ice. If 250 cm^3 of liquid did not drain during the 12 h, a vacuum was applied to remove enough water to reach 250 cm^3. Measurements were made to the top of the soil in each sample tube to detect changes in volume during drainage. Afterwards, these samples were placed vertically in a freezer box with dry ice to freeze the sample. The samples were allowed to freeze for a minimum of 12 h before transportation. The frozen samples then were placed in cushioned transport racks. Measurements were also made to the top of the frozen samples during freezing. Both the nonfrozen and frozen samples were measured before and after transportation to determine if volume change occurred during transportation, which would indicate sample disturbance.

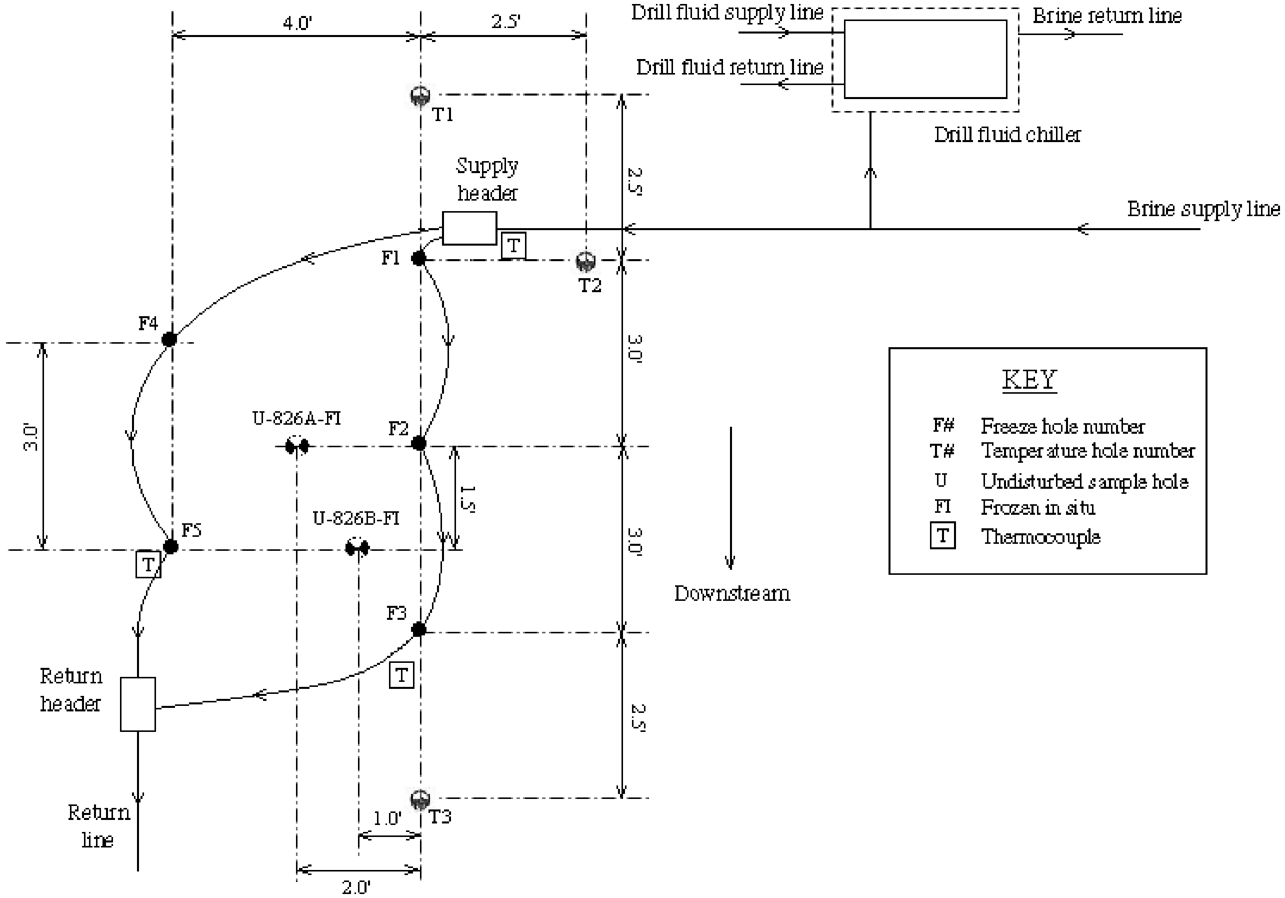
Ground freezing

To obtain the highest quality samples of the saturated foundation sands at Milford Dam, a procedure of one-dimensional freezing (Singh et al. 1979) was used to avoid volume change and disturbance from ice expansion and to ensure that water would drain out ahead of the freeze front. Previous in situ ground freezing at other sites had used an all-around freezing technique. At Fort Peck Dam, Fort Peck, Montana (USACE 1939; Hvorslev 1948), vertical freeze pipes in a circular array were used to create a frozen mass that was subsequently cored with a 0.9 m (36 in.) diameter calyx barrel. Cooled brine was circulated through the pipes to freeze the soil. This layout may have resulted in disturbance of the soil due to volume expansion in the phase change between water and ice. A similar all-around freezing technique was later attempted by Osterberg and Varaksin (1973) who concluded the samples had been disturbed by the expansion of ice in the pore volume. Yoshimi et al. (1978) use radial one-dimensional freezing employing a single freeze pipe and subsequently pulled a frozen column of soil from the ground. This procedure eliminates potential of disturbance from volume expansion of the water into ice but is limited in application to shallow depths and is not appropriate at the toe of an operational dam where piezometric levels may be high or above the ground surface.

One-dimensional freezing allows freezing to occur while drainage of excess water occurs so the porosity of the soil is not altered by water expansion. High-quality samples are then cored from the frozen sand. This one-dimensional ground freezing technique was developed at Milford Dam and allowed high-quality sampling to a depth of 15 m (50 ft) without compromising the structural integrity of the dam (USACE KCD 1986*c*). Core samples of 150 mm (6 in.) diameter were preserved in the frozen state throughout coring and transportation to the laboratory. In the laboratory, the oversized cores were recored to a 70 mm (2.8 in.) diameter for triaxial compression testing. Soils suitable for such a freezing program are relatively clean sands and gravels. The presence of significant fines (material finer than US No. 200 sieve) can prevent drainage of pore water away from the freeze front and (or) migration of pore water toward the freeze front and formation of ice lenses. In either case, volumetric increase is likely to occur, causing disturbance of the soil structure.

In situ freezing at Milford Dam was performed at three downstream locations named first test site, site 1, and site 2 (see upper portion of Fig. 1). A planned third site was not frozen and tested. The one-dimensional ground freezing technique was used at all sites. The first test site includes two borings drilled into the in situ frozen soils, with two in situ frozen ground sample core holes, U-826A and U-826B (see Fig. 3). In situ freezing location site 2 was more extensively investigated than site 1, including an initial shallow test site. Three in situ frozen ground sample core holes, U-826C through U-826E, were drilled at site 2. Two SPT borings, D-785 and D-785A, a CPT sounding (CPT-14) and two Shelby tube sample borings, U-785N (nonfrozen, Shelby tube sample) and U-785F, were also performed at site 2. Borings U-826A and U-826B were used for initially setting up the in situ freezing program at site 2 in addition to obtaining samples and stratigraphic data.

Fig. 3. Boring location diagram at first test site (see Fig. 1) for in situ ground freezing program (USACE KCD 1986*c*). All dimensions in feet (1 ft = 0.3048 m).



Site 1 includes three borings drilled into the in situ frozen soils. Three in situ frozen ground sample core holes, U-825, U-825A, and U-825B, were drilled at site 1. To provide additional data and for comparison with site 2, the following explorations were also performed at site 1 prior to ground freezing: D-781 (SPT boring), CPT-21 (CPT sounding), U-781N (nonfrozen, Shelby tube sample), and U-781F (frozen aboveground, Shelby tube sample).

Coolant consisting of calcium chloride brine chilled by a conventional 9.5×10^4 kJ/h (7.5 t) refrigeration plant (typically used for ground freezing for construction purposes) supplemented with liquid carbon dioxide (LCO_2) was used. The brine was cooled by the primary coolant of the chiller, Freon 12, to about –20 °C. The brine was circulated through an injection chamber where LCO_2 was injected directly into the brine, bringing the temperature to about –30 °C. This added temperature drop was considered necessary because flowing groundwater beneath the dam posed an added energy removal requirement.

Like other ground freezing programs, the one at Milford Dam used vertical pipes for circulating brine coolant. The vertical pipes were placed in a layout to create an area from which coring could occur (see Fig. 3). Figure 3 also shows the location of the various borings at the first test site used to freeze the foundation soils and monitor the progress of the freezing.

The coolant was initially circulated through freeze pipes F1–F3. The freezing radiates symmetrically outward from these pipes and eventually forms a solid mass of frozen soil around each pipe. Temperature of the soil is monitored through brine-filled holes at T1–T3. Pipes F4 and F5 are circulated with coolant after freezing from the three previous freeze holes was nearly complete. If freezing from F4 and F5 had started too early, one-dimensional freezing would not have occurred. The frozen soil created by pipes F4 and F5 created a thick mass of frozen soil quickly and completely.

Coring frozen ground

To obtain high-quality sample cores of frozen ground, a double-tube core barrel with a diamond bit was used. Because the coring occurred in areas of full saturation, the soil voids were completely filled with ice, which was sufficient to resist the torsional forces from the coring equipment. To prevent soil melting during drilling, a chilled drilling fluid was used to maintain the frozen temperature. Diesel fuel is commonly used as the drilling fluid, but propylene glycol was used at Milford Dam because it is more environmentally friendly. The required drilling fluid was circulated through the same chiller system that cools the brine in the vertical pipes. This coring procedure yielded 150 mm (6 in.) diameter sample cores of the frozen foundation sands.

Transportation and storage of frozen ground samples

After field coring, samples were stored onsite in trucks with freezer units to be shipped upon completion of coring at each site. Each sample was sealed in two layers of polyolefin and surrounded by three strips of split PVC pipe to serve as a protective cradle. After shipment, samples were stored in a commercial cold storage facility near the USACE testing laboratory to maintain a temperature of –23 °C. Samples were placed in plastic bags and stored in wooden crates with crushed iced surrounding the samples to prevent sublimation. When the samples were ready for further processing and testing, they were transported in containers with dry ice pellets to the USACE Missouri River Division Laboratory. Freezers set at –18 °C were used for temporary storage at the laboratory.

Laboratory testing of frozen ground samples

Frozen samples were cut to lengths of 180–200 mm (7–8 in.) with a circular masonry saw. To prevent premature thawing, the sample was placed in a container with dry ice pellets when not in immediate use. From the 180–200 mm (7–8 in.) long sample, a 70 mm (2.8 in.) diameter test specimen was cored from the center of the 150 mm (6 in.) diameter sample using a diamond-bit core barrel. The bit was precooled by submerging the tip in an alcohol – dry ice mixture to minimize thawing around the core bit during coring. A clamp held the sample piece vertically during coring. Compressed carbon dioxide (CO_2) gas was used as a drilling fluid to keep all necessary components at temperatures to minimize thawing. The grain size of the sample is small enough to allow recoring (maximum sample diameter, D_{max}, less than one-sixth of the specimen diameter) to obtain a specimen. The recoring is designed to eliminate the majority of the soil disturbance caused by field coring and (or) thawing at the sample edge, and it allowed use of the available triaxial equipment sized for 70 mm (2.8 in.) diameter specimens.

Once a suitable specimen diameter for triaxial compression testing was obtained, the specimen was trimmed to the necessary length using a diamond-tooth hard rock saw. An important feature of the trimming process was the use of a fine moist sand packed around the bottom of the saw-cut location. The fine sand was frozen to the sample. This process allowed a sharper specimen edge to be cut by preventing chunks of sand from breaking off as the blade exited the specimen. Also, an aluminum disk was used to prevent the saw blade from drifting toward the end while making thin cuts. Compressed CO_2 gas and liquid nitrogen were used in this process to keep temperatures at acceptable levels and remove cuttings. End squareness was verified before the specimen was approved for triaxial compression testing.

Final preparation of the specimen for triaxial testing included placing it in a cold trimming block. High spots were trimmed flush with the block, and voids were filled with moist No. 80 sand. Specimen diameter at the top, middle, and bottom were measured to the nearest 0.025 mm (0.001 in.) using a circumferential “Pi” tape. Specimen height was measured with a calibrated dial gauge to the same tolerance. The specimen was finally weighed to the nearest 0.1 g.

As part of the triaxial equipment setup, the bottom end cap and drainage line were filled with deaired water and then frozen. Ice coming from the bottom stone was removed to create a flush surface. The cell base was put in a 300 mm (12 in.) deep tub containing dry ice pellets. The dry ice kept the base frozen and also provided a CO_2 rich environment for specimen setup.

Silicon grease was applied to the top and bottom cap peripheries. The silicon acted as a sealant with the membrane because the O-rings were too stiff to seal during the installation stage. After application of the silicon grease, the frozen sand specimen and membrane were added and a partial vacuum applied (250 mm Hg (1 mm Hg = 133.3 Pa)) to the top cap. Height and diameter measurements of the specimen were made again for comparison purposes. Thermocouples were then applied to the top and bottom caps for temperature monitoring.

The triaxial specimen was now ready to be thawed prior to testing. The lucite chamber for the triaxial cell was put in place, and the desired cell pressure for thawing that particular specimen was applied. This cell pressure is not the consolidation pressure because the consolidation pressure was applied after thawing was completed but before shearing. The specimen was allowed to warm slowly with monitoring of the specimen height and top cap temperature. Water was added to the top drainage line so the desired back pressure could be applied as the top of the specimen began to thaw. Cell and back pressure was applied such that the mean effective stress on the specimen during thawing was

$$[1] \qquad \sigma'_{\mathrm{mean}} = \frac{\sigma'_{\mathrm{v}} + 2\sigma'_{\mathrm{h}}}{3}$$

where the vertical and horizontal stresses, σ'_{v} and σ'_{h}, respectively, were estimated from the in situ stresses. Weights were placed on the loading piston or rod to counteract piston uplift when the cell pressure was applied. Some specimens were thawed with a back pressure equal to the in situ pore pressures, while others were thawed using a back pressure of 345 kPa (50 psi) to expedite saturation and minimize height changes due to thawing.

Specimens were thawed from top to bottom over a period of 8 h. Dry ice at the bottom cap controlled the thaw rate. Several parameters were recorded during the thawing process, including elapsed time, water intake, height reading, and cap temperatures. The volume change during thawing was estimated from the change in specimen height, assuming isotropic volume change. The data obtained suggests that the water intake differed from the 9% theoretical value of ice melting to water, which was attributed to gas in the ice. Upon completion of thawing, the triaxial cell was filled with deaired water, and the *B* pore-pressure coefficient was measured. Stress-controlled cyclic triaxial compression tests and critical-state consolidated–undrained (CU) triaxial compression tests were conducted to obtain values of in situ dynamic and static strengths, respectively.

A key question at this point is whether the thawing process under the consolidation pressure produced any change in sample void ratio, i.e., did the void ratio change as a result of the thawing. If the water intake had been equal to the pore-water volume change expected from thawing and specimen height did not change, then it could be assumed that no change in void ratio occurred. As noted above, the water intake measurement could not be used for this purpose because of apparent presence of air in the ice in the pores.

Table 1 presents the change in void ratio from the initial void ratio, i.e., void ratio after sampling, to void ratio after thawing based on observed changes in specimen height and assuming isotropic volume change. Triaxial specimens reported are U1 through U11 except U2 because the U2 specimen was lost during consolidation. Comparison of these void ratios shows that thawing did not produce a significant change in void ratio. After thawing, the triaxial cell contains an unfrozen, high-quality sand specimen ready for consolidation and shear testing. Of course, additional void ratio change occurred during consolidation, but this change in void ratio was obtained from the volume of water that exited the specimen during consolidation.

Table 1. Change in void ratio due to specimen thawing in laboratory.

Specimen	Void ratio after sampling	Void ratio after thawing
U1	0.479	0.433
U3	0.413	0.400
U4	0.416	0.393
U5	0.441	0.423
U6	0.706	0.673
U7	0.681	0.647
U8	0.691	0.667
U9	0.753	0.718
U10	0.650	0.593
U11	0.538	0.480

Remolded specimens

In addition to the cyclic and monotonic triaxial compression tests performed on the frozen ground specimens, monotonic triaxial compression tests were performed on remolded specimens to establish the critical-state line for the foundation sands. The foundation sands were determined to have similar grain roundness, and thus the slope of the critical-state line could be assumed to be constant for all foundation sands (Castro 1985). Of the four zones of foundation sand that were delineated, zone 3B from a depth of 5–7 m (19–23 ft), or elevation 324–325 m (1062–1066 ft), was found to have the lowest densities and thus controlled liquefaction susceptibility (USACE KCD 1986*c*). It was then decided that all remolded specimens would be mixed to achieve the gradation of the sands in zone 3B. The target gradation for the remolded sand specimens to simulate the sands in zone 3B is shown in Fig. 4. Figure 4 shows the target gradation has a fines content of 2%–3% so the remolded samples correspond to a clean sand.

Remolding was performed using the trimmings of the frozen in situ cyclic triaxial test specimens from zone 3B generated during the specimen trimming process. The soil was thoroughly mixed and separated on standard sieves Nos. 4, 10, 16, 20, 30, 40, 70, 100, and 200. The appropriate amounts from each sieve were then added to achieve the gradation shown in Fig. 4. The sand at the target gradation has a minimum dry unit weight of 15.6 kN/m^3 (99.3 lb/ft^3), a maximum dry unit weight of 18.4 kN/m^3 (117.1 lb/ft^3), and a specific gravity of 2.66. Uniform specimens were prepared in layers using a 12.7 mm (0.5 in.) diameter static weight tamper and a tamper pattern of 38 applications per layer. The static weight could be varied to achieve the desired specimen unit weight. Using this procedure, specimen dry unit weights could be achieved to a maximum of 16.5 kN/m^3 (105 lb/ft^3), which is between the maximum and minimum dry unit weights. Higher unit weights could not be obtained because vibration was not used.

Penetration test data

At both downstream test sites, sites 1 and 2 (see Fig. 1), SPTs and CPTs were performed. At site 1, one SPT boring was performed, D-781, as well as one CPT sounding, CPT-21. At site 2, two SPT borings were performed, D-785 and D-785A, and one CPT sounding, CPT-14. In all cases, the mobile safety hammer was used with one or two wraps of the rope on the cathead. Specific details of the testing procedures were meticulously recorded as suggested in Seed et al. (1985) and Kovacs et al. (1977). The safety hammer, when used with these procedures, typically delivers about 60% of the energy to the ground (Seed et al. 1985). This was confirmed later during subsequent calibration of drill rigs, hammers, and operators at other site investigations performed for the District dam projects using calibration equipment. The SPT data is corrected for an effective overburden pressure of 96 kPa (1 t/ft^2). The resulting corrected SPT blow count values, $(N_1)_{60}$, presented in Fig. 5 are not corrected for fines content or gravel content. The results of the SPTs and CPTs are presented in Figs. 5 and 6, with a profile of the foundation soils at Milford Dam. The ground surface is located around elevation 331 m (1085 ft) at sites 1 and 2, and the top of bedrock is around elevation 313.6 m (1029 ft) at site 1 (D-785 boring) and around elevation 317.0 m (1048 ft) at site 2 (D-781 boring), which includes a boulder layer over the bedrock.

The results of the SPTs in Fig. 5 show significant scatter for the foundation sands at both sites 1 and 2, which start at around elevation 327 m (1072.5 ft) or a depth of about 3 m (10 ft). This scatter is most likely caused by variability in relative density within the alluvial foundation, but a nonsite specific value of the blow count effective stress correction factor, C_N, may also have contributed to some of the scatter with depth by overestimating C_N. The $(N_1)_{60}$ values as low as 7 occur from elevation 328.5 m (1078 ft) to elevation 326 m (1070 ft). As the effective vertical stress increases with depth, $(N_1)_{60}$ values tend to increase to around 25 at elevation 317.2 m (1040 ft), a depth of 13.7 m (45 ft). The high $(N_1)_{60}$ values from borehole hole D-781 at site 1 (solid squares) are most likely caused by the presence of dense foundation sands or the sampler being influenced by large gravel particles (limestone fragments).

The CPT results plotted in Fig. 6 provide a better illustration of the increasing density, or strength, with depth (or elevation) in the foundation sands. The spikes in tip resistance near the ground surface should be ignored because they correspond to the fine-grained blanket (clay) and not the foundation sands, which are the subject of the liquefaction assessment. The spikes in tip resistance in the fine-grained blanket may be caused by desiccation or an inclusion. The CPT sounding CPT-14 at site 2 (dashed line and highest tip resistance in fine-grained blanket) shows values of uncor-

Fig. 4. Target gradation for remolded specimens to simulate the sands in zone 3B for monotonic triaxial compression tests on foundation sand at Milford Dam.

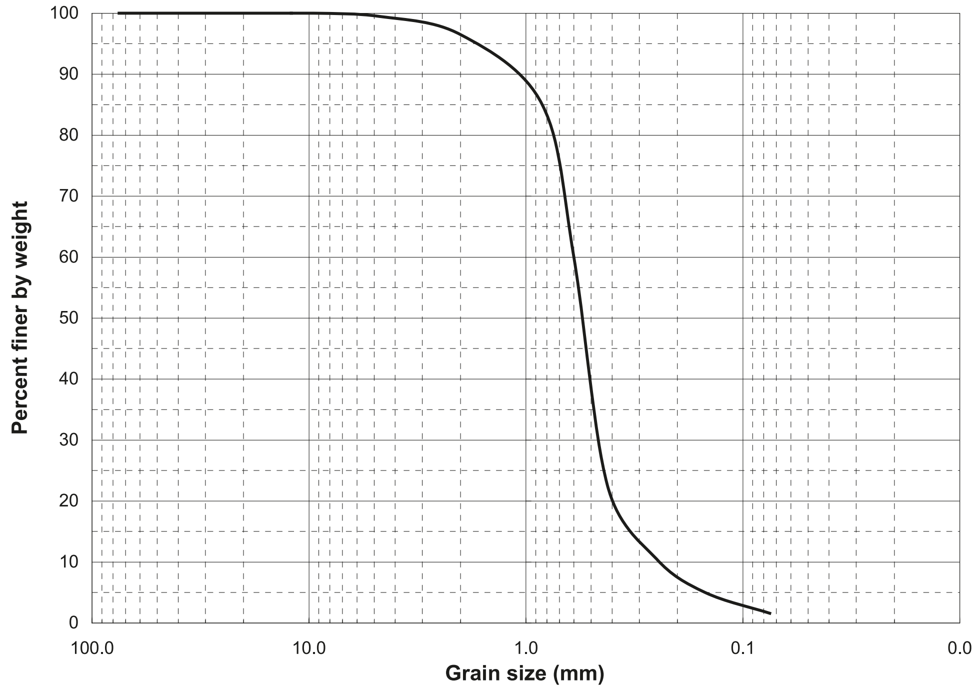


Fig. 5. Average foundation soil profile and SPT results for test sites 1 (D-781) and 2 (D-785). (Note: no fines or gravel corrections applied.)

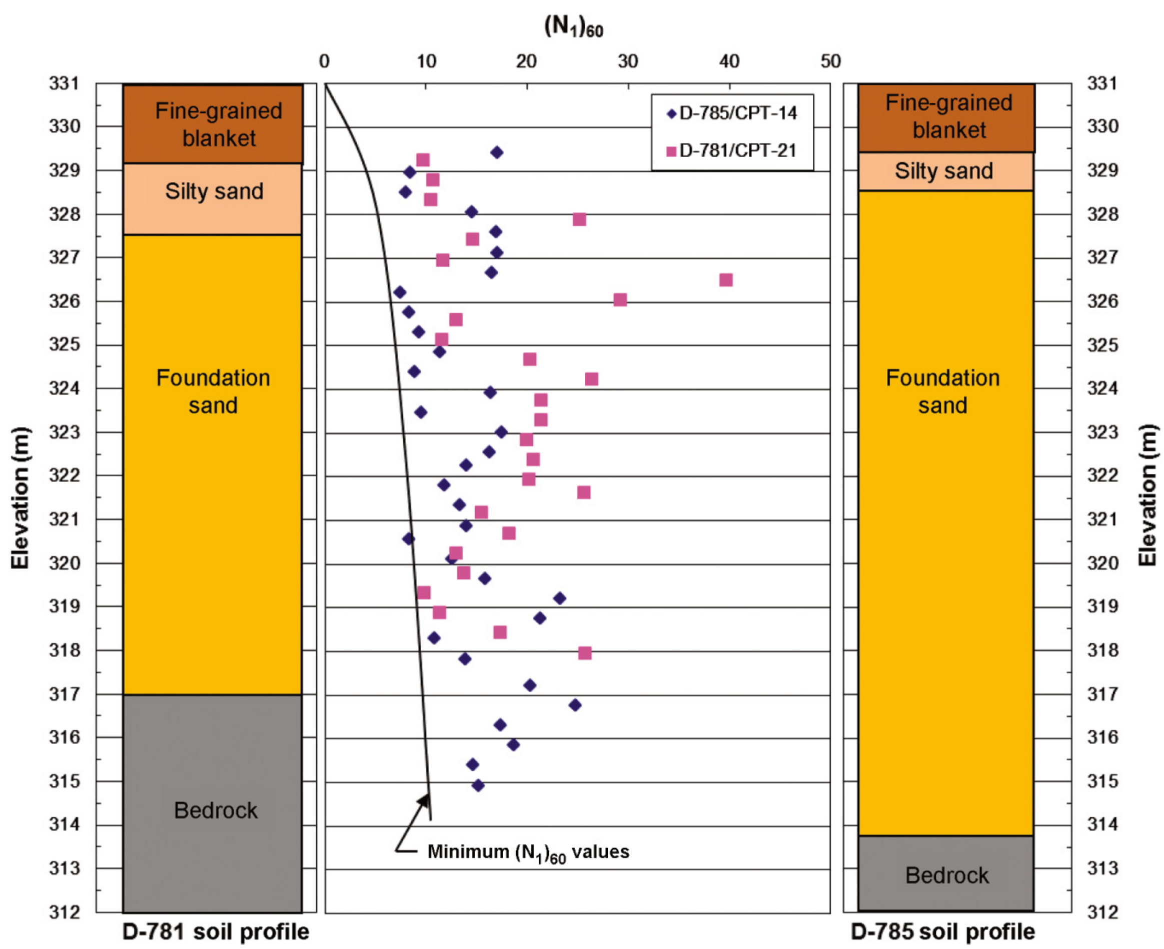
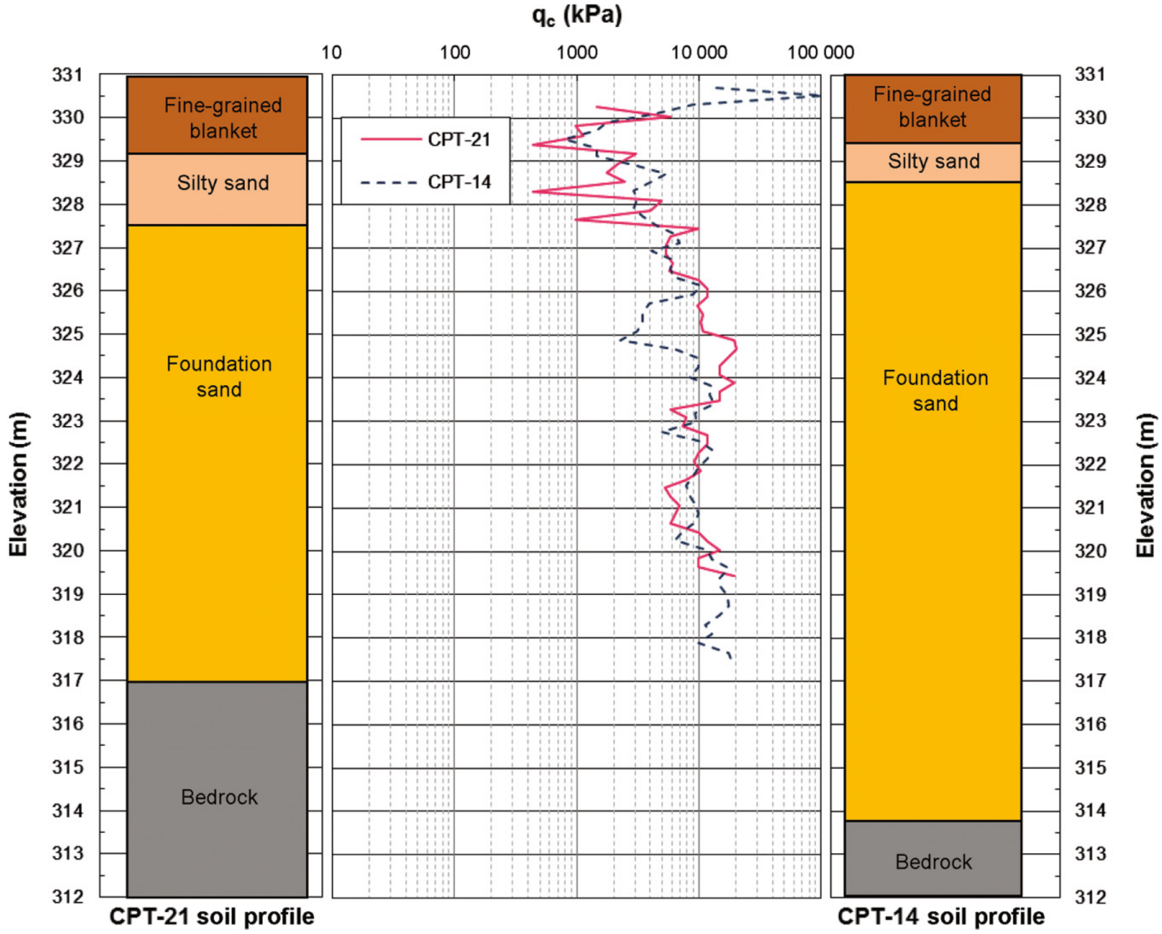


Fig. 6. Average foundation soil profile and CPT results for test sites 1 (CPT-21) and 2 (CPT-14). q_c, uncorrected cone tip resistance.



rected cone tip resistance, q_c, near the bottom of the fine-grained blanket around 1000 kPa (approx 10 kg/cm^2). The tip resistance increases to over 10 000 kPa (approximately 100 kg/cm^2) near the bedrock (elevation 317 m; 1040 ft). The CPT sounding CPT-21 at site 1 produced a similar sounding as CPT-14 from site 2 except for the loose zones in CPT-21 at about elevation 328.5 m (1077.8 ft) and about elevation 327.5 m (1074.5 ft). However, CPT-14 from site 2 shows a loose zone that is not present in CPT-21 at about elevation 324–325.5 m (1063–1068 ft).

Several authors (e.g., Popescu et al. 1997, 1998; Yoshimine et al. 2000) conclude that the loosest zone(s) of foundation sands will control liquefaction potential. The loosest zones of the Milford Dam foundation sands were identified using the minimum $(N_1)_{60}$ and q_c values. A line showing minimum values of $N_{1,60}$ versus depth is shown in Fig. 5. The range of $(N_1)_{60}$ is 7 at elevation 326 m (1070 ft) to 10 or 11 at elevation 318 m (1045 ft). The minimum q_c values are 960 kPa (10 kg/cm^2) at about elevation 327.5 m (1074.5 ft) up to 9600 kPa (100 kg/cm^2) at elevation 320 m (1050 ft).

Fines content comparison

Traditionally split-spoon samples obtained during an SPT are used to provide a representative grain-size distribution for soil classification and liquefaction potential analyses. However, split-spoon samples can experience poor recovery because of the tendency to lose cohesionless or coarser sand from the end of a sample tube during sampler extraction when sampling highly stratified sand and silty sand materials below the groundwater surface. This can occur even though techniques are used to increase recovery, including elimination of liners from the split-spoon sampler and efforts made to adjust and optimize drill fluid viscosity and density. Thus, SPT sampling can result in a nonrepresentative gradation. The SPT split-spoon sample recoveries were generally no more than about 50% at Milford Dam, with a typical recovery of 20%–30%. Frequently, the portion recovered in the SPT split-spoon was the finer grained or more cohesive material, which was not always representative of the overall material. A sample recovery of 20%–30% is probably not typical of other sites, so the fines content comparison presented below may not be applicable to other sites.

A sample recovery of 100% was often achieved at Milford Dam with in situ ground freezing and high-quality fixed-piston unfrozen tube sampling. The difference in sample gradation between the various sampling methods is discussed below by comparing the gradation of the split-spoon samples prior to ground freezing, high-quality 75 mm (3 in.) conventional fixed-piston samples, and 150 mm (6 in.) frozen ground samples from the two test sites. Some investigators (Seed et al. 1985) have reported more fines with SPT samples than with thin-wall push samples and concluded they were contaminated with drill fluid. However X-ray diffraction results from Milford Dam SPT samples show little montmorillonite content, i.e., drill fluid.

The results of the fines content measurements are presented in Fig. 7, with a profile of the foundation soils for comparison purposes. Figures 7*a* and 7*b* show the fines content from 150 mm (6 in.) frozen ground samples and split-spoon samples with elevation and soil type for sites 1 and 2. For site 1 (Fig. 7*a* and boring D-781), the average fines content of the in situ frozen samples is 7.9%, and the average fines content of the split-spoon samples is 9.5% for the foundation sands, i.e., fines content data for the silty sand and fine-grained blanket are not included in the average. A large difference in fines content occurs between elevations 327 and 326 m, where the frozen ground samples exhibit a much higher fines content. With the exception of the four samples that yield a much higher fines content at elevations 327 and 326 m and a sample around elevation 324.5 m, the frozen ground samples exhibit a slightly lower fines content than the split-spoon samples especially at deeper depths. No data from frozen ground samples were obtained below elevation 323.7 m because the samples were lost during the thawing process.

The fines content difference between sampling methods is more dramatic at site 2 (Fig. 7*b* and borings D-785 and D-785A), where the average fines content for the frozen ground samples is 4.5% and the average fines content of the split-spoon samples is 17.3% for the foundation sands, i.e., fines content data for the silty sand and fine-grained blanket are not included in the average. In addition, the difference in fines content are consistent with depth, and there are no frozen core samples that exhibit substantially higher fines content than the split-spoon samples, as was observed for five split-spoon samples at site 1.

Thus, split-spoon samples appear to yield a greater fines content than the frozen ground or in situ soil at Milford Dam probably because stratified sands like those found in alluvial deposits, coarser, more cohesionless material, tend to be lost from the end of the SPT sample tube, leaving the finer grained or more cohesive material with a higher percentage of fines in the sample. This conclusion is based on the split-spoon samples yielding consistently higher fines content than the frozen ground samples at Milford Dam. This result has implications for liquefaction assessments because split-spoon samples may overestimate the actual fines content, possibly resulting in a deposit being classified as nonliquefiable. Based on the Milford Dam data in Fig. 7, split-spoon samples can overestimate the actual fines content by 1%–10%, which can result in an overestimate of the fines content correction in a liquefaction potential analysis. Additionally, the site characterization may result in mischaracterizing the deposit as silty sand when it should actually be characterized as a clean sand with siltier layers.

Cyclic triaxial compression test results: frozen ground (undisturbed) samples

A suite of laboratory triaxial shear tests were performed on samples of the foundation sands that were recovered after in situ ground freezing. Seventy-six samples were subjected to cyclic triaxial compression testing. Consolidation pressures of 96, 431, and 575 kPa (2000, 9000, and 12 000 lb/ft^2) were used for the triaxial compression testing. Additional cyclic triaxial testing was performed on fixed-piston unfrozen thin-wall tube samples for comparison purposes. Four series of tests were performed at 96 kPa (2000 lb/ft^2) to compare the cyclic strength of samples obtained with fixed-piston unfrozen samples to strength obtained by in situ frozen ground samples, i.e., effect of sample disturbance with piston samples. Samples from both sites 1 and 2 were used. Cyclic triaxial test results were grouped based on classification, grain size, and dry unit weight. These results are presented in Fig. 8 and show the number of cycles required to reach 10% double amplitude strain (or approximately the number of cycles to reach 100% pore-water pressure generation) versus cyclic stress ratio, $\sigma_d/2\sigma'_{3c}$, where σ_d is the cyclic deviator stress ($\sigma_1 - \sigma_3$, where σ_1 is the major principal stress and σ_3 is the minor principal stress), and σ'_{3c} is the consolidation stress.

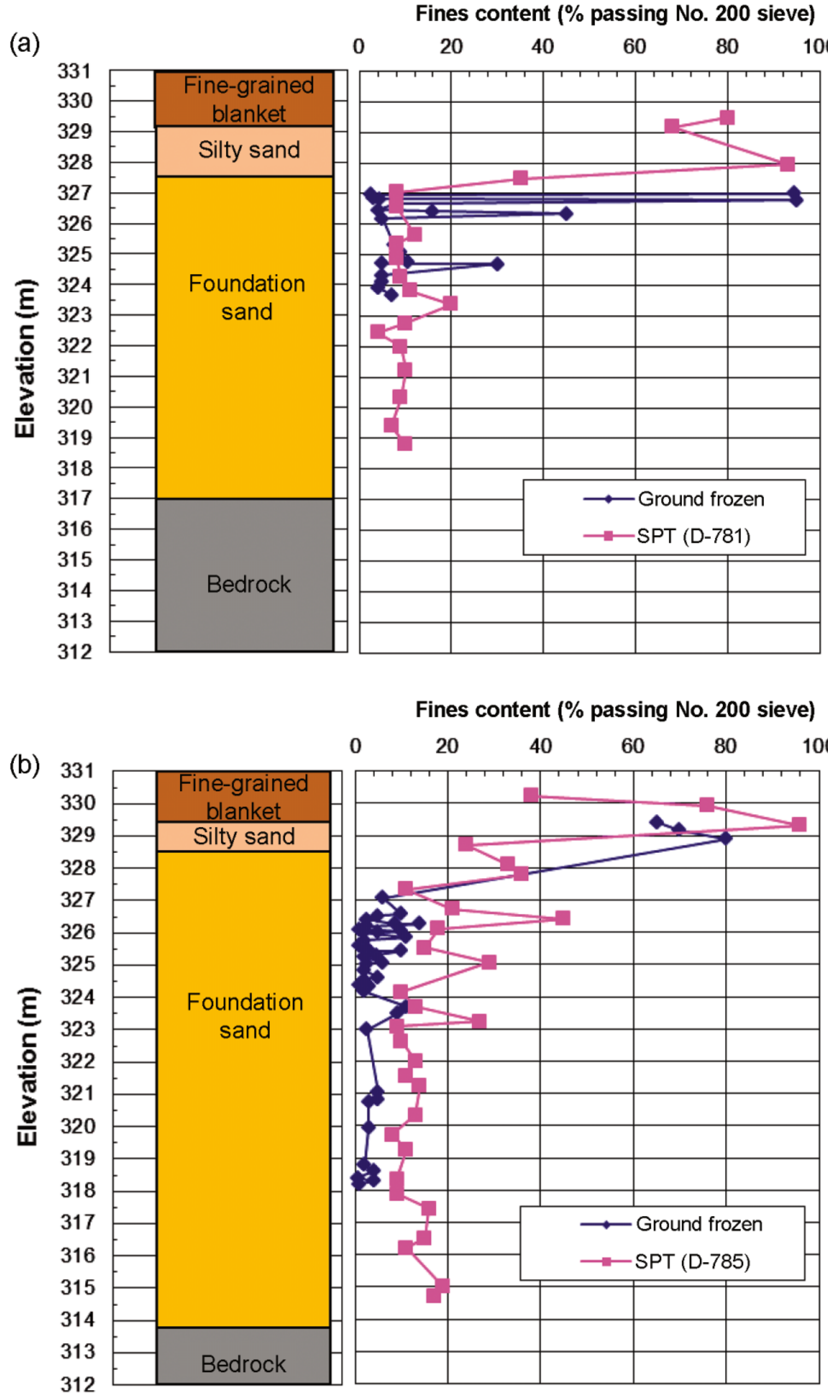
For loose sands (sites 1 and 2 at a depth of 4–7 m (12–22 ft)), the test results show the cyclic triaxial strengths are greater for the conventional fixed-piston unfrozen samples than for the in situ frozen ground specimens (see Fig. 8*a*). Conversely, for dense sands (site 1 at a depth of 7–7.5 m (23–25 ft)), the strength is greater for the in situ frozen ground specimens than fixed-piston unfrozen samples (see Fig. 8*b*). Comparison of the void ratio measured prior to consolidation suggests that the thin-walled sampling densified the loose sands because the void ratio ranges from 0.44 to 0.54 for the unfrozen thin-walled specimens and from 0.50 to 0.66 for the frozen ground specimens. Conversely, the thin-walled sampling appeared to loosen the dense sands because the void ratio is 0.44 for the unfrozen thin-walled specimens and 0.41 for the frozen ground specimens. Thus, the effect of conventional fixed-piston sampling on in situ unit weight of the loose sands is an increase of 0.5–3.2 kN/m^3 (3–7 lb/ft^3) and a decrease of about 0.9 kN/m^3 (2 lb/ft^3) or more for the dense sands. The degree to which aging and development of bonds at the grain-to-grain contact for the Milford Dam sands increases the cyclic strength or liquefaction resistance cannot be assessed on the basis of the comparative testing with other undisturbed samples. However, it is clear that conventional sampling can cause an increase in density that results in an overall strength increase for loose sands even though disturbance likely destroys some of the interparticle bonding that may be present. For dense sands, the strength is decreased probably due to a loss of bonding and decrease in density. This is consistent with the laboratory investigation results obtained by Singh et al. (1979).

Monotonic undrained yield shear strength testing

Eleven frozen ground specimens were loaded only monotonically in CU triaxial tests to understand the monotonic stress–strain behavior and measure the static undrained yield strength and critical state strength. The yield shear strength, Su(yield), is the peak resistance available during undrained loading of a saturated, contractive soil (Terzaghi et al. 1996). The yield strength ratio, Su(yield)/σ'_{1c}, is defined as the yield shear strength normalized by the initial consolidation stress, σ'_{1c}. The yield strength was measured in monotonic CU triaxial tests, but none of the specimens were strained far enough to reach a critical state condition.

The undrained yield strength ratio can be expressed in terms of a flow liquefaction surface (FLS) as suggested by

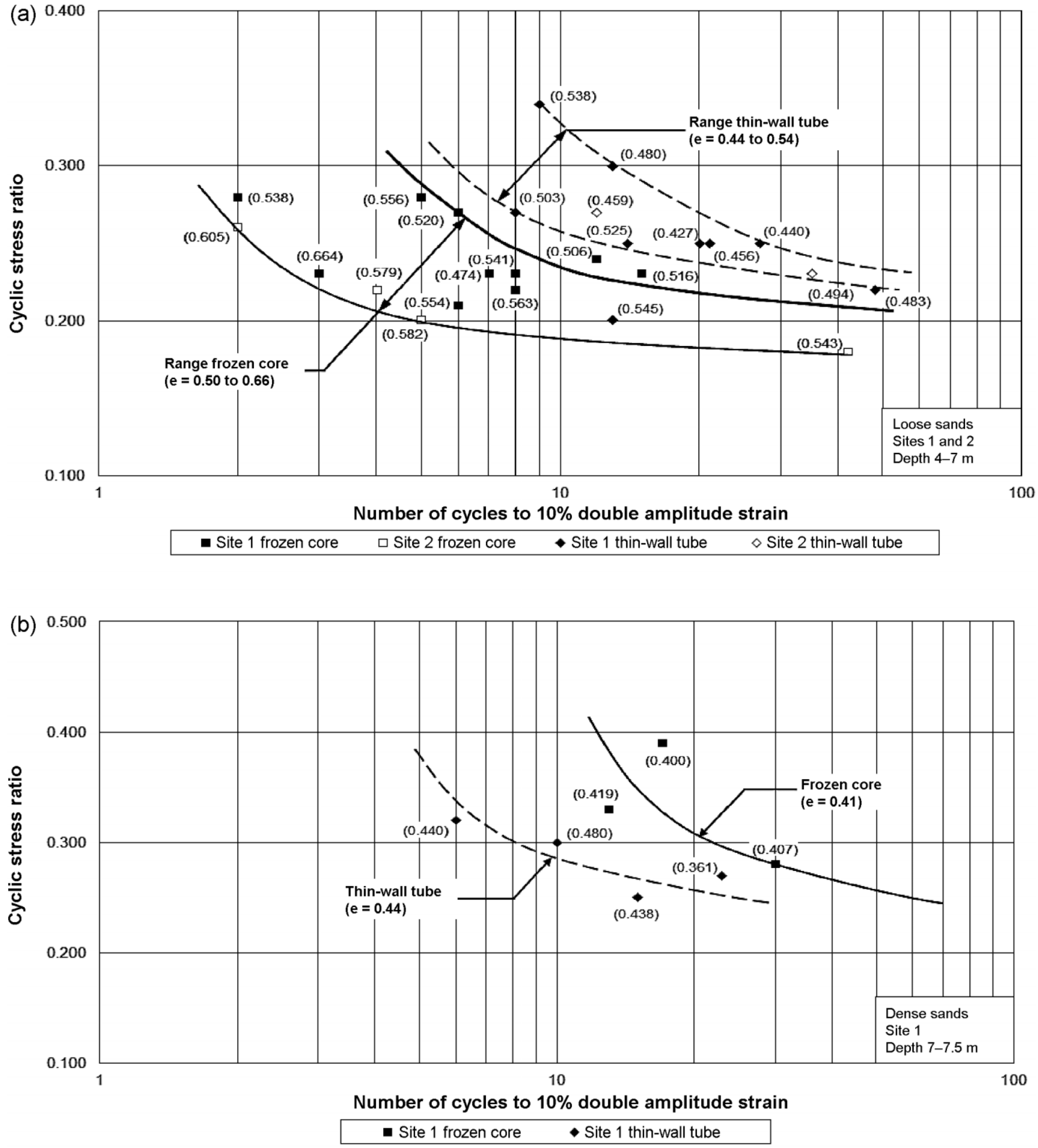
Fig. 7. Fines content of samples with respect to depth and average soil profile at (*a*) site 1 (D-781) and (*b*) site 2 (D-785).



Vaid and Chern (1985). A number of researchers, e.g., Hanzawa et al. (1979), Hanzawa (1980), and Sladen et al. (1985), suggest an analogous surface, i.e., the collapse surface (CS), which is assumed to project linearly through the steady-state point. Since Sladen et al. (1985), the preponderance of available experimental data appears to support projection through the origin (FLS) and not through the steady-state point (CS) (Kramer 1996). Therefore, a FLS is used herein to characterize the undrained yield strength and undrained yield strength ratio.

The frozen ground, i.e., undisturbed, and remolded triaxial specimens produced different shear behaviors, which are visualized in σ'_{mean}–q space, where q is the deviator stress ($q = \sigma_1 - \sigma_3$). Figure 9 shows the stress paths for monotonic CU triaxial specimens U1 through U11 except U2 and U10, which did not produce usable results because the U2 specimen was lost during consolidation and the U10 results are erratic. The stress paths shown in Fig. 9 show that large positive pore-water pressures did not develop in the frozen

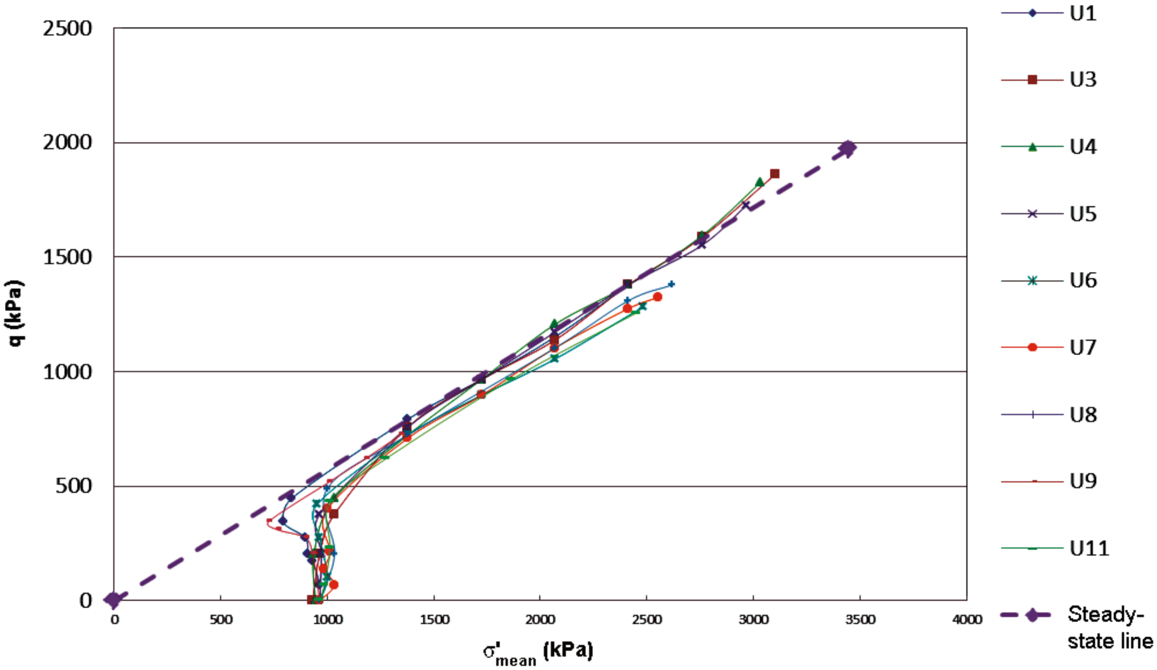
Fig. 8. Comparison of cyclic strength ratios from in situ frozen ground and fixed-piston push tube samples: (*a*) loose sands depth of 4–7 m at sites 1 and 2; (*b*) dense sands depth of 7–7.5 m at site 1. *e*, void ratio.



ground specimens, and thus these specimens did not liquefy or reach a critical strength. Specimens U3, U4, U5, U7, and U8 exhibit a peak strength at axial strain less than about 15%, and then the shear resistance, i.e., *q*, decreases and was still decreasing at the end of the test. The other frozen ground specimens, U1, U6, U9, and U11, exhibit a gradually increasing shear resistance, *q*, until an axial strain of about 20% and was gradually decreasing at the end of the test.

In tests U3–U8, the absolute value of pore pressure reached atmospheric or slightly below, i.e., the induced pore pressure was equal or slightly higher than the back pressure, indicating possible cavitation during the test due to specimen dilation. Thus, the measured peak strength is probably too low because the soil would have continued to dilate and mobilize more resistance had cavitation not occurred. In some tests (U3–U5), the effective minor principal stress becomes essentially constant beyond the strain at which the maximum pore pressure, e.g., atmospheric pressure, is reached. In tests U1 and U11, the pore pressure is close to atmospheric pressure, so air may have been released because back pressure was used to dissolve air bubbles to achieve saturation; therefore, when negative pore pressures were induced by cavita-

Fig. 9. Response of nine frozen ground specimens in isotropically CU triaxial tests in σ'_{mean}–q space (USACE KCD 1986*c*).



tion, some of these dissolved air bubbles may have been released even though the pore pressures are at or above atmospheric pressure. The only specimen in which cavitation did not occur was U9 because the induced pore pressures were such that the absolute pore pressure value remained positive throughout the test.

Conversely, the stress paths for the remolded specimens show that large positive pore-water pressures developed because these specimens were prepared to be loose enough so that a flow liquefaction surface could be derived (see Fig. 10). The steady-state point for the remolded specimens is located near where the FLS intersects the steady-state strength envelope. The steady-state point is at the condition in which a state of deformation at constant shear and normal stress is reached, with the latter value being more reliable. The state of stress moves close to or at a point on the strength envelope, but it moves up or down along the envelope to reach the steady state.

Comparison of SPT and undrained yield strength ratios

Figure 11 shows SPT results before and after applying gravel corrections to account for gravel-size particles at sites 1 and 2. At site 1, some gravel-sized limestone “float” rock was encountered from a depth of about 4–8 m. At site 2, gravel-sized particles were encountered occasionally below 8 m. Figure 12 presents undrained yield strength ratios from triaxial compression tests using frozen ground specimens as a function of equivalent clean sand blow count, $(N_1)_{60\text{-CS}}$. The value of $(N_1)_{60\text{-CS}}$ for the undrained yield strength ratios shown in Fig. 12 was obtained for each triaxial data point by estimating the blow count from Fig. 11 using the depth where the sample was obtained. The blow count from Fig. 11 was then corrected to a clean sand blow count using eq. [2] and the fines content adjustment proposed by Seed et al. (1985). For comparison purposes, Fig. 12 also shows the yield strength relationships suggested by Stark and Mesri (1992). The data in Fig. 12 shows considerable scatter. However, the yield strength relationship presented by Stark and Mesri (1992) provides a reasonable lower bound to the data.

$$[2] \qquad (N_1)_{60\text{-CS}} = (N_1)_{60} + \Delta(N_1)_{60}$$

Comparison of measured cyclic stress ratios and liquefaction triggering relationships

The cyclic stress ratios in Fig. 8 obtained from cyclic CU triaxial tests corresponding to 10% double amplitude strain were also compared with current liquefaction triggering relationships to investigate the effectiveness of in situ ground freezing. The data in Fig. 9 was not used in this comparison because the in situ frozen ground samples used to create Fig. 9 were tested monotonically not cyclically. This explains the difference in the number of data points in Figs. 9 and 12. The cyclic stress ratios at 10% double amplitude strain in Fig. 8 were converted to equivalent cyclic stress ratios at 15 cycles, which corresponds to a magnitude 7.5 earthquake using the following relationship suggested by Idriss and Boulanger (2009):

$$[3] \qquad \frac{N_A}{N_B} = \left(\frac{CSR_B}{CSR_A}\right)^{1/b}$$

where N_A and N_B are the number of cycles (N) equivalent to cyclic stress ratios (CSR) CSR_A and CSR_B, respectively, and $-b$ corresponds to the slope of a log(CSR) versus log(N) plot. A b value of 0.34 was used, which is representative of clean sands. The initial number of cycles to cause 10% double amplitude strain and the corresponding cyclic stress ratio were obtained from Fig. 8. The stress ratios obtained for 15 cycles were then multiplied by 0.6 to obtain the “field” cyclic stress ratio to model a cyclic simple shear mode of shear in the field as suggested by Seed and Peacock (1971) and Castro (1975). According to Seed and Peacock (1971), the conver-

Fig. 10. Response of 11 remolded specimens in isotropically CU triaxial tests in σ'_{mean}–q space with flow liquefaction surface (USACE 1986*d*).

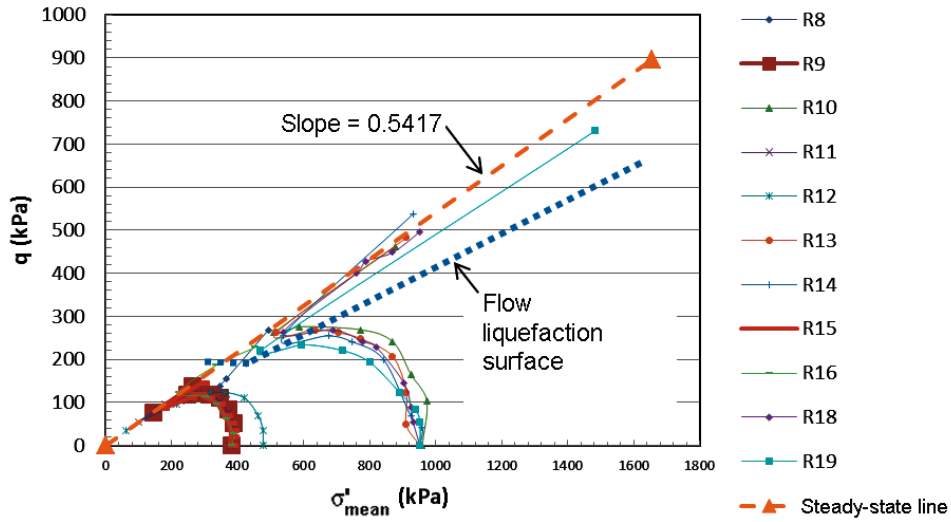
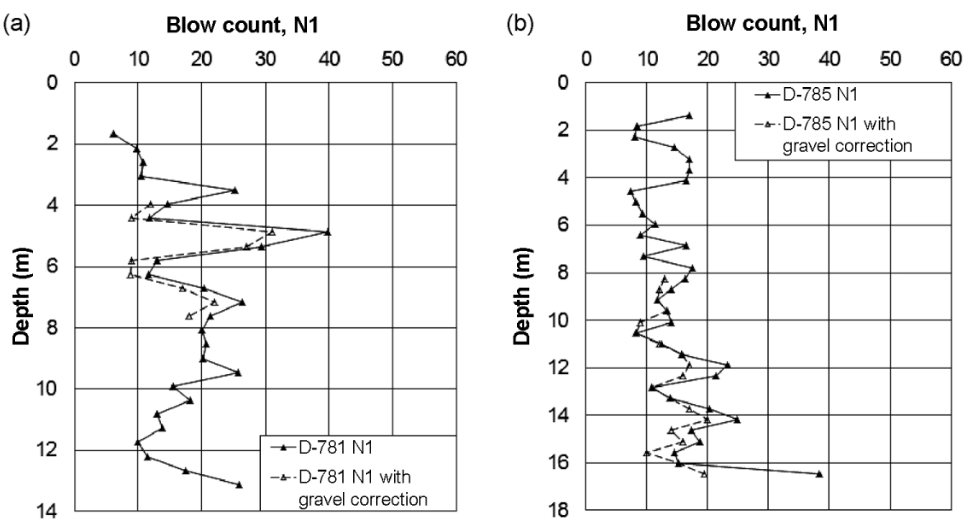


Fig. 11. Comparison of measured blow counts versus depth with gravel correction: (*a*) site 1 (D-781) blow counts; (*b*) site 2 (D-785) blow counts.



sion factor from a triaxial cyclic stress ratio to a "field" cyclic stress ratio, c_r, varies from 0.57 at an overconsolidation ratio (OCR) of 1 to 0.8 at an OCR of 8. The sands sampled at the toe of Milford Dam are slightly overconsolidated because the groundwater level has risen slightly with reservoir impoundment, so a value of c_r equal to 0.6 was used for this conversion. Figure 13 plots the resulting cyclic stress ratios versus equivalent clean sand blow count using the same procedure as used for Fig. 12. In other words, the value of $(N_1)_{60\text{-CS}}$ for the undrained yield strength ratios shown in Fig. 13 was obtained for each triaxial data point by estimating the blow count from Fig. 11 using the depth where the sample was obtained. The blow count from Fig. 11 was then corrected to a clean sand blow count using eq. [2] and the fines content adjustment from Seed et al. (1985). Both frozen ground samples and thin-wall tube samples are plotted in Fig. 13. The data obtained are compared to the liquefaction triggering relationships by Idriss and Boulanger (2004) and Youd et al. (2001). The measured strength ratios from frozen ground samples are in good agreement with both triggering relationships. The majority of these cyclic tests plot slightly below the empirical relationship, i.e., are conservative, due to the redistribution of water in the specimen (Castro 1969). However, thin-walled tube specimens show a higher strength ratio at lower blow counts and a lower stress ratio at high blow counts, which is in good agreement with the previous conclusion that thin-walled tube samples tend to densify the loose sands and loosen the dense sands. This favorable comparison of the measured cyclic stress ratios with existing triggering relationships suggests that the frozen ground sampling technique used at Milford Dam resulted in high-quality specimens that can be used in a liquefaction assessment investigation.

Conclusions

The USACE completed a liquefaction potential analysis as part of the seismic evaluation of Milford Dam in 1986. The 1986 data is used herein to compare fines content (% passing No. 200 sieve) data from in situ frozen and SPT samples and suggest fines content can be overestimated by 1%–10% by SPT samples in stratified sand deposits. This result may have implications for liquefaction assessments because split-spoon

Fig. 12. Undrained yield shear strength ratio as a function of equivalent clean sand blow count and relationship proposed by Stark and Mesri (1992).

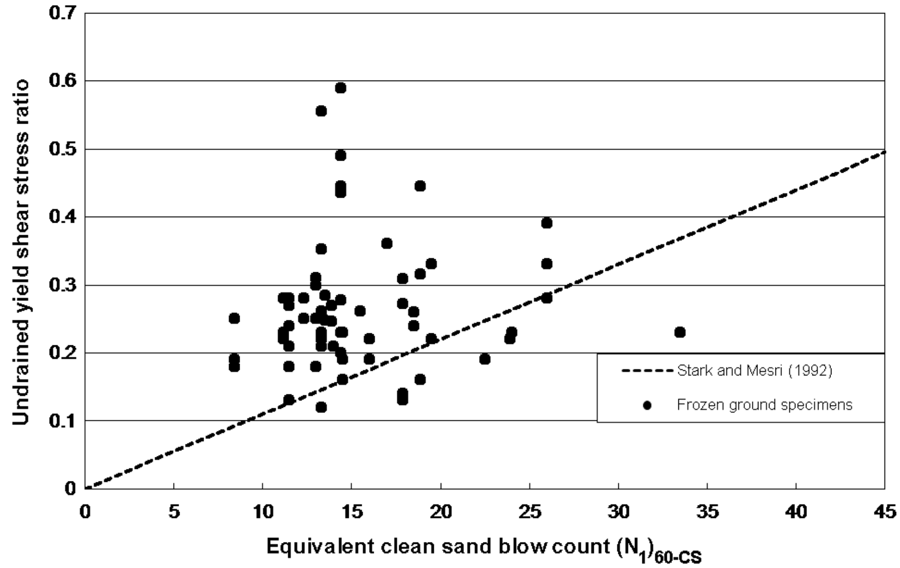
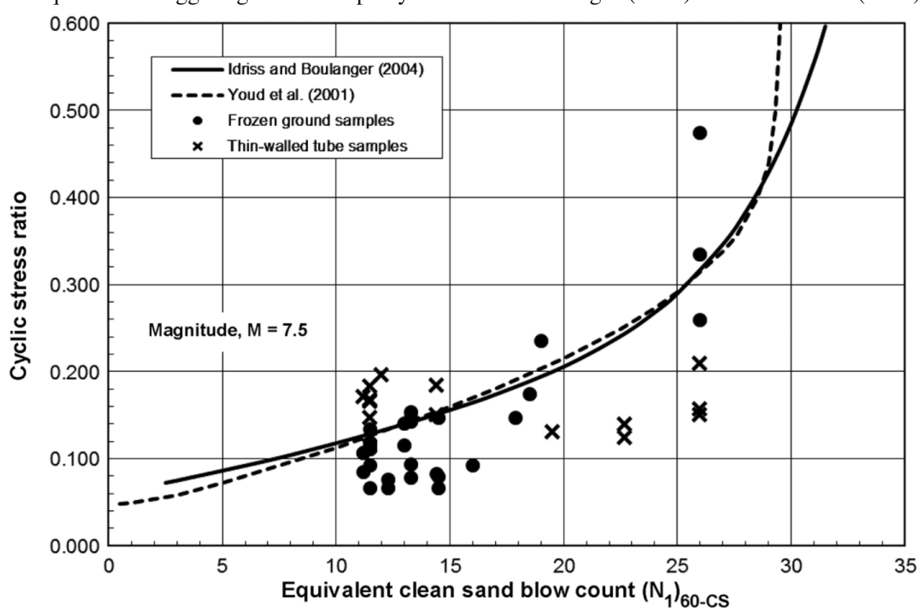


Fig. 13. Measured stress ratios at 10% double amplitude strain from frozen ground and thin-walled tube samples adjusted to 15 cycles (earthquake magnitude 7.5) and liquefaction triggering relationships by Idriss and Boulanger (2004) and Youd et al. (2001).



samples may overestimate the actual fines content, resulting in a liquefiable deposit being classified as nonliquefiable. In addition, the data shows the effectiveness of ground freezing on maintaining in situ soil structure and aging of the foundation sands at Milford Dam, which suggests that high-quality sand samples can be obtained using the in situ ground freezing process described herein but at considerable cost.

Cyclic triaxial compression testing on loose sands (sites 1 and 2 at a depth of 4–7 m (12–22 ft)) obtained using conventional fixed-piston unfrozen specimens gave undrained strengths that are greater than in situ frozen ground specimens. Conversely, for dense sands (site 1 and a depth of 7–7.5 m (23–25 ft)), the undrained strength is greater for in situ frozen ground specimens than fixed-piston unfrozen specimens, which means the loose sands can be densified during sampling while dense sands can be loosened during sampling. Because the densification of loose sands and loosening of denser sands occurred during sampling and prior to freezing, aboveground freezing also was not effective in preserving in situ sand structure. In other words, aboveground freezing of samples does not yield significantly better samples than fixed-piston sampling because disturbance is introduced during the sampling process, i.e., prior to freezing. Therefore, to obtain the best quality samples, in situ ground freezing should be used. This data also suggest that in situ frozen ground samples appear to adequately preserve the aging that was present in the foundation sands prior to fixed-piston sampling.

Finally, a simplified column analysis using the computer program SHAKE (Schnabel et al. 1972) and the results of the triaxial compression testing of the frozen ground showed that the dam has an adequate factor of safety against liquefaction except for a limited extent beneath the downstream toe. Monotonic testing of frozen ground samples did not result in liquefaction or development of a critical strength under field consolidation stresses. As a result, Milford Dam was determined to be safe against liquefaction-induced downstream slope failure, and no improvement of the foundation sands was undertaken (USACE KCD 1986*c*, 1986*d*).

Acknowledgements

The leadership, support, and encouragement provided by Jacob Redlinger (deceased, formerly Chief, Geotechnical Branch, Missouri River Division, US Army Corps of Engineers (USACE)) for the foundation investigations conducted at Milford Dam is gratefully acknowledged. The authors also acknowledge and appreciate the excellent assistance of Luis Pazmino, a graduate research assistant at the University of Illinois, Urbana, Illinois, in finalizing this manuscript.

References

Castro, G. 1969. Liquefaction of sands. Harvard University, Cambridge, Mass. Harvard Soil Mechanics Series No. 81.

Castro, G. 1975. Liquefaction and cyclic mobility of sands. Journal of the Geotechnical Engineering Division, ASCE, **101**(GT6): 551–569.

Castro, G. 1985. Trip report letter to Walberg, F.C. US Army Corps of Engineers, Kansas City District, Kansas City, Mo.

Hanzawa, H. 1980. Undrained strength and stability analysis for a quick sand. Soils and Foundations, **20**(2): 17–29.

Hanzawa, H., Itoh, Y., and Suzuki, K. 1979. Shear characteristics of a quick sand in the Arabian Gulf. Soils and Foundations, **19**(4): 1–15.

Hvorslev, M.J. 1948. Subsurface exploration and sampling of soils for civil engineering purposes. US Army Corps of Engineers Waterways Experiment Station, Vicksburg, Miss.

Idriss, I.M., and Boulanger, R.W. 2004. Semi-empirical procedures for evaluating liquefaction potential during earthquakes. *In* Proceedings of the 11th International Conference on Soil Dynamics and Earthquake Engineering and 3rd International Conference on Earthquake Geotechnical Engineering. University of California, Berkeley, Calif., 7–9 January 2004. *Edited by* D. Doolin, A. Kammerer, T. Nogami, R.B. Seed, and I. Towhata. Stallion Press, Singapore. Vol. 1, pp. 32–56.

Idriss, I.M., and Boulanger, R.W. 2009. Soil liquefaction during earthquakes. MNO-12. Earthquake Engineering Research Institute (EERI), Oakland, Calif.

Kovacs, W.D., Evans, J.C., and Griffith, A.H. 1977. Towards a more standardized SPT. *In* Proceedings of the Ninth International Conference on Soil Mechanics and Foundation Engineering (ICSMFE), Tokyo, Japan, July 1977. Japanese Society of Soil Mechanics and Foundation Engineering, Tokyo, Japan. Vol. 2, pp. 269–276.

Kramer, S.L. 1996. Geotechnical earthquake engineering. 1st ed. Prentice Hall, Saddle Brook, N.J.

Osterberg, J.O., and Varaksin, S. 1973. Determination of relative density below groundwater table. *In* Evaluation of relative density and its role in geotechnical projects involving cohesionless soils. ASTM Special Technical Publication 523. *Edited by* E.T. Selig and R.S. Ladd. American Society for Testing and Materials, Philadelphia, Pa. pp. 364–376.

Popescu, R., Prevost, J.H., and Deodatis, G. 1997. Effects of spatial variability on soil liquefaction: some design recommendations. Géotechnique, **47**(5): 1019–1036. doi:10.1680/geot.1997.47.5.1019.

Popescu, R., Prevost, J.H., and Deodatis, G. 1998. Characteristic percentile of soil strength for dynamic analyses. *In* Geotechnical earthquake engineering and soil dynamics III. GSP 75. *Edited by* P. Dakoulas and M. Yegian. American Society of Civil Engineers, Reston, Va. pp. 1461–1471.

Poulos, S.J., Castro, G., and France, W. 1985. Liquefaction evaluation procedure. Journal of Geotechnical Engineering, **111**(6): 772–792. doi:10.1061/(ASCE)0733-9410(1985)111:6(772).

Schnabel, P.B., Lysmer, J., and Seed, H.B. 1972. SHAKE: a computer program for earthquake response analysis of horizontally-layered sites. Earthquake Engineering Research Center, University of California, Berkeley, Calif. Report No. EERC-72/12.

Seed, H.B. 1981. Earthquake-resistant design of earth dams. *In* Proceedings of the International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Mo. Vol. 3, pp. 1157–1173.

Seed, H.B. 1983. Earthquake-resistant design of earth dams. *In* Proceedings of the Symposium on Seismic Design of Embankments and Caverns, Philadelphia, Pa., 16–20 May 1983. *Edited by* T.R. Howard. American Society of Civil Engineers, New York. pp. 41–64.

Seed, H.B., and Peacock, W.H. 1971. Test procedures for measuring soil liquefaction characteristics. Journal of the Soil Mechanics and Foundations Division, ASCE, **97**(SM8): 1099–1119.

Seed, H.B., Tokimatsu, K., Harder, L.F., and Chung, R. 1985. Influence of SPT procedures in soil liquefaction resistance evaluations. Journal of the Geotechnical Engineering Division, ASCE, **111**(12): 861–878.

Singh, S., Seed, H.B., and Chan, C.K. 1979. Undisturbed sampling and cyclic load testing of sands. University of California, Berkeley, Calif. Report No. UCB/EERC-79/33.

Sladen, J.A., D'Hollander, R.D., and Krahn, J. 1985. The liquefaction of sands, a collapse surface approach. Canadian Geotechnical Journal, **22**(4): 564–578. doi:10.1139/t85-076.

Stark, T.D., and Mesri, G. 1992. Undrained shear strength of liquefied sands for stability analysis. Journal of Geotechnical Engineering, **118**(11): 1727–1747. doi:10.1061/(ASCE)0733-9410(1992)118:11(1727).

Terzaghi, K., Peck, R.B., and Mesri, G. 1996. Soil mechanics in engineering practice. 3rd ed. John Wiley and Sons, Inc., New York.

USACE. 1939. Report on the slide of a portion of the upstream face of the Fort Peck Dam. US Army Corps of Engineers (USACE), War Department, Fort Peck, Mont.

USACE. 1986*a*. Report of seismic study for Milford Dam, Republican River, Kansas. Phase II — undrained steady strength testing of foundation sands, part 1 (remolded specimens). US Army Corps of Engineers (USACE), Missouri River Division Laboratory, Omaha, Nebr.

USACE. 1986*b*. Report of seismic study for Milford Dam, Republican River, Kansas. Phase I — cyclic triaxial testing of frozen in situ sand samples. US Army Corps of Engineers (USACE), Missouri River Division Laboratory, Omaha, Nebr.

USACE KCD. 1986*c*. Milford Dam, Republican River, Kansas. Design Memorandum No. 35. Seismic evaluation: evaluation of embankment and foundation earthquake resistance, November. US Army Corps of Engineers (USACE), Kansas City District (KCD), Kansas City, Mo.

USACE KCD. 1986*d*. Milford Dam, Republican River, Kansas. Design Memorandum No. 35A. Seismic evaluation: liquefaction potential based on undrained steady state strength, supplement A, November. US Army Corps of Engineers (USACE), Kansas City District (KCD), Kansas City, Mo.

Vaid, Y.P., and Chern, J.C. 1985. Cyclic and monotonic undrained response of saturated sands. *In* Advances in the art of testing soils under cyclic conditions. *Edited by* V. Khosla. American Society of Civil Engineers, New York. pp. 120–147.

Walberg, F.C. 1978. Freezing and cyclic triaxial behavior of sands. Technical Note. Journal of the Geotechnical Engineering Division, ASCE, **104**(GT5): 667–671.

Yoshimi, Y., Hatanaka, M., and Oh-oka, H. 1978. Undisturbed sampling of saturated sands by freezing. Soils and Foundations, **18**(3): 59–73.

Yoshimine, M., Robertson, P.K., and Wride (Fear), C.E. 2000. Undrained shear strength of clean sands to trigger flow liquefaction. Canadian Geotechnical Journal, **36**(5): 891–906. doi:10.1139/t99-047.

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., and Stokoe, K.H. 2001. Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. Journal of Geotechnical and Geoenvironmental Engineering, **127**(10): 817–833. doi:10.1061/(ASCE)1090-0241(2001)127:10(817).