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# Liquefaction Analysis for Rubber Dam and Review of Case Histories of Liquefaction of Gravels

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# Liquefaction Analysis for Rubber Dam and Review of Case Histories of Liquefaction of Gravels

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SYNOPSIS: A geotechnical investigation for the design and construction of an air-inflated rubber dam (for ground-water recharge) was performed, including evaluation of the liquefaction potential of the dam's foundation. The dam, about 300 feet long and 13 feet high when fully inflated is located along Alameda Creek at the upper part of the alluvial fan near Fremont, California. The sediments at the site are generally sandy gravels and gravelly sands.

In order to fully investigate the liquefaction potential of the gravelly soils, the soils themselves were studied, and case histories involving similar conditions were reviewed. These case histories indicate that, in all cases where liquefaction appears to have occurred, the gravelly soils were loose to very loose and at shallow to moderate depths. For most of the reported cases the gravelly soils contained significant quantities of fines, and it was the fine matrix which actually liquefied. There are also numerous cases where earthquake induced liquefaction of gravelly soils (loose to dense) did not occur, but which are not given adequate coverage in the literature. The results of percolation tests, SPT blow count data, and comparison of particle-size distributions with gravelly soils that liquefied in past events all indicate that the soils at the rubber dam site have a very low probability of liquefaction.

## REVIEW OF CASE HISTORIES

Actual documented cases of liquefaction of saturated gravelly soil deposits during past earthquakes are quite rare. This is due to the fact that, in nature, these materials are generally free-draining and will not develop high excess pore pressures during an earthquake. However, under certain conditions where the materials are in a loose state, contain large quantities of fines, or have impeded drainage, it is conceivable that they could liquefy or develop high excess pore-water pressures during strong earthquake ground-shaking.

In recent years, a number of cases in which earthquake-induced liquefaction of gravelly deposits has supposedly been observed have been described in the literature (Harder, 1988, Hynes-Griffin, 1988). These include the following:

- Liquefaction of a gravelly-sand alluvial fan deposit in the 1948 Fukui earthquake in Japan (Ishihara, 1985).
- The flow slide at Valdez in an alluvial fan containing large zones of gravelly sand and sandy gravel in the March 27, 1964 Alaska earthquake (Coulter and Migliaccio, 1966). A review of a study of bridge foundation behavior during this earthquake was also performed (Ross et al., 1969).
- The slide in the upstream gravelly-sand shell of Shimen Dam in the 1975 Hanking earthquake in China (Wang, 1984).
- The slide in the upstream sandy-gravel slope protection layer of Baihe Dam during the 1974 Tangshan earthquake in China (Wang, 1984).
- The liquefaction of gravelly soils in level ground at the Pence Ranch, and in sloping ground at Whiskey Springs, both during the 1983 Mt. Borah, Idaho earthquake (Youd et al., 1985; Andrus et al., 1987, 1989; Harder, 1988, Stokoe et al, 1989).

### Case 1 - Ishihara (1985)

A review of this extensive report by Prof. Ishihara of the University of Tokyo finds only a very brief statement regarding the reported liquefaction of gravelly soils during the 1948 Fukui earthquake:

> *"The hazards associated with soil liquefaction during earthquakes has been known to be encountered in deposits consisting of fine to medium sands and sands containing low-plasticity fines. Occasionally, however, cases are reported where liquefaction apparently occurred in gravelly soils. For instance, at the time of the Fukui earthquake of June 28, 1948 in Japan, signs of disastrous liquefaction were reportedly observed in a gravelly sand in an area of fan deposit near the epicenter of the earthquake."*

No further documentation is provided regarding the observed liquefaction by which one can judge the validity of the observed liquefaction. Without detailed documentation to substantiate the reported liquefaction, little validity should be given to this case history or any other similar reported observations.

Case 2 - Coulter and Migliaccio (1966); Ross et al. (1969)

A review of the report entitled "Effects of the Earthquake of March 27, 1964 at Valdez, Alaska" by the authors finds that very few instances, if any, of liquefaction of gravelly soils are mentioned. Several excerpts from the reports are presented herein:

> *"A site for relocating the town of Valdez has been designated. It is situated on the Mineral Creek fan - an area underlain by coarse alluvial gravel. The absence of evidence of ground breakage on the Mineral Creek fan indicates that the coarse subsoils at the relocation site react favorably under seismic conditions. A comprehensive subsurface investigation was made of the central part of the Mineral Creek fan. The subsurface data show that the Mineral Creek alluvial fan is underlain by more than 100 feet of medium dense to very dense cobble gravel in a matrix of medium to coarse sand."*

The subsurface conditions in the vicinity of Valdez and in the Port of Valdez are described in the report as follows:

> *"Subsurface investigations by the Alaska Department of Highway indicates that the delta is composed of a thick section of poorly consolidated silt, fine sand, and gravel. The silt and fine sand occur as beds and stringers within the section and also are widely disseminated throughout the coarser fractions."*

> *"Insofar as it is known, three exploratory borings put down by the Department of Highways are the only deep borings ever made on the Valdez waterfront or, for that matter, along the entire eastern shore of Port Valdez. These three borings, which reached depths of 105, 82 and 132 feet, show a remarkable degree of horizontal continuity and vertical uniformity. There are two distinct layers. The first, or surface layer, is a loose to medium-dense sandy gravel with cobbles and silt. This layer is 20 to 30 feet thick. Conversations with local residents and examination of old photographs indicate that this material is fill moved in during development of the harbor facilities. The gravel layer is underlain by an undetermined thickness of loose to medium-dense gravelly sand containing thin lenses of silt. This zone persists to the maximum depths drilled."*

A description of the mechanism of the submarine slide which occurred within the Port of Valdez is described as follows:

> *"During the earthquake several factors apparently combined to produce the slide. First, the actual shocks or ground vibrations were of the critical intensity and duration required to create a condition of spontaneous liquefaction in the loose to medium-dense saturated sand. During liquefaction an increase in pore-water pressure, in effect, transformed a normal sediment into a concentrated suspension with a minimal shear strength. Second, the water withdrew from the beaches almost simultaneously with the initial shocks, according to eyewitnesses. The withdrawal may have been accompanied by a sudden decrease in hydrostatic head."*

Descriptions of ground breakage, including development of fissures and sand boils during the earthquake, include the following:

> *"During the earthquake and while the slide was taking place along the waterfront, an extensive system of fissures developed across the Valdez delta. Some of these fissures reportedly were opening and closing during the tremors, and considerable volume of water and suspended silt and sand were pumped from many of them."*

> *"The largest individual longitudinal-fissure segment observed was located 1500 feet east of Dike Road and 800 feet north of Richardson Highway. A large volume of fine sand and silt was ejected from this fissure."*

> *"Large volumes of ejected silt, sand and in some places pebbles characterize the central part of the longitudinal complex."*

This last excerpt is the only place in the entire report where reference is made to the fact that granular materials may have actually liquefied during the 1964 Alaska earthquake.

Based on a careful review of the above-referenced report, we cannot find any direct statement to support statements made by various investigators that earthquake-induced liquefaction of gravelly soils occurred during the 1964 Magnitude 8.4 Alaska earthquake. However, indirect evidence does exist to suggest that liquefaction of the loose to medium-dense sandy gravel surficial deposits (probably loosely dumped fill material) within the Port of Valdez probably occurred during the earthquake as a result of various related phenomena.

The massive submarine slide which occurred within the Port of Valdez was more than likely due to the following phenomena: strong ground-shaking, wave-induced liquefaction, and rapid drawdown conditions. The combined effects of these phenomena were most likely responsible for the slide which occurred.

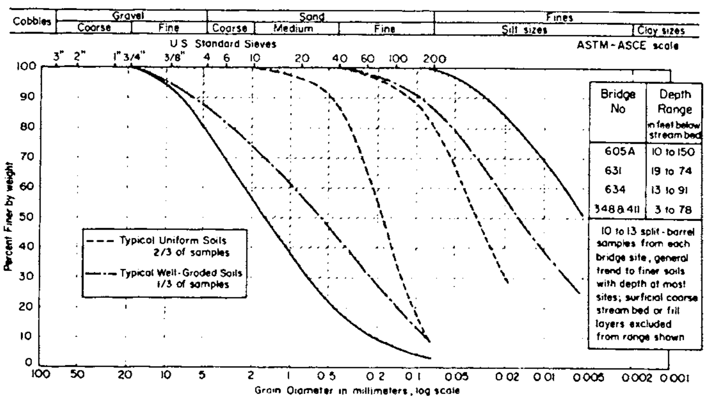
The information presented in the report also clearly shows that the medium-dense to dense gravelly deposits within the Mineral Creek fan area behaved quite well during the earthquake, and did not undergo any liquefaction or lateral spreading.

Extensive damage to highway bridges was caused by the March 27, 1964 Alaska earthquake. A paper by Ross et al. (1969), which documented the various observed bridge damages that occurred along several highways in Alaska, was reviewed in an attempt to evaluate the effects of foundation-support conditions on bridge behavior. The following summary is taken from the above-referenced report:

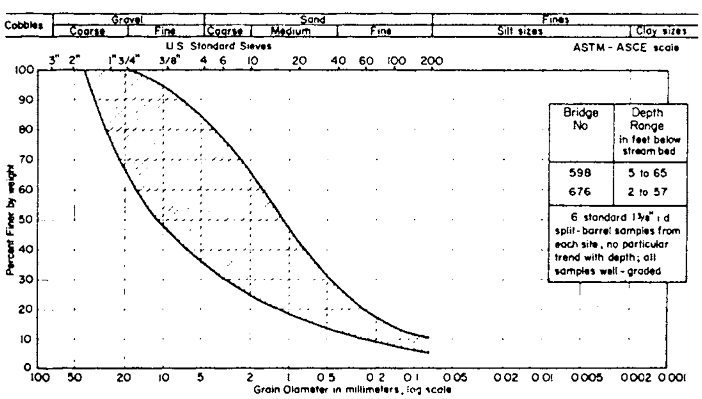
> 1. *No case of evident foundation displacement was reported for bridges known to be founded wholly on bedrock (total of 9 bridges).*

2. *Moderate to severe foundation displacement was reported for two bridges founded on distinctly differing support conditions, with bedrock at one end and pilings founded in cohesionless soils on the other.*

3. *The greatest concentration of bridges that sustained severe foundation movements were founded on pilings driven through saturated sands and silts of low-to-medium relative density (measured blow counts less than 20). The range of grain-size distribution for soils in this category is shown at the top of* Figure 1. *The samples from which this range was derived were obtained from bridge sites at which severe foundation displacements occurred during the earthquake.*




Typical grain-size distribution range for sands and silts at bridge sites where severe foundation displacements occurred




Typical grain-size distribution range for gravels and gravelly sands

Fig. 1 Grain size Distributions of Foundation Soils at Alaskan Bridge Sites (After Ross et al., 1969)

4. *Bridges founded on piles that were driven through loose to medium-dense sands and silts into denser sands and silts seemed to fare no better than those founded on piles that were embedded in loose to medium-dense sand and silt without reaching denser strata.*

5. *Bridge foundations that were founded in gravels and gravelly sands (regardless of N-values) rather than sands and silts behaved relatively well, with a generally even distribution between "nil" and "moderate" foundation displacement, and one or two cases of moderate-to-severe displacement. The grain size distribution range for samples obtained at the bridge sites where the subsoil consisted of gravels and gravelly sands, and where foundation displacements were minor to moderate, is shown at the bottom of* Figure 1. *These samples were generally well-graded, with uniformity coefficients, $C_u$, of about 15 to 50. As shown on the figure, they contained little fine sand and silt sizes. It should be noted that some subsoils classified as gravels may have included strata or lenses of sands or silts, a condition that would probably have influenced the observed performance."*

Cases 3 and 4 - Wang (1984)

Shortly after the occurrence of the Magnitude 7.3 Hanking earthquake of February 4, 1975 in China, a slide occurred within the saturated upstream shell of the Shimen Dam. Earthquake intensities at the dam, which was located a distance of approximately 33 km from the earthquake epicenter, were estimated to be on the order of VII (MM). The upstream shell of the dam consisted of a loosely placed well-graded sand-gravel mixture (Figure 2). The slide was very shallow having a maximum depth of about 15 feet (infinite slope failure-type) and occurred some 80 minutes after the earthquake. Failure has been attributed

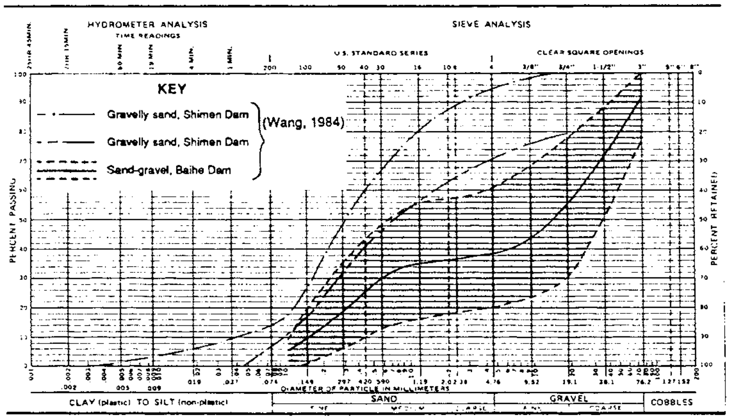



Fig. 2 Gradation of Soils Which Liquified During Past Earthquakes in China

to development of high excess pore-water pressures or liquefaction of the surface zone of the relatively loose and not-very-pervious sand-gravel shell.

During the July 28, 1976 Magnitude 7.8 Tangshan earthquake, a flow slide occurred within the saturated upstream protection zone of Baihe Dam. Peak ground horizontal accelerations recorded at the dam site near the dam crest were on the order of 0.15 g. Accelerations of about 0.05 g were recorded near the downstream toe. The upstream protection zone of the dam overlays the sloping clay core of the dam and consists of a gap-graded sand-gravel mixture containing up to 50% fines (and averaging about 32%; see Figure 2). This slide was also of the infinite slope failure-type and extended over the entire length of the dam. Lateral spreading occurred within the reservoir as far as 100 meters from the upstream toe of the dam.

No information is provided on the method or degree of compaction of the sand-gravel protection zone. However, a relative density of 56%, dry density of 103 pcf, and a permeability of $10^{-3}$ cm/sec have been assigned to the fine-grained portion of the material (d < 5 mm). These data indicate that the materials are generally loose and not very free-draining. The author states that segregation between gravel and sand particles was observed in test pits dug in the upstream protection zone and that the fine-grained portion of the mixture actually governs the stability of the slope. Based on these data, it appears that the fine-grained portion of the protection zone was mainly responsible for the high excess pore-water pressures and/or liquefaction which occurred leading to slope failure of the dam and lateral spreading of the upstream slope of the dam.

Cases 5 and 6 - Youd et al. (1985); Andrus et al. (1987, 1989); Harder (1988); Stokoe et al. (1989)

The liquefaction of gravelly soils in level ground at the Pence Ranch, and in sloping ground at Whiskey Springs, both in Idaho during the 1983 Magnitude 7.3 ($M_s$) Borah Peak earthquake, represent the best documented case histories studied to date. Extensive field investigations and analyses have been carried out by various investigators at both sites in order to present a well-documented account of all the factors pertinent to each case.

On October 29, 1983, central Idaho was shaken by the Borah Peak earthquake ($M_s$ = 7.3). Earthquake effects included a 38-km-long (28-mile) surface rupture, landslides, disruptions of ground water, and several liquefaction effects. This included field observation of several gravelly-sand boil deposits in the zone of fissuring located at the Pence Ranch, and lateral spreading, fissuring, and sand boils in a coarse-grained sediment in the Whiskey Springs area. Approximate epicentral distances and estimates of peak ground accelerations at various sites are tabulated below (after Harder, 1988). Values shown in parentheses are estimates of peak ground acceleration by others.

| Soil Site | Approximate Epicentral Distance (Miles) | Estimated Peak Ground Acceleration (g) |
|---|---|---|
| Town of MacKay | 14 | 0.19 |
| Mackay Dam | 10 | 0.22 |
| Pence Ranch | 5 | 0.29 (0.30-0.35)* |
| Whiskey Springs Slide | 1 | 0.40 (0.50-0.70)* |

*Estimates by Andrus & Youd (1989).

It is important to note that field reconnaissance conducted by Youd (1985) along a 5-mile-long segment of the Big Lost River, which lies between 2 and 4 miles from the southern terminus of the fault rupture, did not produce any evidence of significant liquefaction effects. They state that "the probable reason for the absence of effects in this area is that the sediments may be too coarse and well drained to liquefy or to create surficial liquefaction effects."

Field investigations were conducted at both the Pence Ranch and the Whiskey Springs lateral spread sites during July 1985. These included Cone Penetration Tests (CPT), Standard Penetration Tests (SPT), Becker Hammer Tests, measurements of in-situ shear wave velocities using the Spectral-Analysis-of-Surface-Waves (SASW) Method, and sampling of the materials using various sampling devices.

1. Pence Ranch Site

The Pence Ranch site is located on a low terrace along the Big Lost River approximately 5 miles south of the southern edge of the fault rupture. Several gravelly sand boil deposits were observed in the zone of fissuring and lateral spreading after the earthquake. Samples of these deposits were found to be generally clean sand with gravel content ranging from 5% to 25% present, with pebbles as big as 1 inch across. Grain-size distribution curves for samples taken from various sand boil deposits are shown in Figure 3. The subsurface soils underlying the Pence Ranch site range from a clean gravelly sand (SP-GP) to a clean sandy gravel (GP) with fines content generally less than 5% (Figure 4). A zone (Unit C) located within a depth of about 5 to 13 feet had measured blow counts (SPT "N" values) which ranged from 1 to 16. Corrected SPT "N" values [$(N_1)_{60}$] were less than 8. Below this depth, the blow counts were considerably higher (greater than 20). Values of equivalent SPT blow counts $(N_1)_{60}$ were also estimated by Harder (1988) from blow counts obtained using the Becker Hammer. He also concluded that, within Unit C, the equivalent corrected SPT blow count is approximately 8. This increases to 18 in the depth interval between 13 and 20 feet.

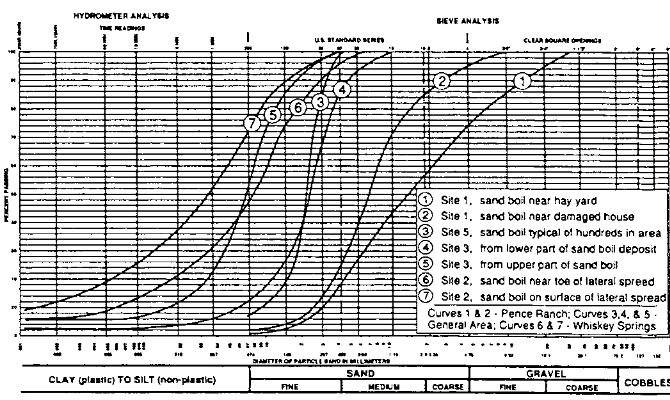
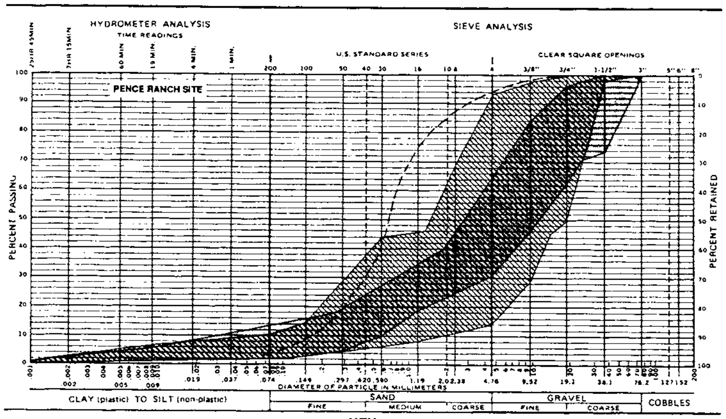



Fig. 3 Gradation of Soils Which Liquefied During 1983 Borah Peak Earthquake (Youd et al., 1985)

Evaluations of the liquefaction potential of Unit C were carried out by Stokoe et al. (1989) using the corrected SPT blow count data, and by Harder (1988) using the equivalent corrected SPT blow counts estimated from the Becker Hammer blow counts. Both studies were able to predict liquefaction of the very loose gravels, as was observed during the 1983 earthquake, as indicated on the plot presented on Figure 5. It should be noted that the simplified liquefaction analyses conducted by Andrus and Youd (1987) and by Stokoe et al. (1989) were carried out using the procedures developed by Seed et al. (1984) for clean sands without any modifications whatsoever for the gravelly nature of the deposits.



KEY

SAMPLES FROM BECKER SOUNDINGS, DEPTHS = 0 - 39 FT.
SPT SAMPLES OBTAINED BETWEEN DEPTHS = 0 - 14.5 FT. (SUSPECT LIQUEFIED SOIL LAYER) } (Harder, 1988)

SAMPLE OF SAND BOIL OBTAINED NEAR RANCH HOUSE (AFTER YOUD ET AL., 1985)

Fig. 4 Gradation of Soils Which May Have Liquefied During 1983 Borah Peak Earthquake, Pence Ranch Site

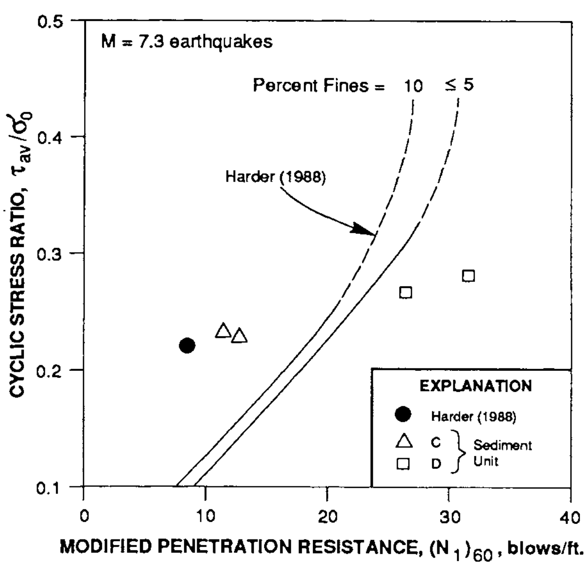


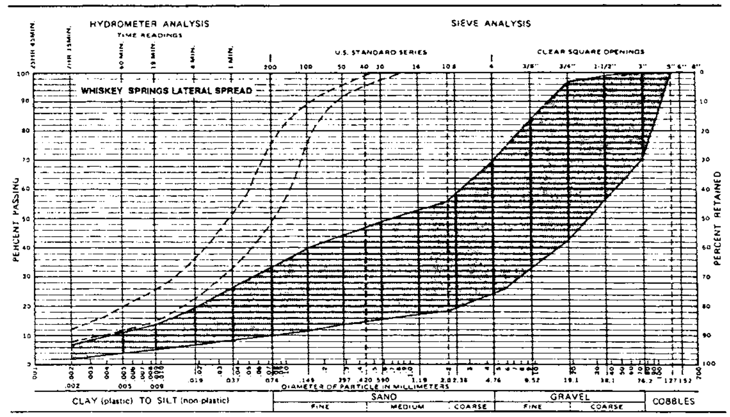

Fig. 5 Liquefaction Potential Chart, Pence Ranch Site (Modified from Andrus and Youd, 1989)

2. Whiskey Springs Slide Site

Field investigations and studies similar to those carried out at the Pence Ranch were also conducted within the Whiskey Springs Slide area by various investigators. The Whiskey Springs slide is a lateral spread which occurred close to the epicenter of the 1983 Borah Peak earthquake. Estimates of peak ground accelerations at this site range between 0.40 g and 0.70 g (see previous table). Because of the rather large lateral movements (roughly 4 feet) which occurred on a relatively flat slope (approximately 5°) at this site, it was concluded that liquefaction was probably associated with the observed behavior. This was supported by the fact that several sandy silt boils were found near the fissures and sod buckles.

A dense to very dense silty sandy gravel caps the Whiskey Springs site (Units A and B). Looser and finer-grained gravelly sediments (GM), Units C1 and C3, lie beneath the dense gravel. SPT's performed within a hollow-stem auger casing gave measured N-values for Unit C1 ranging from 5 to 14 blows per foot. Blow-count measurements were made for each inch of penetration. Plots of these data were essentially uniform in any given test and appear to indicate that the measured N-values were not affected by the presence of gravels.

Andrus and Youd (1987) conclude that the lateral spread was caused by liquefaction and shear deformation within Unit C1. This unit ranges in depth from a minimum of 8 feet near the toe of the slide zone to a maximum of 55 feet near the head scarp of the lateral spread zone. Gradation characteristics of soil samples from the suspect liquefied soil layer are presented in Figure 6.



KEY

14 SAMPLES OF CRITICAL LAYER (ELEV. 43-58 FT.) OBTAINED USING EITHER 5" I.D. AUGER SAMPLER OR 4.3" I.D. BECKER SAMPLING BIT (AFTER HARDER, 1988)

SAMPLES OF 2 SILT BOILS OBTAINED NEAR BUCKLED SOD (AFTER YOUD ET AL., 1985)

Fig. 6 Gradation of Soils from Suspect Soil Layer Liquefied during 1983 Borah Peak Earthquake, Whiskey Springs Site

Harder (1988) performed Becker Hammer Penetration Tests at the various sites investigated by Andrus and Youd (1987) within the Whiskey Springs Slide area. Based on these tests, he established equivalent corrected SPT blow counts for the measured Becker Hammer blow counts. The average corrected equivalent SPT blow counts $(N_1)_{60}$ for Unit C1 was approximately 8. It was Harder's contention that the results obtained from the SPT investigation gave penetration resistance values that were on the high side. However, he also concluded that the silty gravel layer (Unit C1) was the zone that liquefied and thus triggered the lateral spread. Based on the gradation of these materials shown in Figure 6, Harder concluded that the larger gravel particles are simply floating without direct contact with each other in a silty sand matrix having a fines content of 40% or more.

An evaluation of the liquefaction potential of the subsurface soils underlying the Whiskey Springs lateral spread was performed by Andrus and Youd (1987) using corrected SPT blow counts, simplified procedures developed for sandy soils, and a peak ground acceleration ranging from 0.50 g to 0.70 g. The results of these analyses are presented in Figure 7. Harder (1988) performed similar analyses using an equivalent corrected SPT blow count of 8, a peak ground acceleration of 0.40 g, and a somewhat more elaborate but similar liquefaction analysis. His results are also shown of Figure 7. It can be seen that in both cases liquefaction of the suspect silty gravel layer (Unit C1) is predicted.

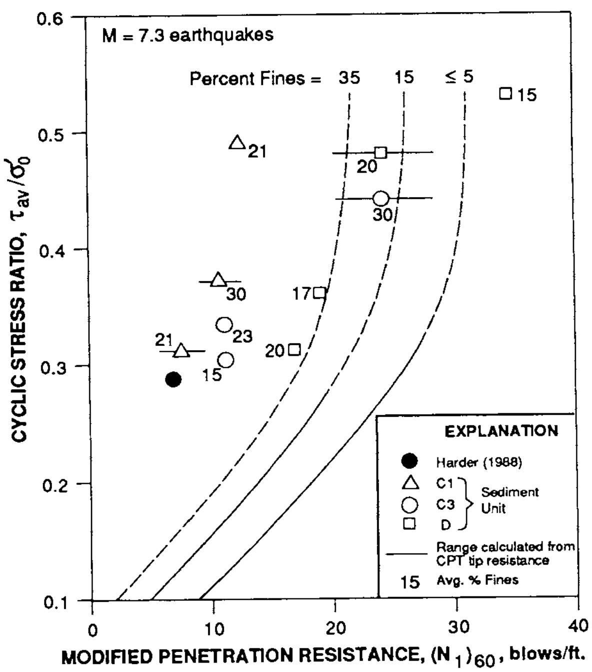



Fig. 7 Liquefaction Potential Chart, Whiskey Springs Site (Modified from Andrus and Youd, 1989)

Summary of Case Histories

Gradation of soils which liquefied during the 1983 Borah Peak, Idaho earthquake are presented on Figures 3, 4, and 6 (Youd et al., 1985; Harder, 1988). Gravelly soils which were part of the upstream embankment of two dams in China, and which liquefied during past earthquakes, are shown on Figure 2. Based on these plots, it can be seen that the gradational characteristics of soils which may be subject to liquefaction can range from clean gravels to silts.

It should be noted, however, that, in all cases of suspected liquefaction of gravelly soils, one or more of the following conditions was observed:

- All gravel deposits were loose to very loose. In the best-documented case where SPT or equivalent SPT blow counts were measured in the gravelly deposits which liquefied, they ranged from 1 to 15 blows per foot.

  In the clean gravel deposits, the quantity of sand filling the voids was insufficient to fill the voids between contacting gravel particles, allowing the larger particles to contact each other in a "clasp-supported structure."

  In the silty/sandy gravel deposits the larger gravel particles were simply floating without direct contact with each other in a silty sand matrix. Thus, liquefaction behavior was controlled by the fine matrix and not by the gravel particles.

RUBBER DAM SITE CONDITIONS

The study site lies near the head of an alluvial fan, where deposition took place in a primarily high energy regime. The sequence of coarse-grained materials is thought to be about 300 feet thick and is in basal contact with bedrock. The alluvial fan is an important ground water source in the area. The stratigraphy at the site consists of stratified, lenticular, normally consolidated deposits ranging in grain size from sand size to mostly gravel size, commonly with less than 10% low-plasticity fines Standard Penetration Test blow counts ranged between 20 and 50 indicating medium-dense to dense materials, although the occurrence of gravel makes these tests less reliable. The depth to the ground-water table is variable, and can be expected to fluctuate between about 0 and 20 feet below ground surface depending upon the season and whether or not water is impounded.

The seismic setting of the area is dominated by the active Hayward fault which is located about 1/2 kilometer downstream of the study area. Because of its proximity and known activity, the Hayward fault is regarded as the controlling seismic source at this site, and the assigned Maximum Credible Earthquake (MCE) magnitude of 7-1/2 was used for our analyses with a corresponding horizontal ground surface acceleration of 0.53 g.

FIELD AND LABORATORY INVESTIGATION

The original field investigation consisted of five hollow-stem auger borings, ranging in depth from 13 to 26.5 feet, and a test pit that was about 12 feet in depth. SPT's were conducted at 5-foot intervals inside the hollow-stem auger casing using a CME automatic hammer system (Riggs et al., 1983, 1984).

Subsequent field investigations, conducted after liquefaction of the gravels became a concern, utilized rotary wash borings, as deep as 65.3 feet. These borings were drilled downstream of the rubber dam site, along both levees, to investigate the liquefaction potential of the materials comprising and underlying the levees. Blow counts taken during the SPT's were recorded for every 1/10 of one foot to provide better data for evaluating the effect of gravel content on the measured blow counts. Figure 8 illustrates typical field results when blow counts are recorded every 0.1 foot. As can be seen from the figure when blow counts are plotted cumulatively (as was done for Andrus and Youd; 1987), it is not always obvious to detect the presence of gravels or to establish the effect of gravel content on the measured blow counts.

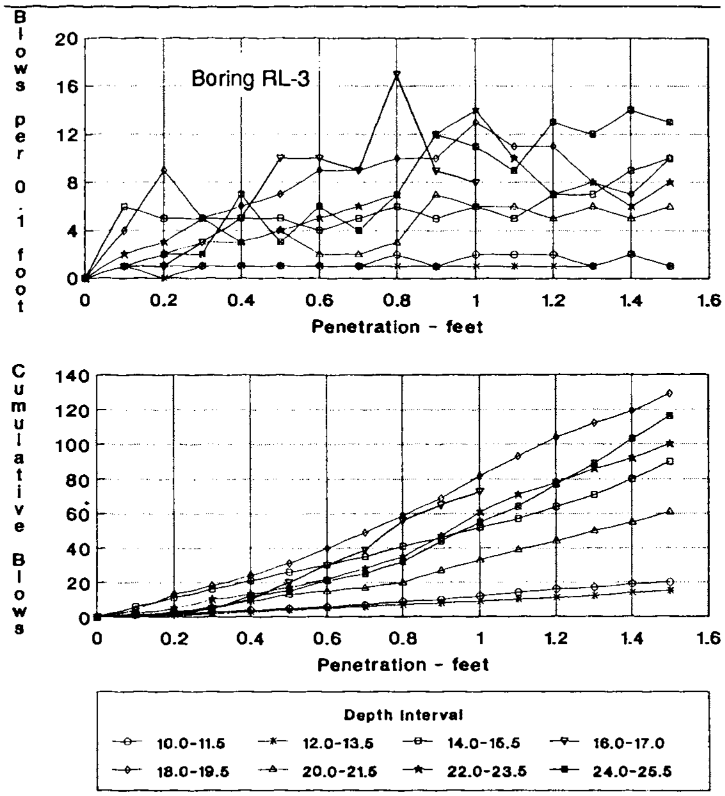



Fig. 8 Field Measured Blow Counts

Additional field investigations were performed during the construction of the cutoff wall foundation for the rubber dam. Geologic mapping of the cutoff wall open-cut excavation was conducted and bag samples of representative materials were taken at various locations and depths for sieve analyses and relative density tests. In addition, sand cone and percolation tests were performed in order to establish the density and drainage characteristics of the in-situ soils.

Sand-cone in-situ density tests were performed in accordance with ASTM D1556 using a 12-inch-diameter U.S. Bureau of Reclamation sand cone. For comparison with the in-situ density of the on-site soils, two relative density tests were performed on representative samples according to ASTM D4253 and D4254. The computed relative densities, which gave the most reasonable results and best simulated field conditions, were based on the wet method (ASTM D4254). The relative densities ranged from 60% to 100%, indicating medium-dense to dense materials in situ.

Percolation tests were performed in the cutoff wall excavation in excavated pits that were about 2 to 3 feet in both diameter and depth. Measurements were taken of the dimensions of the percolation pit, the rate at which water was added to the pit, and the rate at which water was infiltrating the surrounding ground. Whenever possible, the objective was to reach a steady state of infiltration. The tests indicated free-draining material and qualitatively high permeability values. Steady-state infiltration in the three holes ranged from about 12 to 15 gallons per minute, except for one location for which the computed infiltration rate was only about 1-1/4 gpm.

Laboratory sieve analyses were performed on SPT samples, in accordance with ASTM D422-63, often with the hydrometer test omitted. Based on the results of the sieve analyses, the percentage of cobbles, gravels, sands, and fines, the $D_{50}$ size, and the coefficient of uniformity were calculated for each sample. The sand and gravel envelopes for the sieve analyses are shown in Figures 9 and 10, and are discussed in more detail below.

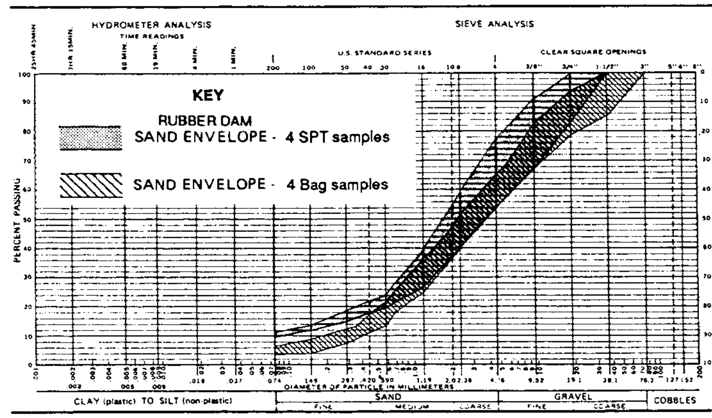



Fig. 9 Sand Envelopes

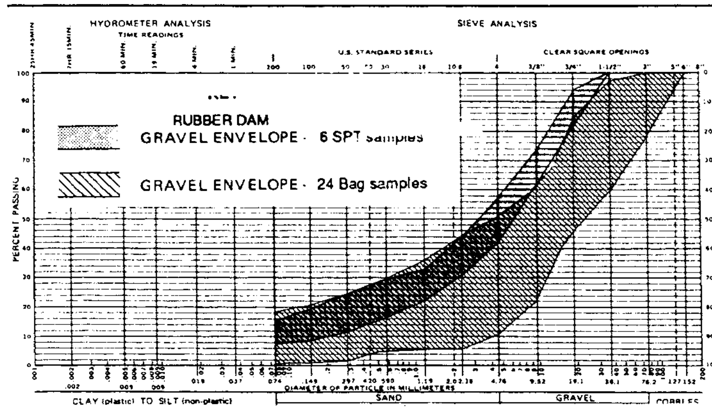



Fig. 10 Gravel Envelopes

## LIQUEFACTION POTENTIAL EVALUATION, RUBBER DAM SITE

The liquefaction potential of the sandy and gravelly soils underlying the rubber dam site was evaluated using various procedures which have been developed for evaluating the liquefaction potential of saturated soil deposits consisting of clean sands, silty sands, or silts (non-plastic) under earthquake loading conditions.

Gradation Characteristics of On-Site Soils

A summary of the gradation characteristics at the rubber dam site is shown on Figure 9 for the sandy soils, and on Figure 10 for the gravelly soils. A comparison of the sand and gravel envelopes for soils at the site with the case history envelopes for liquefiable soils indicates that, on the basis of gradation alone, it is possible that the gravelly soils at the site could liquefy or develop high excess pore pressures during strong ground shaking at the site. As previously mentioned, however, review of the case histories of suspected liquefaction of gravelly soils indicated that, in all cases, the materials were very loose to loose, whereas the gravelly soils present at the study site are generally medium dense to dense and free-draining.

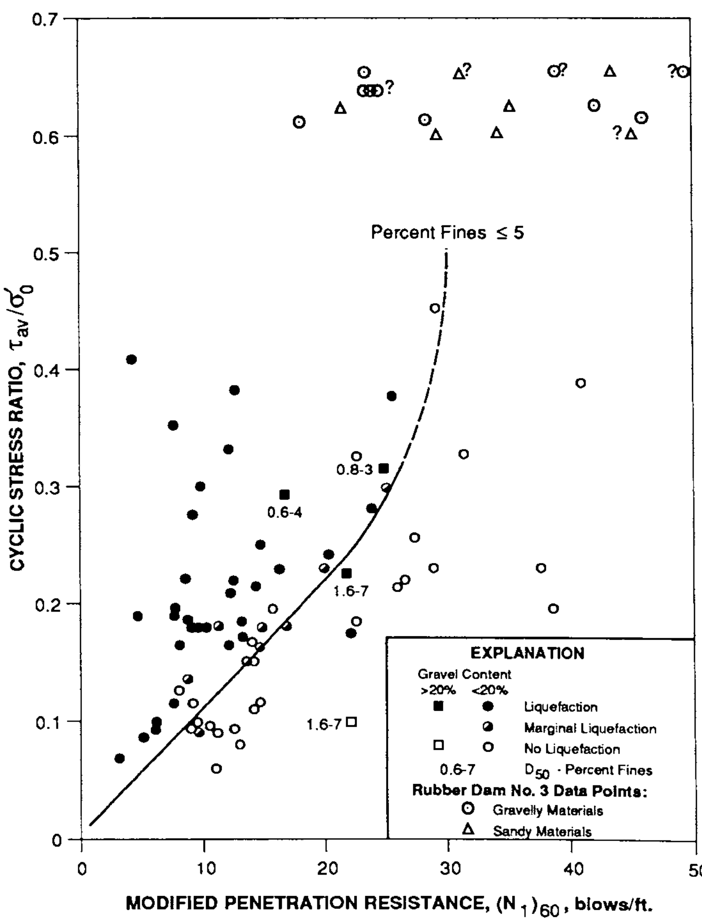
The liquefaction potential of a soil is dependent on many factors other than gradation. Among these are peak ground acceleration and duration of strong shaking, relative density of the soil, and permeability or drainage characteristics of the soil deposit. As discussed previously, in-situ density and relative density tests performed on representative samples of the in-situ gravelly soils indicate medium-dense to dense gravels. Blow-count data likewise indicate medium-dense to dense sands and gravels.

Although complete liquefaction of a gravelly soil deposit may not occur, it is possible that high excess pore-water pressures could develop as a result of earthquake ground shaking. This could occur if the gravelly deposit is capped by less pervious soil layers and/or contains significant quantities of fines, which considerably reduce its overall permeability characteristics. These two factors appeared to have contributed to the liquefaction which occurred at the Whiskey Springs lateral spread site during the 1983 Borah Peak earthquake. As indicated previously, results of the percolation tests indicate that, in general, the soils are free-draining. Permeability estimates based on the $D_{10}$ size (Cedergren, 1967), though not strictly applicable, indicate a range of permeability from 1 cm/sec to $10^{-4}$ cm/sec, with an average of about 0.3 cm/sec. It should be noted, however, that the average fines content (passing the No. 200 sieve) for the bag and bulk samples is on the order of 6.5%. Where liquefaction is most critical for the rubber dam foundation is in the upper 12 feet, the depth of the cutoff wall. Since the gravelly soils at these depths are relatively shallow, the surface acts as a free-drainage boundary. In addition, excess pore water pressure will also be dissipated laterally within the deposits. It should also be noted that Alameda County Water District (Owner of the project) has extensive data on the ground-water recharge rates in the area, as well as data on their well field pumping operations, thus providing further support of the free-draining characteristics of the gravel deposits underlying the dam site.

Liquefaction Analysis

An evaluation of the liquefaction potential of the sandy and gravelly soils at the site was performed using blow-count data obtained from Standard Penetration Tests (SPT's) performed at the site during ESA's initial investigation and the "Simplified Seed Method" for a horizontal soil deposit (Seed et al., 1967). This method was originally developed for evaluating the liquefaction potential of saturated clean sand and silty deposits, but has recently been used for evaluating the liquefaction potential of loose sandy gravels and gravelly sands (Andrus and Youd, 1987; Stokoe et al., 1989).

On Figure 11, a plot of SPT-corrected blow count $(N_1)_{60}$ versus cyclic stress ratio $(\tau_{avg}/\sigma_o')$ required to cause liquefaction is presented for a Magnitude 7.5 earthquake (Seed et al, 1983, 1984). The data points shown on this figure represent a comprehensive collection of site conditions where evidence of liquefaction or no liquefac-




Liquefaction susceptibility chart with data prepared by Seed et al. (1984) for clean sands and a 7.5 magnitude earthquake. Data points having more than 20% gravel are denoted by squares. The mean grain-size and fines content for each of these gravelly sites are also shown.

Fig. 11 Liquefaction Analysis

tion is known to have occurred during past earthquakes. Relationships of this type have been developed for different magnitude earthquakes and for different fines contents. It should be noted that the majority of liquefaction case histories shown on this figure have occurred in clean sand and silty sand deposits at shallow depths on the order of 30 feet or less.

In order to evaluate the liquefaction potential at the site using the "Simplified Seed Method," it is first necessary to distinguish between SPT blow-count data obtained in the sandy materials and those measured in the gravelly soils. Secondly, it is important to correct the applicable blow-count data for various effects such as overburden effects and hammer energy ratio. It is also helpful to statistically evaluate the blow-count data on the basis of material type as an indication of whether or not the materials sampled are representative of actual site conditions.

Results of the field investigation and laboratory sieve analyses were used to help classify the soil samples and distinguish the most reliable blow-count data in the sandy materials. A comparison of laboratory sieve analyses from SPT split-spoon samples and bag samples was made. The gradation envelopes presented in Figures 9 and 10 indicate that there is a sample bias when using the SPT (split-spoon) sampler. The split spoon cannot sample material greater than about 1-3/8 inches, the internal diameter of the sampler. Hence, the SPT samples are finer when compared to the bag samples. As would be expected, the shift is much more pronounced for the more gravelly materials.

In general, the SPT blow count data indicated medium-dense to dense materials according to relationships given by Peck, Hanson, and Thornburn (1953) and Terzaghi and Peck (1948). However, the reliability of the data for the gravelly soils is questionable. SPT blow-count data were corrected for overburden effects, $(N_1)_{60}$, according to relationships provided by Seed et al. (1984) based on data and analyses from Marcuson and Bieganousky (1977), and was corrected for drill rod stiffness by a factor of 0.75 for length of drill rod less than 10 feet (Seed et al., 1985). It should be noted that the points shown on Figure 11 have not been corrected for the effects of hammer efficiency ($N_c$ = 1.30) or for using the SPT sampler without liners ($N_c$ = 1.2) (Riggs et al., 1983; Seed et al., 1985). Nevertheless, the corrected $(N_1)_{60}$ blow counts neglecting these factors indicated medium-dense to dense materials (relative densities ranging from 60% to 95%) based on the relationships developed for sand and presented in Tokimatsu et al. (1987).

A total of 18 blow-count data points are plotted on Figure 11. An additional four data points which met refusal or got hung up in gravels were not plotted since they were considered unreliable. Thus, the total data set consists of 22 points. As indicated by the queried points on Figure 11, five of the data points are considered questionable upon close evaluation of the blow counts and recovery. These five data points, together with the four data points which met refusal, means that nine of the 22 data points (41%) are considered unreliable (or, conversely, that 59% are reliable). Seven of the remaining 13 data points are in gravels (54%), four are in sands (31%), and two are in borderline sandy gravels/gravelly sands (15%). The distribution of the data points among the various material types appears to indicate that the SPT blow-count data are probably representative of the actual field conditions.

The liquefaction potential of the soils was evaluated using the "Simplified Seed Method" previously described, and the relationship shown on Figure 11. Points falling to the left of the curve indicate liquefaction, while points to the right indicate no liquefaction. Corrected SPT blow-count data in the site materials were obtained after interpretation of the sieve analyses and blow-count data. Although SPT data in gravel are often misleading and can lead to erroneously high blow-count values, given the consistency of the measured blow count data, the recovery, and the field geologist observations, the data points corresponding to the gravelly materials present at the site were also plotted. This was actually done by Andrus and Youd (1987) and Stokoe et al. (1989), who used SPT values in gravelly soils to evaluate liquefaction potential as a result of the Magnitude 7.3 Borah Peak earthquake in Idaho in which loose gravelly soil deposits actually liquefied.

The cyclic stress ratios ($\tau_{avg}/\sigma_o'$) induced by the postulated earthquake ground motions were computed using a mean peak horizontal ground surface acceleration ($a_{max}$) of 0.53 g for the Magnitude 7.5 controlling earthquake on the nearby Hayward fault. A conservative average unit weight of 130 pcf was assumed based on in-situ field density tests, and the ground water table was assumed to be at the ground surface. For the rubber dam site, the induced cyclic stress ratio varies between about 0.66 at the surface and 0.59 at a depth of 30 feet.

Of the 18 data points (including the five questionable data points) in sands and gravels, corrected for overburden effects and drill rod stiffness, six points (five in gravel, one in sand) indicate liquefaction, three points (one in gravel, two in sand) indicate marginal liquefaction, and nine points (four in gravel, five in sand) indicate no liquefaction, as shown in Figure 11. Carrying the analyses a step further by correcting for hammer efficiency, all of the data points would indicate no liquefaction.

## CONCLUSIONS

Based on the above studies, we concluded that the gravelly soil deposits underlying the foundation of the rubber dam are medium dense to dense and would not undergo liquefaction during the postulated Magnitude 7.5 earthquake on the nearby Hayward fault. Nevertheless, we believe that it is prudent to at least consider the possibility that some localized liquefaction or development of high excess pore pressures could occur within certain zones of the foundation, and to consider what consequences these occurrences would have on the performance of the rubber dam foundation and structure, and associated equipment.

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