Liquefaction and Embankment Failure Case Histories, 1988 Armenia Earthquake

By M. K. Yegian,[1] V. G. Ghahraman,[2] Members, ASCE, and R. N. Harutiunyan[3]

Abstract: The 1988 $M_s = 6.8$ earthquake in Armenia resulted in more than 40,000 human casualties and massive destruction of the northwestern region of Armenia. The effects of local geology and soil conditions upon the earthquake-induced damage were analyzed and reported by the writers in other publications in the ASCE Geotechnical Journal. This paper presents data and analysis of liquefaction and liquefaction-induced embankment failure case histories, the significance of which stems from the fact that the liquefied sands had a high gravel content (up to 50%). There are only a few well-documented field observations of liquefaction of gravels and gravelly sands, and the information presented in this paper augments this limited database. Our analyses lead to the conclusion that loose to medium-dense gravelly soil deposits that are not confined against drainage can withstand large (0.5 g–1.0 g) peak ground accelerations without liquefying. However, a mere 30 cm thick impermeable topsoil can impede drainage, thus causing liquefaction of such soils leading to significant deformations and lateral spreading. The residual shear strength of the gravelly soils investigated (with field SPT of ≈36 blows/m (12 blows/ft)) was back-estimated to range between 5 to 13 kPa (100–260 psf), values comparable to the residual shear strength of loose clean sands. The observations and conclusions from liquefaction of gravelly soils in Armenia compare well with the well-documented cases from the 1983 Borah Peak, Idaho, earthquake.

INTRODUCTION

The earthquake of December 7, 1988 in Armenia caused extensive damage to buildings, roadways, and other engineered facilities. Soon after the earthquake, a U.S. reconnaissance team traveled to Armenia and prepared a preliminary earthquake-damage assessment report (Wyllie and Filson 1989). It was evident that the geologic, geotechnical, and structural aspects of the earthquake warranted further investigation.

Following the earthquake, the writers, supported by the National Science Foundation, embarked on a research effort aimed at evaluating the geotechnical aspects of the earthquake. The major findings and conclusions from this research are presented in Yegian et al. (1994a–d). This paper presents case histories of liquefaction and liquefaction-induced failure of two embankments, along with one case of no liquefaction from the Armenia earthquake. One of the important characteristics of these case histories is that the liquefied sands have a very high (40–50%) gravel content.

While there is an abundance of case histories involving liquefaction of sands, there have been a very few field observations of liquefaction of gravels and gravelly sands (Liao 1986). In recent years, it has been recognized that saturated gravelly soils may liquefy as readily as saturated sands under

[1]Prof. and Chmn., Dept. of Civ. Engrg., Northeastern Univ., Boston, MA 02115.
[2]Dept. of Civ. Engrg., Northeastern Univ., Boston, MA.
[3]Geotech. Lab. Dir., ARMNIISA, Research Inst. for the Ministry of Constr., Yerevan, Armenia.

Note. Discussion open until August 1, 1994. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on August 18, 1992. This paper is part of the ***Journal of Geotechnical Engineering***, Vol. 120, No. 3, March, 1994. ©ASCE, ISSN 0733-9410/0003-0581/$2.00 + $.25 per page. Paper No. 4636.

certain conditions. There are several noteworthy case histories, as discussed by Harder and Seed (1986) and Evans et al. (1992) in which liquefaction of gravels and gravelly soils have occurred.

The first well-documented case of observed liquefaction of gravelly soils has been provided by the 1983 Borah Peak, Idaho, earthquake ($M_s \approx 7.3$). Results and subsurface investigations, analyses, and correlations with penetration test data have been published by Youd et al. (1985), Andrus and Youd (1987, 1989), and Andrus et al. (1991). A conclusion of those studies was that the empirical liquefaction assessment method based on the standard penetration test $(N_1)_{60}$ values, developed for sandy soils (Seed et al. 1985), is also applicable to gravelly sands. Critical to the occurrence of liquefaction was the presence of a low permeability crust that did not allow the dissipation of the excess pore-water pressures as they were being generated during the earthquake. The observed liquefaction effects included numerous sand boils as well as liquefaction-induced lateral spreading.

In the past few years, laboratory research on liquefaction of gravelly soils has provided some understanding of their cyclic behavior (Evans and Seed 1987; Hynes 1988; Kokusho et al. 1991; Tanaka et al. 1991; Evans et al. 1992). A major problem encountered in experimental investigation of gravelly soils has been associated with the size of the particles involved. For example, large particles in gravelly soils preclude conventional sampling and testing of recovered samples in the laboratory. In addition, the large particles create severe membrane compliance problems that artificially reduce the laboratory liquefaction potential, invalidating the results unless a correction is employed (Evans et al. 1992). Therefore, case histories, including the ones presented herein, can play a vital role in further advancing our knowledge and understanding of liquefaction susceptibility of gravelly soils.

EARTHQUAKE

On December 7, 1988 a surface magnitude $M_s = 6.8$ earthquake struck northern Armenia resulting in a 27 km fault break shown in Fig. 1. The devastated region extended to about 30–40 km from the fault. Over 40,000 people lost their lives and more than 1,000 multistory buildings either collapsed or were damaged beyond repair. The ground motion was recorded with a strong-motion instrument in the town of Ghoukasian, Armenia, 25 km from the northwestern tip of the fault. The peak ground acceleration (PGA) was about 0.2 g for both north-south (N-S) and east-west (E-W) components (A. Der-Kiureghian, personal communication, 1989). The geotechnical profile at the recording station consists of 35–40 m of dense gravelly sands and stiff clays of lacustrine origin. Yegian et al. (1994b) describes the effect of the soil condition at this site upon the ground surface record. Rock outcrop motion was back-estimated from these recorded ground surface motions using soil amplification analysis. Fig. 2 shows the two (N-S and E-W) components of the computed rock motion in Ghoukasian. Yegian et al. (1994b) demonstrated that the spectral shape of these rock motions is representative of the rock motions in the near field (within 20–30 km from the fault). Of course, the PGA of these rock motions would depend on the distance of a particular site from the fault.

In the city of Spitak, Armenia (Fig. 1), 1–2 km from the fault, PGA was estimated from analysis of the response of grave markers to be between 0.5 and 1.0 g (Yegian et al. 1994b). This estimation is consistent with the nearly total destruction of buildings and houses observed throughout the near-fault region. At the locations of the case histories analyzed here, which are also

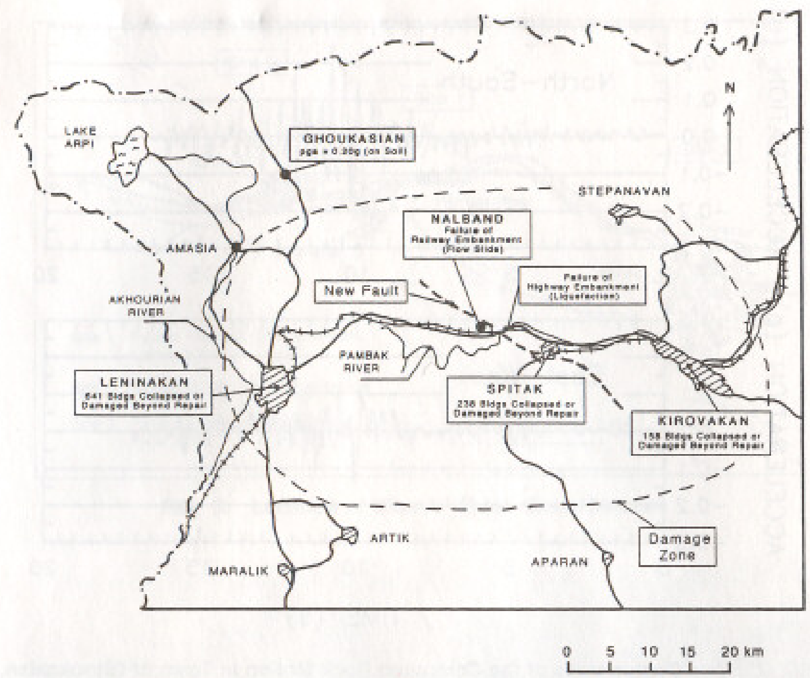


FIG. 1. Map of Western Armenia and Earthquake Damage Region with Points of Interest

1–2 km from the fault and near the city of Spitak, the PGA therefore is estimated to be also between 0.5 and 1.0 g.

CASE HISTORIES

Fig. 3 shows the location of the three sites that are investigated. At site 1, liquefaction of gravelly sands was observed in the toe region of a failed highway embankment. At site 2, gravelly sands, similar to those found at site 1, surprisingly showed no evidence of liquefaction. At site 3, a railway embankment, also founded on gravelly silty sands, failed in the form of a flow slide.

To document these geotechnical case histories from the Armenia earthquake, the writers transported standard- and cone-penetration-test equipment to Armenia. The standard-penetration-test (SPT) equipment included: a Pilcon trip monkey type hammer having a weight of 0.62 kN (140 lb); 1.52 m (5 ft) AW drill rods; and 5.1 cm (2 in.) diameter split spoon samplers without liners. Selected field tests were made to obtain estimates of the soil properties and to confirm profile geometries defined earlier by Armenian engineers. Recognizing difficulties associated with in situ testing of gravels and gravelly sands, only the field (uncorrected) values of the SPT were used in this investigation. These SPT values provide a measure of the densities of the gravelly sands encountered and allow comparison of the observations made from these case histories with those from other case histories on similar gravelly soils. In the present paper, the relevant geotechnical information

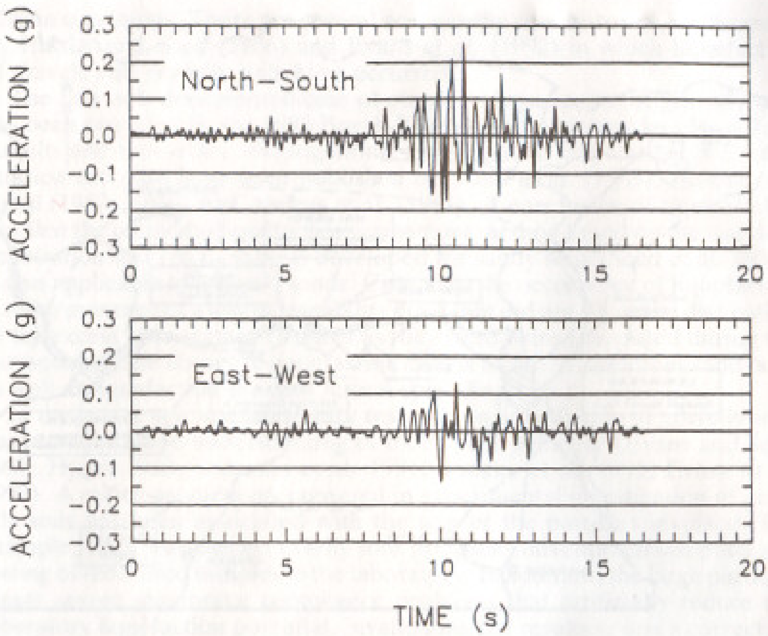


FIG. 2. Two Components of the Computed Rock Motion in Town of Ghoukasian, 25 km from the Northwestern Tip of Fault

is presented for each site and the results of our analyses of the field observations (failure or no-failure) follow.

Sites 1 and 2

At site 1, one km from the fault (Fig. 3), a highway embankment failed. Fig. 4 shows a photograph of the embankment taken after its failure. This embankment supported a highway that was the primary link between the three major Armenian cities of Leninakan (now called Kumayri), Spitak, and Kirovakan, located in the earthquake disaster region (Fig. 1). Its failure seriously impeded the relief efforts immediately after the earthquake.

The failed highway embankment was near a tributary of the Pambak River (Fig. 3), where the water table was near the ground surface. Fig. 5 depicts a geologic cross section of the Pambak Valley crossed by the highway (A. Vehouni, personal communication, 1990). The valley is filled primarily with 140 m deep alluvium (silty gravelly sands) with intermittent layers of tuff (volcanic rock). Immediately after the earthquake, the embankment was reconstructed so that the highway could be put back into service.

During our reconnaissance trips and surveys in the vicinity of the fault, northwest of Spitak, we observed sand boils, with silty sands spread on the ground surface and gravels lodged in the blow-out holes (Fig. 6). These sand boils were located as close as 15 m from the toe of the failed section of the highway embankment.

Fig. 7 shows the embankment cross section before and after the failure determined by the writers. The embankment material is silty, sandy compacted fill. The foundation soils in the toe of the embankment include a top 30–40 cm agricultural soil, described as low-plasticity sandy silts with

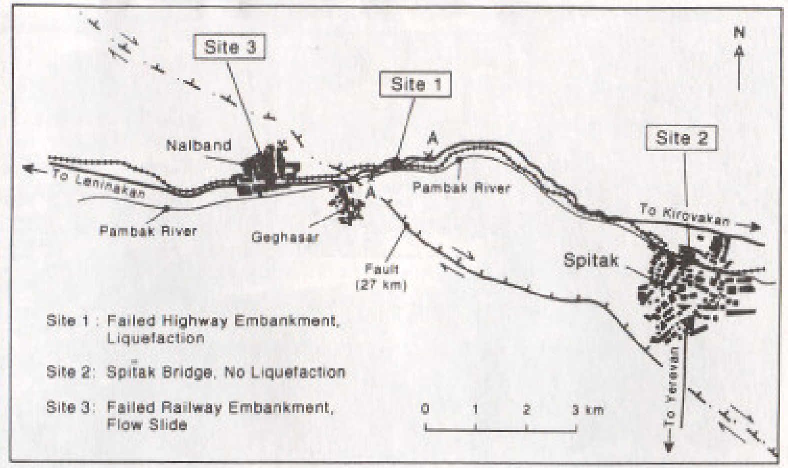


FIG. 3. Location of Sites of Three Case Histories

FIG. 4. Photograph of Failed Spitak Highway (Site 1)

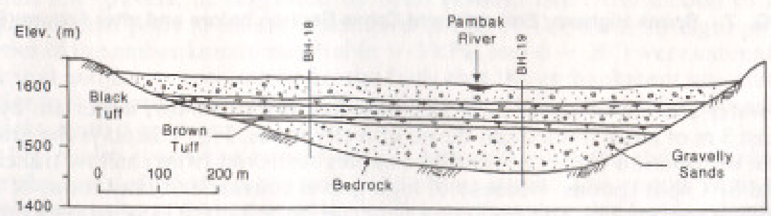


FIG. 5. Geological Cross-Section A-A (Fig. 3) of Pambak River Valley (from A. Vehouni, Personal Communication, 1990)

FIG. 6. Evidence of Sand Boils Adjacent to Failed Spitak Highway Embankment (Site 1)

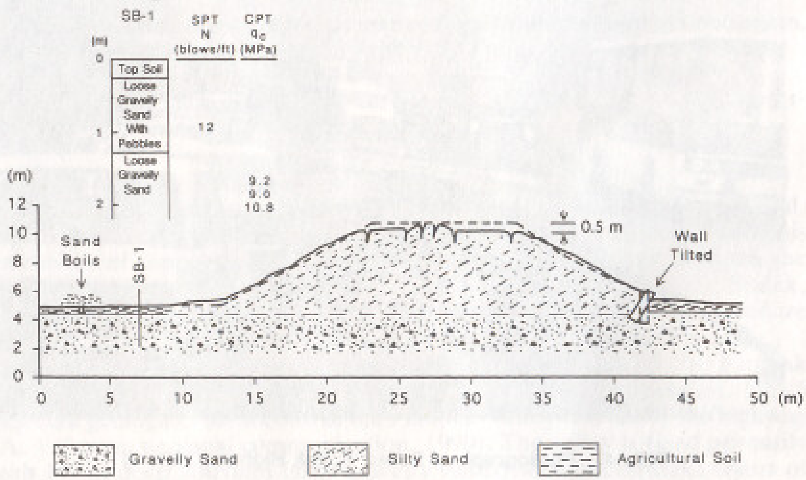


FIG. 7. Spitak Highway Embankment Cross Section before and after Failure (Site 1)

a water content of about 82% and liquidity index of 34%, underlain by at least 3 m of loose to medium-dense gravelly sands. Fig. 8 displays the grain-size distribution curves of the soil samples retrieved from shallow trenches and SPT split spoons. Because of high gravel content and limitations of the drilling equipment, SPT and cone penetration tests (CPT) were performed to a depth of only 2 m. This was not a major drawback since, as the field evidence reveals, the embankment failure surface was limited to a shallow

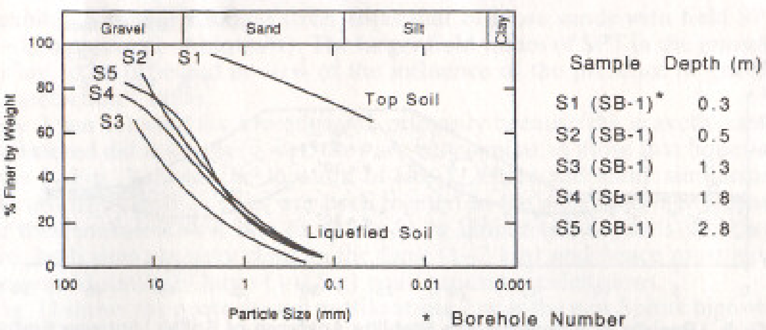


FIG. 8. Grain-Size Distribution Curves of Soil Samples from Site 1

depth within this gravelly sand layer. The field blow counts for this layer were about 36 blows/m (12 blows/ft). The cone tip resistance data ($q_c \approx 9$ to 10.8 MPa) were also utilized to indirectly estimate "equivalent" SPT blow counts for the gravelly sands using relationship of Meigh (1987). The estimated field SPT values range between 12 and 48 blows/m (4 and 16 blows/ft) with an average of 24 blows/m (8 blows/ft). The water table at the time of our field tests was 20 cm below the ground surface; it fluctuates very little during the year.

Our field investigations and observations have led us to the following explanation of the cause of failure of the highway embankment. As stated earlier, failure was limited to the region near a tributary of the Pambak River where the highway was founded on loose gravelly sands with the water table near the ground surface. Because of the proximity of the embankment to the fault, the ground motion at this site was extremely strong (PGA > 0.5 g). But even during such high accelerations, free draining gravels and gravelly sands may not be expected to liquefy (as will be demonstrated at site 2). In this case, however, the presence of an overlying 30–40 cm relatively impermeable soil layer prevented vertical dissipation of the excess pore-water pressures, as they were being generated by the shaking. As a consequence, high pore-water pressures triggered liquefaction or at least substantially reduced the shearing resistance of this deposit beneath, and particularly near the toes of the embankment. This led to instability of the embankment, resulting in cracking and slip deformations (Fig. 7). Eventually, the failed section of the embankment reached an equilibrium position. Stability analyses of the final deformed shape of the embankment were performed to obtain estimates of the residual shear strength of the liquefied sands and gravels, as suggested by Seed (1986). The cross section of the embankment prior to failure is sketched in Fig. 9. The shear strength properties of the embankment material ($c = 5$ kPa, and $\phi = 30°$) were estimated by trial and error, accounting for the facts that the embankment was stable prior to the earthquake (FS > 1.0) and that after failure it exhibited tension cracks up to 1 m deep.

Fig. 9 also shows the critical circles, assumed to be the sliding surfaces on the basis of photographs and field observations. The results from the stability analyses show that prior to the earthquake, the factor of safety associated with these slip surfaces is about 1.7, which is close to FS = 1.5 commonly used in the design of such embankments. This leads to the con-

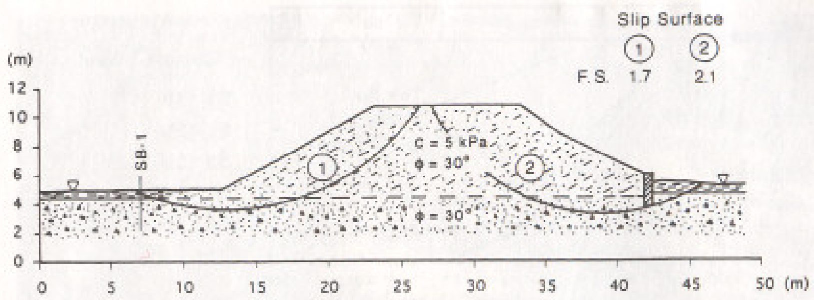


FIG. 9. Results of Preearthquake Stability Analyses of Spitak Highway Embankment (Site 1)

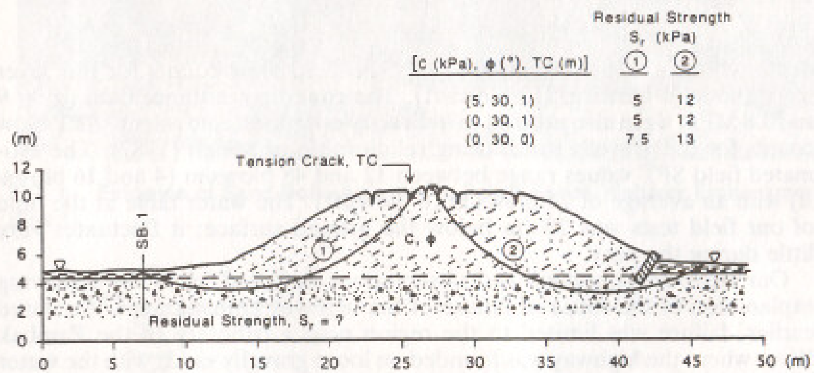


FIG. 10. Results of Postearthquake Stability Analyses of Spitak Highway Embankment (Site 1)

clusion that the estimated material properties (c = 5 kPa, and ϕ = 30°) are reasonable.

As already stated, liquefaction of the gravelly sands (especially in the toe region of the embankment) resulted in shallow slip failures. The final shape of the embankment is depicted in Fig. 10 along with critical circles for postearthquake stability analyses. The minimum value of the residual shear strength was determined by trial and error to achieve a factor of safety FS = 1 on such shallow critical surfaces. The uncertainty in the properties of the embankment material and the depth of the cracks was accounted for parametrically: the cohesion c of the embankment fill was varied from 0 to 5 kPa, and the tension-crack depth from 0 to 1 m. Thus, the calculated residual shear strength of the liquefied gravelly sands ranged between 5 and 13 kPa.

Comparison of these residual shear strength values with those of clean sands published by Seed et al. (1989), leads to the observation that the shallow (1–2 m deep) gravelly soil layer with field SPT ≈ 36 blows/m (12 blows/ft) retained, during liquefaction, a residual strength comparable to that of clean sands [$N_1(60)$ ≈ 15–33 blows/m (5–11 blows/ft)]. Under similar site conditions (depth ≈ 1–2 m and water table near the ground surface) the field SPT of these sands will range between 6 and 12 blows/m (2 and 4 blows/ft). Thus, the gravelly sands with field SPT ≈ 36 blows/m (12 blows/

ft) exhibited similar residual strength as that of loose sands with field SPT ≈ 6–12 blows/m (2–4 blows/ft). The larger field values of SPT in the gravelly soils are to be expected in view of the influence of the presence of gravels ("Liquefaction" 1985).

Site 2 was selected for investigation primarily because the gravelly sands encountered did not liquefy, yet, they are very similar to those that liquefied at site 1. Fig. 3 shows the location of site 2. There are many similarities between sites 1 and 2. They are both located in the same geologic region, near the Pambak River, and therefore have similar soil deposits. Furthermore, both sites are very close to the fault (1–2 km) and hence must have experienced similarly large (>0.5 g) peak ground accelerations.

Fig. 11 shows the geotechnical profile at site 2 near the new Spitak highway bridge. Our field surveys showed no evidence of sand boils, ground deformations, or any tilt or settlement of the bridge piers, thereby confirming that liquefaction did not take place at this site. Fig. 11 also shows that the site consists of an upper 3–4 m thick layer of highly permeable (k of the order of 30 m/s) medium-dense gravel, underlain by about 5 m thick layer of loose to medium-dense silty gravelly sands with permeability of the order of 0.5 m/s. Fig. 12 plots the grain-size distribution of these two layers. Note that the gravelly silty sand (layer 2) at this site, except for its higher silt content (about 22%), is very similar to that encountered at site 1. The question is, why this primarily gravelly sand layer liquefied at site 1 but not at site 2?

There are two likely factors. Perhaps the most important is that, the gravelly sands at site 1 were below a practically impermeable cap (topsoil

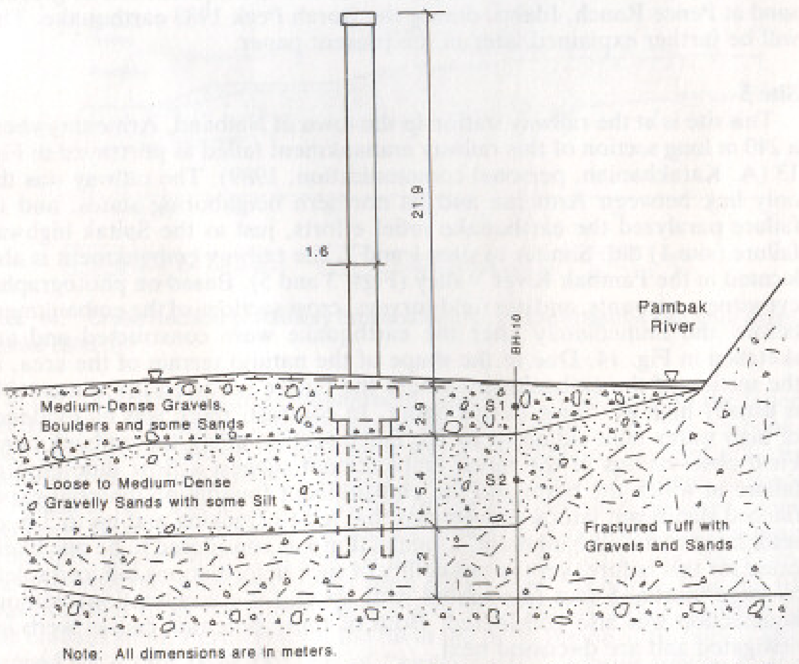


FIG. 11. Soil Profile at Location of Spitak Bridge (Site 2) (Jughourian, ARMNIISA, Personal Communication, 1989)

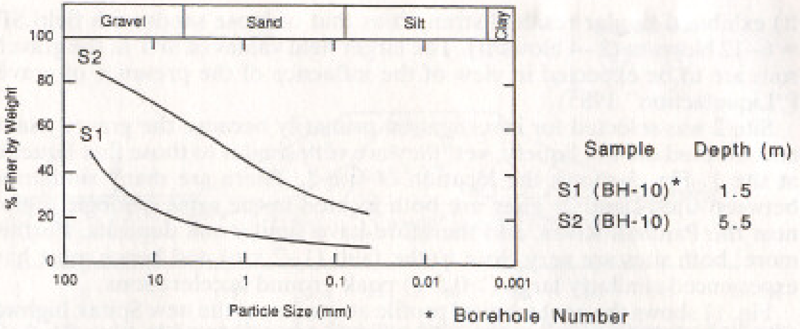


FIG. 12. Grain-Size Distribution Curves of Soil Samples from Site 2

and presence of embankment), whereas at site 2 the top layer covering the gravelly sand is very permeable, allowing the rapid dissipation of excess pore pressures. The higher fines content at site 2 (22% compared to <5% at site 1) may have also contributed to a lesser degree to its greater resistance to liquefaction. However, the third case history (site 3), presented in this paper, has shown that this was not a significant factor; liquefaction of a similar gravelly sand with 25% silt content did take place.

In conclusion, gravelly sands that are not confined by impermeable layers can withstand extremely large (0.5–1.0 g) peak ground accelerations without experiencing liquefaction, whereas the presence of a thin (30–40 cm) impermeable cap can make such soils susceptible to liquefaction. There is a similarity between this case of liquefaction and the liquefaction of gravelly sand at Pence Ranch, Idaho, during the Borah Peak 1983 earthquake. This will be further explained later in the present paper.

Site 3

This site is at the railway station in the town of Nalband, Armenia, where a 240 m long section of this railway embankment failed as portrayed in Fig. 13 (A. Karakhanian, personal communication, 1989). The railway was the only link between Armenia and its northern neighboring states, and its failure paralyzed the earthquake relief efforts, just as the Spitak highway failure (site 1) did. Similar to sites 1 and 2, the railway embankment is also located in the Pambak River Valley (Figs. 3 and 5). Based on photographs, eyewitness accounts, and our field surveys, cross sections of the embankment before and immediately after the earthquake were constructed and are sketched in Fig. 14. Due to the shape of the natural terrain of the area, in the section of the embankment where the failure occurred the water table is usually high as shown in this figure. In contrast, there was no evidence of high water table adjacent to the failed section along the embankment. Field observations of the failed embankment suggest a flow slide type of failure in which the berm and the embankment foundation material experienced significant lateral movements that led to cracking and up to 3 m of settlement of the embankment. Liquefaction was considered to be one likely cause for this failure; yet the possibility of such deformations induced solely by large inertial forces (associated with up to 1.0 g acceleration) without liquefaction was also considered. Both of these potential causes were investigated and are discussed next.

To obtain relevant soil properties, profile geometries and stratification, we conducted field exploration and testing at two locations (NB-1 and NB-

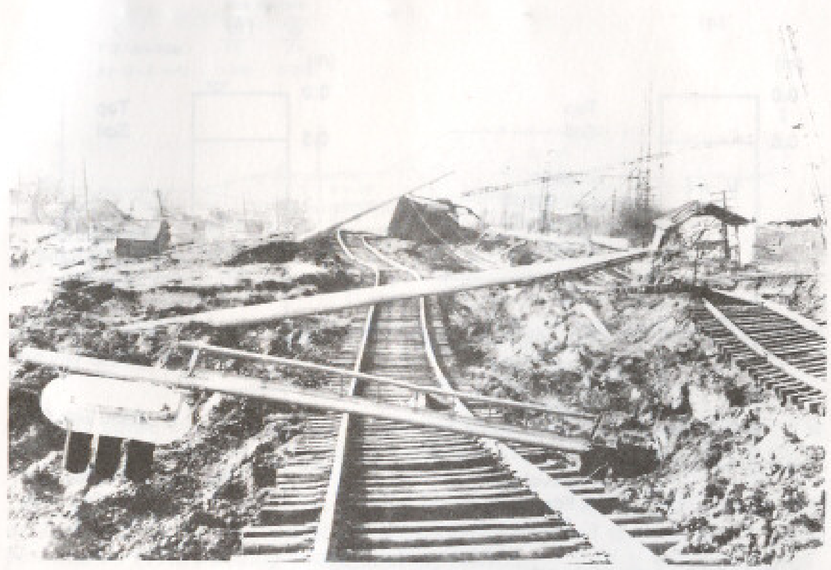


FIG. 13. Photograph of Failed Embankment at Railway Station in Nalband (Site 3) (Photo Courtesy of A. Karakhanian, 1989)

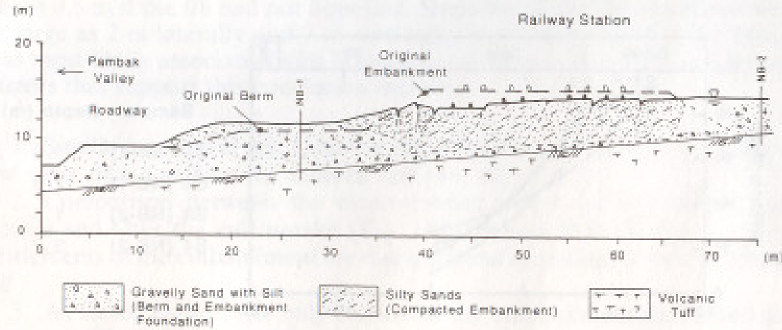
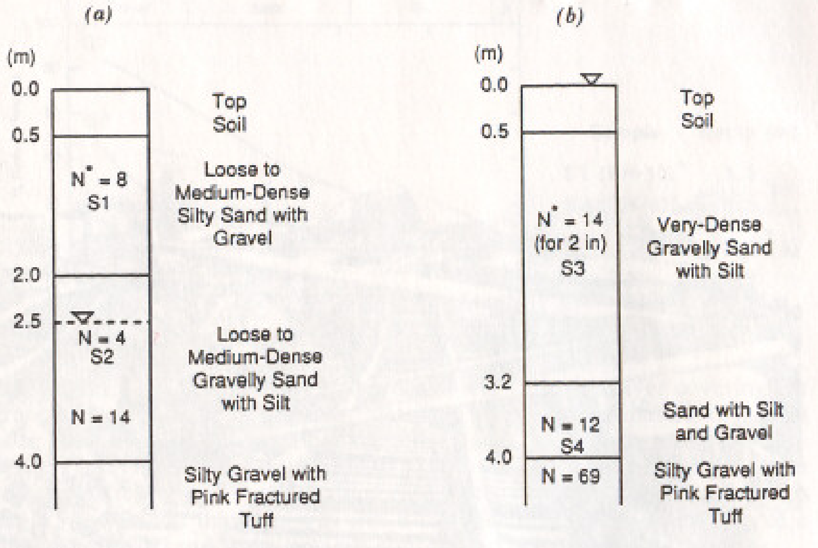


FIG. 14. Cross Section of Railway Embankment in Nalband before and after Earthquake (Site 3)

2 in Fig. 14) in the existing embankment, immediately adjacent to the failed section. Fig. 15 summarizes the results of this investigation. The embankment consists of compacted silty sands and is founded on a layer of gravelly sand fill with up to 25% fines, loosely placed on a naturally-sloping deposit of volcanic tuff. The field SPT values of the gravelly-sand fill layer range between 12 and 42 blows/m (4 and 14 blows/ft). Grain-size distributions from split spoon samples of this fill material are plotted in Fig. 16. Comparison with the gradation curves from sites 1 and 2 confirms that the fill was obtained from the alluvial deposits of the Pambak River adjacent to the embankment. Furthermore, the fill in the berm region at this site is also capped by a thin layer (50 cm) of "impermeable" topsoil similar to the condition encountered at site 1.

Stability analyses were performed, using cross sections corresponding to



* Field SPT values in blows/ft.

FIG. 15. Boring Logs NB-1 and NB-2 at the Nalband Railway Station (Site 3): (*a*) Borehole NB-1; (*b*) Borehole NB-2

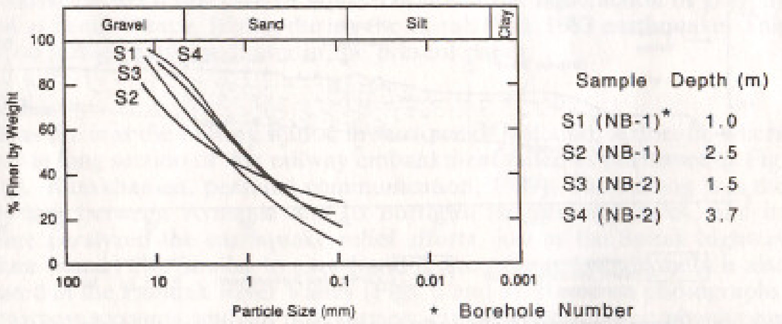


FIG. 16. Grain-Size Distribution Curves of Soil Samples from Site 3

preearthquake and postearthquake conditions, to explore the causes of this failure. Fig. 17 depicts the results of the stability analyses of the original (preearthquake) cross section. The shear strength properties of the embankment were back-estimated in a similar trial and error manner as explained for the highway embankment at site 1. The friction angle for the gravelly sand fill was also obtained from the laboratory analysis of similar material encountered at site 2. Sensitivity analyses indicate that slight uncertainties in the values of shear strength parameters have little influence upon the overall conclusions drawn from stability analyses. From the results of these preearthquake stability analyses (Fig. 17), it is concluded that prior to the earthquake the factors of safety associated with critical failure surfaces were greater than 2.4. Hence, if excess pore pressures did not develop within the embankment, the lateral acceleration required to initiate failure (FS =

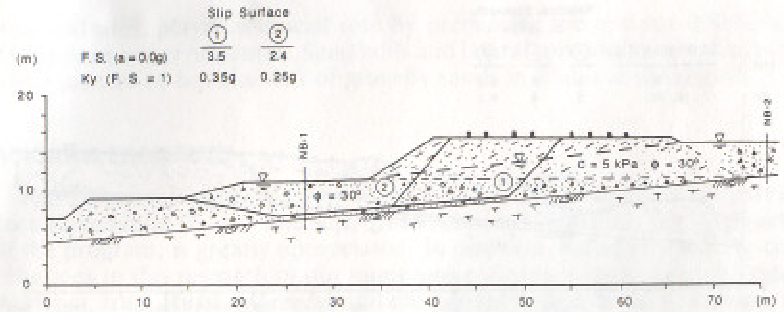


FIG. 17. Results of Preearthquake Stability Analyses of Embankment at Nalband Railway Station (Site 3)

1) along these surfaces would be greater than 0.25 g. This acceleration is commonly referred to as yield acceleration, k_y (Makdisi and Seed 1978; Yegian et al. 1991).

To assess if the failure was due to large inertial forces and without liquefaction, analysis of permanent earthquake-induced deformations using Newmark's (1965) sliding block analogy with $k_y = 0.25$ g was performed. The time history of the rock outcrop motion inferred at Ghoukasian (Fig. 2) was used as excitation after scaling to an upper bound PGA of 1.0 g. The results show that the expected permanent deformations are less than about 0.5 m if the fill had not liquefied. Since the actual deformations were as large as 2 m laterally and 3 m vertically, it is concluded that the failure was most likely associated with liquefaction-induced flow slide. Additional factors that support this conclusion include:

1. Similar material at site 1, having impermeable cap and located at about the same distance from the fault as this site, did liquefy
2. Comparison between the reconstructed shapes of the embankment before and after the earthquake (Fig. 14) indicates that the large vertical settlements of the embankment are due to lateral spreading of the underlying fill
3. As stated earlier, the only section of the embankment that failed was the one with high water table

An examination of the postfailure cross section, sketched in Fig. 18, leads to the observation that, once instability was initiated, lateral movement continued until the driving forces came to equilibrium with the shearing resistance of the gravelly sand fill and the embankment material. The shearing resistance of the liquefied gravelly sand, associated with this condition (FS = 1), referred to earlier as residual strength, was calculated in a similar manner as for site 1. The results, shown in Fig. 18, indicate that the residual shear strength of the gravelly sands with field SPT values of 12 to 42 blows/m (4 to 14 blows/ft), ranged between 5 and 6.2 kPa. This range of values of residual shear strength is similar to that estimated from the analysis of site 1 (5–13 kPa), thus confirming the earlier conclusion that loose to medium-dense gravelly sands, even with 25% silt, when confined against drainage, can liquefy. The residual shear strength of such soils is similar to that of loose clean sands.

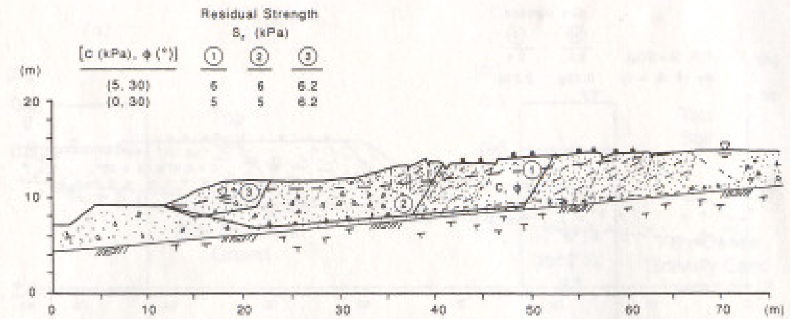


FIG. 18. Results of Postearthquake Stability Analyses of Embankment at Nalband Railway Station (Site 3)

CONCLUSIONS

Case histories of liquefaction-induced embankment failures from the 1988 Armenia earthquake are presented. The liquefied materials consist of sands with about 40–50% gravel. In view of the fact that there is limited knowledge and field experience regarding the behavior of gravels and gravelly sands during earthquakes, these case histories provide valuable information. Field observations and analyses of the data from three sites (two liquefaction, and one no-liquefaction cases), lead to the following conclusions regarding the liquefaction vulnerability of the investigated gravelly sands:

1. When not capped with a relatively impermeable upper layer, these materials can withstand very large (0.5–1.0 g) peak ground accelerations without liquefaction
2. A thin (as little as 30 cm) impermeable layer can impede drainage preventing dissipation of excess pore pressures, thus causing liquefaction of the gravelly sands
3. Liquefaction of the gravelly sands investigated triggered flow slides (in the presence of static shear forces) leading to permanent derformation and settlement of an overlying embankment
4. The residual shear strength of the loose to medium-dense gravelly sands with field SPT values between 12 and 42 blows/m (4 and 14 blows/ft), range between 5 and 13 kPa (100 and 260 psf). These values are back-estimated from stability analyses
5. Comparison of the residual shear strength values of these materials with values corresponding to clean sands published by Seed et al. (1989), indicates that the residual shear strength of loose gravelly sands is similar to that of loose clean sands

There is a noteworthy similarity between the presented case histories and the liquefaction of gravelly sand in the 1983 Borah Peak, Idaho, earthquake, as reported by Andrus et al. (1991). The two events were of slightly different magnitude (6.8 in Armenia, and 7.3 in Idaho), but all sites in both events were located within a few kilometers of the surface breakout of the fault, and are thus likely to have experienced very similar levels of acceleration. The gradation curves in the Armenia and Idaho sites were comparable. More importantly, a practically impermeable soil cap, present in all the

liquefied sites, played a crucial role by preventing the upward diffusion of excess pore-water pressures. Sand boils and lateral spreading were the visible consequences of liquefaction of gravelly sands in Idaho and Armenia.

ACKNOWLEDGMENTS

This research was made possible through a grant from the National Science Foundation (NSF). The support of NSF and Clifford J. Astill, director of the program, is greatly appreciated. In addition, we appreciate the contributions to this research of our numerous colleagues in the United States, Armenia, Iran, Russia, Greece, and the United Kingdom. In particular, we acknowledge the invaluable technical and logistical assistance provided by the following individuals and institutions: Temour Markaryan and Suren Shahinyan of ARMNIISA, Stepan Zargaryan and Azat Vehouni of Yerevan Polytechnic Institute, Lenser Aghalovyan of Armenia Academy of Sciences, Hakob Aloyan of Architectural and Civil Engineering Institute of Yerevan, Razmik Atabekyan of Kirovakan Polytechnic Institute, Dean Paul King of Northeastern University, and to United Armenian Fund for transporting the field investigation equipment to Armenia. Gratitude is expressed to George Gazetas and Mark Evans for the review of this manuscript and their valuable comments.

APPENDIX. REFERENCES

Andrus, R. D., and Youd, T. L. (1989). "Penetration tests in liquefiable gravels." *Proc. 12th ICSMFE*, Balkema Publishers, Rotterdam, The Netherlands, 679–682.

Andrus, R. D., and Youd, T. L. (1987). "Subsurface investigation of liquefaction-induced lateral spread, Thousands Islands, Idaho." *Misc. Paper GL-87-8*, U.S. Army Corps of Engineers, Geotechnical Laboratory, Vicksburg, Miss.

Andrus, R. D., Stokoe, K. H., and Roesset, J. M. (1991). "Liquefaction of gravelly soil at Pence Ranch during the 1983 Borah Peak earthquake." *Proc., Soil Dyn. and Earthq. Engrg., V*, Computational Mech. Publ., Southampton, U.K., 251–262.

Evans, M. D., and Seed, H. B. (1987). "Undrained triaxial testing of gravels—the effect of membrane compliance," *Rep. No. UCB/EERC-87/08*, Univ. of California, Berkeley, Calif.

Evans, M. D., Seed, H. B., and Seed, R. B. (1992). "Membrane compliance and liquefaction of sluiced gravel specimens," *J. Geotech. Engrg.*, ASCE, 118(6), 856–872.

Harder, L. F., and Seed, H. B. (1986). "Determination of penetration resistance for coarse-grained soils using the becker hammer drill." *Rep. EERC-86/06*. Univ. of California, Berkeley, Calif.

Hynes, M. E. (1988). "Pressure generation characteristics of gravel under undrained cyclic loading," PhD thesis, Univ. of California, Berkeley, Calif.

Kokusho, T., Tanaka, Y., and Yoshida, Y. (1991). "In-situ dynamic property evaluation of gravelly soil." *Proc., Soil Dyn. and Earthq. Engrg., V*, Computational Mech. Publ., Southampton, U.K., 177–188.

Liao, S. (1986). "Probabilistic analysis of liquefaction," PhD thesis, Dept. of Civ. Engrg., M.I.T., Cambridge, Mass.

"Liquefaction of soils during earthquakes." (1985). National Research Council, National Academy Press, Washington, D.C.

Makdisi, F., and Seed, H. B. (1978). "Simplified procedure for estimating dam and embankment earthquake-induced deformations." *J. Geotech. Engrg.*, ASCE, 104(7), 849–867.

Meigh, A. C. (1987). "Cone penetration testing: methods and interpretations," CIRCA ground engineering report, Butterworths, London, U.K.

Newmark, N. M. (1965). "Effects of earthquakes on dams and embankments," *Géotechnique*, London, U.K., 15(2), 139–160.
Seed, H. B. (1986). "Design problems in soil liquefaction," *Rep. No. UCB/EERC-86/02*, Univ. of California, Berkeley, Calif.
Seed, H. B., Tokimatsu, K., Harder, L. F., and Chang, R. M. (1985). "Influence of SPT procedures in soil liquefaction resistance evaluations." *J. Geotech. Engrg.*, ASCE, 111(12), 1425–1445.
Seed, H. B., Seed, R. B., Harder, L. F., and Jong, H. L. (1989). "Re-evaluation of the lower San-Fernando Dam. Report 2: Examination of the post-earthquake slide of February 9, 1971." *Contract Report GL-89-2*, Dept. of Army, U.S. Army Wat. Exp. Sta., Vicksburg, MS.
Tanaka, Y., Kokusho, K., Kudo, K., and Yoshida, Y. (1991). "Dynamic strength of gravelly soils and its relation to the penetration resistance." *Proc. 2nd Int. Conf. on Adv. in Geotech. Earthq. Engrg. and Soil Dyn.*, 1, Univ. of Missouri–Rolla, Rolla, Mo., 399–406.
Wyllie Jr., L. A., and Filson, J. R., eds. (1989). "Armenia earthquake reconnaissance report." *Earthq. Spectra*, Special supplement.
Yegian, M. K., Ghahraman, V. G., and Gazetas, G. (1994a). "1988 Armenia earthquake, I: seismological, geotechnical and structural overview," *J. Geotech. Engrg.*, ASCE, 120(1), 1–20.
Yegian, M. K., Ghahraman, V. G., and Gazetas, G. (1994b). "1988 Armenia earthquake, II: damage statistics versus geologic and soil profiles," *J. Geotech. Engrg.*, ASCE, 120(1), 21–45.
Yegian, M. K., Ghahraman, V. G., and Gazetas, G. (1994c). "Ground-motion and soil-response analyses for Leninakan, 1988 Armenia earthquake." *J. Geotech. Engrg.*, ASCE, 120(2), 330–348.
Yegian, M. K., Ghahraman, V. G., and Gazetas, G. (1994d). "Seismological, soil and valley effects in Kirovakan, 1988 Armenia earthquake." *J. Geotech. Engrg.*, ASCE, 120(2), 349–365.
Yegian, M. K., Marciano, E. A., and Ghahraman, V. G. (1991). "Earthquake-induced permanent deformations: probabilistic approach," ASCE, *J. Geotech. Engrg.*, 117(1), 35–50.
Youd, T. L., Harp, E. L., Keefer, D. K., and Wilson, R. C. (1985). "The Borah Peak, Idaho earthquake of Oct. 28, 1983—Liquefaction." *Earthq. Spectra*, 2(1), 71–89.