

Case History of Observed Liquefaction-Induced Settlement Versus Predicted Settlement

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SUMMARY:

A comparison of the predicted liquefaction-induced settlement using procedures consistent with United States practice with the observed liquefaction-induced settlement of a site in the western San Fernando Valley during the January 17, 1994 Northridge earthquake with a moment magnitude of 6.7 is presented. The observations were made a few days after the Northridge earthquake and were documented in photographs. Estimates of the liquefaction-induced settlement were possible because existing buildings at the site were supported on piles driven to bear in bedrock. The soil profile beneath the site consists of Holocene age alluvium with predominantly silt and sand overlying Miocene age bedrock at depths between 40 and 60 feet (12.2 and 18.3 m). The observed settlement of the ground surface is substantially smaller than the settlement predicted by currently used methods for estimating settlement for clean sands.

Keywords: Liquefaction, settlement, Northridge earthquake

1. INTRODUCTION

Simplified methodologies for estimating liquefaction-induced settlement have been developed for clean sands and have been used for analysis and design of structures in liquefaction-prone areas. The application of these methodologies to sites with soils that are not purely clean sands has been discussed and debated although there is a general sense in the professional community that the settlements of finer grained soils should be smaller than for clean sands. A case history for a site in the western San Fernando Valley that experienced liquefaction-induced settlement during the 1994 Northridge earthquake is presented. Using available data from previous geotechnical investigations conducted at the site, an analysis was made using available methods for predicting liquefaction-induced settlement. The observed and predicted settlements are compared and discussed.

2. OBSERVATIONS OF LIQUEFACTION-INDUCED SETTLEMENT IN THE 1994 NORTHRIDGE EARTHQUAKE

On January 17, 1994, a moment magnitude 6.7 earthquake occurred on the Northridge blind thrust fault in the San Fernando Valley which is part of the City of Los Angeles, California. Hall (1996) reported that although evidence of liquefaction with associated surface features such as sand boils was widespread, it generally had little impact on structures. A few days after the earthquake, the first author visited a site in the western San Fernando Valley, as shown in Figure 2.1, and observed evidence of liquefaction-induced settlement. Surface manifestations, such as sand boils or lateral spreading, were not observed. The epicentral distance to the site was less than 4 miles (6.4 km).

2.1. Site Conditions

The site consists of a complex of three- to five-story buildings, some of which have one basement level; the buildings were constructed in two phases, the first in the early 1980s and the second in the

early 1990s. The site is located on the southern edge of the floor of the western portion of the San Fernando Valley. The geotechnical report for the site indicates the upper soils consist of Holocene age alluvium; the alluvium consists of poorly sorted alluvial fan deposits of sand and silt, with lesser amounts of gravel and clay (LeRoy Crandall and Associates, 1980). Subsurface explorations indicate that the alluvium extends to depths between 40 and 60 feet (12.2 and 18.3 m) where it is underlain by Miocene age deposits. The Miocene age deposits consist of marine deposited siltstone, diatomaceous shale, fine sandstone and shale which are believed to extend to depths of about 4,000 feet (1,220 m). Groundwater was encountered in exploration borings at depths of 19 to 20 feet (5.8 to 6.1 m) below the ground surface in 1980.

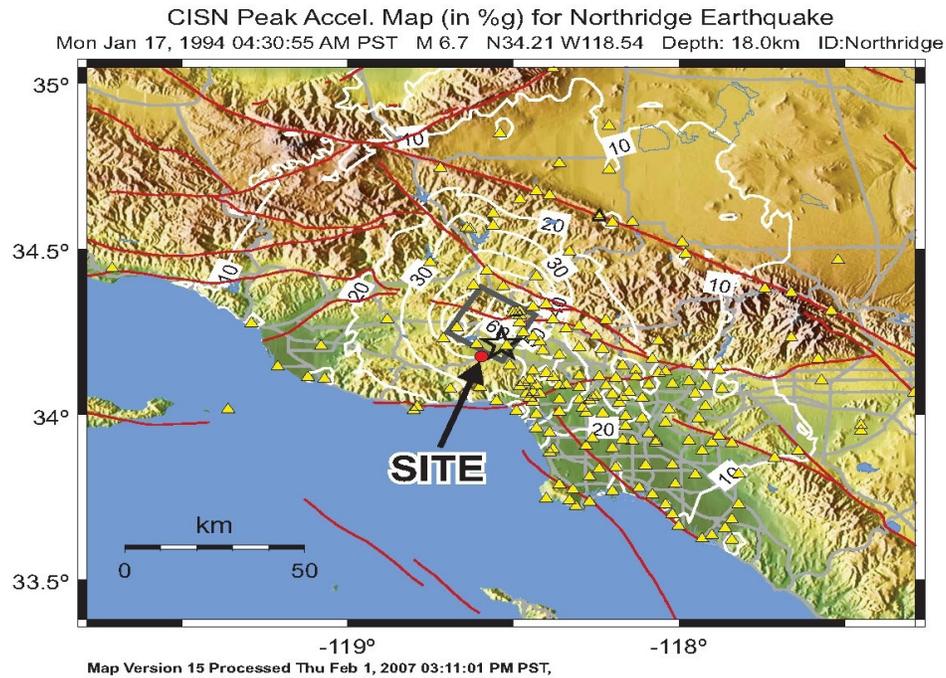


Figure 2.1. Site Location and ShakeMap for the Northridge Earthquake (After California Integrated Seismic Network, 2012)

The idealized soil profile is shown in Figure 2.2; also are the raw uncorrected standard penetration test resistances (“N-values”) associated with each idealized layer.

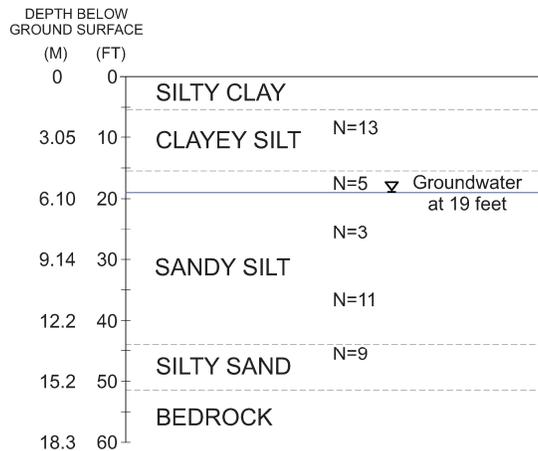


Figure 2.1. Idealized soil profile

As shown in Figure 2.3, the particle size distribution shows that the alluvial deposits are predominantly sandy silt or silty sand; the fines contents by weight (passing the No. 200 sieve) is between about 40 to 70 percent.

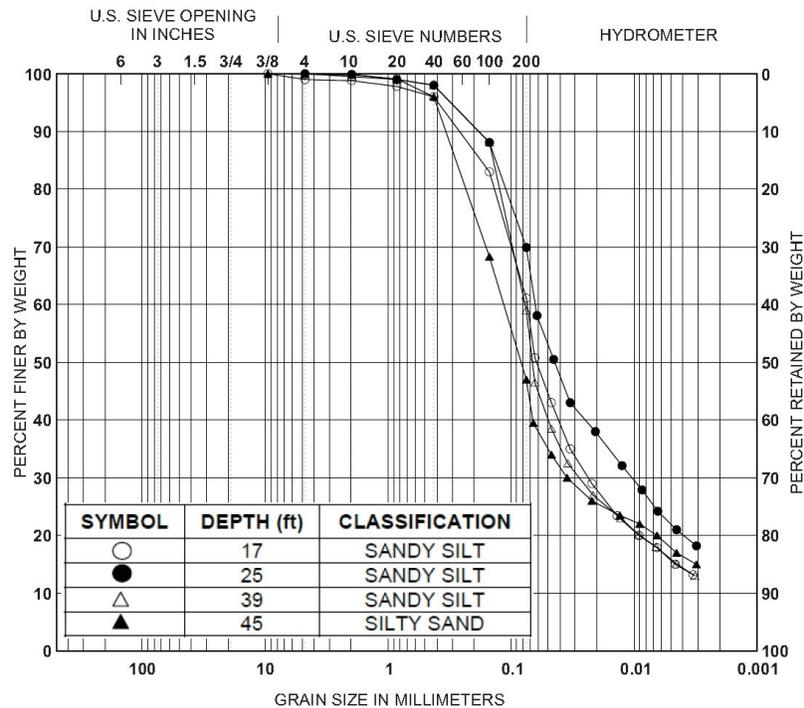


Figure 2.3. Particle size distribution of soils

2.2 Estimated Ground Motions

There was no seismic instrumentation at the site during the Northridge earthquake. However, the peak ground acceleration at the site may be estimated by the ShakeMap published by the California Integrated Seismic Network as shown in Figure 2.1. Based on the contours in the map, the peak ground acceleration at the site was about 0.40g.

2.3. Observations of Liquefaction-Induced Settlement

On February 19, 1994, the first author performed a site reconnaissance of several affected properties two days after the Northridge earthquake. One of the sites was at the location that is the subject of this paper in the western San Fernando Valley of Los Angeles. There was minor structural damage to the three- to five-story steel framed buildings at the site and the buildings remained in service after the earthquake. There was more severe damage to a five-story parking structure constructed of reinforced concrete. Observed damage to the parking structure included shear failures in the columns and shear cracking and crushing at boundary elements of shear walls which necessitated the closure of the parking structure. There was no collapse of the structure but extensive repairs were needed before the structure could be put back into service.

It was also observed that there was a general subsidence or settlement of the ground surface relative to the buildings at the site. Examples of the observed settlement are shown in Figures 2.4 and 2.5 which show the settlement of the concrete slabs on grade relative to the buildings. The relative settlement of the slabs on grade was observed to be between about 1.5 to 2 inches (3.8 to 5.1 cm). This settlement was observed throughout the site adjacent to several buildings. Visual observations of the paved areas

near the buildings did not indicate that there was significant differential liquefaction-induced settlement of the ground surface. In particular, there was a canopy structure adjacent to one of the buildings on the site that was supported on shallow spread footings established in the upper soils (shown in Figure 2.6); the canopy is about 150 feet (45.7 m) long and about 20 feet (6.1 m) wide. There was no observed significant differential settlement of the canopy structure columns which are 20 or 30 feet (6.1 or 9.1 m) apart nor was there significant differential settlement of the concrete paving beneath or surrounding the canopy.



Figure 2.4. Liquefaction-induced settlement of exterior slab-on-grade



Figure 2.5. Liquefaction-induced settlement of exterior concrete steps

The existing three- to five-story buildings at the site are supported on precast concrete pile foundations that were driven into bedrock. Therefore, it is logical to assume that the buildings did not settle during the earthquake and the observed surface settlement is the total settlement of the ground surface due to liquefaction of the soils beneath the site due to the earthquake ground shaking.



Figure 2.6. Canopy structure supported on shallow spread footings

3.0 ANALYTICAL ESTIMATES OF LIQUEFACTION-INDUCED SETTLEMENT

There are several generally accepted methods of estimating the liquefaction-induced settlement of saturated sand soils used by the professional community in the United States. Please note that Idriss and Boulanger (2008) refer to liquefaction-induced settlement as “post-liquefaction reconsolidation settlement.” The two most commonly used methods are those by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). The Tokimatsu and Seed method uses a correlation between the normalized standard penetration test resistance $(N_f)_{60}$ and relative density and an estimate of the shear strain potential of liquefied soil from $(N_f)_{60}$ and cyclic stress ratio so that volumetric strain after liquefaction in a magnitude 7.5 earthquake can be estimated directly from the cyclic stress ratio and standard penetration test resistance (Kramer, 1996). The Ishihara and Yoshimine method uses either the factor of safety against liquefaction or the maximum cyclic shear strain, and the relative density, standard penetration test resistance or cone penetration test tip resistance to estimate the postliquefaction volumetric strain (Kramer, 1996). Both of these methods were developed for clean sands.

The following assumptions based on the expected ground motion at the site during the Northridge earthquake were used in the analyses to estimate the liquefaction-induced settlement by the two methods for the soil profile shown in Figure 2.2:

- Moment magnitude = 6.7
- Maximum ground acceleration = 0.40g
- Groundwater depth = 19 feet (5.8 m)

The analyses were performed considering the alluvial deposits below the 19-foot (5.8-m) groundwater surface down to the contact with bedrock at about a depth of 51 feet (15.5 m). An analysis of the potential for liquefaction triggering of these deposits using the simplified method described by Youd and Idriss (2001) determined that these soils have a factor of safety of significantly less than unity during the Northridge earthquake event indicating that the soils did liquefy as the estimated cyclic stress ratios greatly exceeded the cyclic resistance ratios. This is not a surprising result as the standard penetration test resistances recorded in the alluvial deposits were quite low as shown on Figure 2.2.

The results of the liquefaction-induced settlement from the analyses by the Tokimatsu and Seed method and by the Ishihara and Yoshimine method are given below in Table 3.1; the average estimated settlement is about 10 inches (25.4 cm).

Table 3.1. Predicted Liquefaction-Induced Settlement

Analysis Method	Estimated Ground Surface Settlement	
	Inches	cm
Tokimatsu and Seed	8.5	21.6
Ishihara and Yoshimine	11.5	29.2

4. DISCUSSION ABOUT OBSERVED VERSUS CALCULATED SETTLEMENTS

The average of the predicted ground surface settlements are on the order of about 6 times greater than the average of the observed ground surface settlements. The reason for this discrepancy may be that the actual soils at the site are sandy silts and silty sands with significant fines content; the particle size analyses, as shown in Figure 2.3, indicate that these soils have 40 to 70 percent fines content by weight passing the No. 200 sieve.

It has been recognized that the liquefaction behavior of soils is significantly affected by the fines content (Martin and Lew, 1999). In the simplified liquefaction triggering analysis originally introduced by Seed and Idriss (1982) for clean sands, there is a fines correction procedure. For post-liquefaction settlement analysis, Ishihara (1993) recommended increasing the cyclic shear strength of the soils if the Plasticity Index (PI) of the fines is greater than 10; this would increase the factor of safety against liquefaction and thus decrease the seismically induced settlement using the Ishihara and Yoshimine method. In addition, field observations by O'Rourke et al. (1999) and Egan and Wang (1991) suggest that the Tokimatsu and Seed method results in overestimation of liquefaction-induced settlements.

Seed (1987) recommended that a correction based on the fines content be applied to the normalized standard penetration test resistance, $(N_1)_{60}$, before determining the post-liquefaction settlements; these corrections are presented in Table 4.1. Because this recommendation predated the publishing of the Ishihara and Yoshimine method, these corrections are presumed to apply to the method proposed by Tokimatsu and Seed.

Table 4.1. Corrections to $(N_1)_{60}$ for fines content for settlement analyses (After Seed, 1987)

Fines content (percent)	N_{corr} (blows per foot [0.31 m])
10	1
25	2
50	4
75	5

Idriss and Boulanger (2008) suggest that consideration of sands with significant fines content can be approximately taken into account by use of the equivalent clean sand penetration resistance and state that this approximation is considered reasonable for practical purposes. Idriss and Boulanger have also developed the standard penetration test (SPT) based liquefaction correlation for clean sands assuming an earthquake with magnitude equal to 7.5 and vertical effective stress at consolidation equal to one atmosphere as shown in Figure 4.1; the figure shows the variation of volumetric strains as a function of the corrected standard penetration resistance, $(N_1)_{60cs}$, during post-liquefaction reconsolidation based on the methodology of Ishihara and Yoshimine.

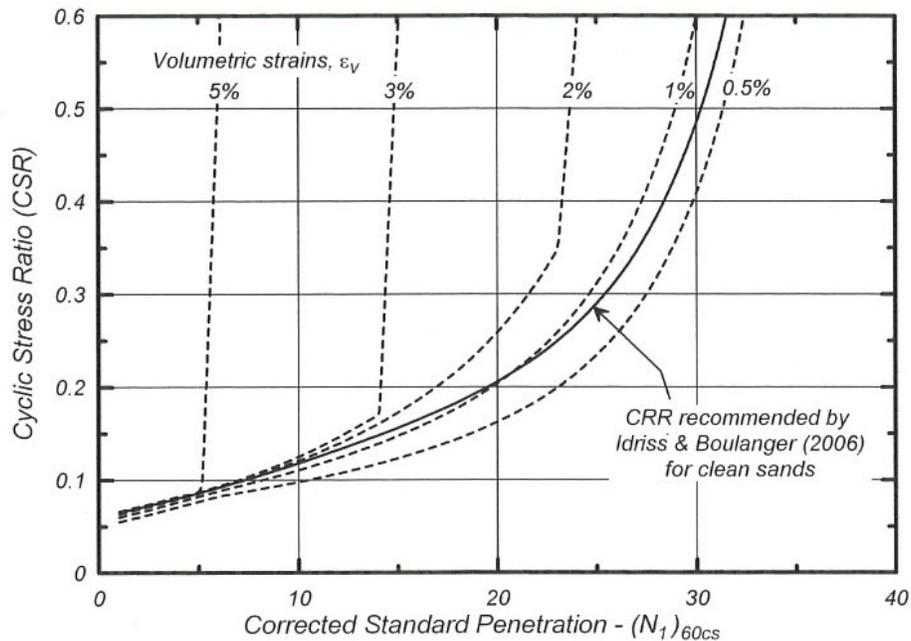


Figure 4.1. SPT-based liquefaction correlation for clean sands with $M=7.5$ and $\sigma'_{vc} = 1$ atm (After Idriss and Boulanger, 2008)

As stated earlier, the estimated liquefaction-induced settlement was about 8.5 to 11.5 inches (21.6 to 29.2 cm) according to the Tokimatsu and Seed method and the Ishihara and Yoshimine method. Using the volumetric strains from the Idriss and Boulanger correlation shown in Figure 4.1, the estimated settlement would be about 13 inches (33 cm), which is somewhat greater than the estimates calculated by the Tokimatsu and Seed or the Ishihara and Yoshimine methods.

Thus the average of the analytical estimates of liquefaction-induced settlement is almost 6 times greater than the average of the observed settlement at the site. It would appear that the fines content adjustment for triggering of liquefaction does not adequately account for the effects of the fines content in liquefaction-induced settlement evaluations in this case. This would leave a few questions that would need to be considered.

The first question would be: What is the quality of the original geotechnical data? The available data from the site was from geotechnical investigations conducted in 1980 for the existing buildings. The borings were spread throughout the site and standard penetration tests were performed in some of the borings at intervals of approximately 10 feet (3.05 m). The standard penetration test blow counts used in the analyses may have some uncertainty as this method of exploration has been shown to have much variability due to sampling techniques, drilling equipment, and stress-strain behavior of the soil during sampling (Youd and Idriss, 2001 and Rogers, 2006). A few particle size distribution tests were performed but, unfortunately, Atterberg Limits tests were not performed to determine the plasticity of the soils. However, the field logs from the borings indicate that the soils were low in plasticity but there was some clay encountered during the drilling but discrete sampling did not produce samples

containing significant amounts of clay. The lack of samples at this point in time does not make it possible to determine if the suspect soils might have sufficient plasticity that they might be excluded from the analysis because of “clay-like” behavior as described by Idriss and Boulanger (2008). It is possible that there may be some clay or clay-like materials that may not be liquefiable, however, it is not likely that the amount of clay or clay-like material was that large that it would account for the substantial differences in liquefaction-induced settlement observed and calculated.

A second question might be: Is the current practice of fines content adjustment adequate for soils with fines content exceeding 35 percent? For this case history, the sandy silt/silty sand soils had fines contents in the range of 40 to 70 percent. The correction for fines content (FC) greater than about 35 percent is constant; i.e., the FC correction for 35 percent is to be used for soils with more than 35 percent fines. According to Idriss and Boulanger (2008), this is consistent with experimental observations that the behavior of silty sand with higher fines content is largely governed by the matrix of fines, with the sand particles essentially floating with the matrix. While use of the correction for fines content may be applicable to the determination of triggering of liquefaction, it may not be possible that the correction is not applicable to the evaluation post-liquefaction reconsolidation settlement. This may be because the fines matrix may not to behave like sand particles when it comes to settlement behavior.

To obtain a comparable liquefaction-induced settlement on the order of 1.5 to 2 inches (3.8 to 5.1 cm) for this soil profile using the Tokimatsu and Seed method, the corrected standard penetration test resistance in each layer for clean sand, $(N_1)_{60cs}$, would need to be increased by 7 or 8 blows per foot. This in effect is essentially increasing the fines correction factor (corresponding to 35 percent fines) by slightly more than a factor of 2.

5. IMPLICATIONS OF OVERESTIMATES OF LIQUEFACTION-INDUCED SETTLEMENT

There are some significant implications of overestimates of liquefaction-induced settlements. One major concern would be for the development of new construction. Overestimates of the liquefaction-induced settlement could put a greater economic burden on the cost of construction. As the settlement estimates increases, it becomes more difficult to justify the use of shallow spread footing foundation systems to accommodate the total and differential settlements and may cause consideration of more substantive foundation systems such as mat or pile foundations which may be more costly. If the settlement is even larger, mat foundations may not be feasible and some form of deep foundation, such as piles, would be required. In the City of Los Angeles, the Department of Building and Safety (LADBS) will generally only allow the use of spread footings for combined static settlement and liquefaction-induced settlement of up to 1.5 inches (3.8 cm). LADBS will generally allow the use of a mat foundation for combined static and liquefaction-induced settlement of up to 3 inches (9.7 cm). For combined static and liquefaction-induced settlements greater than 3 inches (9.7 cm), deep pile foundations extending to stable non-liquefiable soils would be necessary.

Besides the total liquefaction-induced settlement, differentially liquefaction-induced settlements are generally more important than the total settlement as differential settlements can cause more structural load demands that cause damage and disruption to structures by causing deformations over short distances. Furthermore, these deformations may inhibit function of the structure and limit or prevent access and/or egress threatening life safety. The California Geological Survey in its Guidelines for Evaluating and Mitigating Seismic Hazards in California (California Geological Survey, 2008) suggest that “... localized differential settlements on the order of up to two-thirds of the total settlements anticipated should be assumed unless more precise predictions of differential settlements can be made.” Thus if the total liquefaction-induced settlements are overestimated, the differential settlements would also be overestimated.

If structural foundation support systems are not used to accommodate the structural settlements due to static and liquefaction, ground improvement technologies would need to be used; these technologies

are described by Kramer (1996) and by Idriss and Boulanger (2008). These methods include soil densification and soil hardening (mixing) techniques. Soil densification techniques include vibro-compaction, vibro-replacement (commonly known as stone columns), deep dynamic compaction (impact densification), and compaction (pressure) grouting (Hayden and Baez, 1994). Another soil densification technique is explosive compaction. Excavation and recompaction or replacement is another technique but may be problematic if extending deep below the groundwater level. Hardening and/or mixing techniques seek to reduce the void space in the liquefiable soil by introducing grout materials either through permeation, mixing mechanically, or jetting; these methods are known as permeation grouting, soil mixing, or jet grouting, respectively. In some cases, drainage systems consisting of vertical drains can be installed to allow for increased dissipation rates for excess pore water pressure or modify dissipation patterns (Idriss and Boulanger, 2008).

Some combinations of these structural and/or ground improvement techniques can also be used to mitigate the liquefaction hazard and improve performance.

Overestimation of the liquefaction-induced settlement can therefore cause the economic costs of developing a construction project to increase. Whether in the form of using mat or deep foundation systems or having to perform extensive ground improvement, the cost of development will significantly increase. For smaller construction projects, the increased costs can be prohibitive and make a project unfeasible.

6. CONCLUSIONS

The case history presented an account of observations of liquefaction-induced settlement at a site where liquefaction was triggered during the Northridge earthquake in 1994. The soils below the groundwater level consisted of Holocene age alluvium that was predominately sand and silt with lesser amounts of clay and gravel. Settlements of the ground surface on the order of 1.5 to 2 inches (3.8 to 5.1 cm) were observed adjacent to buildings that were supported on pile foundations that were driven into bedrock. Liquefaction-induced settlements were estimated by the Tokimatsu and Seed method as well as the Ishihara and Yoshimine method for the conditions at the site during the Northridge earthquake using a moment magnitude of 6.7 and estimated maximum ground surface acceleration of 0.40g. The two methods predicted that the ground surface settlement would be 8.5 to 10.5 inches (21.6 to 29.2 cm), which was much greater than the observed ground settlement.

The available geotechnical information about the site is somewhat limited by the lack of Atterberg Limits testing results, however, standard penetration test blow counts are available from several borings. Particle size analyses for several samples of the alluvial soils show that the soils have fines content (i.e., passing the No. 200 sieve – less than 0.074mm) in the range of about 40 to 70 percent by weight. The liquefaction analyses did not exclude any of the soils because of clay or clay-like behavior; the amount of clay reported in the geotechnical logs of borings was minor and was thus neglected. If there were significant amounts of clay or clay-like materials, the amount of estimated settlement would be reduced, however, any potential reduction in settlement would not account for all of the large discrepancy between observed and calculated settlements. It is apparent that fines content correction factors used in liquefaction triggering analyses may not be applicable for liquefaction-induced settlement evaluations.

Considerable engineering judgment will need to be exercised in the evaluation of liquefaction-induced settlements when there are soils with considerable fines contents greater than 35 percent. Reconciliation of the settlements between the analytical techniques and the observed settlements could be achieved by increasing the fines content correction factor, however, the case history presented herein is only a single case history. Additional case histories should be studied.

The implications or consequences of overestimation of liquefaction-induced settlement can cause considerable economic impact on construction. Overestimation of the settlements can lead to more

complicated and expensive foundation systems or very costly ground improvement techniques to be used. The additional cost of structural or ground improvement mitigation may not be necessary or not as extensive. Overestimation may also cause proposed construction to be deemed uneconomical and not proceed.

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