CHAPTER 9: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

9.1 Introduction

Many existing highway bridges are potentially vulnerable to damage from earthquake-induced liquefaction because they are located in areas where liquefiable soils are present and were constructed at a time when the mechanisms and risk of earthquake-induced liquefaction were not well understood. Ground improvement can be used to reduce the potential of damage to bridges and other structures due to this phenomenon, but there is a need for an increased understanding of the factors and phenomena that affect the behavior and performance of the improved ground and supported structure system. In addition the feasibility of using certain ground improvement methods to reduce ground and structure movements to tolerable levels needs to be better understood. The objectives of this research were to address these issues for ground improvement used for liquefaction mitigation at existing highway bridges.

To fulfill the research objectives the scope of work completed for this dissertation included:

- Completing a literature review on the performance of improved ground for liquefaction mitigation relevant to existing highway bridges, as well as analytical procedures for predicting ground deformations, accelerations and pore water pressures under earthquake loading;
- Identifying factors and phenomena affecting improved ground behavior and design;
- Selecting and evaluating some simplified analytical methods for predicting the performance of an improved ground and supported structure system;
- Choosing and assessing the accuracy of a more comprehensive analytical method for predicting performance, particularly in regards to predicting permanent deformations; and

- Investigating the suitability of various ground improvement types and configurations for mitigating potential damage to a bridge and developing a greater understanding of the different factors that affect the performance.

The focus of the research work was on cases involving small to medium bridges supported on shallow foundations only. Parametric studies were conducted for ground improvement at a bridge pier and a stub abutment at the top of an approach embankment supported on shallow foundations. Even though a larger number of existing bridges are supported on deep foundations, such as piles or drilled shafts, the shallow foundation case was evaluated because it is fundamentally a less complex problem in soil-structure interaction than the deep foundation case. Therefore, an emphasis was placed on understanding this less complex problem first. In addition, the results from the shallow foundation case should have some applicability to deep foundations that do not extend completely through liquefiable soil, as well as provide some insights into the behavior of improved ground that is useful for deep foundations that completely penetrate the liquefiable soils.

The literature on the effects and performance of improved ground zones in liquefiable deposits is primarily concerned with studies involving numerical modeling using various computer codes and physical modeling using shaking table or centrifuge tests. Some key observations made in these studies that were relevant to this research work, particularly the parametric study of ground improvement for a stub abutment on an approach embankment and a bridge pier, are given below.

Embankments

- An improved zone is more effective when located under the slope of an embankment. As the zone is moved progressively outward from underneath the embankment slope, the lateral displacement of the embankment gets larger. The zone is ineffective in limiting embankment deformations when one edge of it is located at the embankment toe and the remainder completely outside of the slope. (Riemer et al., 1996; Yanagihara et al., 1991)
- As the distance that a treated zone extends underneath an embankment from the embankment toe increases, the lateral movement of the embankment progressively

decreases. However, the incremental reduction in the displacement gets progressively smaller. (Riemer et al., 1996)

- There is a consistent trend between the predicted permanent displacements and the post-earthquake factor of safety for stability of an embankment with ground improvement. As the factor of safety goes up the predicted displacements go down. However, the factor of safety can not be used as a reliable indicator of displacements because it can not take into account the effect of the input motion frequency (higher frequency motion induces less movement than a low frequency motion having the same amplitude and duration) and ground motion amplification on the displacements. (Riemer et al., 1996)
- The stiffer the improved ground used to contain or restrict the movement of liquefiable soils at an embankment or sloping ground, the smaller the lateral displacement and settlement of the embankment and ground that occur. (Adalier et al., 1996; Yasuda et al., 1991)

Shallow Foundation

- As the width of treatment increases under a model footing placed at the surface of a liquefiable layer, the settlement of the footing decreases. However, the incremental reduction of the footing settlement gets progressively smaller with increasing width of treatment, and the settlement appears to approach a lower bound value such that further increases in the width do not appreciably reduce the settlement. (Hatanaka et al., 1987)
- For a given treatment width beyond the edge of a structure, the measured settlement decreases as the width of the structure increases, even if the bearing pressure of the structure remains constant. On the other hand, as the height-to-width ratio of the structure increases the predicted settlement of the structure increases. (Hatanaka et al., 1987)
- As the depth of treatment beneath a structure increases, the predicted settlement of the structure decreases. However, there appears to be a limiting treatment depth such that additional treatment below that depth does not result in a substantial change in settlement. Using a treatment depth that is close to the limiting value, but less than the full depth of the liquefiable soils, results in lower accelerations being transmitted

to the structure sometime after the start of shaking. The lower transmitted accelerations are due to liquefaction in the untreated portion under the improved zone, which isolates the footing and zone from the base motion. (Liu and Dobry, 1997)

The research conducted for this study was designed to build on this information from the literature, with particular focus on the use of ground improvement for limiting the movements of bridge piers and stub abutments on shallow foundations to tolerable levels. In order to reach this goal the investigation also focused on the performance requirements for bridges, the factors and phenomena affecting the performance of the improved ground and a supported structure, and simple and comprehensive methods for evaluating this performance. Summaries and conclusions from these phases of work are presented below.

9.2 Summary and Conclusions

9.2.1 Performance Requirements

A key performance requirement for foundations supporting bridges is the tolerable movements. For the seismic performance of a bridge, the tolerable movements are generally a function of the serviceability required for the bridge following the design earthquake. The serviceability criteria used for bridges include: (1) minor damage that allows immediate use of the bridge, (2) repairable damage allowing service to be restored at a later date, and (3) non-repairable damage that does not cause collapse but requires bridge replacement.

Movement limits typically given in the literature for bridges are for maintaining the bridge in a serviceable state under non-earthquake loading conditions. Vertical movement limits are usually defined in terms of either distortion (distortion defined in Chapter 4, Section 4.2.1) or gross movement. Horizontal movement limits are given in terms of gross movements. Distortion limits given for continuous and simple span bridges are 0.004 and 0.005 to 0.008, respectively (Moulton et al, 1985; Duncan and Tan, 1991; Lok and Mitchell, 1994). Gross movement limits generally range from 50 to 100 mm and 25 to 38 mm for vertical and horizontal movements, respectively (Duncan and Tan, 1991; Lok and Mitchell, 1994). Youd (1998) and

Youd et al. (1998) give displacement criteria based on the observed performance of bridges subjected to earthquake loading and liquefaction. Their ground movement limits for maintaining serviceability of bridges supported on shallow foundations are 100 to 200 mm for lateral displacement and 25 to 100 mm for settlement, with the larger values being appropriate for modern, well-built bridges. The movement criteria given above are for all post-construction movements, including those that occur prior to an earthquake, with the exception of those by Youd and Youd et al. which are only for seismically-induced movements.

The stability safety factor for bridge foundations under earthquake loading can be evaluated as a potential indicator of bridge performance. Specific recommendations for the factor of safety for earthquake and post-earthquake conditions are not given in many bridge codes, but typically the values for these cases should range somewhere between 1.1 and 1.5. There are several shortcomings in using the factor of safety as an indicator of performance under seismic loading and liquefaction, including: (1) it does not reflect the influence of earthquake frequency and ground motion amplification on seismic performance (Riemer et al., 1996) and (2) values less than or greater than one from pseudostatic slope stability analyses do not necessarily indicate inadequate or adequate performance because of the transient nature of the loading and deformation that occur.

Based on information available from the literature the conclusions below are drawn regarding performance requirements for bridges.

- The movement criteria cited are useful as a starting point for evaluating whether a bridge is likely to remain in a serviceable state after an earthquake. However, movement criteria should be established by a bridge engineer on a case by case basis, particularly where the interest is only in maintaining the repairability of the bridge or preventing collapse. In this research, the foundation movement limits adopted were 10 cm vertically and 10 cm laterally/horizontally.
- The factor of safety for stability of bridge foundations for earthquake loading conditions can only be used as an indicator of potential performance. It does not provide enough information to make definitive judgements about bridge performance under earthquake loading. The factor of safety is only a definitive indicator of

performance when it is less than one for post-earthquake conditions, which indicates there is a high potential for failure after shaking stops.

9.2.2 Factors and Phenomena Affecting Performance

The key factors in the design of ground improvement for liquefaction mitigation at existing highway bridges, as well as other structures, are the type, size, and location of the improved zone used for remediation. There are also a number of phenomena discussed in the literature that affect the performance of treated ground zones in liquefiable soils during and after earthquake shaking. These phenomena include: (1) excess pore water pressure migration, (2) ground motion amplification, (3) inertial force phasing, (4) dynamic component of liquefied soil pressure, (5) influence of a supported structure, and (6) lateral spreading forces.

Conclusions are given below concerning the impact of these factors and phenomena on improved ground zone design and performance based on information in the literature, particularly from physical modeling (i.e. – shaking table and centrifuge tests) and numerical modeling studies.

- There is clear evidence that the type, size, and location of ground improvement supporting a structure in liquefiable soils has a direct effect on the performance of that structure during shaking and liquefaction.
- Pore water pressure migration from untreated to treated zones will occur during and after shaking, particularly when the improved zone consists of densified soil. This migration causes an increase in pore pressures in the improved zone that reduces stability to some extent. The magnitude of the migration is dependent on things such as the soil permeability, compressibility, and loading conditions. No physical modeling test has been found where excessive settlement or failure of a structure occurred due to pore water pressure migration into a treated zone, but such a failure could possibly occur.
- Ground motions tend to be amplified by treated zones, particularly those extending completely through liquefiable deposits, resulting in higher supported structure accelerations and inertial forces.

- Data from some physical modeling studies indicate the phase difference between accelerations acting on a structure and an improved ground zone may vary depending on the specific problem conditions. It would be conservative to assume that these accelerations, and the associated inertial forces, are completely in-phase for simplified, pseudostatic type analyses.
- A dynamic component of liquefied soil pressure has been measured in full-scale tests on a retaining wall. Such a component has not been measured on an improved zone in liquefied soil that typically is not continuous or as rigid as a retaining wall. Although it would be conservative to include the dynamic component of pressure in design of improved zones, reducing the total pressure available for maintaining stability at some points during shaking, further verification of this pressure component would be desirable before adopting this practice.
- The presence of a structure has a clear influence on the behavior of the soil supporting it. A structure can produce dilative behavior in the soil underneath, resulting in increased soil strength and stiffness, but also increasing the potential for pore pressure migration. Increasing the width of a structure generally decreases its settlement and increasing its height relative to its width generally increases its settlement.
- Forces are exerted by liquefied and overlying non-liquefiable soil layers that undergo lateral spreading and move relative to a stationary structure. Such forces would likely be exerted on an improved zone and therefore should be estimated and included in design. The lateral spreading force exerted by the liquefied soil can be estimated by treating the material as a viscous fluid or a solid having a reduced modulus and using an appropriate analytical solution, such as a drag force calculation.

Analyses of the performance of improved ground zones supporting existing highway bridges, or other structures, should incorporate these factors and phenomena, as appropriate.

9.2.3 Simplified Methods for Predicting Performance

A simplified method of predicting the performance of an improved ground zone and supported structure was investigated. The method was based on the assumption that the important phenomena influencing the performance of the improved ground could be evaluated using a series of uncoupled analyses, with the results from one or more analyses being used in other analyses, as appropriate. The stability and movements of the improved ground and structure are evaluated using pseudostatic stability and deformation analyses performed with slope stability and static finite element programs, respectively (i.e. – UTEXAS3 and SAGE). Pseudostatic coefficients for the stability and deformation analyses are estimated from two-dimensional (2-D), equivalent-linear dynamic response analyses (i.e. – QUAD4M) of the system. The dynamic response analyses also provide an estimate of the shear strains and stresses that develop within the improved zone during shaking, as well as the factor of safety against liquefaction in the zone, allowing pore water pressures that develop due to cyclic loading to be estimated. Additional pore water pressures in the zone due to migration from the liquefied soils are estimated using transient seepage analyses (i.e. – MODFLOW). A dynamic component of liquefied soil pressure and lateral spreading forces were not considered in the proposed simplified method, but can be incorporated as additional forces and pressures within the pseudostatic stability and deformation analyses if desired.

The feasibility of using each of these uncoupled analyses for predicting a particular behavior associated with an improved zone within a liquefied soil deposit was investigated by analyzing some test cases. Based on the analytical results the conclusions below were reached about using these simplified analyses for predicting improved ground behavior.

- Dynamic Response Analyses: The ground accelerations and excess pore water pressures predicted in a treated zone can be highly dependent on the properties assumed in the equivalent linear response analysis for the adjacent loose, liquefiable sand. Using a shear modulus for the loose sand based on initial effective stresses and a modulus degradation curve, or assigning it a very low shear modulus representative of fully-liquefied soil, resulted in underprediction of the acceleration and pore water pressures measured in a dense sand zone in a centrifuge test. Predicted excess pore water pressures and ground accelerations were closer to the measured values when a reduced shear modulus was used for the loose sand that gave a first natural period for the entire loose-dense sand system close to the period of the input motion. This indicates that a peak response for an improved ground system will likely occur when the system degrades to the point that its first natural period is similar to the predominant period of the input motion, such that it reaches resonance. Using an equivalent linear dynamic response analysis would require systematically reducing the stiffness of the liquefiable soil until a peak response is obtained, which is time consuming and inconvenient.

- Seepage Analyses: Transient seepage analyses can potentially provide reasonable estimates of pore water pressures that develop in a densified zone due to migration and dissipation of pore water pressures from an adjacent liquefied zone. An analysis of pore water pressure migration into a dense zone from an adjacent liquefied zone using MODFLOW gave trends similar to those measured in a shaking table test both during and after shaking. However, the magnitudes of the predicted pore water pressures did not agree very well with the measured values until the coefficient of volume compressibilities for the loose and dense sand were adjusted. The magnitude of pore water pressures that develop in densified zones during and after shaking are dependent on the duration of the shaking and the properties of the soils, particularly the soil hydraulic conductivity and volume compressibility, which change throughout the process. For some cases where the densified zone hydraulic conductivity is high, its compressibility low, and the earthquake duration is long, the pore water pressures that develop in the zone during shaking due to migration can be estimated using a steady state seepage analysis, which provides an upper bound estimate of the pressures. Analyses show that pore water pressure migration into a three-dimensional (3-D) zone that is approximately equidimensional in plan can be underestimated using a 2-D seepage analysis.
- Stability and Deformation Analyses: Analyses performed for a bridge pier on a densified zone indicate that logical trends for the predicted stability safety factor and movements of the pier footing with changes in zone factors (i.e. - width, stiffness, strength, excess pore water pressures, and pseudostatic seismic coefficient of the zone) can be obtained from pseudostatic analyses with UTEXAS3 and SAGE. However, there are several difficulties associated with the pseudostatic method. including: (1) progressive accumulation of ground and structure movements during earthquake shaking are not handled, (2) solution becomes unstable when analyzing a case where there is transient failure of the improved zone and supported structure

during shaking, and (3) effects of the ground motion frequency on the deformations are not inherently included.

Methods could possibly be developed to overcome the shortcomings of the analyses described above. However, developing and using such adjustments would likely be time consuming and still not account for some of the interdependency of various factors controlling behavior. Therefore using a series of uncoupled analyses for evaluating improved ground performance does not appear feasible. However, these analyses do provide some general insights into improved ground behavior. Transient seepage analyses with MODFLOW are particularly useful for understanding the effect of using a 2-dimensional representation of a 3-dimensional seepage problem and could potentially be used to develop a factor for incrementing the pore water pressures obtained in 2-D coupled analyses to account for 3-D seepage effects.

9.2.4 Comprehensive Method for Predicting Performance

More comprehensive methods were investigated for evaluating the behavior of improved ground and a supported structure using a coupled analytical approach. Comprehensive methods considered were based on effective stress analyses that included some aspect of pore water pressure development, stress-strain softening, strength reduction, and groundwater flow in soil under dynamic loading conditions.

The computer code FLAC (Fast Lagrangian Analyses of Continua) was selected for performing these analyses because of its reported success in predicting the liquefiable soil and improved ground behavior, ability to incorporate a number of the key factors and phenomena affecting that behavior, flexibility for adding new formulations, and its growing acceptance by the geotechnical engineering profession. The FLAC code uses a partially-coupled solution to handle the mechanical equilibrium, pore pressure generation, and groundwater flow aspects of the liquefaction problem. Modifications were made to the liquefaction code to incorporate a hyperbolic type, non-linear stress-strain behavior for the soil based on a formulation by Pyke (1979) and an updated formulation for plastic volumetric strain by Byrne (1991), which is used for pore water pressure generation during shaking. This modified version of the liquefaction model is referred to as the Pyke-Byrne model.

The ability of FLAC with the Pyke-Byrne model to predict the behavior of liquefiable soils with and without improved ground zones and structures was evaluated by simulating a number of centrifuge tests performed by other researchers and one field case history involving earthquake shaking and liquefaction. The centrifuge models included: (1) a uniform medium dense sand layer; (2) model footing on a medium dense sand layer; (3) dense embankment on a loose sand deposit; (4) gravity retaining wall on a medium dense sand deposit; (5) adjacent, vertical zones of loose and dense sand; and (6) clayey sand embankment on a loose sand layer with and without ground improvement. Properties used for the soils in the FLAC simulations of the centrifuge tests were obtained directly from available laboratory test data or from FLAC simulations of those tests. "Typical" sand properties were used for the soils in the FLAC simulation of the field case history.

Comparisons were made of the predicted and measured time histories of pore water pressures, accelerations, and displacements for the centrifuge tests and field case. The conclusions below were drawn regarding the ability to predict the performance of improved ground and a supported structure in liquefiable soil using FLAC with the Pyke-Byrne model.

- Ground and structure accelerations may not be adequately predicted in all cases. In some cases there is a tendency to underpredict the peak acceleration at the top of an improved zone, embankment, or structure by as much as a factor of three.
- The development of excess pore water pressures beneath and immediately adjacent to structures during shaking is sometimes poorly predicted, with the peak pore water pressure overpredicted or underpredicted by a factor of three or more in some cases. On the other hand, the trends of excess pore pressure in the free-field and densified zones seem to be predicted fairly well. In many cases, with or without structures, the excess pore water pressures that develop during shaking dissipate faster than measured after shaking stops.
- Due to the deviation of the predicted accelerations and pore water pressures from measured values, these predicted values can likely only be used to get a sense of the trends in these responses as opposed to judging the performance of an improved ground and supported structure system.
- The trends and magnitudes of vertical and horizontal displacements of structures and supporting ground are predicted reasonably well and can be used to judge performance. The permanent deformations/displacements of a structure or

embankment supported in liquefiable soil, with or without improvement, can be predicted within a factor of about two, with the predicted values likely being anywhere from 0.5 to 2 times the actual values. The relative reduction in movements of a structure in liquefiable soils using different types of ground improvement can be predicted reasonably well.

On the basis of these conclusions, FLAC with the Pyke-Byrne model was used to investigate the feasibility of using different ground improvement methods for liquefaction remediation at a bridge pier and stub abutment supported on shallow foundations.

9.2.5 Ground Improvement for a Single Pier

The case evaluated consisted of a multicolumn pier supported on a shallow foundation having a length (transverse to the longitudinal axis of the bridge), width (parallel to the longitudinal axis of the bridge), and thickness of 8 meters, 4 meters, and 1 meter, respectively. The pier itself consisted of two, four-meter-high columns with a one-meter-high cap beam across their tops that applied a dead load of approximately 866 kN to the top of each pier column. The load carried by the pier was predominantly from the superstructure on each side of it. The superstructure spans were 21 meters in length, with one span having pinned connections at the top of the pier cap beam and the other span having roller bearings.

The bottom of the pier footing was founded one meter below the surface of a 3-meter-thick medium dense sand layer, which was underlain by an 8-m-thick loose sand layer followed by stiffer, non-liquefiable deposits. Both sand layers had a fine to medium gradation and were liquefiable. The groundwater level was at the ground surface, which was the top of the medium dense sand layer.

Performance of the pier footing with and without ground improvement was analyzed using a North-South component of horizontal acceleration from the 1995 Kobe earthquake measured on Port Island at 16 m depth scaled to 0.23g. This motion was applied to the base and sides of the 2-D finite difference grid representing a longitudinal cross-section through the soil profile and bridge. The pier columns and bridge superstructure were modeled using structural elements and the pier footing using grid elements having high stiffness.

Ground improvement schemes investigated for the pier included no improvement, densification under and around the pier using compaction grouting, cementation under and around the pier using chemical grouting, and containment of the soils under the pier using a pair of jet-grouted walls (one on each side of the pier footing). These methods were qualitatively judged to have high to moderate applicability to this case. Both the densified and chemically-grouted soils were modeled using a Mohr-Coulomb failure envelope and the non-linear stress-strain formulation by Pyke. The jet-grouted material was modeled using the Mohr-Coulomb failure envelope and a linear elastic-perfectly plastic stress-strain formulation. No pore water pressures above static levels were allowed to develop in the chemically-grouted or jet-grouted zones.

Results obtained from the FLAC analyses of the bridge pier included time histories of the pore water pressures, stresses, accelerations, and displacements at select locations of the grid. These results justify the conclusions below regarding ground improvement for the test problem and other cases having similar conditions.

- Bridge pier settlements can be reduced to a tolerable range of 5 to 10 cm by using an improved ground block under and around the pier footing created by densification using compaction grouting or cementation using chemical grouting. Tolerable settlements were obtained for densified zones having relative densities of 75 and 85 percent and grouted soil with an unconfined compressive strength of 300 kPa.
- The width of the treated zone needed to reduce pier settlements to the acceptable range is generally smaller for improved soil having higher strength and stiffness. A smaller width of treatment can likely be used for a chemically-grouted zone than for a densified zone. The width-to-depth ratio (distance treatment extends beyond the edge of the footing in the lateral direction divided by the depth of treatment below the footing), W/D, of the improved zone needed for chemical grouting in the test problem was on the order of 0.3 to 0.5, as opposed to 0.5 and 1.0 for densified zones having relative densities of 85 and 75 percent, respectively. In these analyses the treatment extended downward 10 meters from the footing bottom to the bottom of the liquefiable soils.
- As the width of the treated zone at the pier footing increases the settlement of the footing decreases. However, it appears that increasing the treated zone width beyond

a certain point does not result in any additional reduction in settlement that is significant, particularly for densified zones.

- A higher relative density for a densified zone results in a narrower width of treatment being required to reduce pier settlements to acceptable levels, but results in higher pore water pressures developing under the footing due to pore pressure migration from surrounding liquefied soils. Although no migration-induced failure was observed in the test problem evaluated with FLAC, post-shaking failures of structures supported on liquefiable soils have been documented in past earthquakes and may have been the result of pore pressure migration. FLAC analyses show that drains along the edge of a densified zone should reduce the pore water pressures that developed in it due to migration. Excess pore water pressure migration could be a potential problem for chemically-grouted zones, but insufficient information was available to model pore water pressure development and migration for chemically-grouted soil.
- Chemically-grouted zones having sufficient width and strength can reduce pier settlements down to a level close to zero. On the other hand, there will likely be a few centimeters (i.e. – on the order of 2 cm) of settlement for a pier on a densified zone, even if the width of treatment is large and the relative density high (i.e. – on the order of 85 percent).
- An improved ground zone below a pier footing that only partially penetrates the underlying liquefiable soils will reduce the ground motion that is transferred to the pier during the later parts of shaking due to pore pressure development and softening in the untreated zone. In the test problem a densified zone extending below the pier footing a distance equal to 90 percent of the liquefiable soil thickness had tolerable settlements and significantly reduced the pier accelerations during the later stages of shaking. However, partial treatment depths at a pier are not recommended due to the potential for additional settlements associated with densification of the untreated zone and horizontal movements caused by lateral spreading.
- Horizontal displacements of a pier footing supported on an improved zone that extends downward completely through the underlying liquefiable soils tend to follow similar trends as the vertical displacements with changes in the treated zone

properties and size. The accumulated back-and-forth horizontal displacement of the pier provided the most consistent trends. The predicted final horizontal displacements at the end of shaking were erratic. Final horizontal displacements are the values most readily comparable to tolerable horizontal movement criteria. Therefore, pier performance on improved zones can not be directly evaluated in terms of horizontal movements using FLAC and the Pyke-Byrne model at this time.

- Results of the FLAC analyses for jet-grouted walls installed on each side of a bridge pier footing were inconsistent because changes in conditions that should have produced smaller predicted settlements resulted in larger, unacceptable settlements. Further improvements in the analyses for this case are needed.

Based on the results of the parametric study performed, it appears that chemical grouting and densification by compaction grouting can be used to limit the movements of pier footings to acceptable levels. If the treatment widths required at the piers are similar to the bridge span lengths, then a continuous treatment zone may be required under the entire bridge. The conclusions drawn above were based on the results of 2-D analyses of the test problem. Although the magnitude of the predicted response might differ somewhat between 2-D and 3-D analyses, many of the trends seen in the 2-D analyses would likely also be seen in the 3-D analyses.

9.2.6 Ground Improvement for a Stub Abutment

Ground improvement was investigated for a stub abutment consisting of an L-shaped beam seat constructed of reinforced concrete and supported on a 2.4-m-wide (parallel to the longitudinal axis of the bridge) by 8-m-long (perpendicular to the longitudinal axis) spread footing at the top of a 6-m-high approach embankment constructed of dense, well-graded gravelly sand. Roller bearings were assumed to transfer the load from the superstructure beams to the beam seat. The stub abutment was evaluated in the 2-D FLAC analyses using a cross-section taken along the longitudinal axis of the bridge and approach embankment, with a concentrated load of 100 kN/m applied to the beam seat to represent the applied superstructure load.

Soil and groundwater conditions beneath the approach embankment and stub abutment were assumed to be the same as for the pier case, described above. The input motion used in the pier case was also used for the stub abutment case, with the motion being applied to the bottom and vertical sides of the finite difference grid. Due to the fact that the approach embankment was directly in contact with one of the vertical boundaries of the grid, where the input motion was applied, the model simulated a case where the liquefiable soil ends at some distance behind the stub abutment and the approach embankment comes into direct contact with stiff, non-liquefiable soil.

The effect of improvement of the liquefiable soils under the approach embankment on the performance of the stub abutment was evaluated for no improvement, densification by compaction grouting, and cementation by chemical grouting or jet grouting. In addition, use of a buttress fill placed against the toe and slope of the approach embankment was investigated. The densified, chemically-grouted and jet-grouted soils were modeled in the same way as described for the pier cases. The buttress fill was assumed to be composed of the same material as the approach embankment. Both the approach embankment and buttress fill were modeled with the Pyke-Byrne model, with the volumetric strain constants C_1 and C_2 taken as zero because both were located above the ground water table and little volumetric strain of the dense, gravelly sand was expected. The pore pressure at the top of the medium dense sand layer under the embankment was fixed to zero to avoid the complication of partially-saturated flow in the embankment material.

Time histories of pore water pressures, stresses, accelerations, and displacements were obtained at select locations in the analysis of the stub abutment with different improvement schemes. Based on the results of the analyses, the conclusions below were reached regarding ground improvement at the stub abutment for the test problem and other similar cases.

- Abutment movements can be reduced to the tolerable range of approximately 5 to 10 cm in the both vertical and horizontal directions using a densified zone created by compaction grouting or a cemented zone created using chemical or jet grouting. The improved zone must be properly positioned and have adequate size, strength, and stiffness to prevent deformations caused by three different mechanisms: (1) large shear deformations in the soils under and immediately outside the approach embankment, (2) localized slope stability failure at the abutment and embankment slope, and (3) pushing of the softened, untreated soils under the embankment towards the embankment slope. All of these mechanisms have the tendency to move the abutment laterally in the direction of the free-field, as well as cause some vertical displacement.

- The size and location of the improved ground zone needed to limit deformations to acceptable levels is dependent in part on the strength and stiffness properties of the improved material. As the strength and stiffness of the treated material increase, the size of the treated zone needed generally decreases. At a minimum, the treated zone in the liquefiable soils likely needs to extend laterally from beneath the stub abutment at the embankment crest to the embankment toe. In addition, the zone must extend downward through all of the liquefiable soil layers.
- For remediation using chemical grouting or densification by compaction grouting, the treated zone under the abutment and embankment slope will likely need to be extended behind the abutment and beyond the embankment toe some distance to provide additional resistance against the three mechanisms causing deformation. In the test problem a densified zone having a fairly high relative density (Dr of 85 percent) and large width (on the order of 40 to 50 meters) was required to reduce settlements to the tolerable range. The width of treatment required for a chemically grouted zone having an unconfined compressive strength of 300 kPa was on the order of 15 to 20 meters less than the densified zone, indicating smaller treatment widths can likely be used with chemical grouting than compaction grouting. The lower limits of abutment movement that can be achieved appear to be smaller for chemical grouting than densification by compaction grouting.
- The use of jet grouting to produce a continuous block of high-strength material will allow a smaller treated zone to be used under the abutment than possible for chemical grouting or densification by compaction grouting. However, even when the material in the jet-grouted block does not fail, excessive movements can be caused by permanent rotational displacements of the block during shaking. The block must be sufficiently large to limit these rotational movements, and the associated abutment and embankment displacements, to the tolerable range. In addition it must limit the movements due to the other three deformation mechanisms previously mentioned above.

- A narrow jet-grouted wall installed at the toe of an approach embankment and penetrating deep into non-liquefiable soil strata beneath liquefiable soils will not likely be successful in limiting abutment movements to tolerable levels due to potential overstressing and failure of the wall, ground movement toward the embankment slope, and densification of the untreated soils beneath the abutment.
- A limited size buttress fill placed against the slope of an approach embankment supported on thick, liquefiable deposits will not likely be successful by itself in reducing stub abutment movements to tolerable levels.
- Excess pore water pressure migration from untreated, liquefiable soils into a densified zone appears to be a potential concern for the portion of the zone in the vicinity of the approach embankment toe, when the width of treatment beyond the toe is not large. In the FLAC analyses performed, migration in this area resulted in an increase in the number of elements with fully mobilized strength between the abutment and embankment toe, with the pattern starting to approach a continuous failure surface. Installing vertical drains near the edge of the densified zone in the vicinity of the embankment toe could reduce the likelihood of this failure. Excess pore water pressure migration could also be a problem for chemically-grouted zones, but there was insufficient information available to model pore water pressure development and migration in the chemically-grouted soil.

Based on the results of the FLAC analyses for the test problem, it appears that a densified zone created by compaction grouting and a cemented zone created by chemical or jet grouting can be used to reduce stub abutment movements to acceptable levels. For compaction and chemical grouting remediation, the treated zone at the stub abutment may need to be combined with the treated zone at the adjacent bridge pier. Like the pier case, the conclusions drawn above for the stub abutment were based on the results of 2-D analyses of the test problem. No attempts were made to adjust the 2-D results for 3-D effects. However, the trends in the results seen in the 2-D analyses would likely also be seen in the 3-D case, although the magnitudes of response would probably vary somewhat.

9.3 Recommendations for Future Research

The research performed on the use of ground improvement for liquefaction remediation at existing highway bridges has provided additional insight and understanding regarding:

- Factors and phenomena that affect the behavior and performance of improved ground used for liquefaction remediation at existing structures;
- Potential use and limitations of simplified, uncoupled analyses for predicting the behavior of improved ground and a supported structure;
- Use and reliability of a more comprehensive, partially-coupled analytical method for evaluating an improved ground and supported structure system; and
- Behavior and performance of improved ground zones created in thick deposits of liquefiable soils for liquefaction remediation at an existing bridge pier and stub abutment supported on shallow foundations.

Future research can build on the work and results of this study to further develop understanding of the behavior of improved ground zones used for liquefaction remediation at existing highway bridges and other structures, as well as improved methods for analyzing and designing those zones. Some areas in which this research can be continued are discussed below.

- Phenomena Affecting Performance: Additional work is needed to verify whether a dynamic component of liquefied soil pressure acts on an improved ground zone during shaking. This research will likely require physical modeling studies using either centrifuge or shaking table tests. More information is needed on the phasing between inertial forces acting on an improved ground zone and the structure it supports.
- Simplified Analytical Methods: Given the difficulties and limitations of using a series of uncoupled analyses to predict the behavior of improved ground zones, further development of this approach does not appear justified. Greater benefit would likely be gained by using more comprehensive partially- or fully-coupled codes to perform additional parametric studies for the purpose of developing simple design

charts or guidelines for use in the preliminary design of improved ground zones for some "common" cases. Even with simplified design charts, a large portion of the ground improvement cases will likely have to be individually designed using partially- or fully-coupled analyses because of the great number of factors that affect the design and performance, and the difficulty of incorporating all of these factors in simple charts or guidelines.

- Comprehensive Analytical Methods: There is still considerable room for improvement in predicting the behavior of improved ground and supported structures within liquefiable deposits using partially- or fully-coupled analytical methods. Constitutive soil models should continue to be developed and improved for the purpose of predicting the behavior and performance of improved ground within liquefiable soil deposits with or without supported structures. Numerical modeling codes for performing dynamic analyses and implementing constitutive soil models should continue to be improved. Extensive verification of these models and codes should be conducted for a number of different cases relevant to the problems to be numerically simulated. Additional measured data from physical modeling studies and field case histories are needed for use in the verification work, as well as to increase understanding of improved ground behavior.
- Modeling Studies: Evaluation of improved ground zones supporting bridge abutments and piers on shallow foundations should be continued with a focus on different soil stratigraphies and earthquake motions. In particular, a soil stratigraphy having a non-liquefiable, low permeability surficial layer over the liquefiable soils should be evaluated. The modeling work should then shift to ground improvement for bridges supported on deep foundations, with particular emphasis on deep foundations that completely penetrate the liquefiable strata into underlying non-liquefiable soil layers. The effect of lateral spreading forces on improved ground zones should be investigated since this phenomenon is typically present at most bridge sites where liquefaction occurs. In addition the behavior of improved ground zones that support more than one bridge component, such as a stub abutment and an adjacent pier, needs to be investigated and understood along with the behavior of an entire ground improvement system for a bridge. Although most of this research can

be conducted using numerical modeling, some physical modeling will be necessary to provide measured data for verification of the numerical modeling methods. Additional work is needed to understand the limitations of using 2-dimensional analyses for modeling improvement cases that are 3-dimensional and, if possible, developing methods to adjust 2-dimensional analyses to account for 3-dimensional effects. Investigations should also be conducted regarding the width of ground improvement needed beyond the bridge foundations in the direction transverse to the longitudinal axis of the bridge and cases involving transverse and vertical components of earthquake motion. Pore pressure development and migration in chemically-grouted zones should also be implemented in the modeling.

These topics demonstrate the many challenges that still lie ahead in developing practical means of analyzing and designing ground improvement for liquefaction remediation. As progress continues to be made in these areas, the ability of the geotechnical engineer to provide effective and economical means of mitigating the damaging effects of earthquake-induced liquefaction on existing highway bridges, and other structures, will continue to improve.