

**GROUND IMPROVEMENT FOR LIQUEFACTION MITIGATION
AT EXISTING HIGHWAY BRIDGES**

by
Harry G. Cooke

Dissertation submitted to the Faculty of the
Virginia Polytechnic Institute and State University
in partial fulfillment of the requirements for the degree of

Doctor of Philosophy
in
Civil Engineering

Approved by:

Dr. James K. Mitchell, Chair
Dr. J. Michael Duncan
Dr. Mahendra P. Singh
Dr. Thomas L. Brandon
Dr. James R. Martin

July 13, 2000
Blacksburg, VA

Keywords: Liquefaction, remediation, ground improvement, bridges, numerical modeling, earthquakes

GROUND IMPROVEMENT FOR LIQUEFACTION MITIGATION AT EXISTING HIGHWAY BRIDGES

by

Harry G. Cooke

Dr. James K. Mitchell, Chair

Via Department of Civil and Environmental Engineering

ABSTRACT

The feasibility of using ground improvement at existing highway bridges to mitigate the risk of earthquake-induced liquefaction damage has been studied. The factors and phenomena governing the performance of the improved ground were identified and clarified. Potential analytical methods for predicting the treated ground performance were investigated and tested.

Key factors affecting improved ground performance are the type, size, and location of the treated ground. The improved ground behavior is influenced by excess pore water pressure migration, ground motion amplification, inertial force phasing, dynamic component of liquefied soil pressure, presence of a supported structure, and lateral spreading forces.

Simplified, uncoupled analytical methods were unable to predict the final performance of an improved ground zone and supported structure, but provided useful insights. Pseudostatic stability and deformation analyses can not successfully predict the final performance because of their inability to adequately account for the transient response. Equivalent-linear dynamic response analyses indicate that significant shear strains, pore water pressures and accelerations will develop in the improved ground when the treated-untreated soil system approaches resonance during shaking. Transient seepage analyses indicate that evaluating pore pressure migration into a three-dimensional improved zone using two-dimensional analyses can underestimate the pore pressures in the zone.

More comprehensive, partially-coupled analyses performed using the finite difference computer program FLAC provided better predictions of treated ground performance. These two-dimensional, dynamic analyses based on effective stresses incorporated pore pressure generation, non-linear stress-strain behavior, strength reduction, and groundwater flow. Permanent

movements of structures and improved soil zones were predicted within a factor of approximately two. Predictions of ground accelerations and pore water pressures were less accurate.

Dynamic analyses were performed with FLAC for an example bridge pier and stub abutment on an approach embankment supported on shallow foundations and underlain by thick, liquefiable soils with and without improved ground zones. Ground improvement that restricted movements of the pier and stub abutment to tolerable levels included improved zones of limited size extending completely through the underlying liquefiable soils and formed through densification by compaction grouting or cementation by chemical grouting or jet grouting. A buttress fill at the abutment was unsuccessful.

ACKNOWLEDGEMENTS

Throughout this research work, numerous people have provided support and encouragement. I would like to extend my sincere thanks to my advisor and Ph.D. committee chair, Dr. James K. Mitchell, for his enthusiasm, insights, and support for this work over the last six years. Likewise, I appreciate the interest and insights of my committee members: Drs. J. Michael Duncan, Mahendra P. Singh, Thomas L. Brandon, and James R. Martin. Other faculty members providing useful suggestions and information included Drs. George M. Filz, T. Kuppusamy, and Richard M. Barker.

In the process of performing the numerical analyses for this research using the computer program FLAC, several people shared their knowledge and offered advice. I would like to thank Dr. Peter M. Byrne of the University of British Columbia, Vancouver, Canada, for providing the soil model UBCSAND. Dr. Byrne and graduate student Michael Beaty of UBC also provided useful insights and suggestions regarding dynamic analyses with FLAC and UBCSAND. Dr. Peter Cundall and the Itasca Consulting Group offered valuable technical support for FLAC, helping to address issues regarding use of the program for this work.

My graduate work would not have been possible without the generous financial support of several organizations. Funding for this work was primarily provided by the Federal Highway Administration through the Multidisciplinary Center for Earthquake Engineering Research in Buffalo, New York, under FHWA Contract DTFH61-92-C-00106, Seismic Vulnerability of Existing Highway Construction. Support also came from a nine-month fellowship for earthquake hazards reduction research from the Federal Emergency Management Agency through the Earthquake Engineering Research Institute in Oakland, California. The Center for Geotechnical Practice and Research at Virginia Tech generously provided a one-semester fellowship to allow completion of my research.

During the course of my graduate studies at Virginia Tech, I had the good fortune of working and becoming friends with other students and visiting scholars. Graduate student Jennifer Schaeffer made contributions to this research through her Master's degree project and report on excess pore water pressure migration into improved ground zones. Numerous other students shared their knowledge and good humor with me. I particularly would like to thank

graduate students Chris and Diane Baxter and Carmine Polito for their encouragement and support from the beginning of my studies. I am also grateful to Dr. Moussa Wone for his understanding, advice, humor and friendship over the last three years.

My family has been a constant source of love, encouragement, and support throughout my graduate studies and my life. I am grateful to my mother Dorothy, sister Kathy, and brother-in-law Steve for their love, patience, and understanding over the last six years. Likewise, I am grateful to my father, Harry, who was a source of love, joy, and wisdom in my life and in our family.

I would like to express my gratitude to my family at Christ Episcopal Church in Blacksburg for the love and encouragement they have offered during my time at Virginia Tech. Above all, praise and thanks to a gracious and loving God, for the strength, hope, and mercy provided throughout my life.

TABLE OF CONTENTS

Abstract	ii
Acknowledgements	iv
List of Tables	xi
List of Figures	xiii
Chapter 1: Introduction	1
1.1 Overview of Problem	1
1.2 Objectives of Research.....	3
1.3 Scope of Study	3
1.4 Contents of Dissertation.....	4
Chapter 2: Background on Liquefaction-Induced Failure and Ground Improvement Relative to Bridges	6
2.1 Liquefaction-Induced Failure Mechanisms.....	6
2.1.1 Types of Failure Mechanisms	6
2.1.1.1 Lateral Spreading	6
2.1.1.2 Loss of Bearing Capacity and Settlement	7
2.1.1.3 Ground Oscillation	7
2.1.1.4 Flow Failure	7
2.1.2 Implications for Bridges.....	8
2.2 Remediation Using Ground Improvement.....	8
2.2.1 Improvement Categories and Methods	8
2.2.2 Applicability to Different Bridge Types	8
2.2.2.1 Bridge Configurations Considered.....	9
2.2.2.2 Assessment of Applicability.....	10
2.3 Focus of Research	10
Chapter 3: Literature Review on Observation and Prediction of Improved Ground Performance	25
3.1 Introduction	25
3.2 Improved Ground Performance from Previous Studies	25
3.2.1 Abutment/Embankment Performance	25
3.2.1.1 Treatment Size and Location.....	25
3.2.1.2 Treatment Type	28
3.2.1.3 Effects of Non-Liquefiable Surface Layer	31
3.2.2. Shallow Foundation Performance	33
3.2.2.1 Treatment Width	33
3.2.2.2 Treatment Depth.....	34
3.3 Numerical Modeling of Liquefaction and Ground Behavior Under Seismic Loading	35
3.3.1 Overview of Methods.....	35
3.3.2 Uncoupled Analyses.....	36

3.3.3 Coupled Analyses.....	37
3.4 Issues Requiring Further Investigation	40
Chapter 4: Requirements and Phenomena Influencing Performance	56
4.1 Overview	56
4.2 Performance Requirements	56
4.2.1 Movement Criteria	57
4.2.2 Stability Criteria	58
4.3 Phenomena Affecting Performance	59
4.3.1 Pore Water Pressure Migration	59
4.3.2 Ground Motion Amplification	60
4.3.3 Inertial Force Phasing.....	60
4.3.4 Dynamic Fluid Pressure	61
4.3.5 Influence of a Structure	62
4.3.6 Lateral Spreading Forces.....	62
4.4 Implications for Design and Research	63
Chapter 5: Assessment of a Simplified Method for Predicting Performance	71
5.1 Overview	71
5.2 Dynamic Ground Response.....	73
5.2.1 Centrifuge Test Details.....	73
5.2.2 QUAD4M Modeling and Results.....	74
5.2.3 Implications for Use of QUAD4M.....	79
5.3 Seepage Analyses.....	80
5.3.1 Analytical Approach	80
5.3.2 Verification of Method.....	81
5.3.3 Observations Regarding Migration and Dissipation	83
5.3.4 Implications for Design and Analysis	85
5.4 Stability and Deformation Analyses	86
5.4.1 Test Problem	86
5.4.2 Pseudostatic Stability Analyses.....	88
5.4.3 Pseudostatic Deformation Analyses.....	89
5.4.4 Results of Analyses	91
5.4.5 Assessment of Pseudostatic Approach.....	94
5.5 Implications for Prediction of Performance	94
Chapter 6: Assessment of a Comprehensive Method for Predicting Performance	134
6.1 Overview	134
6.2 Selection of Numerical Modeling Tools	135
6.2.1 Computer Code	135
6.2.2 Soil Models	136
6.2.2.1 First Code Modification	138
6.2.2.2 Second Code Modification.....	139
6.2.2.3 UBCSAND.....	141
6.2.2.4 Selected Model.....	142
6.2.3 Simplifications and Adjustments	143

6.3 Calibration of Model	144
6.3.1 Nevada Sand.....	145
6.3.1.1 General Properties	145
6.3.1.2 Strength Properties	146
6.3.1.3 Maximum Shear Modulus.....	146
6.3.1.4 Volumetric Strain Constants	146
6.3.2 Typical Sand.....	148
6.3.2.1 Strength	149
6.3.2.2 Maximum Shear Modulus.....	149
6.3.2.3 Volumetric Strain Constants	149
6.4 Verification Analyses.....	151
6.4.1 Centrifuge Tests	151
6.4.2 Field Case History.....	153
6.4.3 Results	155
6.4.4 Assessment of Soil Model.....	158
6.5 Implications for Model Use.....	160
Chapter 7: Study of Ground Improvement for a Single Pier	210
7.1 Overview	210
7.2 Case Evaluated	211
7.2.1 Pier Configuration and Loads	211
7.2.2 Soil Profile	213
7.2.3 Earthquake Motion.....	213
7.2.4 Ground Improvement Types and Factors.....	215
7.2.5 Grid and Boundary Conditions	216
7.2.6 2-Dimensional versus 3-Dimensional	217
7.3 No Improvement	217
7.3.1 Single Pier	217
7.3.2 Multiple Piers	218
7.3.3 Implications	219
7.4 Densification	219
7.4.1 Properties and Factors	219
7.4.2 Effects on Response and Performance.....	220
7.4.2.1 General	220
7.4.2.2 Treatment Width and Degree of Densification	221
7.4.2.3 Treatment Depth.....	224
7.4.2.4 Excess Pore Water Pressure Migration	225
7.4.3 Implications for Use	228
7.5 Chemical Grouting	230
7.5.1 Properties and Factors	230
7.5.2 Effects on Response and Performance.....	232
7.5.2.1 General	232
7.5.2.2 Treatment Width	233
7.5.2.3 Extended Duration.....	234
7.5.3 Implications for Use	235

7.6	Jet Grouting.....	235
7.6.1	Properties and Factors.....	235
7.6.2	Analyses and Results.....	237
7.6.2.1	Effect of Wall Spacing.....	237
7.6.2.2	No Footing Case.....	239
7.6.3	Implications for Use.....	240
7.7	Insights Regarding Remediation at Piers.....	240
Chapter 8: Study of Ground Improvement for a Stub Abutment.....		273
8.1	Overview.....	273
8.2	Case Evaluated.....	274
8.2.1	Stub Abutment Configuration and Loads.....	274
8.2.2	Soil Profile.....	275
8.2.3	Earthquake Motion.....	275
8.2.4	Ground Improvement Types and Factors.....	276
8.2.5	Grid and Boundary Conditions.....	276
8.2.6	2-Dimensional versus 3-Dimensional.....	277
8.3	No Improvement.....	278
8.3.1	Observed Behavior.....	278
8.3.2	Implications.....	280
8.4	Densification.....	280
8.4.1	Properties and Factors.....	280
8.4.2	Effects on Response and Performance.....	281
8.4.2.1	General.....	281
8.4.2.2	Location, Width, and Degree of Densification.....	282
8.4.2.3	Excess Pore Water Pressure Migration.....	285
8.4.3	Implications for Use.....	287
8.5	Chemical Grouting.....	288
8.5.1	Properties and Factors.....	288
8.5.2	Effects on Performance and Response.....	290
8.5.2.1	General Effects.....	290
8.5.2.2	Treatment Location and Width.....	291
8.5.2.3	Treatment Depth.....	293
8.5.2.4	Extended Duration.....	294
8.5.3	Implications for Use.....	295
8.6	Jet Grouting.....	296
8.6.1	Properties and Factors.....	296
8.6.2	Effects on Performance and Response.....	297
8.6.3	Implications for Use.....	299
8.7	Buttress Fill.....	301
8.7.1	Properties and Factors.....	301
8.7.2	Effects on Performance and Response.....	302
8.7.3	Implications for Use.....	302
8.8	Insights Regarding Remediation at Stub Abutments.....	303

Chapter 9: Summary, Conclusions and Recommendations.....	341
9.1 Introduction	341
9.2 Summary and Conclusions.....	344
9.2.1 Performance Requirements	344
9.2.2 Factors and Phenomena Affecting Performance	346
9.2.3 Simplified Methods for Predicting Performance	347
9.2.4 Comprehensive Method for Predicting Performance.....	350
9.2.5 Ground Improvement for a Single Pier	352
9.2.6 Ground Improvement for a Stub Abutment	355
9.3 Recommendations for Future Research	359
References	362
Vitae.....	372

LIST OF TABLES

Table 2.1	Categories of Ground Improvement Methods for Liquefaction Mitigation at Existing Highway Bridges	12
Table 2.2	Summary of Ground Improvement Methods for Liquefaction Remediation at Existing Highway Bridges.....	13
Table 3.1	Comparison of Improved and Unimproved Embankment Response for Adalier et al. (1998) Centrifuge Tests	42
Table 4.1	Distortion and Movement Criteria for Bridges	64
Table 4.2	Past Studies of Phenomena Affecting Improved Ground Performance	65
Table 5.1	Properties Used for Loose and Dense Sands in QUAD4M Analyses.....	96
Table 5.2	Shear Modulus Reduction Factor, G/G_{max} , and Damping Ratio for Sand at Different Strain Levels from Idriss (1990).....	97
Table 5.3	Soil Properties Reported by Iai et al. (1988) for Model Study	97
Table 5.4	Soil Properties Used for Modeling Study (after Schaeffer, 1998)	97
Table 5.5	Forces and Moments Acting at Bottom of Pier Column Based on AASHTO Group I Service Loads	98
Table 5.6	Forces and Moments Acting on Stub Abutment Based on AASHTO Group I Service Loads.....	98
Table 5.7	Forces and Moments Acting at Bottom of Pier Column Based on AASHTO Group VII Service Loads for Primary Loading in Longitudinal Direction.....	99
Table 5.8	Equivalent Forces or Stresses at Bottom of Footing for Use in Two-Dimensional Pseudostatic Analyses of Pier Subjected to Longitudinal Seismic Loading	99
Table 5.9	Cases Evaluated and Parameter Values Used in Pseudostatic Stability Analyses.....	100
Table 5.10	Cases Evaluated and Parameter Values Used in Pseudostatic Deformation Analyses.....	101

Table 6.1	Properties of Nevada Sand	162
Table 6.2	Peak Friction Angles and Shear Modulus Numbers Used for Nevada Sand Based on VELACS Test Data	162
Table 6.3	Volumetric Strain Constants for Nevada Sand Obtained from Simple Shear and Centrifuge Simulations.....	162
Table 6.4	Shear Modulus Number, K_{2max} , for Clean Sand.....	163
Table 6.5	Volumetric Strain Constants for “Typical” Sand.....	163
Table 6.6	Summary of Centrifuge Tests and Field Case Histories Used for Verification.....	164
Table 6.7	Parameter Values Used in Simulation of Centrifuge Tests.....	165
Table 6.8	Parameter Values Used in FLAC Simulation of Wildlife Site	166
Table 6.9	Comments on Predicted Results from FLAC Simulations in Comparison to Measured Response	167
Table 6.10	Comparison of Predicted and Measured Movements for Verification Cases.....	168
Table 7.1	Properties of Structural Members Used in Pier Analyses	243
Table 7.2	Properties Assigned to Soil Strata in Parametric Study	243
Table 7.3	Cases Evaluated and Properties Assumed for Densified Zone in Parametric Study	244
Table 7.4	Long-term Analyses Performed for Pier on Densified Zone	245
Table 7.5	Properties Assigned to Grouted Sand in Parametric Study.....	245
Table 7.6	Properties Assigned to Jet-Grouted Material in Parametric Study	246
Table 8.1	Properties Assigned to Approach Embankment Soil	307
Table 8.2	Summary of Cases Evaluated for Densified Zone at Stub Abutment.....	308
Table 8.3	Properties Assigned to Dense Sand Layer at Base.....	309
Table 8.4	Summary of Cases Evaluated for Chemically-Grouted Zone at Stub Abutment.....	310

LIST OF FIGURES

Figure 1.1	Potential Effects of Liquefaction on Highway Bridge	5
Figure 2.1	Lateral Spreading Mechanism.....	17
Figure 2.2	Bearing Capacity Failure.....	17
Figure 2.3	Ground Oscillation Phenomena	18
Figure 2.4	Flow Failure	18
Figure 2.5	Some Typical Bridge Abutments	19
Figure 2.6	Some Typical Bridge Piers.....	20
Figure 2.7	Ground Improvement at Bridge Pier.....	21
Figure 2.8	Ground Improvement at Stub Abutment.....	22
Figure 2.9	Mixed-in-Place or Jet-Grouted Wall at Bridge Pier.....	23
Figure 2.10	Buttress Fill at Stub Abutment.....	24
Figure 3.1	Soil Profile and Properties for Highway I-57 Bridge Used in Study by Riemer et al.	43
Figure 3.2	Impact of Treated Zone Size on Predicted Performance of Highway I-57 Bridge Abutment.....	44
Figure 3.3	Impact of Treated Zone Location on Predicted Performance of Highway I-57 Bridge Abutment	45
Figure 3.4	Shaking Table Test of Embankment With and Without Ground Improvement by Yanagihara et al.....	46
Figure 3.5	Improvement Schemes for Embankment Modeled in Centrifuge Tests by Adalier.....	47
Figure 3.6	Ground Deformations (in Prototype Scale) Observed in Centrifuge Tests by Adalier.....	48

Figure 3.7	Schematic of Centrifuge Test with Improved (Dense) Zone by Balakrishnan et al.	50
Figure 3.8	Sections Showing Structure on Densified Zone Modeled in Shaking Table Tests by Hatanaka et al.	51
Figure 3.9	Relationship Between Settlement and Width Ratios for Improved Zone in Shaking Table Tests	51
Figure 3.10	Relationship Between Settlement Ratio and Ratio of Treatment Width Beyond Structure to Treatment Depth for Improved Zone	52
Figure 3.11	Relationship Between Settlement Ratio and Height-to-Width Ratio of Structure on Unimproved Soil.....	52
Figure 3.12	Centrifuge Model of Footing on Densified Zone Tested by Liu and Dobry.....	53
Figure 3.13	Foundation Settlement versus Normalized Compaction Depth for Centrifuge Tests	54
Figure 3.14	Normalized Footing Acceleration versus Normalized Compaction Depth for Centrifuge Test	55
Figure 4.1	Phenomena Affecting Improved Ground Performance.....	69
Figure 5.1	Adjacent Loose/Dense Sand Zones in Centrifuge Tests by Adalier	102
Figure 5.2	Coarse Mesh Used in QUAD4M Analyses.....	102
Figure 5.3	Fine Mesh Used in QUAD4M Analyses.....	103
Figure 5.4	Typical Relationship Between Residual Excess Pore Pressure Ratio and Factor of Safety Against Liquefaction for Sand.....	103
Figure 5.5	Variation of Predicted Liquefaction Safety Factor for Dense Zone with System Period for Different Side Boundaries	104
Figure 5.6	Variation of Predicted Shear Strain for Dense Zone with System Period for Different Side Boundaries	104
Figure 5.7	Variation of Predicted Excess Pore Pressure Ratio at CP7 in Dense Zone with System Period for Different Side Boundaries	105
Figure 5.8	Variation of Predicted Excess Pore Pressure Ratio at CP8 in Dense Zone with System Period for Different Side Boundaries	105

Figure 5.9	Variation of Predicted Excess Pore Pressure Ratio at CP2 in Dense Zone with System Period for Different Side Boundaries	106
Figure 5.10	Variation of Predicted Excess Pore Pressure Ratio at CP9 in Dense Zone with System Period for Different Side Boundaries	106
Figure 5.11	Variation of Predicted Excess Pore Pressure Ratio at CP10 in Dense Zone with System Period for Different Side Boudaries	107
Figure 5.12	Variation of Predicted Peak X-Acceleration at CA4 in Dense Zone with System Period for Different Side Boundaries	107
Figure 5.13	Variation of Predicted Peak X-Acceleration at CA7 in Dense Zone with System Period for Different Side Boundaries	108
Figure 5.14	Variation of Predicted Peak X-Acceleration at CA5 in Dense Zone with System Period for Different Side Boundaries	108
Figure 5.15	Effect of Mesh Gradation on Predicted Liquefaction Safety Factor for Dense Zone.....	109
Figure 5.16	Effect of Mesh Gradation on Predicted Excess Pore Pressure Ratio at CP9 in Dense Zone.....	109
Figure 5.17	Variation of Predicted Liquefaction Safety Factor for Dense Zone with System Period for Different Damping Ratios	110
Figure 5.18	Variation of Predicted Shear Strain for Dense Zone with System Period for Different Damping Ratios	110
Figure 5.19	Variation of Predicted Excess Pore Pressure Ratio at CP9 in Dense Zone with System Period for Different Damping Ratios	111
Figure 5.20	Variation of Predicted Peak X-Acceleration at CA7 in Dense Zone with System Period for Different Damping Ratios	111
Figure 5.21	Variation of Predicted Liquefaction Safety Factor for Dense Zone with System Period for Different Pore Fluids	112
Figure 5.22	Variation of Predicted Shear Strain for Dense Zone with System Period for Different Pore Fluids	112
Figure 5.23	Variation of Predicted Excess Pore Pressure Ratio at CP9 in Dense Zone with System Period for Different Pore Fluids.....	113

Figure 5.24	Variation of Predicted Peak X-Acceleration at CA-7 in Dense Zone with System Period for Different Pore Fluids	113
Figure 5.25	Variation of Predicted Liquefaction Safety Factor for Dense Zone with System Period for Different Input Motion Periods	114
Figure 5.26	Variation of Predicted Shear Strain for Dense Zone with System Period for Different Input Motion Periods	114
Figure 5.27	Variation of Predicted Excess Pore Pressure Ratio at CP9 in Dense Zone with System Period for Different Input Motion Periods	115
Figure 5.28	Variation of Predicted Peak X-Acceleration at CA7 in Dense Zone with System Period for Different Input Motion Periods	115
Figure 5.29	Combined Migration and Dissipation Analysis	116
Figure 5.30	Example of Excess Pore Pressure Ratio vs. Time Plot for Point A in Dense Sand from 2D Combined Migration and Dissipation Analysis.....	117
Figure 5.31	Cross-section of Shaking Table Test by Iai et al.....	117
Figure 5.32	Migration and Dissipation Analysis of Shaking Table Tests by Iai et al.....	118
Figure 5.33	Densified Zone Configuration Used in Study of Pore Pressure Migration and Dissipation.....	119
Figure 5.34	Predicted Excess Pore Pressure Ratios at Two Points in Densified Block	120
Figure 5.35	Elevation View of Bridge Used for Test Problem	121
Figure 5.36	Plan and Elevation Views of Bridge Stub Abutment for Test Problem.....	122
Figure 5.37	Plan and Section Views of Bridge Pier for Test Problem.....	123
Figure 5.38	Improved Ground Zone Used at Bridge Pier	124
Figure 5.39	Improved Zone at Bridge Stub Abutment.....	125
Figure 5.40	Configuration for Pseudostatic Stability Analysis of Bridge Pier Footing	126
Figure 5.41	Finite Element Mesh and Boundary Conditions Used in Pseudostatic Deformation Analyses for Densification to 10 Meters Outside Footing	127
Figure 5.42	Example of Critical Failure Surface for Pseudostatic Stability Analysis of Bridge Pier Footing	128

Figure 5.43	Typical Direction of Displacements Observed in Pseudostatic Deformation Analyses.....	129
Figure 5.44	Effect of Soil Strength on Stability Safety Factor for Pier Footing on Densified Zone.....	130
Figure 5.45	Effect of Soil Stiffness on Predicted Maximum Displacements of Pier Footing on Densified Zone.....	130
Figure 5.46	Effect of Width/Depth Ratio of Dense Zone on Predicted Stability Safety Factor for Pier Footing.....	131
Figure 5.47	Effect of Width/Depth Ratio of Dense Zone on Predicted Maximum Displacements of Pier Footing.....	131
Figure 5.48	Effect of Seismic Coefficient on Predicted Stability Safety Factor for Pier Footing on Densified Zone.....	132
Figure 5.49	Effect of Seismic Coefficient on Predicted Maximum Displacements of Pier Footing on Densified Zone.....	132
Figure 5.50	Effect of Excess Pore Pressure Ratio in Dense Zone on Predicted Stability Safety Factor for Pier Footing.....	133
Figure 5.51	Effect of Excess Pore Pressure Ratio in Dense Zone on Predicted Maximum Displacements of Pier Footing.....	133
Figure 6.1	Grain Size Distribution Curve for Nevada Sand.....	169
Figure 6.2	Single Element Model in FLAC for Simulating Cyclic Simple Shear Test.....	169
Figure 6.3	Predicted Response from FLAC for Cyclic Simple Shear Test on Nevada Sand at $Dr = 40\%$, VELACS Test No. CSS 40-09.....	170
Figure 6.4	Simplified Base Curve Recommended for Calculation of Cyclic Resistance Ratio (CRR) from SPT Data (after NCEER, 1997).....	171
Figure 6.5	Uniform Medium Dense Sand Layer in Centrifuge Test by Liu.....	172
Figure 6.6	Footing on Medium Dense Sand Layer in Centrifuge Test by Liu.....	172
Figure 6.7	Dense Sand Embankment on Loose Sand Foundation in Centrifuge Test by Adalier.....	173
Figure 6.8	Submerged Gravity Block Wall with Surface Load on Medium Dense Sand Layer in Centrifuge Test by Zeng.....	174

Figure 6.9	Adjacent Loose/Dense Sand Zones in Centrifuge Test by Adalier.....	175
Figure 6.10	Embankment on Loose Sand Layer with Densification Improvement in Centrifuge Test by Adalier.....	176
Figure 6.11	Mesh Used in FLAC Analysis of Centrifuge Test on Uniform, Medium Dense Sand Layer.....	177
Figure 6.12	Mesh Used in FLAC Analysis of Centrifuge Test of Footing on Medium Dense Sand Layer.....	177
Figure 6.13	Mesh Used in FLAC Analysis of Centrifuge Test of Dense Sand Embankment on Loose Sand Foundation.....	178
Figure 6.14	Mesh Used in FLAC Analysis of Centrifuge Test of Submerged Gravity Block Wall with Surface Load on Medium Dense Sand Layer	179
Figure 6.15	Mesh Used in FLAC Analysis of Centrifuge Test on Adjacent Loose/Dense Sand Zones	180
Figure 6.16	Mesh Used in FLAC Analysis of Centrifuge Test of Embankment on Loose Sand Layer with Densification Improvement.....	181
Figure 6.17	Plan of Sand Boils, Lateral Spreading, and Cracks at Wildlife Liquefaction Array After 1987 Superstition Hills Earthquake.....	182
Figure 6.18	Section A-A' Showing Soil Profile and Instrumentation at Wildlife Liquefaction Array	183
Figure 6.19	Mesh Used in FLAC Analysis of Centrifuge Test of Wildlife Site During 1987 Superstition Hills Earthquake.....	184
Figure 6.20	Actual and FLAC Base Acceleration Records for Uniform, Medium Dense Sand Layer in Centrifuge Test	185
Figure 6.21	Measured vs. Predicted X-Acceleration at Ah-3 for Uniform, Medium Dense Sand Layer in Centrifuge Test	185
Figure 6.22	Predicted vs. Measured X-Acceleration at Ah-5 for Uniform, Medium Dense Sand in Centrifuge Test.....	186
Figure 6.23	Predicted vs. Measured Excess Pore Pressure at Pi for Uniform, Medium Dense Sand in Centrifuge Test.....	186
Figure 6.24	Predicted vs. Measured Excess Pore Pressure at Pa for Uniform, Medium Dense Sand in Centrifuge Test.....	187

Figure 6.25	Predicted vs. Measured Vertical Displacement at LVDT_S for Uniform Medium Dense Sand in Centrifuge Test	187
Figure 6.26	Actual and FLAC Input Base Acceleration Records for Footing on Medium Dense Sand Layer in Centrifuge Test.....	188
Figure 6.27	Predicted vs. Measured X-Acceleration at Ahs for Footing on Medium Dense Sand Layer in Centrifuge Test	188
Figure 6.28	Predicted vs. Measured X-Acceleration at Ah4 for Footing on Medium Dense Sand Layer in Centrifuge Test	189
Figure 6.29	Predicted vs. Measured Excess Pore Pressure at PF2 for Footing on Medium Dense Sand Layer in Centrifuge Test.....	189
Figure 6.30	Predicted vs. Measured Excess Pore Pressure at PC2 for Footing on Medium Dense Sand Layer in Centrifuge Test.....	190
Figure 6.31	Predicted vs. Measured Vertical Displacement at LVDT_I for Footing on Medium Dense Sand Layer in Centrifuge Test.....	190
Figure 6.32	Actual and FLAC Input Base Acceleration Records for Dense Sand Embankment on Loose Sand Layer in Centrifuge Test	191
Figure 6.33	Predicted and Measured X-Acceleration at A5 for Dense Sand Embankment on Loose Sand Layer in Centrifuge Test.....	191
Figure 6.34	Predicted and Measured X-Acceleration at A8 for Dense Sand Embankment on Loose Sand Layer in Centrifuge Test.....	192
Figure 6.35	Predicted and Measured Excess Pore Pressure at P5 for Dense Sand Embankment on Loose Sand Layer in Centrifuge Test	192
Figure 6.36	Predicted and Measured Excess Pore Pressure at P3 for Dense Sand Embankment on Loose Sand Layer in Centrifuge Test	193
Figure 6.37	Predicted and Measured Vertical Displacement at L1 for Dense Sand Embankment on Loose Sand Layer in Centrifuge Test	193
Figure 6.38	Actual and FLAC Input Base X-Acceleration Record for Submerged Gravity Retaining Wall on Medium Dense Sand Layer in Centrifuge Test	194
Figure 6.39	Actual and FLAC Input Base Y-Acceleration Record for Submerged Gravity Retaining Wall on Medium Dense Sand Layer in Centrifuge Test	194

Figure 6.40	Predicted and Measured X-Acceleration at ACC8 for Submerged Gravity Retaining Wall on Medium Dense Sand Layer in Centrifuge Test.....	195
Figure 6.41	Predicted and Measured Excess Pore Pressure at PPT2 for Submerged Gravity Retaining Wall on Medium Dense Sand Layer in Centrifuge Test	195
Figure 6.42	Predicted and Measured Excess Pore Pressure at PPT5 for Submerged Gravity Retaining Wall on Medium Dense Sand Layer	196
Figure 6.43	Predicted and Measured X-Displacement at LVDT1 for Submerged Gravity Retaining Wall on Medium Dense Sand Layer in Centrifuge Test.....	196
Figure 6.44	Horizontal (X) Acceleration Measured in North-South Direction at SM1 at Wildlife Site and Input in FLAC Analysis.....	197
Figure 6.45	Vertical (Y) Acceleration Measured at SM-1 at Wildlife Site and Input in FLAC Analysis.....	197
Figure 6.46	Predicted and Measured Horizontal (X) Acceleration in North-South Direction at SM2 at Wildlife Site.....	198
Figure 6.47	Predicted and Measured Vertical (Y) Acceleration at SM2 at Wildlife Site ...	198
Figure 6.48	Predicted and Measured Excess Pore Pressure at P3 at Wildlife Site.....	199
Figure 6.49	Predicted and Measured Excess Pore Pressure at P2 at Wildlife Site.....	199
Figure 6.50	Predicted Horizontal (X) Displacement of Ground Surface at Instrument Array at Wildlife Site	200
Figure 6.51	Actual and FLAC Input Base Acceleration Records for Adjacent Loose-Dense Sand Zones in Centrifuge Test	201
Figure 6.52	Predicted and Measured X-Acceleration at A5 for Adjacent Loose-Dense Sand Zones in Centrifuge Test.....	201
Figure 6.53	Predicted vs. Measured X-Acceleration at A7 for Adjacent Loose-Dense Sand Zones in Centrifuge Test.....	202
Figure 6.54	Predicted vs. Measured Excess Pore Pressure at P7 for Adjacent Loose-Dense Sand Zones in Centrifuge Test.....	202
Figure 6.55	Predicted vs. Measured Excess Pore Pressure at P9 for Adjacent Loose-Dense Sand Zones in Centrifuge Test.....	203

Figure 6.56	Predicted vs. Measured Vertical Displacement at L4 for Adjacent Loose-Dense Sand Zones in Centrifuge Test	203
Figure 6.57	Actual and FLAC Input Base Acceleration Records for Embankment on Loose Sand Layer with Dense Zones in Centrifuge.....	204
Figure 6.58	Predicted and Measured X-Acceleration at A6 for Embankment on Loose Sand Layer with Dense Zones in Centrifuge Test.....	204
Figure 6.59	Predicted and Measured X-Acceleration at A11 for Embankment on Loose Sand Layer with Dense Zones in Centrifuge Test.....	205
Figure 6.60	Predicted and Measured Excess Pore Pressure at P5 for Embankment on Loose Sand Layer with Dense Zones in Centrifuge Test.....	205
Figure 6.61	Predicted and Measured Excess Pore Pressures at P6 for Embankment on Loose Sand Layer with Dense Zones in Centrifuge Test.....	206
Figure 6.62	Predicted and Measured Vertical Displacement at L3 for Embankment on Loose Sand Layer with Dense Zones in Centrifuge Test.....	206
Figure 6.63	Predicted Excess Pore Pressure at Pa for Uniform Medium Dense Sand in Centrifuge Test Using Re-Solidification Concept	207
Figure 6.64	Predicted Excess Pore Pressure at P7 for Adjacent Loose-Dense Sand Zones in Centrifuge Test Using Re-Solidification Concept.....	207
Figure 6.65	Predicted and Measured Crest Settlements for Embankment on Loose Sand with Different Improvements for Centrifuge Test by Adalier (1996).....	208
Figure 6.66	Predicted and Measured Total Settlements for Crest of Embankment Accumulated Over Three Shaking Events in Centrifuge Test by Adalier (1996)	209
Figure 6.67	Predicted and Measured Maximum Horizontal Displacement Under Toe of Embankment Accumulated Over Three Shaking Events in Centrifuge by Adalier (1996)	209
Figure 7.1	Representation of Pier, Superstructure, and Footing in FLAC Analyses.....	247
Figure 7.2	Soil Profile Used in SHAKE Analyses to Obtain Acceleration Records at 11-m Depth for Loma Prieta and Saguenay Earthquakes	248
Figure 7.3	Normalized Response Spectra for Horizontal Acceleration Records Obtained Using Four Different Earthquake Motions.....	249

Figure 7.4	Corrected and Scaled Horizontal Motion Records from Port Island Array at 16-m Depth During 1995 Kobe Earthquake Used as Base Input.....	250
Figure 7.5	Grid Used for FLAC Analysis of Pier in Parametric Study.....	251
Figure 7.6	Expanded Section A-A' of Grid Used for FLAC Analysis of Pier.....	252
Figure 7.7	Vertical Displacement of Footing vs. Time for No Improvement Case	253
Figure 7.8	Relative X-Displacement Between Footing and Base of Grid vs. Time for No Improvement Case.....	253
Figure 7.9	Excess Pore Water Pressure Ratio vs. Time for No Improvement	254
Figure 7.10	X-Velocity at Base of Grid and Top of Footing vs. Time for No Improvement.....	254
Figure 7.11	Dimensions Used for Calculating Width-to-Depth (W/D) and Depth-to-Thickness (D/H) Ratios for Improved Zones.....	255
Figure 7.12	Vertical Displacement of Footing vs. Time for Densified Zone with W/D = 1, D/H = 1, and Dr = 75%.....	256
Figure 7.13	Relative X-Displacement Between Footing and Base of Grid vs. Time for Densified Zone with W/D = 1, D/H = 1, and Dr = 75%	256
Figure 7.14	Excess Pore Water Pressure Ratio vs. Time for Densified Zone with W/D = 1, D/H = 1, and Dr = 75%	257
Figure 7.15	X-Velocity of Footing and Grid Base vs. Time for Densified Zone with W/D = 1, D/H = 1, and Dr = 75%	257
Figure 7.16	Average Predicted Settlement of Pier vs. Width-to-Depth Ratio of Densified Zone for Two Relative Densities and D/H = 1	258
Figure 7.17	Predicted Final X-Displacement of Pier vs. Width-to-Depth Ratio of Densified Zone for Two Relative Densities and D/H = 1	258
Figure 7.18	Predicted Maximum X-Displacement of Pier vs. Width-to-Depth Ratio of Densified Zone for Two Relative Densities and D/H = 1	259
Figure 7.19	Predicted Peak-to-Peak X-Displacement of Pier vs. Width-to-Depth Ratio of Densified Zone for Two Relative Densities and D/H = 1.....	259

Figure 7.20	Predicted Accumulated X-Displacement of Pier vs. Width-to-Depth Ratio of Densified Zone for Two Relative Densities and $D/H = 1$	260
Figure 7.21	Average Predicted Settlement of Pier vs. Depth-to-Thickness Ratio of Densified Zone for $Dr = 75\%$ and $W = 10$ m.....	261
Figure 7.22	Predicted Accumulated X-Displacement of Pier vs. Depth-to-Thickness Ratio of Densified Zone for $Dr = 75\%$ and $W = 10$ m.....	261
Figure 7.23	Relative X-Displacement Between Footing and Base of Grid for Densified Zone with $D/H = 0.9$, $W = 10$ m, and $Dr = 75\%$	262
Figure 7.24	X-Velocity for Footing and Grid Base vs. Time for Densified Zone with $D/H = 0.9$, $W = 10$ m, and $Dr = 75\%$	262
Figure 7.25	Excess Pore Water Pressure Ratio in Free-Field and Densified Zone with $W/D = 0.28$, $D/H = 1$, and $Dr = 85\%$	263
Figure 7.26	Number of Elements in Densified Zone with Fully-Mobilized Strength vs. Time for $W/D = 0.28$, $D/H = 1$, and $Dr = 85\%$	263
Figure 7.27	Excess Pore Water Pressure Ratio in Densified Zone ($Dr = 85\%$) at Point P4 with and without “Perfect” Drain at Boundary.....	264
Figure 7.28	Vertical Displacement of Footing vs. Time for Chemically-Grouted Zone with $W/D = 0.5$ and $D/H = 1$	265
Figure 7.29	Relative X-Displacement Between Footing and Base of Grid vs. Time for Chemically-Grouted Zone with $W/D = 0.5$ and $D/H = 1$	265
Figure 7.30	Predicted X-Velocity of Footing and Grid Base vs. Time for Chemically-Grouted Zone with $W/H = 0.5$ and $D/H = 1$	266
Figure 7.31	Average Predicted Settlement of Pier vs. Width-to-Depth Ratio of Chemically-Grouted Zone for $D/H = 1$	267
Figure 7.32	Predicted Accumulated X-Displacement of Pier vs. Width-to-Depth Ratio of Chemically-Grouted Zone for $D/H = 1$	267
Figure 7.33	Predicted Peak-to-Peak X-Displacement of Pier vs. Width-to-Depth Ratio of Chemically-Grouted Zone for $D/H = 1$	268
Figure 7.34	Predicted Maximum X-Displacement of Pier vs. Width-to-Depth Ratio of Chemically-Grouted Zone for $D/H = 1$	268

Figure 7.35	Predicted Final X-Displacement of Pier vs. Width-to-Depth Ratio of Chemically Grouted Zone for $D/H = 1$	269
Figure 7.36	Number of Elements in Grouted and Untreated Zones with Fully-Mobilized Strength vs. Time for $W/D = 0.28$ and $D/H = 1$	269
Figure 7.37	Vertical Displacement of Footing vs. Time for Jet-Grouted Walls Located 1.4 Meters from Footing.....	270
Figure 7.38	Relative X-Displacement Between Footing and Base of Grid vs. Time for Jet-Grouted Walls Located 1.4 Meters from Footing	270
Figure 7.39	Average Predicted Settlement of Pier Footing vs. Distance Between Footing and Jet-Grouted Wall.....	271
Figure 7.40	Variation of Excess Pore Pressure Ratio, r_u , in Soils Between In-Ground Walls with Spacing from Centrifuge Tests	271
Figure 7.41	Excess Pore Water Pressure Ratio, r_u , at 9.5-m Depth vs. Time for Free-Field and in Soil Between Walls at Two Different Spacings.....	272
Figure 7.42	Excess Pore Water Pressure Ratio, r_u , at 4.1-m Depth vs. Time for Free-Field and in Soil Between Walls at Two Different Spacings.....	272
Figure 8.1	Representation of Stub Abutment and Superstructure Load in FLAC Analyses	311
Figure 8.2	Grid Used for FLAC Analysis of Stub Abutment in Parametric Study	312
Figure 8.3	Expanded Section A-A' of Grid Used for FLAC Analysis of Stub Abutment.....	313
Figure 8.4	Displacement Pattern in Vicinity of Stub Abutment for Case of No Ground Improvement.....	314
Figure 8.5	Relative X-Displacement Between Stub Abutment and Base of Grid vs. Time for No Improvement Case.....	315
Figure 8.6	Vertical Displacement of Stub Abutment vs. Time for No Improvement Case.....	315
Figure 8.7	X-Velocity of Grid Base and Stub Abutment vs. Time for No Improvement Case.....	316
Figure 8.8	Excess Pore Water Pressure Ratio Under Embankment vs. Time for No Improvement Case.....	316

Figure 8.9	Excess Pore Water Pressure Ratio Under Stub Abutment vs. Time for No Improvement Case.....	317
Figure 8.10	Excess Pore Water Pressure Ratio in Free-Field vs. Time for No Improvement Case	317
Figure 8.11	Relative X-Displacement Between Stub Abutment and Base of Grid for 39-m-wide Densified Zone ($D_r = 85\%$).....	318
Figure 8.12	Vertical Displacement of Stub Abutment vs. Time for 39-m-wide Densified Zone ($D_r = 85\%$).....	318
Figure 8.13	X-Velocity of Grid Base and Stub Abutment vs. Time for 39-m-wide Densified Zone ($D_r = 85\%$).....	319
Figure 8.14	Excess Pore Water Pressure Ratio in 39-m-wide Densified Zone ($D_r = 85\%$) Under Stub Abutment vs. Time	319
Figure 8.15	Excess Pore Water Pressure Ratio in Unimproved Soils Under Embankment vs. Time for 39-m-wide Densified Zone ($D_r = 85\%$)	320
Figure 8.16	Cases Evaluated in Parametric Study of Densified Zones at Stub Abutments	321
Figure 8.17	Variation of Stub Abutment Displacements with Densified Zone ($D_r = 75\%$) Size and Location for Group I Cases	322
Figure 8.18	Displacement Pattern in Vicinity of Stub Abutment for Densified Zone ($D_r = 75\%$) Under Embankment Slope and Outside Embankment (Case B) ..	323
Figure 8.19	Displacement Pattern in Vicinity of Stub Abutment for Densified Zone ($D_r = 75\%$) Under Entire Embankment (Case C)	323
Figure 8.20	Displacement of Stub Abutment vs. Distance Densified Zone ($D_r = 75\%$) Extends in Front of Toe for Group 2 Cases	324
Figure 8.21	Displacement of Stub Abutment vs. Distance Densified Zone ($D_r = 75\%$) Extends Behind Crest for Group 3 Cases.....	324
Figure 8.22	Displacement of Stub Abutment vs. Distance Densified Zone ($D_r = 85\%$) Extends Behind Crest for Group 4 Cases.....	325
Figure 8.23	Displacement Pattern in Vicinity of Stub Abutment for 39-m-wide Densified Zone ($D_r = 85\%$) Starting 19 m Behind Embankment Crest (Case J).....	325

Figure 8.24	Excess Pore Water Pressure Ratio Under Embankment in Untreated and Treated Areas for 39-m-wide Densified Zone at $D_r = 85\%$ (Case J).....	326
Figure 8.25	Excess Pore Water Pressure Ratio in Treated Area at Toe of Embankment and Free-Field for 39-m-wide Densified Zone at $D_r = 85\%$ (Case J).....	326
Figure 8.26	Number of Elements with Fully-Mobilized Strength in Different Areas for 39-m-wide Densified Zone at $D_r = 85\%$ (Case J)	327
Figure 8.27	Pattern of Grid Elements with Fully-Mobilized Strength at a Time of 180 Seconds for 39-m-wide Densified Zone with $D_r = 85\%$	327
Figure 8.28	Expanded Section A-A' of Grid Used for FLAC Analysis of Stub Abutment with Chemically-Grouted Zone.....	328
Figure 8.29	Relative X-Displacement Between Stub Abutment and Base of Grid vs. Time for 26-m-wide Chemically-Grouted Zone	329
Figure 8.30	Vertical Displacement of Stub Abutment vs. Time for 26-m-wide Chemically-Grouted Zone.....	329
Figure 8.31	X-Velocity of Grid Base and Stub Abutment vs. Time for 26-m-wide Chemically-Grouted Zone.....	330
Figure 8.32	Cases Evaluated in Parametric Study of Chemically-Grouted Zones at Stub Abutment	331
Figure 8.33	Displacement of Stub Abutment vs. Distance Chemically-Grouted Zone Extends in Front of Toe for Group 1 Cases	332
Figure 8.34	Displacement of Stub Abutment vs. Distance Chemically-Grouted Zone Extends Behind Crest for Group 2 Cases.....	332
Figure 8.35	Displacement of Stub Abutment vs. Distance Chemically-Grouted Zone Extends in Front of Toe for Group 3 Cases	333
Figure 8.36	Displacement Pattern in Vicinity of Stub Abutment for Chemically-Grouted Zone Under Embankment Slope and Outside Embankment (Case D)	333
Figure 8.37	Displacement Pattern in Vicinity of Stub Abutment for 26-m-wide Chemically-Grouted Zone Starting 6 m Behind Embankment Crest (Case G)	334

Figure 8.38	Number of Elements with Fully-Mobilized Strength in Different Areas for 26-m-wide Chemically-Grouted Zone (Case G).....	334
Figure 8.39	Cases Evaluated in Parametric Study of Jet-Grouted Zone at Stub Abutment	335
Figure 8.40	Displacement Pattern in Vicinity of Stub Abutment for 5.5-m-wide Jet-Grouted Zone.....	336
Figure 8.41	Relative X-Displacement Between Stub Abutment and Base of Grid vs. Time for 15-m-wide Jet-Grouted Zone	337
Figure 8.42	Vertical Displacement of Stub Abutment vs. Time for 15-m-wide Jet-Grouted Zone.....	337
Figure 8.43	X-Velocity of Grid Base and Stub Abutment vs. Time for 15-m-wide Jet-Grouted Zone.....	338
Figure 8.44	Expanded Section of Grid Used for FLAC Analysis of Stub Abutment with Buttress Fill Placed Against Embankment Slope.....	339
Figure 8.45	Relative X-Displacement Between Stub Abutment and Base of Grid vs. Time for Buttress Fill	340
Figure 8.46	Vertical Displacement of Stub Abutment vs. Time for Buttress Fill	340