2009-08-07

Liquefaction Mitigation in Silty Sands Using Stone Columns with Wick Drains

Michael James Quimby
Brigham Young University - Provo

LIQUEFACTION MITIGATION IN SILTY SANDS USING

STONE COLUMNS WITH WICK DRAINS

by

Michael J. Quimby

A thesis submitted to the faculty of

Brigham Young University

in partial fulfillment of the requirements for the degree of

Master of Science

Department of Civil and Environmental Engineering

Brigham Young University

December 2009

BRIGHAM YOUNG UNIVERSITY

GRADUATE COMMITTEE APPROVAL

of a thesis submitted by

Michael J. Quimby

This thesis has been read by each member of the following graduate committee and by majority vote has been found to be satisfactory.

Date	Kyle M. Rollins, Chair
Date	Richard J. Balling
Date	Norman L. Jones

BRIGHAM YOUNG UNIVERSITY

As chair of the candidate's graduate committee, I have read the thesis of Michael J. Quimby in its final form and have found that (1) its format, citations, and bibliographical style are consistent and acceptable and fulfill university and department style requirements; (2) its illustrative materials including figures, tables, and charts are in place; and (3) the final manuscript is satisfactory to the graduate committee and is ready for submission to the university library.

Date	Kyle M. Rollins Chair, Graduate Committee

Accepted for the Department

E. James Nelson
Graduate Coordinator

Accepted for the College

Alan R. Parkinson
Dean, Ira A. Fulton College of Engineering and Technology

ABSTRACT

LIQUEFACTION MITIGATION IN SILTY SANDS USING
STONE COLUMNS WITH WICK DRAINS

Michael J. Quimby

Department of Civil and Environmental Engineering

Master of Science

Stone column treatment is commonly used to mitigate liquefaction hazard in sandy soils. Research and experience indicate that this method is effective for clean sands but that it may not be effective for silts and sands with fines contents greater than 15-20%. An alternative to the stone column method involves supplementing stone column treatment with pre-fabricated vertical wick drains installed prior to the stone columns installation. Although this method is used in practice, there has not been a formal academic study of its effectiveness. This thesis evaluates seven different case histories where wick drains were used and one where wick drains were not used, for comparison purposes. The site locations varied as well as the soil properties and treatment plans. CPT testing was done at 3 sites and SPT testing was performed at the other 5 sites. CPT data were correlated to SPT data to facilitate comparisons. One of the case histories includes a

unique study in which three different variations of the stone column treatment were applied at the same site, providing a direct comparison of the effectiveness of each method. A 26% area replacement ratio (A_r) with drains was determined to be more effective overall than a 26% A_r without drains and more effective in increasing low initial blow counts than the 34% A_r without drains. The areas with drains were more likely to exceed the minimum project criteria consistently throughout the site. Significant scatter were observed in the results and probable causes for the scatter are noted. Final blow count coefficients of variation ranged from 28% to 77%. Increased fines contents required increased A_r in order to maintain similar average final blow counts. Site improvements were evaluated separately and collectively. Individual site results were compared to clean sand curves developed by Baez (1995). Sites with average fines contents less than 20% which were improved using drains and an 11-15% A_r treatment were comparable to clean sand sites without drains and with 5-10% A_r. To achieve similar improvement at sites with 40-46% fines necessitated drains and A_r values of 23-26%. Design recommendations are provided.

ACKNOWLEDGMENTS

I would like to thank all those who have helped me to complete this thesis. I'd like to express my appreciation to Dr. Kyle M. Rollins for his guidance, patience, and direction as my advisor. I appreciate Dr. Richard J. Balling and Dr. Norman L. Jones for serving on my committee. Thanks goes to the Utah Department of Transportation for the funding to make this project possible. The mitigation work done on the I-15 sites was funded by the Utah Department of Transportation. RB&G Engineering, Inc. performed the engineering design and quality control for the I-15 projects. Bradford Price and Robert Johnson from RB&G were very helpful throughout the whole process answering questions, providing data, and offering guidance. Thanks go to Lisheng Shao and Mark Koelling from Hayward Baker for allowing me to visit their offices and gather needed data. I'd like to thank Lisheng for all of the time he volunteered in helping me to complete the Silver Reef, Marina del Rey, and San Pedro portions of my research. Hayward Baker provided the data from these three sites in order to allow me to broaden my research. Thanks to Emily Dibbs for her help in getting this project started and providing the data and preliminary analyses of the Shepard Lane, Cherry Hills, and Salmon Lake projects. I'd like to thank my parents for giving me the means and the encouragement to be able to get to this point. I also wish to thank my wife Dené for her

tireless support and encouragement, without which I would not have been able to finish this thesis.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1 Introduction

1.1 Background

The United States Geological Survey (USGS) estimates that each year several million earthquakes occur worldwide. Of these earthquakes, on average there are approximately 134 large earthquakes with magnitude of 6.0 or greater. Based on information collected by the USGS National Earthquake Information Center from 2000 to 2008, the worldwide average number of deaths per year due to earthquakes is 51,509 (USGS, 2009). In the 1994 Northridge, California earthquake alone there were over 60 deaths, 5,000 injuries, and 25,000 people left homeless (FEMA 2006). These statistics show the devastating damage and loss of life that occur in the world and in the United States each year due to earthquakes.

One of the means by which an earthquake causes such significant damage is through the process of soil liquefaction. Liquefaction is the loss of the structural stability of a soil due to an increase in pore water pressure during an earthquake. Liquefaction is common in saturated loose granular soils such as sands as well as non-plastic soils with poor drainage such as silty sand. Soil liquefaction results in significant damage to buildings, transportation systems, and lifelines in most major earthquake events. Liquefaction and the resulting loss of shear strength can lead to landslides, lateral spreading of bridge abutments and wharfs, loss of vertical and lateral bearing support for foundations, and excessive foundation settlement and rotation. Liquefaction resulted in nearly $1 billion worth of damage during the 1964 Niigata Japan earthquake (NRC, 1985), $99 million in damage in the 1989 Loma Prieta earthquake (Holzer, 1998), and over $11.8 billion in damage just to ports and wharf facilities in the 1995 Kobe earthquake (EQE, 1995). The loss of these major port facilities subsequently led to significant indirect economic losses. The port facilities in Oakland, Los Angeles, and Seattle are potentially vulnerable to similar losses.

Figure 1-1 shows several apartment buildings which fell over due to liquefaction of the soil supporting the structures during the 1964 Niigata, Japan earthquake. In order to minimize the destructive effects of an earthquake, liquefaction hazard in all soils supporting structures must be mitigated.

Figure 1-1 Settlement and tilting of apartment buildings in Niigata, Japan due to a M6.8 Earthquake in 1964 (source, Earthquake Engineering Research Center Library, University of California at Berkeley).

There are several methods of liquefaction hazard mitigation, with one of the most common methods being the installation of stone (gravel) columns in soils susceptible to liquefaction. Stone columns are installed in a vibratory manner in order to further densify the soil. Stone columns have been effectively used to mitigate liquefaction in sand, but they are typically much less effective in silty sands or soils with high fines content (Mitchell, 1981; Baez, 1995; Rollins et al., 2006). Mitchell (1981) indicates that vibratory compaction techniques are relatively ineffective when fines content exceeds 10 to 20%, as seen in Figure 1-2. Higher fines contents reduce permeability and provide increased soil strength, both of which make it more difficult to increase density during vibratory loading.

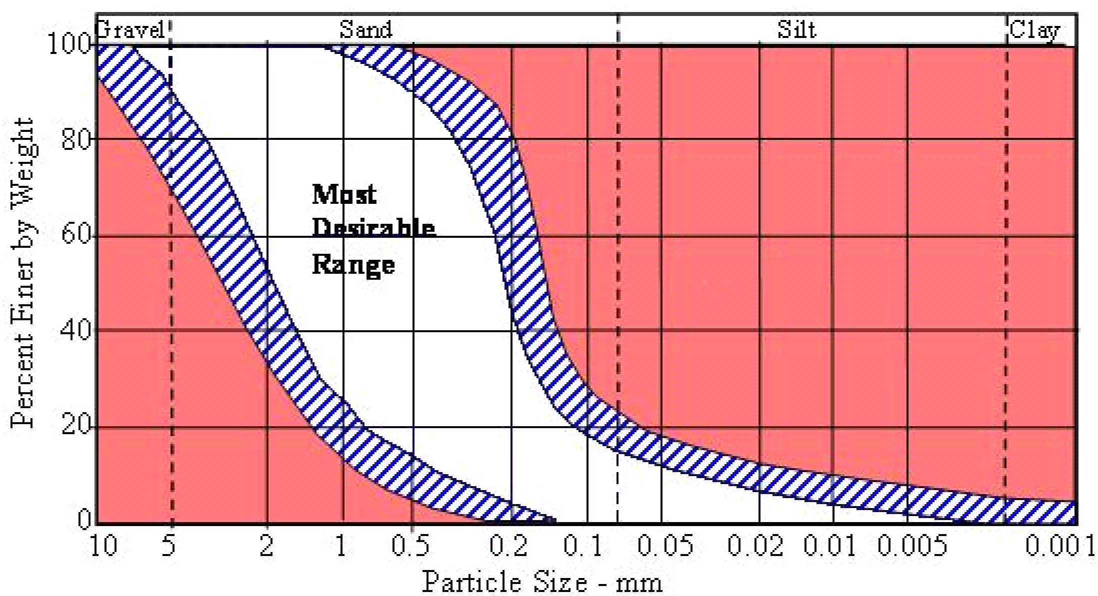


Figure 1-2 Effectiveness of vibratory compaction techniques based on fines content (Mitchell, 1981).

In order to improve the efficiency of the stone column method, practicing engineers have tried installing pre-fabricated vertical wick drains between the stone columns prior to column installation to assist in pore pressure relief through drainage. This method has been used at a few sites in recent years but there has yet to be an academic study of its effectiveness in mitigating liquefaction hazards.

1.2 Objectives

The purpose of this research was to determine whether the practice of installing wick drains in combination with stone columns is effective in mitigating liquefaction. The practice of installing stone columns alone in silty sands tends to be more expensive and less effective than in clean sands. Thus it is important to find a more economical and effective method. The stone column with wick drains method has been performed by contractors previously, but there has never been a direct comparison test at a single site in order to compare the stone column method to the stone column with wick drains method.

The objectives of this research are as listed below.

1. Determine if the use of wick drains with stone columns has a beneficial effect and how substantial that effect might be.
2. Develop methods to predict final blow count as a function of initial soil parameters such as blow count, area replacement ratio, and fines content for stone columns with drains.
3. Identify conditions which will limit the effectiveness of stone column treatment with drains.
4. Develop recommendations regarding design of stone columns with drains in silty sands.

1.3 Scope

To accomplish the objectives of this study it was necessary to collect available case history data relating to stone column treatment with wick drains. A thorough review of the literature and discussion with specialty geotechnical contractors indicated that data was available for several case histories. Specifically, data for the following sites were available; Esprit Apartments, Marina del Rey, California; Home Depot, San Pedro, California; Silver Reef Casino Expansion, Silver Reef, Washington; Shepard Lane Bridge Abutment, Farmington, Utah; Cherry Hill Bridge Abutment, Kaysville, Utah; and Salmon Lake Dam, Salmon Lake, Washington. All available data was collected and analyzed to determine the effectiveness of the remediation procedures undertaken at each site. Results from each site are presented in this thesis. Finally, after each site had been individually analyzed, the data and results were compared for all of the sites together.

To determine the effectiveness of the addition of wick drains to the stone column method, it is desirable to compare the two methods side by side at a single test site. The review of the case history data indicated that this had not been done previously, so as part of this study the treatment zone at the I-15 and 24th Street Bridge in Ogden, Utah was divided into two areas; one test area with wick drains and stone columns and a second adjacent test area with stone columns only. This provided data that could be compared more directly than data from different sites where the soil conditions may vary. Pre- and post-improvement SPT borings were performed throughout the test section to evaluate the effectiveness of the soil improvement procedures. Results for the test section were analyzed and are presented.

In addition to the test section at the I-15 and 24th Street Site, the overall site data for the I-15 & 24th Street site was analyzed and the results presented. Another UDOT site, the I-15 and UPRR Railroad bridge site, which was similarly improved using stone columns with wick drains, was analyzed and the results presented. Both sites had pre- and post-improvement SPT borings performed. Additional soil testing including sieve analysis and hydrometer analysis were performed in order to classify the soil and determine the factors which affect improvement in the soil. These additional tests are a part of the UDOT funded research and required direct involvement between UDOT, Brigham Young University, the engineering firm RB&G Engineering, and the construction company Hayward Baker.

Each project site studied in this thesis was initially analyzed separately. A collective analysis was also performed and is discussed in Chapter 11 - Compiled Analysis of All Sites. The individual analyses were generally performed using the same method to allow comparison of the results from different sites.

Depending on the site, SPT or CPT testing was used to evaluate the effectiveness of the mitigation procedures. To enable comparisons between the different project sites, normalized SPT blow count, $(N_1)_{60}$, was used as the measure of improvement for all sites. For the sites where CPT testing had been used instead of SPT testing, correlated $(N_1)_{60}$ values as well as correlated fines contents are based on Robertson and Wride's (1998) method, unless otherwise noted. At sites with correlated $(N_1)_{60}$ values, the conversions were performed previously by the geotechnical engineers involved with the project, except where noted.

Data were analyzed using the steps below.

1. Created scatter plots of initial and final blow counts vs. depth for the entire site. Average lines were shown for each series.
2. Created plots of companion pre- and post-treatment $(N_1)_{60}$ values versus depth wherever direct pre- and post-treatment profiles were available. Possible trends were investigated and discussed. (Note that Steps 2- 6 only apply to sites with direct comparison data between initial and final CPT or SPT tests).
3. Created a scatter plot of the pre- and post-treatment $(N_1)_{60}$ values versus depth for all of the direct comparison data together. Averages were shown for each soil type and treatment method.
4. Where available, the following properties were compared to improvement using scatter plots; fines content, depth, area of replacement, clay content, initial $(N_1)_{60}$, and plasticity index. Improvement was defined as the change in the normalized penetration resistance value or $\Delta(N_1)_{60.}$
5. For properties that exhibited a potential cause and effect relationship with improvement and which didn't exhibit significant data scatter, linear regression was performed.
6. Properties deemed significant from step 4 were compared to the final blow counts for all possible direct comparison data.
7. Averages properties and improvements for all the available direct comparison data were calculated and presented in tabular form. A table of average improvement for the entire site was also presented.

8. Where data were available, the effect of the amount of time that elapsed between improvement and testing (time after improvement) was investigated using scatter plots and regression when applicable.
9. Created a scatter plot with a logarithmic trend line for initial $(N_1)_{60\text{-}cs}$ versus final $(N_1)_{60\text{-}cs}$. Created a plot comparing the clean sand logarithmic trend line to clean sand (<15% fines) curves developed by Baez (1995).
10. Any other relevant analyses or plots may also be presented throughout the results and analysis sections where applicable.

2 Literature Review

2.1 Soil Liquefaction

2.1.1 Liquefaction Mechanics

Loose cohesionless soils, such as sands, are susceptible to liquefaction during an earthquake because of their tendency to densify under cyclic loading. During the rapid cyclic loading of an earthquake, saturated soils typically do not drain rapidly enough to relieve excess pore water pressures. As a cohesionless soil undergoes earthquake loading, the soil tends to densify which in turn increases the pressure applied to any water present in the soil. Once the excess pore water pressure reaches the vertical effective stress, the load begins to be carried by the water instead of the soil. In this situation, the soil loses its strength and begins to act more like a liquid than a soil, thus the phrase "liquefaction" (Mogami and Kubo, 1953). In soils with poor drainage, the same problem is encountered as the excess water is unable to escape the soil and the soil is then susceptible to liquefaction. Poor drainage is a problem in soils with high fines contents. Higher fines content tends to increase the strength of the soil structure but significantly limits drainage since the fines fill in the void spaces that previously acted as a drainage path for excess water. Soils with high fines content also densify less due to the reduction of voids as the fines fill in the available void spaces. Thus saturated, loose, cohesionless soils as well as cohesionless soils with high fines content are susceptible to liquefaction due to the effects of densification and poor drainage during the rapid cyclic loading of an earthquake.

2.1.2 Liquefaction Testing

Liquefaction potential is determined by a variety of soil testing procedures with the two most common procedures being Cone Penetration Testing (CPT) and Standard Penetration Testing (SPT). Both tests are in situ tests performed in the field. In the Standard Penetration Test a thick-walled sample tube is driven into the ground by blows of a 140 lb. slide hammer dropped from a height of 30 inches. The number of blows required to drive the tube every 6 inches, up to 18 inches, is recorded. The sum of the last two values is taken as the standard penetration resistance or N (blows per foot). The blows per foot, or N value, is used to determine the liquefaction potential of the soil being tested.

The Cone Penetration Test is performed by pushing a cone into the ground at a controlled rate. The cone has instrumentation on it to record the penetration resistance at the tip of the cone, otherwise referred to as the tip resistance or q_c, as well as to record the sleeve friction or f_s on the sides of the cone. The ratio of the sleeve friction and the tip resistance is known as the friction ratio, Rf. Oftentimes the tip resistance, q_c, is used to determine the liquefaction potential of the soil being tested, similar to the N value for the SPT test.

A geotechnical earthquake analysis is performed for a site and the minimum values of N or q_c required to ensure that the soil will not liquefy during a possible earthquake is determined. The geotechnical analysis takes into consideration the geographical location of the site, the maximum magnitude earthquake anticipated, as well as the soil properties.

2.2 Stone Column Liquefaction Mitigation Technique

There are several different liquefaction techniques currently used in practice. These techniques include vibro-compaction, drainage, explosive compaction, deep soil mixing, deep dynamic compaction, permeation grouting, jet grouting, and stone (gravel) columns. Baez (1995) gives a short overview of each of these methods. The research done for this project focused on the stone column method only with the emphasis being on the effectiveness of the stone column method in silty soils, thus only the stone column method is addressed here.

2.2.1 Overview of Stone Column Method

Stone column installation as a liquefaction mitigation technique is a widespread technique currently being used by construction companies both inside and outside the United States. Significant research has been done for the stone column method in the United States as well as in Japan (e.g. Seed and Booker, 1977; Ishihara and Yamazaki, 1980; Boulanger et al., 1998; Baez and Martin, 1995). Construction companies such as Hayward Baker in the United States (http://haywardbaker.com) currently use this technique.

Stone column installation begins by vibrating a probe into the soil along with jets of water (wet method) or compressed air (dry method). The gravel is then fed into the column through the tip of the vibrator (bottom feed method) or by pushing gravel around the top of the probe (top feed). The vibrator is then raised and lowered in order to compact the gravel. The gravel is inserted into the hole and compacted in multiple lifts with each lift typically being about 1 meter in length. This method is illustrated in Figure 2-1. All of the case histories in this study employed the dry bottom feed method for treatment.

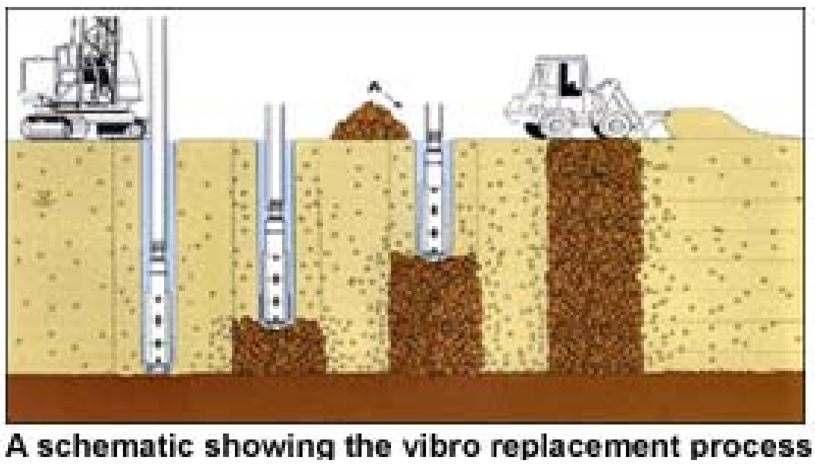


Figure 2-1Vibro-stone column installation method (source, www.kellerasia.com).

The vibration during installation ensures that the gravel is compacted thoroughly throughout the entire length of the column. The operator can evaluate compaction efficiency by monitoring the amperage of the probe and the working time within each lift.

The amount of soil displaced by the stone columns is quantified by the area replacement ratio, or A_r. The area replacement ratio (A_r) is calculated as the area of the stone column cross section (A_c) divided by the tributary area of the stone column (A_e). The two primary stone column arrangements are (a) squares and (b) equilateral triangles, as shown in Figure 2-2. For equilateral triangles, the equivalent diameter of the tributary area (D_e) is calculated as 1.05 times the center-to-center spacing between stone columns. For squares, the tributary area is as shown in Figure 2-2. The area replacement ratio can be increased by increasing the diameter of the stone columns or by decreasing the center-to-center spacing.

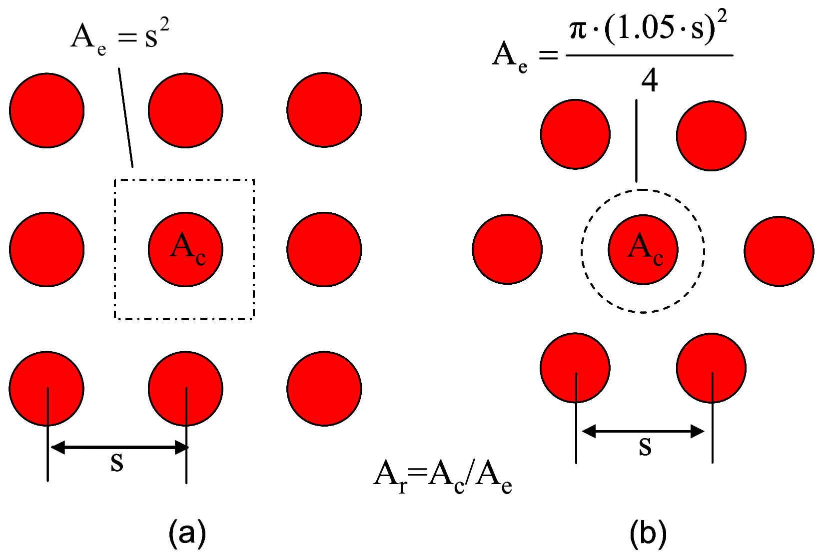


Figure 2-2 Stone column area replacement ratios (A_r) for the (a) square arrangement and the (b) equilateral arrangement.

2.2.2 Stone Column Liquefaction Mitigation Mechanisms

Stone columns are thought to reduce liquefaction hazard and improve soil performance in four main ways. First, the stone column provides increased drainage of the soil surrounding the columns which results in a reduction of pore water pressure during a seismic event. Second, densification of the soil occurs around the stone column during installation which increases the resistance to liquefaction. Third, the stone column serves as reinforcement to the treated soil area since it is stiffer and stronger than the surrounding soil. Fourth, the method increases the lateral stresses in the soil surrounding the column (Adalier and Ahmed, 2004; Baez and Martin, 1992; Baez, 1995; and Rollins et. al, 2006).

The drainage of the soil during installation and as well as the potential drainage path through the stone column in the event of an earthquake are considered to contribute to liquefaction mitigation. Baez and Martin (1992) conducted a field study of a stone column liquefaction mitigation site where it was observed that the stone columns seemed to act as a drainage path even during installation. This observation was based on the lack of significant pore pressure generation at their depth of interest. The soil profile was 10 meters of poorly graded sand with a 2 meter layer of sandy silt at 6 meters below the surface. Millea (1990) used a theoretical earthquake simulation model to test a saturated clean sand deposit with and without stone columns in order to determine the drainage effects of the columns. Millea determined that pore pressures were effectively reduced up to two diameters away from the stone columns when compared against the pore pressures of the test without columns. Full scale tests of stone column installation in loose cohesionless soils by Ashford et al. (2000a,b) indicate that excess pore water pressure generation was reduced and the rate of pore pressure dissipation increased due to stone column installation.

On the other hand, Boulanger et al. (1998) investigated the drainage effects of the stone column in varying soil conditions and concluded that although increased drainage is possible, it is highly affected by construction procedures and that vibro-replacement stone columns tend to mix the native soil with the gravel of the columns leading to reduced permeability. As a result it was recommended that the primary mechanism of liquefaction mitigation be densification without regard to drainage. Any possible contribution due to drainage should be considered as a secondary effect when using stone columns according to their recommendations. Sasaki and Taniguchi (1982) performed large scale shake table tests at Tsukuba Science City using clean sands and determined that although a reduction in pore water pressure is possible, high frequency strong motion earthquakes would lead to a quick build up of excess pore water pressures in the native soils. The drainage effects of the stone column method are still debatable and as such they are not always considered to contribute to liquefaction mitigation when using the stone column method.

Another factor thought to contribute to the liquefaction mitigation of the stone column method is the stiffening effect of the stone columns on the area treated. When considering a soil profile, the replacement of the native soils with a compacted stone column in various locations causes the soil profile to have a greater stiffness. In liquefaction applications, the soil being replaced has a stiffness that is far less than that of the stone column. The increased stiffness of the column provides additional resistance to the dynamic lateral loading during a seismic event and reduces the stresses to the surrounding soil.

In the United States, the most widely accepted of the factors contributing to liquefaction mitigation is the densification of the surrounding soil. Many of the liquefaction mitigation methods currently employed in the United States only consider densification of the soil as contributing to liquefaction mitigation (Adalier K. and Ahmed E., 2004).

The vibratory method ensures that the gravel is thoroughly compacted and it also aids in the densification of the surrounding soil. The vibration and the compression of the gravel in the column presses gravel into the soils adjacent to the column as the column is compacted, further densifying the surrounding soil. The stone column is denser than the surrounding soil, further adding to the liquefaction resistance.

Barksdale and Bachus (1983) proposed the use of a relative density chart when designing stone column improvements (See Figure 2-3). The penetration resistance in sands, as measured by the relative density, is compared with the soil classification in order to determine the tributary area per compaction probe to be used in preliminary design. The chart was obtained from vibro densification improvements to a sand backfill. As explained by Baez (1995), the chart could be used to estimate densification potential with stone columns if a 2.5 ft diameter column is used.

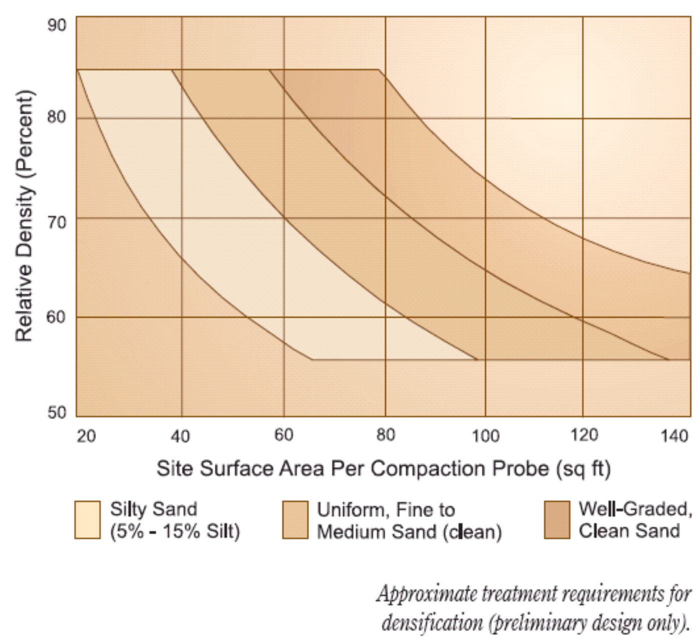


Figure 2-3 Hayward Baker densification potential chart (source, Hayward Baker "Vibro Systems, 2004" brochure, www.haywardbaker.com, after Barksdale and Bachus, 1983).

Baez and Martin (1993) introduced a simplified procedure which accounts for all three liquefaction mitigation factors and Baez (1995) has further proposed a more advanced design procedure for practicing engineers. Shenthan et al. (2003) has also introduced a design procedure which takes into consideration pore pressure dissipation (drainage) as well as densification. The method also takes into consideration the use of supplemental wick drains as part of the design procedure; however, the method has not been validated with field performance.

2.2.3 Stone Column Liquefaction Case Histories and Field Studies

Mitchell and Wentz (1991) evaluated several vibro-replacement stone column sites following the 1989 Loma Prieta earthquake. The earthquake was magnitude 7.1 with recorded peak ground accelerations ranging from 0.11g to 0.45g near the epicenter. All but one of the sites consisted of manmade fill. The sites examined and their soil conditions are found below in Table 2-1 from the 12 sites that they evaluated, they determined that the vibro-replacement procedure was effective in preventing liquefaction. None of the sites examined showed any signs of liquefaction.

One specific example was the Medical / Dental Clinic in Treasure Island, CA, where construction was underway at the time of the earthquake. The soil profile consisted of between 31 to 43 feet of loose to medium dense hydraulically placed sands underlain by 30 feet of soft Bay Mud. The building footings were cast and showed no cracking. A portion of the elevator shaft filled with sand and it was determined that the soil from 22 feet to 40 feet had liquefied. The treatment depth of the stone columns was 22 feet. The only surface manifestations of liquefaction likewise occurred outside of the treated portion of the site. The stone column treatment was thus determined to be successful in mitigating liquefaction.

Table 2-1 General information about the sites that were investigated by Mitchell and Wentz (1991).

No.	Location	Project Type	Site Characteristics (Soil Profile)	Initial Soil Properties	Reference
1	Treasure Island San Francisco, CA	Medical / Dental Bldg. (40% complete, 2-story steel-frame structure)	6 m loose to med. dense hydraulic sand fill over 6 m very loose silty sand fill over Bay Mud, GWT @ 2.3 m depth	$(N_1)_{60}$ = 4-62 avg. $(N_1)_{60}$ = 27, <10% fines in upper sand layer	Mitchell and Wentz (1991) Bolt (1990)
2	Treasure Island San Francisco, CA	Office building No. 450 (3-story steel-frame with concrete walls and floors)	9 m loose to med. dense hydraulic sand fill over 2.5 m med. dense sand over Bay Mud	$(N_1)_{60}$ = 3-54 avg. $(N_1)_{60}$ = 19, <10% fines	Mitchell and Wentz (1991) Bolt (1990)
3	Treasure Island San Francisco, CA	Facilities 487, 488, 489 (3-story concrete bldgs.)	6 m very loose to med. dense hydraulic sand fill over 5 m med. dense silty sand fill over Bay Mud, GWT @ 3.0 m depth	<12% fines in upper sand layer	Mitchell and Wentz (1991) Bolt (1990)
4	Treasure Island San Francisco, CA	Approach Area Pier 1	1.3 m loose to med. dense hydraulic sand fill over Bay Mud, GWT @ 3.0 m depth	<10% fines	Mitchell and Wentz (1991) Bolt (1990)
5	Treasure Island San Francisco, CA	Building No. 453 (4-story concrete bldg.)	8 m loose to med. dense hydraulic sand fill over 5.5 m loose silty sand fill over Bay Mud, GWT @ 2.8 m depth	$(N_1)_{60}$ = 3-46 avg. $(N_1)_{60}$ = 14, <12% fines in upper sand layer	Mitchell and Wentz (1991) Bolt (1990)
6	Richmond, CA	Marina Bay Esplanade, Buttress Against Lateral Spreading	4 m med. dense to dense sandy and gravelly artificial fill over 3.5 m loose silty sand hydraulic fill over Bay Mud, GWT @ 1.4 m depth	$(N_1)_{60}$ = 11-22 avg. $(N_1)_{60}$ = 15, avg. q_{c1} =45 kg/cm2, <55% fines in silty sand layer	Mitchell and Wentz (1991) Bolt (1990)
7	Emeryville, CA	East Bay Park Condominiums	3-6 m med. dense hydraulic sand fill over Bay Mud, GWT @ 1.5 m depth	avg. $(N_1)_{60}$ = 18, <5% fines in sand	Mitchell and Wentz (1991) Bolt (1990)
8	Alameda. CA	Perimeter Sand Dike, Harbor Bay Business Park	Loose to dense silty sand hydraulic fill overlying dense sand with pockets of soft to med. stiff silty clay with peat, GWT @ 3 m depth	Fill: N_{60} = 2 to ≥ 25 q_c = 10 to ≥ 80 tsf, <11% fines in hydraulic sand fill	Mitchell and Wentz (1991) Bolt (1990)
9	Union City, CA	Hanover Properties, 5 tilt-up panel bldgs. cover an area of 18,580 m2	0.6-0.9 m hard clayey silt fill over 0.6 m of alternating layers of loose sand and firm silt over Bay mud, GWT @ 2.1 m depth	n/a	Mitchell and Wentz (1991) Bolt (1990)
10	South San Francisco, CA	Kaiser Hospital Addition	2.4 m of unconsolidated fill over 8 m of loose to med. dense hydraulic sand fill	avg. $(N_1)_{60}$ = 19	Mitchell and Wentz (1991) Bolt (1990)
11	Santa Cruz, CA	Riverside Avenue Bridge	1.5 m of sat. loose to med. dense sandy gravel over 3.4 m dense gravelly sand; soils are submerged	<5% fines	Mitchell and Wentz (1991) Bolt (1990)
12	Santa Cruz, CA	Adult Detention Facility	1.2-3.6 m of firm to very stiff clays and silts and medium to very dense sands and gravels over 6-21 m of soft to stiff sandy silts and loose to med. dense silty sands over siltstone bedrock, GWT @ 4.5 m depth	$(N_1)_{60}$ = 4-27 avg. $(N_1)_{60}$ = 13	Mitchell and Wentz (1991) Bolt (1990)

Baez (1995) evaluated two stone column sites following the January, 1994 Northridge earthquake. The type of soil at the sites was not referenced but is assumed to have been sand or silty sand based on the potential liquefaction hazard. The earthquake was magnitude 6.8. The first site was a building which experienced peak ground accelerations greater than 0.7g. There was no ground distress or liquefaction apparent around the building. The second site was the approaches to an elevated railroad track 30 miles from the epicenter. No ground acceleration records were available. No signs of liquefaction were evident following the earthquake and CPT soundings indicated a significant degree of densification as a result of the earthquake.

Iai et al. (1994) reported on a quarry wall at Kushiro Port following the January, 1993 Kushiro-Oki earthquake. The earthquake was magnitude 7.8 and the site experienced peak ground accelerations of approximately 0.47g (Iai et al., 1995). The soil profile consisted of approximately 11 meters of fill and loose sand underlain by medium to dense gravelly sand deposits. Stone columns (gravel drains) and sand compaction piles were used to mitigate liquefaction. The quarry experienced no damage while everywhere else stuck by the earthquake had moderate to severe damage.

2.2.4 Factors Affecting Improvement

The effectiveness of the stone column method is based on several different factors. Some of the factors that researchers have identified as important include the following; soil type, silt content, clay content, plasticity of the soil, degree of pre-densification, relative densities, hydraulic conductivity, vibrator type, stone shape and durability, stone column spacing and area (area replacement ratio, A_r), construction sequencing, improvement gains over time, and depth beneath the surface (Baez, 1995; Adalier and Ahmed, 2004; Shenthan et al., 2003; Shenthan, 2006; Rollins et al., 2006). Each of these factors contributes to the stone column treatment's ability to mitigate liquefaction through densification, drainage, or stiffening of the treated soil. Listed below are some of the different conclusions presented by researchers concerning these different factors.

In his design procedure, Baez (1995), notes that there are three densification mechanisms for the stone column method. The first mechanism is the controlled liquefaction induced by the vibrator during installation. As the column is compacted and the vibrator is employed in compaction, the soils in the immediate vicinity undergo a controlled, localized liquefaction. The second mechanism was the confining effect of installing columns in groups of equally spaced columns instead of installing columns as single members. Groups of columns were more effective than individual columns. The third mechanism was the effect of improved resistance with time. Baez noted that results of up to one year after treatment were better than those immediately following treatment (Baez, 1995). There were also short-term improvements as well.

Shenthan et al. (2003) conducted research similar to what is presented in this paper with more of an emphasis on numerical modeling. The stone column and wick drain layout used in the author's simulation was an equilateral triangle with drains being spaced equidistant between all adjacent columns. Wick drains were installed prior to the stone columns. The authors' research concerning the use of wick drains to supplement stone columns suggests that an area replacement ratio of at least 20% is required for the wick drains to contribute significantly to drainage or densification. In the authors' model, the greater the area replacement ratio, the closer the spacing of the wick drains from the columns. The authors also noted that soils with a hydraulic conductivity less than about 10^{-6} m/s saw no significant improvement unless wick drains were employed in the mitigation procedures.

Several researchers note that the stone column method is ineffective in silty sands and soils with fines contents greater than about 15-20% (Baez, 1995; Adalier and Ahmed, 2004; Rollins et al., 2006). Baez (1995) concludes that densification and drainage by vibro stone columns does not contribute to liquefaction resistance in soils with fines contents greater than 15%. Baez suggests that there is a need for additional methods of liquefaction mitigation for soils with fines contents greater than 15%. Figure 2-4 below shows Baez's best fit curves of improvement, n, based on normalized pre-SPT blow count. Improvement is measured as the normalized post-SPT blow count divided by the normalized pre-SPT blow count. Figure 2-5 shows the model's prediction of normalized post-SPT blow count based on the area replacement ratio and the normalized pre-SPT blow count. The model is only accurate for soils with fines contents less than 15%.

Shenthan (2005) studied why the stone column method is ineffective in silty soils. Shenthan concluded that the primary reason was that low coefficient of consolidation associated with silty soils leads to slower pore pressure dissipation during installation. The slower rate of pore pressure dissipation is what hinders the densification of the soil around the stone columns. The low coefficient of consolidation also causes the stone columns to be less effective as a drainage routes during an earthquake.

Andrews (1997) reported on a vibro-stone column site at the Fern Hill Water Treatment Plant near Portland, Oregon. The 15 meter thick non-plastic layer of silty soil was improved using the dry bottom feed method with an area replacement ratio of 20%. The site fines content was approximately 85%. The design earthquake was magnitude 7.3 with peak ground acceleration of 0.3g. Test results from SPT, CPT, and other testing indicated that soil improvement was effective when the fines were non-plastic.

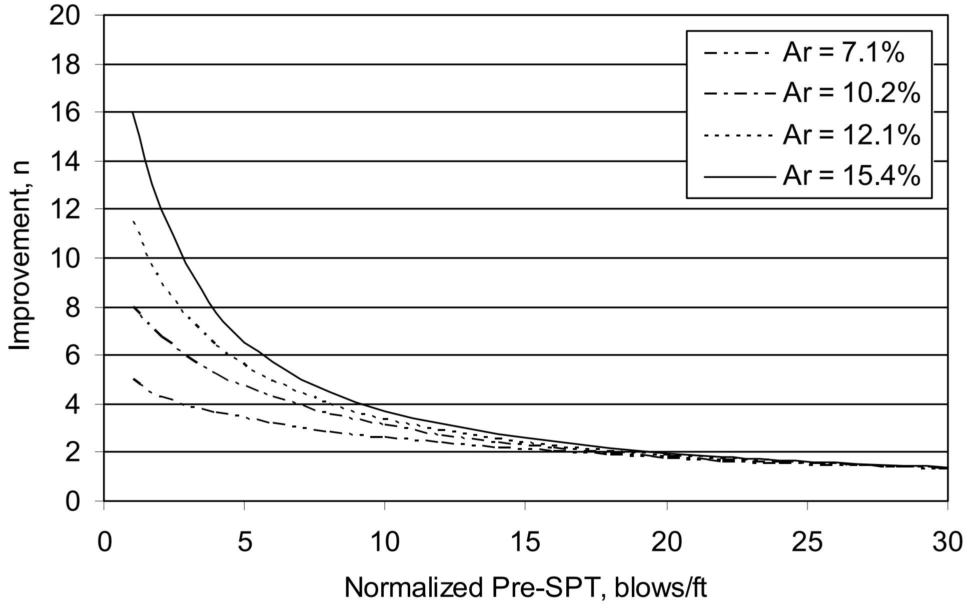


Figure 2-4 Baez (1995) best fit curves for observed data at sites with uniform fine to medium silty sands (<15% passing No. 200 sieve). Improvement (n) is measured as the final blow count divided by the initial blow count.

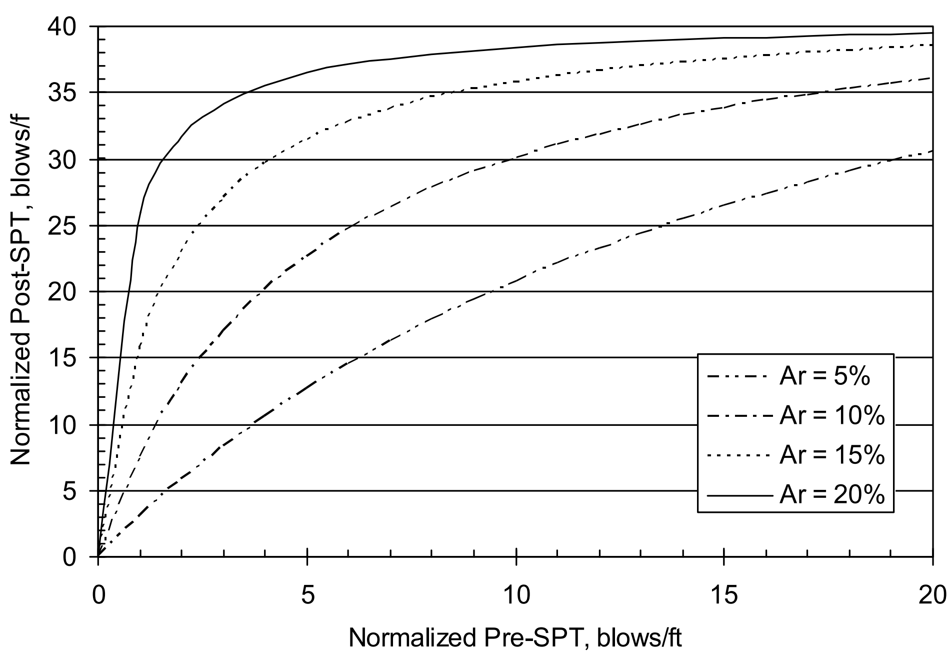


Figure 2-5 Baez (1995) model's prediction of normalized post-SPT blow count based on normalized pre-SPT blow count and area of replacement.

Current research regarding techniques of liquefaction mitigation in soils with high fines contents is limited. In practice, construction companies have adopted several different methods to address the problem. One such method is the use of wick drains to supplement the stone columns (see Figure 2-6).

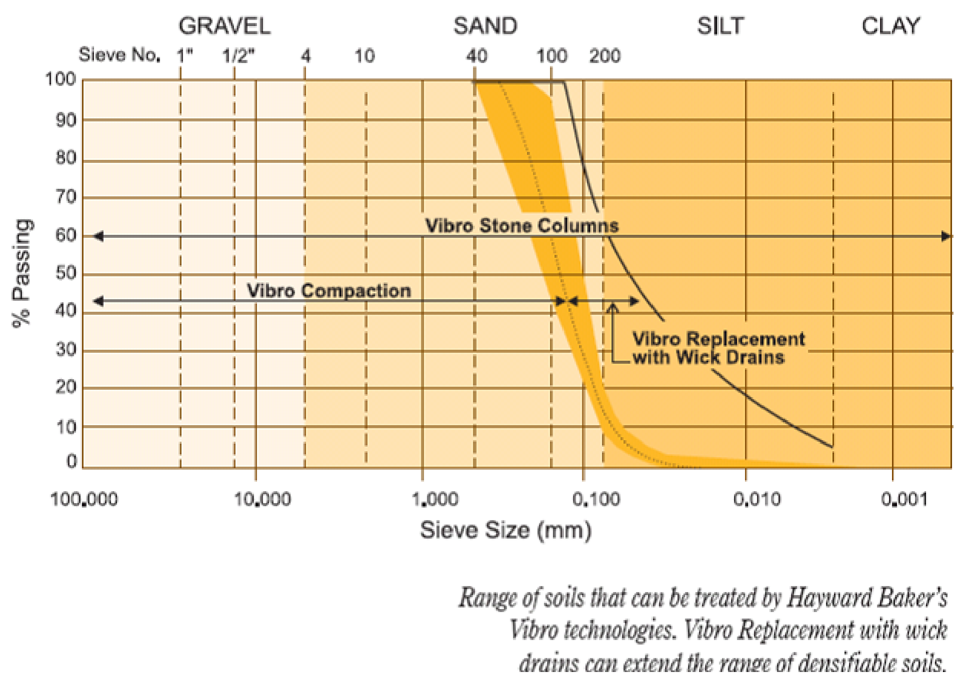


Figure 2-6 Figure showing improvement techniques employed by Hayward Baker, Inc. with wick drains used for soils with high fines contents (Source, haywardbaker.com).

2.3 Stone Column with Wick Drains Liquefaction Mitigation Approach

To the author's knowledge, there is very little research available concerning the use of supplemental wick drains installed prior to stone column installation. Engineers and construction companies have begun to implement the supplemental wick drain method, but it is based on local experience and expensive test section must be employed at each site in order to determine the effectiveness of the method. One such test site was the Salmon Lake Dam, Washington where wick drains were used to supplement stone columns for liquefaction mitigation.

Luehring et al. (2000, 2001) reported on the stone column with wick drain approach to liquefaction mitigation at Salmon Lake Dam, WA. The site had high fines contents with fines contents up to 60%. A test section was used to determine the effectiveness of the supplemental wick drain method. Wick drains were installed prior to stone columns equidistant from adjacent column locations. Water and air bubbles were observed exiting the drains during the stone column installations (see Figure 2-7). Piezometer readings also showed low pore pressures around columns with wick drains nearby and high pore pressures around columns without wick drains. The supplemental wick drains were determined to be more effective than stone columns alone and the rest of the site was completed using supplemental wick drains.

Shenthan (2005) developed a numerical method for the stone column with wick drain method to assist practicing engineers in applying this method. Based on the numerical model and the test verifications of the model, Shenthan suggested that wick drains are needed for soils with hydraulic conductivity lower than about 10^{-5} m/s. The flow charts presented for the numerical model depend on three parameters. The parameters needed are the current normalized clean sand equivalent SPT blow counts for the soil layers of concern, the in situ hydraulic conductivity of the layer, and the desired normalized clean sand equivalent SPT blow count for the layers. The model assumes that the stone columns are in a triangular pattern with wick drains pre-installed equidistant between adjacent columns, and that the soil deposit is uniform, loose, and normally consolidated. In addition to this the stone columns and wick drains are assumed to have infinite permeability. Shenthan concludes that high replacement ratios are needed for wick drains to be effective. Also, even though the method is effective for soils containing non-plastic silt and hydraulic conductivity as low as 10^{-7} m/s, the degree of improvement decreases with increasing silt content and decreasing hydraulic conductivity.

Figure 2-7 Water escaping through previously installed wick drains during stone column installation (source, Hayward Baker, Inc.).

3 I-15 and 24th Street Bridge Abutment, Ogden, Utah

3.1 Site Overview and Soil Conditions

The 24th Street Bridge site was located at the 24th Street overpass on Interstate 15 in Ogden, Utah. The 24th Street Bridge was part of the Utah Department of Transportation (UDOT) I-15 NOW project and was also known as Bridge 16. The site consisted of existing twin bridges that were replaced with newer, wider bridges. The abutments to the bridges were built on treated soil and the construction was done in two phases to accommodate traffic demands and treatment objectives.

RB&G Engineering prepared the report of geotechnical observation and testing for the site. The design earthquake for the site was a magnitude 7.4 earthquake with a peak ground acceleration of 0.57g. The peak ground acceleration was based on a 2% probability of being exceeded in 50 years (~2500 yr recurrence interval). The site is located within a about a mile of the Wasatch fault.

Prior to treatment, nearly continuous SPT sampling was performed at the test site to determine liquefaction potential and soil characteristics at the site. The soil profiles varied somewhat but were relatively consistent across the site. A generalized soil profile is provided in Figure 3-1 along with profiles of fines content and clay content. The preliminary geotechnical investigation classified the soil as silty sand (SM) layers to about 13 feet with an average fines content of 26% and a clay content of 6%. This was underlain by sandy silt (ML) and silty sand to about 20 feet with fines and clay contents increasing. The average fines content increased to 35% and that average clay content to 11%. Below 13 feet the variability of the fines and clay contents increased as the soil became more interbedded with depth. The soil profile below 20 feet to a depth of about 40 feet consisted of interbedded sandy silt and silty sand layers with an average fines content of 43% and an average clay content of 14%. Below 40 feet the soil profile mainly consisted of clayey silt to silt clay to the depth of exploration. The variations in soil properties were expected to produce significant variations in the success of the stone column treatment and attention will be given to these variations in the subsequent analysis section.

The overall site averages were 34% fines content and 13% clay content. It should be noted that while the analyses consider only the sand and silt soil combinations, the overall site profile (average clay and fines contents) consider all soil combinations. This is considered to be an appropriate assumption due to the fact there were relatively few clays and other soils found within the soil profile as a whole. Averages for the SM and ML data varied by only a few percent from the averages presented in this case history.

Soil liquefaction was deemed a potential hazard at the site and mitigation efforts were required. Liquefiable deposits were most frequently encountered within the upper 40 feet of the soil profile. Liquefaction analyses were performed by RB&G using the "Simplified Procedure" developed by Seed and Idriss (1971) with refinements presented at the 1996 NCEER workshop (Youd, et al., 1997).

To prevent liquefaction the minimum average post-improvement $(N_1)_{60}$ value for a single SPT boring was specified as 23 with the minimum individual value set at 18. The average initial blow count for the site was 16 with the average initial blow count for SM/ML layers being 18.

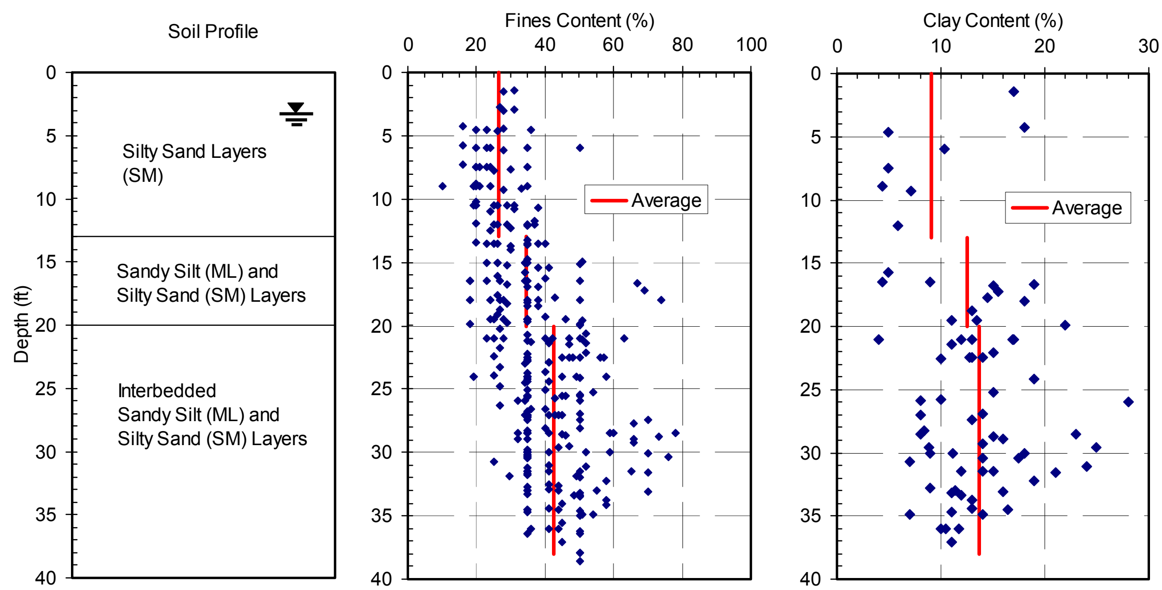


Figure 3-1 Idealized soil profile along with fines content and clay content profiles for the 24th Street Bridge case history.

3.2 Treatment Method

The main concern regarding the mitigation of the liquefiable zones was the amount of non-plastic fines in those areas. The fines contents ranged from about 34 to 62% in susceptible layers encountered in the pre-treatment borings. RB&G recommended and UDOT approved the vibro-replacement method of stone column installation for liquefaction mitigation. Based on previous experience at the Cherry Hills and Shepard Lane sites, RB&G and UDOT chose to supplement stone columns with wick drains in order to mitigate the liquefaction hazard posed by the sandy silts and silty sands. Wick drains were utilized in order to reduce pore pressures and to increase the densification effects of the vibro-stone columns in soils with high fines. The contractor hired to perform the soil improvement at the site was Hayward Baker and soil improvement was finished by 2007.

As part of this research study, the initial mitigation efforts (Phase I) focused on a test area with the intent of determining the effectiveness of utilizing wick drains in concert with stone columns. The test area was divided into four sections with stone columns alone installed in two of the sections and stone columns with wick drains installed in the remaining two sections. Phase II consisted of installing stone columns throughout the remainder of the site. Further investigation of the best treatment method included increasing the diameter of the stone columns in one location instead of installing wick drains; however, it was determined that the wick drains were more effective so the remainder of the site had wick drains installed in addition to the stone columns. The change in stone column diameter instead of utilizing wick drains provides a unique comparison of three different stone column treatment approaches at the same project site. The test area is believed to be the first and only direct comparison of the stone column treatment with and without wick drains at the same site.

Stone columns were installed in a center to center equilateral triangular spacing of 6.5 feet. Stone column diameters were approximately 3.5 feet. The stone column layout resulted in an area replacement ratio (A_r) of 26%. Wick drains were installed equidistant between any two columns prior to column installation. The stone column and wick drain layout for the test area is shown in Figure 3-2. The one section of the site that utilized an increased stone column diameter of approximately 4.5 ft had an area replacement ratio of about 34%. The remainder of the site was treated according to the layout of the wick drain sections of the test area. Stone columns and wick drains were installed to a depth of 40 feet. Stone columns were installed using the dry, bottom-feed approach. A Keller System S23120 vibrator (380 Volts, 1775 rpm) was used to install the columns. Maximum amperage during installation ranged from 150 to 300 amps and the time for installation was typically about 30 minutes.

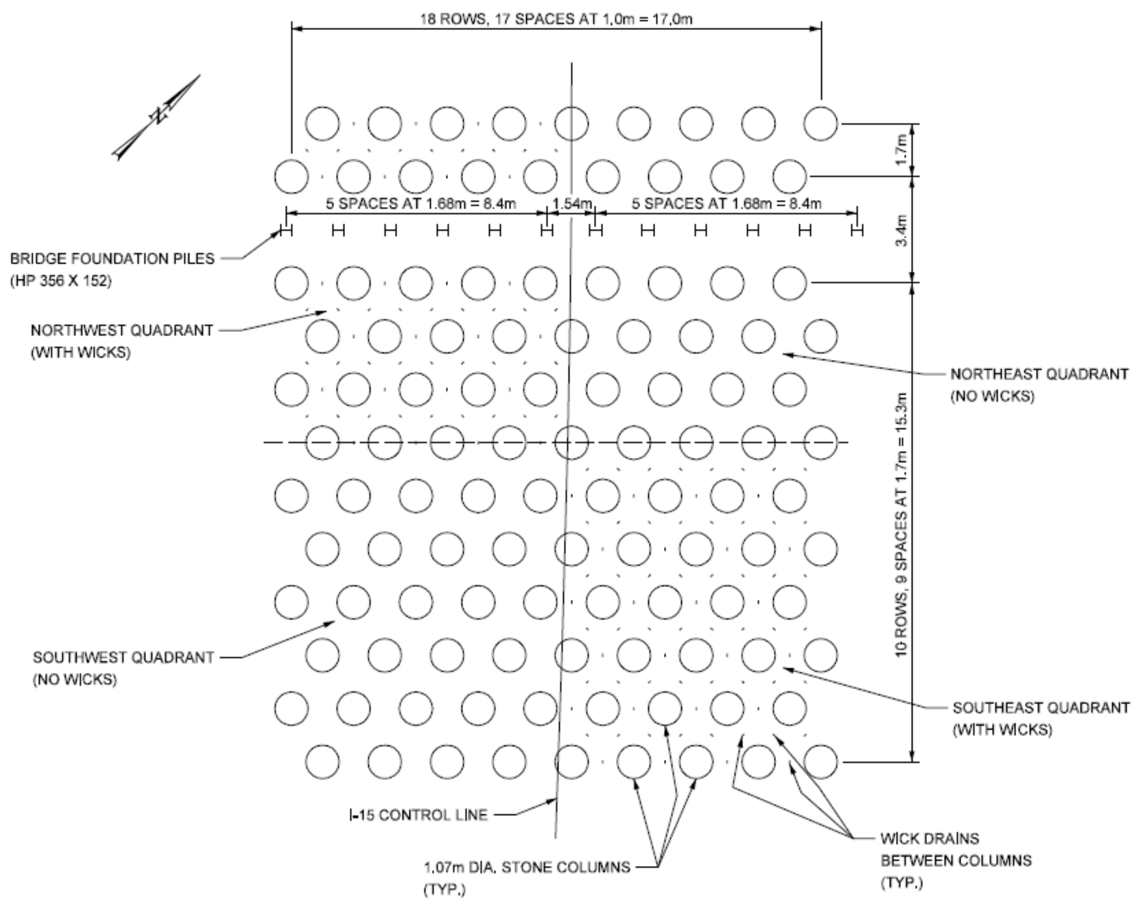


Figure 3-2 Layout of stone columns and wick drains in the test area at the 24th Street Bridge case history.

3.3 Results and Analysis

Due to the unique nature of the test site of the 24th Street Bridge case history, analyses were divided into two separate sections. The first section focuses on the test section of the site and the second section focuses on the additional information that was observed for the site as a whole.

3.3.1 Test Section Results and Analysis

Post-treatment testing was performed within 1-3 days after completion of stone column installations. Test holes were drilled in each of the four test areas with nearly continuous sampling as was done previously for the pre-treatment testing. Post-treatment test holes were located as close as possible to the pre-treatment test holes. Near continuous sampling helped identify potentially soft layers and their thicknesses. All of the test section data is for an area replacement ratio of 26%.

To examine the effectiveness of the drains, the initial and final blow counts versus depth for the treatment area test holes were plotted as shown in Figure 3-3 and Figure 3-4. The two test holes in areas without drains are shown in Figure 3-3, while the two test holes in areas with drains area shown in Figure 3-4. Data includes silts and sands with occasional silty clay data points. The silty clays were not removed for these figures (as was done in other case histories) because of the generally distinct SM/ML layer and because the clay points were typically individual points that do not seem to affect the data significantly (averages and trends remained essentially the same when they were removed). By including all of the data points the profiles are clearer and easier to visualize.

Initial post-treatment SPT borings were performed between 1-3 days following stone column treatment. Since there were several locations where the minimum criteria was not achieved it was decided to test again between 13-16 days in anticipation of gains in penetration resistance with time. Increases in blow count with time have previously been observed with other liquefaction mitigation methods (Mitchell and Solymar 1984, Schmertmann 1991). Figure 3-3 and Figure 3-4 both show the subsequent testing that took place following the initial post-treatment testing. One additional post-treatment SPT test was performed at 20 days for Boring 4 (see Figure 3-3) where minimum values had not been obtained. From a research standpoint it would have been desirable to conduct SPT testing over the entire treatment depth, but to minimize testing costs, re-testing was only performed in depth intervals where minimum blow counts had not been achieved.

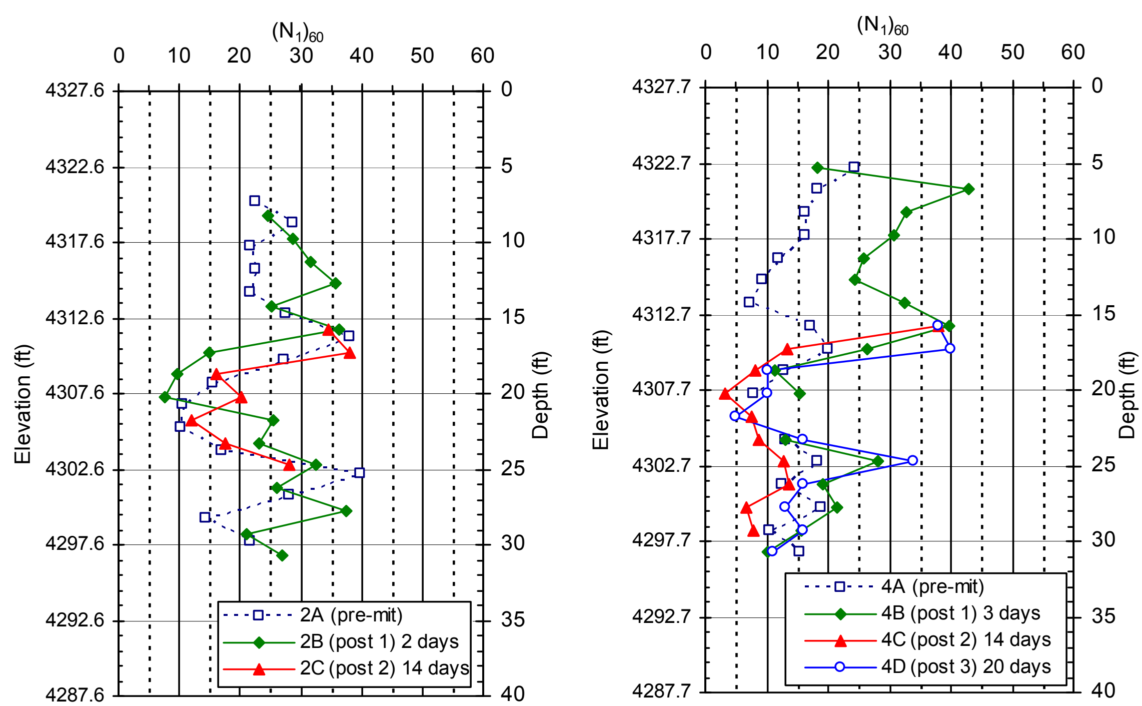


Figure 3-3 Results from SPT test holes 2 and 4 in the test areas (26% A_r) without wick drains at the 24th Street Bridge case history.

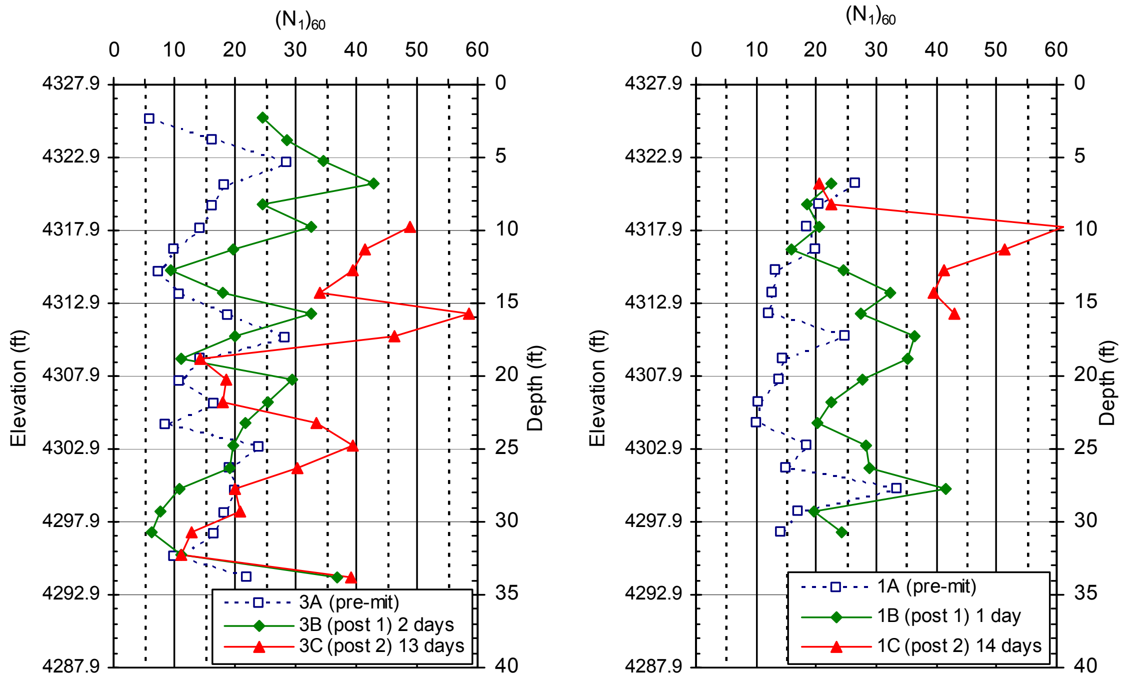


Figure 3-4 Results from SPT test holes 1 and 3 in the test areas (26% A_r) with wick drains at the 24th Street Bridge case history.

The minimum average SPT blow count of 23 was achieved in both the test area with drains as well as the test area without drains; however, there were a number of locations where the minimum blow count of 18 was not achieved. In the drained areas, 16% of the blow counts did not meet the minimum criteria of 18 as opposed to 44% in the areas without drains. Some of the low blow count layers were eliminated from further evaluation due to clay contents equal to or greater than 15%; however, the majority of the low blow counts were not able to be excluded due to high clay content.

Testing following the initial 1-3 day post-treatment testing generally did not yield consistent improvement as had been anticipated but instead resulted in a range of increased and decreased blow counts. The most consistent improvements with time after treatment were seen in the areas with wick drains, as shown in Figure 3-4. There were gains as well as losses although the areas with wicks did exhibit more consistent gains with time. Boring 4 in Figure 3-3 was the only location tested three times following improvement and was tested the longest after treatment at 20 days. The additional testing did not indicate consistent gains with time for the areas without drains. There was roughly the same number of points that yielded gains relative to prior testing as there were that had losses when compared to prior testing. Relatively consistent gains from increasing the time after improvement until testing were only noted in the areas with drains while the areas without drains did not exhibit any significant net gains overall.

The treatment was accepted although every data point did not meet the minimum blow count because of project provisions that allowed for reduced acceptance criteria in the test section. Nevertheless, all areas achieved the average final blow count minimum of 23. The areas with wick drains generally performed better than the areas without wick drains with an average final blow count of 32 in the drains areas and 25 in the no drains areas.

As noted, the average trend was that there was more improvement in the areas with drains than there was in the areas without drains. To examine this trend more closely, Figure 3-5 presents initial and final blow count profiles for all the data from the treatment area. Figure 3-5 uses all of the test area data and is divided into plots of (a) the areas with drains and (b) the areas without drains. Both plots show varying increases from initial blow counts to final blow counts. The drains data shows a consistent improvement of at least 6 while the no drains data shown little to no improvement in all areas except the 5-15 ft interval. There are even some negative improvements in the upper 5 ft of the soil profile.

The dashed lines in both plots indicate the minimum final blow count criteria of 18 blows per foot. There are significantly more final values below the minimum of 18 for the areas without drains than those with drains. This criterion is site specific but it indicates that when designing for liquefaction mitigation with stone columns at this site the addition of drains increased the success of the stone column treatment. Most engineers will be interested in achieving a final SPT or CPT value by which success is ultimately judged and from this standpoint the addition of wick drains is likely to be a success based on the average results from the 24th Street Bridge case history's average results. The final blow count criteria will vary by site but based on the data shown here, for a site with similar characteristics to the 24th Street Bridge case history (predominantly SM and ML soils, 34% fines content, and 13% clay content), additional improvements would be anticipated by the addition of wick drains to a vibro-stone column treatment plan..

To determine the factors affecting improvement, the change in blow count, $\Delta(N_1)_{60}$, was plotted versus fines content, initial $(N_1)_{60}$, and depth. The plot showing $\Delta(N_1)_{60}$ versus fines content is provided in Figure 3-6. The data indicates that improvement decreases as the fines content increases for both areas. However, the areas without drains appear to decrease much less than the areas with drains as fines contents increases. The curves also indicate that greater improvement is obtained with the drains for a given fines content, suggesting that the drains have a positive impact on improvement. The data suggests that the effectiveness of the drains is reduced as the fines content increases until the improvement is comparable at high fines contents. Unfortunately the data is too scattered to infer any direct relationships or to consider regression analyses and the R-squared values for both trend lines are below 0.22. It is probable that fines content explains only a portion of the variance in improvement and that initial blow count, clay content, and other factors not included in this figure may explain the remaining variation in improvement.

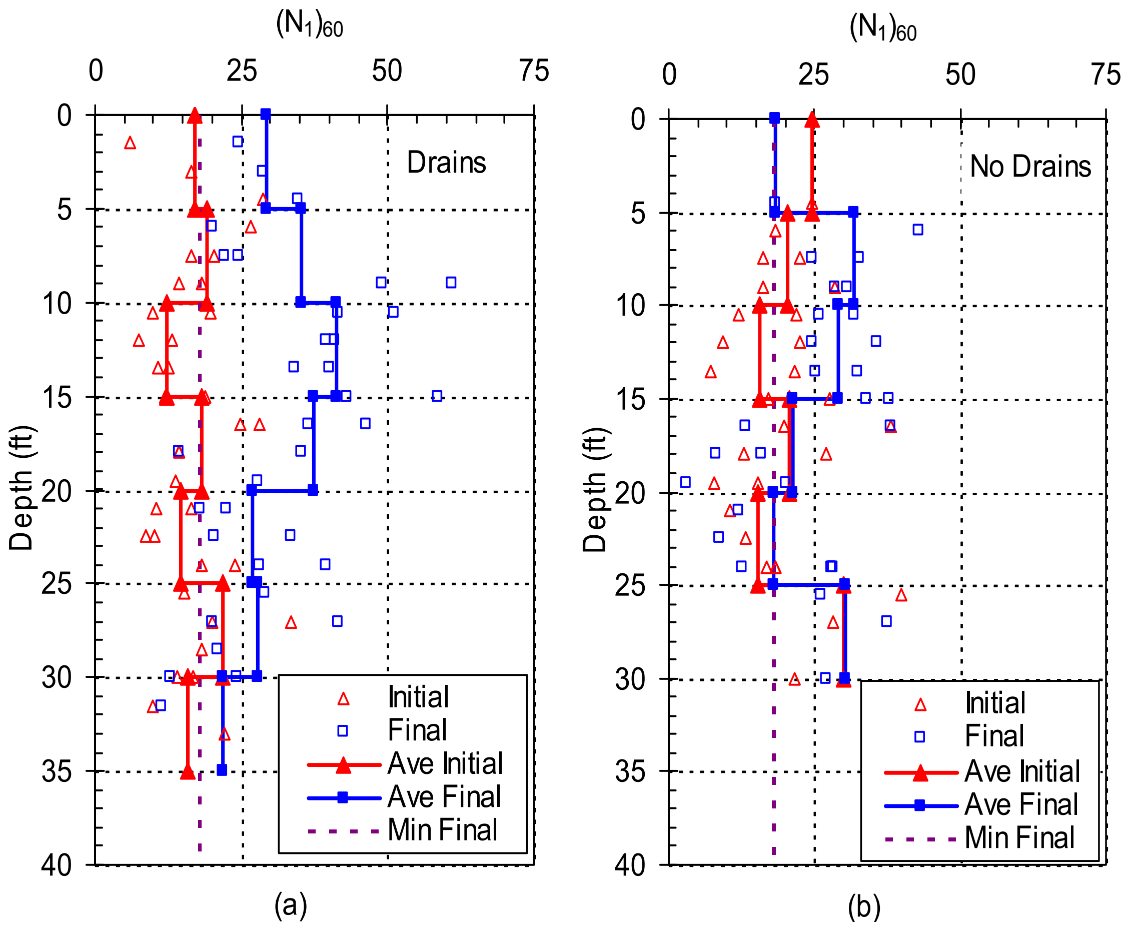


Figure 3-5 Initial and final blow counts versus depth for the test areas (26% A_r) (a) with drains and (b) without drains at the 24th Street Bridge case history. A dashed line indicates the minimum acceptable final blow count for the treatment plan.

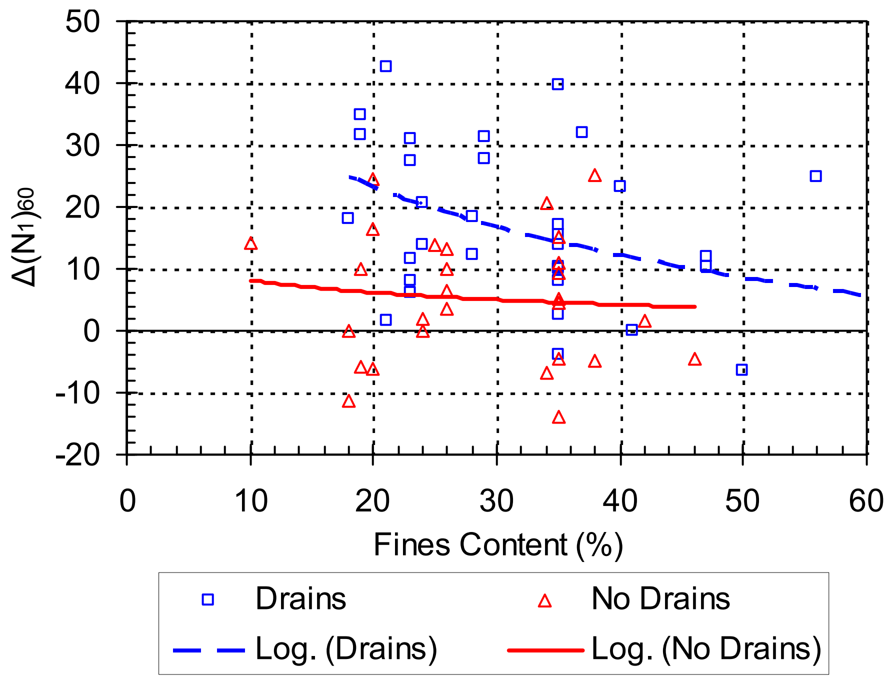


Figure 3-6 $\Delta(N_1)_{60}$ versus fines content for the test areas (26% A_r) with and without drains for the 24th Street Bridge case history.

$\Delta(N_1)_{60}$ versus initial $(N_1)_{60}$, is presented in Figure 3-7. The data points in the figure do not suggest any direct relationships between initial blow count and improvement. The data is very scattered and the R-squared values for the logarithmic regression lines are both less than 0.13. The data does confirm that there were more negative improvements within the areas without drains than there were in the areas with drains. The data indicates that negative improvements in the areas without drains were unaffected by increasing initial blow counts. The logarithmic trend lines also show that both areas exhibited greater improvement at low initial blow counts than at high initial blow. The trend is not surprising as it is expected that loose soils with low initial blow counts will improve more than dense soils with high initial blow counts. The shapes of the trend lines are remarkably similar with the trend line with drains being consistently about 10 blows above the trend line without drains.

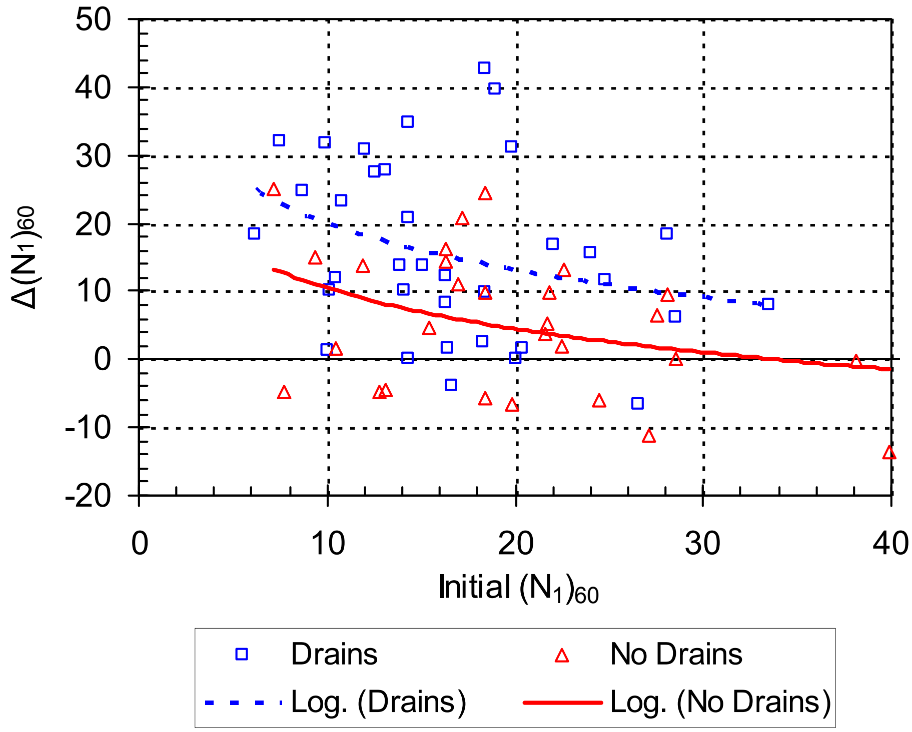


Figure 3-7 $\Delta(N_1)_{60}$ versus initial $(N_1)_{60}$ for the test areas (26% A_r) with drains and without drains at the 24th Street Bridge case history.

Depth versus $\Delta(N_1)_{60}$ is presented in Figure 3-8 below. The data points in the figure do not suggest any distinct relationship between depth and improvement. Nevertheless, the data does seem to indicate a general decrease in blow count with depth but the data are very scattered. The data show that there are more negative improvements in the areas without drains. This indicates that treatment was more effective in the areas with drains than in the areas without drains. The majority of the negative improvement points was in the no drains areas and was 15 feet or more beneath the surface. Fines content and clay content were noted to increase with depth at the site. It is possible that the areas with drains were less affected by increasing fines and clay contents in terms of negative improvements due to the presence of drains.

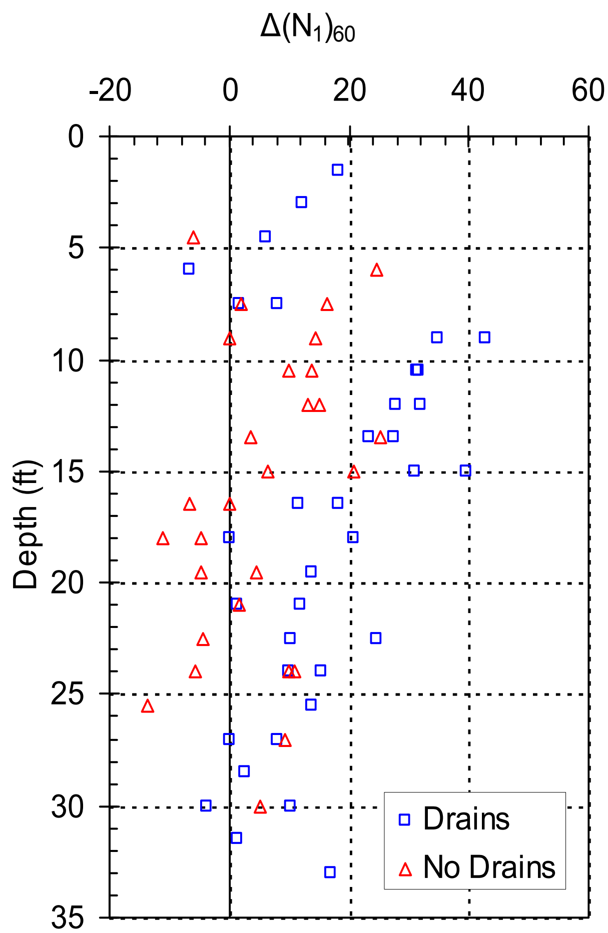
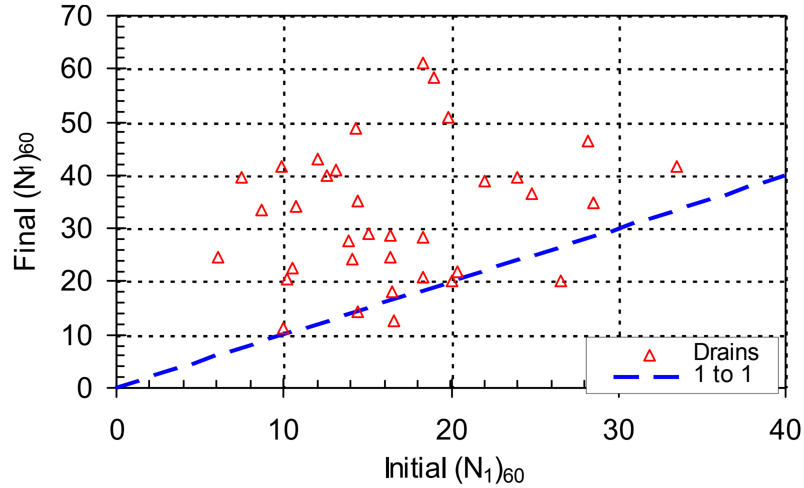
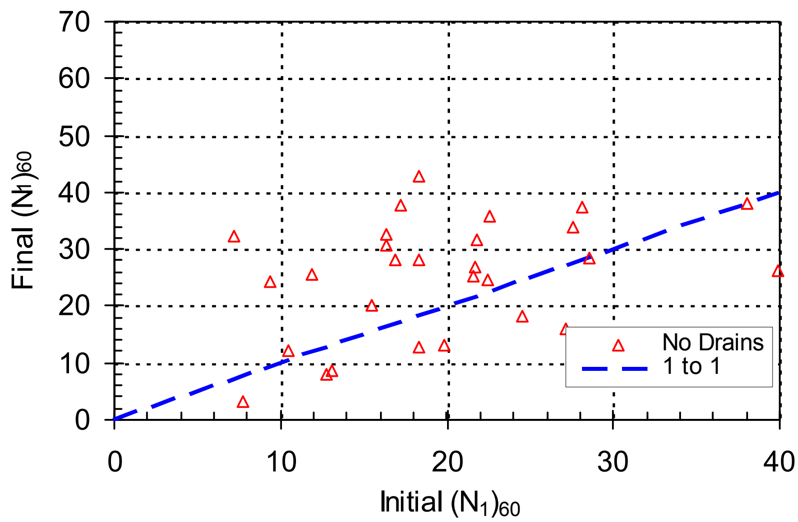


Figure 3-8 Depth versus $\Delta(N_1)_{60}$ for the test areas (26% A_r) with drains and without drains for the 24th Street Bridge case history.

As noted previously for this case history, final blow count was the measure of success for the mitigation efforts, as is often the case in similar case histories. Final $(N_1)_{60}$ versus initial blow $(N_1)_{60}$ is presented in Figure 3-9. Data above the 1 to 1 line indicate improvement. The data confirm that the areas with drains had fewer negative improvements and had fewer low (less than 20) final blow counts than areas without drains but there are no clear trends.



(a)



(b)

Figure 3-9 Final $(N_1)_{60}$ versus initial $(N_1)_{60}$ for the test areas (26% A_r) with drains (a) and without drains (b) for the 24th Street Bridge case history.

The overall average blow counts and fines contents as well as the number of points below the minimum criteria for the test area are shown in Table 3-1. Both areas had similar initial blow counts (≈18) while the fines content in the drains area was 35% which was slightly higher than that in the no drains area where the fines content was 29%. The areas with drains improved their average final blow count by 11 more than the areas without drains despite the higher average fines content. Most sites using the stone column treatment establish a minimum post-treatment SPT or CPT value by which to establish the success of their liquefaction mitigation efforts. As mentioned previously, the minimum final value for the 24th Street Bridge was 18. There were 7 values (25%) that did not pass this minimum from the areas without drain while there were only 4 values (11%) that did not pass from the areas with drains despite post-treatment testing which considered gains in penetration with time. Based on the increase in the average improvement and the relatively small number of points that did not meet the minimum final blow count criteria, the addition of drains to the stone column treatment plan was a success. The areas with drains were more effective on average than the areas without drains. Phase II included a trial area with increased column diameters and no drains which will be discussed later; however, based on their performance in the test section, drains were ultimately used at the remainder of the site.

Table 3-1 Test area (26% A_r) average blow counts, fines contents, and other values for the 24th Street Bridge case history.

	Average $(N_1)_{60}$				Ave. Fines Content (%)	Values Below Required	Sample Size	Standard Dev. (Final)
Test Area	Initial	Final	Change	Increase				
Drains	17	32	16	94%	35	4	35	12.5
No Drains	20	25	5	27%	29	7	28	10.3

3.3.2 Overall Site Results and Analysis

To further understand the effectiveness of the wick drain treatment at the site and to investigate the effect of time after treatment, the data for the remainder of the site were added to the test section data and the results and analyses are presented below. The data for the areas with no drains and 26% A_r remained the same since the remainder of the site all had drains installed, except the area with a 34% A_r. The area with the increased column diameters and no drains will be compared to the drains data for the entire site. The information for the areas with drains changed somewhat with the addition of the remaining data; however, the general trends usually remained the same. Some of the averages varied and additional clay content information made it possible to do further analyses. In instances where the general trend remained the same, the discussion of the figures and tables will be minimal and the reader will be referred to the more detailed discussion in the Test Section Results and Analysis section. The additional data also made it possible to investigate the time after improvement effect in more detail.

The test section results indicate that areas with drains exhibited some improvement in final blow counts with increased time between treatment and testing. The initial post-treatment testing in the test section was done 1-3 days following treatment. Subsequent testing was done following the initial post-treatment testing in hopes of gains with time. Based on the positive results of the areas with drains in the test section, the remainder of the site was tested 6-13 days after treatment instead of 1-3 days after.

Initial and final blow count profiles for the additional SPT test holes at the site are shown in Figure 3-10 to Figure 3-12. The dashed line in the figures for $(N_1)_{60}$ = 18 indicates the minimum acceptable final blow count for the project. The data shows an increase in blow count for most of the points with some decreases in blow count evident. By the project provisions, samples with clay contents greater than 15% were exempt from the minimum blow count criterion. The data with clay contents greater than 15% are indicated with an "X" through the data point. The data show that a majority of the post-improvement blow counts that fell below the minimum blow count of 18 were classified as having high clay content (>15%). Final blow counts below 18 occurred throughout the soil profile but there were a high number of occurrences in the bottom 5-10 feet of the profile. Much of the data with high clay content were also in this zone. Increasing fines and clay contents may have limited improvement in these zones.

The data for the entire site is plotted as depth versus initial and final blow count in Figure 3-13. The data is very similar to that of the test area (Figure 3-5) and there is consistent improvement in the areas with drains and as well as decreasing improvement with depth. Decreasing improvement with depth may be due to increasing fines and clay contents with depth.

$\Delta(N_1)_{60}$ versus fines content was plotted in Figure 3-14 for the entire site with the data being split into series based on areas with drains and areas without drains. Logarithmic trend lines are shown for both series. With the additional site information there was enough clay content data to indicate the data points with high fines content (>15%) with an open box around the data point.

Logarithmic trend lines indicate similar trends to those observed in the test section (Figure 3-6). Scatter is significant for all of the data but it is particularly large for low fines contents in the areas with drains. Due to scatter, the trend lines are representative of the average values across the site instead of an actual prediction of improvement based on fines content. Data with high fines content exhibit less scatter and smaller improvements. High clay contents are prevalent in the data with little improvement although there were data with large improvements among the high clay content data. The improvement for the no drains data seems to be relatively constant regardless of fines content. The reason for this trend is not fully understood.

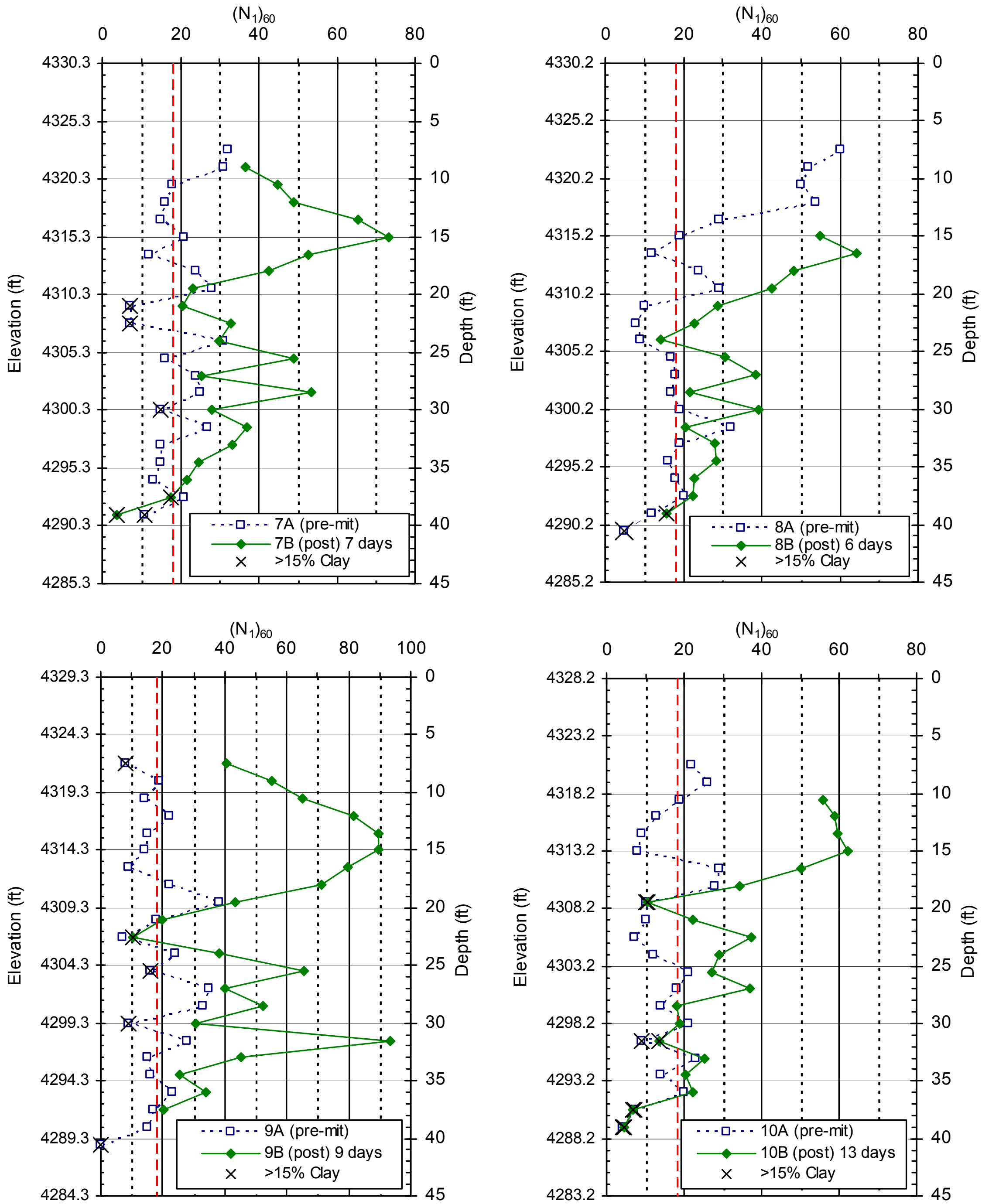


Figure 3-10 Results from SPT test holes 7-10 in 26% A_r areas with wick drains outside of the test section at the 24th Street Bridge case history. The dashed line indicates the project's minimum final blow count criterion for acceptance of treatment.

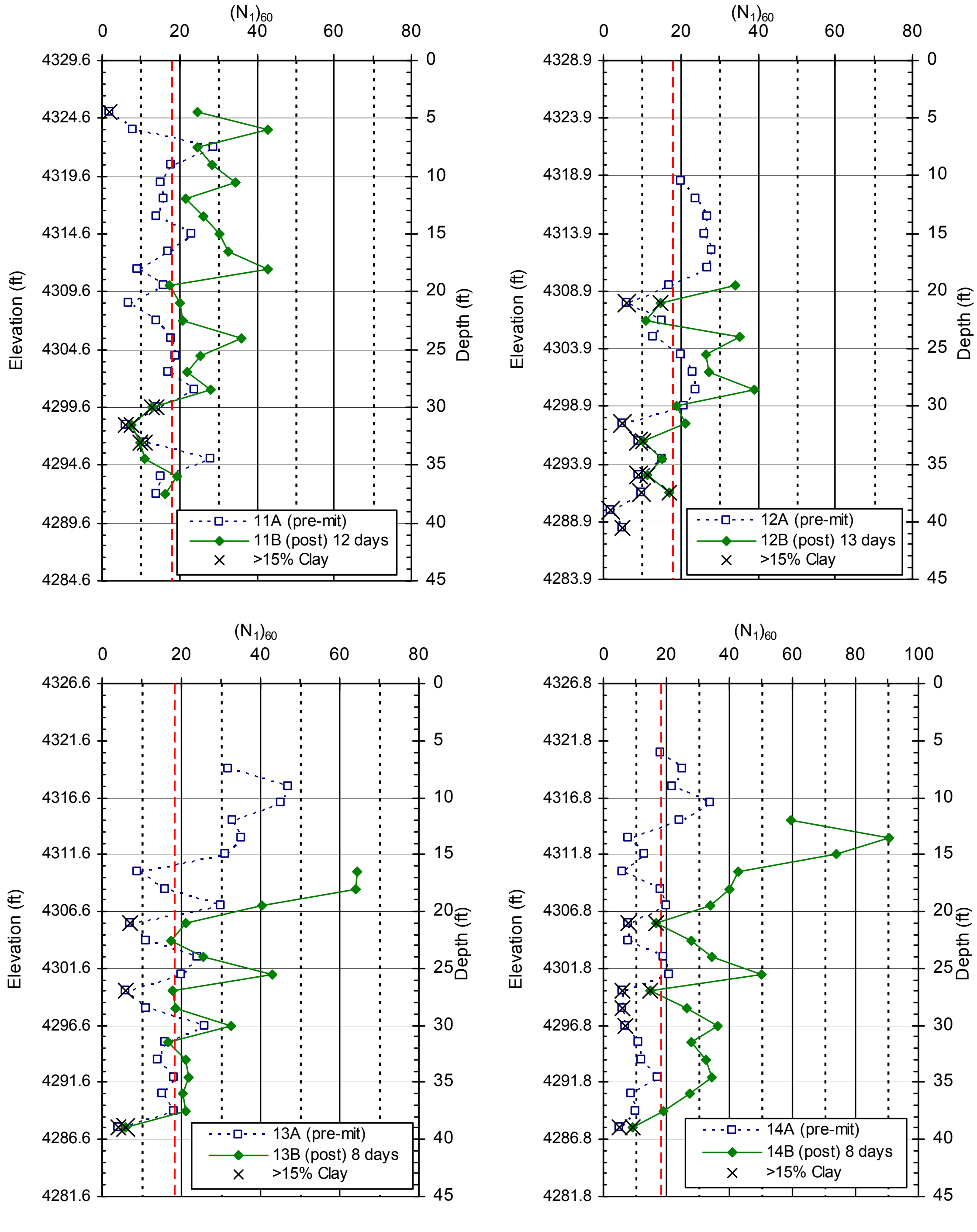


Figure 3-11 Results from SPT test holes 11-14 in 26% A_r areas with wick drains outside of the test section at the 24th Street Bridge case history. The dashed line indicates the project's minimum final blow count criterion for acceptance of treatment.

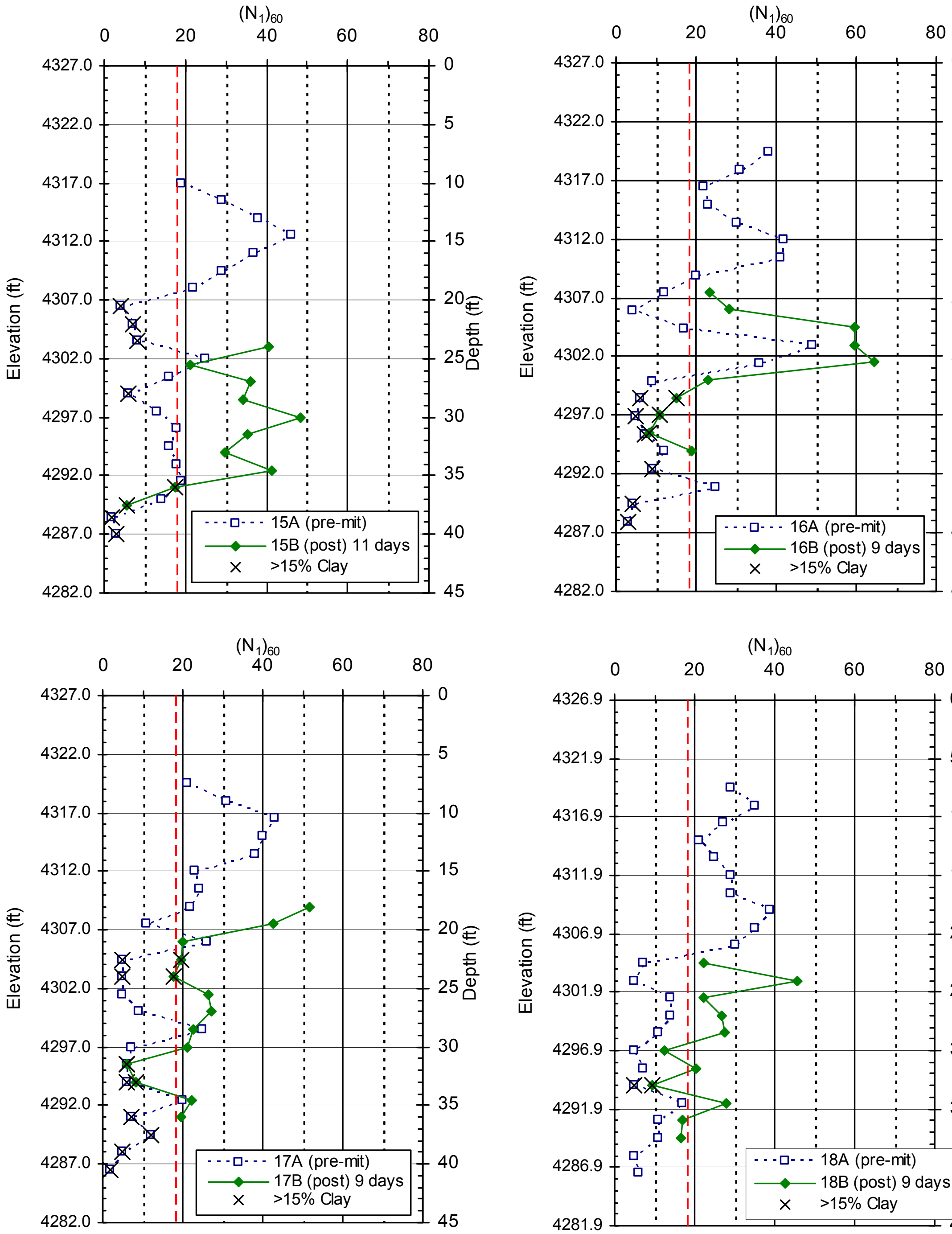


Figure 3-12 Results from SPT test holes 15-18 in 26% A_r areas with wick drains outside of the test section at the 24th Street Bridge case history. The dashed line indicates the project's minimum final blow count criterion for acceptance of treatment.

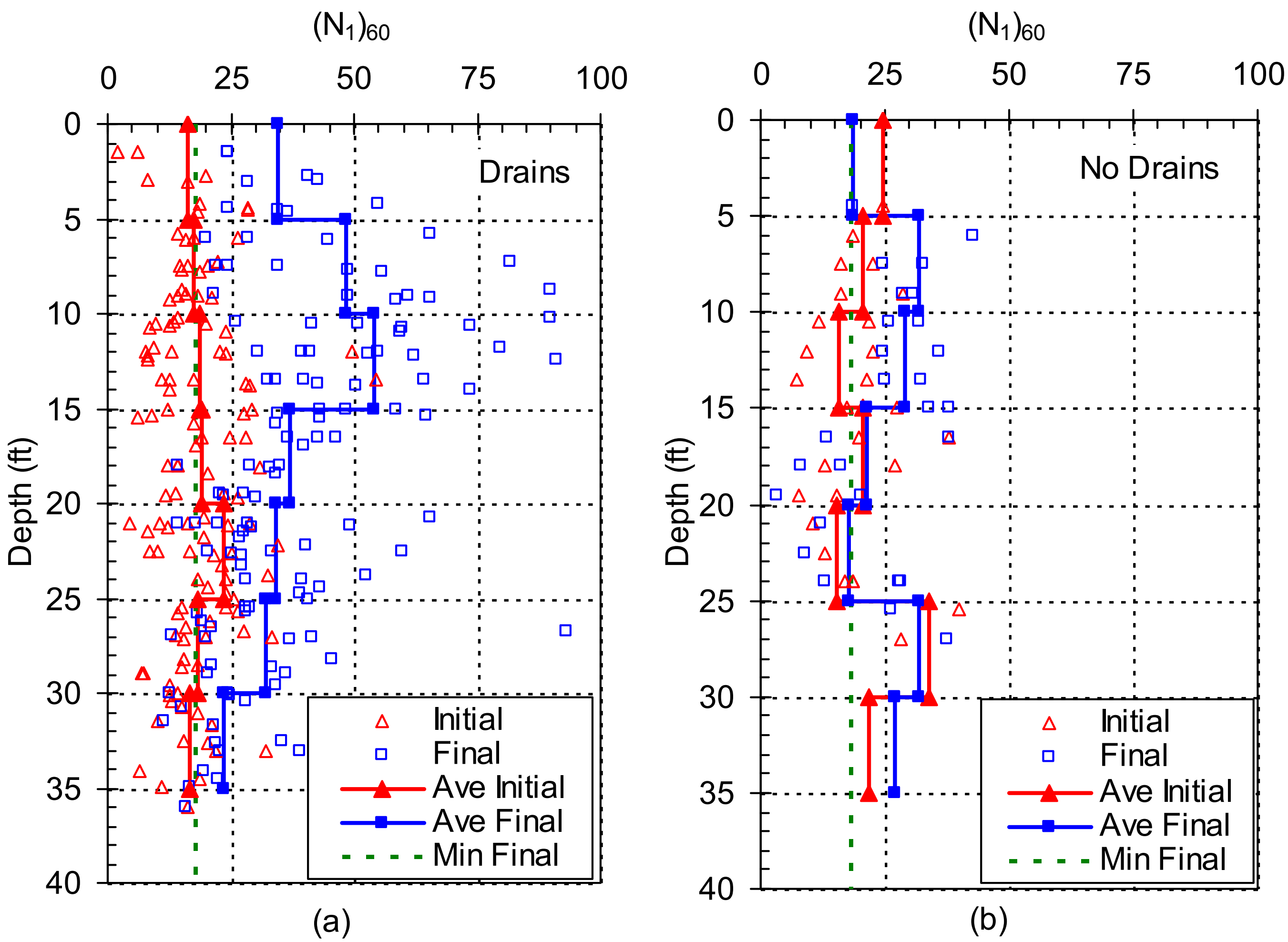


Figure 3-13 Initial and final blow count profiles for all of the 26% A_r areas at site split into (a) areas with drains and (b) areas without drains at the 24th Street Bridge case history. A dashed line indicates the minimum acceptable final blow count for the treatment plan.

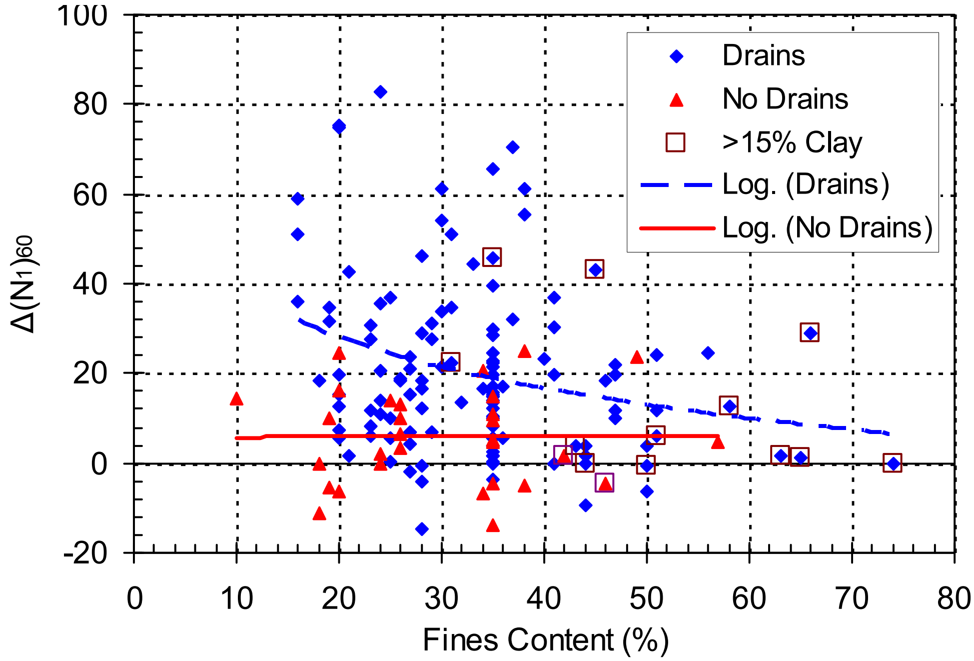


Figure 3-14 $\Delta(N_1)_{60}$ versus fines content for the entire site in 26% A_r areas with and without drains for the 24th Street Bridge case history.

$\Delta(N_1)_{60}$ versus initial $(N_1)_{60}$ is plotted in Figure 3-15. The trend appears to be nearly the same as that observed for the test area data (Figure 3-7) except that the drains data values are higher for the entire site, especially at low initial blow counts. The trend line for the drains data is approximately 26 for an initial value of 10 in Figure 3-15 while the test area trend line is only about 10 at the same location. Both decrease with increasing initial blow count but the test area data does so more rapidly. The difference is attributed to the increase in the amount of data available for the entire site versus the test area. The more data there is, the more likely it is to represent the average and nullify outliers in the data. The higher values for the entire site data are therefore considered more accurate.

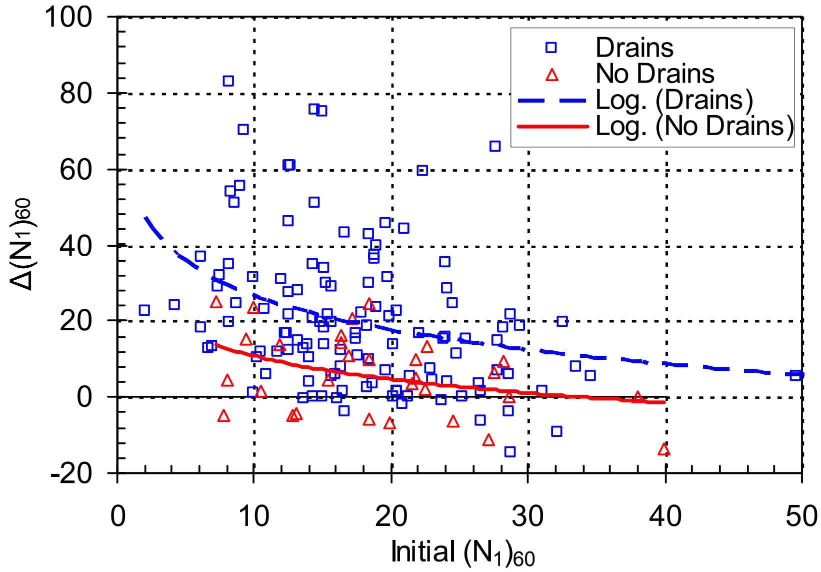


Figure 3-15 $\Delta(N_1)_{60}$ versus initial $(N_1)_{60}$ for the entire site in 26% A_r areas with and without drains for the 24th Street Bridge case history.

Once again the improvement decreased for the drained areas as the initial blow count increased. To investigate this trend and simultaneously consider the increasing fines and clay contents with depth, the drains data for the entire site are plotted a second time in Figure 3-16 with series represented data above 20 feet and below 20 feet. The soil profile was noted to change at around 20 feet as seen in the idealized soil profile in Figure 3-1. Data points with clay contents greater than 15% are marked with an empty square box around them. Separating the data as done in Figure 3-16 is the most representative improvement of the drains data since it attempts to account for initial blow counts as well as fines content and clay content trends simultaneously. Above 20 feet the average fines content was 31% and the average clay content was 11%. Below 20 feet the average fines increased to 43% while the average clay content increased to 14%.

Both layers exhibited decreasing improvement with increasing initial blow count in Figure 3-16. The data in the upper layer showed greater improvement than the lower layer. For an initial blow count of 10, improvement in the upper layer was almost 20 blows higher than the improvement in the lower layer. The gap between the two trend lines narrowed to about 7 at an initial blow count of 30. The data with high clay contents predominately exhibited improvements less than 10. The data indicate that increasing fines content and clay content significantly decrease improvement by as much as 7-20 blows per foot in areas with drains. Although there were several negative improvements, the trend line for the upper layer data indicates average improvements of 10 or more depending on the initial blow count. The data is too scattered (R-squared values less than 0.18) to use regression to predict an equation for improvement, but the overall trends are significant and useful to a practicing engineer who is considering using drains with a vibro-stone column treatment plan.

For the sake of comparison, $\Delta(N_1)_{60}$ is plotted as a function of depth in Figure 3-17. There are no differences worth mentioning when compared to the test site data in Figure 3-8. There is more data but the data do not indicate any trends that differ from those mentioned previously.

Figure 3-18 provides a plot of final blow count versus initial blow count for all of the drain data for the site to allow comparison to the test section data plotted in Figure 3-9. There are more positive and negative improvements, as indicated by values plotting above and below the 1 to 1 line respectively. The measured average and plus and minus one standard deviation lines are shown. The data is still very scattered, the standard deviation is large (18) and no new trends are apparent. A second figure presenting final blow count versus initial blow count is shown in Figure 3-19.

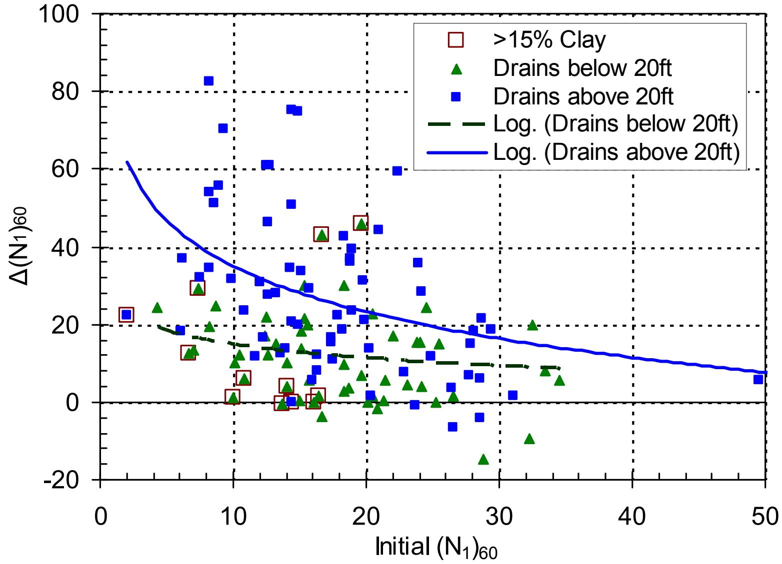


Figure 3-16 $\Delta(N_1)_{60}$ versus initial $(N_1)_{60}$ for the 26% A_r areas with drains divided by data above 20 feet and data below 20 feet for the 24th Street Bridge case history. Empty squares indicate data with clay contents greater than 15%.

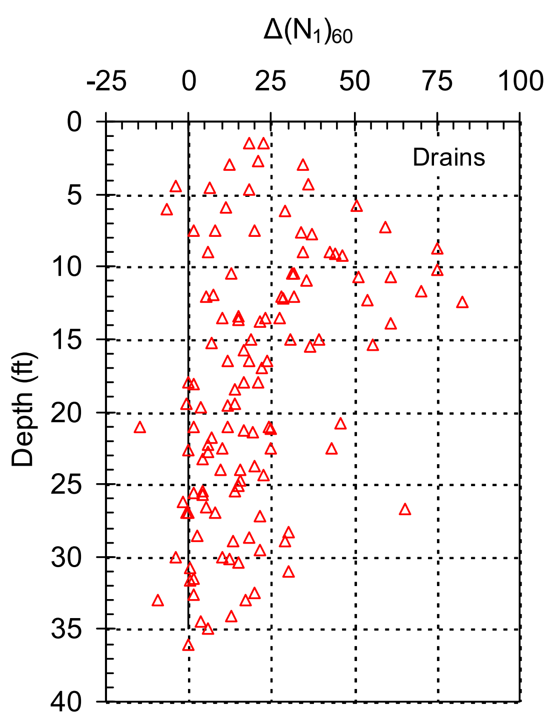


Figure 3-17 Depth versus $\Delta(N_1)_{60}$ for the 26% A_r data with drains for the 24th Street Bridge case history.

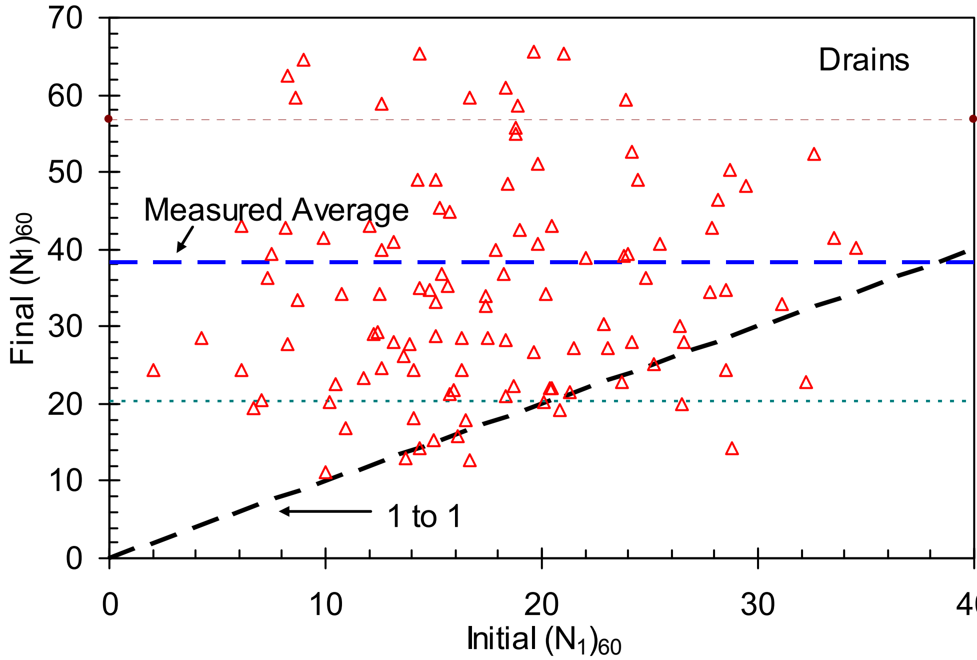


Figure 3-18 Final $(N_1)_{60}$ versus initial $(N_1)_{60}$ for the 26% A_r data with drains at the 24th Street Bridge case history. The measured average final blow count as well as plus and minus one standard deviation lines are shown in addition to a 1 to 1 line.

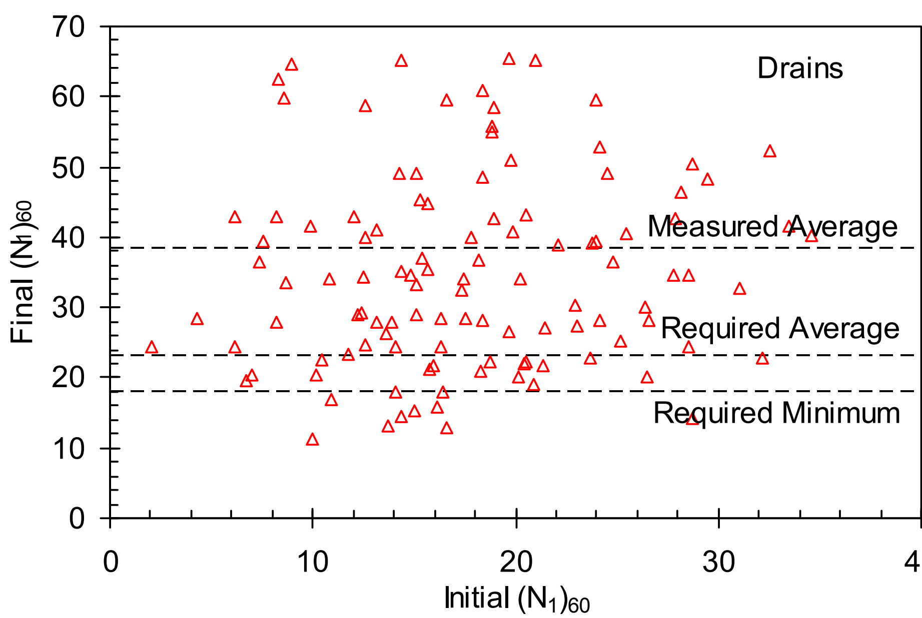


Figure 3-19 Final $(N_1)_{60}$ versus initial $(N_1)_{60}$ for the 26% A_r data with drains at the 24th Street Bridge case history. The measured average final blow count as well as the required final average and final minimum blow counts are indicated by dashed lines.

Figure 3-19 shows the average final blow count line as well as the required minimum and average blow count criteria for the site. The treatment succeeded in producing final blow counts above the minimum final criterion (93% passing) and the measured average of 38 was significantly higher than the minimum average of 23.

Average values for the entire site are presented in Table 3-2 below. The data were split into areas with drains and areas without drains, both above and below 20 feet. The data above 20 feet had an average fines content of about 28%, an average clay content of about 11%, and an average initial blow count of about 19 for both the areas with and without drains. Although the initial site conditions were roughly equivalent, the areas with drains experienced an average increase in blow count of 148% while the no drains areas only increased 35%. Both areas had relatively high final blow counts though and initially it appears that, based on the average minimum final blow count of 23 prescribed for the site, drains might not have been needed to meet the treatment objectives in this layer; however, as seen in the test section, the areas without drains still had more blow counts below the minimum individual final blow count of 18. The standard deviations for the final blow counts in all categories were high, between 40-50% of the average final blow counts.

Between 20-40 feet the average fines content increased to about 40% in the drains areas and 32% in the no drains areas. The average clay content increased to about 14% in both areas. The initial blow count stayed at 18 in the drains areas and increased to 21 in no drains areas. The drains area exhibited the most improvement despite the higher fines contents. Therefore, the results present a conservative picture of the beneficial effect of the drains. In the 20-40 ft layer the difference between the areas with drains and without drains was very distinct, with average improvement in the areas with drains at 69% as opposed to only 8% in the areas without drains. In regards to final blow count, it is noted that the project acceptance criterion of a minimum average of 23 was only barely met by the areas without drains (average of 23) while the areas with drains clearly passed with an average of 30. There were very few data points (only 8) in the no drains areas for this layer though so these results could be considered less reliable and should not be relied upon heavily; however, the improvement in the drains areas was consistent across the entire site.

Table 3-2 Averages of data analyzed for all of the 26% A_r areas including blow counts, fines contents, and the number of values used at the 24th Street Bridge case history.

Depth Interval	Drainage State	Ave. % Fines	Ave. Initial $(N_1)_{60}$	Ave. Final $(N_1)_{60}$	Ave. % Increase in $(N_1)_{60}$	Sample Size	Standard Deviation (Finals)
0-20 ft	Drains	29	18	45	148	67	18
0-20 ft	No Drains	27	19	26	35	20	10
20-40 ft	Drains	40	18	30	69	55	15
20-40 ft	No Drains	32	21	23	8	8	10

It would be desirable to include more data from areas without drains in order to make sure the values are representative of the site as a whole; however, considering that there were only two small areas within the test section that did not have drains it is still an appropriate representation of the site. The data clearly indicate that layers with higher fines content experience less improvement on average. The areas with drains were shown to experience almost twice the improvement of the areas without drains in both areas with fines content of 29% as well as areas with fines content of 40%. The addition of wick drains to the vibro-stone column treatment plan was very effective in increasing the blow counts for the site with its overall average fines content of 34% and clay content of 13%.

The effect of time on improvement was mentioned previously in the Test Section Results and Analysis. It was observed that there were not typically consistent gains with time and that in several cases there were losses with time. The areas with drains appeared to have more zones with gains; however, it was not consistent and there were only two test holes to analyze. With the addition of the rest of the site data the effect of time on improvement was examined more thoroughly.

The test results from the entire site were separated into two time intervals (1-6 days following treatment and 7-14 days following treatment) and plotted versus the pre-treatment blow count. Other time after treatments existed but only the most common times, as noted above, were utilized in this analysis. The test results were also analyzed separately for test areas with and without wick drains. The results for the areas with drains were plotted in Figure 3-20, while the results for areas without drains were plotted in Figure 3-21.

The drains plot shows significant scatter in the data which is more severe for the 7-14 day interval than for the 1-6 day interval. The no drains plot also shows some scatter. The R-squared values are all less than 0.12 except for the no drains, 7-14 days after treatment series which had an R-squared value of 0.4942. The scatter can, at least partially, be attributed to the variation in fines content and clay content which are not accounted for in these plots. Despite significant scatter in the test results, the trend lines show an average increase of 10 to 20 blows for the 7-14 day time interval relative to the 1-6 day time interval where drains were present. The improvement with time is greater for the soils with lower pre-treatment blow counts. In contrast, the data for test sites without wick drains show no increase with time after treatment, but actually show a decrease. The decision to initially test the data between 6-13 days following treatment instead of 1-3 days following treatment was successful since there was no need to retest any of the treatment area a second or third time following initial post-treatment testing.

The scatter is very significant with final blow counts varying from approximately 10 to 80 in the 7-14 days drains data. Due to the significant scatter, a definitive conclusion cannot be reached although on average increasing the time between treatment and testing from 3 days to 10 days in areas with drains typically yielded an average increase of about 10 blows per foot in penetration resistance.

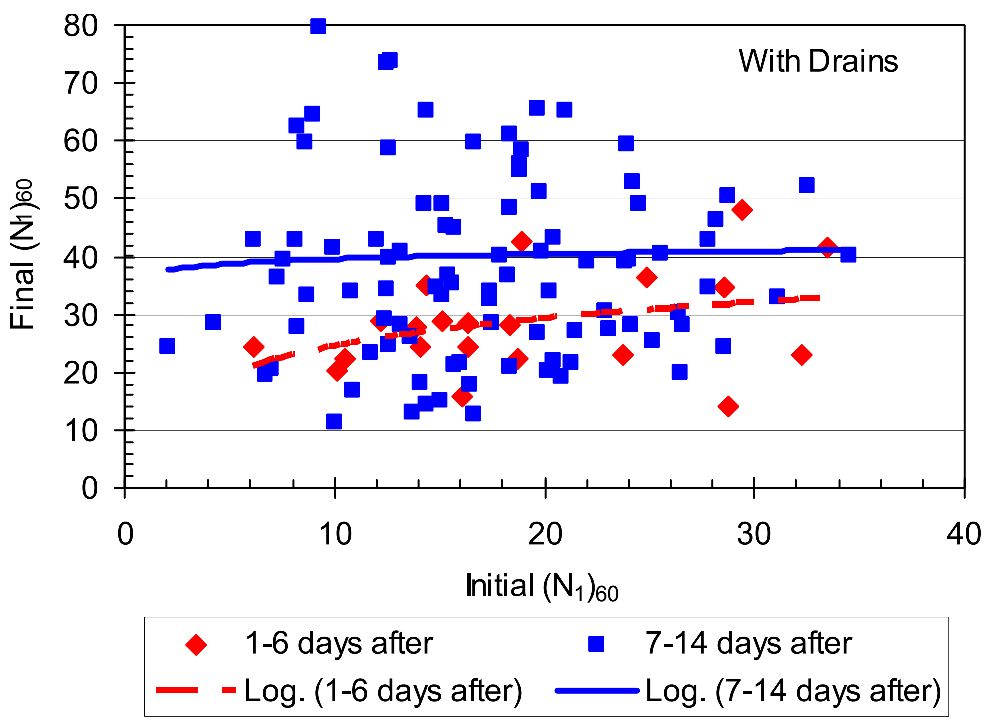


Figure 3-20 Final $(N_1)_{60}$ versus initial $(N_1)_{60}$ for the 26% A_r areas with drains split into two time intervals after stone column treatment at the 24th Street Bridge case history.

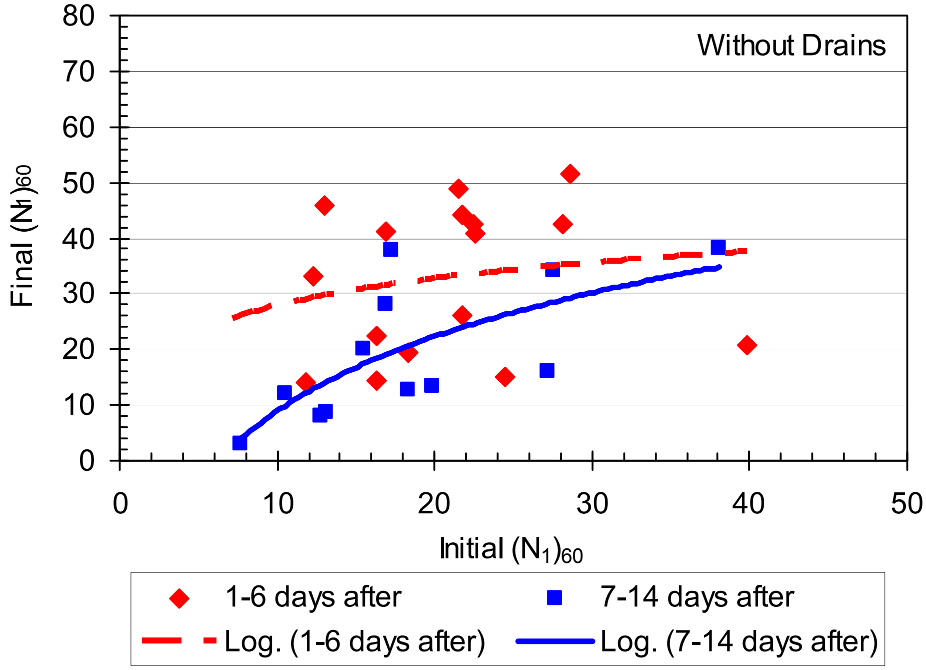


Figure 3-21 Final $(N_1)_{60}$ versus initial $(N_1)_{60}$ for the 26% A_r areas without drains split into two time intervals after stone column treatment at the 24th Street Bridge case history.

The one section of the site without drains where the A_r was increased to 34% provides a comparison of the increased effectiveness of the wick drains versus additional stone columns. The test section clearly showed that a general increase in performance is to be expected with the addition of wick drains at sites similar to the 24th Street Bridge case history. The question as to whether adding wick drains or simply increasing the replacement ratio would be more effective is now addressed. Figure 3-22 presents the SPT testing results for test holes 5 and 6 which were located in the 34% A_r areas without wick drains.

The data shows general improvement with some low or negative improvements at the lower boundary of the treatment zone. This is expected to be due to the influence of increasing clay contents and clay layers. The treatment may also be less effective at the lower boundary since the vibratory installation of the stone column may not be as effective in densifying soils right at the lower boundary of the treatment depth. Boring 5 was tested twice following treatment but there did not seem to be any consistent gains with time between the first and second post-treatment testing. The majority of the zones that showed improvement did typically have quite large improvements in blow count. There were multiple points with increases greater than 20.

To compare the three different treatments from the site, the initial blow count is plotted versus the final blow count for each category in Figure 3-23. Each series has a corresponding logarithmic trend line which indicates the general trend of the data, although there is significant scatter in the data. The scatter in the data is the most pronounced for the drains data, as noticed previously.

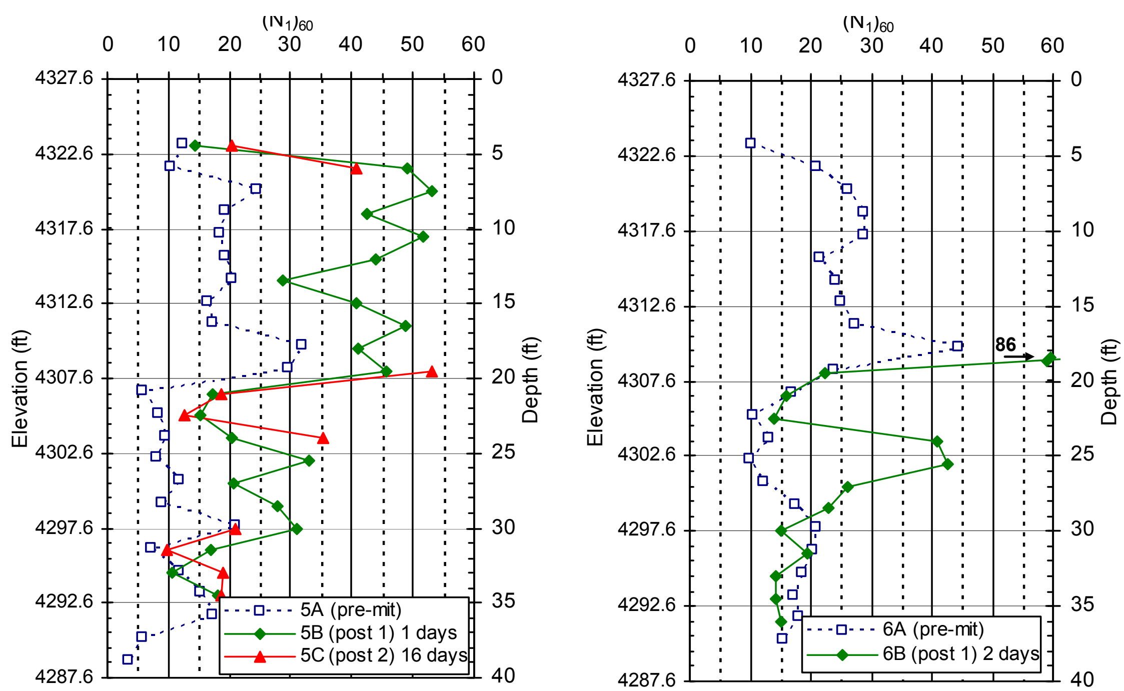


Figure 3-22 Results from SPT test holes 5 and 6 in the 34% A_r areas without drains at the 24th Street Bridge case history. These test holes were located outside of the test section. The dashed line indicates the project's minimum final blow count criterion for acceptance of treatment.

The data in Figure 3-23 shows that increasing the diameter of the stone columns to produce an A_r of 34% was more effective than the primary arrangement with an A_r of 26%. The improvement increased as the initial blow count increased. The increased improvement is good; however, the most important soil to improve is the soil with relatively low initial blow counts. The data for sites with drains shows even better improvement than was achieved with a 34% A_r using no drains except at high initial blow counts. It is notable that the improvement produced with drains was much better than the 34% A_r data at low blow counts, where it is the most important. The drains data had the most scatter but also the drain data accounted for almost all of the final blow counts above 50. There was not as much data for the 34% A_r areas as there was for the drains areas; however, the data indicates that adding drains to the stone column treatment was more effective than increasing the area replacement ratio overall. This is why RB&G elected to use wick drains for the majority of the site.

Multiple linear regression was performed for the data in an attempt to simultaneously account for the contribution of both fines content and initial blow count to the final blow count. Previous attempts at regression using linear or logarithmic trend lines produced very low R-squared correlation values. Likewise, multiple regression using fines content and initial blow count to predict final blow count produced a multiple R-squared regression value of only 0.3762. The results of the multiple linear regression are presented in Figure 3-24 with measured final $(N_1)_{60}$ values versus predicted final $(N_1)_{60}$ values and a one to one line. Multiple regression appears to generally predict an average final values to be about 40 with scatter between 30 and 50. The accuracy of the regression is very poor.

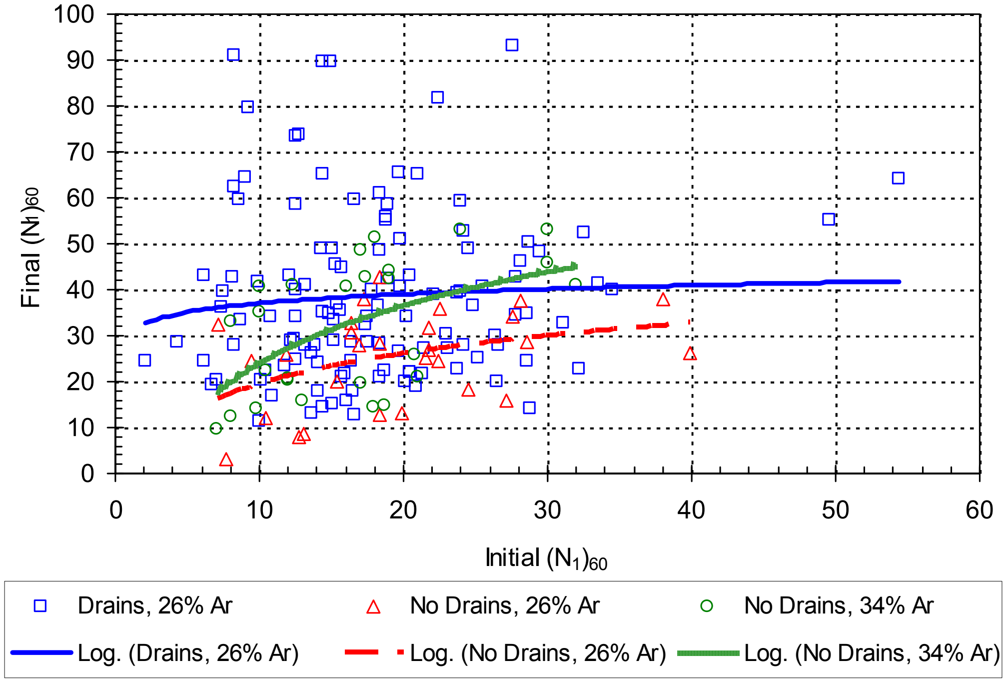


Figure 3-23 Final $(N_1)_{60}$ versus initial $(N_1)_{60}$ for the 26% A_r areas with and without drains as well as the 34% A_r area without drains at the 24th Street Bridge case history. Logarithmic trend lines are shown for each series.

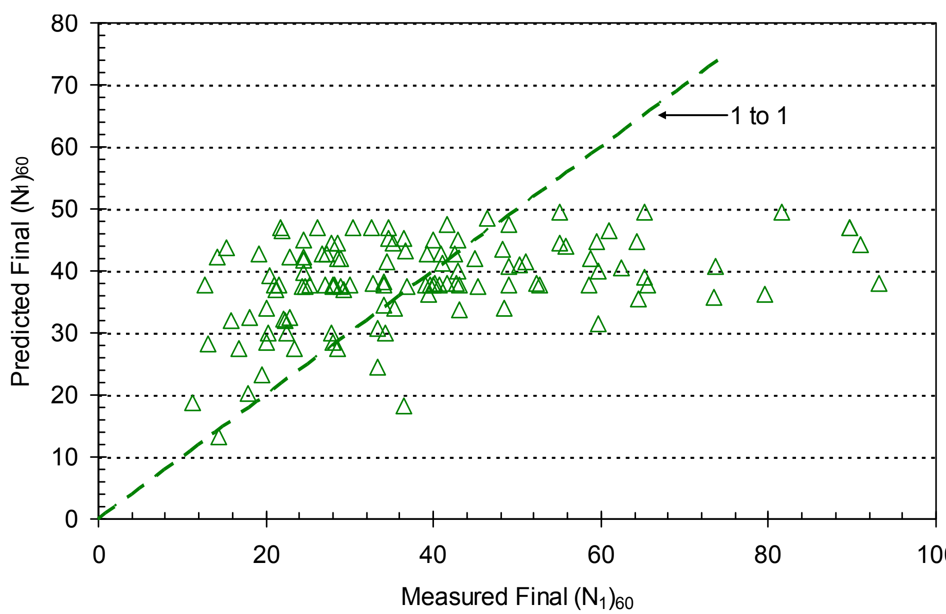


Figure 3-24 Measured final $(N_1)_{60}$ values versus predicted final $(N_1)_{60}$ values for multiple regression using fines content and initial $(N_1)_{60}$ as predictors. A one to one line is also shown.

To compare the results for this case history to the published literature, final $(N_1)_{60\text{-}cs}$ values are plotted versus initial $(N_1)_{60\text{-}cs}$ and logarithmic trend lines are presented in Figure 3-25. The data is split according to depth with trend lines for depths below 20 ft, above 20 ft, and for the full depth. This is according to the previous observation that the soil layer above 20 ft had lower average fines content (28%) than the layer below 20 ft (40%) and that the overall profile had an average fines content of 34%. The R-squared values for the trend lines were all below 0.12 and the trend lines are best viewed as average trends in the data. The influence of fines content on final blow count seems significant at low initial blow counts but less significant at blow counts greater than about 35. No new trends are observed by comparing the $(N_1)_{60\text{-}cs}$ trend lines with the $(N_1)_{60}$ trend lines, which are found in Appendix A, Figure 13-1.

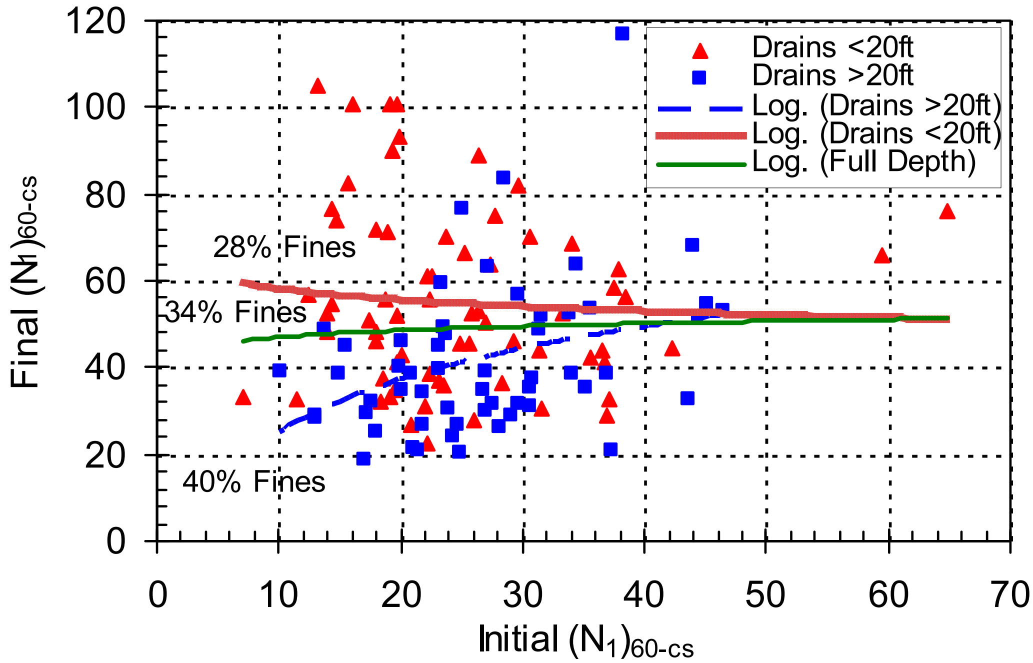


Figure 3-25 Initial $(N_1)_{60\text{-}cs}$ versus final $(N_1)_{60\text{-}cs}$ for the 26% A_r data with drains at the 24th Street Bridge case history.

In each case history, the clean sand trend line will be compared to the set of curves for clean sand (<15% fines content) developed by Baez (1995). Comparing the trend lines will allow the reader to visualize general trends in the data although scatter in the results is to be expected. The clean sand value is used to minimize the variation in penetration resistance associated with differences in fines content. The correction for clean sand will allow for a more direct comparison to the clean sand values presented by Baez. The influence of fines content on the ability of the treatment plan to improve the soil would still be present in the data but the influence of fines content on the penetration resistance will be removed or at least muted to a large degree.

The clean sand trend lines are not highly representative of the data due to scatter; however, by comparing them to the Baez clean sand curves the effectiveness of the 24th Street Bridge site treatment plan may be compared generally. It should be noted that the trend lines truncate at the limits of the data from which they were obtained, in order to most accurately represent the available data. The 24th Street Bridge site with its 34% average fines content and 26% A_r with wick drains showed average improvements of about 10 blows greater than clean sand site with less than 15% fines and an A_r of 20% without wick drains. The other trend lines indicate that an average fines content of 40% limited improvement to a clean sand equivalent A_r of 5-10% while an average fines content of 28% provided improvement greater than the 20% A_r clean sand curve by about 15-20 blows. The drains were effective in mitigating liquefaction in all areas, but the areas with fines contents less than 35% saw significantly greater improvement than the area with 40% fines and the improvement was roughly comparable that of the clean sand curves presented by Baez (1995).

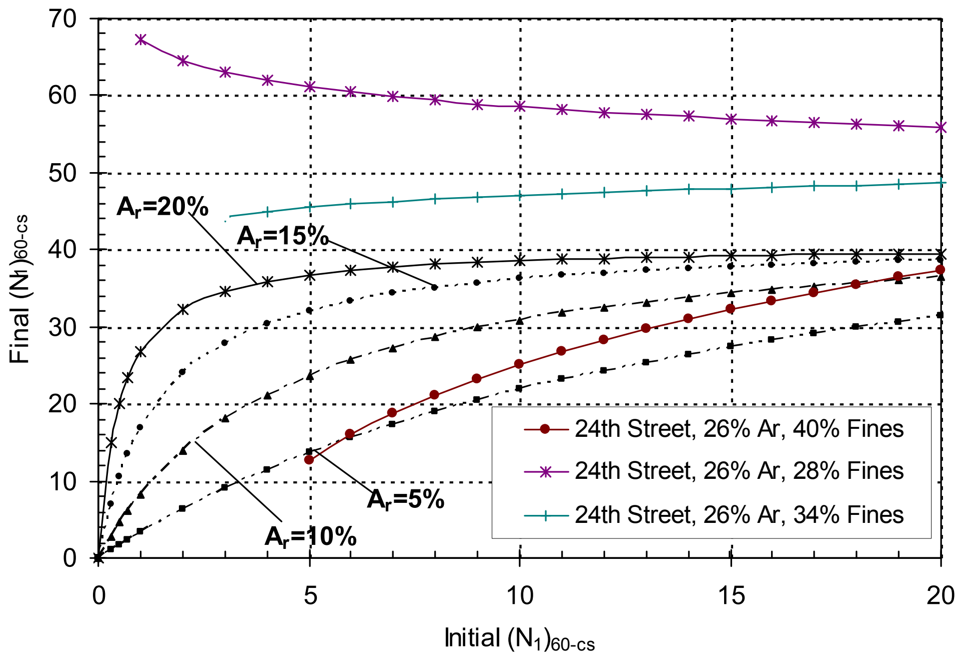


Figure 3-26 24th Street case history SPT clean sand results compared with clean sand (<15% fines) curves developed by Baez (1995). The 24th Street data is represented by trend lines for the full depth, for the layer above than 20 ft which had an average fines content of 28%, and the layer above 20 ft which had an average fines content of 40%.

3.4 Conclusions

3.4.1 Test Section Conclusions

- The test section was valuable to wick drain research because it allowed a direct comparison of areas with and without drains where the soil profile and characteristics were nearly the same.
- Areas with drains consistently had fewer negative improvements ($\Delta(N_1)_{60}<0$) and fewer low final blow counts (<18) than areas without drains. Data from areas with drains had 7% negative improvement as opposed to 32% in the areas without drains. Final blow count was used as the acceptance criterion at the site and areas with drains only had 11% not

passing while the areas without drains had 25% not passing. Both areas passed the average final blow count minimum of 23.

- The 26% area replacement ratio yielded a 94% increase in blow count for the areas with drains and a 27% increase in areas without drains. The average fines content for the test area was 32% and the average clay content was 13%.
- SPT testing performed 13–16 days after treatment did not show consistent gains or losses over data tested 1–3 days following treatment for areas without drains. In contrast, areas with drains exhibited some improvements in penetration resistance with time.
- Testing was only performed at the depths of interest which limited the overall understanding of improvement across the soil profile.

3.4.2 Additional Conclusions for the Entire Site

- High clay contents (>15%) were most often associated with small increases in blow count although some high clay contents were found in data with large increases in blow count.
- The soil profile was divided into layers at 20 feet with the layer above averaging about 28% fines content and 11% clay content and the layer below averaging about 40% fines content and 14% clay content.
- When the results were divided according to the upper and lower layers, the areas with drains showed much better improvement in the layer with lower fines and clay contents. Improvement in areas with drains consistently

outperformed that in areas without drains. In the upper layer improvement was 148% versus 35% improvement for drained versus undrained areas, and in the lower layer improvement was 69% versus 8%.

- The decision to increase the time between treatment and post-treatment testing from 1-3 days after treatment to 6-13 days after treatment helped to eliminate the need for additional post-treatment testing. It is recommended that post-treatment testing not be performed before 3 days when possible and preferably after 6 days to assess improvement more accurately.
- Adding wick drains to the 26% A_r stone column treatment plan was more effective in increasing the final blow counts than increasing the A_r to 34% was for initial blow counts less than 24. At an initial blow count of 10 the final blow count for the 26% A_r drains data was on average about 10 blow counts higher than the 34% A_r no drains data. The data was very scattered and these results are only given as a general trend instead of as a prediction of a specific final blow count.
- The 26% A_r treatment with wick drains at the site (34% average fines content) showed average improvements of about 10 blows greater than the clean sand (<15% fines) curve developed by Baez (1995) for an A_r of 20% without wick drains.

4 I-15 and UPRR Railroad Bridge Abutment, Ogden, Utah

4.1 Site Overview and Soil Conditions

The UPRR Railroad Bridge site was in Ogden, Utah where Interstate 15 crossed over the Denver and Rio Grande (D&RG) Line of the Union Pacific Railroad (UPRR). The UPRR Bridge was part of the Utah Department of Transportation (UDOT) I-15 NOW project and was also known as Bridge 18. The site consisted of existing twin bridges that were replaced with newer, wider bridges. The abutments to the bridges were constructed over improved soil and the construction was performed in three phases to accommodate traffic demands and treatment objectives.

RB&G Engineering prepared the report of geotechnical observations and testing for the site. The design earthquake for the site is a magnitude 7.4 earthquake with a peak ground acceleration of 0.57g. The peak ground acceleration is based on a 2% probability of being exceeded in 50 years (~2500 yr recurrence interval). The site is located within about mile of the Wasatch fault.

Prior to treatment, nearly continuous SPT sampling was performed at the test site to determine liquefaction potential and soil characteristics at the site. The soil profile was characterized by multiple deposits of potentially liquefiable sandy and silty soils. The soil profile varied across the site; however, a generalized soil profile is found in Figure 4-1.

The layering was not consistent across the site so Figure 4-1 is representative of the most common layers found at the site.

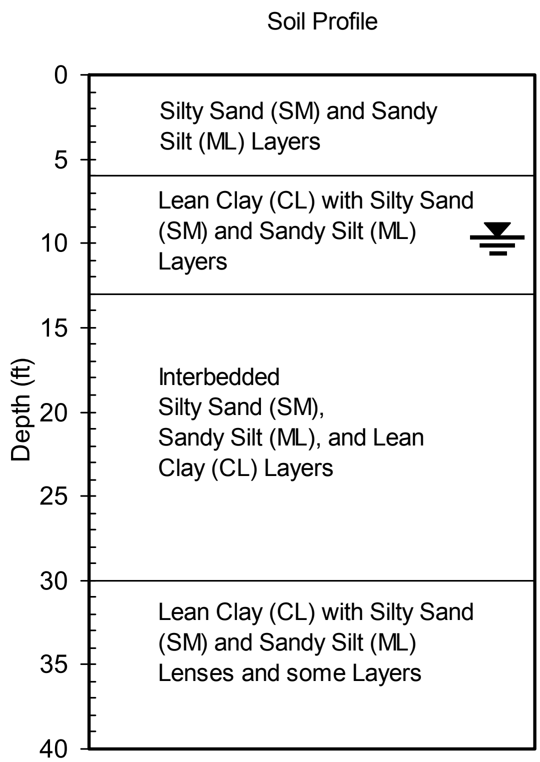


Figure 4-1 Idealized soil profile for the UPRR Bridge case history.

The preliminary geotechnical investigation classified the soil as silty sand (SM) and sandy silt (ML) layers to about 6 feet. Between 6-13 feet the soil was predominantly firm to stiff lean clayey soils with silt and sand layers which was underlain by interbedded silts, sands and clay layers to about 30 feet. Below 30 feet the soil profiles varied but the soil predominantly consisted of lean clay with silt and sand lenses and layers. The measured fines content and clay content profiles for the site are found in Figure 4-2. The focus of this study is silts and sands so subsequent analyses did not use any data classified as clay mixtures, clay lenses, or clay layers; however, to accurately portray the interbedded nature of the site, (a) profiles of the all the data (including clay data) as well as (b) profiles of only the silts and sand data are both presented. The most consistent layer of silts and sands was at approximately 10-17 ft beneath the surface.

The data in Figure 4-2 show a general increase in fines content with depth. The overall average fines content for the site was 47% fines with the average for the silts and sands being 46% fines. There were only 30 measured fines contents for the silt and sand data versus 96 for the overall data. There were even fewer measured clay contents, 23 overall and only 9 for the silt and sand data. The overall average for the site was 17% with the silt and sand data having an average clay content of 12%. There were not enough data to determine clay content trends with depth; although it is assumed that there were not any significant trends due to the interbedded, variable nature of the soil profile. The remainder of the analyses will use the silt and sand data set unless otherwise noted.

Soil liquefaction was deemed a potential hazard at the site and mitigation efforts were required. Liquefiable deposits were most frequently encountered within the upper 25-30 feet of the soil profile. Liquefiable deposits below depths of 30 feet were not continuous and significant structural damage due to liquefaction of these layers was considered to be unlikely. Liquefaction analyses were performed by RB&G using the “Simplified Procedure” developed by Seed and Idriss (1971) with refinements presented at the 1996 NCEER workshop (Youd, et al., 1997). At many locations, the liquefiable layers were occasionally interrupted by deposits of clays and moderately dense sands, which were not expected to liquefy during the design event. In several cases, boring samples compared at similar elevations encountered completely different soils which varied from loose granular deposits to denser granular and/or plastic clayey soil deposits.

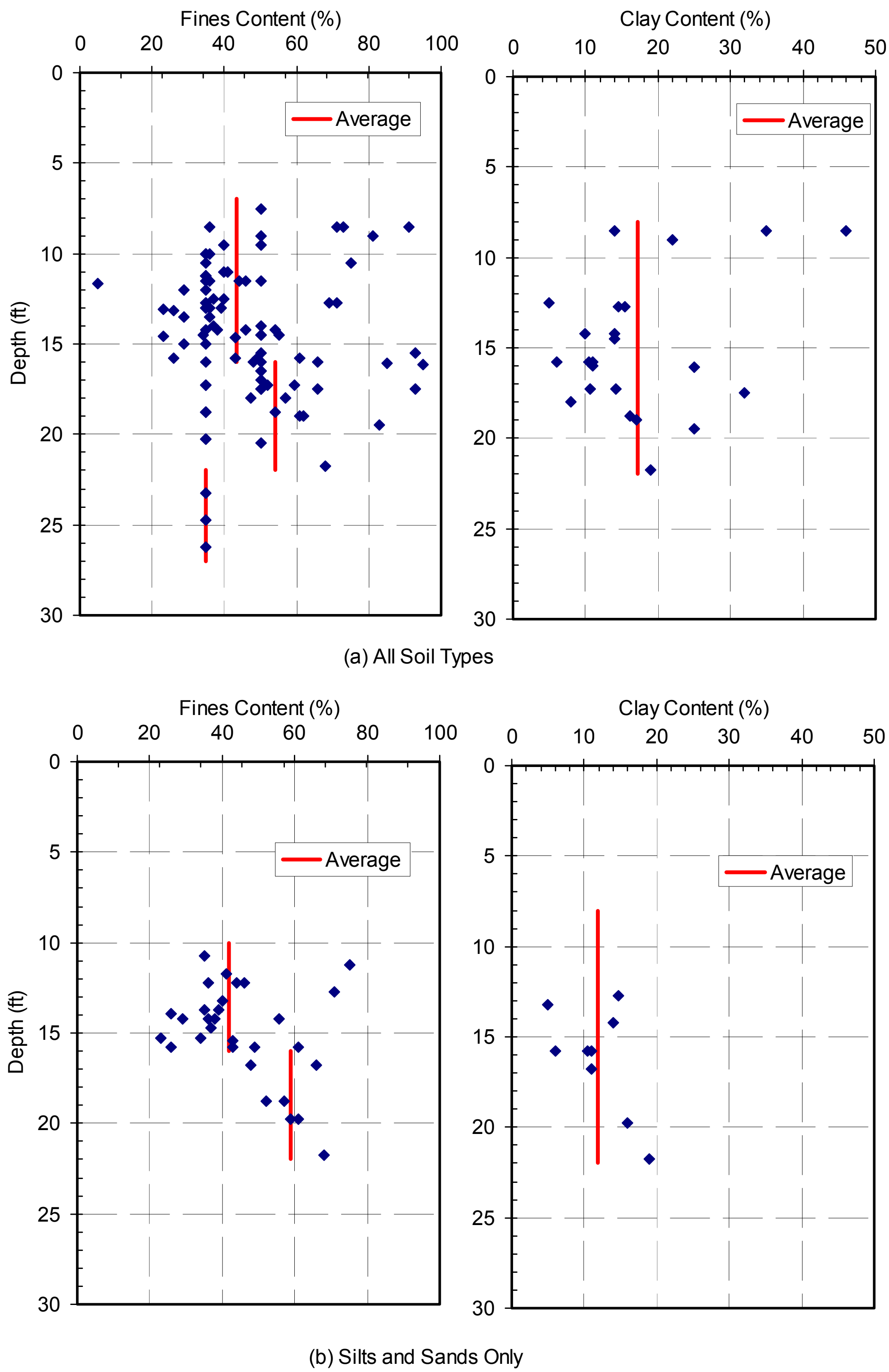


Figure 4-2 Measured fines content and clay content profiles for (a) all soil types and (b) silts and sands only at the UPRR Bridge case history. Average lines over 5-9 foot intervals are shown.

To prevent liquefaction the minimum average post-improvement $(N_1)_{60}$ value for a single SPT boring was specified as 23 with the minimum individual value set at 18. The average initial blow count for the overall site was 10 with the average initial blow count for silt and sand data being 13.

4.2 Treatment Method

The main concern for the mitigation of the liquefiable areas was the amount of non-plastic fines in those areas. The fines contents ranged from about 23 to 75% in susceptible layers encountered in the pre-treatment borings. RB&G recommended and UDOT approved the vibro-replacement method of stone column installation for liquefaction mitigation. Based on previous experience at the Cherry Hills and Shepard Lane sites, as well as concurrent work on the test section of the 24th Street Bridge, RB&G and UDOT chose to supplement stone columns with wick drains in order to target the mitigation of sandy silts and silty sands. Wick drains were utilized in order to reduce pore pressures and to increase the densification effects of the vibro-stone columns in soils with high fines. The contractor hired to perform the soil improvement at the site was Hayward Baker.

The treatment plan for the UPRR Bridge site was essentially the same as the 24th Street Bridge site. Stone columns were installed in a center to center equilateral triangular pattern with a spacing of 6.5 feet. Stone column diameters were approximately 3.5 feet. The stone column layout resulted in an area replacement ratio (A_r) of 26%. Wick drains were installed equidistant between any two columns prior to column installation. The stone column and wick drain layout for the site is shown in Figure 4-3. Stone columns and wick drains were installed to a depth of 25 feet. Stone columns were installed using the dry, bottom-feed approach with a Keller System S23120 vibrator (380 Volts, 1775 rpm). Maximum amperage during installation ranged from 150 to 250 amps and the time for installation was typically about 30 minutes. Treatment was finished during 2007.

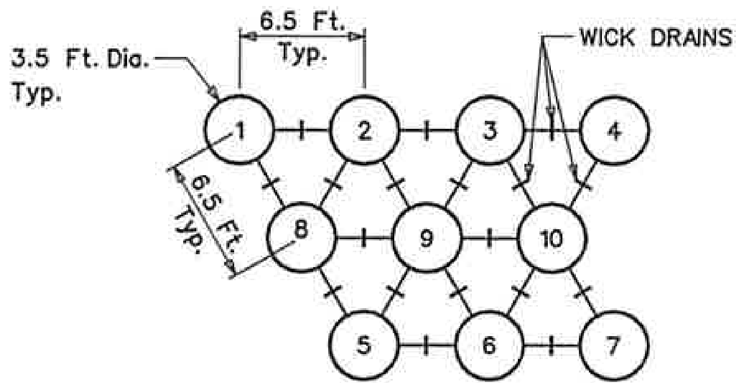


Figure 4-3 Layout of stone columns and wick drains in the test area at the UPRR Bridge case history.

4.3 Results and Analysis

Post-treatment testing was performed within 1-17 days after completion of stone column installations. Test holes were drilled with nearly continuous sampling as was done previously for the pre-treatment testing. Post-treatment test holes were located as close as possible to the pre-treatment test holes, typically within about 6 feet. All of the pre-treatment borings for the site were directly comparable to the post-treatment borings.

To examine the effectiveness of the drains, initial and final blow counts versus depth for the treatment area were plotted as shown in Figure 4-4. Averages over 3 ft intervals are shown to help visualize the trend with depth. Plots of the results for (a) all of the soil types as well as (b) the silt and sand soil types only are both presented.

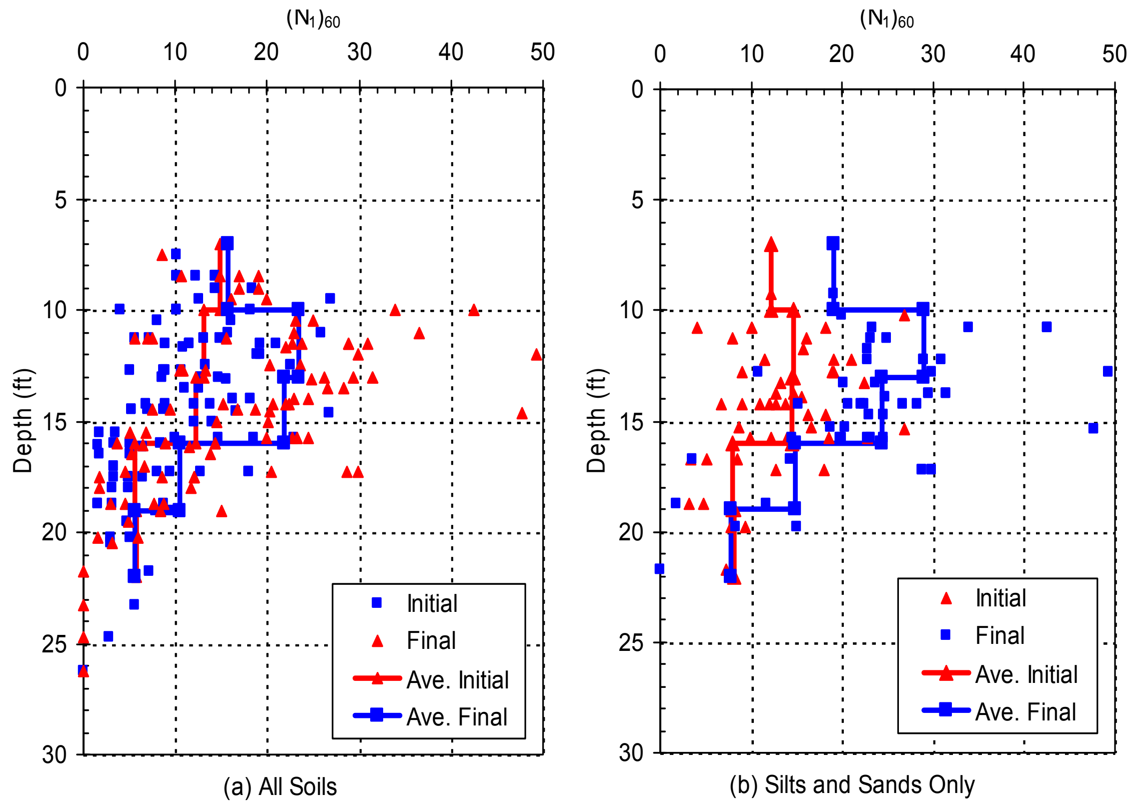


Figure 4-4 Results from pre- and post-treatment SPT testing for (a) all the soil types as well as (b) silt and sand soil types only for the UPRR Bridge case history. Averages over 3 ft intervals are also presented.

The data and average lines in Figure 4-4 show general improvement following treatment. The silt and sand data shows slightly greater improvement over most intervals. The overall average improvement [$\Delta(N_1)_{60}$] for all soil types was 7 blows/ft relative to the initial average blow count of 11 while the silt and sand average improvement was 10 for an even higher initial average blow count of 13. The improvement decreased drastically over the 19-22 ft interval. However, there were much fewer data points available for this interval which makes the results less reliable. The fines content was also noted to increase with depth which may be a factor. The effect of fines content will be examined more closely later in the analysis section.

Figure 4-5 to Figure 4-8 present the companion SPT boring results for initial and final blow counts versus depth. The interbedded nature of the soil was thought to be a potential factor affecting improvement and was also considered later in the analysis process. To visualize the interbedded nature of the soils easier, all of the data tested is presented in the individual boring results. Data with high clay contents (>15%) are noted with an "X" over the data point in the plot. These high clay content data were either soils classified as clay or soils classified as silt or sand which had clay contents greater than 15%. The minimum final $(N_1)_{60}$ criterion of 18 is indicated on each plot by a dashed line.

Due to varying surface elevations across the site, the pre- and post-treatment companion borings are best compared using elevation instead of depth in order to match the correct pre-treatment soil layering with the corresponding post-treatment soil layering. The results presented in Figure 4-5 to Figure 4-8 are based on elevation and pre-mitigation depths. The remainder of the analyses will use an average of the pre- and post-treatment depths whenever there was a difference in surface elevations. Typically the difference in pre- and post-treatment test hole surface elevations was only 1-2 feet which would not have affect the results significantly.

Similar to the 24th Street case history, testing of the entire soil profile following treatment would have been desirable from a research standpoint; however, to minimize testing costs, post-treatment testing was limited to the liquefiable portions of the soil profile. This limits the ability of the data to present a comprehensive picture of what happened as a result of treatment, what factors affected improvement, and to what degree the factors affected improvement. Nevertheless, trends are still observable and conclusions can still be drawn from the data, but specific quantifiable relationships between improvement and all factors contributing to improvement are limited.

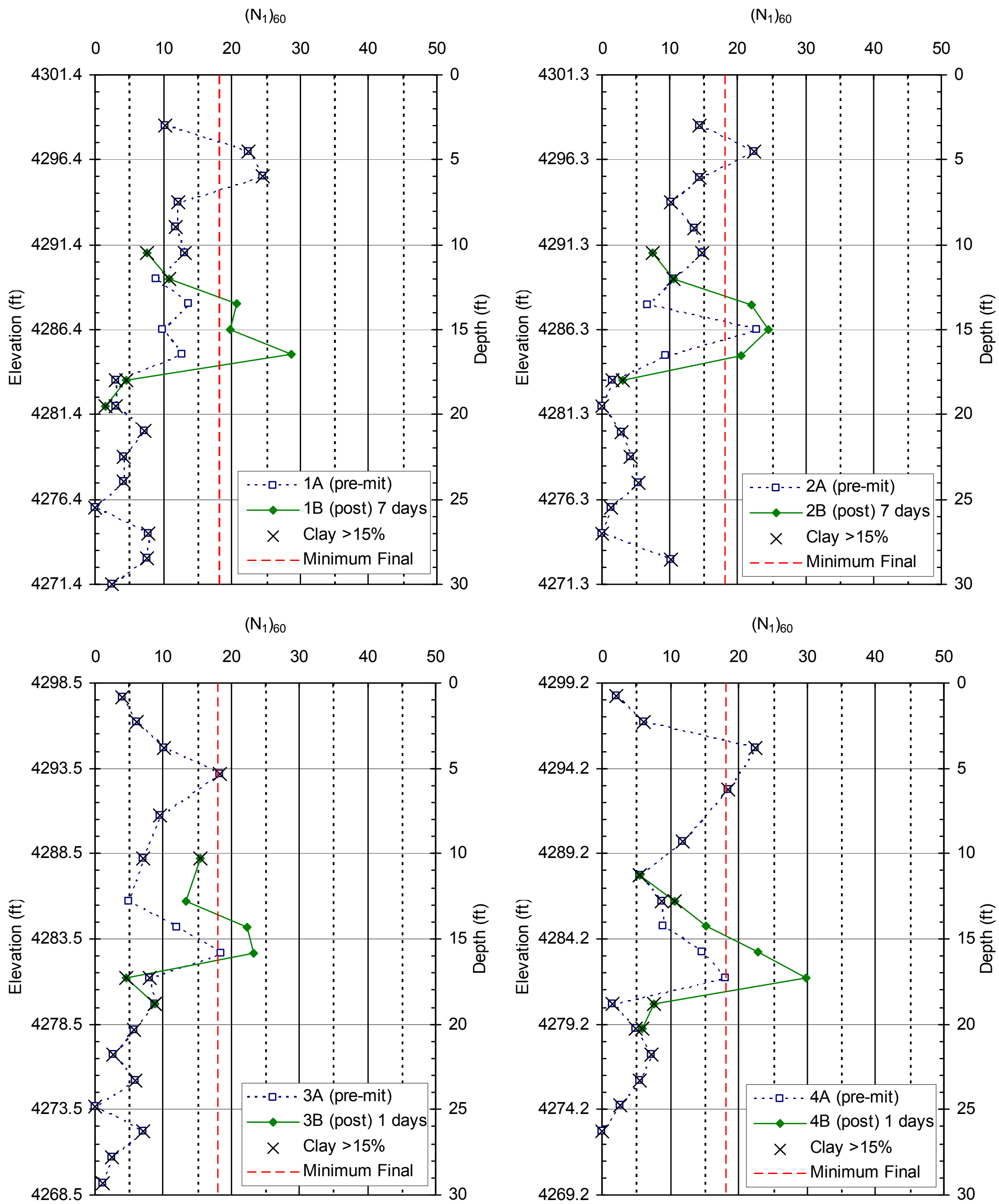


Figure 4-5 Results from SPT test holes 1-4 at the UPRR Bridge case history. The dashed line indicates the project's minimum final blow count criterion for acceptance of treatment. The data with clay contents >15% are marked by an "X".

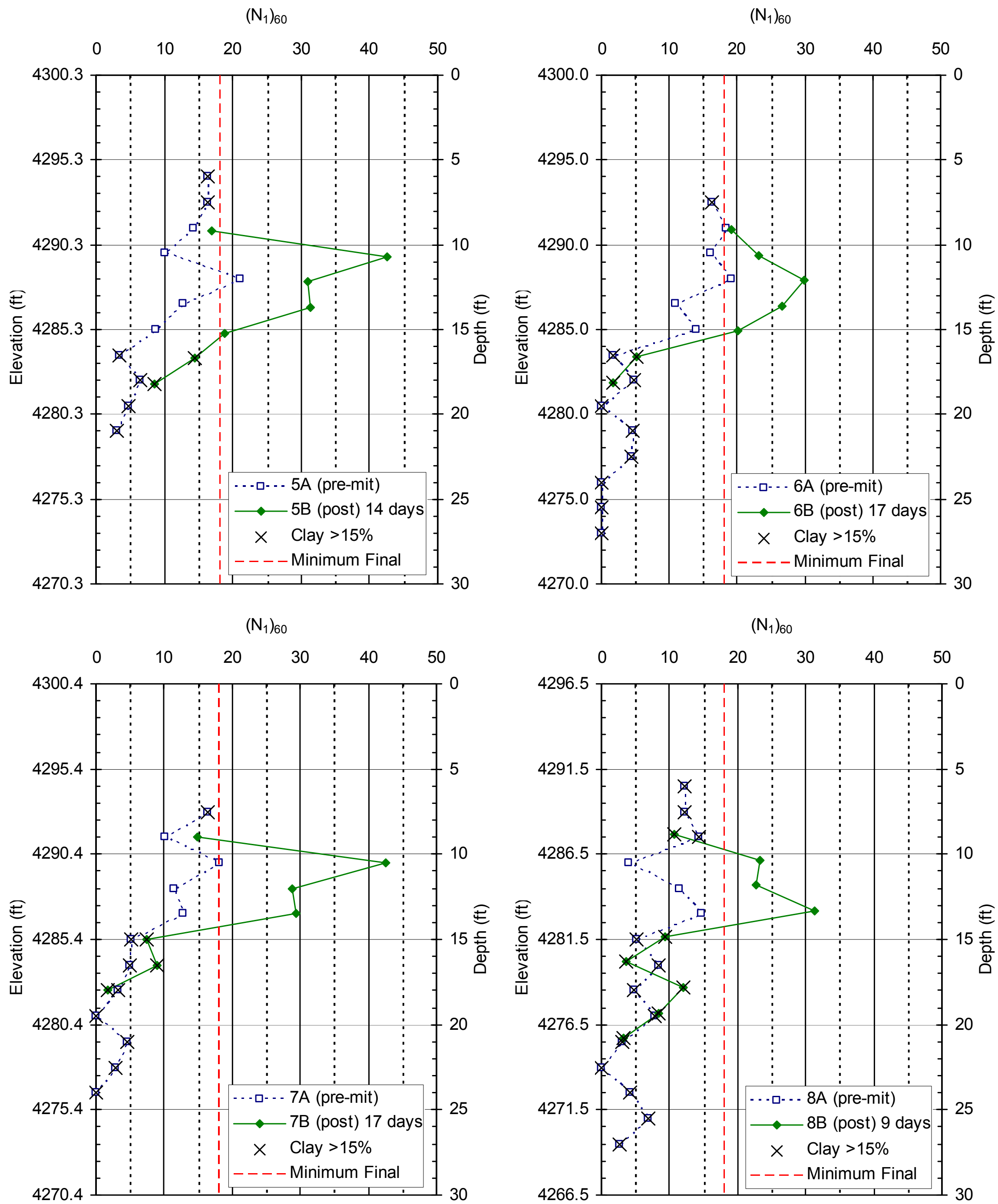


Figure 4-6 Results from SPT test holes 5-8 at the UPRR Bridge case history. The dashed line indicates the project's minimum final blow count criterion for acceptance of treatment and data with clay contents >15% are marked by an "X".

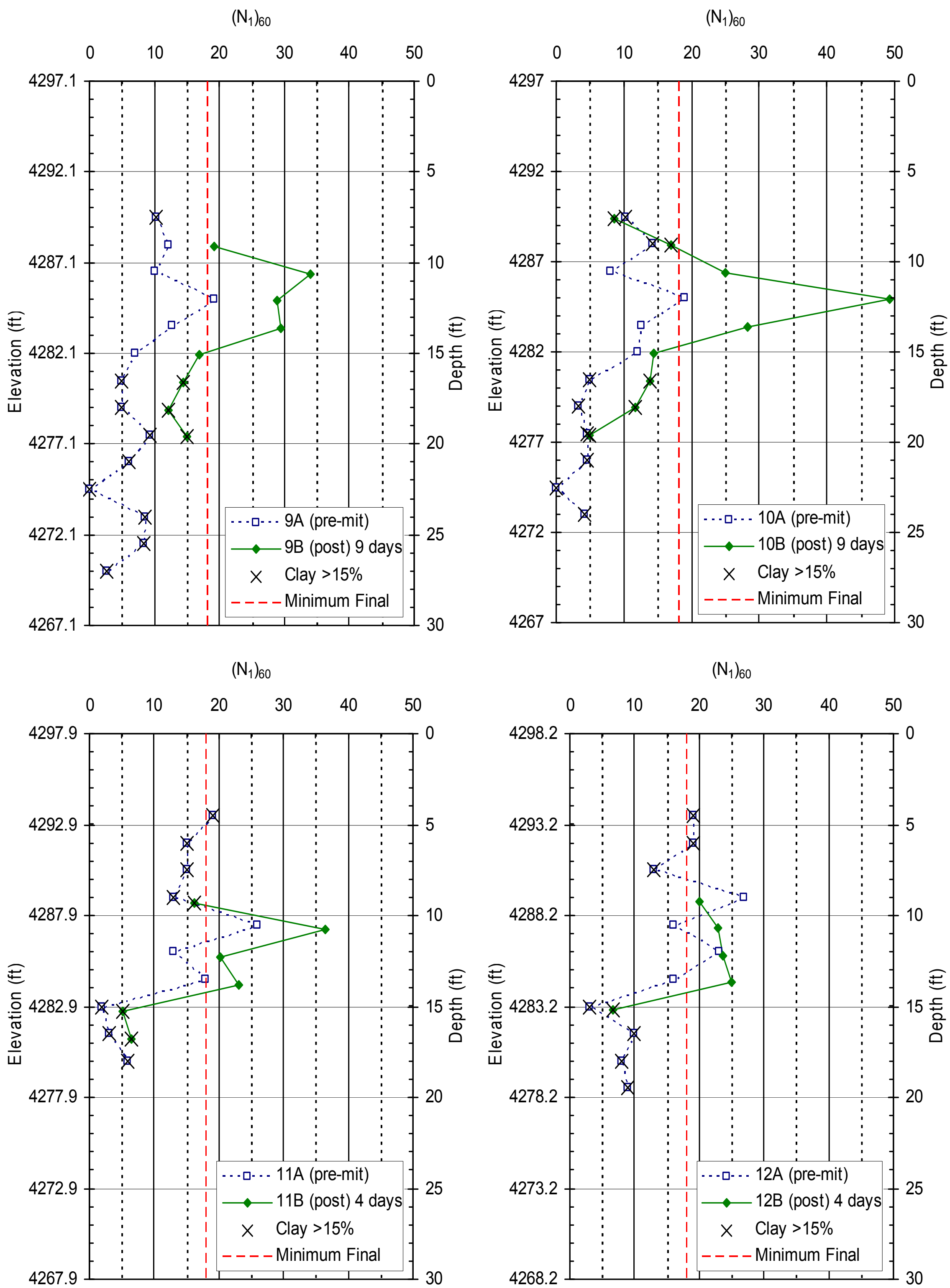


Figure 4-7 Results from SPT test holes 9-12 at the UPRR Bridge case history. The dashed line indicates the project's minimum final blow count criterion for acceptance of treatment and data with clay contents >15% are marked by an "X".

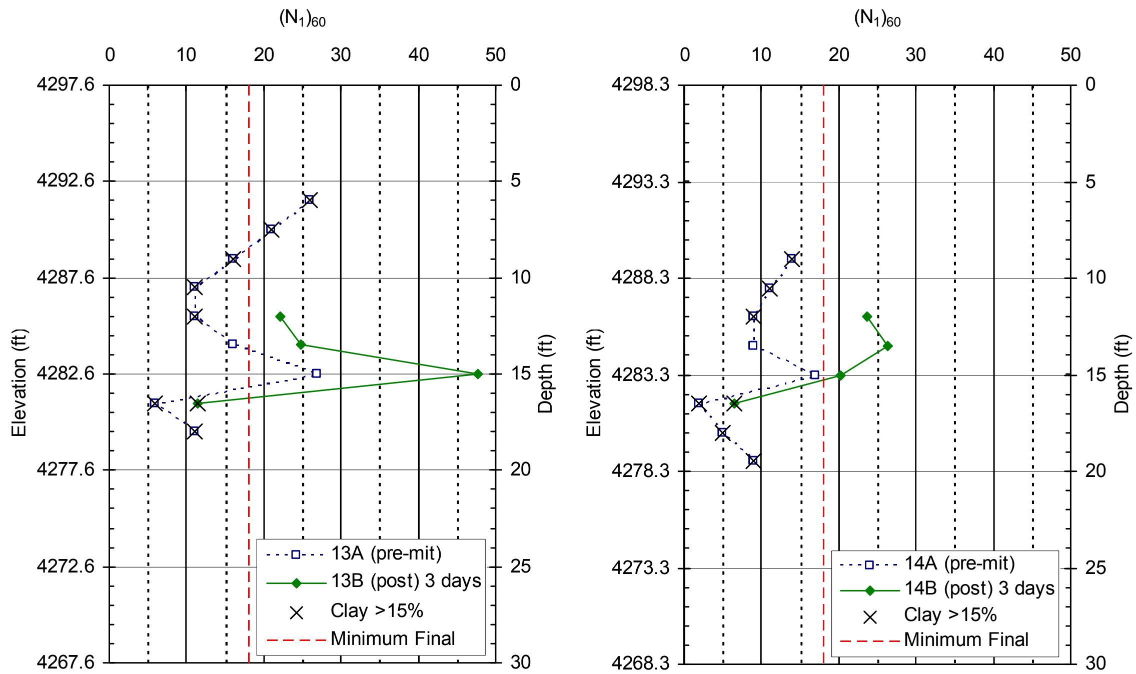


Figure 4-8 Results from SPT test holes 13-14 at the UPRR Bridge case history. The dashed line indicates the project's minimum final blow count criterion for acceptance of treatment. The data with clay contents greater than 15% are marked by an "X".

The companion boring results show general improvement within the liquefiable zone across the site. There are some data with negative improvements but the majority of the data are positive improvements. The post-treatment testing was done over potentially liquefiable layers only and there is a common trend in the individual boring data plots where the negative improvements are typically at the top or bottom of the post-treatment testing interval. It is possible that the soil at the boundaries of the silt and sand layers is affected by the clay layers adjacent to them. The layers with high clay contents typically had less improvement in blow counts. This is noted since the data with high clay content (with an "X" through the data point) are typically the points with lower blow counts. Thus the clay layers may have caused the treatment to be less effective on the boundaries of the silt and sand layers. It is also noted that many of the values with negative improvement have high clay contents and may have even been classified as clayey soils since all soil types are included in the individual boring results. The project criterion considered soils with clay contents greater than 15% to be non-liquefiable according to the Modified Chinese Criteria (Seed, et al., 1983). The effect of clay content will be discusses later in the analysis section of the study once the clayey soil samples are removed from the data set.

To determine the factors affecting improvement, the change in blow count, $\Delta(N_1)_{60}$, was plotted versus depth, initial $(N_1)_{60}$, fines content, and clay content. The plot showing depth versus $\Delta(N_1)_{60}$ is located in Figure 4-9. The scatter is significant and regression is not appropriate for the data; however, there does appear to be a trend toward lower improvement as depth increases. It was noted previously that the average fines content increased with depth which may have caused the improvement to decrease with depth. This will be investigated further later in the analyses.

The plot showing fines content versus $\Delta(N_1)_{60}$ is presented in Figure 4-10. A linear trend line is shown and data with clay contents greater than 15% are marked by an "X". All of the fines content data are measured rather than from correlations. The data in Figure 4-10 suggests that there is a trend where increasing fines content yields decreasing improvement. The trend line shows that at about 25% fines content the expected improvement would be 14 blows per foot while above approximately 65% fines content the expected improvement would be less than 5 blows per foot. There is significant scatter in the data with the R-squared value for the linear trend line being only 0.1307, meaning that only 13% of the improvement can be accounted for by fines content variations. Therefore, the exact equation of the trend line is not useful due to the significant scatter; only the general decrease in improvement with increasing fines content is useful. The downward slope of the trend line appears to be tied to some of the data points where clay content exceeds 15%. Therefore, this reduction in improvement may be somewhat exaggerated.

Initial $(N_1)_{60}$ versus $\Delta(N_1)_{60}$ is plotted in Figure 4-11. A linear trend line is shown and data with high clay contents (>15%) are marked by an "X". The scatter in the data is significant. The R-squared value for the trend line was extremely small at 0.0002. It appears that there is no change in improvement with increased initial blow count. This is contrary to the trend that has been noted at other sites where increased initial blow counts leads to decreased improvement. This is interesting but not of particular value since the data is very scattered.

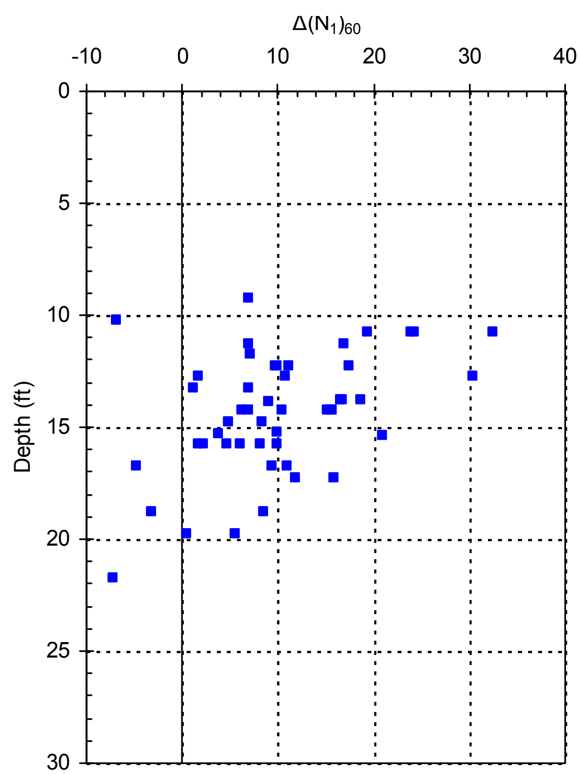


Figure 4-9 Depth versus $\Delta(N_1)_{60}$ for the UPRR Bridge case history.

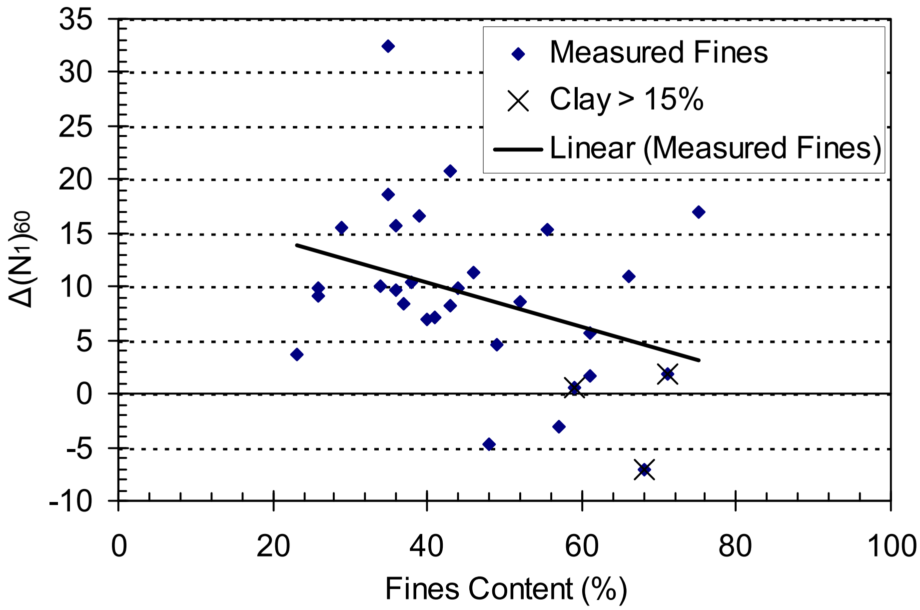


Figure 4-10 Fines content versus $\Delta(N_1)_{60}$ with a linear trend line for the silt and sand data at the UPRR Bridge case history. Samples with clay content greater than 15% have an "X" through their data point.

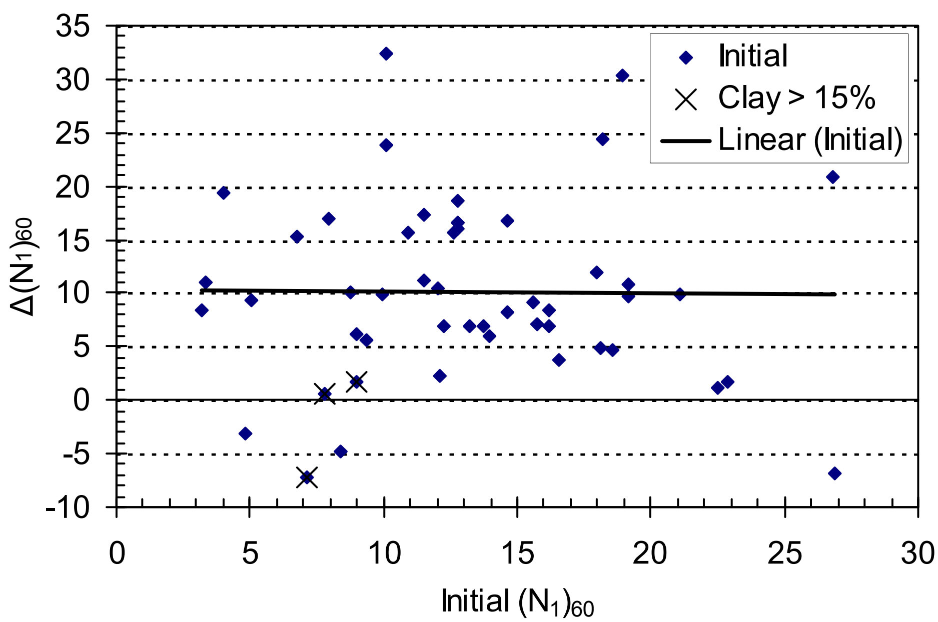


Figure 4-11 Initial $(N_1)_{60}$ versus Δ $(N_1)_{60}$ with a linear trend line for the UPRR Bridge case history. High clay contents (>15%) are marked with an "X" through the data point.

The overall improvement at the site is approximately 10 blows per foot. The few data points with measured clay contents greater than 15% all exhibited very low improvement. There were only 9 measured clay contents in the silt and sand data which was not enough to represent the site properly; however, all of the clay contents that were above 15% did have low improvement.

Clay content versus $\Delta(N_1)_{60}$ was plotted with a linear trend line in Figure 4-12. The R-squared value for the trend line was only 0.2441. As noted previously, there were only 9 measured clay contents in the silt and sand data. Nevertheless, the trend observed based on the limited data was that the improvement decreased as the clay content increased. In the previous figures it was noted that high clay contents typically indicated low improvement. In Figure 4-12 there is additional data that indicates that low clay contents tend to have higher improvement. There are several outliers though so additional testing would be required to verify these trends. To reduce testing costs, the clay contents were typically only measured for samples expected to liquefy or which did not show adequate improvement. Several additional measurements were taken for this research study; however, there are still not enough data to reliably show specific measurable trends in the data.

Final $(N_1)_{60}$ is plotted versus initial $(N_1)_{60}$ in Figure 4-13 along with a linear trend line showing no improvement (1 to 1 line). Data with high clay contents are marked with an “X”. Although this plot shows results similar to that in Figure 4-11, (which plots initial $(N_1)_{60}$ versus $\Delta(N_1)_{60}$) presenting the data in terms of final blow counts allows an engineer to more easily determine whether the desired final blow count is likely to be met. As was observed in Figure 4-11, the data shows a fairly constant improvement when compared to the 1 to 1 line and the high clay contents correspond to low final blow counts.

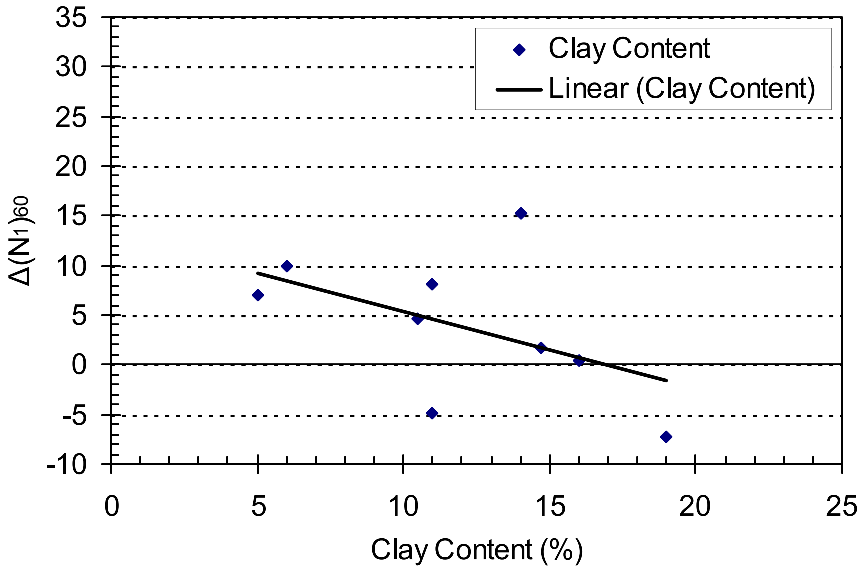


Figure 4-12 Clay content versus $\Delta(N_1)_{60}$ with a linear trend line for the UPRR Bridge case history.

The additional observations made from this figure are that there seems to be a high concentration of data in the 20-30 final blow count range and there are very few data points below about 15 blows per foot for the final blow count. The average final blow count over the initial blow count range 0-10 was 13 blows, for the 10-20 range the final blow count average increased to 27 blows and for the 20-30 range it increased even further to 29 blows. On average the final blow count increased primarily due to the increase in the initial blow count. Typically there is a decrease in improvement due to increasing initial blow counts. This was not observed here which may be due to the fact that any questionable results were tested for high clay contents and several were excluded. It is also possible that fines content may be the main factor affecting both initial blow count and final blow count. Figure 4-10 shows that increased fines content decreased improvement, which may have affected the initial and final blow count relationship as well. Figure 4-14 shows fines content versus final $(N_1)_{60}$ which confirms seems to confirm that fines content appears to be the most significant factor with increased fines contents resulting in decreased final blow counts.

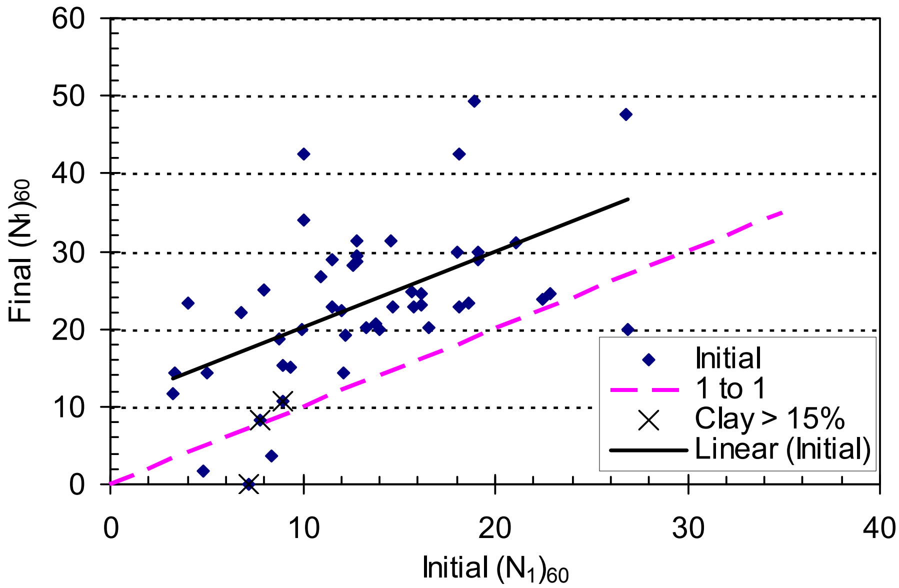


Figure 4-13 Initial $(N_1)_{60}$ versus final $(N_1)_{60}$ with a linear trend line and a 1 to 1 line for the UPRR Bridge case history. Data with high clay content (>15%) are marked by an "X".

To investigate the effect of increased time between treatment and testing, initial $(N_1)_{60}$ versus final $(N_1)_{60}$ is plotted in Figure 4-15 with the data being separated into time intervals based on the time after treatment. There are three time intervals represented, the first being 1-4 days, the second 7-9 days, and the third 1-17 days after treatment. Logarithmic trend lines are included for each series. There is scatter in the data but there is less scatter than in many of the other plots that have been presented for this case history. The R-squared values are as follows; 0.5541 for the 1-4 days series, 0.2744 for the 7-9 days series, and 0.4414 for the 14-17 days series. Although the R-squared values are not particularly high, they are much better than the other R-squared values noted previously in this case history. Testing following about one week produced results about 10 blow counts higher than testing following 1-4 days for an initial blow count of 8 with improvement being similar at an initial blow count of 23. Testing following about two weeks produced even larger improvements; however, at low initial blow counts (<10) it was only slightly greater than testing following about one week. The data shows that when grouped by time intervals, there were noticeable gains in the final blow count with increased time between treatment and testing; however, there is still significant scatter in the data which makes a direct relationship unreliable. Project costs may increase with increased time before post-treatment testing so possible gains by increasing testing from one week after to two weeks after may justify increased costs; however, it is recommended that testing be done at least one week after treatment when possible.

Average values for the site are presented in Table 4-1 (silt and sand only). The overall site saw a 77% increase in blow count from an initial blow count of 13. The overall site average fines content was 46% and the overall average clay content was 12%. The post-treatment $(N_1)_{60}$ values met the project criterion of a minimum average final blow count of 23. The area of replacement was 26% which was effective in meeting the project objectives despite the interbedded nature of the soil with its high average fines content. The standard deviations for the final blow counts in all of the categories were 8-10 which is relatively large compared to the average final blow counts. The large final blow count standard deviations (32-80% coefficients of variation) confirm the visual observation of significant scatter in the data. The average values for the site, including all soil types, are presented in Table 4-2.

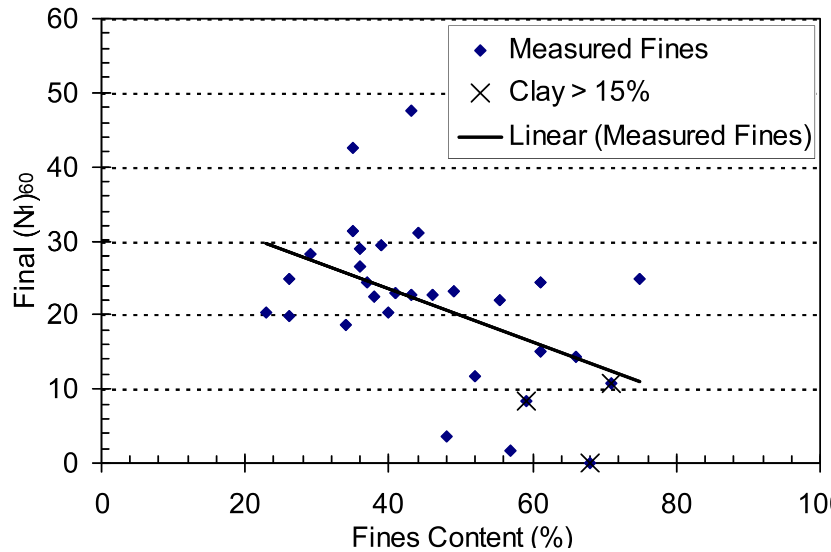


Figure 4-14 Fines content versus final $(N_1)_{60}$ with a linear trend line and a 1 to 1 line for the UPRR Bridge case history. Data with high clay content (>15%) are marked by an "X".

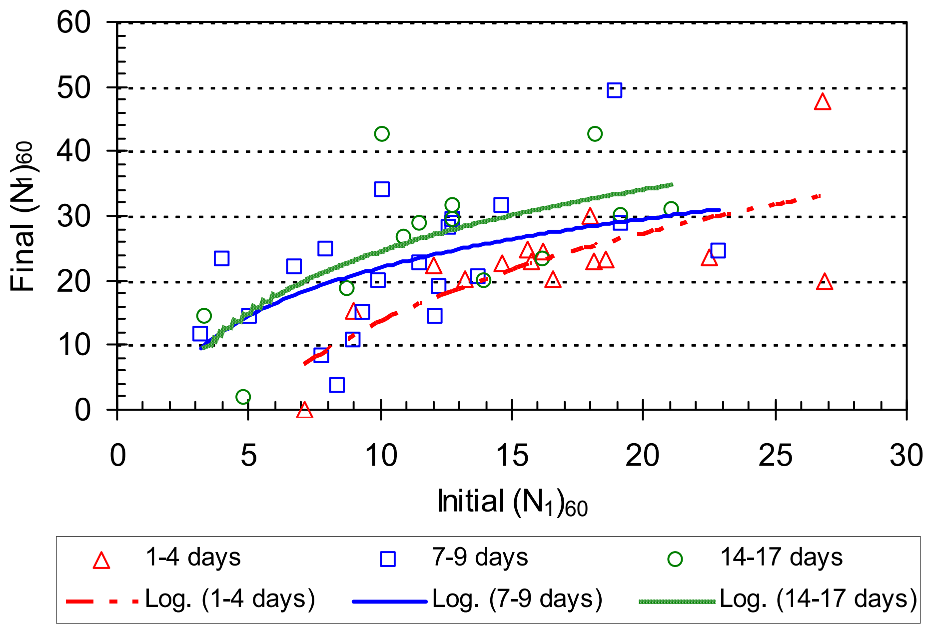


Figure 4-15 Initial $(N_1)_{60}$ versus final $(N_1)_{60}$ separated by the time after improvement before testing for the UPRR Bridge case history. Logarithmic trend lines are also shown for each series.

Table 4-1 Average values for the silt and sand data at the UPRR Bridge case history.

Depth Interval	Ave. % Fines	Ave. % Clay	Ave. Initial $(N_1)_{60}$	Ave. Final $(N_1)_{60}$	Ave. $\Delta(N_1)_{60}$	Ave. % Increase in $(N_1)_{60}$	Sample Size	Standard Deviation (Final)
10-16 ft	42	10	14	26	11	79	40	8
16-20 ft	59	15	8	13	5	60	10	10
10-20 ft	46	12	13	23	10	77	50	10

Table 4-2 Average values for all soil types at the UPRR Bridge case history.

Depth Interval	Ave. % Fines	Ave. % Clay	Ave. Initial $(N_1)_{60}$	Ave. Final $(N_1)_{60}$	Ave. $\Delta(N_1)_{60}$	Ave. % Increase in $(N_1)_{60}$	Sample Size	Standard Deviation (Final)
7-20 ft	47	17	11	18	7	68	89	10

The data in Table 4-1 is also divided according to depth intervals which exhibited different characteristics. The upper layer, 10-16 feet below the surface, had an average fines content of 42% and an average clay content of 10%. This layer was underlain by a layer, 16-20 feet beneath the surface, with increased fines and clay contents where the average fines content increased to 59% and the average clay content increased to 15%. The upper layer also had a higher initial blow count of 14 versus 8 in the lower layer. The upper layer exhibited much greater improvement in terms of increased blow count and final blow count. The upper layer had an average final blow count of 26 while the lower layer only had an average of 13. Unfortunately there were only 10 data points available for the lower layer so these trends are less reliable. These trends match those seen in the 24th Street Bridge case history but they cannot be relied on because they may not accurately portray the site since there are so few data points. The full depth averages are thus considered more useful and reliable than the layered averages.

The average values for the site, including all soil types, are presented in Table 4-2. These values show the effects of the treatment for all soils in the interbedded soil profile. Comparing the values for all soil types to those for the silt and sand soil types, the average fines contents are similar at about 47% while the clay content is higher for the all soil types category, 17% versus 12% for the silts and sands. The initial blow count was slightly lower for all soils and the final blow count was lower by 5 blows per foot on average. The all soils average final blow count was only 18 while the silts and sands blow count was 23. This is not of particular concern though because the additional soil types included in the all soils category are typically clayey soils which are not expected to liquefy. The silt and sand category had 77% improvement overall while the all soils category only had 68% improvement. These values are useful when considering the interbedded nature of the soils. The data shows that in an interbedded soil profile with high fines content (47%) and high clay content (17%) that the potentially liquefiable silt and sand layers are likely to see more improvement than the non-liquefiable clay layers and that improvements as much as a 77% increase in $(N_1)_{60}$ are possible.

To compare the results for this case history to the published literature, final $(N_1)_{60-cs}$ values are plotted versus initial $(N_1)_{60-cs}$ and a logarithmic trend line is obtained in Figure 4-16. The logarithmic trend line is then compared to the curves developed by Baez (1995) for clean sands with fines contents less than 15% which are also presented in Figure 4-17. The trend line is not highly representative of the data due to scatter (R-squared value of 0.338); however, by comparing it to the Baez clean sand curves the effectiveness of the UPRR Bridge site treatment plan may be compared generally. The logarithmic trend line for the UPRR Bridge site was developed from data with initial clean sand blow counts greater than 8 blows so the trend line that is compared to the Baez curve likewise begins at 8 blows.

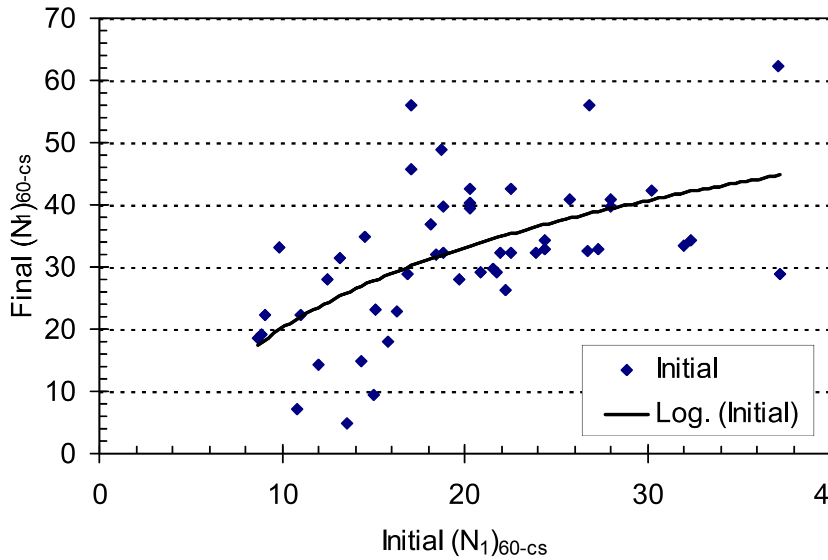


Figure 4-16 Initial $(N_1)_{60\text{-}cs}$ versus final $(N_1)_{60\text{-}cs}$ with a logarithmic trend line for the UPRR Bridge case history.

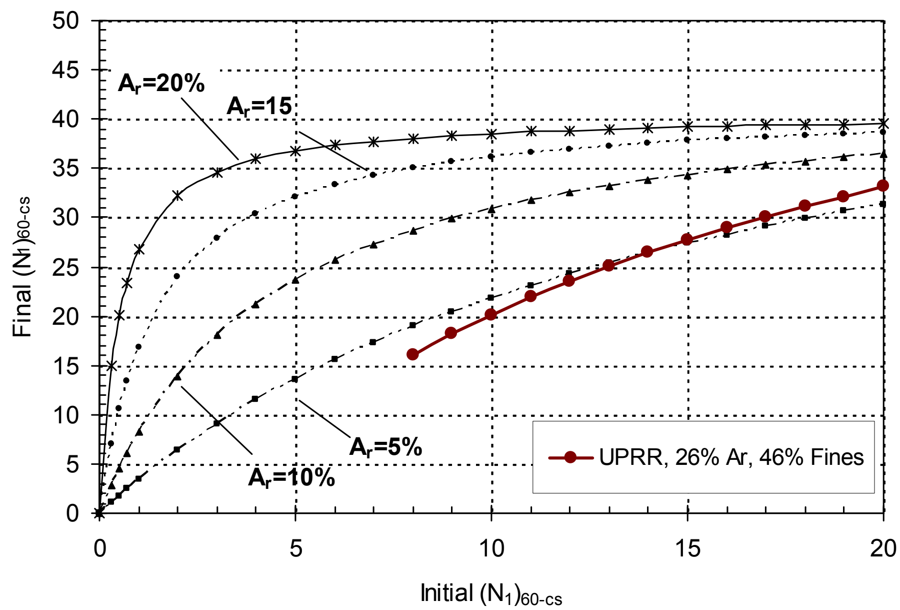


Figure 4-17 Initial $(N_1)_{60\text{-}cs}$ versus final $(N_1)_{60\text{-}cs}$ with the clean sand curves developed by Baez (1995) as well as the logarithmic trend line for the UPRR Bridge case history silt and sand data.

The UPRR Bridge site with its 46% average fines content and 26% A_r with wick drains showed similar improvement to a clean sand site with less than 15% fines and a 5% A_r without wick drains. The wick drains did not supplement the stone columns enough to counteract the negative effect of the fines content, although sufficient improvement was attained to meet the project objectives.

4.4 Conclusions

- An area replacement ratio of 26% was used at the site where the average fines content was 46% and the average clay content was 12%. There was a 77% increase in blow count following treatment with an average initial blow count of 13 increasing to a final blow count of 23. The standard deviation for the final blow counts was 10 for the site.
- Despite the interbedded nature of the soil profile, an average of 77% improvement was noted in the silts and sands layers, up from 68% improvement for the site when including clayey soils.
- Initial $(N_1)_{60}$ did not directly affect $\Delta(N_1)_{60}$ and there was a constant average improvement of 10 blows per foot independent of initial $(N_1)_{60}$.
- High clay contents (>15%) were typically associated with low improvement (<5 blows per foot) although measured clay contents were scarce and additional testing must be done to verify this trend.
- Gains in final $(N_1)_{60}$ with time were observed following treatment based on increasing time intervals and initial blow counts. Scatter was significant though and although general improvement may exist, data with

little improvement is also to be expected. At low initial blow counts (≈8) final blow count improvements of up to 10 blow counts were noted when testing was done 7-9 days following treatment instead of 1-4 days following treatment. It is recommended that post-treatment testing be done at least one week following treatment when possible.

- The treatment plan of 26% A_r and wick drains improved the site (46% average fines content) approximately the same amount as a 5% A_r treatment plan without wick drains would be expected to improve a clean sand (<15% fines) site.

5 Esprit Apartments, Marina del Rey, California

5.1 Site Overview and Soil Conditions

The Esprit apartment development is located in Marina del Rey, Los Angeles County, California. The Esprit complex is a large apartment complex located on a narrow peninsula in Los Angeles' only yacht harbor (see Figure 5-1). The development consisted of constructing four- and five-story wood-frame apartment buildings on top of a reinforced concrete parking garage with the lower level being partially below grade. The buildings are supported by a mat foundation over the improved soil.

Figure 5-1 Esprit Apartments site, Marina del Rey, CA (source, Hayward Baker, Inc.).

MACTEC Engineering and Consulting, Inc. prepared the report of geotechnical observation and testing for the site. The design earthquake for the site is a magnitude 6.8 earthquake with a peak ground acceleration of 0.5g. The preliminary geotechnical investigation classified the soil as liquefiable silty sand (SM) and sandy silt (ML). The upper 17 feet of the soil profile were determined to need soil improvements in order to prevent liquefaction and lateral spreading. Figure 5-2 presents a typical cross section of the soil profile at the site.

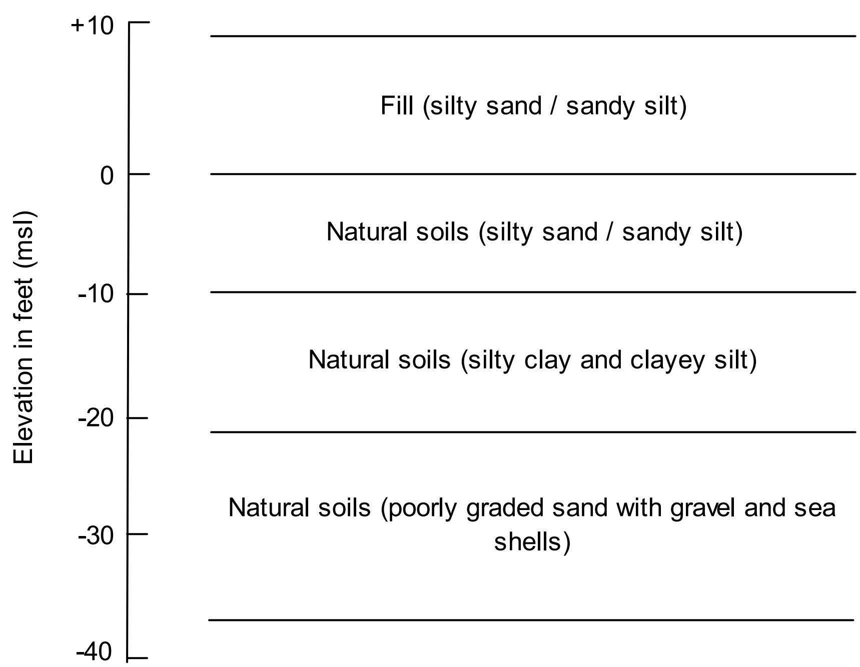


Figure 5-2 Marina del Rey typical cross section.

A test section, Test Section 1, approximately 95-feet long by 55-feet wide on the West (Pan-Handle) side of the site was used to determine the necessary soil improvement plan for that portion of the site. A second test section, Test Section 2, was located in the East (Pan) portion of the site and was approximately 60-feet long by 60-feet wide.

During the preliminary investigation in Test Sections 1 and 2 it was determined that the CPT data were not consistent in their soil type and fines content predictions relative to the samples obtained from the SPT data. This can be seen in Figure 5-3 where the correlated CPT fines contents are compared to the measured SPT fines contents. As a result, further investigation was undertaken by MACTEC and it was determined that the CPT data was indicating a coarser grained soil than SPT data indicated. To account for the observed change, MACTEC adjusted (increased) the I_c parameter in the post-improvement CPT data by 0.235. Based on measured fines contents, the average fines content for the site to a depth of 25 feet was 24.5% although measured fines contents varied from 7.6% to 94.1% with fines generally increasing with depth.

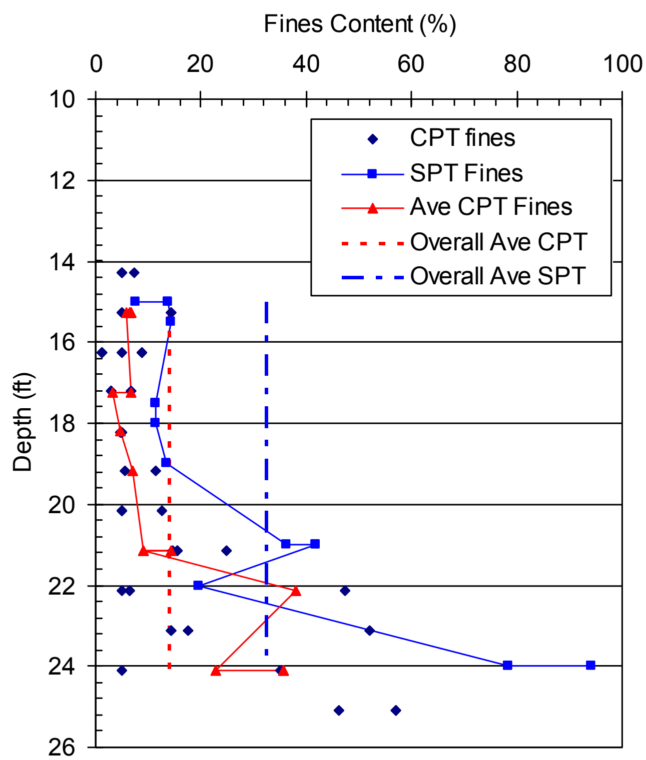


Figure 5-3 Comparison of correlated CPT fines contents and measured SPT fines contents for the Marina del Rey case history.

Sieve analyses from pre-improvement SPT borings resulted in the gradation curves found in Figure 5-4. The gradation curves are remarkably similar and indicate that the sand was fine grained. It was also visually observed that the sand at the site was very fine. Fines contents typically ranged from 25 to 40%. Although fine sand is not considered part of the fines content, it can potentially reduce the effectiveness of the mitigation efforts.

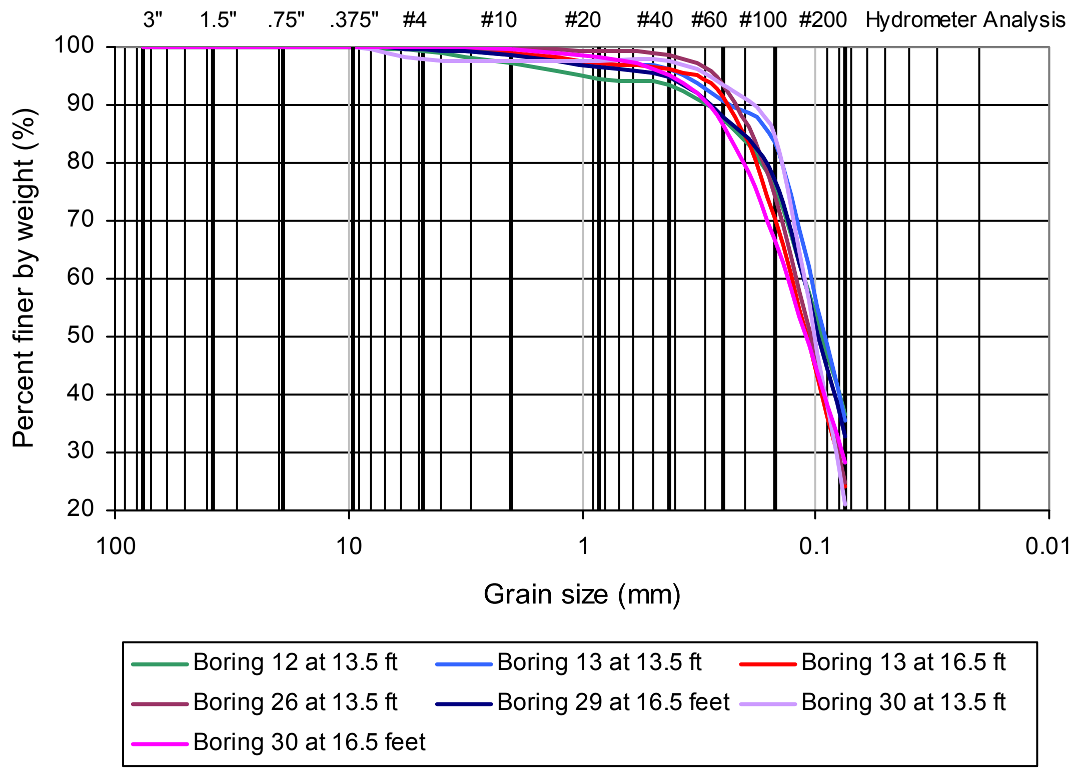


Figure 5-4 Gradation plots from pre-treatment SPT samples for the Marina del Rey case history.

5.2 Treatment Method

The contractor hired to perform the soil improvement at the site was Hayward Baker. The soil improvements took place during 2004. It was determined that the soil improvement plan should consist of installing stone columns with wick drains pre-installed in-between columns as well as soil cement columns around the perimeter.

The wick drains extended to a depth of 20 feet below the surface with the stone columns extending to a depth of 17 feet below the surface as illustrated in Figure 5-5. The stone columns had a diameter of approximately 3 feet. The columns were spaced in an 8 ft x 8 ft square pattern. The stone column layout resulted in an area replacement ratio of 11%. The drains were installed at the midpoints between adjacent columns as well as in the center of every stone column grid as shown in Figure 5-6. Based on this layout there were approximately 3 wick drains per stone column. A S23 series vibrator with 120 kW of power was used to install the stone columns using the dry bottom feed method.

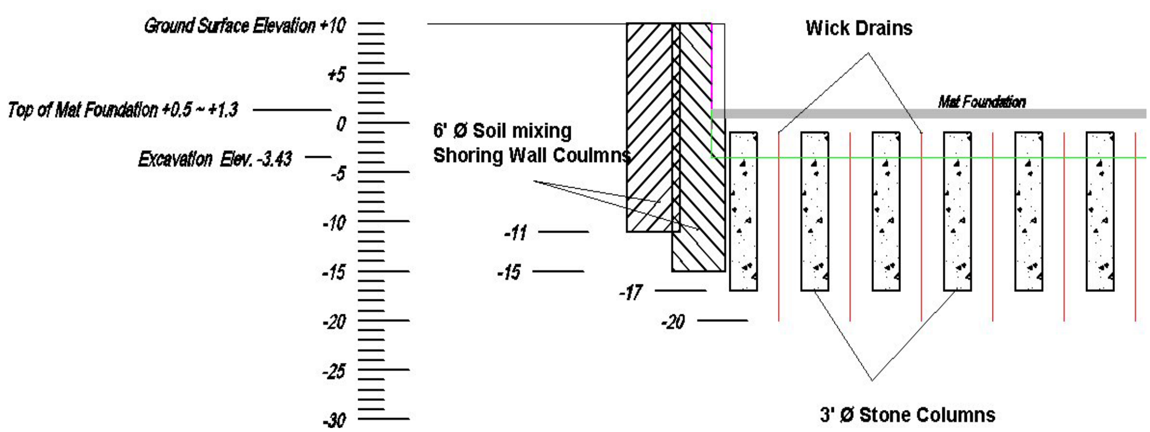


Figure 5-5 Elevation of the stone column and wick drain treatment at the Marina del Rey case history.

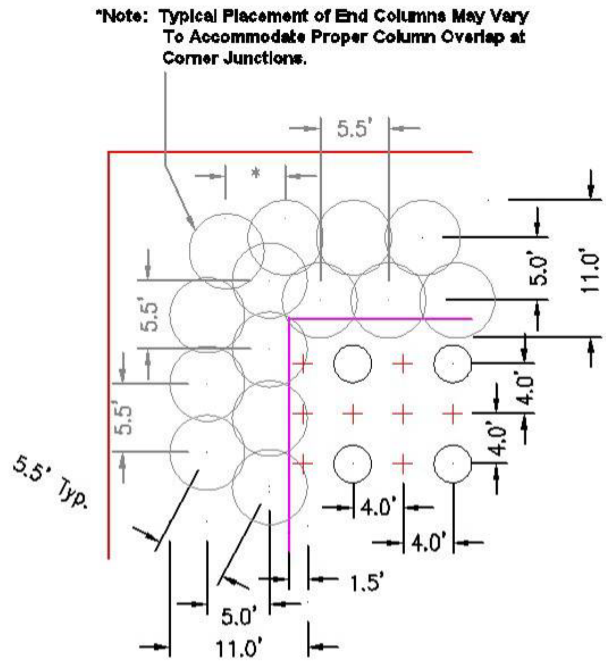


Figure 5-6 Typical plan detail for the stone column and wick drain treatment at the Marina del Rey case history.

5.3 Results and Analysis

CPT's were used by Hayward Baker to evaluate the effectiveness of the mitigation procedures. The post-treatment CPT's were done mostly within 2 weeks of the date of treatment. For the Marina del Rey site, CPT-SPT as well as the apparent fines correlations were applied by MACTEC during the preparation of their analyses using the Robertson and Wride (1998) correlation.

Pre-treatment and post-treatment CPT borings within 15 feet of each other were compared directly. The remaining boring data were not compared directly but instead they were included in the overall site scatter plot analyses as well as in the table of averages for the entire site.

A scatter plot showing depth versus initial and final $(N_1)_{60}$ values for the entire site is shown below in Figure 5-7. Figure 5-7 (a) shows the values for the data points with low fines contents while Figure 5-7 (b) shows the values for the data points with high fines contents. The cutoff between low and high fines contents is taken as 15% fines contents. Where fines contents were not available, the cutoff was taken to be an I_c value of 2.05 or a SBT value of 6. Values below I_c = 2.05 or greater than or equal to SBT = 6 indicate soils with fines contents low enough to be classified as clean sands with fines contents likely below 15%.

$(N_1)_{60}$ values vary from approximately 5 to 50 blows per foot for plot (a) and from approximately 5 to 40 blows per foot for Figure 5-7 (b). The average lines in Figure 5-7 (a) show that from 10 to 20 feet there was noticeable improvement when fines contents were low (less than 15%). The only negative improvement is above 10 feet where there are a limited number of final values available or below the treatment depth of 20 feet where improvement is not expected. Average improvement ranged from 3 to 14 blow counts in the positive improvement zone. The average lines in Figure 5-7 (b) show that the soil with high fines contents (greater than or equal to 15%) exhibited little improvement with some areas of negative improvement. The average improvements ranged from -2 to 2 blows counts. In general the method was not effective for soils with high fines contents. As noted previously, and as seen in Figure 5-4, the sand was very fine grained which may possibly have inhibited mitigation efforts in a manner similar to soils with high fines contents.

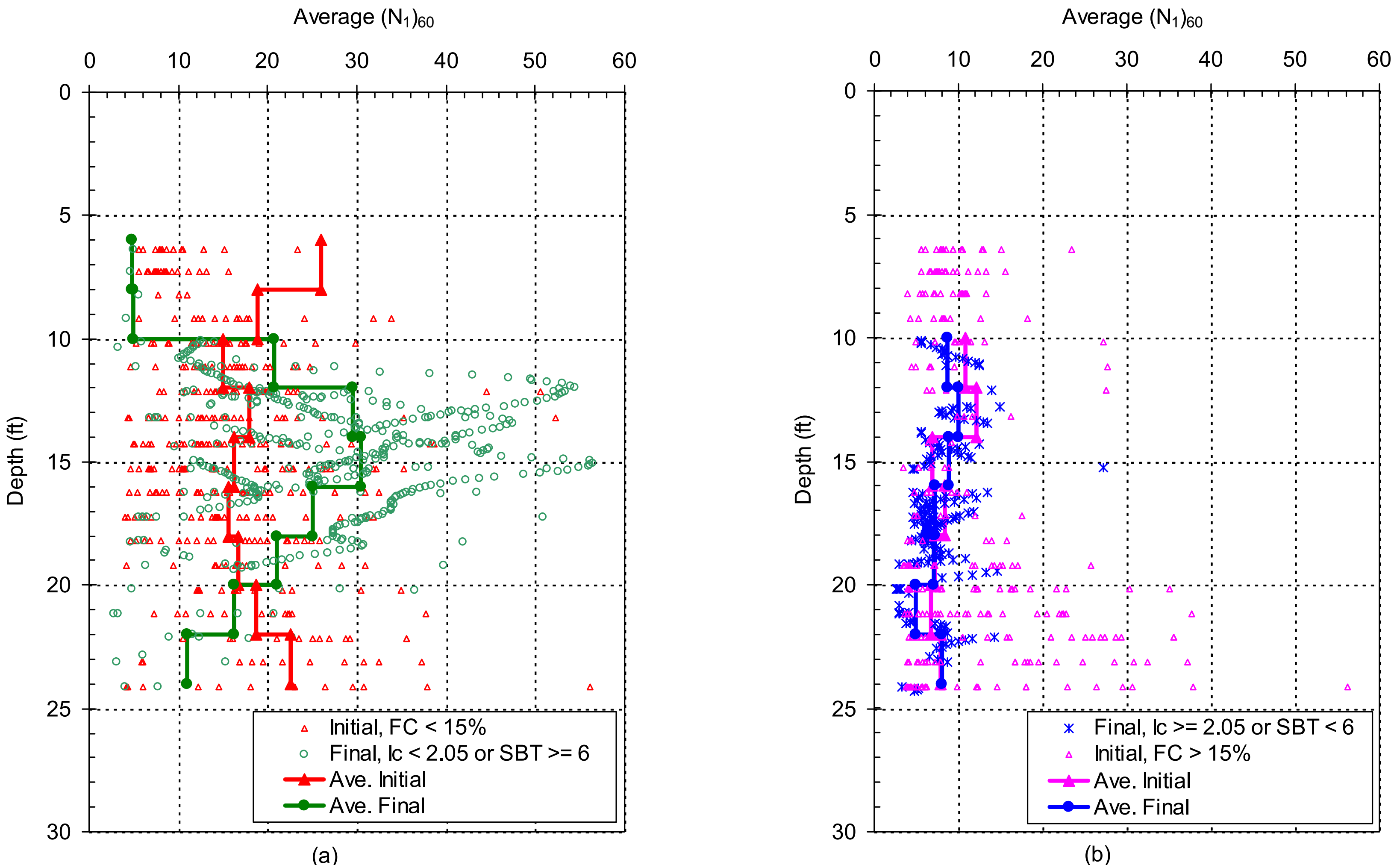


Figure 5-7 Depth versus initial and final $(N_1)_{60}$ values for the entire site divided into figures with (a) fines contents less than 15% or the equivalent and (b) fines contents greater than or equal to 15% or the equivalent for the Marina del Rey case history.

Direct comparisons of pre- and post-CPT data are plotted in Figure 5-8 through Figure 5-10. It can be seen that the data are rather varied. There are some cases where the final value is less than the initial value; however for most cases the final value is greater than the initial value. The comparison between soundings 1 and 12 in Figure 5-9(a) illustrates this. Some values show positive improvement, some show negative improvement, and still others stay about the same. One possible explanation for this peculiar phenomenon is that the soundings were not close enough to directly compare due to the variations in the soil profile.

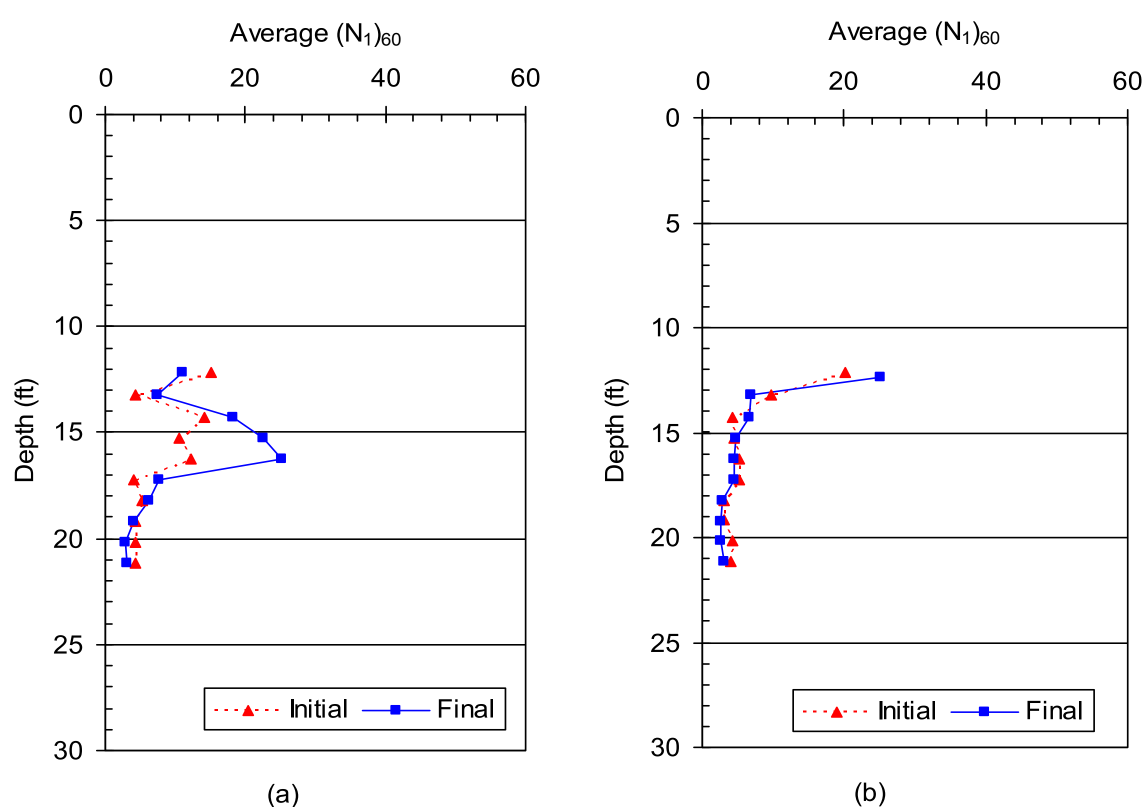


Figure 5-8 Marina del Rey direct comparison plots of $(N_1)_{60}$ vs. depth for pre-post soundings (a) 17 - 27, and (b) 24 - 47.

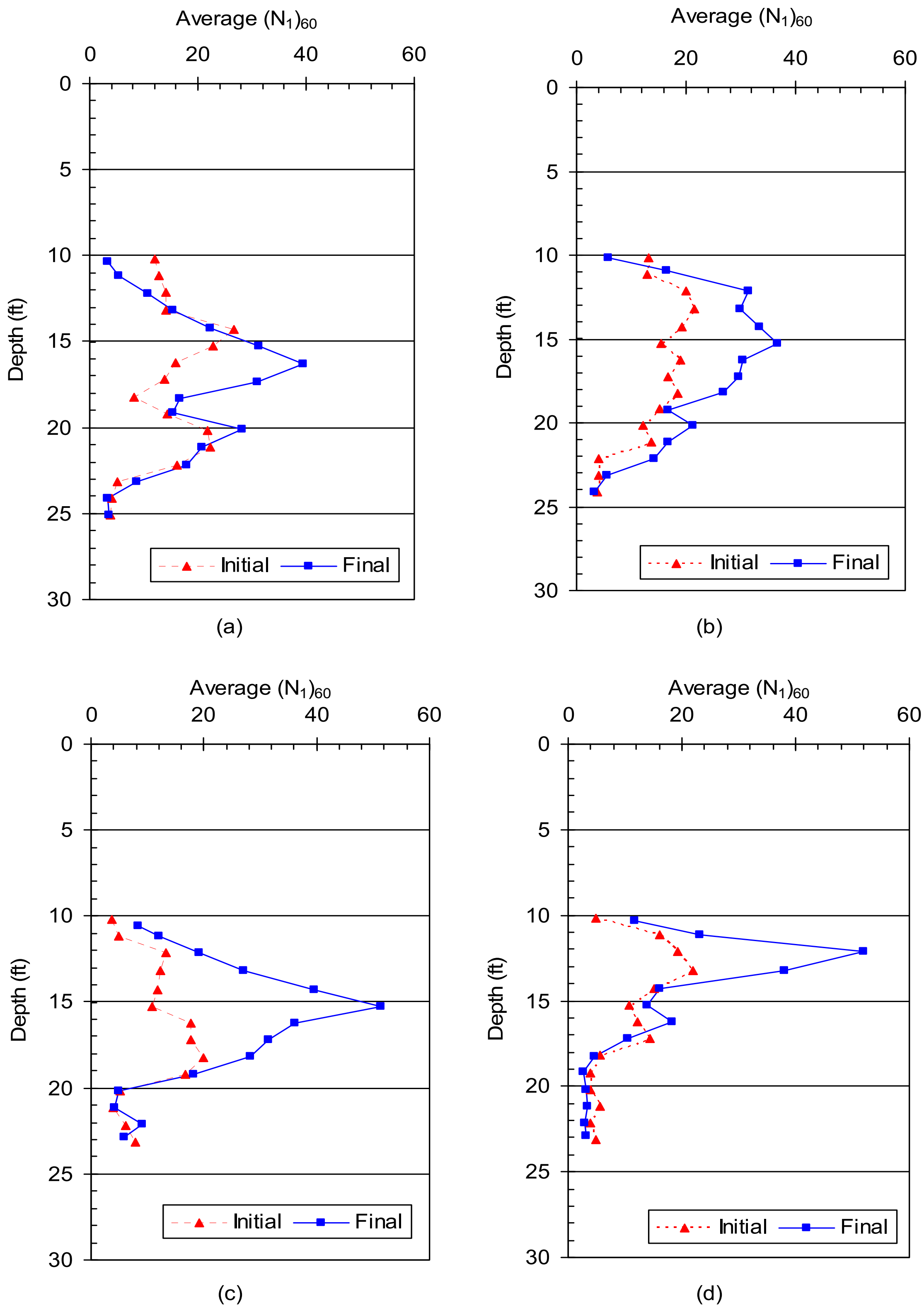


Figure 5-9 Marina del Rey direct comparison plots of $(N_1)_{60}$ vs. depth for pre-post soundings (a) 1 – 12, (b) 2 - 15, (c) 7 - 60, and (d) 9 - 63.

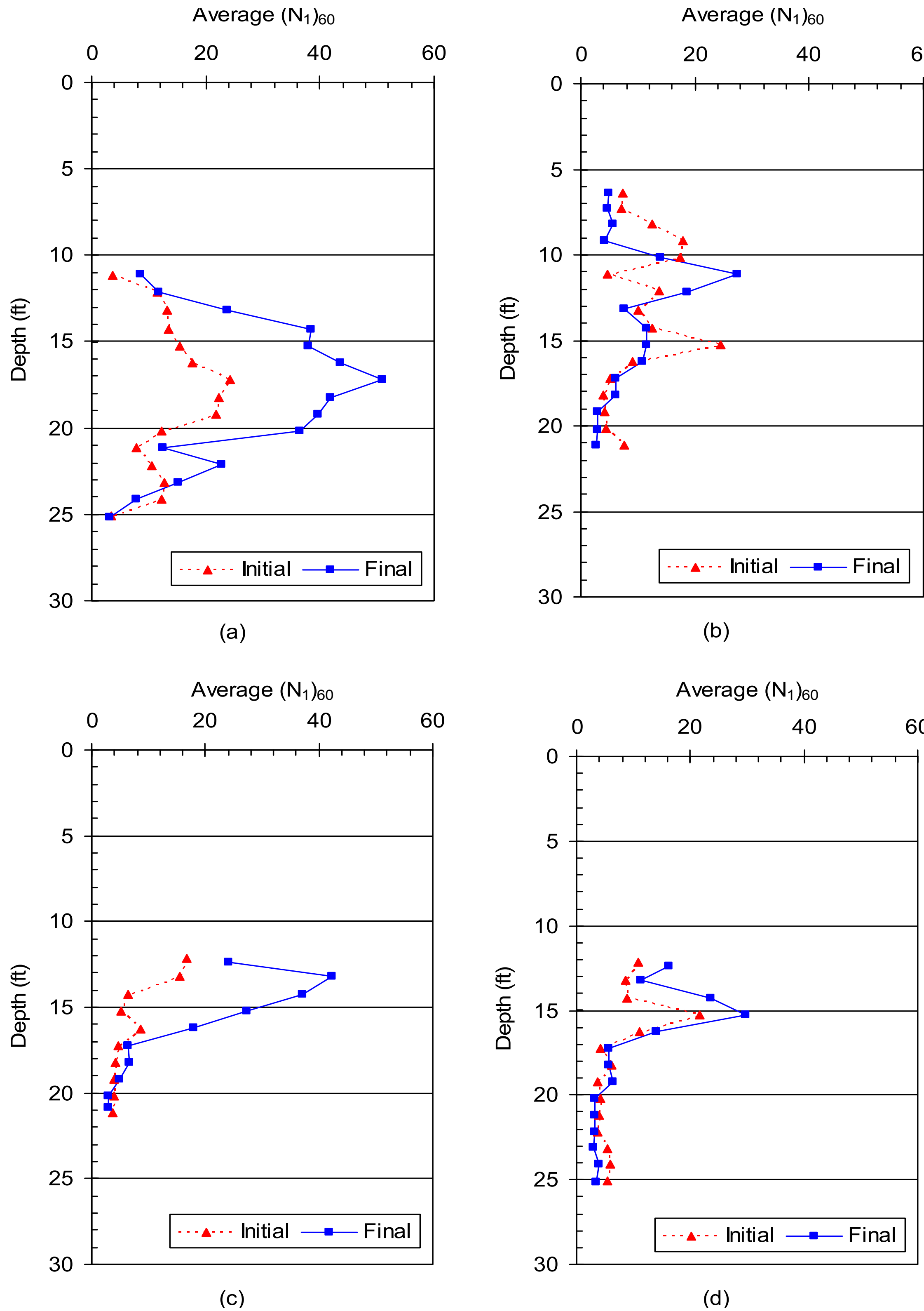


Figure 5-10 Marina del Rey direct comparison plots of $(N_1)_{60}$ vs. depth for pre-post soundings (a) 3 - 17, (b) 16 - 28, (c) 13 - 54, and (d) 15 – 38A.

On average there was an overall improvement for the direct comparison soundings at the site. This can be seen in Figure 5-11 where all of the pre- and post-$(N_1)_{60}$ data for the direct comparison CPT's is plotted versus depth. The averages for every 2 foot depth interval are also plotted. The averages indicate that in general there was improvement throughout the soil profile except as the boring neared the depth of 25 feet. This is likely due to the treatment depth being only 20 feet. It is also seen from Figure 5-11 that there was greater improvement for the soils with fines contents less than 15%.

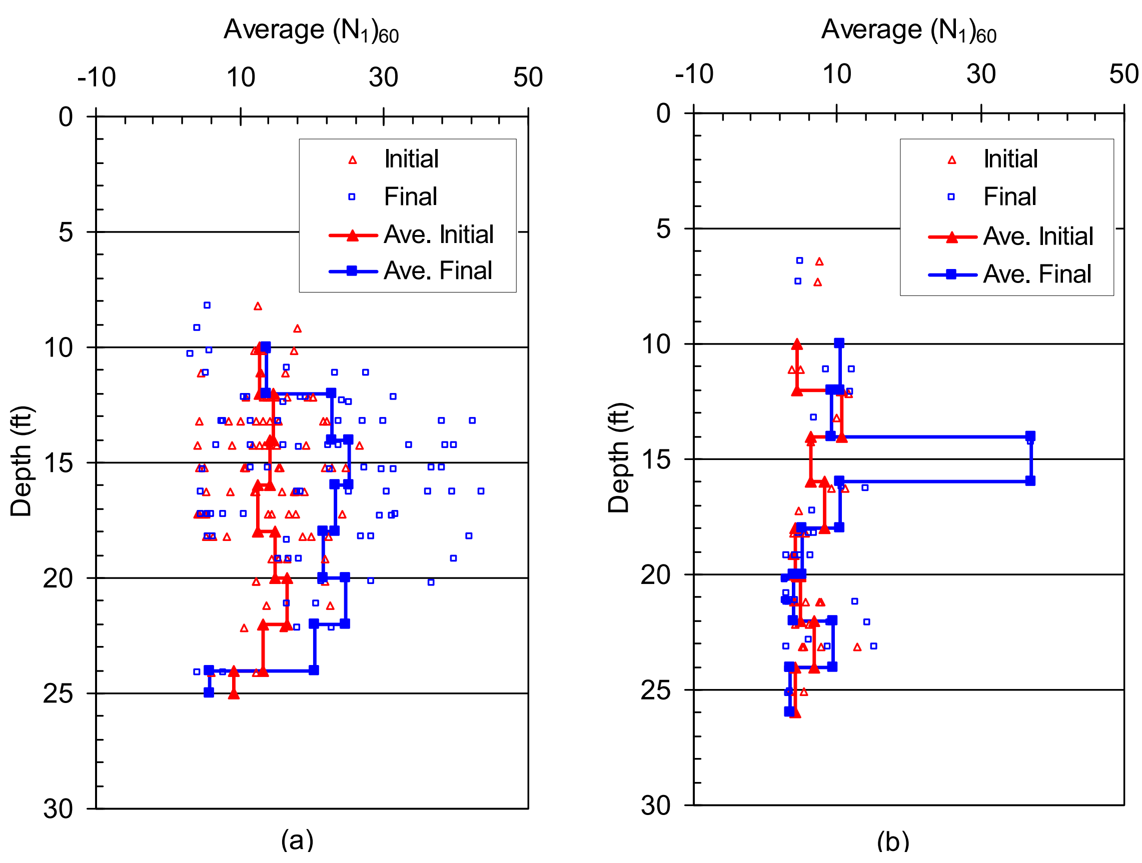


Figure 5-11 Marina del Rey direct comparison plots of $(N_1)_{60}$ vs. depth with averages for (a) fines content less than 15%, and (b) fines content greater than 15% but less than 50%.

It should also be noted that the fines contents are the correlated fines contents from the CPT data. This means that the actual fines contents are slightly higher based on the SPT and CPT fines comparisons chart shown previously in Figure 5-3. Another factor is that the sand was very fine sand, also contributing to the soil acting more like a fine soil instead of a granular soil.

To determine the factors affecting improvement, the change in blow count, $\Delta(N_1)_{60}$, was plotted versus initial $(N_1)_{60}$, fines content, and depth. The plot showing $\Delta(N_1)_{60}$ versus fines content is located in Figure 5-12. The logarithmic trend line has a low R-squared value, only 0.0896; however the data points in the figure suggest that the fines content has a large impact on the improvement.

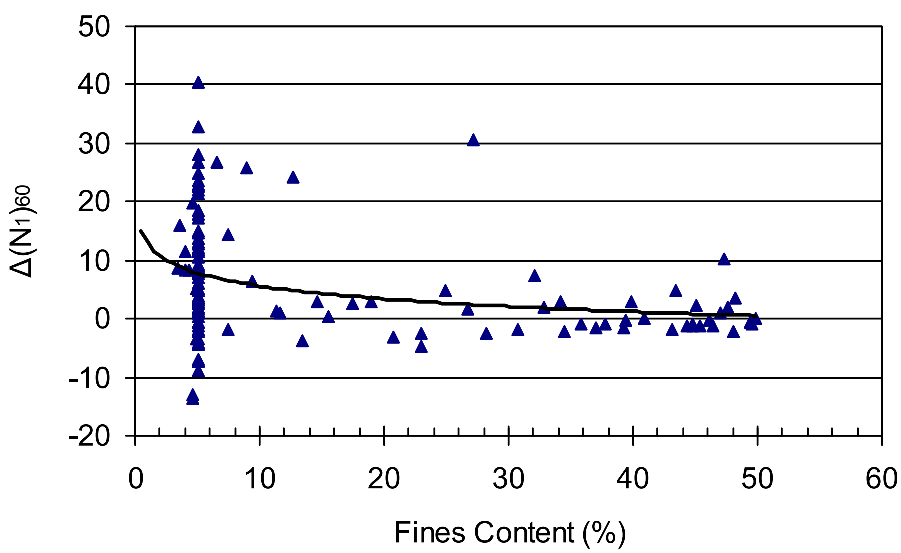


Figure 5-12 $\Delta(N_1)_{60}$ vs. fines content with a logarithmic trend line for the direct comparison at the Marina del Rey case history data.

For fines contents less than 10% the improvement is extremely varied, but in some cases it was as much as 40 blows per foot improvement. As the fines content increased above 15% the improvement continued to vary, but by much less than for fines content less than 15%. For fines contents higher than 15% the data points basically scatter about the zero improvement line. With one exception, the improvement in blow count for fines greater than 15% was 10 blows per foot or less. Similar to stone columns without wick drains, high fines content detrimentally affects the efficiency of the stone columns with wick drains.

$\Delta(N_1)_{60}$ is plotted versus initial $(N_1)_{60}$ in Figure 5-13. Although there appears to be a slight trend of increased improvement with increased initial $(N_1)_{60}$, the scatter is significant and it is not reasonable to use linear regression to predict improvement based upon initial $(N_1)_{60}$ values alone. A linear trend line is shown in the figure, but it only gives the reader a general idea of the trend instead of accurately portraying the data. The R-squared value for the linear regression line shown was 0.075 which indicates that the linear relationship between $\Delta(N_1)_{60}$ and initial $(N_1)_{60}$ is insignificant.

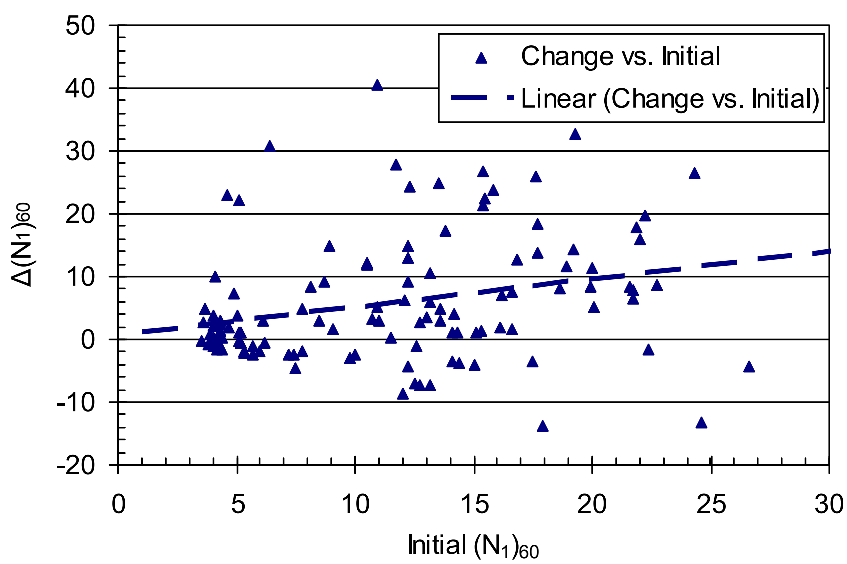


Figure 5-13 $\Delta(N_1)_{60}$ vs. Initial $(N_1)_{60}$ for all direct comparison data of the Marina del Rey case history.

$\Delta(N_1)_{60}$ is plotted versus depth in Figure 5-14 below. There is a slight trend in the data but it is not significant enough to conclude a correlation between $\Delta(N_1)_{60}$ and depth.

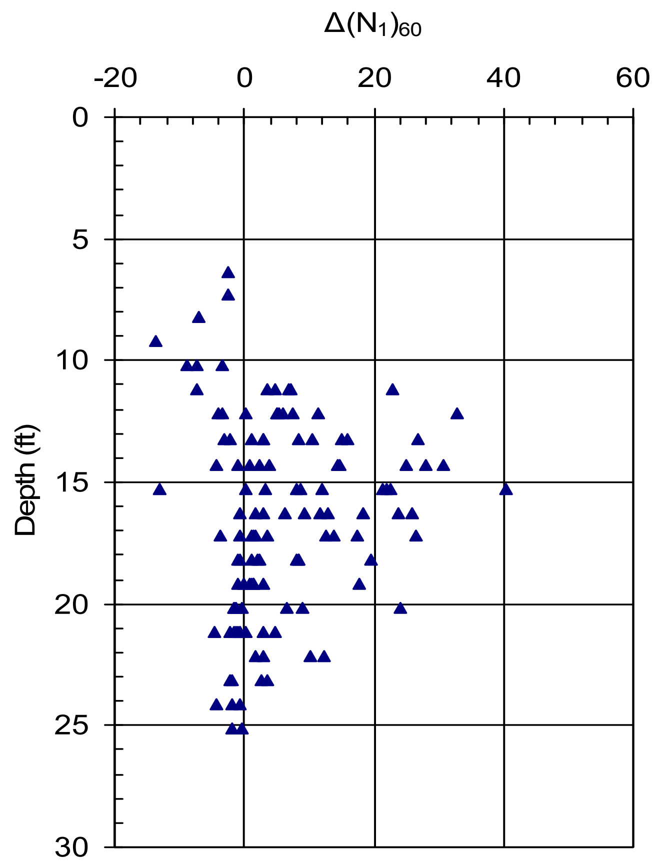


Figure 5-14 $\Delta(N_1)_{60}$ vs. depth for the direct comparison data of the Marina del Rey case history.

An alternative to using $\Delta(N_1)_{60}$ as the dependant variable is to use the final $(N_1)_{60}$ as the measure of improvement. In most cases the practicing engineer will be more interested in the final blow count than the $\Delta(N_1)_{60}$ value because usually a minimum or average $(N_1)_{60}$ value will be the measure of their mitigation efforts. Final $(N_1)_{60}$ versus initial $(N_1)_{60}$ is plotted below in Figure 5-15. Linear regression between final $(N_1)_{60}$ and initial $(N_1)_{60}$ yields an R-squared value of 0.5039 which is significantly better than the R-squared value between $\Delta(N_1)_{60}$ and initial $(N_1)_{60}$. However it is observed that there is a significant degree of scatter in the data and that linear regression would be inappropriate in determining a direct linear relationship.

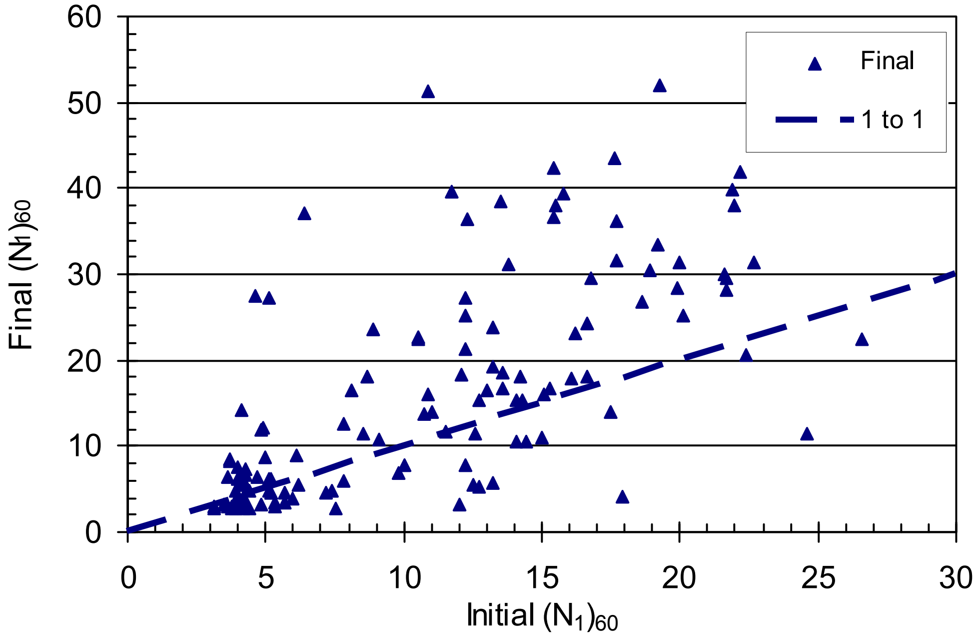


Figure 5-15 Final $(N_1)_{60}$ vs. initial $(N_1)_{60}$ with a 1 to 1 line for the direct comparison data of the Marina del Rey case history.

Average $(N_1)_{60}$ values for the direct comparison data of the Marina del Rey case history are found below in Table 5-1. Average $(N_1)_{60}$ values for all of the data at the Marina del Rey case history are found below in Table 5-2. The values are divided according to fines contents less than and greater than 15%. The fines contents were all generally less than 50% throughout the site.

Table 5-1 Average $(N_1)_{60}$ values based on fines contents for the direct comparison data of the Marina del Rey case history.

Fines (%)	Sample Size	Standard Deviation (Finals)	Average $(N_1)_{60}$			
			Initial	Final	Change	Increase
0-15	91	13	18	22	4	22%
15-50	42	6	6	7	1	15%
0-50	133	16	14	17	3	20%

Table 5-2 Average $(N_1)_{60}$ values based on fines contents for all of the data at the Marina del Rey case history.

Fines (%)	Sample Size	Standard Deviation (Finals)	Average $(N_1)_{60}$			
			Initial	Final	Change	Increase
0-15	798	13	18	26	9	48%
15-50	379	3	9	8	-1	-11%
0-50	1177	14	15	21	6	39%

The average values show that the $(N_1)_{60}$ values are much less in areas with high fines content. In the 15-50% fines content category the initial and final $(N_1)_{60}$ values are both less than 10 and the change is only 1 blow count for the direct comparison data and -1 blow count for the entire site data. Both values indicate that if the fines contents are high and the initial value is low then there will be very little improvement. In all the case histories examined in this thesis, a final $(N_1)_{60}$ value of 7 or 8 would not have met the improvement criteria. In contrast, the average $(N_1)_{60}$ values in the 0-15% fines content category went from an initial blow count of 18 to a final blow count of 22 in the direct comparison data and 26 in the overall site data. Increasing the number of points used in analysis typically yields a better representation of the data. By examining the entire site data we see an even greater increase in the low fines category and a decrease in the high fines category. This emphasizes the observation that high fines content appears to significantly limit improvement for the stone column with wick drains method.

The standard deviations for Table 5-1 were very high, 59-95% of the final values from which they were derived. The standard deviations for Table 5-2 were also high, 39-67% of the average final values; however, they were less significant than those of the direct comparison tests. This is possibly due to poor comparisons when directly compared being less influential on the overall test site averages. The direct comparison data were considered to be close enough and representative of similar enough soils to use in analysis; however, the overall site average would naturally be more representative of the data since the sample size was so much larger and outlying data would not affect the averages as much. The large standard deviations should be noted when referencing the average data for the site.

The area replacement ratio for the Marina del Rey site was only 11% with a stone column spacing of 8 ft. by 8ft. with wick drains centered between columns, as shown in Figure 5-6. It appears that this column and drain spacing is inadequate to improve soils with fines contents greater than 15% and low initial blow counts. Approximately two thirds of the data for the Marina del Rey site had fines contents less that 15% though so the overall objective of the site was achieved. The majority of the post-improvement CPT soundings were taken within 2 weeks of installing the wick drains and stone columns. This time frame would have allowed for the initial improvement to occur, but testing at a later date would likely have yielded greater gains with time.

To compare the results for this case history to the published literature, final $(N_1)_{60\text{-}cs}$ values are plotted versus initial $(N_1)_{60\text{-}cs}$ and a logarithmic trend line is obtained in Figure 5-16. The logarithmic trend line is then compared to the curves developed by Baez (1995) for clean sands with fines contents less than 15% which are also presented in

Figure 5-17. The trend lines should be considered to be prediction of average final blow count than a prediction of specific final blow counts due to the significant scatter with R-squared values being less than 0.02; however, by comparing those to the Baez clean sand curves the effectiveness of the Marina del Rey site treatment plan may be compared generally. The logarithmic trend lines for the Marina del Rey site were developed from data with initial clean sand blow counts greater than 5-10 initial blow count so the trend lines that are compared to the Baez curve likewise begin at 5-10 initial blow count.

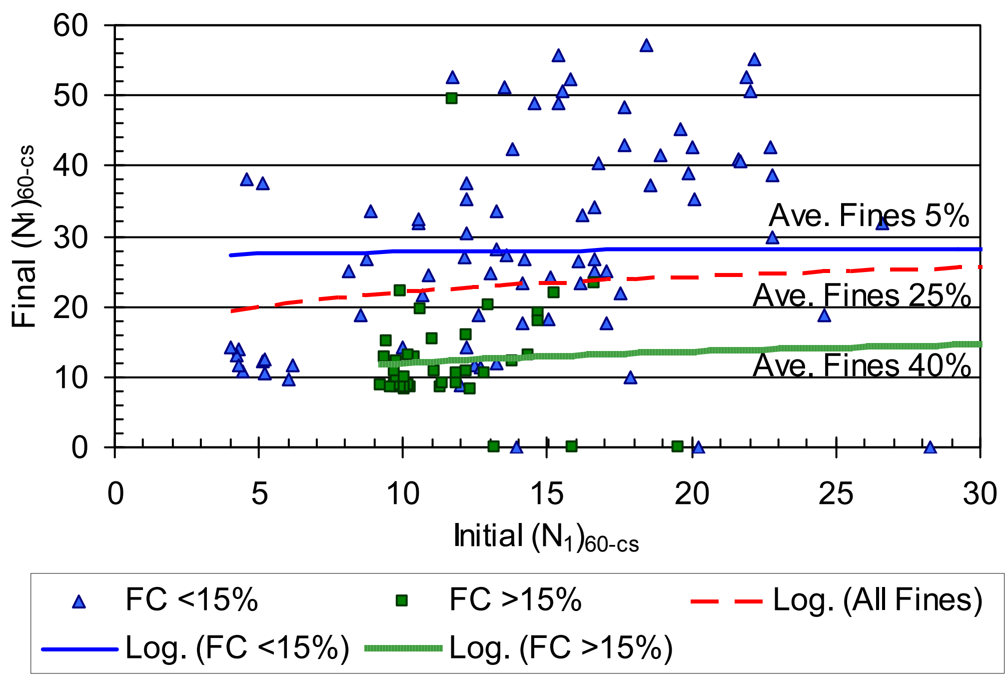


Figure 5-16 Initial $(N_1)_{60\text{-cs}}$ versus final $(N_1)_{60\text{-cs}}$ with logarithmic trend lines for the different fines content categories (<15%, >15%, all fines) of the Marina del Rey case history.

The Marina del Rey site with its 25% average fines content and 11% A_r with wick drains is not very comparable to the curves presented by Baez. The Marina del Rey trend lines have very small slopes and predict a relatively constant final clean sand blow count.

Some error may be present since the clean sand values are corrected using correlated apparent fines contents obtained from CPT test results. The overall site with an average fines content of 25% predicted 20-25 blows/ft for the final blow counts which is comparable to the 5% A_r clean sand curve at an initial clean sand blow count of 10, but for higher initial blow counts it predicted much less improvement (about 8 blows/ft less at an initial blow count of 20). The wick drains did not supplement the stone columns enough to counteract the negative effect of the fines content at high initial blow counts, although sufficient improvement was attained to meet the project objectives. Increasing fines content clearly decreased improvement once the clean sand blow counts were utilized.

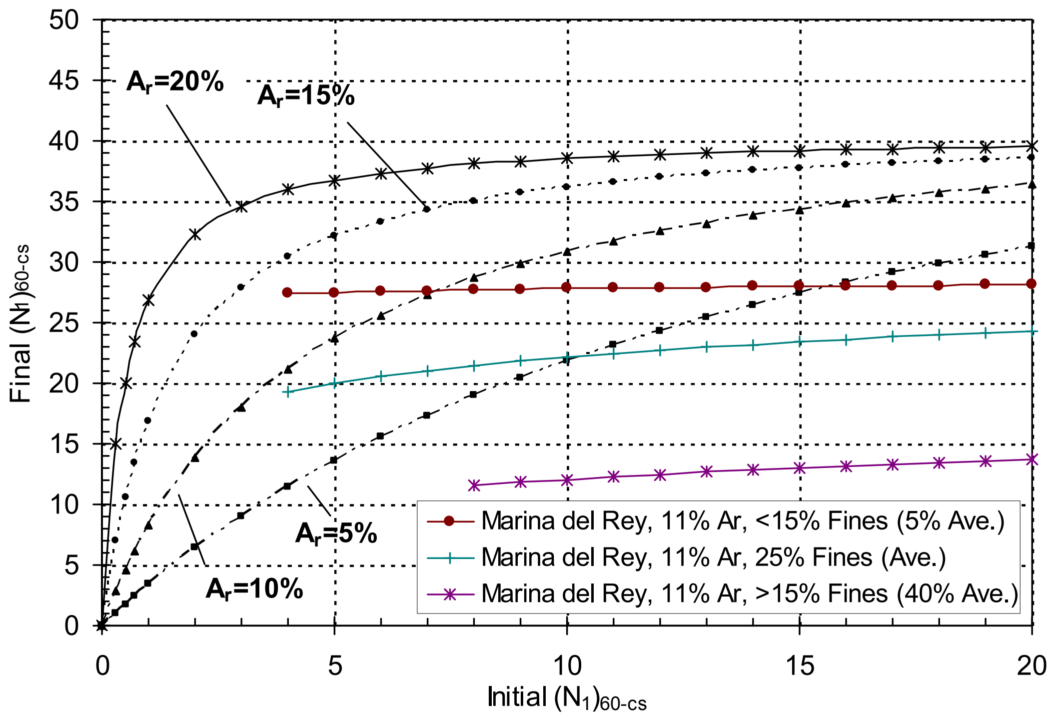


Figure 5-17 Initial $(N_1)_{60\text{-cs}}$ versus final $(N_1)_{60\text{-cs}}$ with the clean sand curves developed by Baez (1995) as well as the logarithmic trend lines for the Marina del Rey case history silt and sand data.

5.4 Conclusions

- The data for the Marina del Rey case history were too scattered to perform linear regression and the linear regression that was attempted yielded poor R-squared correlation values. Standard deviations of the final blow counts were large, 39-95% of the average final values.
- Fines content and initial $(N_1)_{60}$ appear to have some effect on improvement. High fines contents and low initial $(N_1)_{60}$ values tend to limit improvement significantly.
- Sands that are very fine grained appear to have little improvement similar to soils with high fines contents. The site had an average fines content of 24.5% with very fine sands.
- On average, for the entire site, there was 39% improvement with final $(N_1)_{60}$ values of 26 blows/ft for soils with fines contents less than 15% (average fines 5%) and 8 blows per foot for soils with fines contents greater than 15% (average fines 40%). The standard deviation of the final $(N_1)_{60}$ values was high (14).
- The Marina del Rey site did not exhibit similar results as the clean sand curves developed by Baez (1995) and the improvement was generally more constant than a clean sand site (<15%). For an initial blow count of 5 the improvement was close to the 10% A_r clean sand curve but at an initial blow count of 20 the improvement was less that that of the 5% A_r clean sand curve. The data with an average fines content of 40% consistently performed worse than the clean sand 5% A_r curve.

- The 11% area replacement ratio was insufficient to effectively improve soils with fines contents greater than 15%. In order to improve the soils with high fines contents a closer drain spacing or higher area replacement ratio is suggested.

6 Home Depot, San Pedro, California

6.1 Site Overview and Soil Conditions

The Home Depot site being considered is located at the corner of Gaffery Street and Westmont Drive in the area of San Pedro of the City of Los Angeles, California. The facility consists of a 134,000 square foot store and garden center facility. The building is supported by a foundation system of shallow spread footings over improved soil.

Kleinfelder was the geotechnical engineer of record with Hayward Baker, Inc. providing the design of the ground improvement. Geotechnical reports from both Kleinfelder and Hayward Baker were referenced in preparing this case history. The design earthquake for the site is a magnitude 7.1 earthquake with peak ground acceleration of 0.6g. The site is within 1 mile of the Palos Verdes Hills fault. The site mean sea level (MSL) elevations ranged from 28 to 70 feet.

The preliminary geotechnical investigation classified the soil as artificial fill and alluvium deposits. The upper 2 to 12.5 feet of the soil profile consisted of undocumented fill (silty to gravelly sand, sandy silt, and silty clay), underlain by alluvial soils consisting of interbedded silty sand, sandy to clayey silt, silty clay, and gravelly sand to approximately 65 feet. Pleistocene deposits of medium stiff to stiff sandy clay were noted in the top 7 to 15 feet of the soil profile. The clays encountered were typically moderately overconsolidated. Ground water depths varied between 12 feet and 25 feet in the treatment area. The soil behavior types identified by the CPT sounding in this case history were predominantly silty sand (SM) to sandy silt (ML) in the upper 9 to 11 feet with some silty clay (CL) and clayey silt (ML) layers. From about 11 feet to 30 feet it was predominantly silty clay and clayey silt with some sandy silt and silty sand layers, and then from 30 feet to 65 feet it was generally silty clay and clayey silts.

Figure 6-1 presents a generalized cross section of the soil profile at the site based on the preliminary geotechnical investigation as well as the CPT data used for this case history. The soil was intermixed with multiple layers of the differing soil types present in each section of the soil profile. Figure 6-2 presents depth versus fines content for the site using two plots. Plot (a) presents the entire site's data including all soil types while plot (b) presents the silt and sand mixtures whose data points were used for the analyses.

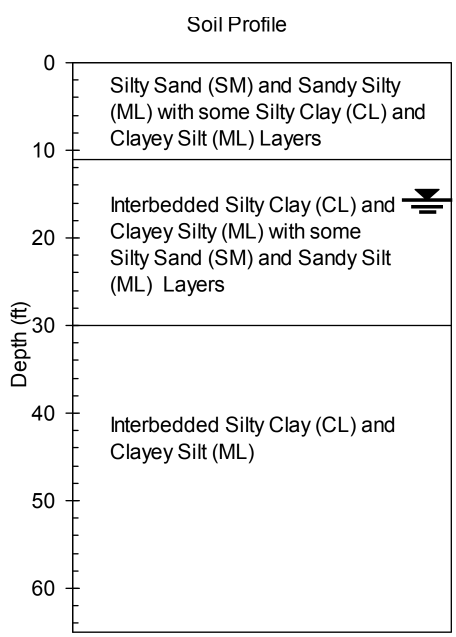


Figure 6-1 San Pedro Home Depot case history generalized cross section.

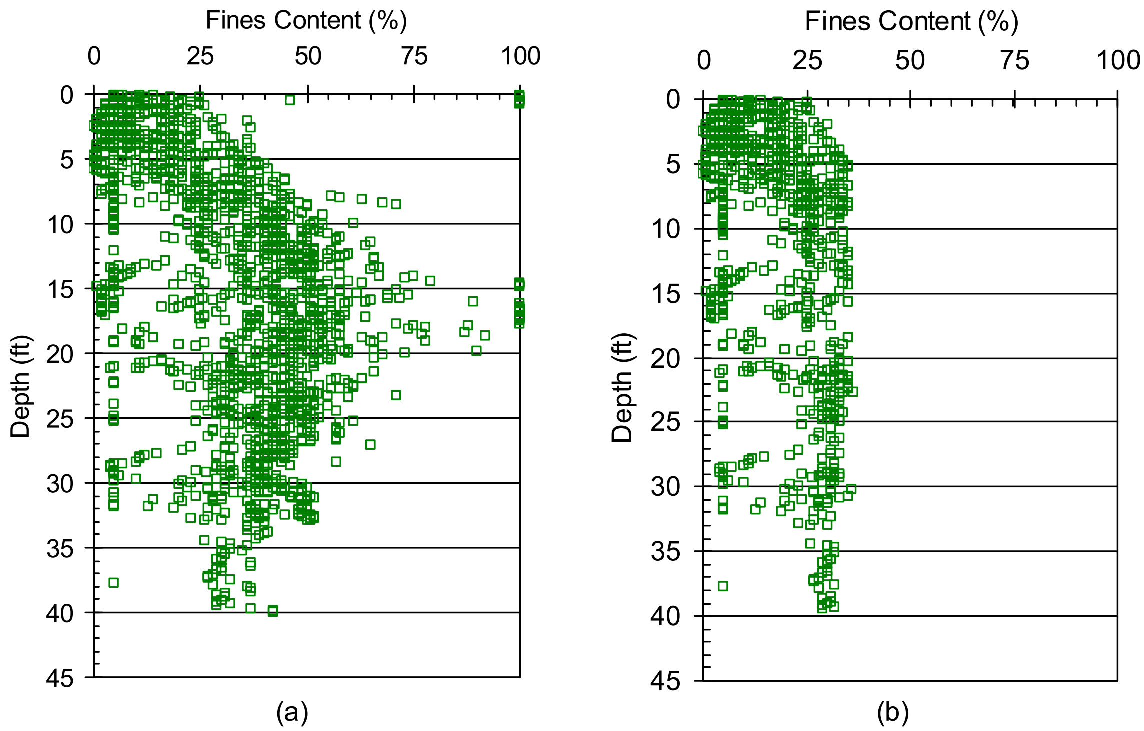


Figure 6-2 Apparent fines content versus depth for the entire treatment depth including all soil types (a) and apparent fines content versus depth for the treatment depth only including silt and sand mixtures only (b) for the San Pedro Home Depot case history.

Hayward Baker performed the correlation between the CPT data and the apparent fines content during the preparation of their analyses using the Robertson and Wride (1998) correlations. All of the fines contents referenced in this case history are correlated apparent fines contents. The average apparent fines content for the site (including all soil types) was 33% while the average apparent fines content for the silt and sand mixtures used in analyses was 18%. The apparent fines contents tend to increase with depth for both plots with fines contents decreasing slightly below 17 feet for plot (a). The CPT method uses the soil index I_c from which the SBT value and the apparent fines content are derived. I_c values of 2.60 yield apparent fines contents of 35% and soil behavior type (SBT) values of 4 which classify as silty clay or clayey silt (Robertson and Wride, 1998).

The focus of this study is to determine the effectiveness of stone columns and wick drains in silt and sand mixtures which is why plot (b) truncates at 35% fines content. I_c values greater than 2.60 with corresponding apparent fines contents above 35% do not fall within the scope of this study. For the liquefaction analyses performed by Hayward Baker, an I_c value of 2.6 was taken as the value above which soils are nonliquefiable. Liquefaction analyses were done according to the Youd and Iddriss NCEER 1997 procedure as well as the Martin and Lew SCEC, 1999 procedure. Liquefiable layers were found to exist with thicknesses varying from 6 inches to 4 feet.

6.2 Treatment Method

The purposes of soil treatment included limiting both liquefaction settlement as well as post construction static settlements. Dry bottom-feed vibro-stone columns were used to accomplish the mitigation goals. Due to high fines contents from the silty clay and clayey silt lenses and layers, pre-installed wick drains were used in addition to stone columns. The primary purposes of the column and wick drain treatment were densification and reinforcement of the surrounding soil; however, drainage of the excess pore water pressure was a secondary objective. Soil improvements took place during 1999.

Stone columns were installed primarily in a 10 foot center-to-center square arrangement with a secondary stone column installed in the middle of the primary pattern for soils determined to have a high liquefaction potential. Additionally, four stone columns were installed at the corners of every building column location. Stone column diameters were approximately 3 feet. Treatment depths varied from 20 feet to 52 feet for both stone columns and wick drains. An S series vibrator with 120 kW of power was used to install the stone columns using the dry bottom feed method. A sample section of the treatment layout is presented in Figure 6-3 where primary stone columns are shown as empty circles, wick drains as hourglasses, secondary columns as circles with an x inside, and stone columns supporting building columns as double lined circles.

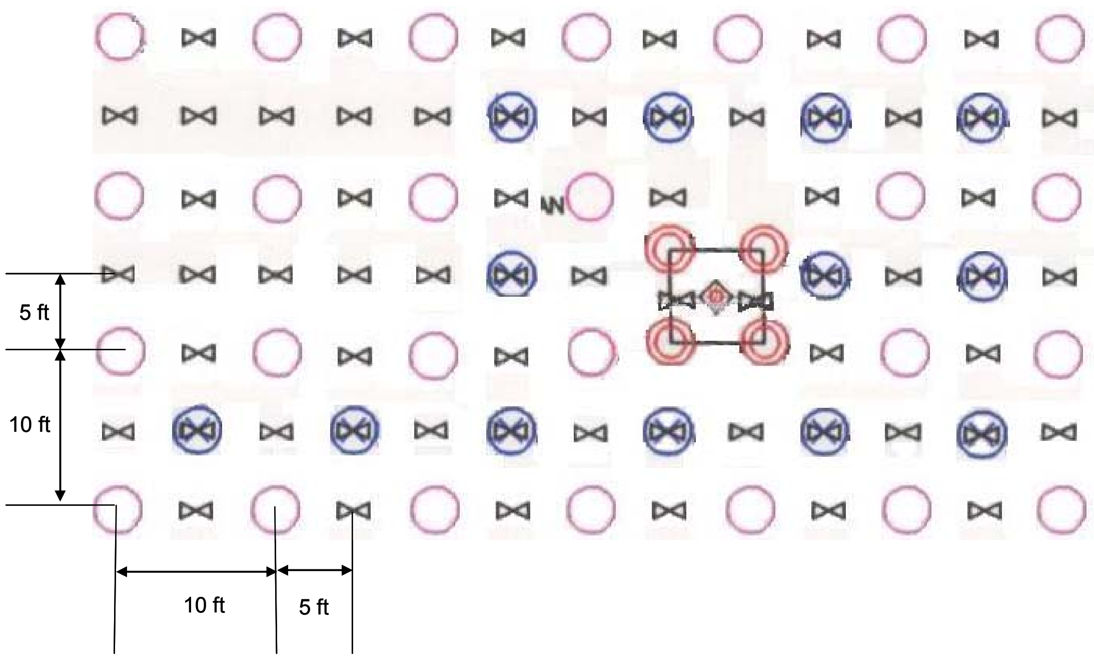


Figure 6-3 Sample stone column and wick drain layout for the San Pedro Home Depot case history. Primary stone columns are shown as empty circles, wick drains as hourglasses, secondary columns as circles with an x inside, and stone columns supporting building columns as double lined circles.

6.3 Results and Analysis

CPT's were used by Hayward Baker to evaluate the effectiveness of the mitigation procedures. The post-treatment CPT's were performed between 6 and 40 days after treatment with the majority of the soundings taken at about 2 weeks after treatment. For the San Pedro site, the correlation between the CPT data and the SPT blow count were applied by Hayward Baker during the preparation of their analyses using the Robertson and Wride (1998) correlations.

Pre-treatment and post-treatment CPT soundings within 10 feet of each other were compared directly. Due to the varying stone column arrangements, especially at building column locations and at areas of high liquefaction potential, the area replacement ratios varied from approximately 7.1% to 17.7%. The majority of the data available for this case history were in the primary stone column arrangement regions which had an A_r of about 7.1%. The remaining data came from areas of high liquefaction potential or at building column locations where additional stone columns were installed to increase the area replacement ratio. There were only one set of comparable CPT soundings for each additional A_r so there was not enough data to utilize these areas in analysis. Therefore, only the areas with 7.1% A_r will be analyzed

The interbedded nature of the soil profile makes it difficult to isolate the silty sand and sandy silt layers for analysis. The focus of this research is to determine the usefulness of wick drains with stone columns in silty sands. CPT soundings typically include the soil behavior type index (SBT) from which the soil classification can be inferred. An SBT value of 5 indicates a silty sand or sandy silt, and a value of 6 indicates a clean sand or silty sand. The SBT classifications relevant to the San Pedro case history site are depicted in Figure 6-4 below. Values just below 5 typically indicate clay mixtures. To isolate silty soils, each direct comparison data set was plotted first with depth versus initial and final $(N_1)_{60}$ and second with depth versus the soil behavior type index (SBT). Once the SBT values were known for a boring, the data set was limited to include only data with SBT values of 5 or 6 in order to only include potentially silty sands. Thin layers of silty sands were also excluded due to the influence of thin layers on the interpretation of penetration test results. Robertson and Wride (1998) explained that theoretical as well as laboratory studies show that CPT results are affected by the soil ahead of and behind the CPT cone and that this causes misinterpretations of the properties of thinly interbedded soils. Thin layers of approximately 1 – 2 feet or less without nearly continuous SBT classifications greater than 5 were excluded from the data set. A sample direct comparison is presented in Figure 6-5 for initial CPT sounding 11 and final CPT sounding 31.

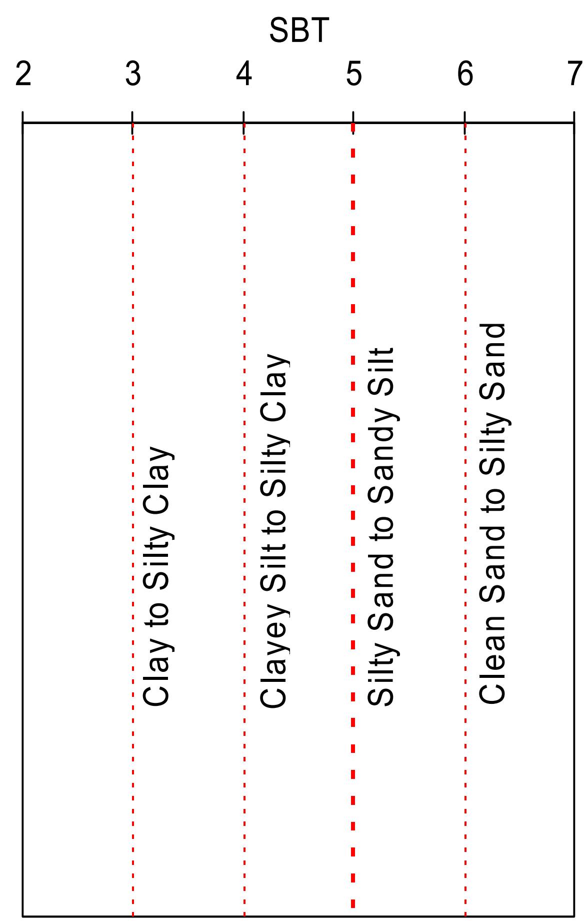


Figure 6-4 SBT classifications relevant to the San Pedro Home Depot case history.

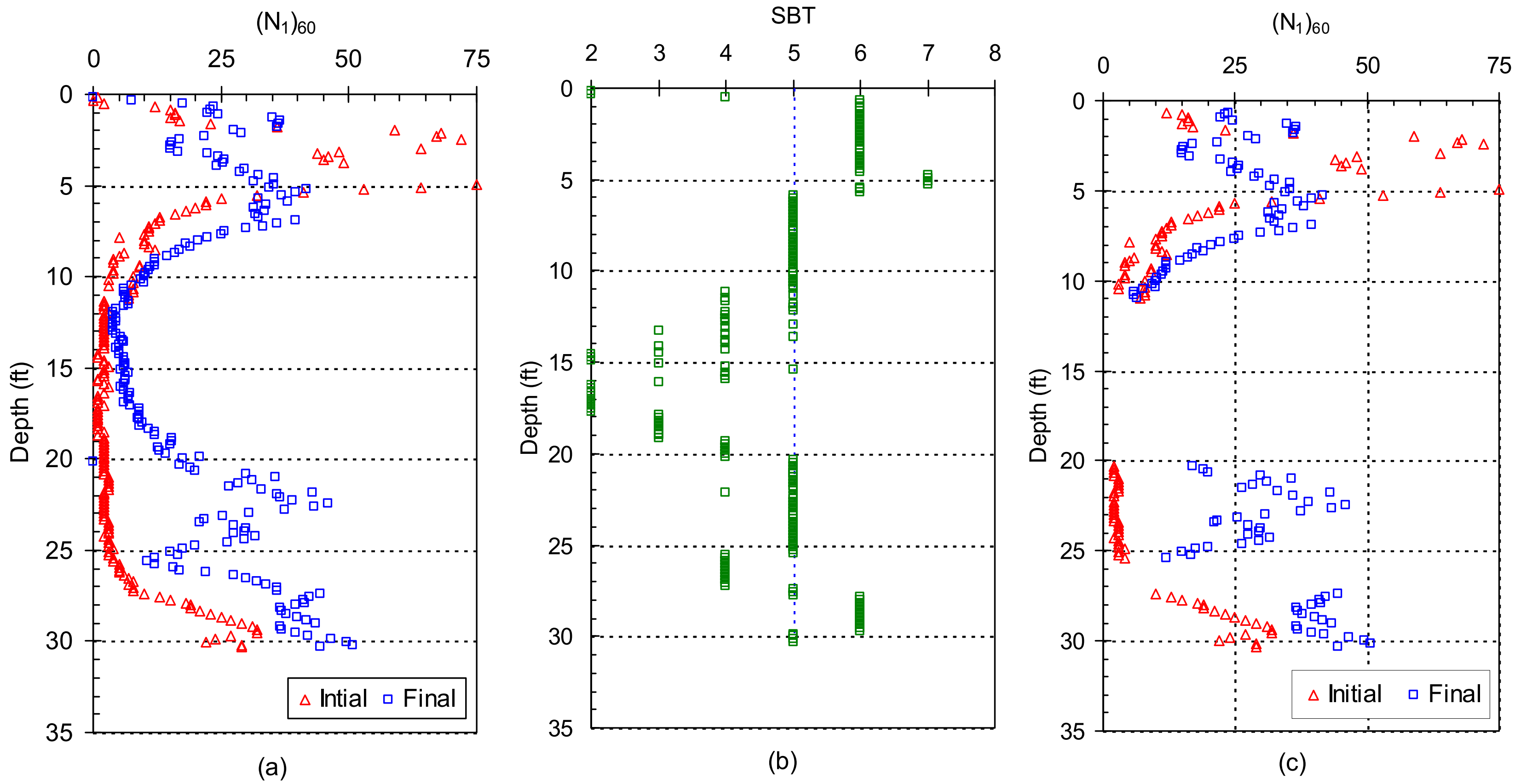


Figure 6-5 Direct comparison of CPT soundings 11 (initial) & 31 (final) plotted as (a) initial and final blow count versus depth, (b) SBT versus depth with a line for SBT = 5 (silty sands and sandy silts), and (c all included initial and final blow counts versus depth based on soil types SBT >= 5 for the San Pedro Home Depot case history.

Plot (a) in Figure 6-5 presents the general improvement of the final $(N_1)_{60}$ values in comparison to the initial $(N_1)_{60}$ values. Plot (b) presents the soil classification of the depth profile as indicated by the SBT values with a dashed line to indicate SBT = 5. Values above SBT = 5 were used in analysis. Plot (c) is the same as plot (a) except that soils with SBT values below 5 were excluded from the plot. The reduced data set for each direct comparison was then used for further analyses.

A scatter plot showing initial and final $(N_1)_{60}$ values versus depth for the 7.1% A_r data at the site is presented in plot (a) of Figure 6-6 and a plot of average $(N_1)_{60}$ values over 2 to 3 foot intervals is shown in plot (b) of the same figure. Only data points with SBT values greater than or equal to 5 are used and thin layers are eliminated for this plot as well as for all subsequent plots and analyses, except where noted.

The plot in Figure 6-6 (a) shows that the data is very scattered. Figure 6-6 (b) allows the reader to see the comparison between the initial and final $(N_1)_{60}$ values more clearly. The data shows that, in all but one interval, the final values were higher than the initial values. The only negative improvement had a high average initial value, greater than 50 blows per foot, and it is not uncommon for a very dense soil to loosen slightly due to soil disturbances that might occur during treatment. Nevertheless, the blow count in this layer is still very high and would not be expected to liquefy in an earthquake. The data shows significant improvement for most of the cross section with final values greater than 23 indicating that the soil improvements were producing a desirable result.

Direct comparison data were available for the San Pedro Home Depot case history. Plots similar to Figure 6-6 plot (c) are presented in Figure 6-7 and Figure 6-8 for all of the direct comparison data with 7.1% A_r that were used in this case history.

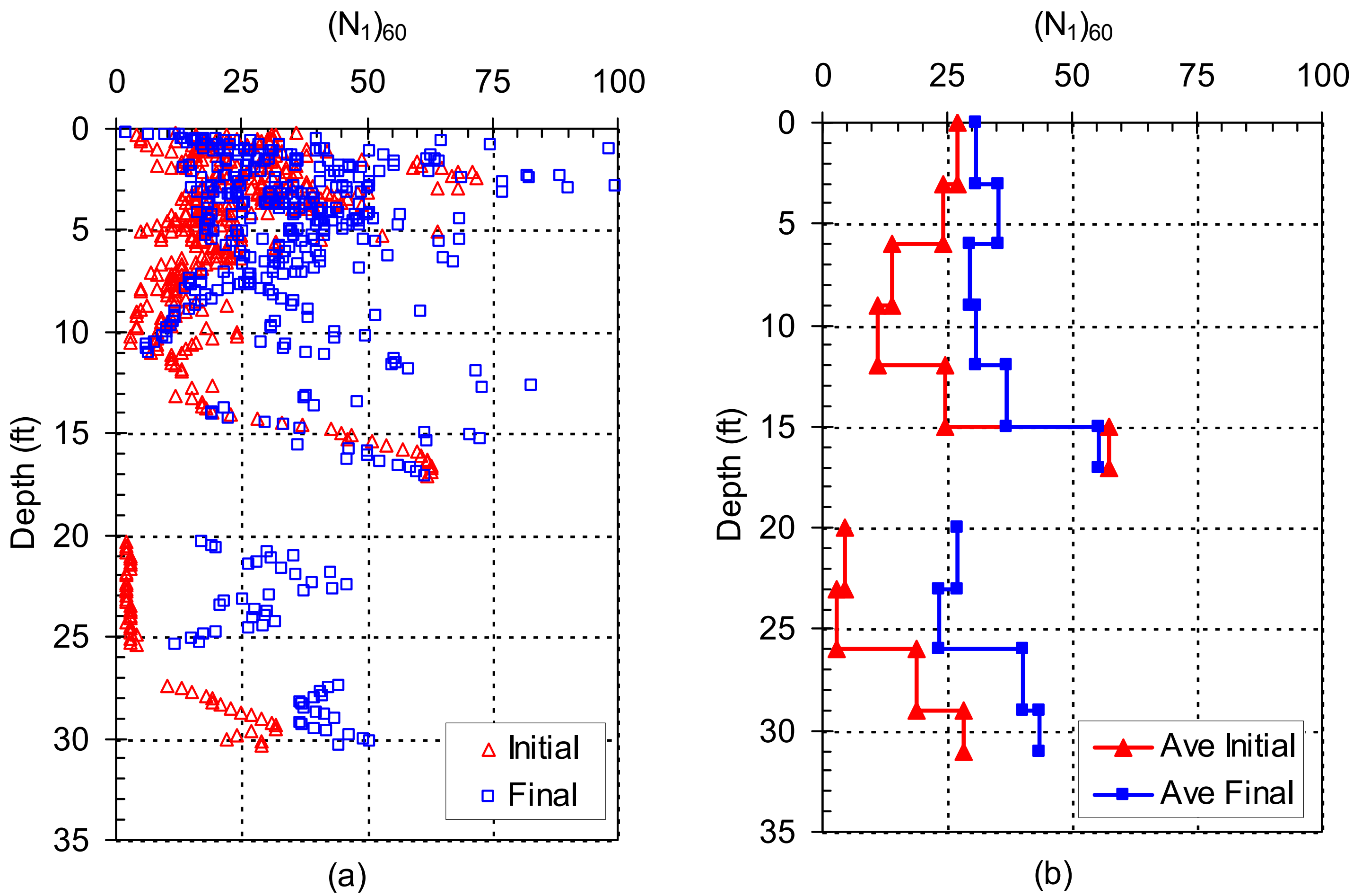


Figure 6-6 Initial and final $(N_1)_{60}$ versus depth for all of the 7.1% A_r data for the San Pedro case history.

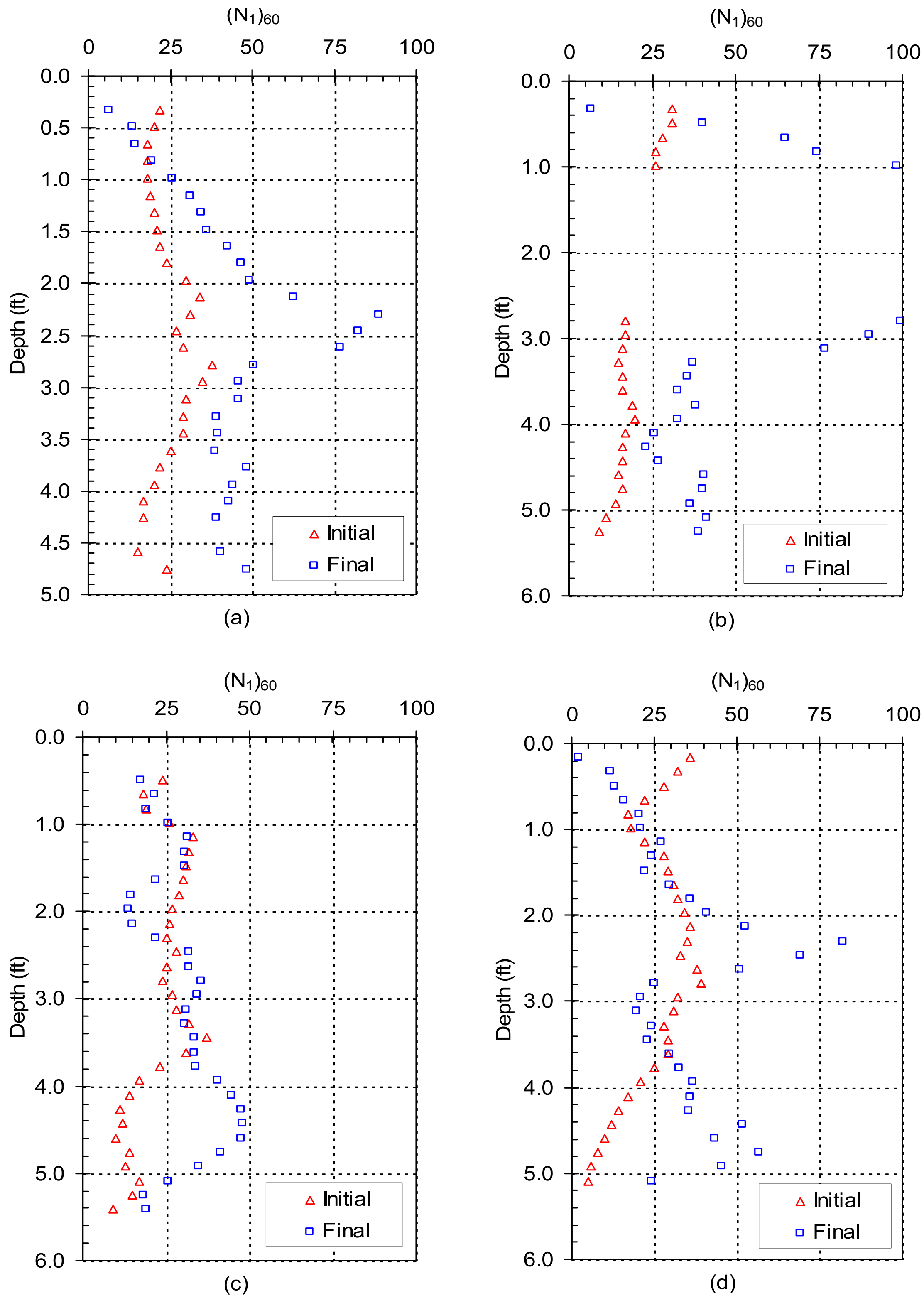


Figure 6-7 Initial and final $(N_1)_{60}$ versus depth for 7.1% A_r direct comparison pre-post soundings (a) 5 – 69, (b) 21 - 71, (c) 15 – 68, and (d) 18 – 49 for the San Pedro case history.

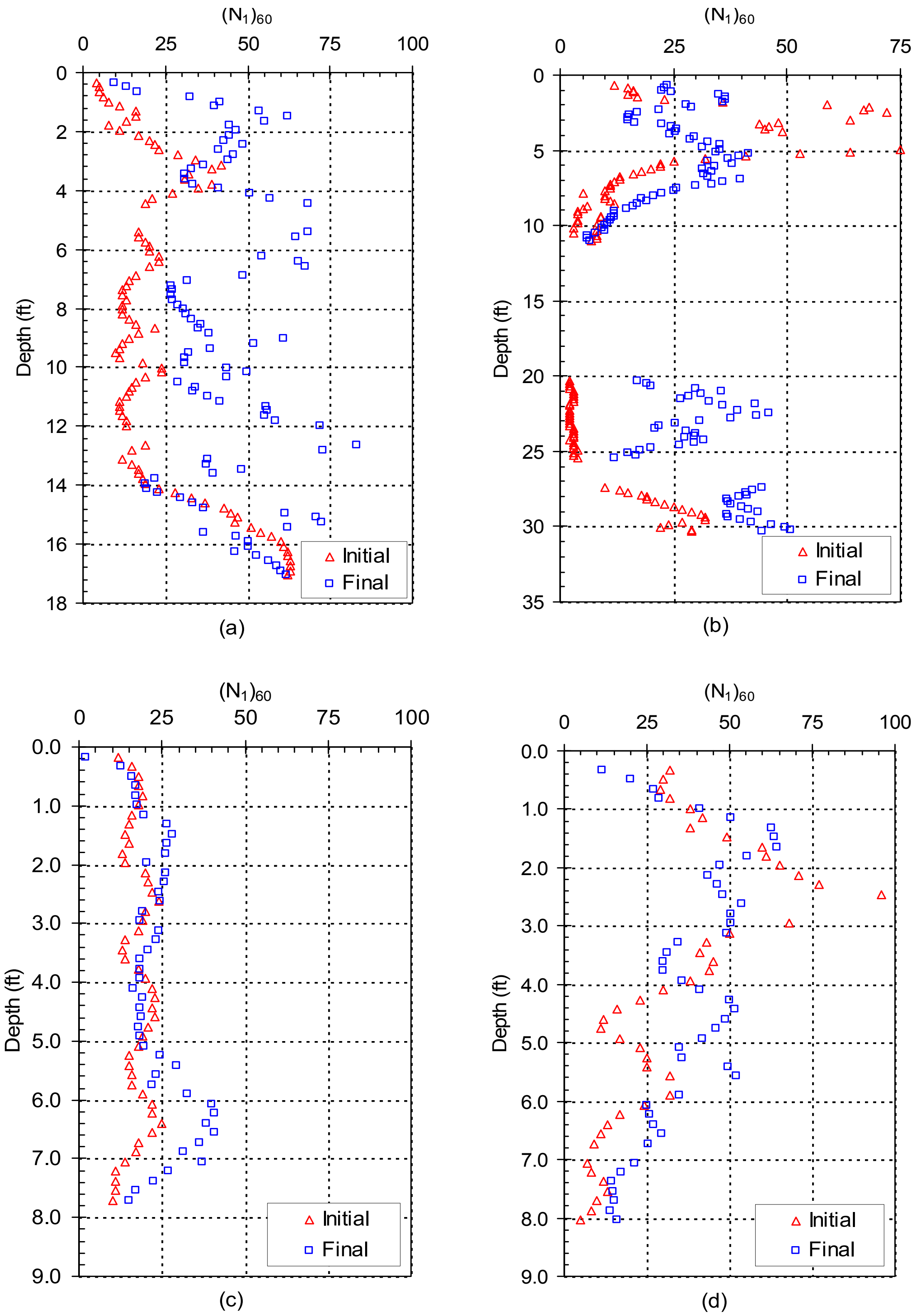


Figure 6-8 Initial and final $(N_1)_{60}$ versus depth for 7. 1% A_r direct comparison pre-post soundings (a) 13 – 30, (b) 11 - 31, (c) 19 - 45, and (d) 20 – 50 for the San Pedro case history.

The direct comparison plots in Figure 6-7 and Figure 6-8 indicate overall improvement from initial $(N_1)_{60}$ to final $(N_1)_{60}$ values. There are several places where the final values are less than the initial values; however, in most cases the negative improvement layers were less than 1-2 feet thick. Investigating these layers using available data, it was noticed that almost without exception the SBT values corresponding to the negative improvements were all equal to 6. An SBT value of 6 indicates clean sand to silty sand with generally low fines content. Each of the direct comparison plots above have a corresponding SBT vs. depth plot in Appendix A (Figure 13-2 and Figure 13-3). There were also multiple layers with SBT values of 6 throughout the remaining soil layers which did exhibit improvement. Many of the negative improvements were within the top few feet or even the top foot of the soil profile where the soil may have been disturbed due to soil improvement construction. The reason for the negative improvement in the SBT equal to 6 layers is not fully understood at this time.

To determine the factors affecting improvement, the change in blow count, $\Delta(N_1)_{60}$, was plotted versus initial $(N_1)_{60}$, fines content, and depth. $\Delta(N_1)_{60}$ is plotted versus initial $(N_1)_{60}$ in Figure 6-9 with a linear trend. The data and trend line show that there is a relationship between increasing initial blow count and decreasing change in blow count. This trend is expected since high initial blow counts are often difficult to improve; however, the R-squared value is only 0.37. This value indicates that approximately 37% of the variance in $\Delta(N_1)_{60}$ is accounted for by the initial $(N_1)_{60}$. The scatter in the data is significant. A linear relationship between the improvement (change in blow count) and final blow count would generally estimate improvement based off of initial blow counts, but significant variation in individual results would be expected. As a result, further regression with the intent to produce a direct equation is not performed for the data.

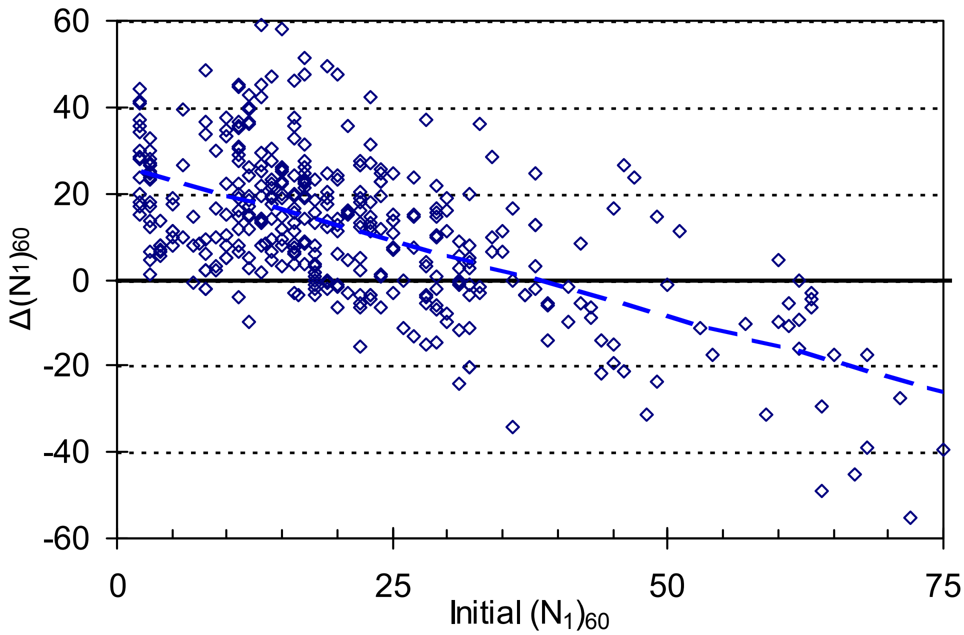


Figure 6-9 $\Delta(N_1)_{60}$ versus initial $(N_1)_{60}$ with linear regression curve shown for primary treatment areas (7.1% A_r) at the San Pedro Home Depot case history.

The plots showing $\Delta(N_1)_{60}$ versus fines content are located in Figure 6-10. The data indicates that increasing fines content is accompanied by an increase in $\Delta(N_1)_{60}$ values. This trend runs counter to what is normally observed in field project. A linear trend line and the accompanying R-squared value are also shown. There is significant scatter in the data and the R-squared value is only 0.28 which indicates a poor linear relationship between fines content and $\Delta(N_1)_{60}$. Apart from the natural variation which would be expected, the scatter may also be partly attributable to uncertainties in the correlations used to obtain the SPT $(N_1)_{60}$ values and the fines contents from the CPT data.

There is a large amount of negative improvements for data with fines contents less than 10%. If these negative values are excluded, the trend line becomes much flatter for the range of fines contents. Typically the improvement decreases as the fines content increases. To investigate further, the residuals from the initial blow count versus improvement linear trend line are plotted versus fines content in Figure 6-11 . Once the influence of initial blow count is accounted for, it can be seen that influence of fines content is negligible with an R-squared value of only 0.0313.

The data indicates that the initial blow count is impacting the improvement more than the fines content, although the reason why increasing fines content does not indicate decreasing improvement is not fully known. The most likely reason for this abnormal trend is that the data being used is correlated from CPT data with imperfect correlations. The data is first correlated to $(N_1)_{60}$ values and then further corrected using the correlated apparent fines contents. The average apparent fines contents can increase or decrease significantly following post-treatment testing at a site as noted by the project engineers; however, realistically the fines contents across the site are not affected by treatment and should not increase or decrease significantly following treatment. The CPT correlations do not properly account for this discrepancy which would cause the correlated clean sand blow counts to also be less accurate. The CPT correlated values are less accurate than the measured SPT values so the trend observed for this site is considered less reliable and although it is contrary to the typical trend, it is not considered to be of significant importance.

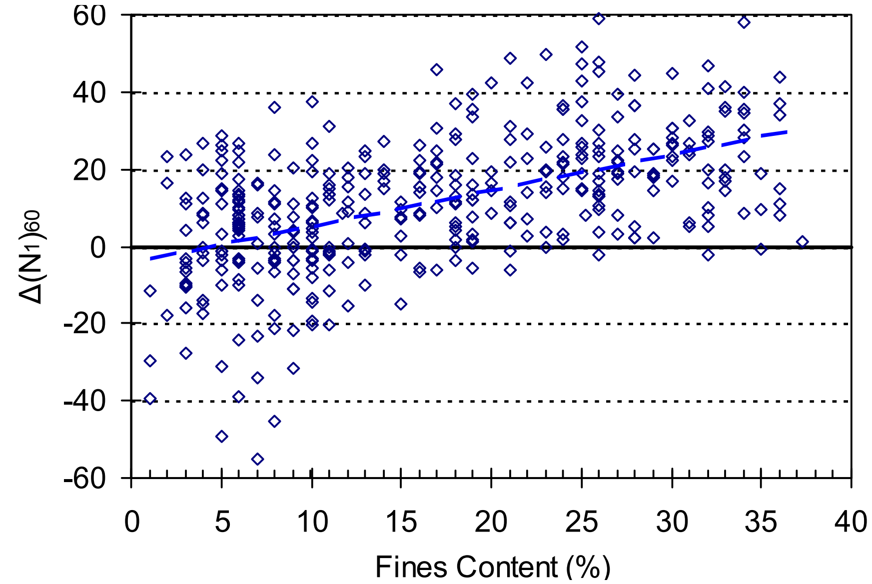


Figure 6-10 $\Delta(N_1)_{60}$ versus fines content for the primary treatment areas (7.1% A_r) at the San Pedro Home Depot case history. A linear regression line is also shown.

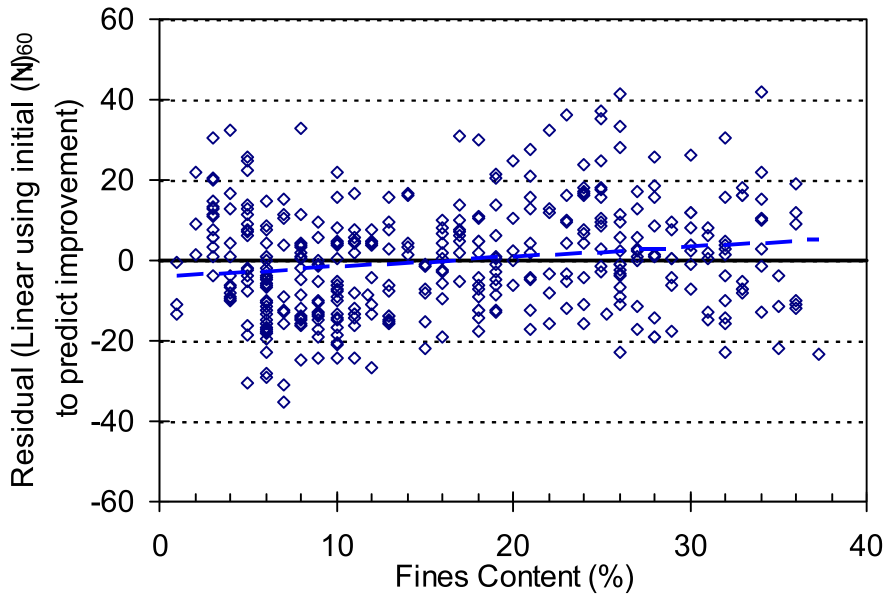


Figure 6-11 Fines content versus the residual values from linear regression on improvement using initial blow count as the predictor for the San Pedro Home Depot case history. A linear trend line is also shown for the data.

$\Delta(N_1)_{60}$ versus depth is plotted in Figure 6-12. The data show that most of the negative improvements were in the upper 5 feet of the soil profile. It is thought that soil treatment and surface conditions may possibly have loosened the soil near the surface. Soils further beneath the surface have a confining effect that may limit any soil disturbances other than the desired densification, however soil near the surface does not have this same confining effect and disturbances to the soil may cause heave or loosening of the soil. There are negative improvement zones beneath 5 feet, but they are minor compared with those above 5 feet. There are not any noticeable trends seen in the figure other than perhaps slightly less variation in $\Delta(N_1)_{60}$ as depth increases.

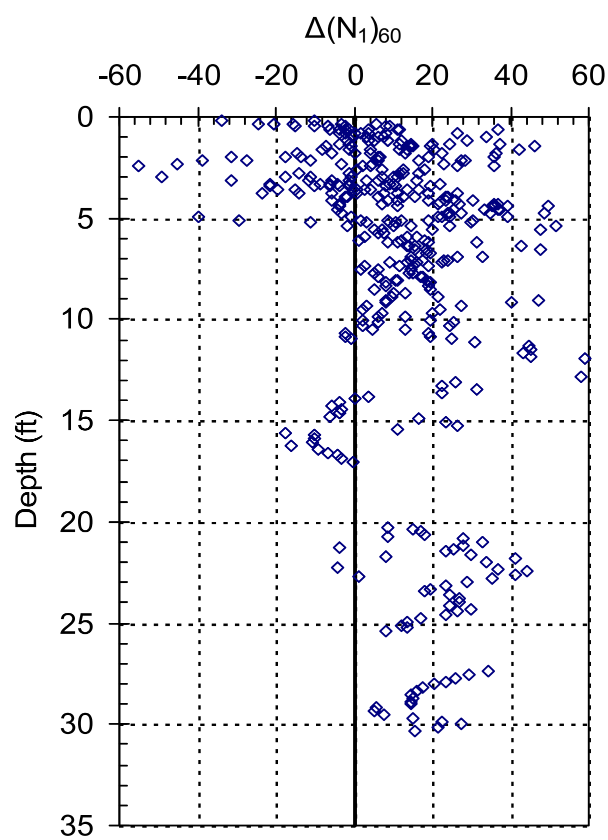


Figure 6-12 $\Delta(N_1)_{60}$ versus depth for the primary treatment areas (7.1% A_r) at the San Pedro Home Depot case history.

Final $(N_1)_{60}$ versus initial $(N_1)_{60}$ is plotted below in Figure 6-13. A 1 to 1 line is also shown with values above the line representing positive improvement and values below the line representing negative improvement. The data shows a significant degree of scatter and using linear regression is inappropriate to determine a direct linear relationship. The majority of the data falls above the 1 to 1 line which verifies that there was a general improvement in blow count as a result of treatment but not a consistent definable trend.

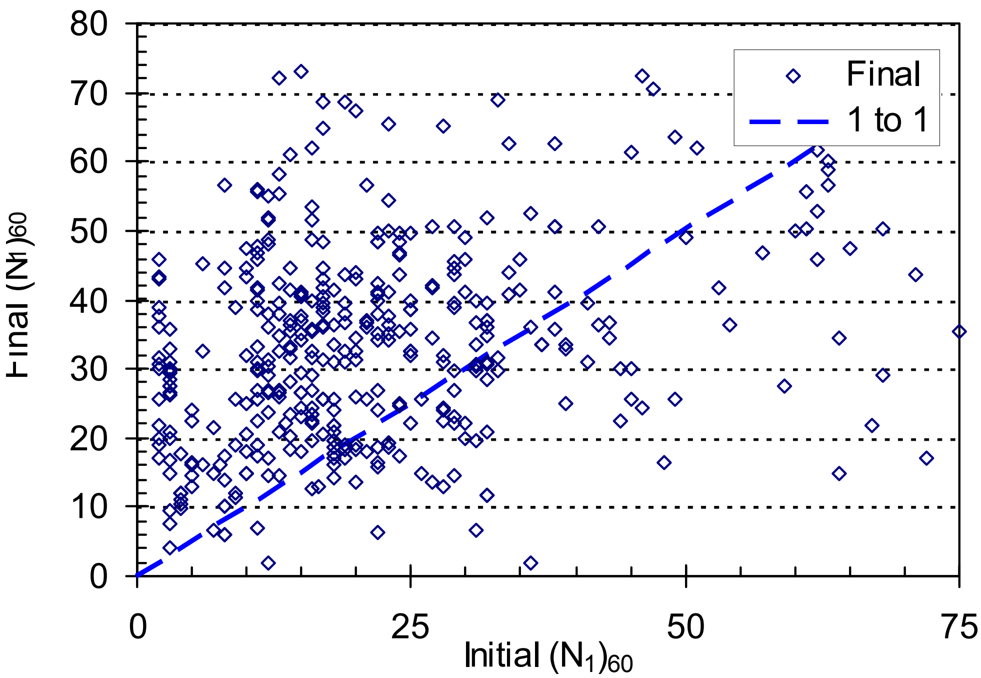


Figure 6-13 Final $(N_1)_{60}$ vs. initial $(N_1)_{60}$ with 1 to 1 line shown for the primary treatment area (7.1% A_r) data at the San Pedro Home Depot case history.

Average values for the 7.1% A_r direct comparison data at the San Pedro Home Depot case history are presented below in Table 6-1. To show the overall site improvement as well as to give the reader a feel for the overall site treatment, averages for all soil types together are also presented within the treatment depth.

The fines content for the silty sands and sandy silts was 15.5%. The average increase in the $(N_1)_{60}$ values was 11 blows per foot which represents an average increase of 52%. The average final blow count was high with a value of 33 blows per foot.

Table 6-1 Averages for the 7.1% Ar direct comparison data as well as the average data for all soil types at the San Pedro Home Depot case history.

Averages	SM/ML (7.1% A_r)	All Soil Types (All A_r's)
Initial $(N_1)_{60}$	22	11
Final $(N_1)_{60}$	33	18
$\Delta(N_1)_{60}$	11	7
Increase (%)	52	68
Time after treatment (days)	26	17
Apparent Fines Content (%)	15.5	33.2
A_r (%)	7.1	10.6
Standard Deviation (Finals)	14.4	15.4
Sample Size	391	2043

The overall site data presented in Table 6-1 show that the interbedded soil profile (All Soil Types category) yielded higher fines content, lower initial and final blow counts, and lower change in blow count than the SM/ML categories. The standard deviation of the final blow counts was much more significant though, approximately 86% of the average final blow count versus 44% for the SM/ML data. Lower initial blow count and higher fines content are to be expected when including clay layers in the averages. Lower final blow count is also expected based on lower initial blow count. The percent increase of $(N_1)_{60}$ was greater for the overall site than for the SM/ML category.

The data shows that the overall treatment with the site's varying area replacement ratios and soil profiles was still effective in increasing the final blow count. Hayward Baker determined that the improvement was adequate based on post-treatment settlement calculations. The maximum post-treatment settlement was to be less than 1 inch total settlement with less than ½ inch of differential settlement in 50 ft. These objectives were met overall across the site. Although the area replacement ratios varied across the site depending on liquefaction hazard, the treatment plan was effective in mitigating liquefaction for the entire site. By varying the treatment plan across the large site the cost of improvement was reduced instead of applying a single uniform treatment based on the worst soil conditions.

To investigate the relationship between time after treatment and final blow count, a scatter plot of final blow count versus initial blow count in days is presented in Figure 6-14. The data is divided into two separate series with the first showing the data for the CPT's tested within 13-19 days after treatment and the second showing the data for the CPT's tested within 28-40 days after treatment. Neither trend line represents the data particularly well, as the 13-19 day trend line had an R-squared value of 0.33 and the 28-40 day trend line had an R-squared value of 0.20. Nevertheless, the trend lines do show the general trend of the data in terms of gains with time after treatment. The data shows that there is a general increase in final blow count with time, especially for data with low initial blow counts. However, reliance on this average increase in penetration resistance with time must be tempered by the fact that significant variations in penetration resistance were observed.

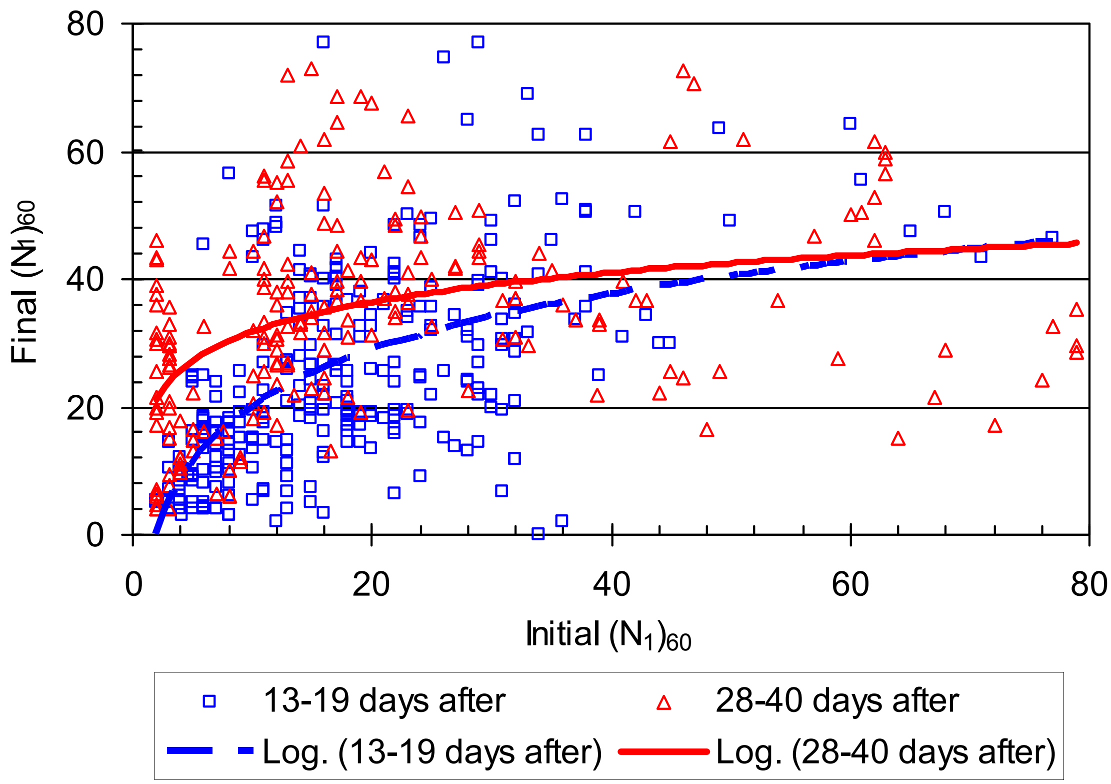


Figure 6-14 $\Delta(N_1)_{60}$ versus initial $(N_1)_{60}$ for the primary treatment areas (7.1% A_r) with series for 13-19 days after treatment data as well as the 28-40 days after treatment data with logarithmic regression curves at the San Pedro Home Depot case history.

To compare the results for this case history to the published literature, final $(N_1)_{60\text{-}cs}$ values are plotted versus initial $(N_1)_{60\text{-}cs}$ and a logarithmic trend line is presented in Figure 6-15. The logarithmic trend line is then compared to the curves developed by Baez (1995) for clean sands with fines contents less than 15% which are also presented in Figure 6-16. The trend line is not highly representative of the data due to scatter (R-squared value of 0.0824); however, by comparing it to the Baez clean sand curves the effectiveness of the San Pedro Home Depot site treatment plan may be compared generally.

The San Pedro Home Depot site with its 16% average fines content and 7% A_r with wick drains showed improvement which was most comparable to that of a clean sand site with less than 15% fines and a 10% A_r without wick drains. The average fines content is very close to the 15% cutoff specified for the Baez curves so it seems that the drains were effective in producing improvements greater than those expected by a clean sand site with a 7% A_r. This may suggest that even for sites with relatively low fines contents (≈15%) an alternative to increasing the area replacement ratio would be to supplement the stone columns with wick drains. It is suggested that additional testing would be required to verify this trend at other sites.

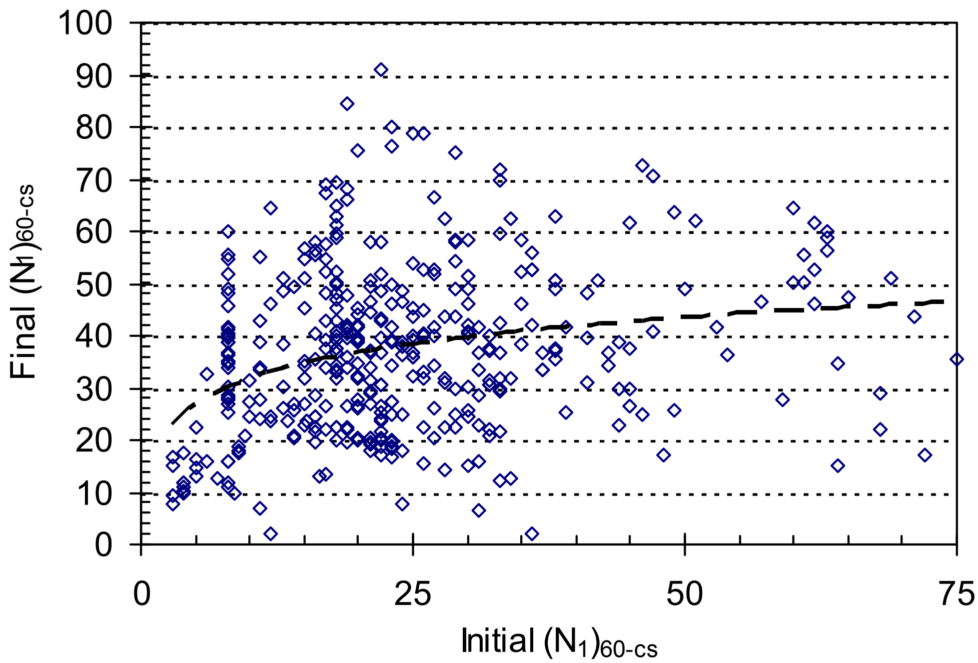


Figure 6-15 Final $(N_1)_{60\text{-}cs}$ vs. initial $(N_1)_{60\text{-}cs}$ with a logarithmic trend line presented for the primary treatment area (7.1% A_r) data at the San Pedro Home Depot case history.

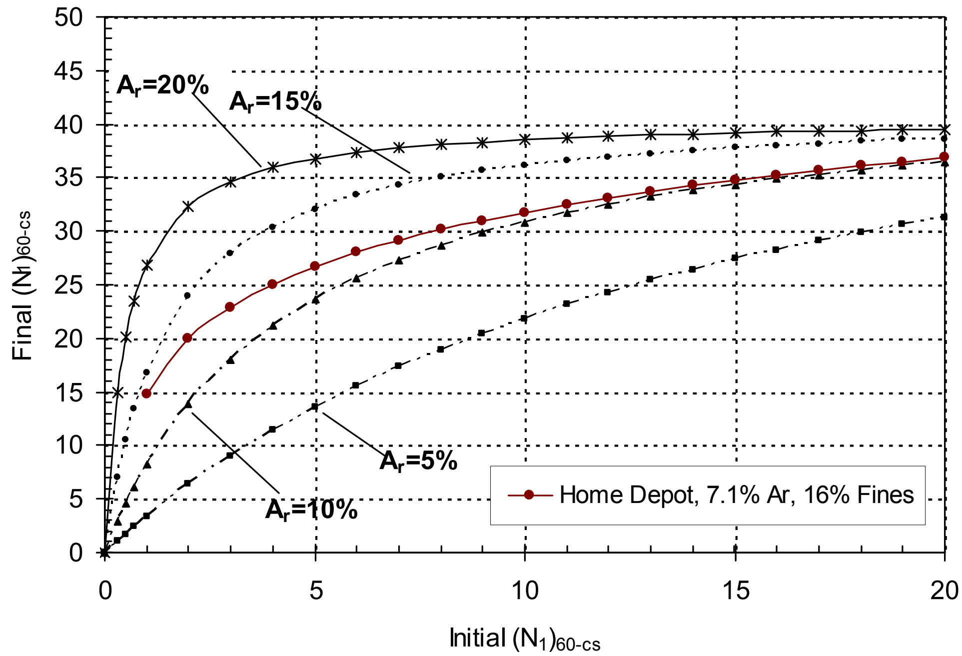


Figure 6-16 Initial $(N_1)_{60\text{-}cs}$ versus final $(N_1)_{60\text{-}cs}$ with the clean sand curves developed by Baez (1995) as well as the logarithmic trend line for the San Pedro Home Depot case history silt and sand data.

6.4 Conclusions

- The data for the San Pedro Home Depot case history were too scattered to perform reliable linear regression and the linear and logarithmic regression that was attempted yielded poor R-squared correlation values. The standard deviations of the final blow count were 43-86% of the average final values depending on the treatment area. Linear regression curves represent average values for the site.
- Fines content and initial $(N_1)_{60}$ appear to have some effect on improvement. High fines contents and low initial $(N_1)_{60}$ values tend to increase improvement. The fines content trend is opposite of the typical trend, this was likely due to the use of correlations between CPT and SPT

data, in particular the correlated apparent fines content correlation when applied to post-treatment data.

- $\Delta(N_1)_{60}$ values varied and were often negative within the upper 5 feet of the soil profile. One possible explanation is that the upper soil does not have the confining effect of soils deep beneath the surface and are thus more susceptible to surface disturbances resulting from treatment.
- On average, for the silty sands and sandy silts in the primary improvement areas (7.1% A_r), there was 52% improvement with final $(N_1)_{60}$ values of 33 blows per foot for soils with 15.5% fines content and 22 blows per foot initial blow count. The standard deviation of the final blow counts was 14.
- The site performed similar to the 10% A_r clean sand curve presented by Baez, suggesting that wick drains are a possible alternative to increase A_r even in relatively low fines contents (≈15%). This observation is based on general trends and potentially significant scatter is to be expected.
- The site soil profile was fairly interbedded with varying soil types. The silty sand and sandy silt layers exhibited high final blow counts of 33 despite the interbedded nature of the soil profile. The overall site (including all soil types) also improved but the final blow count average was lower at 18 blows per foot. The lower blow counts are largely associated with clays, for which liquefaction was not a concern, so overall the site was effectively improved.

- A treatment plan of varying stone column layout was effective in meeting the desired objectives for the site without having to increase the area replacement ratios uniformly across the site.

7 Silver Reef Casino Expansion, Silver Reef, Washington

7.1 Site Overview and Soil Conditions

The Silver Reef Casino is located in the Ferndale area of Whatcom County, Washington. The case history presented here concerns the casino expansion that began in 2005. The expansion involved the addition of a hotel as well as expanding the existing casino to connect to the hotel. The hotel was supported by a mat foundation over improved soils. An aerial view of the casino with the hotel expansion is shown in Figure 7-1 below.

Figure 7-1 Aerial view of the finished Silver Reef Casino Hotel expansion (source, Hayward Baker, Inc.).

Lachel Felice & Associates, Inc. performed the final geotechnical analyses and prepared the geotechnical observation and testing for the site. GeoTech Systems Corp., Hayward Baker, Inc., and GeoEngineers, Inc. were all involved in the geotechnical analyses and/or improvement procedures. Hayward Baker designed and implemented the ground improvements for the expansion.

The design earthquake for the site is a magnitude 7.0 earthquake with a peak ground acceleration of 0.3g. The preliminary geotechnical investigation classified the soil as a three foot layer of medium dense fill overlaying about 40 feet of interbedded loose to very loose silty sand (SM) and medium stiff to soft sandy silt (ML) layers. Underlying the silt and sand layers was approximately 70 feet of interbedded soft silt (ML) and soft to very soft clay (CL). The sand and the clay layers were distinctive. The upper 30 to 40 feet of the soil profile were determined to need soil improvements in order to prevent liquefaction and lateral spread. There was also concern about the settlement of the deep clay layer.

Figure 7-2 presents a typical cross section of the soil profile as well as a plot of apparent fines content versus depth for the site. Fines contents for the site generally ranged between 20 – 50% for the SM/ML layer (personal communication with Mark Koelling of Hayward Baker). The average fines content based on apparent fines contents was approximately 20%. The fines content fluctuates with depth. Although the measured fines contents do not appear consistent with the apparent fines contents, there are not enough measured fines contents to draw any conclusions.

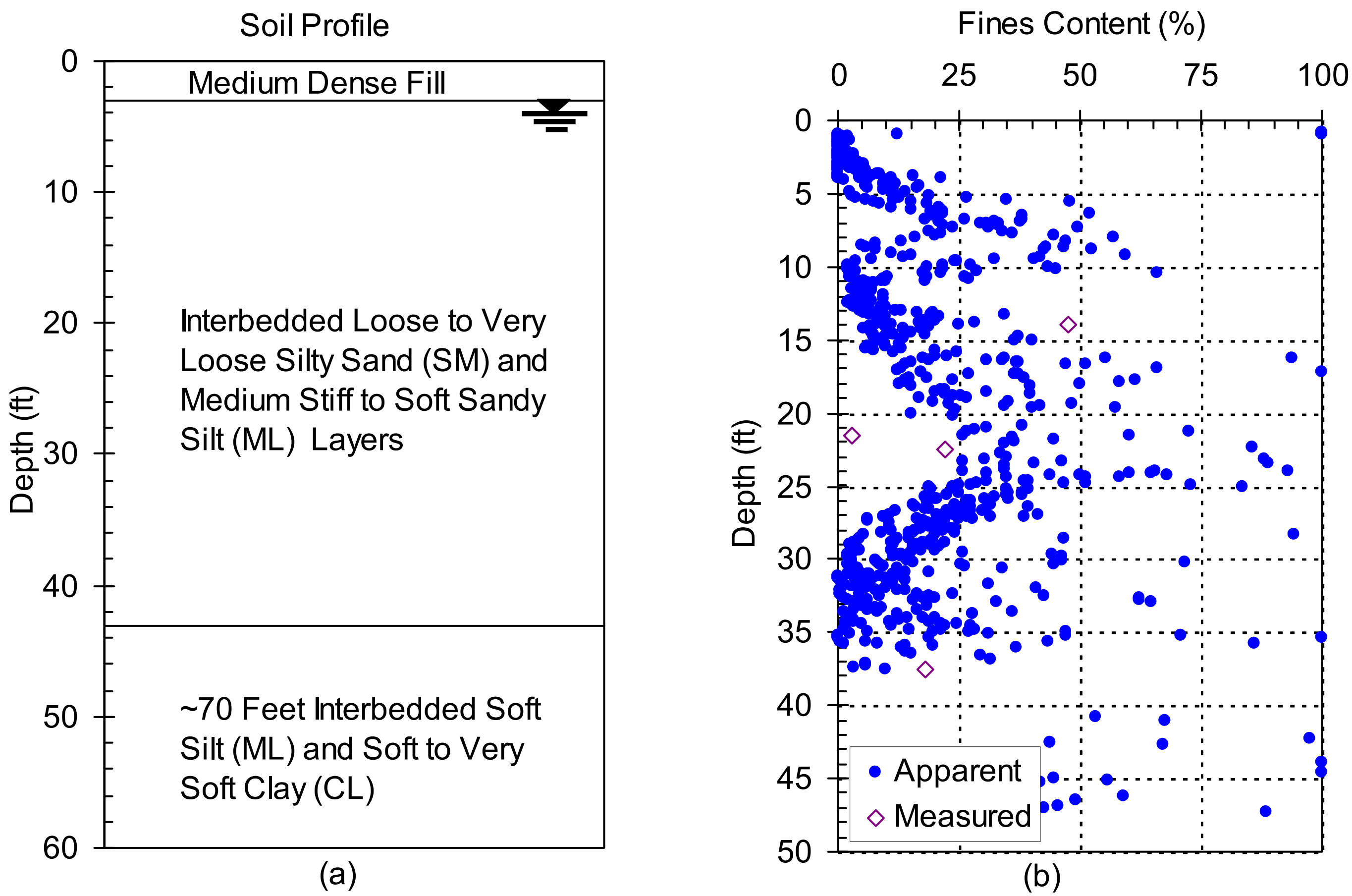


Figure 7-2 Idealized soil profile (a) and apparent and measured fines content versus depth (b) for the Silver Reef Casino case history.

7.2 Treatment Method

Hayward Baker performed the soil improvement at the site which took place during 2005. It was determined that the soil improvement plan should consist of installing stone columns with wick drains pre-installed in-between columns.

The stone columns extended to a depth of 35 to 45 feet below the surface and their primary purpose was to mitigate liquefaction in the upper layer of interbedded silty sand and sandy silt. The average stone column diameter was 3.5 feet. The stone columns were arranged in an 8 ft. by 8 ft. square pattern. The stone column layout resulted in an area replacement ratio of 15%. The spacing of the stone columns was based on the need to densify the loose soil in the upper layer. A S23 series vibrator with 120 kW of power was used to install the stone columns using the dry bottom feed method.

The wick drains extended to a depth of 100 feet below the surface. The primary purpose of the drains was to facilitate consolidation of the deep clay layer. Settlement in the clay layer was pre-induced using a pre-load surcharge in addition to the wick drain system. A secondary purpose of the drains was to facilitate liquefaction mitigation efforts in the upper soil profile by allowing excess pore water pressure to drain from the surrounding soil. The drains were installed at the midpoints between adjacent columns as well as in the center of every stone column grid as shown in Figure 7-3. Based on this layout, there were approximately 3 wick drains per stone column.

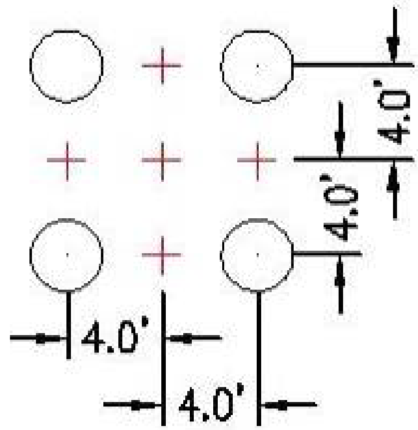


Figure 7-3 Typical plan detail for the stone column and wick drain treatment at the Silver Reef Casino case history.

7.3 Results and Analysis

CPT's were used by Hayward Baker to evaluate the effectiveness of the mitigation procedures. The post-treatment CPT's were done mostly within 2 weeks of the date of treatment. For the Silver Reef site, CPT-SPT correlations were applied by both GeoEngineers and Hayward Baker during the preparation for their analyses based on data from UBC-1983. Apparent fines content calculations were carried out by the author using the Robertson and Wride (1998) correlation.

A layout of the CPT sounding locations was not available for this case history. Without information regarding the proximity of the pre- and post-treatment CPT locations it is not possible to directly compare initial and final CPT data. Therefore, the data for the site is analyzed based on a general scatter plot of depth versus initial and final correlated $(N_1)_{60}$ as well as through the use of a table of averages.

A plot showing initial and final $(N_1)_{60}$ values versus depth for the entire site is shown below in Figure 7-4. For clarity, the scatter plot (a) and the averages plot (b) are shown separately. The averages plot in (b) show that the final average $(N_1)_{60}$ values were greater than the initial average $(N_1)_{60}$ values at all depths below 6 feet. The upper 6 feet had negative improvement. One possible explanation is that the upper 3 feet were medium dense fill which would not improve as much as loose sand and might loosen due to the mitigation efforts. Average improvements below 6 feet ranged from 3 to 33 blows per foot. In general the data shows that there was improvement but it does not show any clear trends.

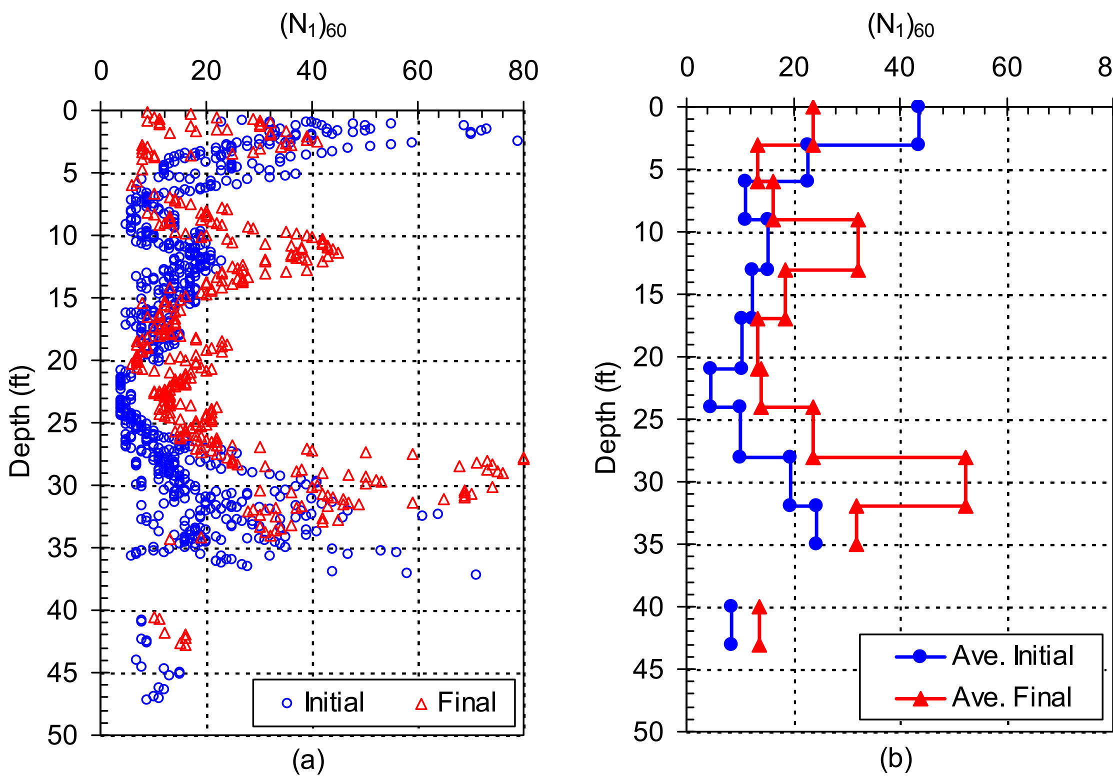


Figure 7-4 Pre- and post-treatment correlated $(N_1)_{60}$ values versus depth with (a) individual points and (b) averages for the Silver Reef Casino case history.

Figure 7-5 presents (a) the average apparent fines contents and (b) the average final $(N_1)_{60}$ values versus depth. The data shows that in all but two cases an increase in the average apparent fines contents leads to a decrease in the average final $(N_1)_{60}$ values and vice versa. This indicates a trend where the final $(N_1)_{60}$ values decreases as the fines content increases. Figure 7-6 presents a comparison of the average initial $(N_1)_{60}$ values with the average apparent fines contents. The figure also includes a logarithmic trend line. The R-squared value for the trend line was 0.7016, indicating a strong relationship. It is thought that fines content affects the initial blow count first which in turn affects the final blow count, thus the most significant independent factor affecting the final blow count is fines content.

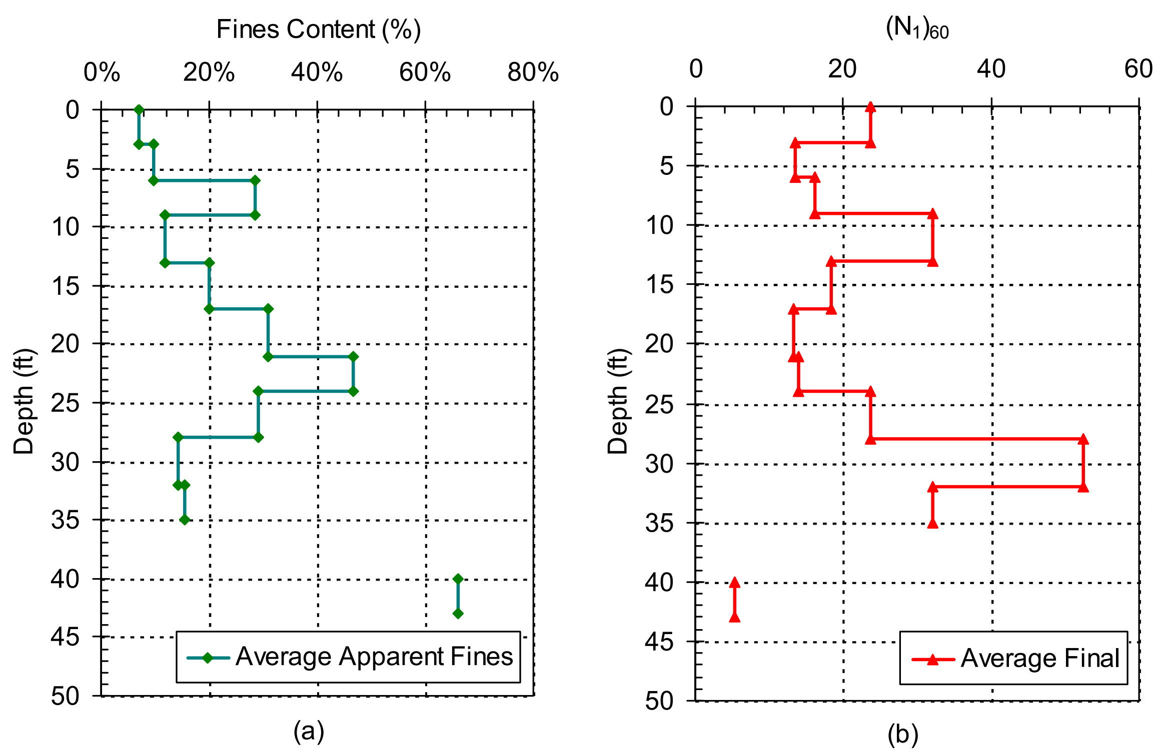


Figure 7-5 Average apparent fines contents (a) and average final $(N_1)_{60}$ (b) versus depth for the Silver Reef Casino case history.

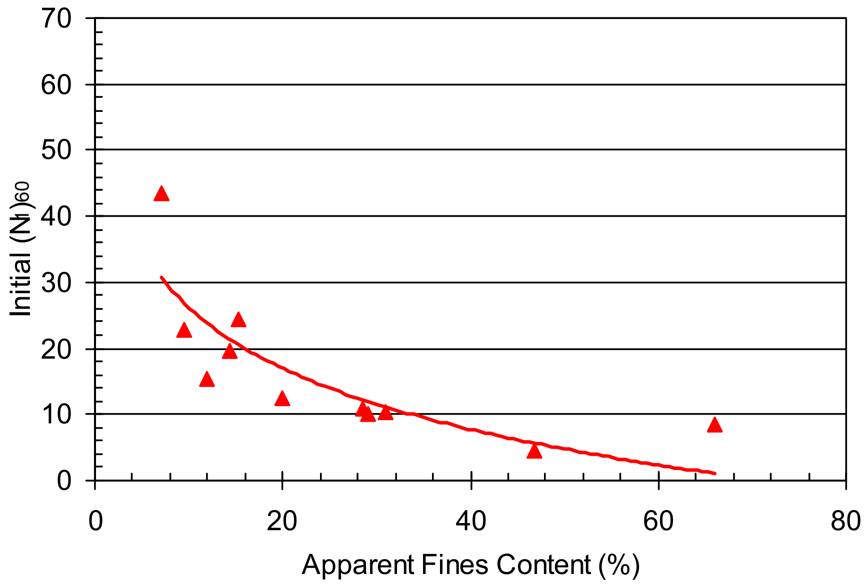


Figure 7-6 Comparison of initial $(N_1)_{60}$ versus average apparent fines content with a logarithmic trend line for the Silver Reef Casino case history.

Since there were no companion soundings, the improvement cannot be directly compared as in other case histories. Final $(N_1)_{60}$ versus fines content is presented in Figure 7-7. The logarithmic trend line indicates a general relationship between increasing fines content and the resulting decrease in final blow count. The R-squared value for this relationship was only 0.1877 which indicates a poor direct relationship between the two variables. Although the direct relationship is poor, it is sufficient to indicate that high fines contents will detrimentally affect the final blow count following stone column and wick drain improvements. A logarithmic regression line fitted to a plot of improvement versus apparent fines content yields an even lower R-squared value of 0.0816; therefore, further analyses with the improvement were not considered. . As seen in Figure 7-6 and as discussed previously, the fines content affects the initial blow count which in turn affects the final blow count so both variables must be considered in order to understand the relationship. A full analysis of this relationship is not possible due to the lack of direct comparison data.

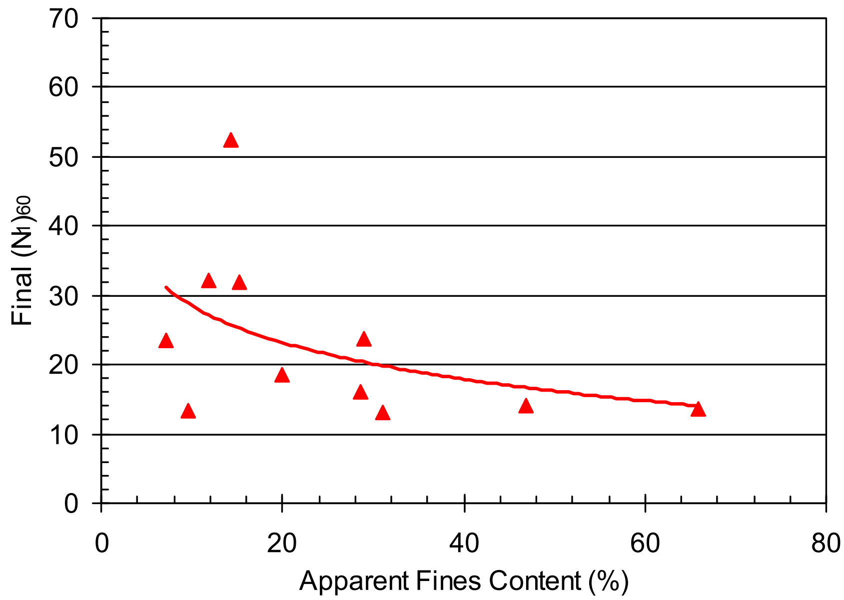


Figure 7-7 Comparison of final $(N_1)_{60}$ versus fines content with a logarithmic trend line for the Silver Reef Casino case history.

Final $(N_1)_{60}$ versus initial $(N_1)_{60}$ is presented in Figure 7-8. The logarithmic trend line indicates a general relationship between increasing fines content and the resulting decrease in final blow count. The R-squared value for this relationship was only 0.1818 which indicates a poor direct relationship between the two variables. The clean sand equivalent of this figure will be considered later in this analysis.

Table 7-1 presents the average values for the entire site. Overall there was a 36% increase in the average $(N_1)_{60}$ value with an average final $(N_1)_{60}$ value of 25. The standard deviation of the final blow counts was 17, or 66% of the average final blow count. As noted previously, high initial $(N_1)_{60}$ values usually indicate that the change in $(N_1)_{60}$ values will not be very large. The average initial blow count was relatively high at 18 but there was still an increase of 7 blows per foot to yield the final value of 25 blows per foot following mitigation.

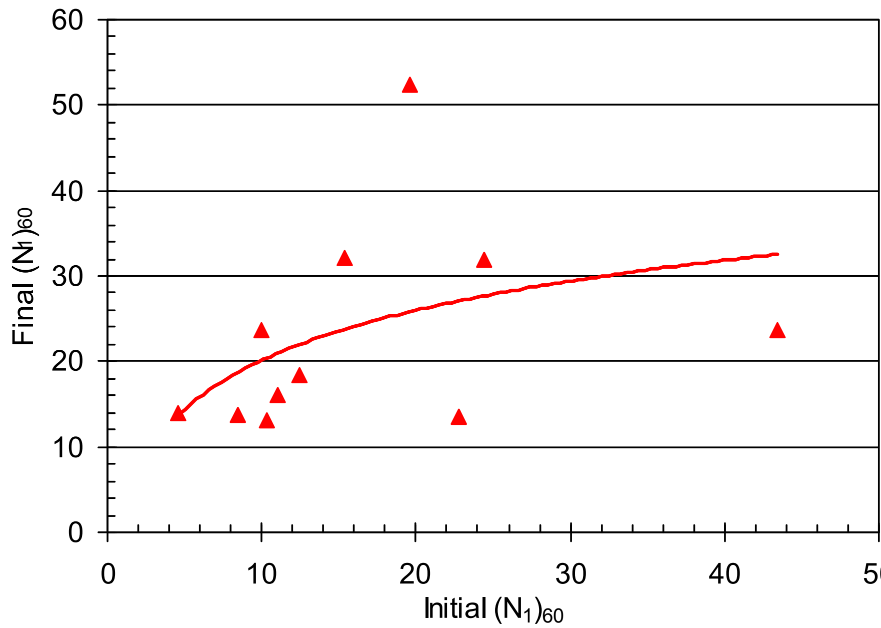


Figure 7-8 Comparison of final $(N_1)_{60}$ versus initial $(N_1)_{60}$ with a logarithmic trend line for the Silver Reef Casino case history.

There were about 1000 points available for analysis although two thirds of them were all initial data points. The large number of data points used reassures that the averages shown are likely reflective of the actual site conditions.

Table 7-1 Average values for the Silver Reef case history.

Depth (ft)	Sample Size	Ave. Fines (%)	Standard Deviation (Finals)	Average $(N_1)_{60}$			
				Initial	Final	Change	% Increase
0 - 45	1003	20.2	17	18	25	7	36

The majority of the post-improvement CPT soundings were taken within 2 weeks of installing the wick drains and stone columns. This time frame would have allowed for the initial improvement to occur, but testing at a later date would likely have yielded greater gains with time.

To compare the results for this case history to the published literature, final $(N_1)_{60\text{-}cs}$ values are plotted versus initial $(N_1)_{60\text{-}cs}$ and a logarithmic trend line is presented in Figure 7-9. It should be noted that the post-treatment CPT soundings were not directly comparable to pre-treatment CPT soundings so the fines content correction used to obtain the clean sand post-treatment blow counts was based on post-treatment apparent fines contents. Typically it is not appropriate to use the post-treatment apparent fines content values since treatment of the soil artificially changes the apparent fines contents in many cases. This was the case for the Silver Reef site as well with the average apparent fines content pre-treatment being 20.2% while the post-treatment average was only 14.3%. To adjust for this, the post-treatment apparent fines contents were individually increased by 6 which raised the average post-treatment apparent fines content to 20.3%.

The logarithmic trend line from Figure 7-9 was compared to the curves developed by Baez (1995) for clean sands with fines contents less than 15% which are presented in Figure 7-10. The trend line is not highly representative of the data due to scatter (R-squared value of 0.0752) and because it is based on averages instead of actual individual data; however, by comparing the trend line to the Baez clean sand curves, the effectiveness of the Silver Reef Casino site treatment plan may be compared generally. The logarithmic trend line for the Silver Reef Casino site was developed from data with initial clean sand blow counts greater than 10 blows so the trend line that is compared to the Baez curve likewise begins at 10 blows.

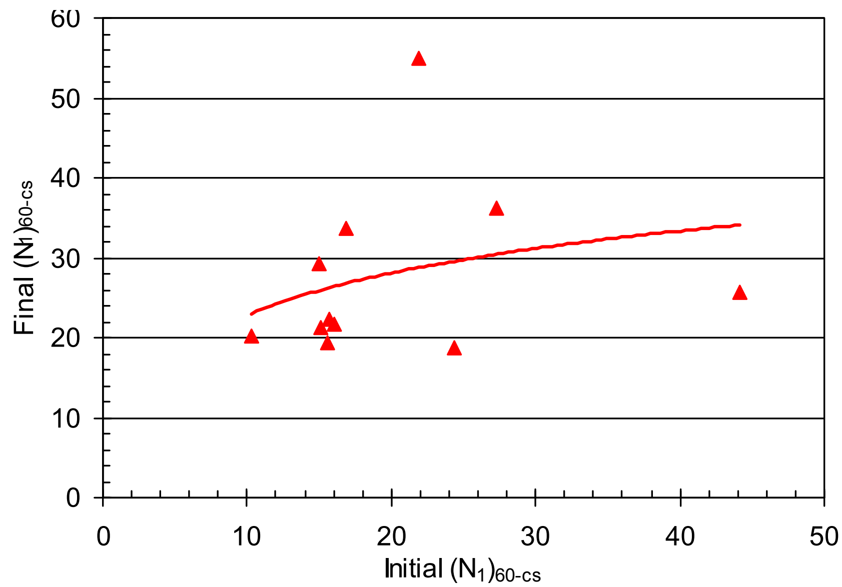


Figure 7-9 Comparison of final $(N_1)_{60\text{-}cs}$ versus initial $(N_1)_{60\text{-}cs}$ with a logarithmic trend line for the Silver Reef Casino case history.

Correlations developed by Baez (1995) for sands with fines contents <15% predict a final $(N_1)_{60}$ value of 38 based on an initial $(N_1)_{60}$ value of 18. Therefore, the increase in blow count is only 35% of what would have been expected for cleaner sands. The trend line for the Silver Reef Casino case history shows slightly less improvement than the 5% A_r clean sand curve. Nevertheless, the area replacement ratio of 15% was effective in producing some increase in the liquefaction resistance for the SM/ML layer which had an average fines content of approximately 20%.

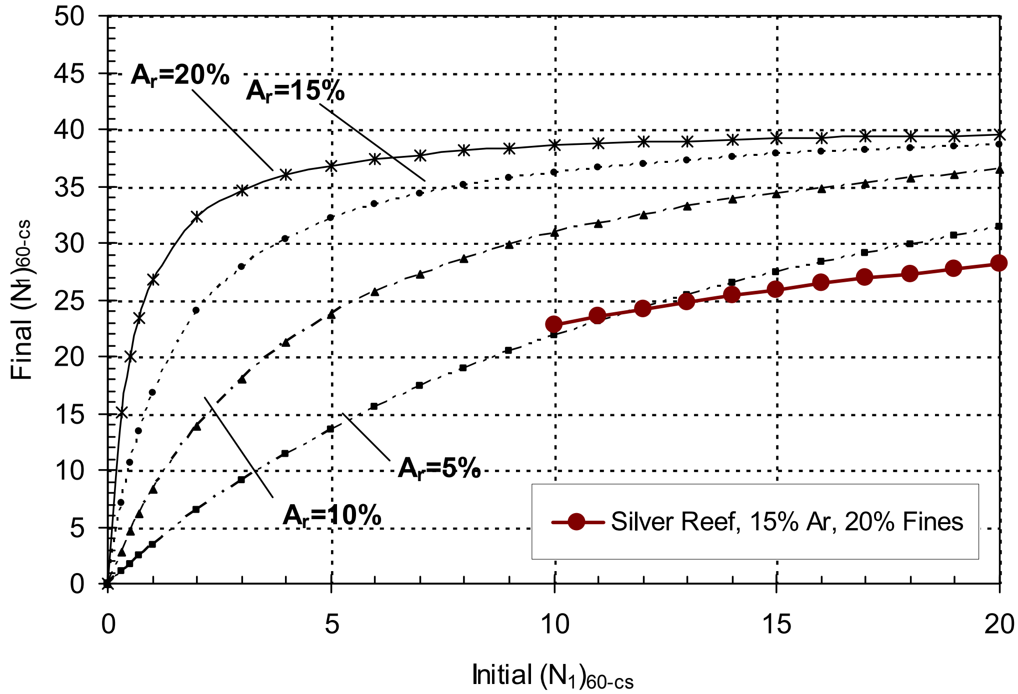


Figure 7-10 Initial $(N_1)_{60\text{-cs}}$ versus final $(N_1)_{60\text{-cs}}$ with the clean sand curves developed by Baez (1995) as well as the logarithmic trend line for the Silver Reef case history silt and sand data.

7.4 Summary and Conclusions

- In the absence of direct comparison data average data were used to analyze the Silver Reef Casino case history and observe general trends.
- Fines content and initial $(N_1)_{60}$ appear to have a combined effect on improvement. High fines contents and initial $(N_1)_{60}$ values tend to limit improvement significantly. Fines content is thought to be the main contributing factor since it affects initial $(N_1)_{60}$ as well as final $(N_1)_{60}$.
- Across the site there was an average of 36% improvement with an average final $(N_1)_{60}$ value of 25 for soils with an average initial $(N_1)_{60}$ value of 18. The standard deviation of the final blow counts was 17, or 66% of the average final blow count. This improvement is only about 35% of the

improvement which would be expected for similar treatment parameters with cleaner sands (<15% fines).

- The 15% area of replacement ratio was sufficient to produce moderate increases in liquefaction resistance of soils with fines contents of 20%.

8 Shepard Lane Bridge Abutment, Farmington, Utah

8.1 Site Overview and Conditions

The Shepard Lane Bridge is an overpass carrying US 89 over Shepard Lane in Farmington, Utah. Previously there had not been a bridge at the site. The bridge abutments and bent were placed over improved soil to prevent liquefaction. Figure 8-1 shows an aerial view of the Shepard Lane Bridge site as well as its close proximity (within 1 mile) of another bridge site considered in this study, the Cherry Hill Bridge site. The Shepard Lane Bridge case history used both stone columns and wick drains while the Cherry Hill Bridge case history only used stone columns.

Figure 8-1 Aerial view of the Shepard Lane overpass which shows its proximity to the Cherry Hill interchange (within 1 mile). The Cherry Hill Bridge case history will be presented in Chapter 9.

RB&G Engineering, Inc. prepared the report of geotechnical observation and testing for the site. The design earthquake for the site is a magnitude 7.4 earthquake with a peak ground acceleration of 0.6g. The peak ground acceleration is based on a 2% probability of being exceeded in 50 years (~2500 yr recurrence interval). The site is located within about a mile of the Wasatch fault.

Prior to treatment, nearly continuous SPT sampling was performed at the test site to determine liquefaction potential and soil characteristics at the site. Near continuous testing also allowed for measurements of fines content throughout the soil profile. The soil profiles were fairly consistent across the site. The layering varied by a few feet in places, but the soil profile was generally continuous between test holes. A generalized soil profile of the site is presented in Figure 8-2.

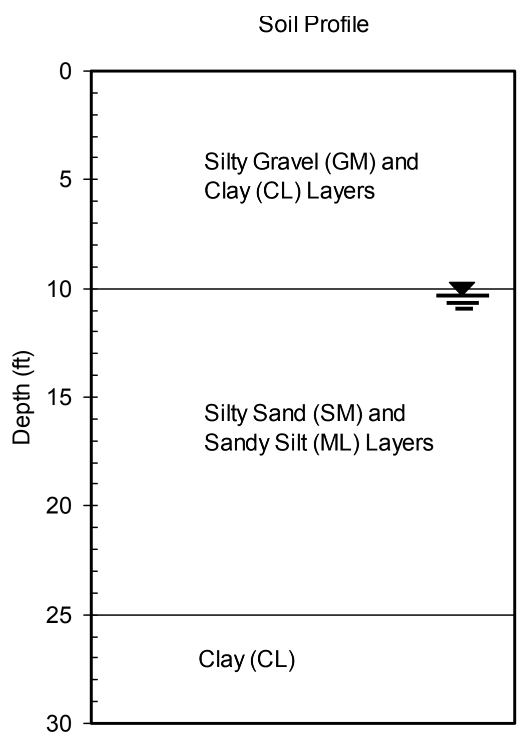


Figure 8-2 Generalized soil profile for the Shepard Lane Bridge case history.

The upper 10 feet were typically silty gravel (GM) and clay (CL) layers. From 10 ft to about 20-25 ft the soil profile consisted of silty sand (SM) and sand silt (ML) layers. The most common lower depth of this layer was 20 ft although there were a few locations where it extended to 25 ft beneath the surface. Likewise, the silt and sand layer typically began at a depth of about 10 ft, but there were some cases where the layer began at depths as little as 5 ft beneath the surface. This layer was underlain by a large clay layer.

The main layer of concern was the silty sand and sandy silt layer from 10-25 ft beneath the surface. For this layer, only the silt and sand data were used in analyses in order to focus on the goals of this study. There were some occasional lenses of clay or silts and sands with high clay contents, but otherwise the silts and sands comprised the majority of the layer. Average values referenced for this layer were obtained from the silt and sand data, unless otherwise noted.

Fines content (a) and clay content (b) profiles are presented for the silt and sand data in Figure 8-3. Average lines for 4 ft intervals are shown for both the fines content and the clay content profiles. The fines contents ranged from 31-88% with a mean value of 46% for the silt and sand data. The plasticity index ranged from non-plastic to about 4% plasticity with an average clay content of 8%. The average data indicate some variation in fines and clay contents with depth, although the data is scattered. The average fines content from 10-14 ft was 37% which was less than both the 47% average in the layer above and the 58% average in the layer below. The overlying and underlying layers both had higher concentrations of clays and soils with high clay contents (>15%). The fines contents in the upper and lower layers were also typically very high, with higher fines contents than the silts and sands layer.

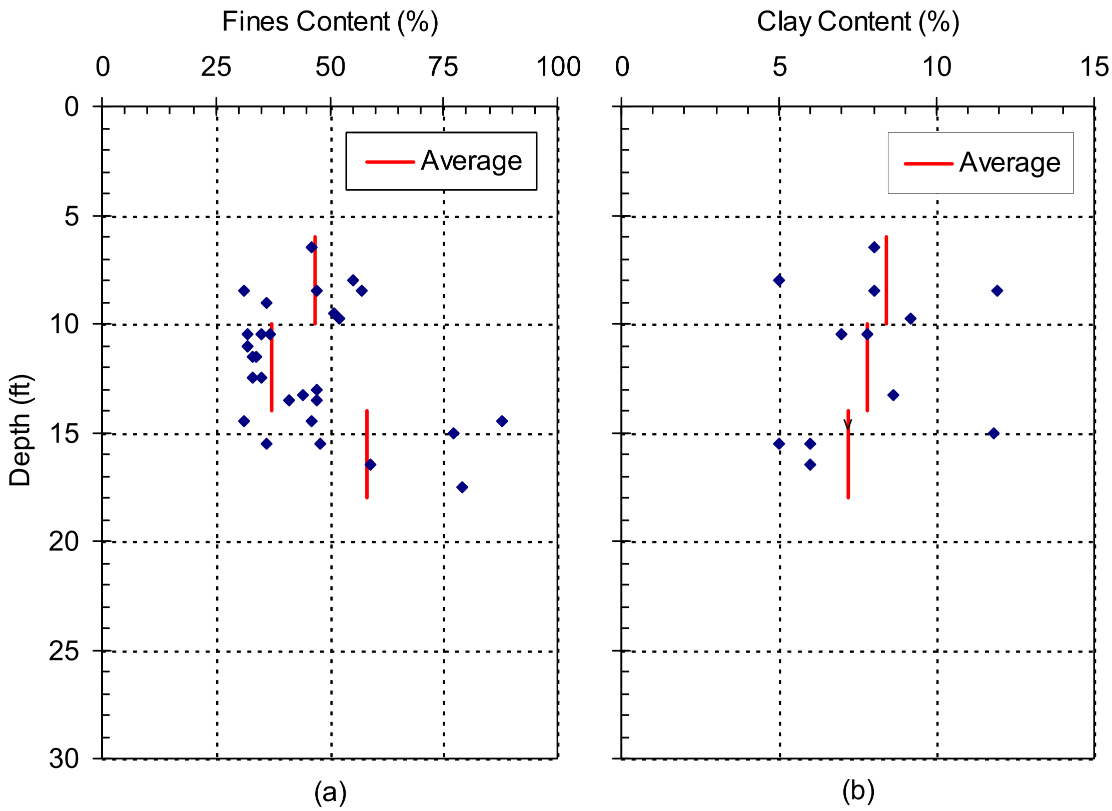


Figure 8-3 Profiles of (a) fines content and (b) clay content with averages for the silt and sand data at the Shepard Lane Bridge case history.

Soil liquefaction was deemed a potential hazard at the site and mitigation efforts were required. Liquefiable deposits were most frequently encountered between the upper 10-25 feet of the soil profile. Liquefaction analyses were performed by RB&G using the “Simplified Procedure” developed by Seed and Idriss (1971) with refinements presented at the 1996 NCEER workshop (Youd, et al., 1997). There were some instances of clay deposits in the liquefiable layer; however, these deposits were relatively small and infrequent. The data for the clay deposits were not included in the analysis data set. Due to slight variations in the soil layering, there were some borings where the liquefiable layer began at depths shallower than 10 ft beneath the surface. The data for these borings are included in analyses in order to examine all of the liquefiable silt and sand data for the site.

To prevent liquefaction, the minimum average post-improvement $(N_1)_{60}$ value for a single SPT boring was specified as 23 while the minimum individual $(N_1)_{60}$ value was specified as 18. The average initial blow count for all soil types at the site was very low with a value of only 8 blows per foot.

8.2 Treatment Method

The stone column treatment plan at the Shepard Lane Bridge case history consisted of vibro-replacement stone columns installed in an equilateral triangle pattern. The typical stone column diameter was 3.5 ft with a center to center spacing of about 6.5 ft. The stone column layout is shown in Figure 8-4. The stone column layout resulted in an area replacement ratio (A_r) of 27.4% except in one location where a decreased column spacing of 6.0 ft was used and the area replacement ratio was 33.9%. The area with decreased spacing was used to determine which spacing would be necessary to meet the site objectives. Following the initial test treatment, it was determined that the 6.5 ft spacing with an A_r of 27.4% was adequate and this layout was used for the remainder of the site.

DGI-Menard was the contractor in charge of the stone column and wick drain treatment plan. Stone columns were installed to varying depths depending on the depth of the liquefiable layer, as determined by the pre-treatment SPT testing. Treatment depths typically ranged from 20-25 ft beneath the surface. Stone columns were installed using the dry, bottom-feed approach. Typical treatment time for each stone column was between 30 and 40 minutes. An Enteco E500 model vibro-float was used to install the stone columns and at a frequency of 50Hz with a power output of 98 kW.

During stone column installation, water was observed exiting the wick drains to distances of up to 20 ft away from the point of installation, as shown in Figure 8-5. This observation suggests that drains in close proximity as well as those further away from the point of installation played a role in reducing excess pore pressure build-up.

There were two treatment areas (one at each abutment), which were each about 40 ft by 165 ft in plan area (see Figure 8-6). Each of the treatment areas was divided into three zones for testing purposes. SPT tests were performed in the centers of triangular stone column groupings prior to and following treatment. The stone column and wick drain treatment was completed in 2003.

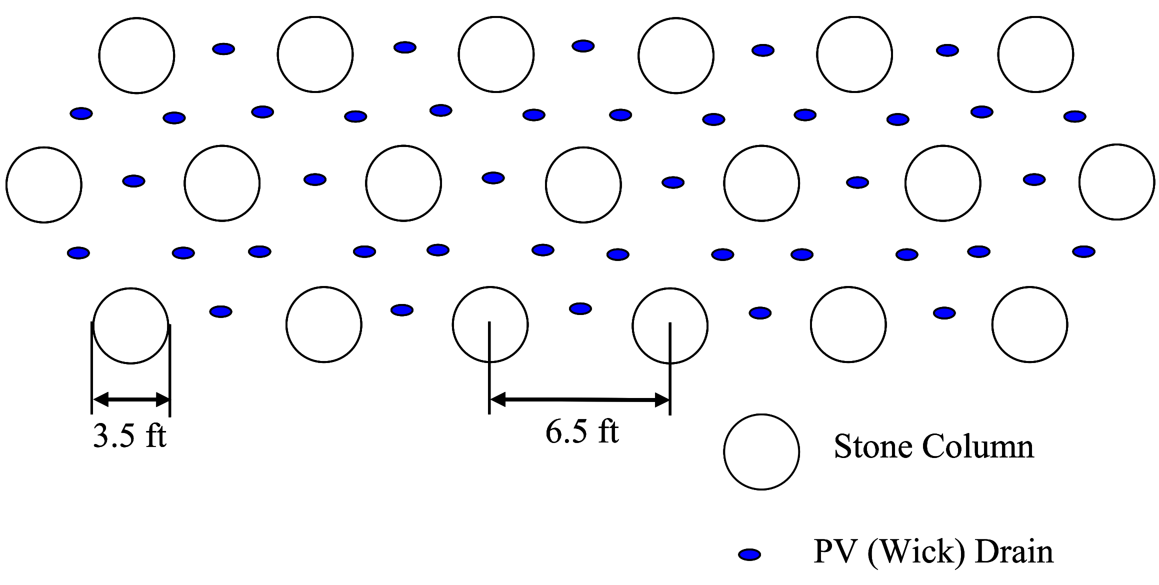


Figure 8-4 Equilateral triangle stone column and wick drain layout for the primary 27% A_r treatment at the Shepard Lane Bridge case history.

Figure 8-5 Photo of water draining from the wick drains during installation of a stone column.

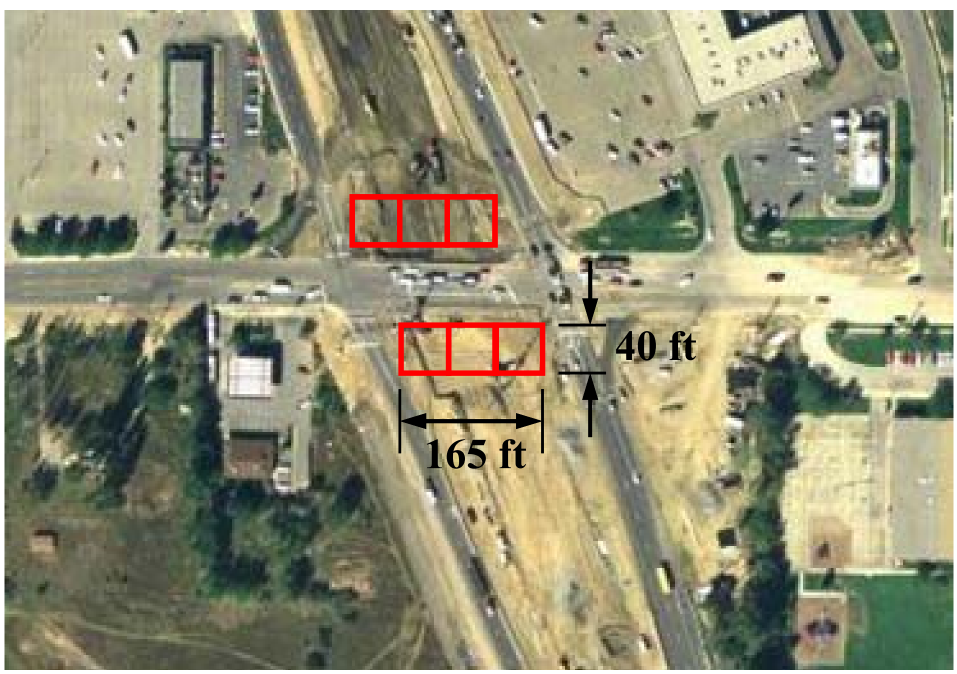


Figure 8-6 Aerial view of the Shepard Lane Bridge case history site with the approximate treatment areas outlined in red.

8.3 Results and Analysis

The primary evaluation of the effectiveness of the treatment was done using SPT testing. Post-treatment testing was done primarily in the center of each section at the midpoint of adjacent stone columns. All of the pre-treatment borings available for this case history were directly comparable to nearby post-treatment borings. Pre- and post-treatment borings were typically within about 7 feet of each other. To evaluate the effectiveness of the treatment plan, plots of pre- and post-treatment blow counts versus depth for (a) all of the soil types and (b) the silt and sand soil types are presented in Figure 8-7. Averages over 2 ft intervals are also shown for both the pre- and post-treatment results. The minimum final blow count criterion is indicated by a dashed line. The data in the figure includes all of the 27.4% A_r sections.

The average lines in Figure 8-7 show that the treatment consistently improved final blow counts in the silt and sand layer. The overall average improvement was 18 blows per foot with the lowest average improvement being 10 blows per foot. The data show that the final blow counts were almost always above the minimum final blow count acceptance criterion of 18 blows per foot for individual samples.

To further investigate the treatment results, the individual pre- and post-treatment boring data were plotted versus depth for the 27.4% A_r areas in Figure 8-8 and Figure 8-9. The minimum final blow count is indicated by a dashed line on each of the plots. Data points with high clay contents (>15%) or which may have had clay lenses or layers in the sample are marked with an "X" through the data points.

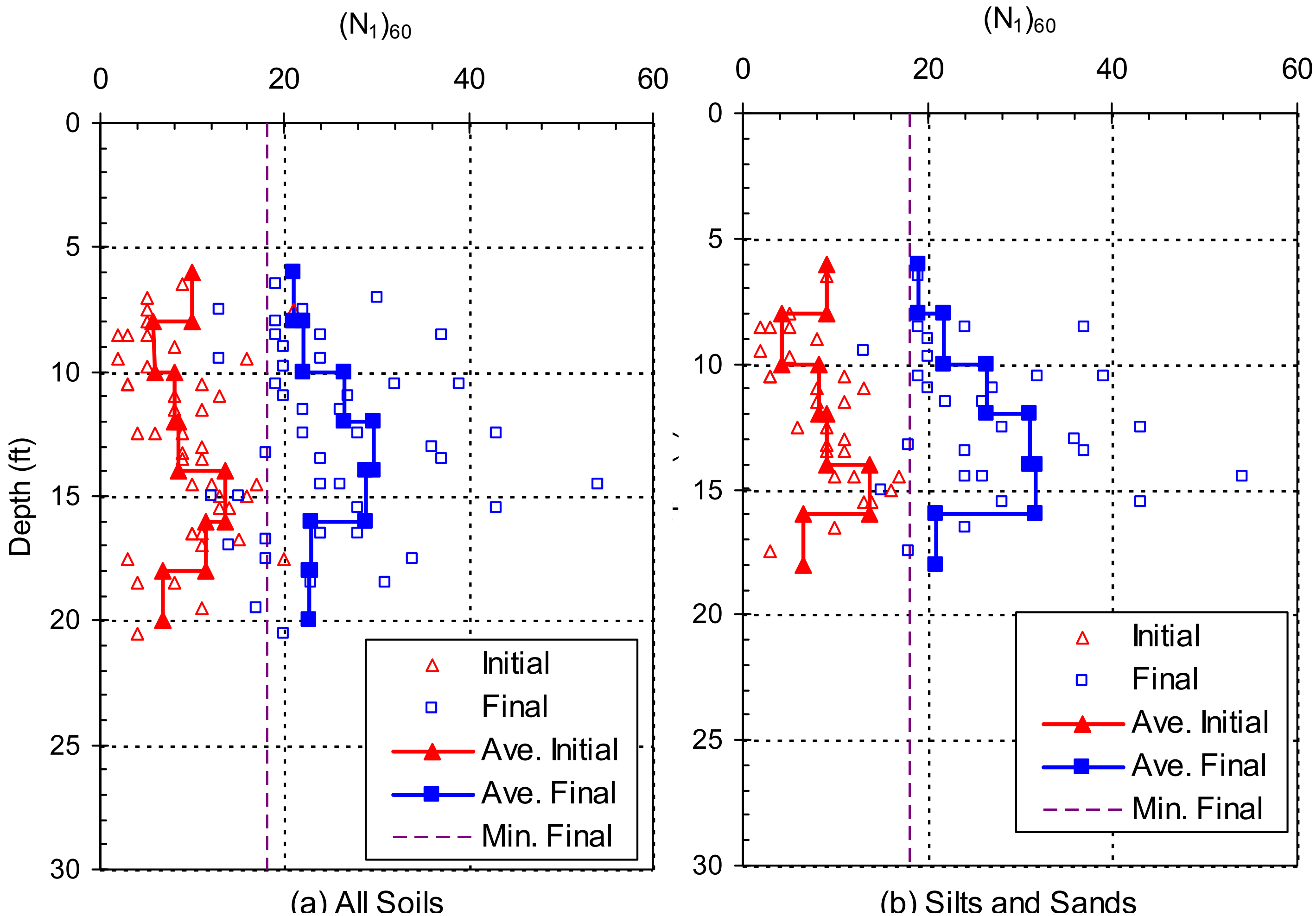


Figure 8-7 Initial and final $(N_1)_{60}$ values versus depth with average lines for (a) all soil types and (b) silt and sand soil types for the 27.4% A_r areas at the Shepard Lane Bridge case history. The minimum final blow count criterion is indicated using a dashed line.

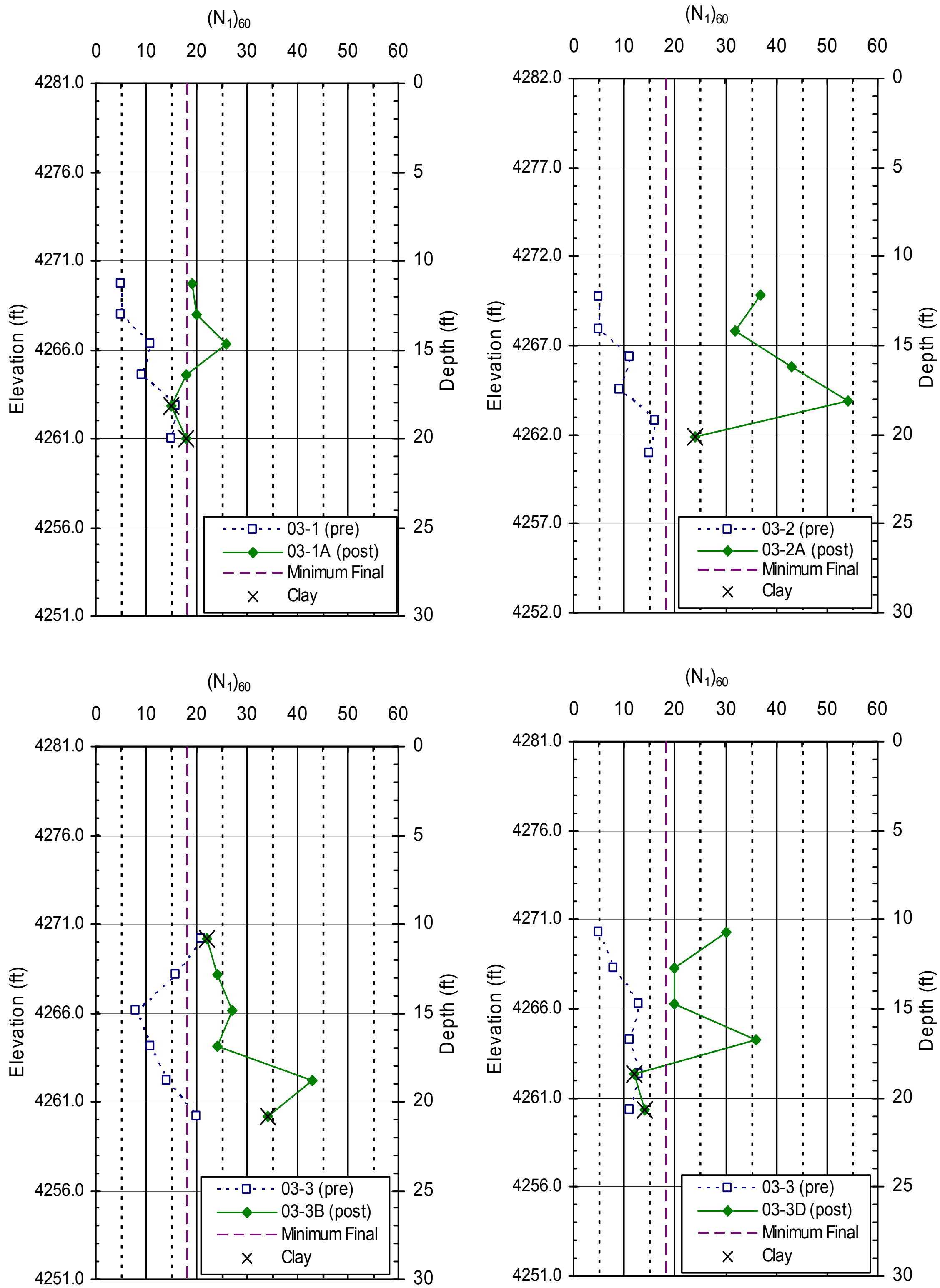


Figure 8-8 SPT results from 27.4% A_r areas for test holes 03-1 to 03-3 at the Shepard Lane Bridge case history. The minimum final blow count criterion is indicated by a dashed line and samples with clayey lenses are marked with an "X". Only the treatment depth results are shown.

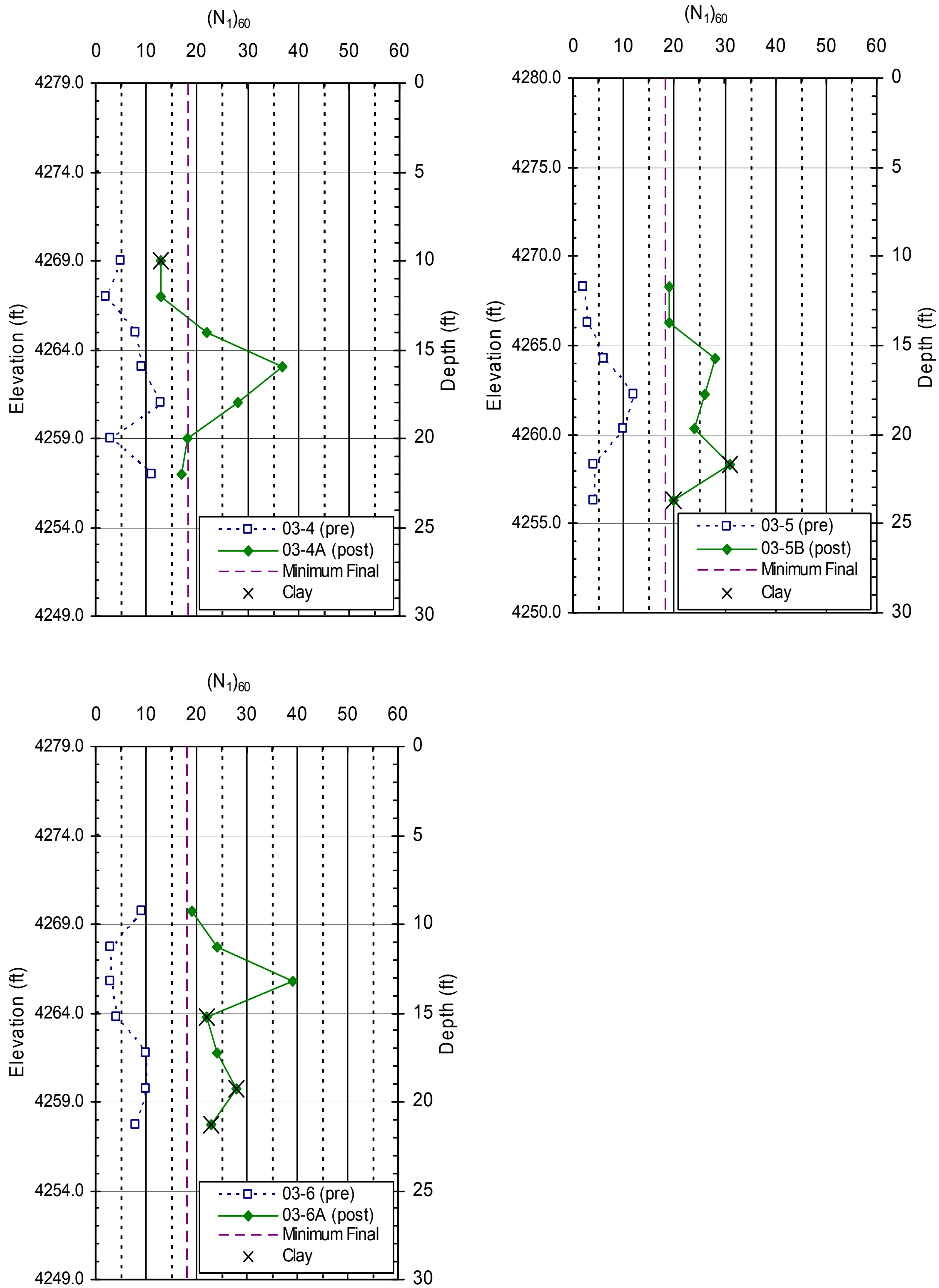


Figure 8-9 SPT results from 27.4% A_r areas for test holes 03-4 to 03-6 at the Shepard Lane Bridge case history. The minimum final blow count criterion is indicated by a dashed line and samples with clayey lenses are marked with an "X". Only the treatment depth results are shown.

The post-treatment testing was typically done within 7 days following treatment; however, the specific time after treatment for each post-treatment test hole was not available for this case history. As a result, the effect of time after treatment cannot be evaluated. The testing was also performed only for the areas of concern instead of for the full profile which saves testing costs and serves the purpose of construction but it limits the academic study of the treatment.

The data in the 27.4% A_r figures show consistent improvement in the blow counts following stone column and wick drain treatment. There were very few final blow counts that did not improve by at least 10 blows per foot from the initial blow count. Those values which did not exhibit significant improvement were generally soils with high clay contents or soils with clay layers or lenses in the sample. There were a few samples below the minimum final blow count which were not excluded based on clay content. These values were still accepted though because they were typically individual samples surrounded by high final blow count layers and as such they were not expected to pose a significant problem if they should liquefy during the design earthquake. Boring 03-1A in Figure 8-8 only had an average final blow count of 21, 2 below the minimum average criterion; however, the overall improvement was deemed acceptable and the treatment was accepted without having to add additional stone columns.

Figure 8-10 shows the SPT results for the test area where the stone column spacing was decreased to 6.0 ft. The pre-treatment test hole was labeled 03-3 and was near the transition between the two different stone column test areas. The corresponding post-treatment boring in the 6.5 ft spacing test section was labeled 03-3B while the boring in the 6.0 ft spacing test section was labeled 03-3C. The minimum final blow count criterion is indicated using a dashed line and samples with clay lenses or layers are marked with an “X”.

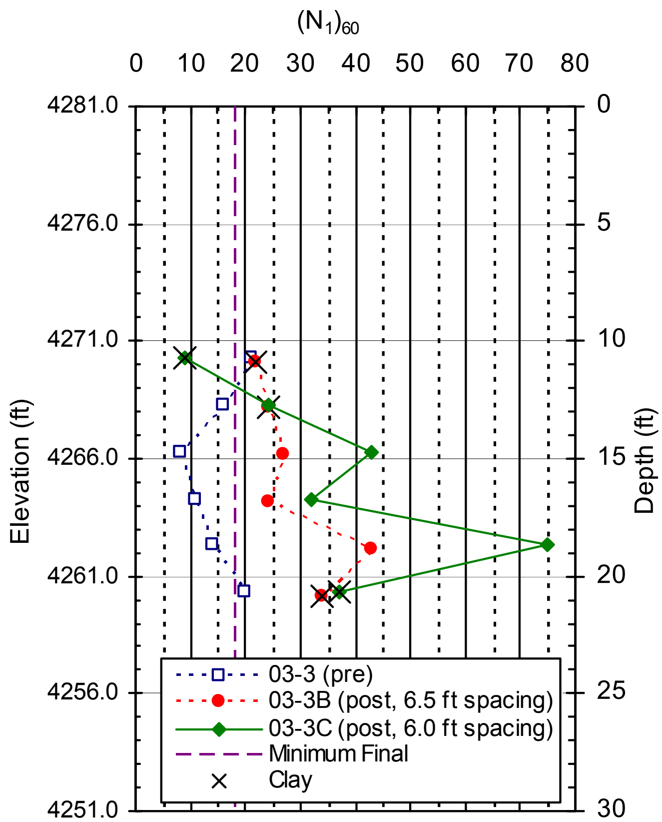


Figure 8-10 SPT results for test holes 03-3B in the 6.5 ft spacing area (27.4% A_r) and 03-3C in the 6.0 ft spacing area (33.9% A_r) of the Shepard Lane Bridge case history. The minimum final blow count criterion is indicated by a dashed line and samples with clayey lenses are marked with an “X”. Only the treatment depth results are shown.

The data shows that both stone column spacings were effective in mitigating the liquefaction potential, as indicated by the final blow counts passing the minimum criteria. There was one value not passing the minimum final blow count, but it was visually classified as having high clay content due to the presence of clay lenses. Table 8-1 presents the average values for both boring 03-3B and 03-3C. The fines contents and average initial blow counts were the same because they both referenced the same initial test hole. The 6.5 ft spacing produced a relatively high final blow count of 30 while the 6.0 ft spacing produced an even higher final blow count of 42 (39% higher). There was a 120% increase in the 27.4% A_r area and a 206% increase in the 33.9% A_r area. Although the overall improvement in the 6.0 ft spacing areas was significantly greater than that of 6.5 ft spacing areas, the acceptance criteria was met by both treatments and the 6.5 ft spacing was used at the remainder of the site.

Table 8-1 Values comparing the results from the 33.9% A_r test hole to the nearby 27.4% A_r test hole at the Shepard Lane Bridge case history.

Test Hole Number	Column Spacing (ft)	A_r (%)	Ave. Fines Content (%)	Ave. Initial $(N_1)_{60}$	Ave. Final $(N_1)_{60}$	Ave. $\Delta(N_1)_{60}$	Increase (%)
03-3B	6.5	27.4	38	14	30	17	120
03-3C	6.0	33.9	38	14	42	28	206

$\Delta(N_1)_{60}$ versus depth is shown in Table 8-1 with averages over 4 ft intervals shown. The scatter is significant but the averages are relatively consistent, with a slight increase over the 10-14 ft interval. In Figure 8-3 it was noted that the fines content decreased to 37% over the same interval. A decrease in fines content may account for increased improvement, although the lower layer improvement is similar to that of the upper layer improvement despite a difference of 10% in average fines contents. It is possible that increasing fines content affects improvement only for fines contents less than about 50%, or that there is some other point at which increasing fines contents does not affect improvement. This is investigated in Figure 8-12 where fines content is plotted versus $\Delta(N_1)_{60}$. A logarithmic trend line is also presented in the figure.

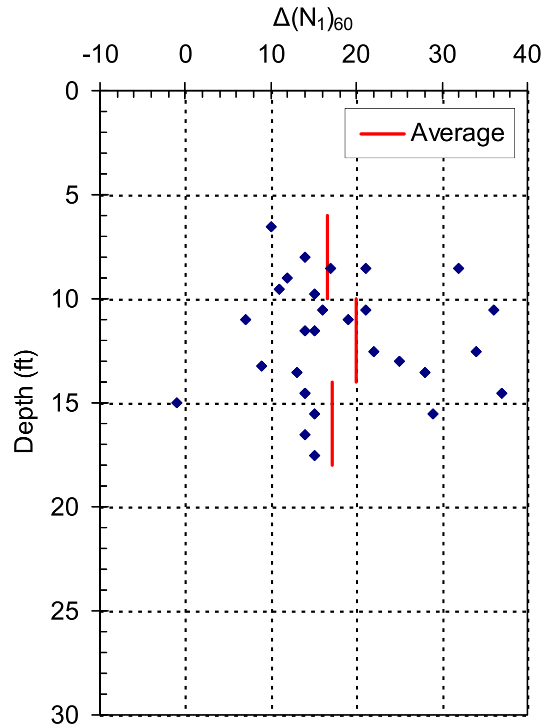


Figure 8-11 $\Delta(N_1)_{60}$ versus depth with average lines for the 27.4% A_r treatment area data at the Shepard Lane Bridge case history.

The data in Figure 8-12 indicates that improvement decreases as fines content increases. The data is scattered and the R-squared value of the trend line is only 0.1512. There is not enough data and the data is too scattered to verify a point at which increasing fines content does not affect improvement. The only negative improvement was for a sandy silt sample with a clay content of 11.8%. If this value were excluded, then the trend line would decrease less with increasing fines content, although definitive conclusions are not possible from the measured data.

There were a several measured clay contents available for the Shepard Lane Bridge case history. Clay content versus $\Delta(N_1)_{60}$ is presented in Figure 8-13 with a logarithmic trend line. The scatter is significant and the R-squared value for the trend line is essentially zero, therefore the line is more or less an average. Most of the samples with high clay content had intermixed clay lenses or layering which was visually observed during testing; therefore, the exact clay contents were not measured. The data indicates that clay content increases between 5-12% do not affect improvement.

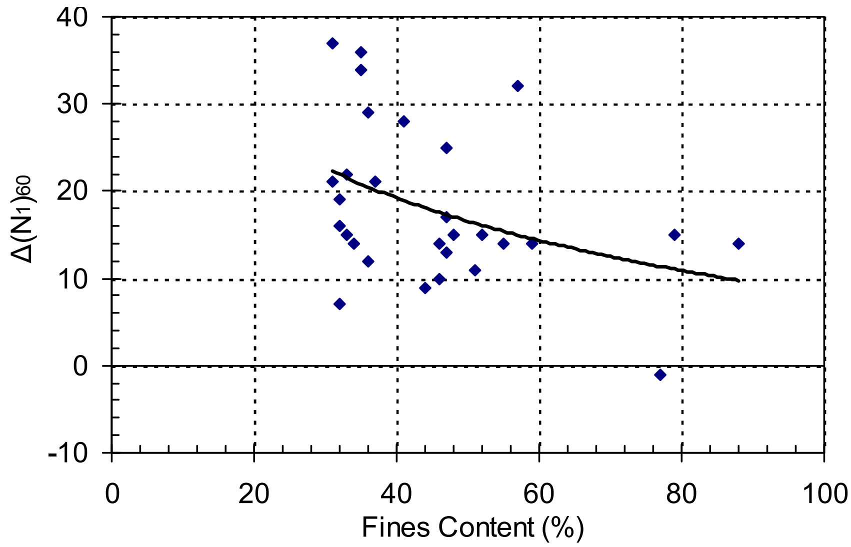


Figure 8-12 Fines content versus $\Delta(N_1)_{60}$ with a logarithmic trend line for the 27.4% A_r treatment data at the Shepard Lane Bridge case history.

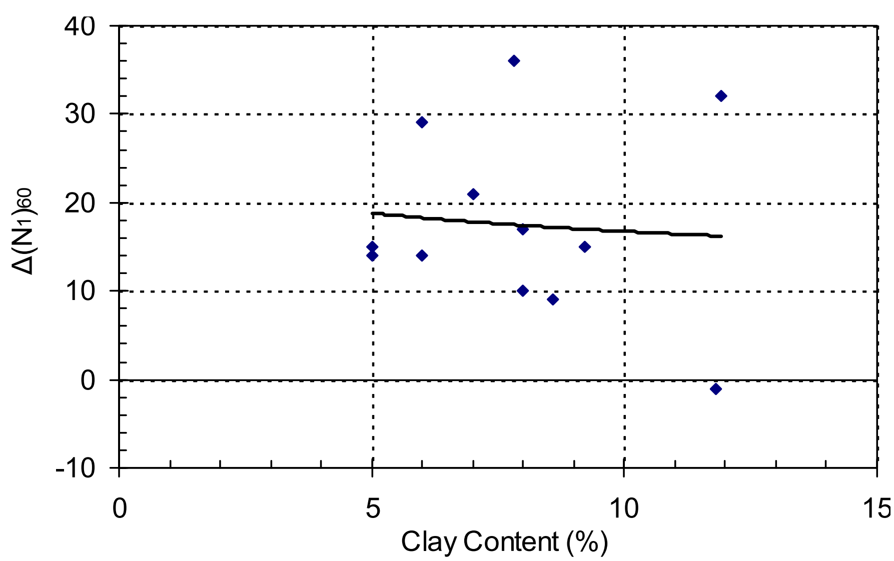


Figure 8-13 Clay content versus $\Delta(N_1)_{60}$ with a linear trend line for the 27.4% A_r treatment data at the Shepard Lane Bridge case history.

Initial $(N_1)_{60}$ versus $\Delta(N_1)_{60}$ is presented in Figure 8-14. Similar to the clay content figure, the data is scattered and the R-squared value of 0.0038 is essentially zero. The R-squared value indicates that 0.3% of the variance in improvement is due to changes in initial blow count. The data shows average improvement of approximately 18 blows per foot with almost all of the improvements being greater than 10 blows per foot. Typically there is expected to be a trend where increasing initial blow counts result in decreased improvement. The data does not indicate such a trend for the Shepard Lane Bridge case history, although this may due to the fact that the initial blow counts are almost all below 13-14 blows per foot.

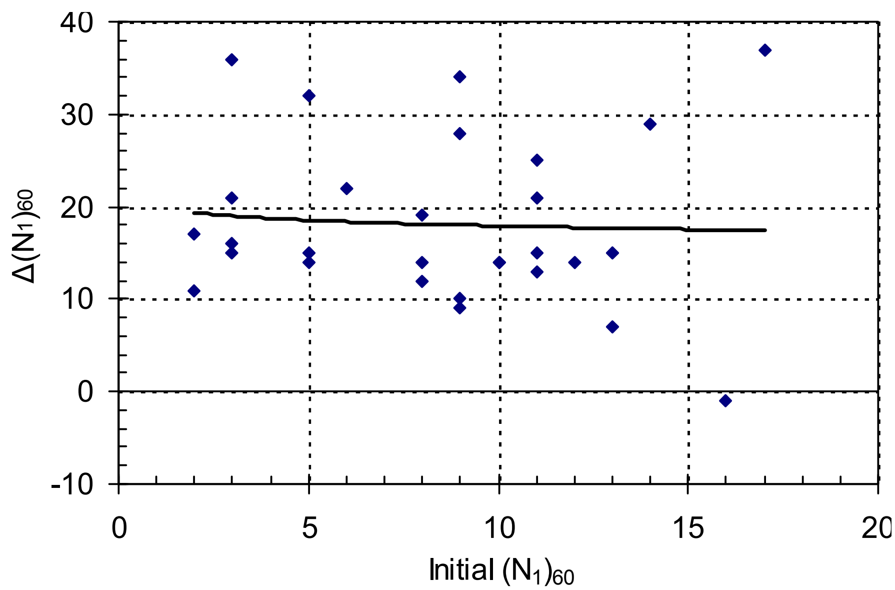


Figure 8-14 Initial $(N_1)_{60}$ versus $\Delta(N_1)_{60}$ with a logarithmic trend line for the 27.4% A_r treatment data at the Shepard Lane Bridge case history.

Initial $(N_1)_{60}$ versus final $(N_1)_{60}$ is presented in Figure 8-15 along with the logarithmic trend line that best represents the data. The R-squared value is only 0.1268. The trend line indicates that increased initial blow counts tend toward increased final blow counts, as would be expected. The initial blow counts were all mostly less than 15 though so this trend is not fully developed in the data. As noted previously, the final blow counts were almost all over 18 and the treatment was successful in meeting the stated final blow count objectives.

To further investigate the effect of decreased stone column spacing and increased A_r, the initial blow counts for both the 27.4% A_r treatment area and the 33.9% A_r treatment area are plotted versus their corresponding final blow counts in Figure 8-16. Logarithmic trend lines that best represent the data are also presented.

The trend lines confirm the previous observation that decreasing the spacing effectively increased the final blow counts by slightly more than 10 blows per foot on average although there are two outliers in the data. The R-squared values are both very low, 0.1268 for the 27.4% A_r trend line and 0.005 for the 33.9% A_r trend line. Unfortunately, there were very few data points for the area with 6.0 ft spacing to compare to the 6.5 ft spacing since only one small test section was treated with the 6.0 ft spacing. Further data may have allowed for additional analyses or more accurate regression. The most accurate comparison of the two different treatments is considered to be the direct comparison of borings 03-3B and 03-3C since they represent roughly equivalent soil conditions and have the same number of data points to compare. The general trend is that increasing the A_r by decreasing the spacing, which subsequently increases the number of columns and drains, results in increased final blow counts.

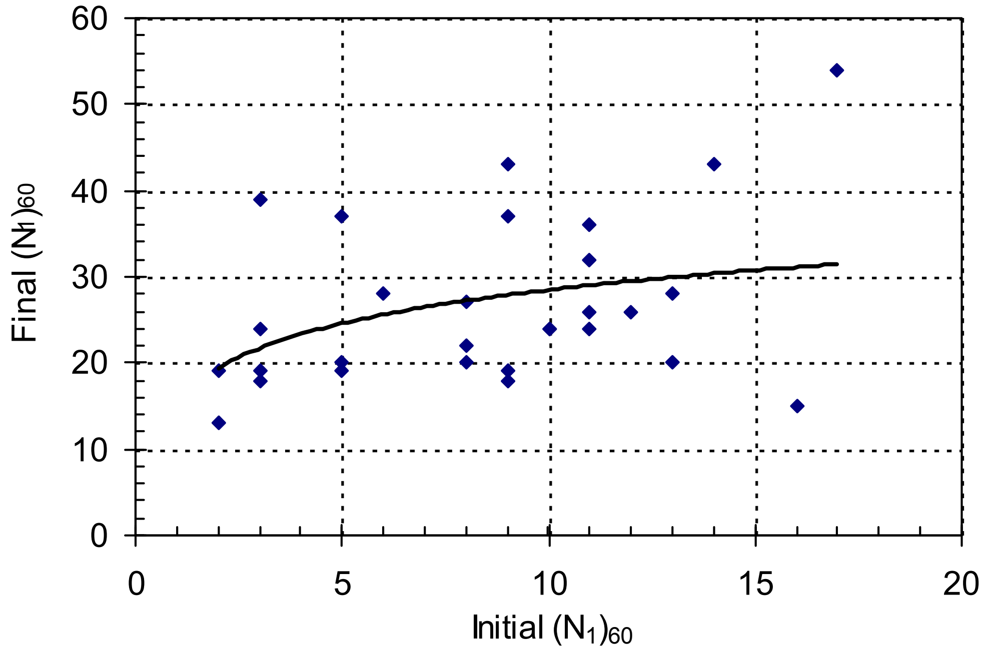


Figure 8-15 Initial $(N_1)_{60}$ versus final $(N_1)_{60}$ with a logarithmic trend line for the 27.4% A_r treatment data at the Shepard Lane Bridge case history.

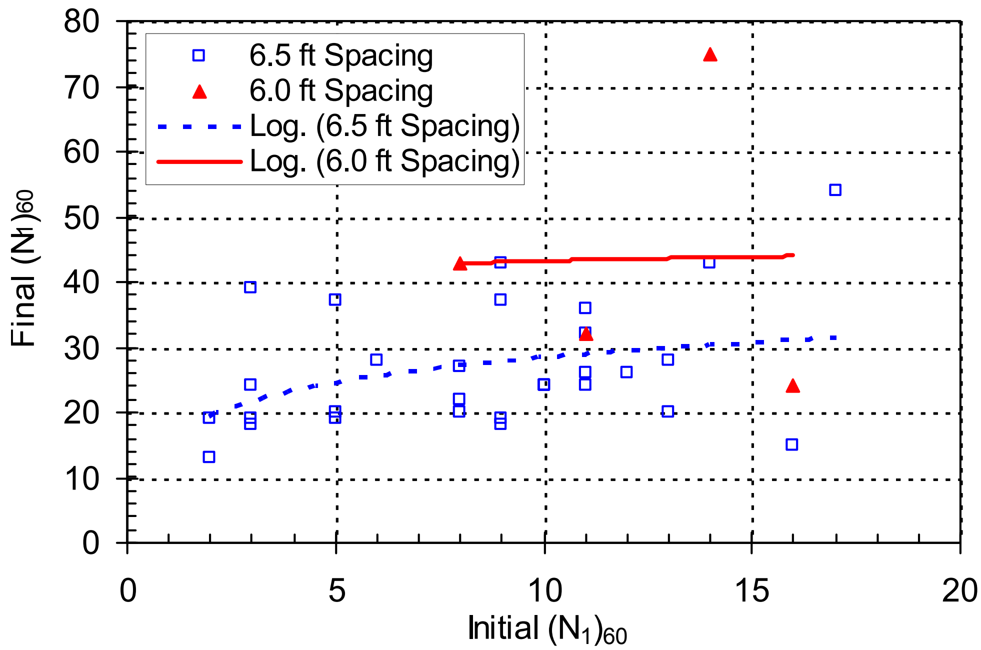


Figure 8-16 Initial $(N_1)_{60}$ versus final $(N_1)_{60}$ with a logarithmic trend lines for the 6.5 ft stone column spacing areas (27.4% A_r) and the 6.0 ft stone column spacing areas (33.9% A_r) of the Shepard Lane Bridge case history.

Average values for silt and sand data in the 27.4% A_r and 33.9% A_r treatment areas as well as average values for all soil types in the 27.4% A_r treatment area are presented in Table 8-2. The differences between the silt and sand data in the two treatment areas have already been discussed previously, but the general trend was an average increased improvement of 10 blows per foot. The percent increases for the two areas were both about 200% but the average final blow count was significantly higher (15 blows per foot) in the 33.9% A_r treatment areas. The data for the 33.9% Ar treatment areas was limited by the number of values available and the scatter in the data; only 5 data points were available and the standard deviation of the final blow counts was 9, or 36% of the average. The 27.4% treatment plan was able to meet the acceptance criteria for mitigating liquefaction potential though, and the average final blow count of 27 exceeded the site acceptance criteria of 23. The standard deviation of the final blow counts for this treatment was 10, or 37% of the average.

The overall site averages, including all soil types, were mitigated successfully with an average final blow count of 25 and 174% improvement. The 33.9% A_r treatment was very successful in silts and sands with 37% fines and 13% clays while the 27.4% A_r was successful in mitigating liquefaction in silts and sands with 46% fines and 8% clay contents.

To compare the results for this case history to the published literature, final $(N_1)_{60\text{-}cs}$ values are plotted versus initial $(N_1)_{60\text{-}cs}$ and logarithmic trend lines for the silts and sands in the 24.7% Ar and 33.9% Ar treatment areas are presented in Figure 8-17. The logarithmic trend lines are then compared to the curves developed by Baez (1995) for clean sands with fines contents less than 15% which are also presented in Figure 8-18.

The trend lines are not highly representative of the data due to scatter (R-squared values less than 0.15); however, by comparing them to the Baez clean sand curves the effectiveness of the Shepard Lane Bridge site treatment plan may be compared generally. The logarithmic trend lines for the Shepard Lane Bridge site were developed from data with initial clean sand blow counts greater than 7-15 blows so the trend lines that are compared to the Baez curve likewise begins at 7-15 blows.

Table 8-2 Averages for the different soil types and area replacement ratios from the Shepard Lane Bridge case history.

	Soil Type		
Properties and Statistical Measures	SM/ML	SM/ML	All Soils
Area Replacement Ratio, Ar (%)	27.4	33.9	27.4
Average Fines Content (%)	46	37	53
Average Clay Content (%)	8	13	14
Average Initial $(N_1)_{60}$	8	14	9
Average Final $(N_1)_{60}$	27	42	25
Average $\Delta(N_1)_{60}$	18	28	16
Average % Increase in $(N_1)_{60}$	215	206	174
Standard Deviation (Finals)	10	20	9
Sample Size	29	5	44

The trends for the $(N_1)_{60\text{-}cs}$ curves are roughly equivalent to the $(N_1)_{60}$ curves with the 33.9% A_r data averaging about 15 blows/ft higher final blow counts than the 27.4% A_r data. The scatter is very significant for the 33.9% A_r data with outliers that are 25-35 blows/ft higher or lower than the average.

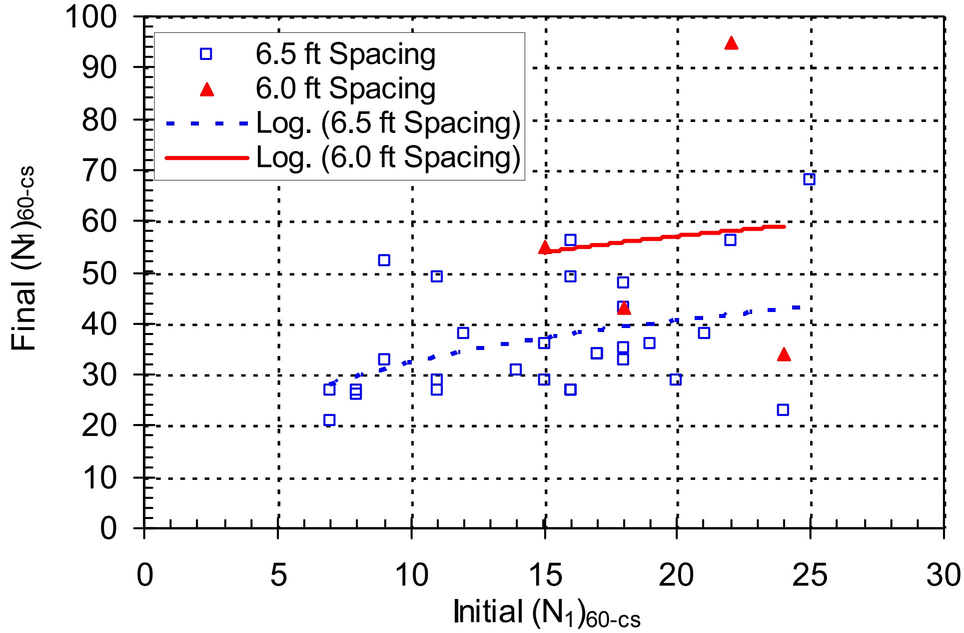


Figure 8-17 Initial $(N_1)_{60\text{-}cs}$ versus final $(N_1)_{60\text{-}cs}$ with a logarithmic trend lines for the 6.5 ft stone column spacing areas (27.4% A_r) and the 6.0 ft stone column spacing areas (33.9% A_r) of the Shepard Lane Bridge case history.

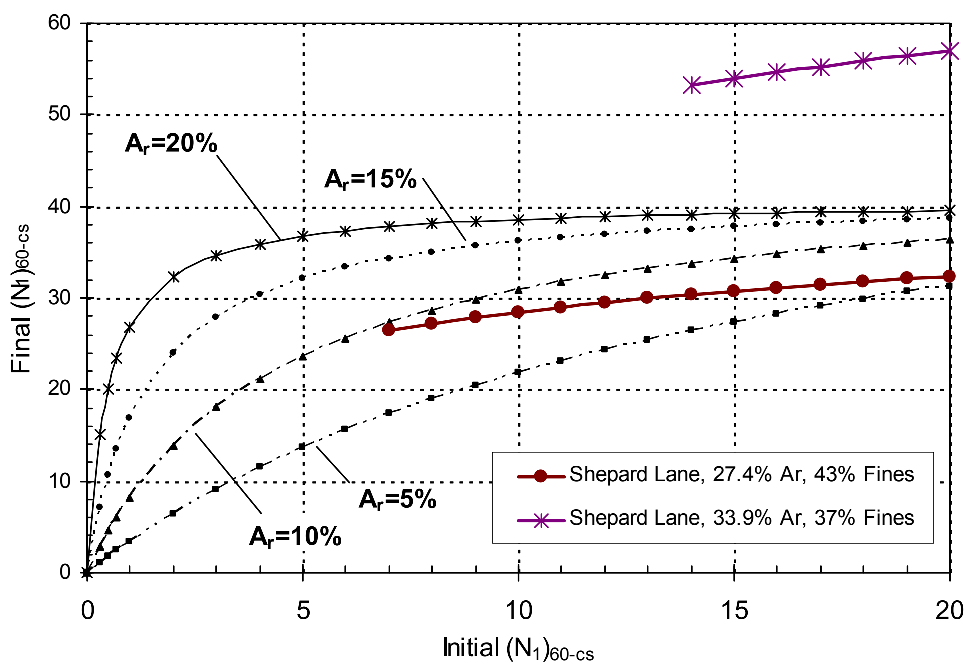


Figure 8-18 Initial $(N_1)_{60\text{-}cs}$ versus final $(N_1)_{60\text{-}cs}$ with the clean sand curves developed by Baez (1995) as well as the logarithmic trend lines for the 6.5 ft column spacing (24.7% A_r) and the 6.0 ft column spacing (33.9% A_r) silt and sand data from the Shepard Lane Bridge case history silt and sand data.

The 33.9% A_r trend line showed average improvement of about 15 blows higher than the clean sand 20% A_r curve, an improvement of about 37%. The average fines content for this data was 37%. The 27.4% A_r trend line showed improvement that was somewhere between the 5% A_r and 10% A_r clean sand curves. The average fines content for this data was 43%. The wick drains did not supplement the stone columns enough to counteract the negative effect of the fines content in the primary 27.4% A_r treatment areas, although sufficient improvement was attained to meet the project objectives.

8.4 Conclusions

- The primary 27.4% A_r stone column and wick drain treatment plan successfully increased the average initial blow count from 8 to 27 in the silt and sand layer, an increase of 215%. The average fines content for this layer was 46%, the average clay content was 8%, and the fines were typically non-plastic. There was no need to add additional stone columns to meet the treatment objectives.
- A preliminary test section where stone column spacing was decreased from 6.5 ft (24.7% A_r) to 6.0 ft (33.9% A_r) yielded final blow counts 39% higher than adjacent 6.5 ft spacing treatment.
- Typical decreases in improvement due to increases in initial blow count were not observed. This is likely due to the initial blow counts all being less than 18 which reduced the range of initial blow counts in the data set to only low values.

- Wick drains contributed to relieving excess pore water pressure during installation up to 20 feet away from the point of installation, as observed by water exiting the wick drains during installation.
- The 27.4% A_r treatment area (average fines content of 37%) produced final blow count results that were comparable to the 10% A_r clean sand curve produced by Baez (1995) at an initial blow count of 7 and to the 5% A_r clean sand curve at an initial blow count of 20. Due to scatter the results represent average improvements at the site.

9 Cherry Hill Bridge Abutment, Kaysville, Utah

9.1 Site Overview and Soil Conditions

The Cherry Hill Bridge is located at the Cherry Hill Interchange on US 89 in Farmington, Utah. Previously there had not been a bridge at the site. The new bridge serves as the northbound off-ramp of I-89 into the Cherry Hill area of Kaysville and Fruit Heights. The bridge abutments and bent were placed over improved soil to prevent liquefaction. Figure 9-1 shows an aerial view of the interchange with the Cherry Hill Bridge circled near the center of the image. It should be noted that wick drains were not used at this case history and it is presented in order to establish some measure of comparison for the stone column method with and without drains are sites with high fines contents.

RB&G Engineering, Inc. prepared the report of geotechnical observation and testing for the site. The design earthquake for the site is a magnitude 7.4 earthquake with a peak ground acceleration of 0.6g. The peak ground acceleration is based on a 2% probability of being exceeded in 50 years (~2500 yr recurrence interval). The site is located within about a mile of the Wasatch fault.

Figure 9-1 Aerial view of the Cherry Hill Bridge at the US 89 and Cherry Hill Interchange in Farmington, Utah.

Prior to treatment, nearly continuous SPT sampling was performed at the test site to determine liquefaction potential and soil characteristics at the site. The soil profiles varied across the site and the layering was not always consistent; however, there were some general layers that were relatively consistent across the site. These layers varied in thickness across the site so the most common thicknesses and depths are used to generalize the soil profile. The generalized soil profile that best represents the layering is presented in Figure 9-2.

The upper 10 feet were typically clay (CL) with gravel and some silty sand (SM) lenses. From 10 feet to about 40 feet the soil profile consisted of silty sand and sandy silt (ML) layers. This layer varied in thickness across the site with thicknesses as little as 25 feet to thicknesses as much as 40 feet thick. Beneath this layer were interbedded layers of silty sand and sandy silt with some clay layers to a depth of about 50 feet, although the depth varied across the site. Underlying the interbedded layers was a clay layer with occasional silty sand and sandy silt lenses.

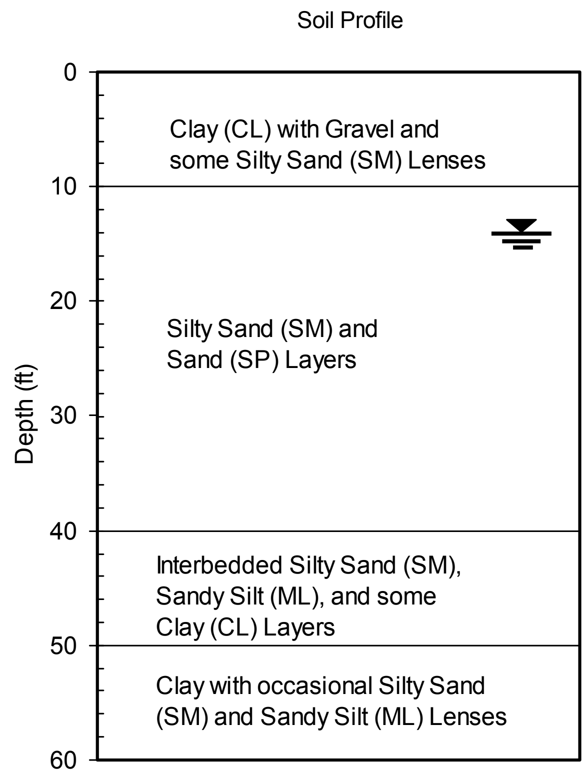


Figure 9-2 Generalized soil profile for the Cherry Hill Bridge case history.

The main layer of concern was the silty sand and sand layer between approximately 10-40 ft beneath the surface. For the 10-40 ft layer, only the silt and sand data were used in analyses in order to focus on the objectives of this study. Average values referenced for this layer were obtained from the silts and sands data, unless otherwise noted. Fines content (a) and clay content (b) profiles are presented for all soil types in Figure 9-3 while the profiles for silts and sand data are presented in Figure 9-4.

Average lines over 5 ft intervals are shown in both figures for the fines and clay content averages with depth.

In the 10-40 ft layer the fines contents typically ranged from 10-50% with an average fines content of 32%. The fines in this layer were typically non-plastic, but with some PIs of 5 or less. The clay content average for 10-40 ft was about 7%. The layers below 40 ft had higher average fines contents (≈40%) and more clayey soils. The upper soil layer also had higher clay contents.

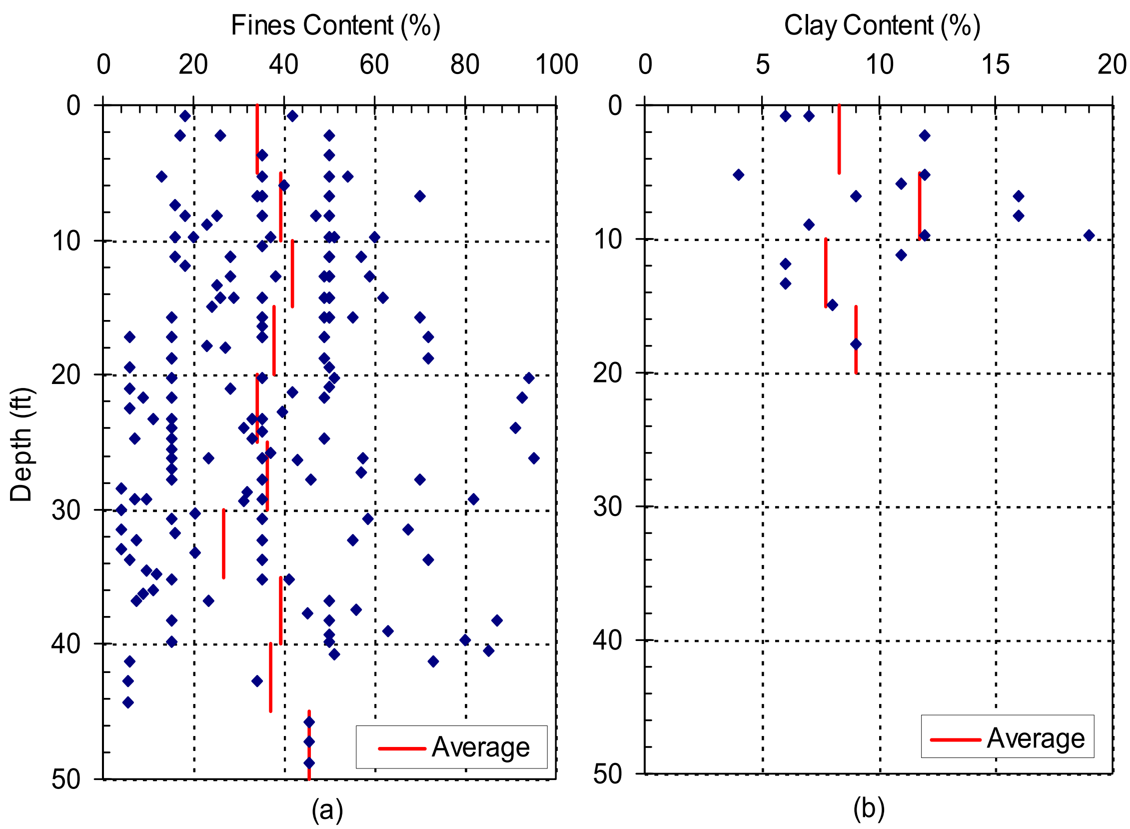


Figure 9-3 Profiles of (a) fines content and (b) clay content for all soil types at the Cherry Hill Bridge case history.

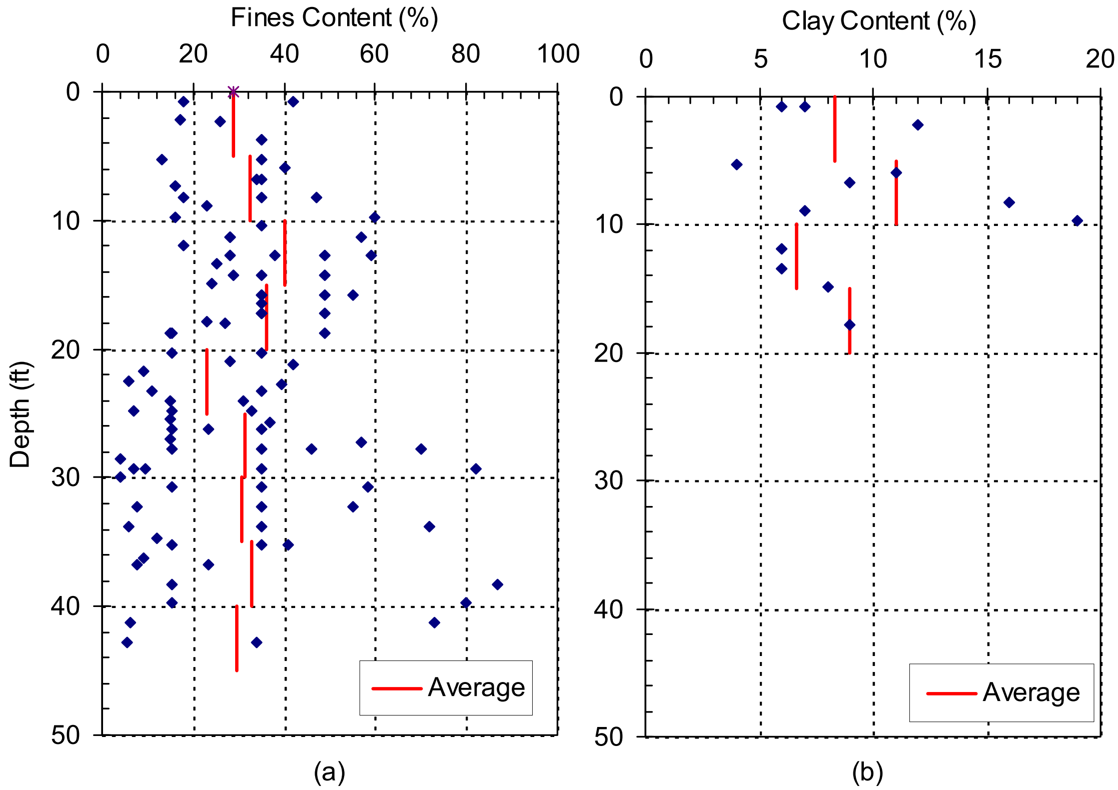


Figure 9-4 Profiles of (a) fines content and (b) clay content for the silts and sands data at the Cherry Hill Bridge case history.

Soil liquefaction was deemed a potential hazard at the site and mitigation efforts were required. Liquefiable deposits were most frequently encountered between the upper 10-50 feet of the soil profile. Liquefaction analyses were performed by RB&G using the "Simplified Procedure" developed by Seed and Idriss (1971) with refinements presented at the 1996 NCEER workshop (Youd, et al., 1997). At some locations, the liquefiable layers were occasionally interrupted by deposits of clays and moderately dense sands, which were not expected to liquefy during the design event. The data for the clay deposits were not included in the analysis data set. Due to variations in the soil layering, there were some borings where the liquefiable layer began at depths shallower than 10 ft beneath the surface. The data for these borings are included in analyses in order to examine all of the liquefiable silt and sand data for the site.

To prevent liquefaction the minimum average post-improvement $(N_1)_{60\text{-}cs}$ (clean sand) value for a single SPT boring was specified as 30 with the minimum individual clean sand value set at 25. For consistency with the other case histories in this study, the normalized blow count was typically referenced instead of the normalized clean sand blow count. The clean sand value was used as the measure of improvement at the site. Some reference is given to the clean sand values in order to address whether the site objectives were met, but the majority of the analyses did not use the clean sand values. The average initial blow count for all soil types at the site was 17 with a clean sand value of 23. The average initial blow count for the silts and sands layers was 18 with a clean sand value of 23.

9.2 Treatment Method

The specialty contractor, Layne Christenson Co., determined the actual stone column layout based on site conditions and guidelines from RB&G Engineering. The stone column treatment plan at the Cherry Hill case history consisted of vibro-replacement stone columns installed in an equilateral triangle pattern. The typical stone column diameter was 2.6 ft with a center to center spacing of about 8.2 ft. The stone column layout is shown in Figure 9-5. The stone column layout resulted in an area replacement ratio (A_r) of 9.3% except in one location where secondary stone columns were installed to meet the project objectives, resulting in a 4.1ft x 7.1ft rectangular arrangement with an area replacement ratio of about 18.6%. Stone columns were installed to varying depths depending on the depth of the liquefiable layer as determined by the pre-treatment SPT testing. Treatment depths typically ranged from about 25-50 ft beneath the surface. Stone columns were installed using the dry, bottom-feed approach. An electrically driven V23 Vibroprobe was used to install the columns. Treatment was performed in the fall of 2000.

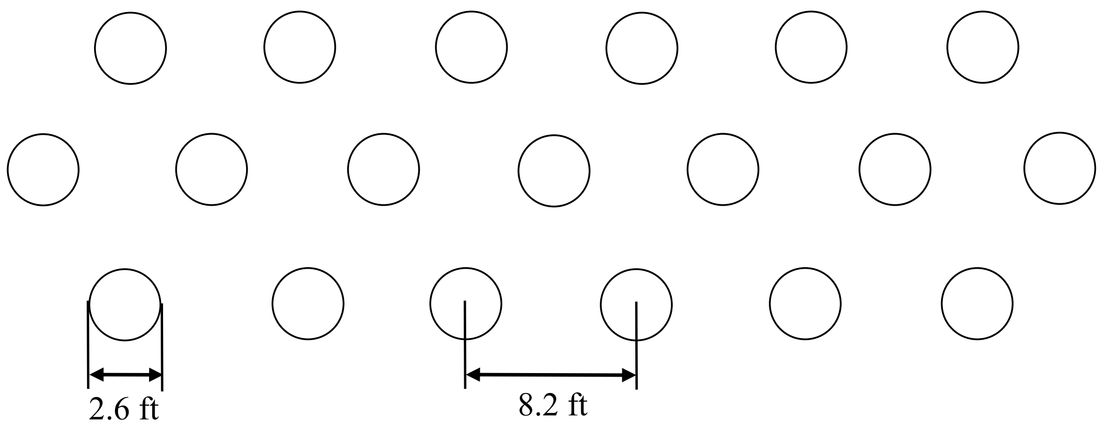


Figure 9-5 Equilateral triangle stone column layout for the Cherry Hill Bridge case history.

There were three treatment areas (one at each abutment and one under the center bent), which were each about 100 ft by 200 ft in plan area (see Figure 9-6). Each of the treatment areas was divided into six zones for testing purposes. SPT and CPT tests were performed in the centers of triangular stone column groupings prior to and following treatment.

Figure 9-6 Aerial view of the three liquefaction mitigation treatment areas at the Cherry Hill Bridge case history.

9.3 Results and Analysis

The primary evaluation of the effectiveness of the treatment was done using SPT testing. Post-treatment testing was done primarily in the center of each section. All of the pre-treatment borings available for this case history were directly comparable to nearby post-treatment borings. Pre- and post-treatment borings were typically within about 7 feet of each other. To evaluate the effectiveness of the treatment plan, a plot of pre- and post-treatment blow counts versus depth is presented in Figure 9-7. Averages over 5 ft intervals are also shown for both the pre- and post-treatment results. The data in the figure includes all of the 9.3% A_r sections.

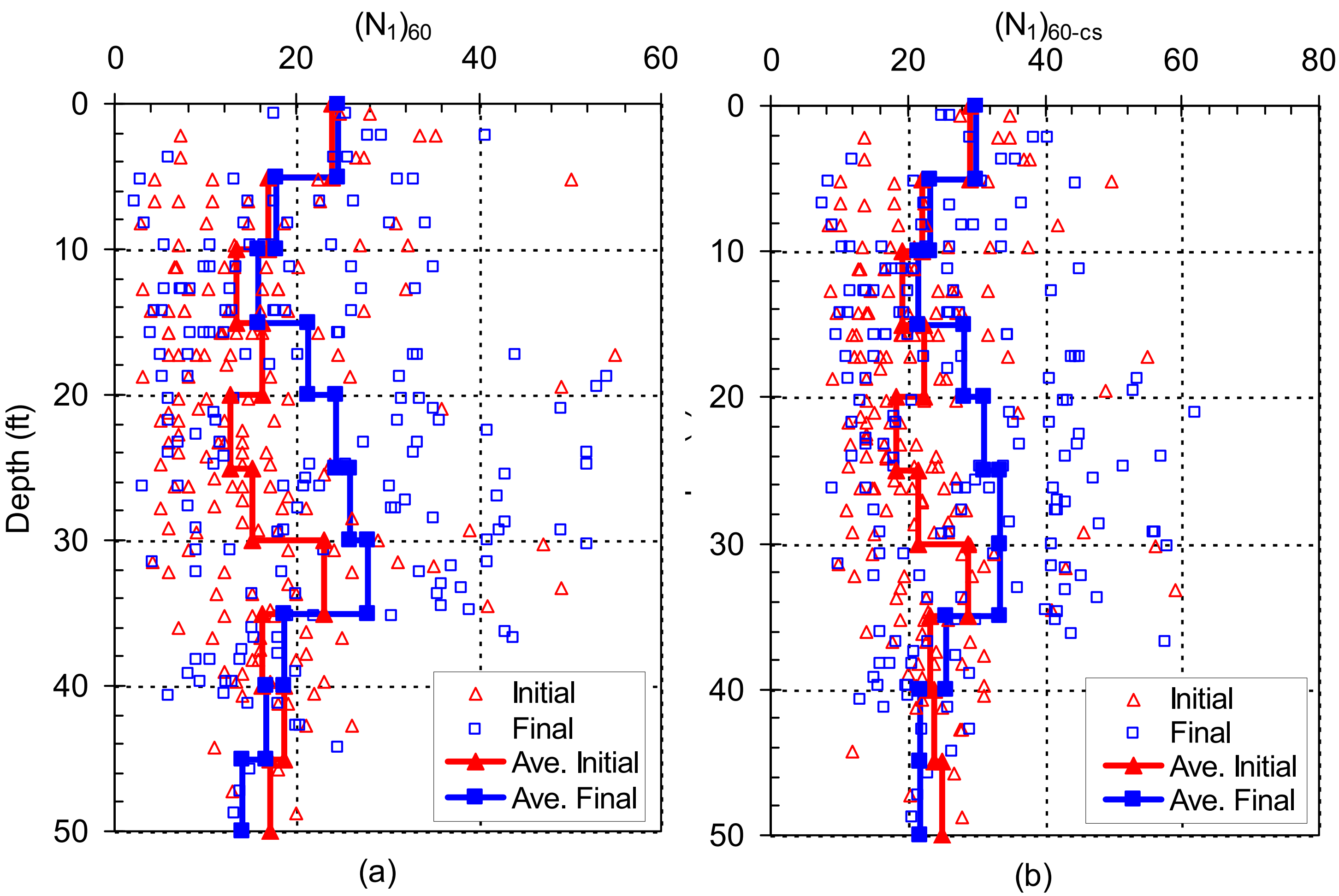


Figure 9-7 Initial and final (a) $(N_1)_{60}$ and (b) $(N_1)_{60\text{-}cs}$ values versus depth for the 9.3% A_r sections of the Cherry Hill Bridge case history. Average lines are shown for 5 ft depth intervals.

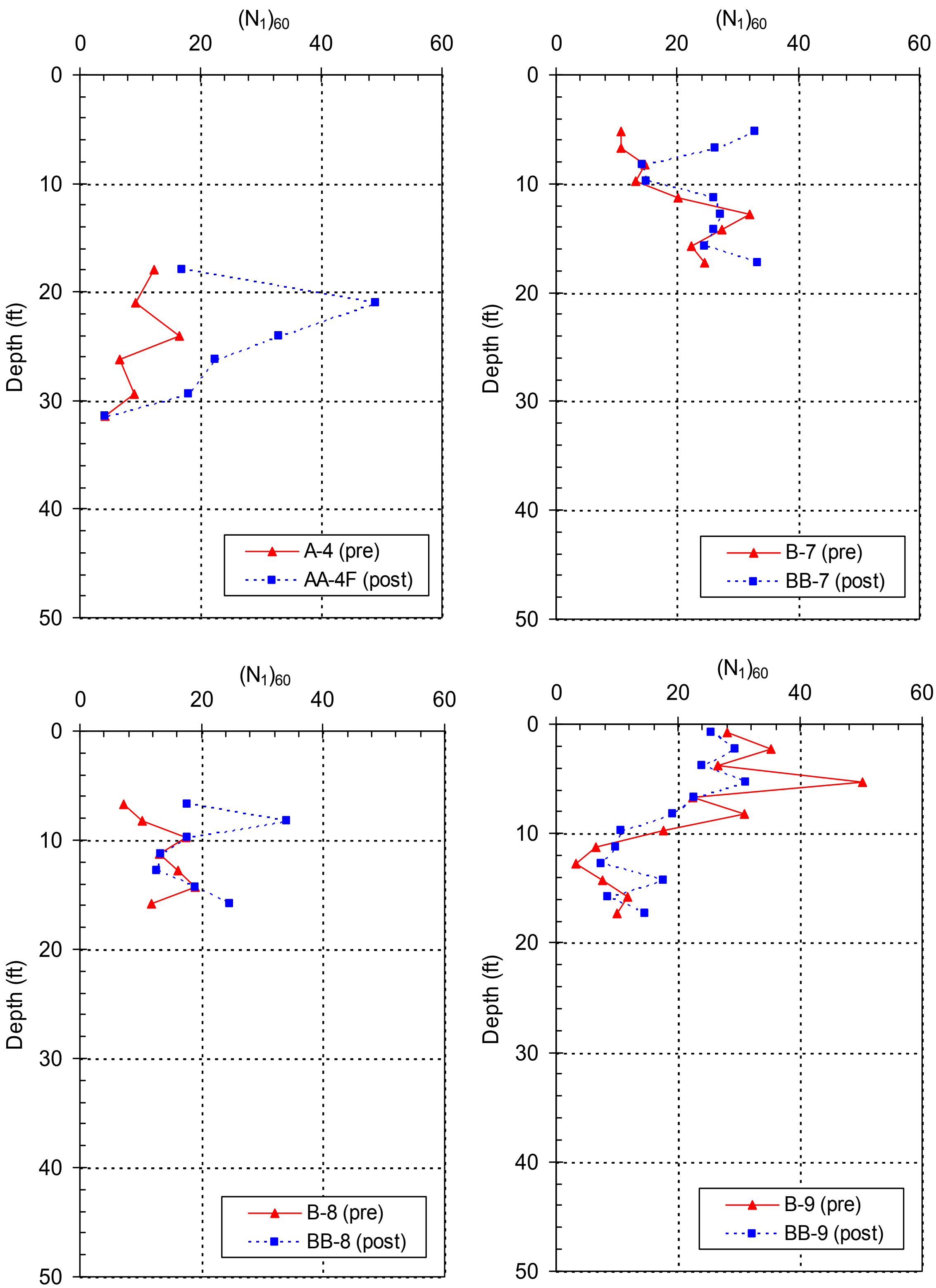


Figure 9-8 SPT test results for direct comparison data from 9.3% A_r treatment sections 4 and 7-9 at the Cherry Hill Bridge case history.

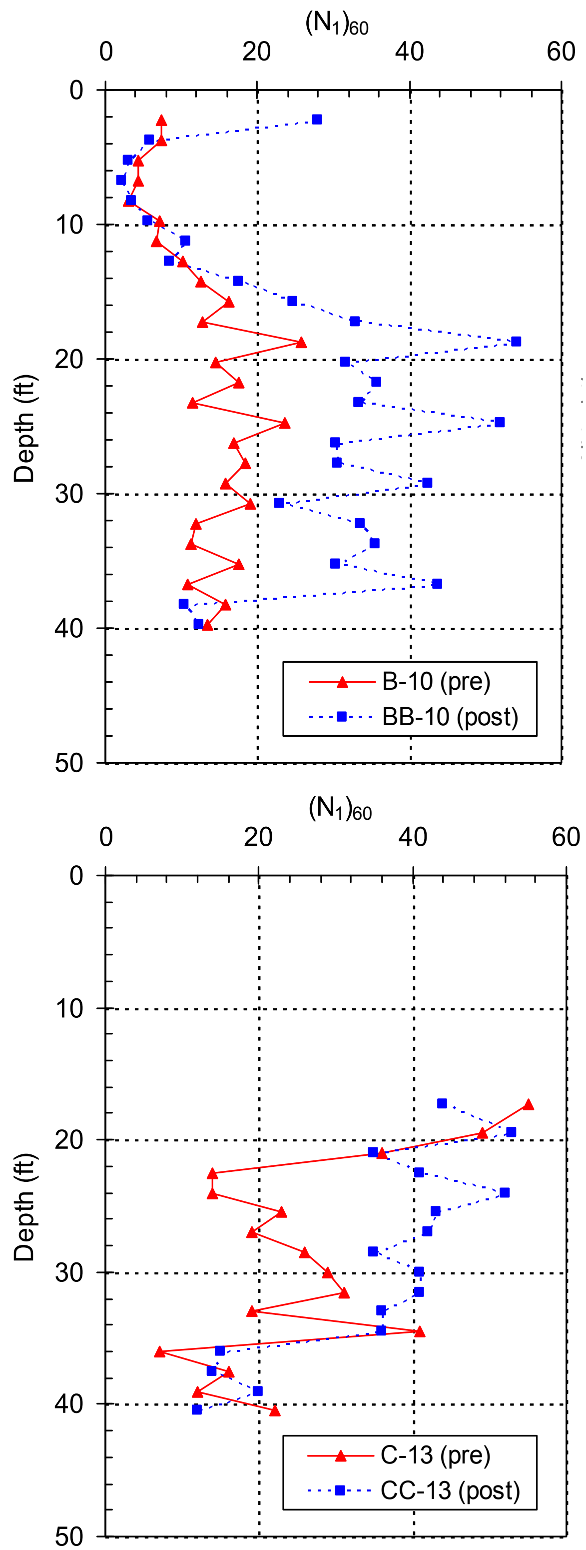


Figure 9-9 SPT test results for direct comparison data from 9.3% A_r treatment sections 10 and 12-14 at the Cherry Hill Bridge case history.

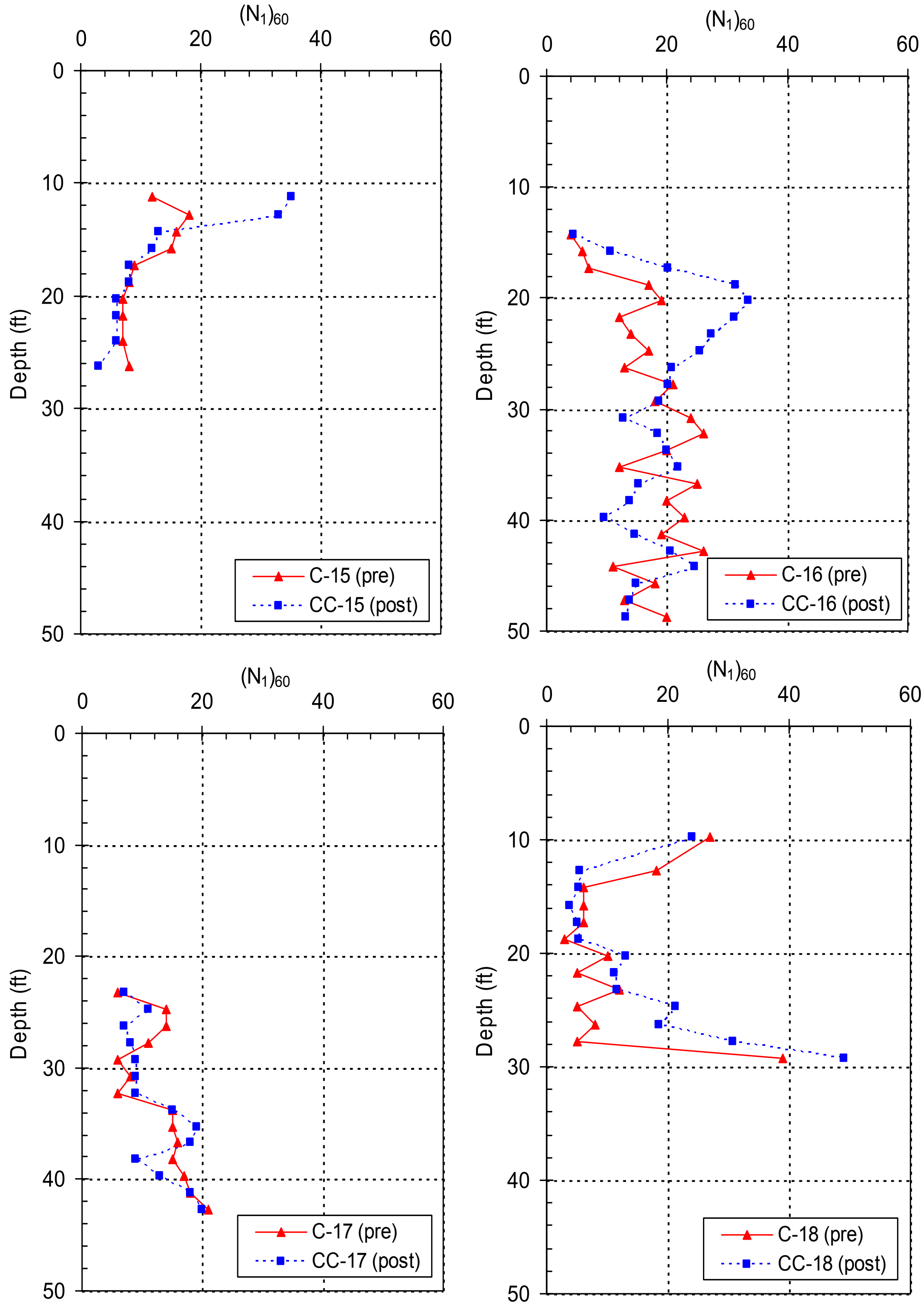


Figure 9-10 SPT test results for direct comparison data from 9.3% A_r treatment sections 15-18 at the Cherry Hill Bridge case history.

The average lines in Figure 9-7 show that the treatment improved final blow counts in the 10-40 ft depth interval; however, values above and below this layer showed little improvement and even some negative improvements. The final blow counts were typically less than 30 on average although there was significant scatter in the data. The clean sand blow count minimum specified for the project was 25 but it can be seen from the data in plot (b) of Figure 9-7 that much of the post-treatment data falls below this minimum value. To further investigate the treatment results, the individual pre- and post-treatment boring data were plotted versus depth for the 9.3% A_r areas in Figure 9-8 to Figure 9-10 and for the 18.6% A_r area in Figure 9-15.

The individual boring results show a very mixed set of results with some areas of significant improvement and other areas of consistent negative improvement. The clean sand charts were not presented in this chapter since the acceptance criteria was of less concern in this study than the amount of improvement in the $(N_1)_{60}$ values; however some explanation of the acceptance criteria will be presented. Boring profiles with both $(N_1)_{60}$ and $(N_1)_{60\text{-cs}}$ values are presented in Appendix A in Figure 13-4 to Figure 13-10 for the reader to reference if desired. The figures in Appendix A also have the minimum final clean sand blow count acceptance criterion indicated by a dashed line.

The data for the 9.3% A_r areas were all deemed acceptable for the treatment objectives; however, the low points were predominantly accepted based on high clay contents or based on being classified as interbedded clay. Some of the data that did not pass was accepted based on the significant improvement in the surrounding layers which would likely prevent the soils not passing the acceptance criterion from liquefying in the event of an earthquake. Unfortunately, clay contents for the data that did not pass the acceptance criteria was typically not available for this study, thus they are not marked on the figures as was done in several of the other case histories. Overall, there were not consistent improvements across the treatment zones, although the treatment was accepted due to the frequent occurrences of clay in the low improvement zones.

Fines content versus $\Delta(N_1)_{60}$ is presented in Figure 9-11 for the 9.3% A_r treatment areas. A logarithmic trend line is shown for the data. The scatter in the data is very large, especially at low fines contents. The higher fines content data (>50%) is less scattered and typically exhibits very low or even negative improvement. The data implies that fines contents greater than 40% typically limit stone column treatment without wick drains to increases of 10 blows per foot or less, with only two exceptions. The R-squared value for the trend line was only 0.015 which is very low and indicates that fines content is not a good direct predictor of improvement ($\Delta(N_1)_{60}$). The data is of limited use other than to note a general decrease in improvement and in the variability of improvement as the fines content increases, especially above 40%. For fines contents less than about 35% there were as much as 50 blows per foot differences in improvement for data with similar fines contents.

$\Delta(N_1)_{60}$ versus initial $(N_1)_{60}$ is presented in Figure 9-12 for the 9.3% A_r treatment areas. The data shows a decrease in improvement as the initial blow counts increase. This is expected since soils with high initial blow counts are typically harder to improve, but it is not of concern since those data point also typically do not liquefy. The data is scattered and the R-squared value for the linear trend line was only 0.0793. There are a very large number of data points showing negative improvement. These data point were often clayey soils or silts and sands with high clay contents based on visual observation;

however, there were multiple silt and sand data points which did not improve. There were only a few measured clay contents available for this case history so a more detailed explanation and analysis was not possible.

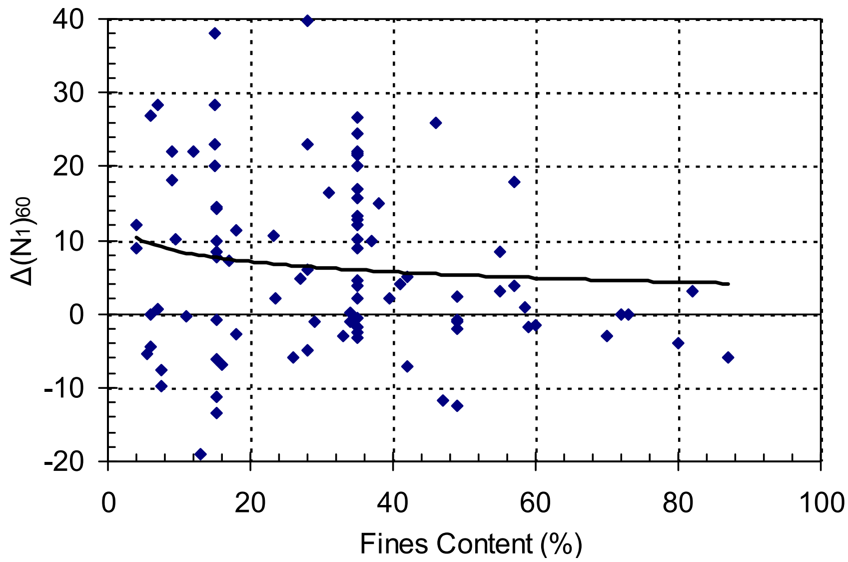


Figure 9-11 Fines content versus $\Delta(N_1)_{60}$ with a logarithmic trend line for the 9.3% A_r data at the Cherry Hill Bridge case history.

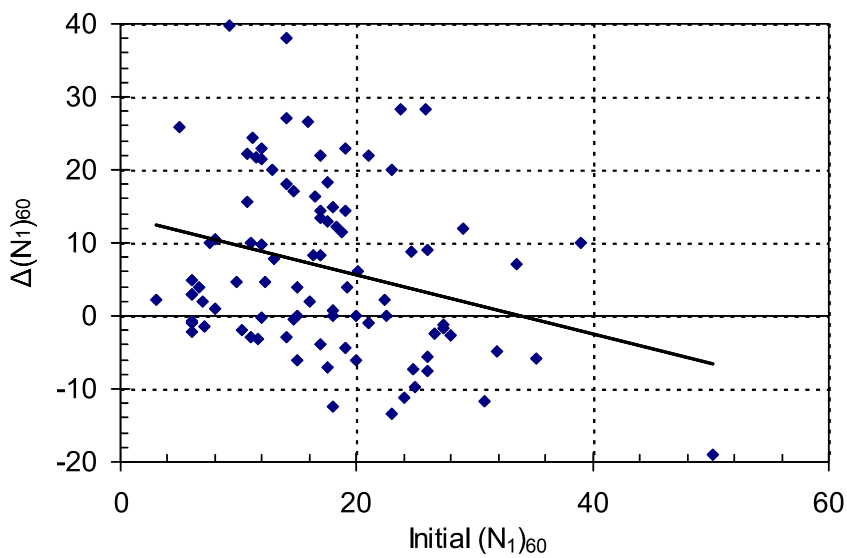


Figure 9-12 Initial $(N_1)_{60}$ versus $\Delta(N_1)_{60}$ with a linear trend line for the 9.3% A_r data at the Cherry Hill Bridge case history.

$\Delta(N_1)_{60}$ versus depth is presented in Figure 9-13 for the 9.3% A_r treatment areas. The data is scattered but has a slight trend showing increasing improvement to a depth of 20 ft beneath the surface at which point the improvement begins to decrease. The fines and clay profiles in Figure 9-4 do not appear to have any distinct trends that would explain this other than the fact that the fines content average from 20-25 ft was only 23%. The data from 20-50 ft would also be more likely to be in the center of the silty sand and sand layer and less likely to be influenced by the clay layers. The majority of the negative improvements are in the upper and lower regions of the soil profile where fines and clay content are higher. Some loss of improvement might be expected in the very top and bottom of the treatment zones but not to the extent that it is seen here. Decreased fines contents and the effect of increased improvement near the center of a liquefiable layer are thought to contribute to the trend; however, a full explanation of this trend is not known. It is possible that stone columns alone were not enough to ensure consistent positive improvements.

Initial $(N_1)_{60}$ versus final $(N_1)_{60}$ is presented in Figure 9-14 for the 9.3% A_r treatment areas. A logarithmic trend line that best represents the data is presented although the R-squared value for the trend line is only 0.1922. Due to the fact that post-treatment testing was only performed for potentially liquefiable layers, there are few initial blow counts above 30. There are a significant number of final blow counts below 20, especially for low initial blow counts. The trend line indicates that the final blow count is not expected to be above 20 if the initial blow count is less than 11. As mentioned previously, the low final blow counts were typically excluded from the minimum final blow count criterion due to high clay contents. If clay contents greater than 15% were not used as an exclusion criterion the treatment would not have passed in multiple locations and additional stone columns would have been necessary, as was the case for the area around boring 11. Data that did not meet the minimum criterion were also excluded in some cases if they were isolated low values; however, there was not consistent uniform improvement across the site and many of the final blow count values were low (<15).

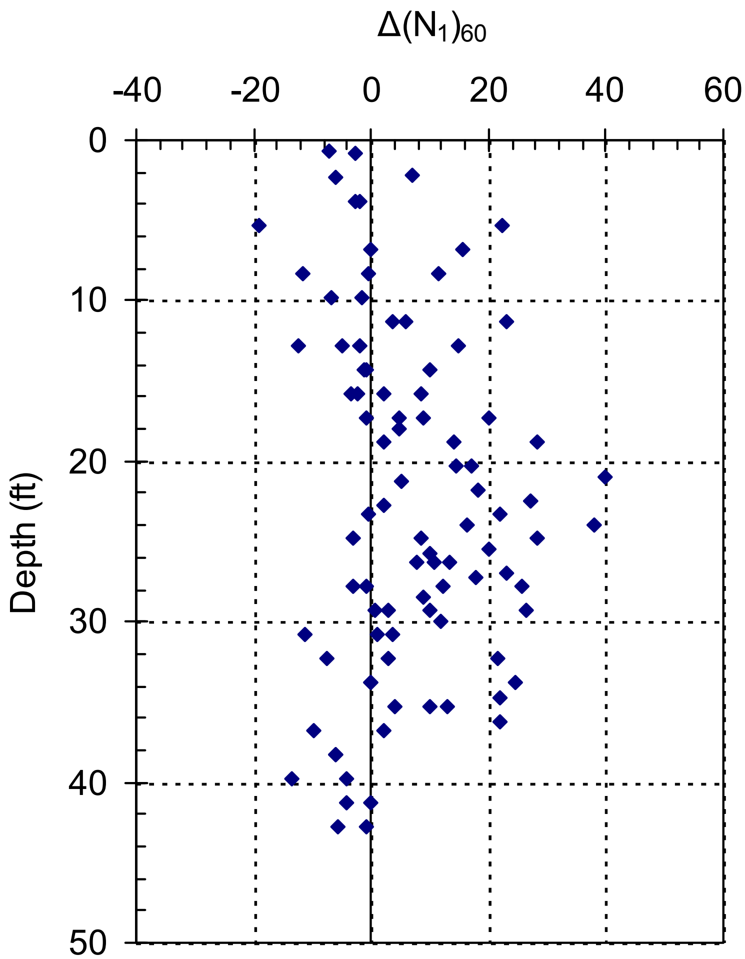


Figure 9-13 $\Delta(N_1)_{60}$ versus depth for the 9.3% A_r data at the Cherry Hill Bridge case history.

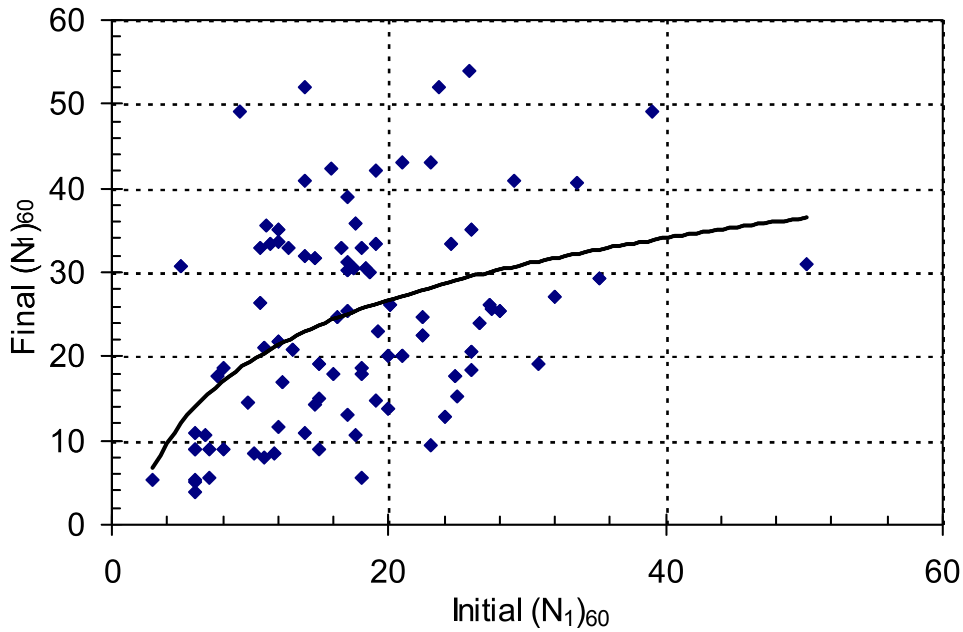


Figure 9-14 Initial $(N_1)_{60}$ versus final $(N_1)_{60}$ with a logarithmic trend line for the 9.3% A_r data at the Cherry Hill Bridge case history.

There was one section which was not accepted during the initial post-treatment testing, section 11. This section had additional stone columns installed and was then retested to evaluate performance. The results are presented in Figure 9-15 for treatment section 11 where the area replacement ratio was increased to 18.6%. The results for this section show mostly improvement with one section of negative improvement where the initial blow count was very high (49). There were two data points not meeting the minimum final clean sand blow count criteria, but the overall improvement was sufficient enough to accept the treatment. The increased area of replacement ratio generally led to good improvement with high final blow counts (≈30) although there were still several points with lower improvements.

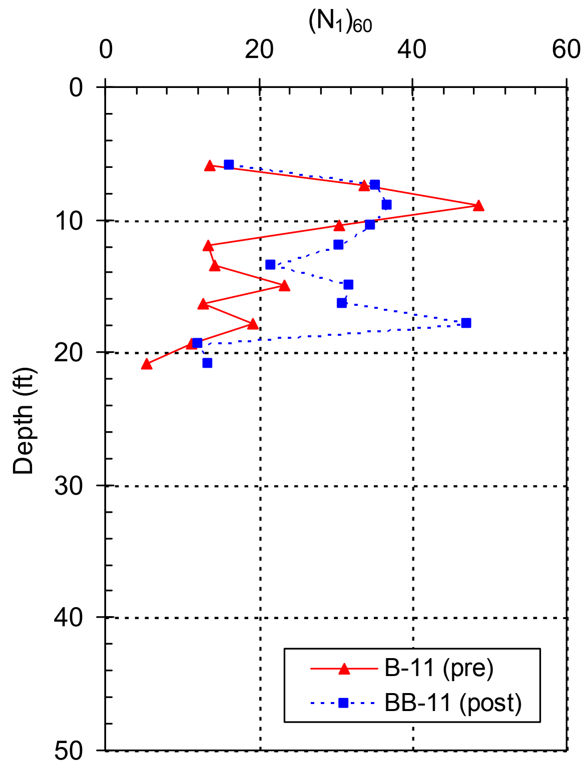


Figure 9-15 SPT test results for direct comparison data from the 18.6% A_r treatment sections 15-18 at the Cherry Hill Bridge case history.

Initial $(N_1)_{60}$ versus final $(N_1)_{60}$ is presented in Figure 9-16 for both the 9.3% A_r and the 18.6% A_r treatment areas. The 18.6% A_r data is from post-treatment testing following the addition of the secondary stone columns. Logarithmic trend lines that best represent the data are presented, although the R-squared value for the 9.3% A_r trend line is only 0.1922 and the R-squared value for the 18.6% A_r trend line is only 0.2453. Doubling the area replacement ratio only produced final blow counts that were about 3-5 blows higher than the 9.3% A_r treatment values. This is not a significant increase but there was only one value below 20 blows per foot for the final $(N_1)_{60}$ values which indicates a fairly successful treatment. The data for the section with secondary columns did not include initial blow counts less than 14.

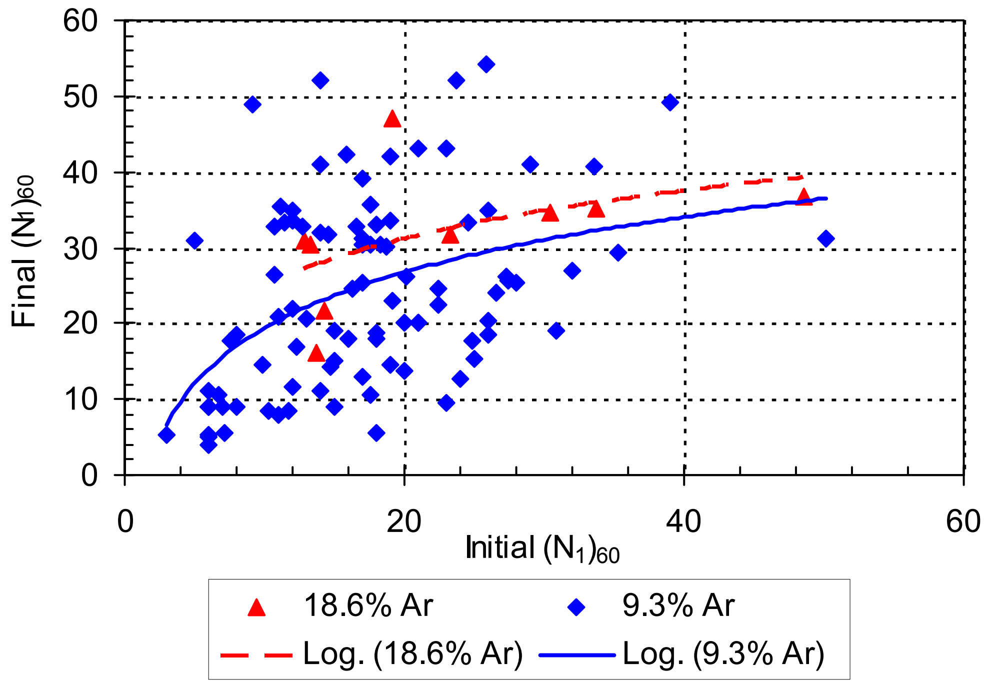


Figure 9-16 Initial $(N_1)_{60}$ versus final $(N_1)_{60}$ with a logarithmic trend lines for both the 9.3% A_r data and the 18.6% A_r data at the Cherry Hill Bridge case history.

A table of averages is presented in Table 9-1 for the different areas at the Cherry Hill Bridge case history. The main categories of interest are the silt and sand soil types although the averages for the entire site, including clay layers, is included to provide a comprehensive view of the site.

The area that required additional stone columns had lower fines content (27%) than the rest of the site (32%) but the initial values were much higher with a value of 23 versus 17 for the rest of the site. The average final blow count in the 9.3% A_r areas was only 24 compared to the 18.6% A_r areas where the final blow count was 32. This average value is tempered by the fact that the 18.6% A_r data did not have any low (<14) initial blow counts which accounted for much of the low (<20) final blow counts in the 9.3% A_r data. There were not very many data point available (9) for the 18.6% A_r areas when compared to the 9.3% A_r areas (92). The percent increase was similar for both sets of data (≈37%). The improvement was greater for the silt and sand data then the data with clays, as would be expected. The standard deviation of the final blow counts varied from 28% of the average final blow count for the 18.6% A_r category to 50-60% in the other two categories. The 18.6% A_r treatment had the least variation in results and the highest average final blow count.

Table 9-1 Table of average values for the Cherry Hill Bridge case history.

	Soil Type		
Properties and Statistical Measures	SM/ML	SM/ML	All Soils
Area Replacement Ratio, A_r (%)	9.3	18.6	9.3
Average Fines Content (%)	32	27	37
Average Clay Content (%)	10	8	11
Average Initial $(N_1)_{60}$	17	23	16
Average Final $(N_1)_{60}$	24	32	21
Average $\Delta(N_1)_{60}$	7	8	5
Average % Increase in $(N_1)_{60}$	38	36	30
Standard Deviation (Finals)	12	9	13
Sample Size	92	9	162

To compare the results for this case history to the published literature, final $(N_1)_{60-cs}$ values are plotted versus initial $(N_1)_{60-cs}$ and logarithmic trend lines are presented in Figure 9-17 The logarithmic trend lines are then compared to the curves developed by Baez (1995) for clean sands with fines contents less than 15%, and both are presented in Figure 9-18. The trend lines are not highly representative of the data due to scatter (R-squared values less than 0.21); however, by comparing them to the Baez clean sand curves, the effectiveness of the 9.3% A_r and the 18.6% A_r site treatment plans may be compared generally. The logarithmic trend line for the Cherry Hill Bridge site was developed from data with initial clean sand blow counts greater than 8-12 blows so the trend line that is compared to the Baez curve likewise begins at 8-12 blows.

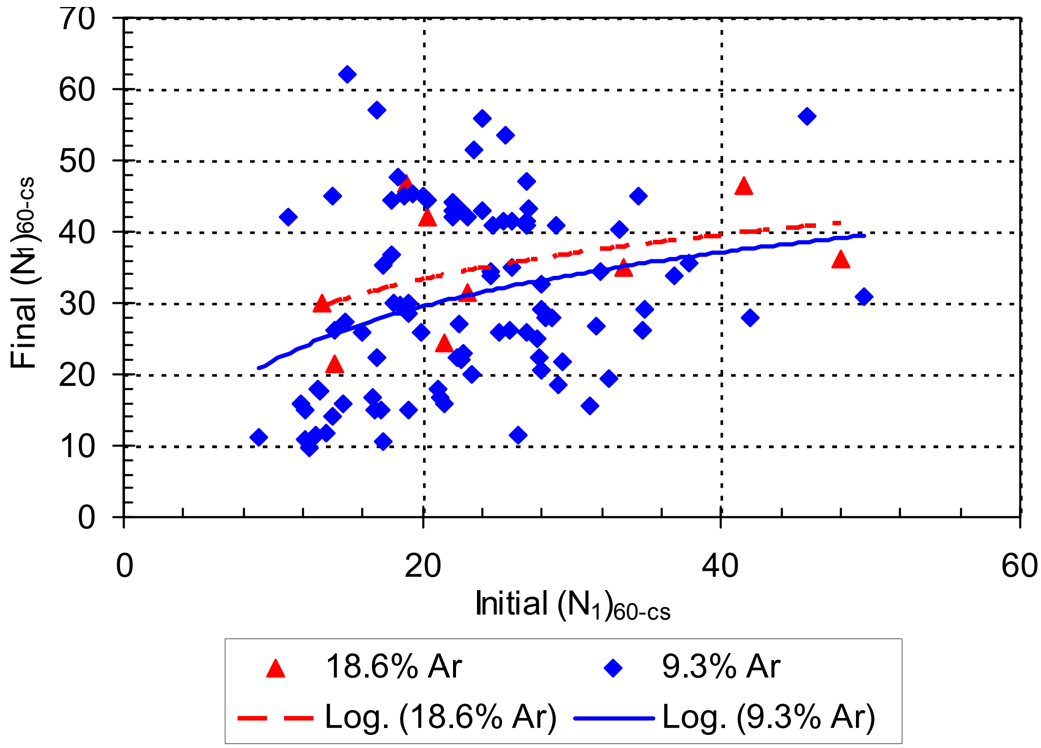


Figure 9-17 Initial $(N_1)_{60\text{-cs}}$ versus final $(N_1)_{60\text{-cs}}$ with a logarithmic trend lines for both the 9.3% A_r data and the 18.6% A_r data at the Cherry Hill Bridge case history.

The 9.3% A_r trend line showed average improvement fairly consistent with that of the clean sand 5% A_r curve. The average fines content for this data was 32%. The 18.6% A_r trend line showed improvement that was somewhere between the 5% A_r and 10% A_r clean sand curves, possibly about 7.5% A_r. The average fines content for this data was 27%. The trend lines are most representative of the average improvement at the site due to the scatter and they indicate that doubling the area of replacement was only about 50%

as effective as doubling the A_r at a clean sand site would be. The wick drains did not supplement the stone columns enough to counteract the negative effect of the fines content in either of the treatment areas. Sufficient improvement was attained to meet the project objectives, although there were still values not meeting the minimum criterion.

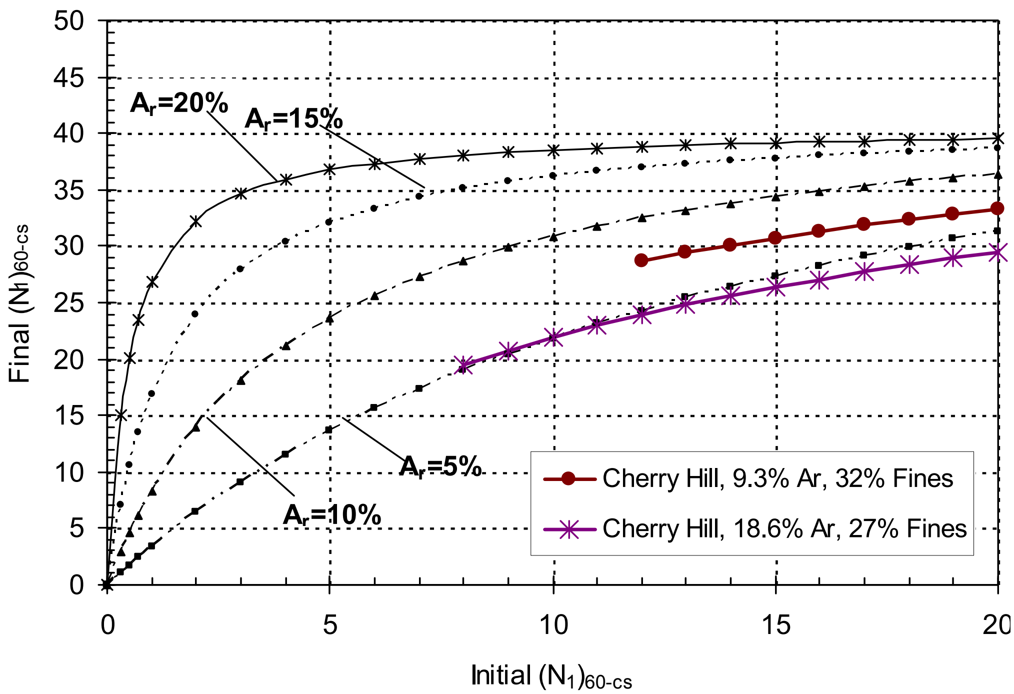


Figure 9-18 Initial $(N_1)_{60\text{-}cs}$ versus final $(N_1)_{60\text{-}cs}$ with the clean sand curves developed by Baez (1995) as well as the logarithmic trend line for the Cherry Hill Bridge case history silt and sand data.

9.4 Conclusions

- The data for the Cherry Hill Bridge case history were too scattered to perform reliable linear regression and the linear and logarithmic regression that was attempted yielded poor R-squared correlation values.
- The 9.3% A_r stone column treatment generally did not provide consistent, reliable increases in blow counts across the site where the average fines

content was 30% and the average clay content was 7%. Negative improvements were generally accepted on the assumption that there was enough clay in the sample or in the surrounding soil to prevent liquefaction. High clay contents (>15%) were used to exclude much of the data not meeting the minimum blow count criterion.

- On average, for the silty sands and sandy silts in the primary improvement areas (9.3% A_r), there was 38% improvement with final $(N_1)_{60}$ values of 24 blows per foot for soils with 32% fines content, 10% clay content, and 17 blows per foot initial blow count. The area with additional stone columns (18.6% A_r) had approximately the same improvement (36%) but had higher initial blow counts (23) and higher final blow counts (32).
- The site trends were consistent with the majority of the case histories with improvement decreasing due to high fines contents and high initial blow counts.
- The center of the silt and sand treatment layer performed better (little to no negative improvements) than the upper and lower boundaries of the layer.
- Fines contents greater than about 40% significantly limited improvement with improvements typically being less than 10 blows per foot. Variations in improvement were most pronounced for low fines contents. For fines contents less than about 35% there were as much as 50 blows per foot differences in improvement for data with similar fines contents.
- The 9.3% A_r treatment (32% fines) showed average improvement fairly consistent with that of the clean sand 5% A_r curve (<15% fines). Doubling

the area replacement ratio to 18.6% (27% fines) produced approximately 50% of the improvement that would be expected by doubling the clean sand A_r from 5% to 10%.

- Scatter in the data limited analysis and the development of a reliable design curve. Scatter was likely due to several factors that could not be evaluated in this study due to insufficient information. These factors may include changes in clay content, proximity of post-treatment testing to stone columns, proximity of pre-and post-treatment testing, plasticity index, and time elapsed between treatment and post-treatment testing.

10 Salmon Lake Dam, Salmon Lake, Washington

10.1 Site Overview and Conditions

The Salmon Lake Dam is located in north-central Washington adjacent to the town of Conconully, Washington (Figure 10-1). The dam was originally constructed in 1921 and consists of a 30 ft high zoned earth-fill embankment with a crest length of 1,260 ft. The case history studied here is that of the liquefaction mitigation efforts performed beginning in 1997.

The Bureau of Reclamation technical memorandum by Snorteland (2003) and an article from the Annual Conference Association of State Dam Safety Officials by Luehring, et al. (1998) were the main references for this case history. More detailed descriptions and analyses of the site may be found in either of these documents.

The design earthquake for the site was a magnitude 6.5 earthquake for a random event at a distance of 19 miles, with an annual probability of occurrence of $2x10^{-5}$ and a peak ground acceleration of 0.26g.

During the investigation stages, CPT and SPT testing were performed across the site to determine the liquefaction potential and soil characteristics at the site. The soil profile consisted of Quaternary fluvio-lacustrine sediments to depth of up to about 300 feet. The sediments were generally cohesionless, interbedded to laminated silty sand, with interbeds and lenses of sandy silt, silt with sand, poorly-graded sand, and silty sand with gravel. Over 80% of the pre- and post-treatment SPT data samples were silty sand (SM) or sandy silt (ML) with another 15% from poorly graded sand with silt (SP-SM). Considering the high percentage of silt and sands, the soil profile is essentially considered to be SM and ML layers and lenses. The presence of sand or gravel was relatively minor and all of the data points were included in this case history under the assumption that the occasional sand or gravel layers did not affect the overall results.

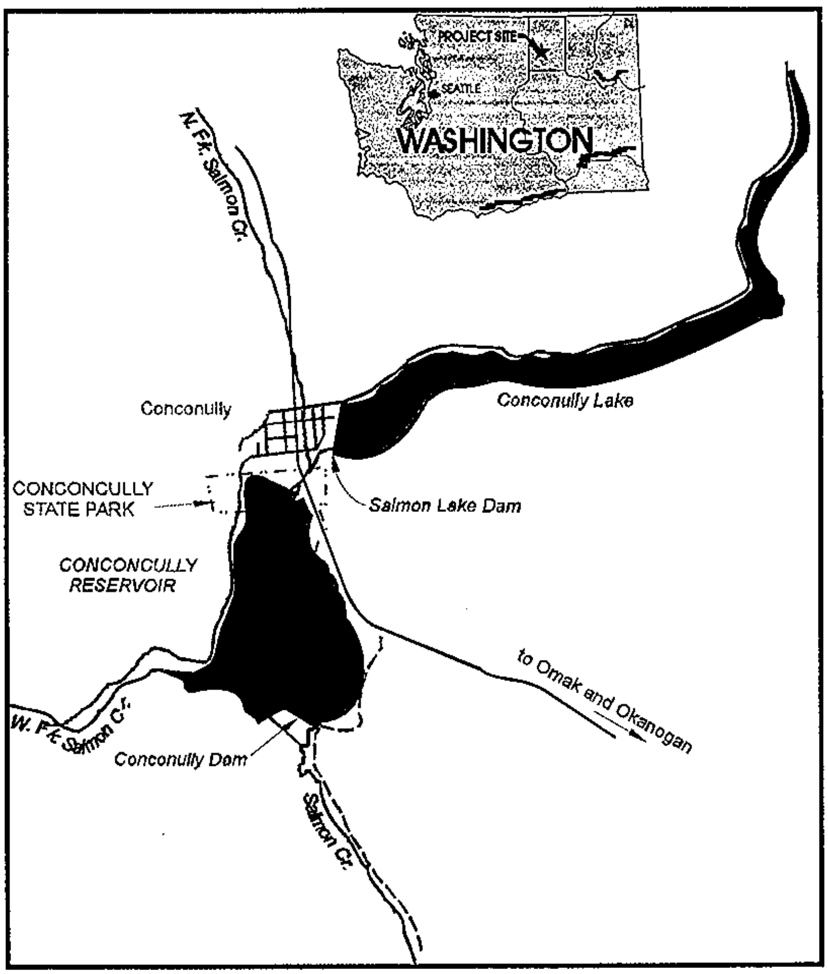


Figure 10-1 Location map of the Salmon Lake Dam case history near Conconully, Washington. (Leuhring, et al., 1998).

The full results of SPT and CPT testing were not available for this case history, only a subset of about 60 data points were used in this case history. The data available for this case history were fairly representative of the overall results; but, where possible, figures from the paper presented by Luehring, et al. (1998) are presented in order to maintain consistency with the overall site results. Unless otherwise noted, the results and averages presented will be those of the overall site as presented in the previously mentioned literature.

The silty sand (SM) layers had an average fines content of 49%, an average clay content of 5%, an average pre-treatment $(N_1)_{60}$ value of 17, and an average pre-treatment $(N_1)_{60\text{-}cs}$ value of 22. The silty sand (ML) layers had an average fines content of 65%, an average clay content of 11%, an average pre-treatment $(N_1)_{60}$ value of 12, and an average pre-treatment $(N_1)_{60\text{-}cs}$ value of 19. Clean sand SPT values were used at this site as a measure of improvement; however, non-clean sand values are also presented in this case history to provide a measure of comparison against the other case histories presented in this study.

A fines content profile is shown in Figure 10-2 with values taken from the available data. The fines contents shown here are slightly lower than the overall site fines contents but, similar to the overall site fines contents, they do not to show any noticeable trends with depth. The overall average fines content for the sight was higher than that shown in the figure with a value of about 43% fines.

Soil liquefaction was deemed a potential hazard at the site and mitigation efforts were required. Liquefiable deposits were encountered to depths of approximately 60 feet. Liquefaction analyses were performed using the "Simplified Procedure" developed by

Seed and Idriss (1971) with refinements presented at the 1996 NCEER workshop (Youd, et al., 1997). To prevent liquefaction, minimum post-improvement $(N_1)_{60\text{-}cs}$ values were determined throughout the soil profile. The minimum values varied according to the soil conditions within the soil profile and will be presented graphically later in this chapter.

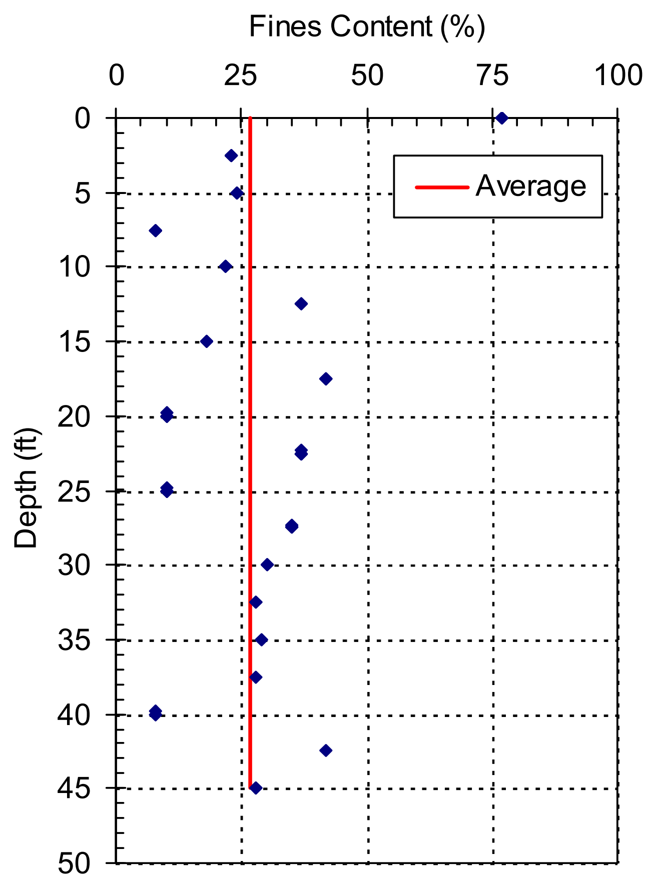


Figure 10-2 Fines content profile with a line indicating average fines content from the available data for the Salmon Lake Dam case history.

10.2 Treatment Methods

A test section was utilized to determine the appropriate layout and sequencing of stone column and wick drain installation. The treatment plan that was chosen for the site included using stone columns and wick drains in varying arrangements. The stone column diameters were 3.0 ft. for the inner columns while the columns on the edges were 3.75 ft in diameter. The stone columns were placed in an equilateral triangle arrangement with a typical center-to-center column spacing of 6 ft which resulted in an area replacement ratio of 22.7%. Wick drains were installed between most of the adjacent stone columns. Stone columns and wick drains were typically installed to depths of about 60 ft beneath the surface. A typical cross section of the Salmon Lake Dam and the stone column treatment is shown in Figure 10-3. The design and as-built treatment layouts are shown in Figure 10-4.

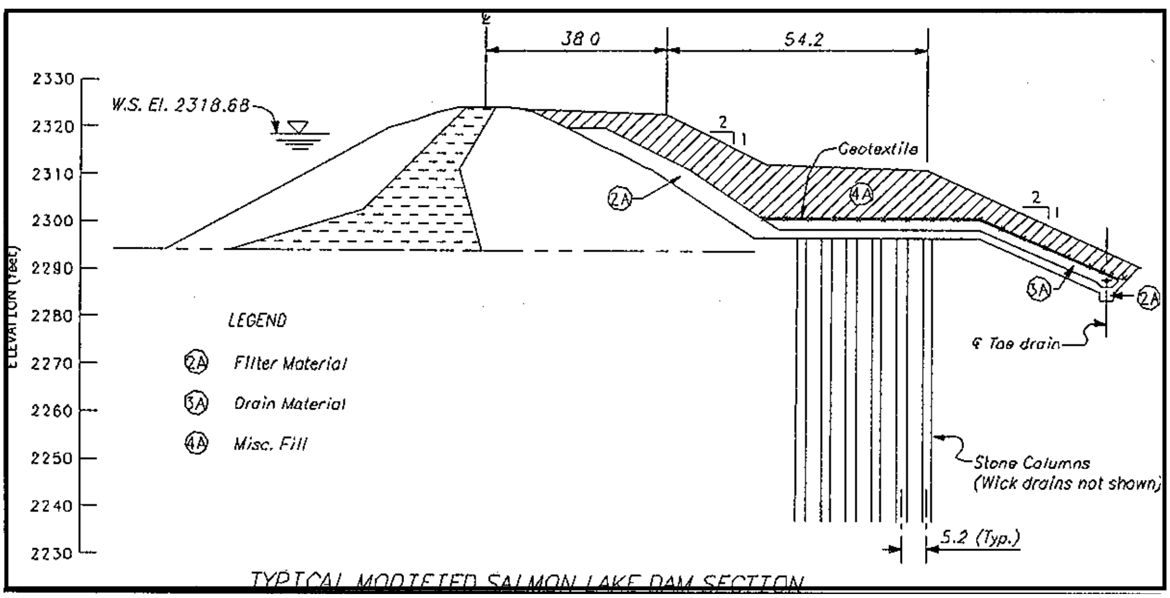


Figure 10-3 Typical cross section of the Salmon Lake Dam (Luehring et al, 1998).

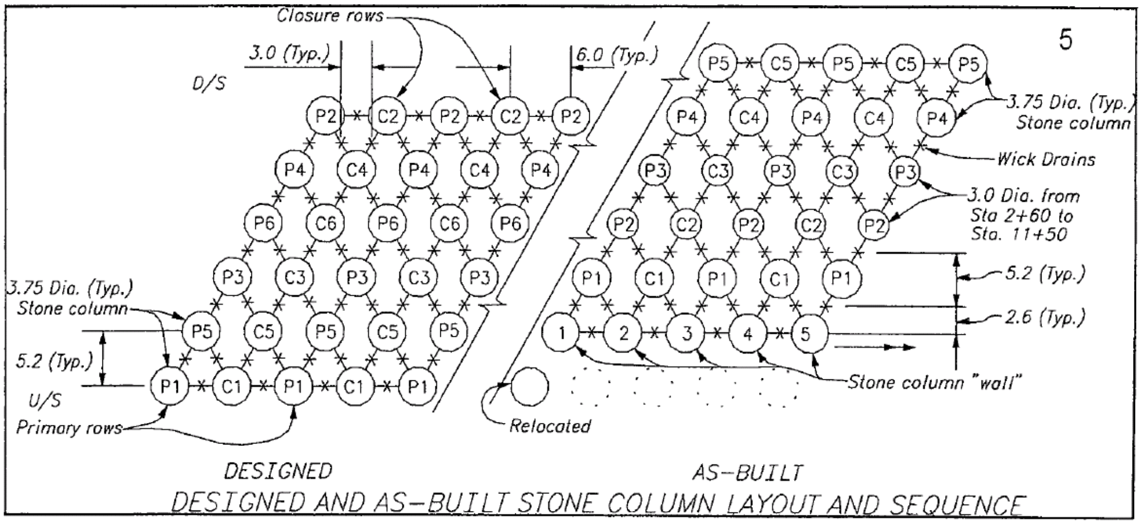


Figure 10-4 Designed and as-built stone column and wick drain treatment at the Salmon Lake Dam case history (Luehring et al, 1998).

10.3 Results and Analysis

During stone column installation, wick drains were observed to actively vent water and air beginning immediately following the start of installation and lasting until after completion of stone column installation. During installation of a single column, up to 40 wick drains were observed venting within an influence zone approximately 15 feet in diameter. This visual observation confirms that the drains definitely contribute to the relief of pore pressure in the soil.

During construction it was noted that the sequencing of the stone column installation was important in order to keep the pore pressures within acceptable levels. Several ideas were suggested and of those that were attempted, the final method was to use an advancing front of column installation with alternating rows in order to create closure columns. By installing a column and then moving onto another area before coming back to install the adjacent column (essentially skipping every other column in a row and then coming back to them once the end of the row is reached), the pore pressures were allowed to dissipate more than if this method were not used.

Closure columns are when a column is installed in between two previously installed stone columns and they are thought to increase the effectiveness of the treatment. The adjacent stone columns provide a confining effect that causes increasing densification of the closure column. The capacity of the equipment used to install the stone columns was important as well since installing closure columns required more effort than installing primary columns.

The post-treatment testing was generally done a minimum of two weeks after treatment and included both SPT and CPT testing. Some of the test holes were directly comparable to adjacent pre-treatment test holes while others were not. A plot of pre- and post-treatment SPT $(N_1)_{60\text{-}cs}$ versus depth is presented in Figure 10-5. The average pre-treatment and post-treatment values are shown using dashed lines and the solid line indicates the minimum final clean sand blow count criteria throughout the depth of the profile.

There is significant scatter in the data; however, the treatment was successful in improving the clean sand blow counts from an average initial value of 22 to an average final value of 40 for the data shown. This represents an increase of 82% which is a significant improvement. The non-clean sand blow counts increased from an average initial value of 17 to an average final value of 33. This is also a significant increase, a 94% improvement. The final blow count criterion was also met for almost all of the data, with a few minor exceptions which were not expected to be of concern in the design earthquake event. There was an increase in both the initial and final blow counts about halfway down the soil profile which Luehring, et al. (1998) attributed to reduced fines contents in these areas.

Fines content versus $\Delta(N_1)_{60}$ is plotted in Figure 10-6. The scatter is significant and the R-squared value for the accompanying logarithmic trend line is only 0.0078. A slight decrease in improvement is noticed with increases in fines content, as might be expected, but the data is too scattered to form any specific conclusions. These results may not be fully representative of the overall site though, due to the limited data set.

Initial $(N_1)_{60}$ versus $\Delta(N_1)_{60}$ is plotted in Figure 10-7. The linear trend line that is presented had an R-squared value of only 0.1756. The general trend is a decrease in improvement with increased initial blow counts. This data is believed to be fairly representative of the overall site results although some variation may exist.

Final $(N_1)_{60}$ versus $\Delta(N_1)_{60}$ is plotted in Figure 10-8. The logarithmic trend line shown in the figure has an R-squared value of only 0.1836 due to the scatter in the data. The data shows that final blow counts seem to level off as the initial blow count increases, which is attributed to the difficulty in improving soil that is already dense before treatment. There are a few low final blow counts (<16) that correspond to low initial blow counts (<10), but the majority of the final blow counts are above 16 blows per foot.

Final $(N_1)_{60\text{-}cs}$ versus $\Delta(N_1)_{60\text{-}cs}$ is plotted in Figure 10-9 with a logarithmic trend line. The scatter is significant and the R-squared value is only 0.1238. There are very few final clean sand blow counts below 20 and the trend seems fairly comparable to that non-clean sand trend observed in Figure 10-8.

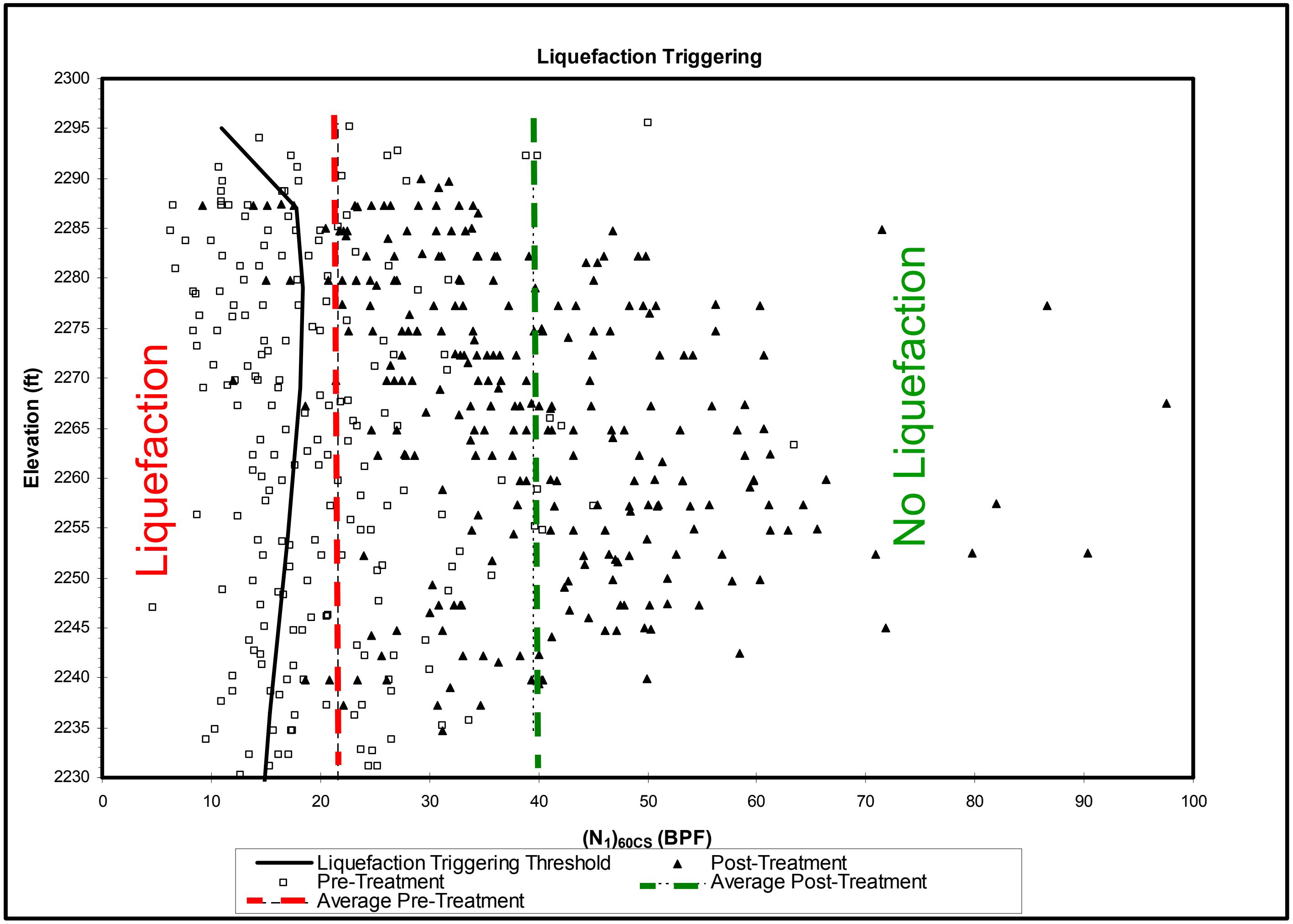


Figure 10-5 Salmon Lake Dam case history SPT clean sand results including average lines and a line representing the liquefaction triggering threshold (Luehring et al, 1998).

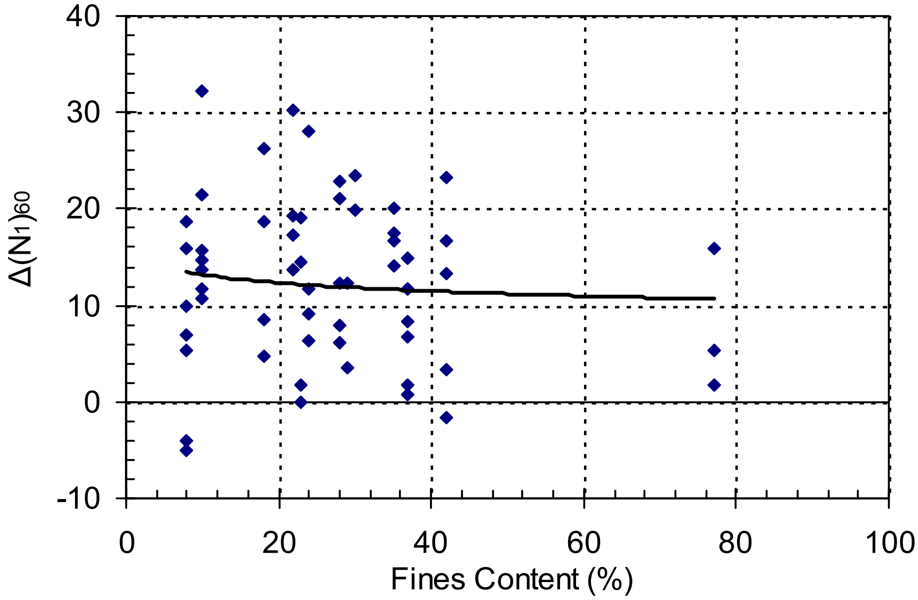


Figure 10-6 Fines content versus $\Delta(N_1)_{60}$ from the available Salmon Lake Dam case history with a logarithmic trend line.

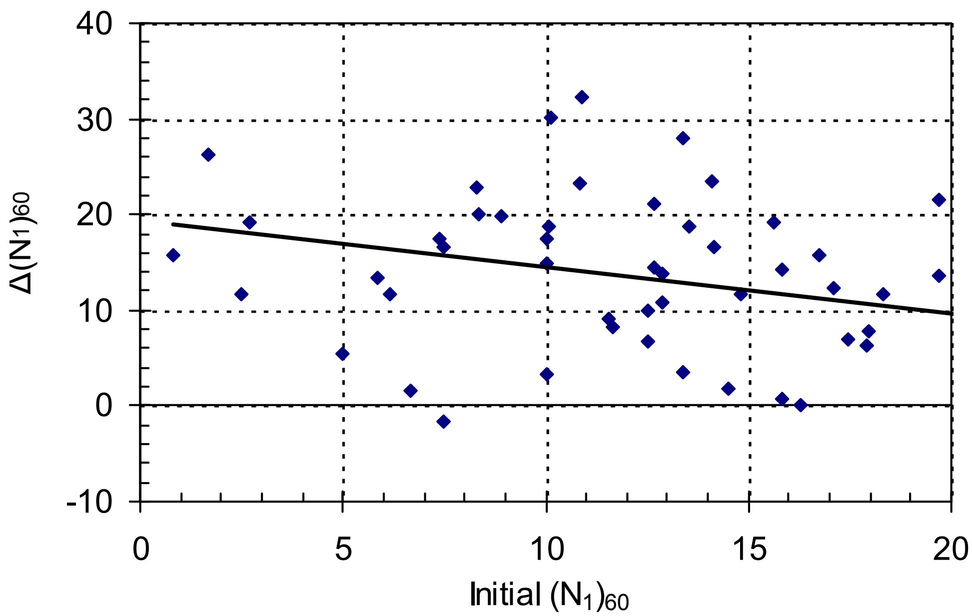


Figure 10-7 Initial $(N_1)_{60}$ versus $\Delta(N_1)_{60}$ with a linear trend line for the available Salmon Lake Dam case history data.

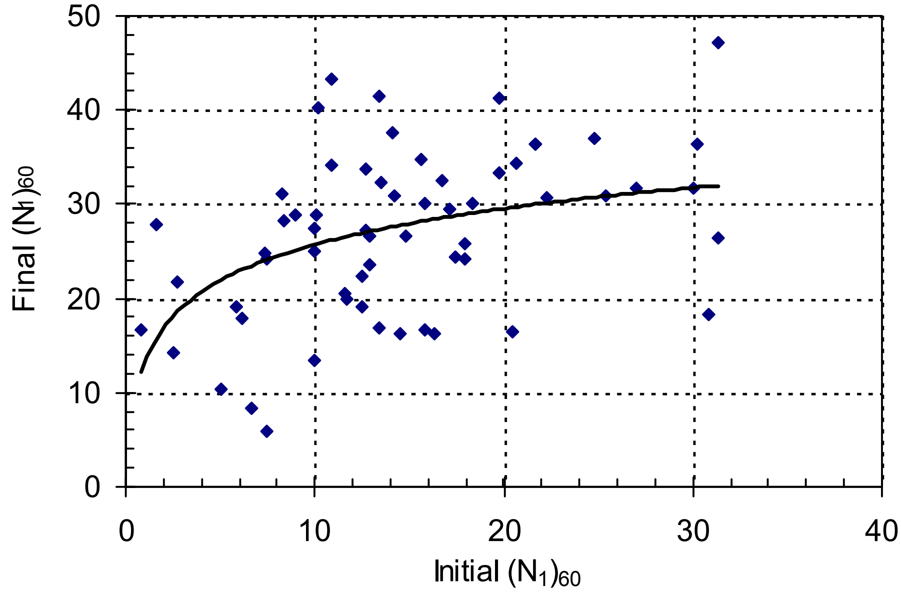


Figure 10-8 Final $(N_1)_{60}$ versus $\Delta(N_1)_{60}$ with a logarithmic trend line for the available Salmon Lake Dam case history data.

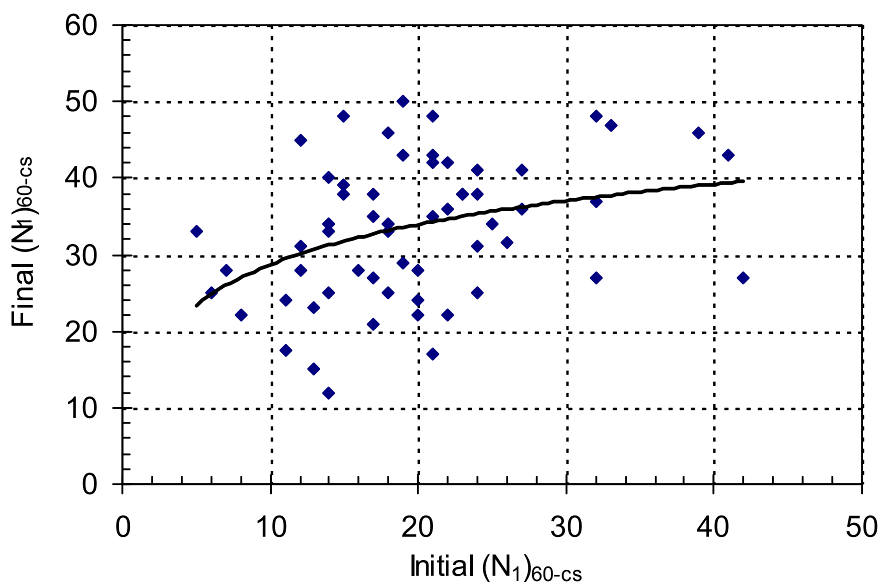


Figure 10-9 Final $(N_1)_{60\text{-cs}}$ versus $\Delta(N_1)_{60\text{-cs}}$ with a logarithmic trend line for the available Salmon Lake Dam case history data.

To investigate the possible causes of scatter, the soil samples were divided by soil classifications and the average values for each soil type are presented in Table 10-1. The majority of the data were silty sands (344 values), followed by silts (188 values). There were consistent improvements of 88-95% in all of the silts and sands and even greater improvement in the gravels (236%). The silts had an average fines content of 65%, the highest for any of the soil types, and the lowest final blow counts. The average final blow count for the silts was only 23 compared to 33 for the silty sands. The overall average for the site, when including all soil types and weighting them by their sample size, was 95% with an average final blow count of 33. The 22.7% A_r treatment with wick drains was successful in increasing the final blow counts by 95% in a site with an average fines content of 46% and predominately silty sand and sandy silt soil types.

Table 10-1 Average values, divided by soil type, for all of the Salmon Lake Dam case history data (Luehring, et al., 1998).

Soil Type	Ave. Fines Content (%)	Ave. Final Clay Content (%)	Number of Values	Ave. Initial $(N_1)_{60}$	Ave. Final $(N_1)_{60}$	Increase (%)
Silt (ML)	65	11	118	12	23	88
Silty Sand (SM)	49	5	344	17	33	95
Poorly-graded sand with silt (SP-SM)	10	2	86	21	40	92
Silty Gravel with sand (GM)	12	3	10	15	52	236

Despite the low correlation in results, the clean sand trend line for initial versus final blow counts in Figure 10-9 is used to compare the site results to the set of curves for clean sand (<15% fines content) developed by Baez (1995). Comparing the trend lines will allow the reader to visualize general trends in the data although scatter in the results is to be expected.

The Salmon Lake Dam case history performed roughly equivalent to a clean sand (<15% fines) site with an area replacement ratio of 10%. The area of replacement for the Salmon Lake site was 22.7% and the average fines content was 43%. The trend line for Salmon Lake was taken from the available data set, so it is possible that it may not be fully representative of the site, although it is believed to be roughly equivalent. If it were inaccurate, it is likely that it is under predicting the final blow counts since the average final clean sand blow count for the data set is only 33 (standard deviation of 9) compared to the average of 40 presented for the entire site (Luehring, et al., 1998).

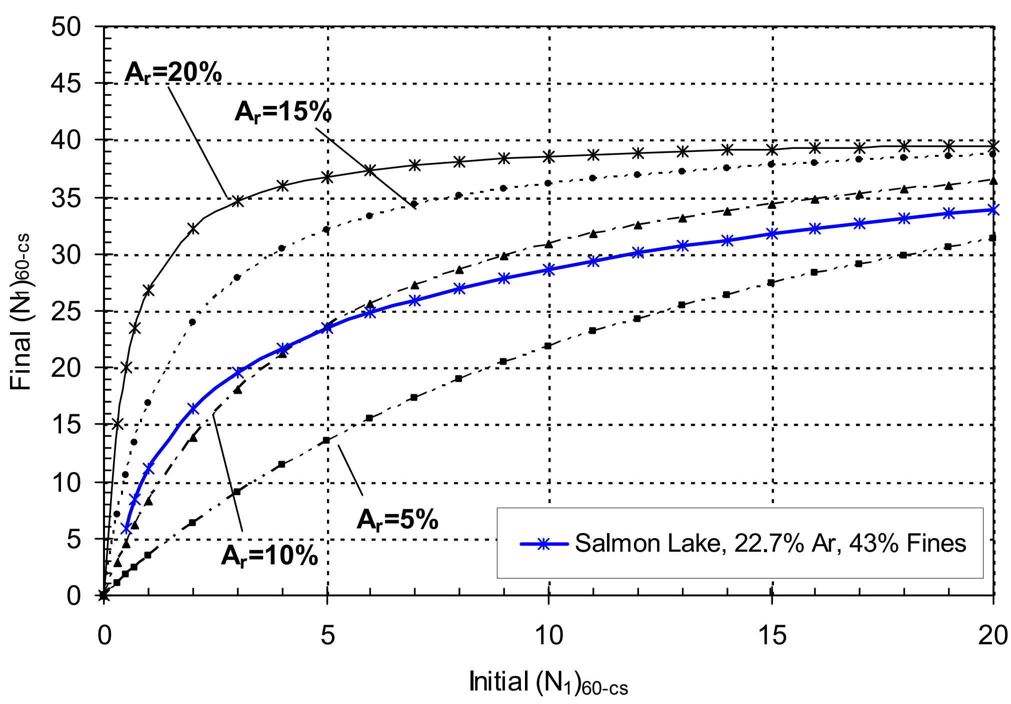


Figure 10-10 Salmon Lake Dam case history SPT clean sand results compared with clean sand (<15% fines) curves developed by Baez (1995).

10.4 Conclusions

- The Salmon Lake Dam treatment plan with wick drains and an area replacement ratio of 22.7% generally improved the site (43% fines, 6% clay) by 95% from an initial $(N_1)_{60}$ value of 17 to a final $(N_1)_{60}$ value of 32.
- The silty sand (SM) data performed better than the silt (ML) data with a slightly higher percent increase in $(N_1)_{60}$ (95% versus 88%) but with a significantly higher final $(N_1)_{60}$ value (33 versus 23).
- Wick drains actively relieved pore pressures within a 15 ft diameter influence area of a stone column during its installation, based on visual observation.
- Sequencing the stone column installation to allow pore water pressure dissipation as well as to allow the installation of closure columns was determined to be an important part of the method.
- On average, the stone column and wick drain treatment for the site (22.7% A_r) performed roughly equivalent to a 10% A_r stone column treatment at a site comprised of clean sand (<15% fines) when compared to the Baez (1995) curves.

11 Compiled Analysis of All Sites

11.1 Overall Analysis of All Sites

There are multiple factors which appear to affect the stone column and wick drain treatment in silts and sands at the case histories presented previously. Some of the factors which may affect the ability of the treatment to adequately improve the soil (as judged by either final blow count or increase in blow count) include but are not limited to the following: stone column area replacement ratio (A_r), arrangement of stone columns, number of drains per stone column (or drains/ft^2), very fine grained sand, fines content (%), clay content (%), plasticity of fines, pre-treatment soil density (initial $(N_1)_{60}$ value), time between completion of treatment and post-treatment testing, depth beneath the surface of treated soils (confining effect of soils at lower depths), interbedded soil layers, variations in the soil profile across the site, pore pressure buildup during installation, thickness of the liquefiable layer, sequencing of stone column and wick drain installation, and whether the treated soil is on the boundary of the treatment. The primary factors analyzed in this study include the stone column area replacement ratio, initial blow count, time after treatment before testing, fines content, and clay content. The other factors which were referenced have been discussed briefly although detailed and consistent analysis was not possible for these factors due to incomplete data or results.

Unfortunately, some of the factors mentioned are not typically measured uniformly across the soil profile, such as clay content, plasticity index, and fines content (especially when using CPT testing). Testing cost and time limit the number of CPT or SPT tests that can be performed, whether the full profile can be tested, and the frequency with which soil properties can be accurately measured. Further, testing accuracy is thought to be influenced by the following: proximity of testing to stone columns and drains, testing method accuracy, correlations applied to test results, and proximity of pre- and post-treatment testing (for direct comparison evaluations).

Due to the many factors which affect both the ability of the stone column and wick drain treatment to improve the soils and the accuracy of the test results, significant scatter has been observed in all of the case histories. In addition, some of the factors which may affect improvement were not available for this study (proximity of pre- and post-treatment test results and proximity of testing to stone columns and wick drains) although in practice they are easily measured. Other factors were not consistently measured (clay content, fines content, plasticity index, and time elapsed between treatment and post-treatment testing) which also limited analysis and understanding of results. The case histories studied all achieved the improvement required at the site so further testing to explain scatter in the results was not necessary and the additional costs were not warranted. As a result, much of the scatter in the case history data is unexplainable. The scatter in the data is discussed more thoroughly later in this chapter and the coefficients of variation are compared to the published literature. Average trends have been noted in the individual case histories and standard deviations of the final blow counts have been presented to quantify the scatter in the results.

Wick drains were observed to relieve pore pressures at several sites based on visual observations of water exiting the drains adjacent to the stone column being installed. Drains closer to the column being installed vented more water than drains further away. Specifically, drains up to 7.5 feet away from the column being installed were seen venting water at the Salmon Lake Dam case history, and drains up to 20 feet away were seen venting water at the Shepard Lane Bridge case history. Other visual observations were noted at the Marina del Rey and 24th Street Bridge case histories. The remaining case histories likely exhibited similar behavior but visual observations information was not available for these case histories. Wick drains contribute to pore pressure relief during stone column installation with varying influence based on the distance between the drain and the column being installed.

During construction at the Salmon Lake Dam case history it was noted that the sequencing of the stone column installation was important in order to keep the pore pressures within acceptable levels. An advancing front of column installation with alternating rows used to create closure columns was used to minimize excess pore pressures. By installing a column and then moving onto another area before coming back to install the adjacent column (essentially skipping every other column in a row and then coming back to them once the end of the row is reached), the pore pressures were allowed to dissipate more than if this method were not used. Closure columns are columns installed between two previously installed stone columns; they were thought to increase the effectiveness of the treatment. The adjacent stone columns provide a confining effect that causes increasing densification of the closure column. The capacity of the equipment used to install the stone columns was important as well since installing closure columns required more effort than installing primary columns.

A table of average soil properties and statistical measures are presented in Table 11-1 for each of the treatment zones at each of the case histories previously analyzed. Most of the values in the table have been previously referenced in the individual case history analyses. The case histories where noticeable layering was observed in the profile or where several different treatments were utilized have several different entries in the table to reflect these varying conditions and treatments. The values from the table will be referenced throughout the comparison of the different case history treatments.

Additional values which are found in the table and which were not presented previously include the number of drains per square foot, the distance from the drains to the columns, and the final blow count variance as a percentage of the final blow count mean. These additional values provide more details from which the effectiveness of the various treatments may be evaluated. The number of drains per square foot was calculated by dividing the area treated by a single stone column by the number of wick drains attributed to that stone column. There were 3 wick drains per stone column in all of the case histories except for the Salmon Lake Dam case history where there were only 2 wick drains per stone column. The spacing and arrangement of the stone columns determined the area attributed to each stone column. The distance from the drains to the columns was taken as the distance from a wick drain to the face of the nearest stone column. The variance as a percentage of the mean (coefficient of variation) gives some indication of the significance of the scatter in the data.

The time between treatment and post-treatment testing (time after treatment) were not included in the table since this information was only available for a few of the sites. The sites where time after treatment information was available for this study include the UPRR Bridge case history and the 24th Street Bridge case history. In some cases the time after treatment was noted to yield higher final blow counts; however, the increases were not uniform and decreased final blow counts are also to be expected. The 24th Street Bridge case history result indicate that testing should not be performed within 3 days of treatment, and preferably at least 6 days after treatment, in order to eliminate the need for multiple post-treatment tests. The UPRR Bridge case history results indicate that testing at least one week after treatment is suggested in order to allow low initial blow count data (blow counts of about 10) to see greater gains (up to 10 blow counts) with time. While consistent gains with time may not be observed, it is suggested that testing be performed no earlier than 6-7 days after treatment to allow for a general increase in final blow count average.

The clay content values were not included in the table due to limited data. The few sites where this data was recorded (24th Street Bridge and UPRR Bridge case histories) indicated that high clay contents (>15%) were typically associated with low final blow counts (<15) and negative improvements; however, some high clay content data were also included in data that exhibited high final blow counts and positive improvements in blow count.

Table 11-1 Summary of the properties and results from each of the case histories (silt and sand data only).

Site	A_r	Stone Column Arrangement	Number of Drains per ft^2*	Distance from Drain to Column (ft)	Category	Ave. % Fines	Average $(N_1)_{60}$				Sample Size	Standard Deviation of Final $(N_1)_{60}$	Coeff. of Variation (%)
							Initial	Final	Change	% Increase			
24th Street	26%	Equilateral Triangles	0.071	1.5	0-20 ft	29	18	45	27	148	67	18	40
					20-40 ft	40	18	30	12	69	55	15	50
			0	n/a	0-20 ft	27	19	26	7	35	20	10	38
					20-40 ft	32	21	23	2	8	8	10	43
UPRR	26%	Equilateral Triangles	0.071	1.5	10-16 ft	42	14	26	11	79	40	8	31
					16-20 ft	59	8	13	5	60	10	10	77
					Full Depth	46	13	23	10	77	50	10	43
Marina del Rey	11%	Squares	0.047	2.5	0-15% Fines	5	18	26	9	48	798	13	50
					15-50% Fines	40	9	8	-1	-11	379	3	38
					Full Depth	25	15	21	6	39	1177	14	67
Home Depot	7%	Squares	0.030	3.5	Full Depth	16	22	33	11	52	391	14	44
Silver Reef	15%	Squares	0.047	2.25	Full Depth	20	18	25	7	36	1003	17	68
Shepard Lane	27%	Equilateral Triangles	0.071	1.5	Full Depth	46	8	27	18	215	29	10	37
	34%		0.083	1.3	Full Depth	37	14	42	28	206	5	20	48
Cherry Hill	9%	Equilateral Triangles	0	n/a	Full Depth	32	17	24	7	38	92	12	50
	19%				Full Depth	32	23	32	8	36	9	9	28
Salmon Lake	23%	Equilateral Triangles	0.056	1.5	Full Depth	53	16	30	15	93	462	9**	30

*Zero indicates that drains were not used. **Salmon Lake standard deviation based on limited data set.

The values from the test section of the 24th Street Bridge case history are not shown in the table since they are presented previously in Chapter 3 and because the inclusion of the additional site data provided more accurate average values; however, the test section data provided a valuable direct comparison of stone columns with and without wick drains in close proximity. In the test section, the areas with drains had fewer negative improvements than the areas without drains (7% versus 32%), fewer post-treatment $(N_1)_{60}$ values below the minimum site criterion of 18 (11% versus 25%), and a greater average increase in $(N_1)_{60}$ following testing (94% versus 27%). The standard deviations of the post-treatment blow counts were typically about 40% of the average final blow counts.

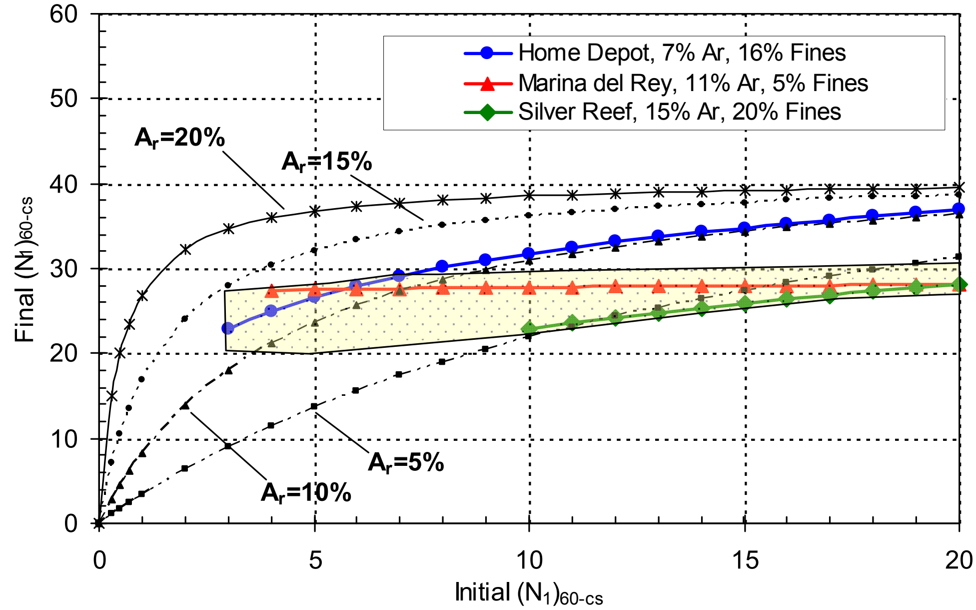
An additional comparison is available for the 24th Street site where the stone column diameter was increased resulting in a 34% A_r. Increasing the stone column diameter (34% A_r) instead of adding wick drains to the primary stone column treatment plan (26% A_r) proved less effective (10 blow counts lower on average) than the addition of wick drains for a low initial blow count of 10 while improvement was comparable at an initial blow count of 24. These improvements are also notable since the areas with drains had higher average fines content (35% versus 29%) and lower average pre-treatment blow counts (17 versus 20), both of which typically limit improvement. The Shepard Lane Bridge site provides a comparison where the A_r was also increased to 34% (by decreasing stone column spacing) but where the wick drains were not eliminated, as was the case at the 24th Street Bridge site. At Shepard Lane, the 34% A_r areas provided similar percent increases in blow counts as the 27% A_r areas (206% versus 215%); however, the average fines content was higher in the 34% A_r areas (46% versus 37%), and the initial blow count average was significantly lower (8 versus 14) so the increase in A_r is thought to have been effective in producing greater improvements. The 24th Street and Shepard Lane Bridge site data indicate that, on average, increasing the area replacement ratio may provide similar results as those obtained by leaving the area replacement ratio the same and adding wick drains to the treatment plan.

Figure 11-1 presents initial $(N_1)_{60\text{-cs}}$ versus final $(N_1)_{60\text{-cs}}$ for the 5-20% fines content category. A general improvement range which may be expected for 5-20% fines and 11-15% A_r is indicated by the highlighted area on the figure. A symbolic equation is also shown which represents the expected improvement range. The trends are not consistent with what might be expected although the trends are similar to average values and the exact shape of the trend line is not necessarily accurate.

The Silver Reef case history performed the worst with improvement comparable to a 5% A_r clean sand. The Silver Reef fines content was 20% and the area replacement ratio was 15%. The average initial $(N_1)_{60}$ value was 18 which is relatively high and which may account for some of the reason why the improvement was only that of a 5% A_r clean sand. The increase in fines content is also a likely factor contributing to decreased improvement. The Marina del Rey site had much lower fines content (5% average fines) with a slightly lower area replacement ratio (11%) and it performed similar to a 10% A_r clean sand in the lower initials and a 5% A_r clean sand in the higher initial blow counts. The Home Depot site performed exceptionally well with improvement similar to a 10% A_r clean sand while the site A_r was only 7.1%. The fines content was close to that of the clean sands. The wick drains were able to increase the performance of the columns to from a 7% A_r to a 10% A_r. Due to the variation in results, a conservative boundary is loosely drawn which may represent potential improvement in a 15-20% fines site with a 11-15% A_r. The coefficients of variation for the final values in this region were 50-68% so significant scatter is to be expected.

Figure 11-2 presents initial $(N_1)_{60\text{-cs}}$ versus final $(N_1)_{60\text{-cs}}$ for the 25-28% fines content category. The coefficients of variation of the final $(N_1)_{60}$ values from which the results in the figure were ultimately (following clean sand corrections) derived range between 38 and 67%. The 25-28% fines content curves seem fairly reasonable. The Cherry Hill site did not have drains but since it had a fines content of 27% it was expected that it would perform worse than the clean sand equivalent. The 18.6% A_r Cherry Hill curve is similar to that of a 5% A_r clean sand curve. An increase of about 10% fines content enough to decrease improvement significantly from what would be expected using the clean sand curves. On the other hand, the Marina del Rey site only had an A_r of 11% but its improvement was somewhat similar to the Cherry Hill site; however, the Cherry Hill site did have higher initial blow counts (23 versus 15) and a lower final blow count coefficient of variation (28% versus 68%). The 24th Street curve was definitely the best with very high final blow counts. The area replacement ratio was high, 26%, and in many cases a lower area replacement ratio might be adequate; however, the improvement was clear. The final blow count coefficient of variation for the 24th Street site was 40% and the average initial blow count was 18. The 24th Street site also had a much higher number of drains per square foot, 0.071 versus 0.047 for the Marina del Rey site. For fines content of 25-28% if the desired average improvement is in the 20-25 final $(N_1)_{60\text{-cs}}$ range then either an 18.6% A_r without drains or an 11% A_r with drains might be reasonable; however a 26% A_r with drains will likely guarantee an average final $(N_1)_{60\text{-}cs}$ value at or above 50 blows per foot.

Figure 11-3 presents initial $(N_1)_{60\text{-}cs}$ versus final $(N_1)_{60\text{-}cs}$ for the 32-37% fines content category. The coefficients of variation of the final $(N_1)_{60}$ values from which the results in the figure were ultimately derived were all approximately equal to 50%. The 32-37% fines content range produced curves that seem consistent with increases in final blow counts resulting from increased A_r and the addition of wick drains to the stone column treatment plan. The Cherry Hills 9.3% A_r treatment without drains had results similar to those of about a 7% A_r in clean sand without drains. The 24th Street 26% A_r treatment with drains produced results greater than a 20% A_r clean sand without drains and with final clean sand blow counts about 10 blows higher than the 20% A_r clean sand curve. The 33.9% A_r treatment with drains at the Shepard Lane also produced very high results, about 15 blows higher than the 20% A_r clean sand curve. A rough outline of potential results for a treatment plan with an A_r greater than 25% and with drains for a site with an average fines content of 32-37% is shown. If the high area replacement ratios (26-34%) and high number of drains per square foot (0.071 for 24th Street and 0.083 for Shepard Lane) are utilized then these results suggest average final clean sand blow counts greater than 40. The equation shown beneath the plot will be reference later as a comparison to the improvement from case histories with average fines contents of 40-46%.



5-20 % Fines + 11-15% A_r + Drains → Improvement ≈ clean sand 5-10% A_r

Figure 11-1 Initial $(N_1)_{60\text{-}cs}$ versus final $(N_1)_{60\text{-}cs}$ logarithmic trend lines for the case histories with 5-20% fines content compared with the clean sand (<15% fines) curves developed by Baez (1995). A symbolic equation comparing the expected improvement to the clean sand curves is also shown.

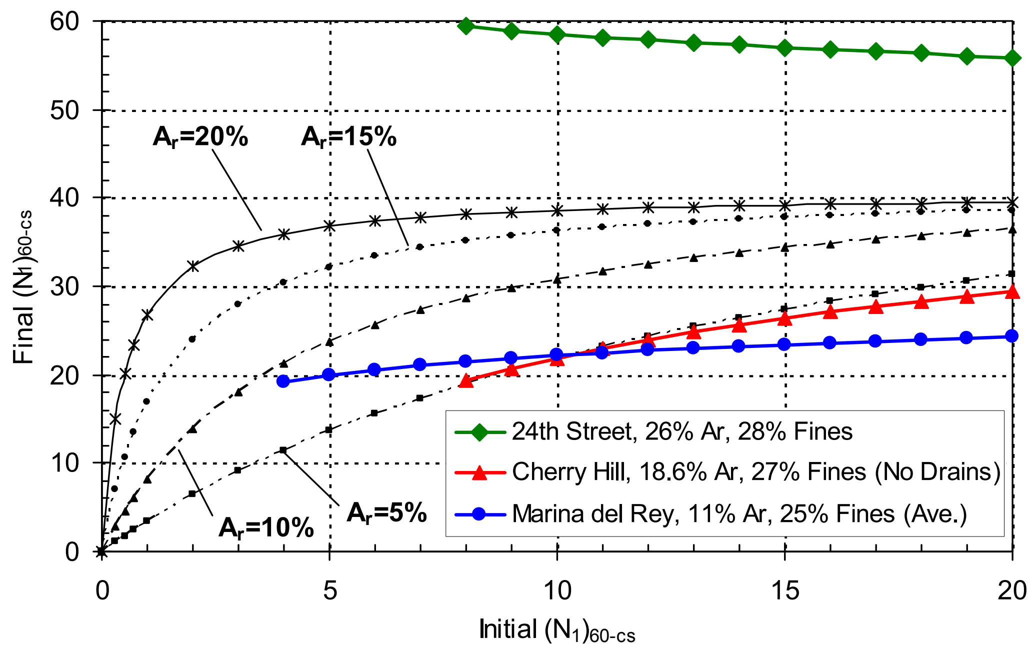


Figure 11-2 Initial $(N_1)_{60\text{-}cs}$ versus final $(N_1)_{60\text{-}cs}$ logarithmic trend lines for the case histories with 25-28% fines content compared with the clean sand (<15% fines) curves developed by Baez (1995).

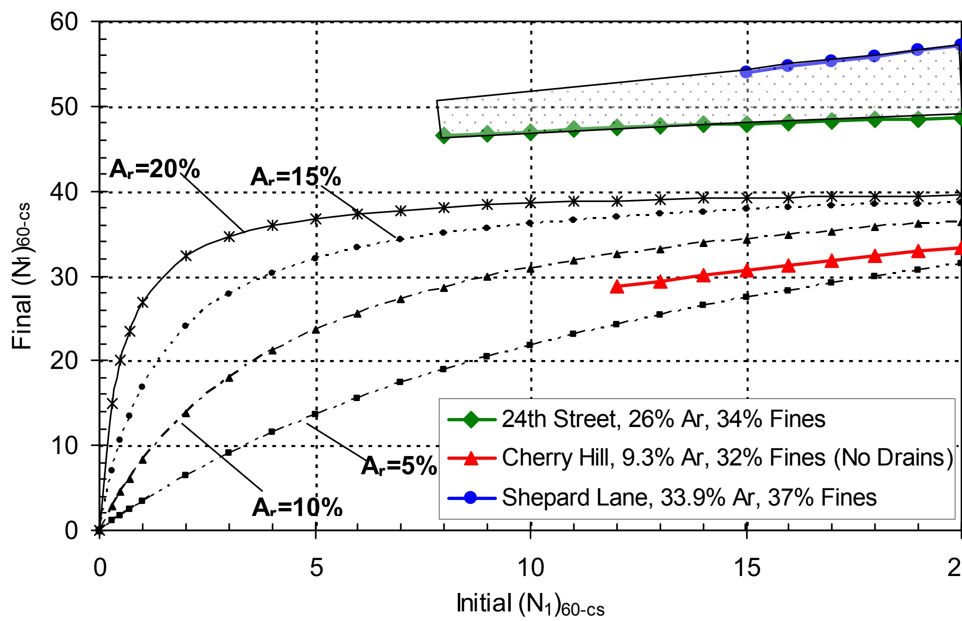


Figure 11-3 Initial $(N_1)_{60\text{-}cs}$ versus final $(N_1)_{60\text{-}cs}$ logarithmic trend lines for the case histories with 32-37% fines content compared with the clean sand (<15% fines) curves developed by Baez (1995).

Figure 11-4 presents initial $(N_1)_{60\text{-}cs}$ versus final $(N_1)_{60\text{-}cs}$ trend lines for the 40-46% average fines content case histories category. A shaded region is presented to represent typical results for the 40-46% average fines content case histories. The coefficients of variation for the final $(N_1)_{60}$ values from which the results in the figure were ultimately derived ranged from 30% at the Salmon Lake site to about 40% at the UPRR and Marina del Rey sites and 50% at the 24th Street Site.

The symbolic equations for the 5-20% average fines content case history results as well as the 40-46% case history results are shown below in Figure 11-5. It is seen that both treatments achieved similar results which were both approximately comparable to the clean sand 5-10% A_r treatment curves presented by Baez (1995). In higher fines content the area replacement ratio must be increased to maintain improvement.

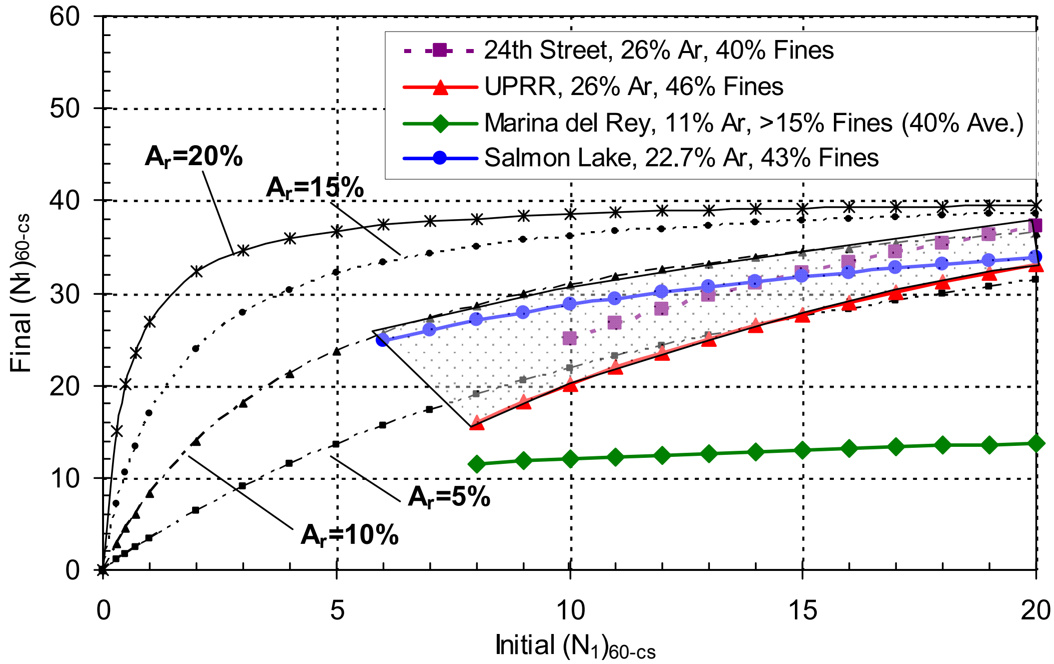


Figure 11-4 Initial $(N_1)_{60\text{-cs}}$ versus final $(N_1)_{60\text{-cs}}$ logarithmic trend lines for the case histories with 40-46% fines content compared with the clean sand (<15% fines) curves developed by Baez (1995).

5-20 % Fines	+ 11-15% A_r	+ Drains →	Improvement	≈ clean sand 5-10% A_r
40-46% Fines	+ 23-26% A_r	+ Drains →	Improvement	≈ clean sand 5-10% A_r

Figure 11-5 Symbolic equations which compare the treatment plans necessary to achieve improvement similar to clean sand sites with 5-10% A_r at sites with average fines contents of 5-20% and 40-46%.

The 22.7% A_r treatment at the Salmon Lake site performed exceptionally well with results that were typically higher than the UPRR and 24th Street sites, except at initial blow counts greater than 15. The 40-46% fines contents decreased improvement significantly although wick drains and area replacement ratios of about 25% were sufficient to raise the final clean sand blow counts to a 5-10% A_r clean sand final clean sand blow count range. In contrast, the 11% A_r Marina del Rey site with drains was not successful in even producing results similar to a 5% A_r clean sand site without drains.

This comparison is shown more clearly in Figure 11-6 with symbolic equations comparing the Marina del Rey improvement to that of the remaining 40-46% fines content sites being shown at the bottom of the figure. For high fines contents the area replacement ratio must be closer to 25% based on the results observed from the case histories.

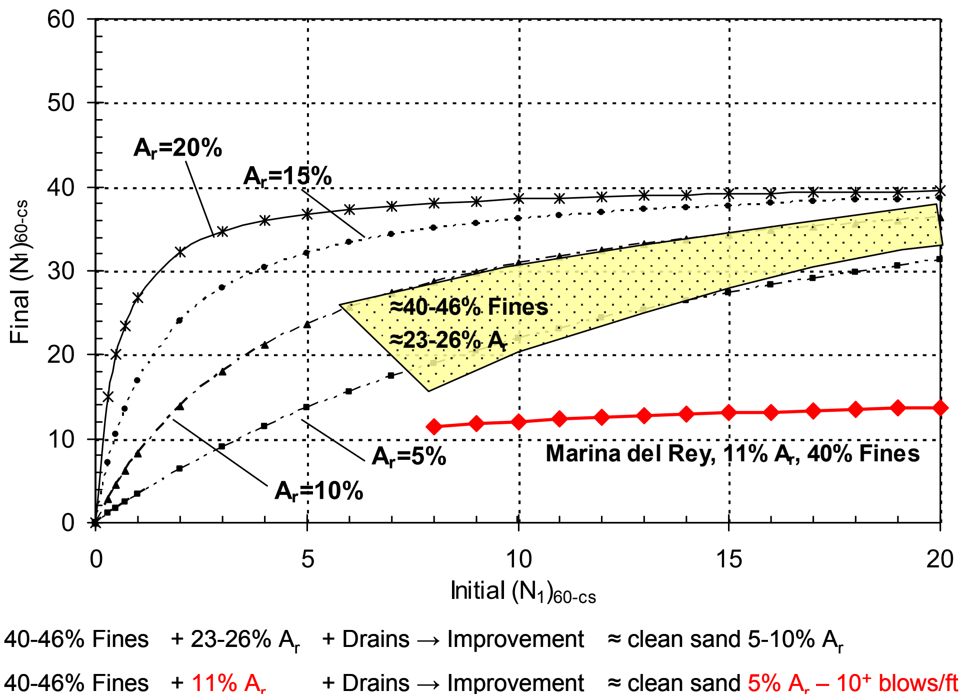


Figure 11-6 Initial $(N_1)_{60-cs}$ versus final $(N_1)_{60-cs}$ expected improvement for case histories with 40-46% fines content compared with the clean sand (<15% fines) curves developed by Baez (1995) and the Marina del Rey case history trend line. Symbolic equations representing improvement are shown.

Figure 11-7 shows final clean sand blow count versus initial clean sand blow count expected improvement for sites with 40-46% average fines content and 23-26% A_r compared to the Baez clean sand curves and the 24th Street case history overall site improvement trend line. Symbolic equations comparing the treatment plans to the clean sand equivalents are also shown. It is seen that for similar area replacement ratios, the improvement was significantly greater at the 24th Street site where the average fines content was only 34% as compared to 40-46% average fines content.

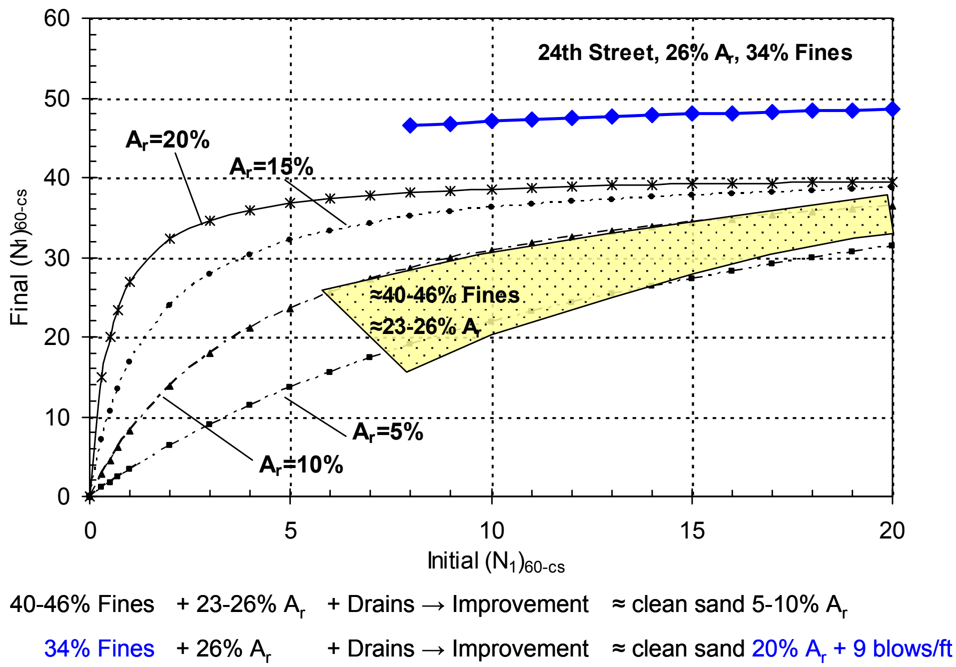


Figure 11-7 Initial $(N_1)_{60\text{-cs}}$ versus final $(N_1)_{60\text{-cs}}$ expected improvement for case histories with 40-46% fines content compared with the clean sand (<15% fines) curves developed by Baez (1995) and the 24th Street average case history trend line.

A summary of the equations presented for the different treatment plans is shown in Figure 11-8. The first two equations show what is necessary to obtain equivalent 5-10% A_r clean sand improvement in areas with fines contents of 5-20% as well as 40-46%. The third equation provides a comparison to the 40-46% fines content sites where a decreased A_r was used. The fourth equation provides a comparison for the 23-26% A_r treatment plans where the fines content was decreased. These have been discussed previously; however, they are provided all together for ease of comparison.

5-20 % Fines	+ 11-15% A_r	+ Drains → Improvement	≈ clean sand 5-10% A_r
40-46% Fines	+ 23-26% A_r	+ Drains → Improvement	≈ clean sand 5-10% A_r
40-46% Fines	+ 11% A_r	+ Drains → Improvement	≈ clean sand 5% A_r – 10^{+} blows/ft
34% Fines	+ 26% A_r	+ Drains → Improvement	≈ clean sand 20% A_r + 9 blows/ft

Figure 11-8 Symbolic equations which compare several of the treatment plans used at the case history sites to the comparable improvement from clean sand sites (Baez, 1995).

The case history results typically showed similar trends where improvement ($\Delta(N_1)_{60}$) decreases as the fines content increases or as the initial blow count increases. The exceptions to these trends were the UPRR Bridge and Shepard Lane Bridge case histories where increasing initial blow counts did not affect improvement and the San Pedro Home Depot case history where it was observed that increasing fines contents yielded increased improvement.

The UPRR Bridge and Shepard Lane Bridge case histories exhibited no trend between increasing initial blow counts and improvement. The UPRR Bridge trend is not fully explainable, although it is thought that the interbedded nature of the soil profile may have contributed to varying results. The interbedded clay layers may decrease the effect of improvement on the high initial blow counts which would have flattened the trend line and made it appear that improvement was independent of initial blow counts. The Shepard Lane Bridge case history trend was thought to be due to the fact that there were no high (>18) initial blow counts. Typically the decreasing improvement is seen as the initial blow counts increase but, since the data was limited, there were not any high initial blow counts which may have shown this decrease in improvement. Generally, increasing initial blow counts limit improvements in blow count.

The San Pedro Home Depot case history fines content and improvement trend is directly contrary to the trends observed in all of the other case histories. It is possible that the correlations used to derive the apparent fines contents may have contributed some to inaccurate results. It was shown in the case history analysis (Chapter 6) that linear regression results indicated that if the effect initial blow counts were removed from the data then the fines contents did not have any significant effect on improvement; however, the reason for this trend is not fully understood.

To further analyze the effect of increasing fines content on improvement, fines content versus improvement trend lines are shown in Figure 11-9 for the case histories with 23-27% A_r. The case histories with 23-27% A_r had the most comparable trend lines which is why they are presented in Figure 11-9. Average initial blow counts for each case history are indicated on the figure. A general improvement region is shaded to indicate how potential improvements are expected to vary with increasing fines contents for 23-26% A_r sites with drains. The R-squared values for the trend lines were all less than 0.16 which indicates that there was significant scatter in the results. The scatter in the results can be seen in the individual case histories which were presented previously.

The trends typically show significant decreases in improvement with increasing fines contents. Lower average initial blow counts typically result in lower improvement, which is apparent when comparing the Shepard Lane and UPRR trend lines. The exceptions are the 24th Street case history and the Salmon Lake case history. A probable reason for the high improvements in the 24th Street case history is that the overall average fines content for this site was 34% compared to 46% for the Shepard Lane and UPRR sites and it is thought that overall improvement would be increased due to lower average fines contents. The fines content is already accounted for in the figure; however, the average fines content at the site might indicate that a soil with low fines content is more likely to be adjacent to a soil with high fines content which would likely limit improvement. The other exception to increased average initial blow counts limiting improvement is the Shepard Lane case history. The data set for this site was limited and may not be fully representative of the site, thus its results are considered less significant than those of the other case histories.

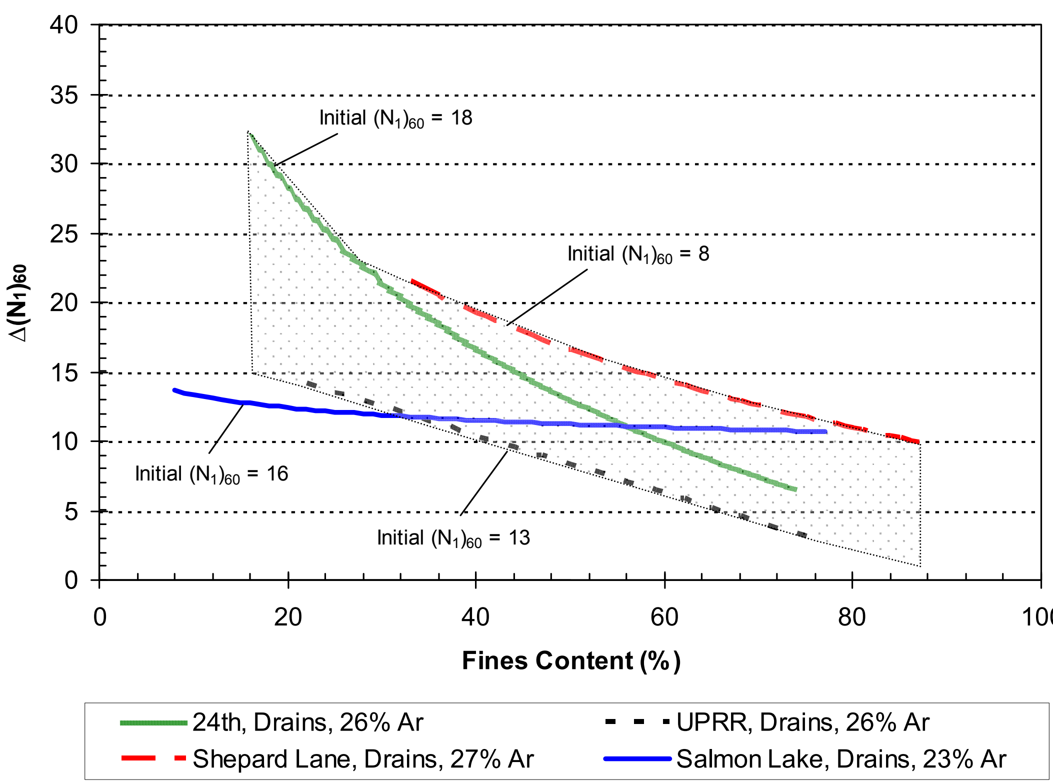


Figure 11-9 Fines content versus $\Delta(N_1)_{60}$ trend lines from the 23-27% A_r case histories with drains (average fines contents at the sites varied from 34 to 46%). Average initial blow counts for each case history are indicated on the figure. A general improvement region is outlined for potential stone column and wick drain sites with 23-27% A_r.

The shaded region in the Figure 11-9 indicates a general range showing how improvement is expected to vary with fines contents within sites with average fines contents of 34-46% and a treatment plan with 23-27% A_r and drains. The range is general and scatter is to be expected; however, the general trend is useful to engineers considering using similar treatment plans at sites with similar fines contents and initial blow counts. Fines contents often vary across a site and the data shows that improvement across the site will decrease as the fines contents increase. Improvement generally ranging from 14-29 blows per foot is expected when fines contents are close to 20% while improvement generally decreases to 2-11 blows per foot for fines contents close to 80%. The lower range of improvement is expected when the initial blow counts are high or the average fines contents are high.

The final blow count data from the case history results indicate that the results were very scattered. The standard deviations were high and the coefficients of variation ranged from 28% to as high as 77%. The average of the case history coefficient of variation values shown in Table 11-1 was 46%. Duncan (2000) reported on values of coefficient of variation for typical geotechnical properties and in situ tests. Geotechnical properties generally had coefficients of variation between about 5-45% while the SPT blow count (N) values ranged from 15-45%. It is also noted that the geotechnical properties dealing with fluid flow in soils (coefficients of permeability and consolidation) had values from about 33% to as high as 90%. The average case history result was 46% for the SPT $(N_1)_{60}$ blow count, which was on the high end of the range specified by Duncan. The values were also as high as 77% which was significantly greater than the upper bound of 45% referenced by Duncan. The use of wick drains increases the fluid flow in the soil during stone column treatment which may account for the increase in the standard deviations and coefficients of variation for the results. The high coefficients of variation for the case history results are thought to be reasonable when compared with the published results presented by Duncan (2000).

Although there are variations in the results and there is not a clear relationship describing the influence of fines content, area replacement ratio, drains per square foot, and initial blow count on the final blow count, the results presented here still offer valuable information that will allow a practicing engineer to decide whether they want to attempt to use wick drains to supplement stone columns. It is clear that high fines contents significantly decrease improvement. For example, it was seen that for an average fines content of 40-46% the area replacement ratio needed to be 23-26% in order to get similar results to of the 11-15% A_r in an average fines content of 5-20%. Also, an 11% A_r treatment with drains performed significantly less than the 26% A_r treatment with drains in high fines zones (40-46% average fines content). Finally, as the fines content increases at a particular site, the improvement is expected to decrease significantly as seen in Figure 11-9.

11.2 Overall Conclusions

- Wick drains were visually observed to relieve pore pressure at several sites during stone column installation. Water was observed exiting drains up to 20 ft from the stone column being installed.
- Standard deviations of the final blow counts tended to be high for each case history (28-77% coefficients of variation) and scatter were significant

so results were representative of average values instead of direct relationships.

- Sequencing the stone column installation to allow pore water pressure dissipation as well as to allow the installation of closure columns was determined to be an important part of the method.
- Despite fines contents greater than 20%, general penetration resistance gains with time are possible using drains with stone column as observed at the 24th Street Bridge and UPRR Bridge sites following treatment; however, significant scatter is to be expected (coefficients of variation of about 40%).
- High clay contents (>15%) were typically associated with low final blow counts (<15) and negative improvements; however, some high clay content data were also included in data that exhibited high final blow counts and positive improvements in blow count.
- Improvement tends to decrease as initial SPT blow count increases; however, less improvement is needed to prevent liquefaction for high initial blow counts.
- Stone column treatment with drains is significantly enhanced relative to treatment without drains at the same area replacement ratio. In the 24th Street test section, the areas with drains had fewer negative improvements than the areas without drains (7% versus 32%), fewer post-treatment $(N_1)_{60}$ values below the minimum site criterion of 18 (11% versus 25%), and a greater average increase in $(N_1)_{60}$ following testing (94% versus

27%). The coefficients of variation of the post-treatment blow counts were typically about 40%.

- The 24th Street and Shepard Lane Bridge site data indicate that, on average, increasing the area replacement ratio may provide similar results as those obtained by leaving the area replacement ratio the same and adding wick drains to the treatment plan.
- Within a site, average improvement in blow count decreases about 5 blows/ft for every 20% increase in fines content. Specifically, sites with 34-46% average fines contents, using stone columns and drains with a 23-27% A_r is expected to produce improvement generally ranging from 14-29 blows per foot in sections of the site where the fines contents are close to 20% while improvement is expected to generally decrease to 2-11 blows per foot when fines contents are close to 80%.
- Sites with 5-20% average fines content and 11-15% A_r with drains produced results similar to a clean sand site without drains and with an area replacement ratio of 5-10%.
- Sites with 25-28% average fines content produced results similar to a clean sand site without drains and with an area replacement ratio of 5-10% for both an 11% A_r with drains and an 18.6% A_r without drains. Significantly more improvement (two to three times more) was produced using an A_r of 26% with drains.
- Sites with 32-37% average fines and 26-34% A_r with drains produced up to 10 blows per foot more than the clean sand 20% A_r without drains

curve. A 9.3% A_r without drains only produced results similar to that of a 7% A_r for clean sands without drains.

- Sites with 40-46% average fines and 23-26% A_r only produced the equivalent results as a 5-10% A_r treatment without drains in clean sands. An 11% A_r treatment with drains produced results significantly less than the 5% A_r clean sand without drains curve.
- Sites with higher average fines contents required higher area replacement ratios to produce similar results. Improvement ranges and treatment plan improvement equations have been defined based on observed case history results.
- The primary factors affecting improvement using the stone column and wick drain method are the area replacement ratio, fines content, clay content, initial blow count, and time between completion of treatment and post-treatment testing.
- Additional factors affecting improvement which have been discussed in this study but for which consistent measurable results were not available for all of the case histories include the following: arrangement of stone columns, number of drains per stone column (or drains/ft^2), very fine sand, plasticity of fines, depth beneath the surface of treated soils, interbedded soil layers, variations in the soil profile across the site, pore pressure buildup during installation, thickness of the liquefiable layer, sequencing of stone column and wick drain installation, and whether the treated soil is on the boundary of the treatment.

- Testing accuracy is thought to be influenced by the following: proximity of testing to stone columns and drains, testing method accuracy, correlations applied to test results, and proximity of pre- and post-treatment testing (for direct comparison evaluations).

11.3 Design Recommendations

- To reduce the need for additional post-treatment testing, it is recommended that post-treatment testing be performed at least 6-7 days following treatment.
- In high fines content areas (>30% fines content), the use of drains is recommended instead of increasing the area replacement ratio.
- General improvement guidelines relative to A_r, initial $(N_1)_{60}$, and fines content have been provided and it is recommended that these guidelines be used to estimate initial stone column and wick drain spacing. The initial treatment plan should be tested using pilot field testing at the site to confirm that adequate improvement is achieved.

12 References

Adalier, K., and Elgamal, A. (2004). "Mitigation of liquefaction and associated ground deformations by stone columns." *J. Engineering Geology*, 72(3-4), 275-291.

Andrews, D. C. A. (1997). “Liquefaction of silty soils: susceptibility, deformation, and remediation.” PhD Thesis, University of Southern California, Los Angeles, CA.

Ashford, S. A., Rollins, K. M., and Baez, J. I., 2000a (2000). “Comparison of deep foundation performance in improved and non improved ground using blast induced liquefaction.” *Proc., Geo-Denver 2000, Soil Dynamics and Liquefaction*, ASCE Geotech. Special Publ., 107, 20-34.

Ashford, S. A., Rollins, K. M., Bradford, S. C., Weaver, T. J., and Baez, J. I. (2000b). “Liquefaction mitigation using stone columns around deep foundations: full scale test results.” *Proc., Transportation Research Board Meeting*, Transportation Research Board, Washington, D.C., p. 110-118.

Baez, J. I., and Martin, G. R. (1992b). “Liquefaction Observations during Installation of Stone Columns using the Vibro-Replacement Technique”, Geotechnical News, 10(3), 41-44.

Baez, J. I. (1995). “A design model for the reduction of soil liquefaction by using vibro-stone columns.” PhD Thesis, Civil Engineering, University of Southern California, Los Angeles, California. 207.

Baez, J. I., and Martin, G. R. (1995). “Permeability and shear wave velocity of vibro-replacement stone columns.” Soil Improvement for Earthquake Hazard Mitigiation. ASCE Geotech. Special Publ., 49, 66-81.

Barksdale, R. D., and Bachus, R. C. (1983). “Design and Construction of Stone Columns Volume I. Report No: FHWA/RD-83/026.” *Federal Highway Administration, Office of Engineering and Highway Operations, Research and Development*, Washington D.C. 1.

Boulanger, R., Idriss, I., Steward, D., Hashash, Y., and Schmidt, B. (1998). “Drainage capacity of stone columns or gravel drains for mitigating liquefaction.” *Proc., Geotech Earthquake Eng. and Soil Dynamics III,* ASCE Geotech Special Publ., No. 75, 678-690.

Construction, H. B. G. (2004). “Vibro Systems I.” Hayward Baker, <http://www.haywardbaker.com> (June 5, 2009)

Council, N. R. (1985). “Liquefaction of Soils During Earthquakes.” National Academy Press: 240p.

Duncan, J. M. (2000). “Factors of Safety and Reliability in Geotechnical Engineering.” *J. Geotech. Eng.*, 126(4), 307-316.

EQE (1995). ““The January 17, 1985 Kobe Earthquake” Summary Report.”

FEMA (2006). “Earthquakes in the United States.” <http://www.fema.gov/hazard/earthquake/usquakes.shtm> (April 12, 2006).

FEMA (2009). “FEMA Prepares New Study of Annualized Earthquake Losses.” <http://www.fema.gov/plan/prevent/hazus/hz_aelstudy.shtm#1> (May 1, 2009).

Holzer, T. L. (1998). “Introduction”, The Loma Prieta, California, Earthquake of October 17, 1989 - Liquefaction, U.S. Geological Survey Professional Paper 1551-B, U.S. Government Printing Office, B1-B8.

Hussin, J. D., and Ali, S. (1987). “Soil improvement at the Trident submarine facility”, *Soil Improvement-A Ten Year Update*, ASCE Geotech Special Publ. No. 12, 215-231.

Iai, S., Matsunaga, Y., Morita, T., Miyata, M., Sakurai, H., Oishi, H., Ogura, H., Ando, Y., Tanaka, Y., Karo, M. (1994). “Effects of remedial measures against liquefaction at 1993 Kushiro-Oki Earthquake.” *Proc., 5th US - Japan Workshop on Earthq. Resistant Design of Lifeline Facil. and Countermeasures against Soil Liquefaction*. Buffalo-NY: 135-152.

Iai, S., Morita, T., Kameoka, T., Matsunaga, Y., Abiko, K. (1995). "Response of a dense sand deposit during 1993 Kushiro-Oki Earthquake." *Soils and Foundations,* 35(1): 115-131.

Ishihara, K., Yamazaki, F. (1980). "Cyclic simple shear tests on saturated sand in multi-directional loading." *Soils and Foundations,* 20(1): 49-59.

Luehring, R., Dewey, B., Mejia, L., Stevens, M., and Baez, J. (1998). “Liquefaction Mitigation of a Silty Dam Foundation Using Vibro-Stone Columns and Drainage Wicks - A Test Section Case History at Salmon Lake Dam”, United States Department of the Interior, Bureau of Reclamation.

Luehring, R., B. Dewey, L. Mejia, M. Stevens, and Baez, J. (2000). “Liquefaction mitigation of a silty dam foundation using vibro-stone columns and drainage wicks - a test section case history at salmon lake dam.”

Management, E. S. R. (2009). "Esprit." <http://www.espritmdr.com/pages/photos.php> (June 2, 2009).

Martin, G. R., and Lew, M. (eds) (1999). "Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating liquefaction Hazards in California." Southern California Earthquake Center, University of Southern California, Los Angeles, CA.

Millea, M. T. (1990). "Liquefaction Mitigation Technology." Technical Note, National Civil Engineering Laboratory, Port Hueneme, CA.

Millea, M. T. (1990). "Liquefaction mitigation technology." Final report, February 1988-June 1989, pp. 37.

Mitchell, J. K. (1981). "Soil Improvement: State-of-the-Art." *10th International Conference on Soil Mechanics and Foundation Engineering.* Stockholm, Sweden. 4: 509 - 565.

Mitchell, J. K., and Solymar, Z.V. (1984). "Time dependent strength gain in freshly deposited or densified sand, ASCE." *J. of the Geotechnical Engineering* 110: 1559-1576.

Mitchell, J. K., and F.L. Wentz (1991). "Performance of Improved Ground during the Loma Preita Earthquake, Report No. UCB/EERC-91/12." *E. E. R. Center*, University of California, Berkeley.

Mitchell, J. K., Baxter, C.D.P, and Munson, T.C. (1995). "Performance of Improved Ground During Earthquakes." ASCE Geotechnical Special Publ. No. 49 - Soil Improvement for Earthquake Hazard Mitigation: pp. 1-36.

Mogami, T., and Kubu, K. (1953). "Plasticity models for seismic analysis of slopes." *Proceedings, 3rd International Conference on Soil Mechanics and Foundation Engineering.* Zurich. 1, 152-155.

Robertson, P. K., and Wride (Fear), C.E. (1998). "Evaluating cyclic liquefaction potential using the cone penetration test." *Canada Geotech. Journal*, 35, 442-459.

Rollins, K. M., Price, B.E., Dibb, E., and Higbee, J. (2006). "Liquefaction mitigation of silty sand in Utah using stone columns with wick drains." *GeoShanghai*. S.-L. S. Ali Porbaha, Joseph Wartman, and Jin-Chun Chai. Shanghai, China, ASCE/GEO Institute: 11.

Sasaki, Y., Taniguichi, E. (1982). "Shaking table tests on gravel drains to prevent liquefaction of sand deposits." *Soils and Foundations,* 22(3), 1-14.

Schmertmann, J. H. (1991). "The mechanical aging of soil." *J. Geotech. Eng.*, 117(9), 1285-1330.

Seed H.B, and Idriss, I.M. (1971). "Simplified procedure for evaluating soil liquefaction potential." *ASCE J. Soil Mech. Found. Div*. 97(9), 1249-1273.

Seed H.B., and Booker, J.R. (1977). "Stabilization of potentially liquefiable sand deposits using gravel drains." *ASCE Journal of Geotechnical Engineering Division* 103(7), 757 - 768.

Seed H.B., Idriss, I.M., and Arango, I. (1983). "Evaluation of Liquefaction Potential Using Field Performance Data." *ASCE Journal of Geotechnical Engineering* 109(3), pp 458-482.

Shenthan, T. (2005). "Liquefaction Mitigation in Silty Soils Using Stone Columns Supplemented with Wick Drains." PhD Thesis, Civil Engineering. Buffalo, New York, State University of New York at Buffalo. 336.

Snorteland, L. Mejia, Luehring, R., N. and Stevens, M. (2001). "Liquefaction mitigation of a silty dam foundation using vibro-stone columns and drainage wicks: a case history at salmon lake dam." Proc. 21st USSD Annual Meeting and Lecture, Denver, CO.

Thevachandran Shenthan, R. G. N., Sabanayafam Thevanayagam, and Geoffrey R. Martin (2003). "Liquefaction Mitigation in Silty Soils Using Composite Stone Columns and Dynamic Compaction." *MCEER* Research Progress and Accomplishments: 16.

U.S. Department of the Interior, U. S. G. S. (2009). "Earthquake Hazards Program." <http://earthquake.usgs.gov/learning/.> (May 1, 2009).

Youd, T. L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L, Harder, L.F., Jr., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marchuson, W.F., III, Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., and Stokoe, K.H, II (1997). "Summary Report. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils." Buffalo, NY. Technical Report NCEER-97-0022, 4-5: pp. 1-40.

13 Appendix A

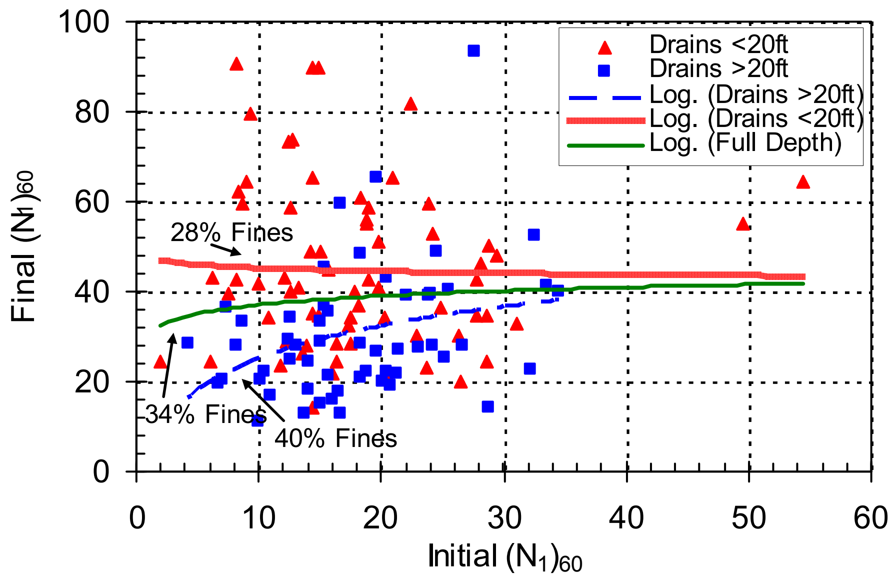


Figure 13-1 Initial $(N_1)_{60}$ versus final $(N_1)_{60}$ for the 26% A_r data with drains at the 24th Street Bridge case history.

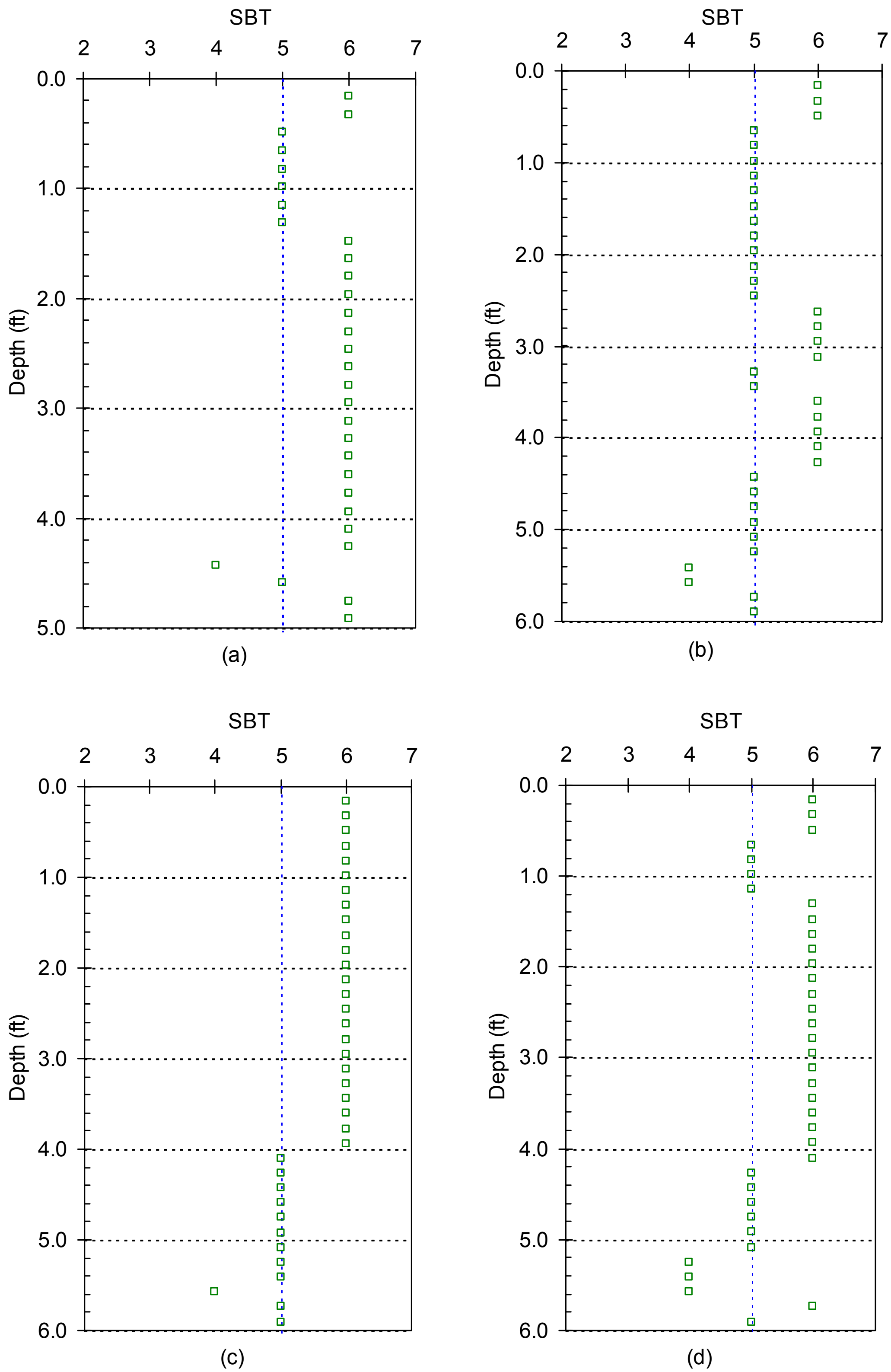


Figure 13-2 Depth versus SBT for the 7.1% A_r direct comparison pre-post soundings (a) 5 – 69, (b) 21 - 71, (c) 15 – 68, and (d) 18 – 49 at the San Pedro Home Depot case history.

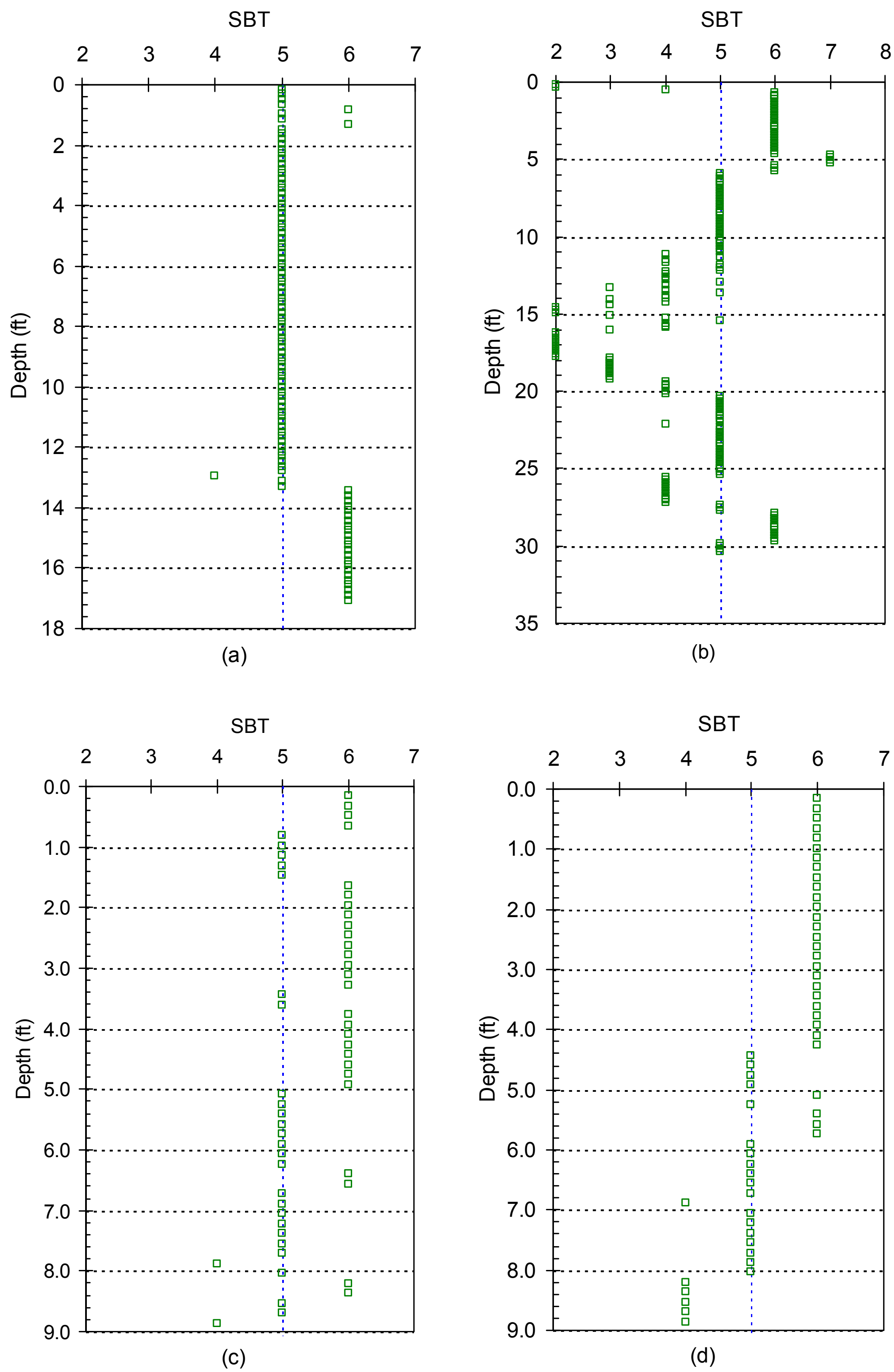


Figure 13-3 Depth versus SBT for the 7.1% A_r direct comparison pre-post soundings (a) 13 - 30, (b) 11 - 31, (c) 19 - 45, and (d) 20 - 50 at the San Pedro Home Depot case history.

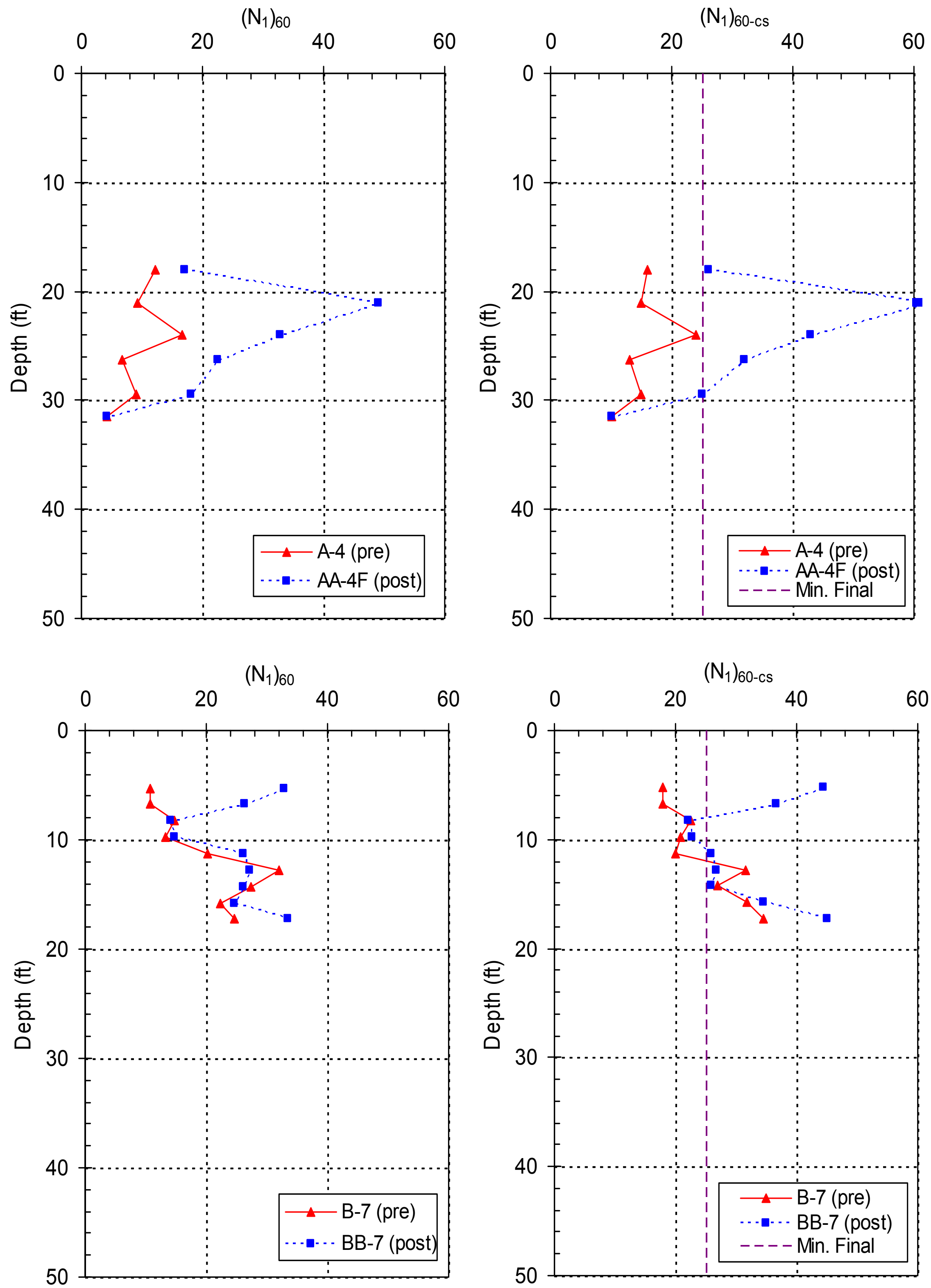


Figure 13-4 SPT $(N_1)_{60}$ and $(N_1)_{60\text{-cs}}$ test results for direct comparison data from 9.3% A_r treatment sections 4 and 7 at the Cherry Hill Bridge case history. The minimum final clean sand blow count criterion for acceptance used at the site is indicated with a dashed line in the clean sand plots.

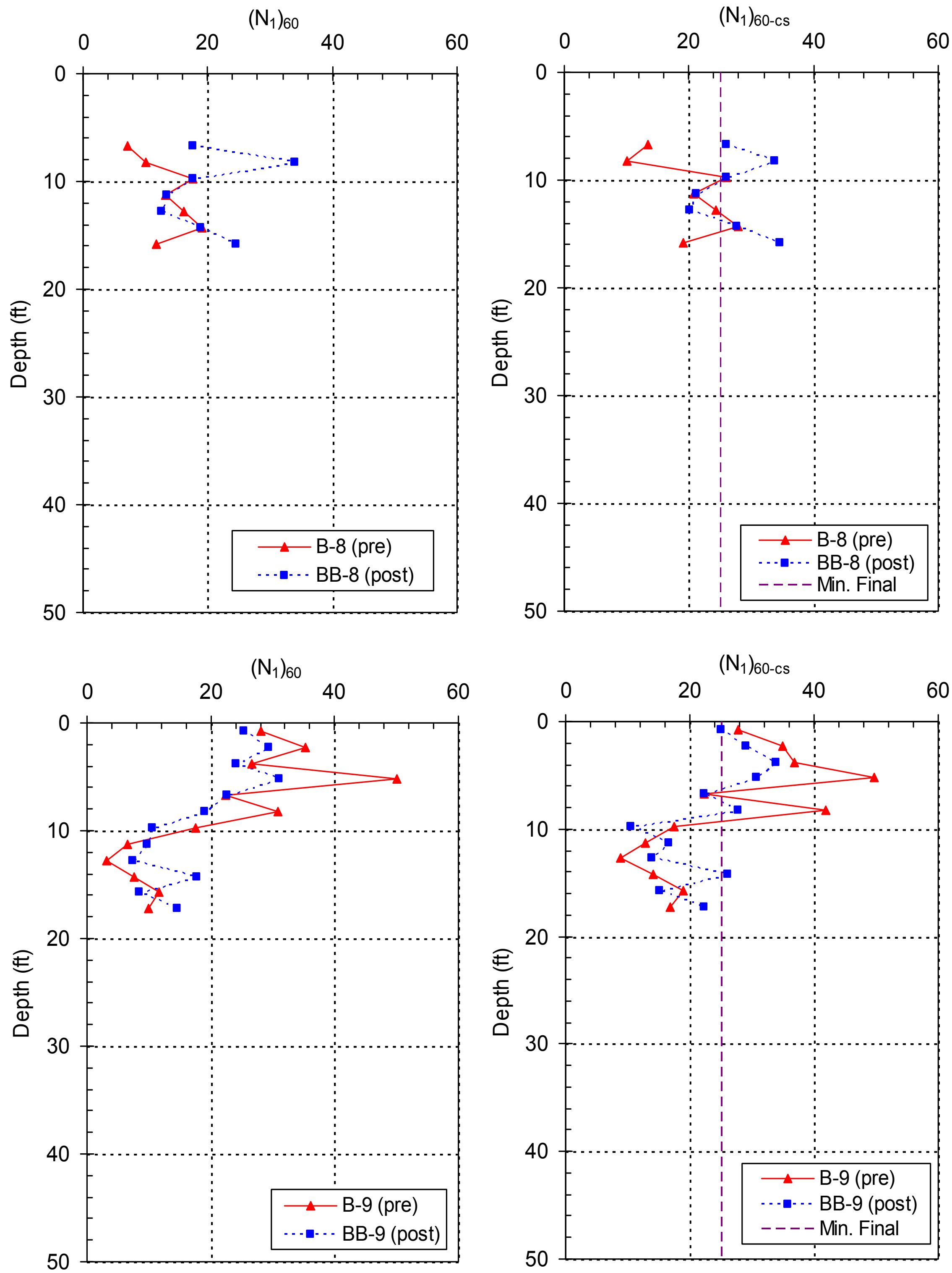


Figure 13-5 SPT $(N_1)_{60}$ and $(N_1)_{60\text{-cs}}$ test results for direct comparison data from 9.3% A_r treatment sections 8 and 9 at the Cherry Hill Bridge case history. The minimum final clean sand blow count criterion for acceptance used at the site is indicated with a dashed line in the clean sand plots.

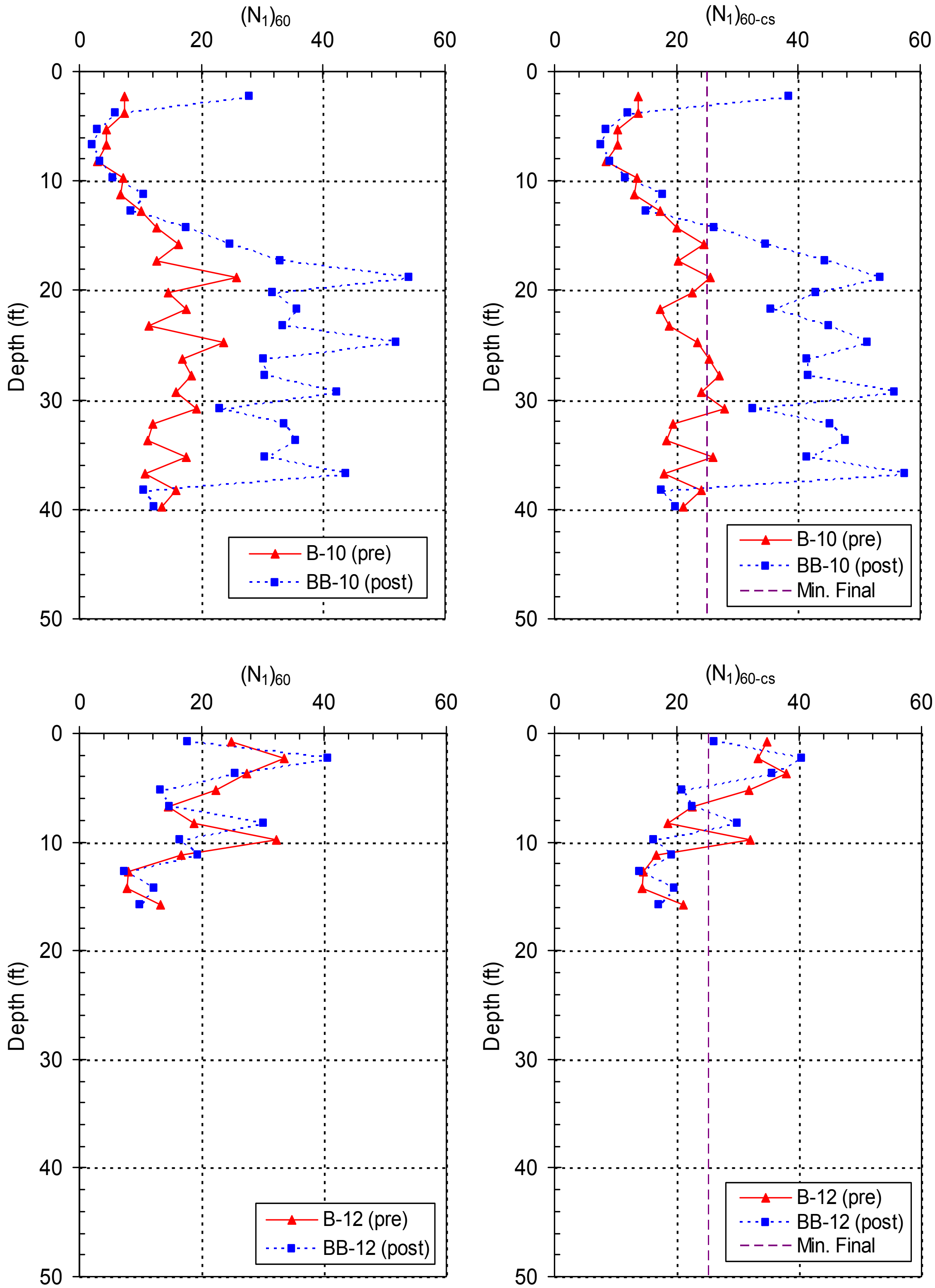


Figure 13-6 SPT $(N_1)_{60}$ and $(N_1)_{60\text{-cs}}$ test results for direct comparison data from 9.3% A_r treatment sections 10 and 12 at the Cherry Hill Bridge case history. The minimum final clean sand blow count criterion for acceptance used at the site is indicated with a dashed line in the clean sand plots.

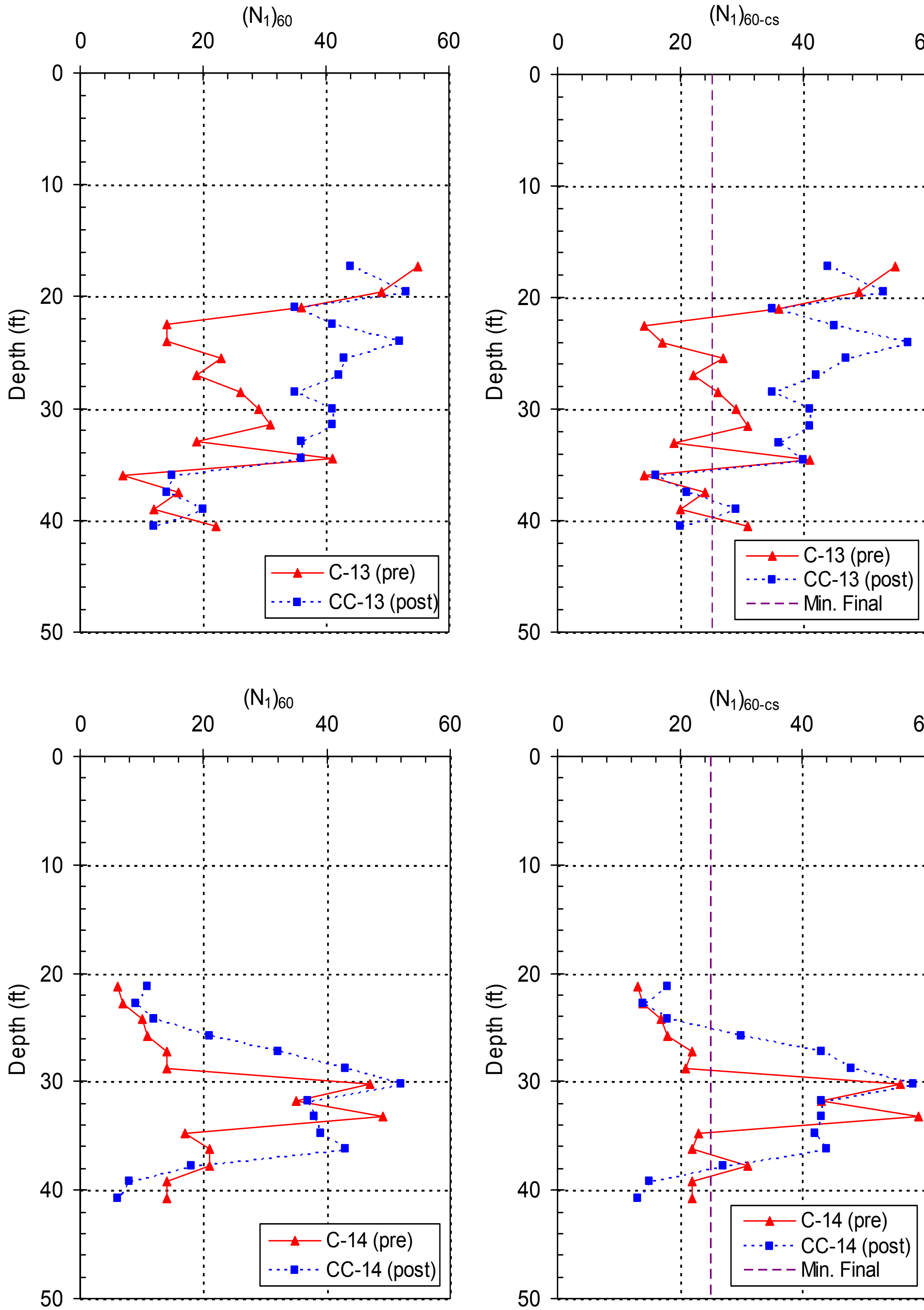


Figure 13-7 SPT $(N_1)_{60}$ and $(N_1)_{60\text{-}cs}$ test results for direct comparison data from 9.3% A_r treatment sections 13 and 14 at the Cherry Hill Bridge case history. The minimum final clean sand blow count criterion for acceptance used at the site is indicated with a dashed line in the clean sand plots.

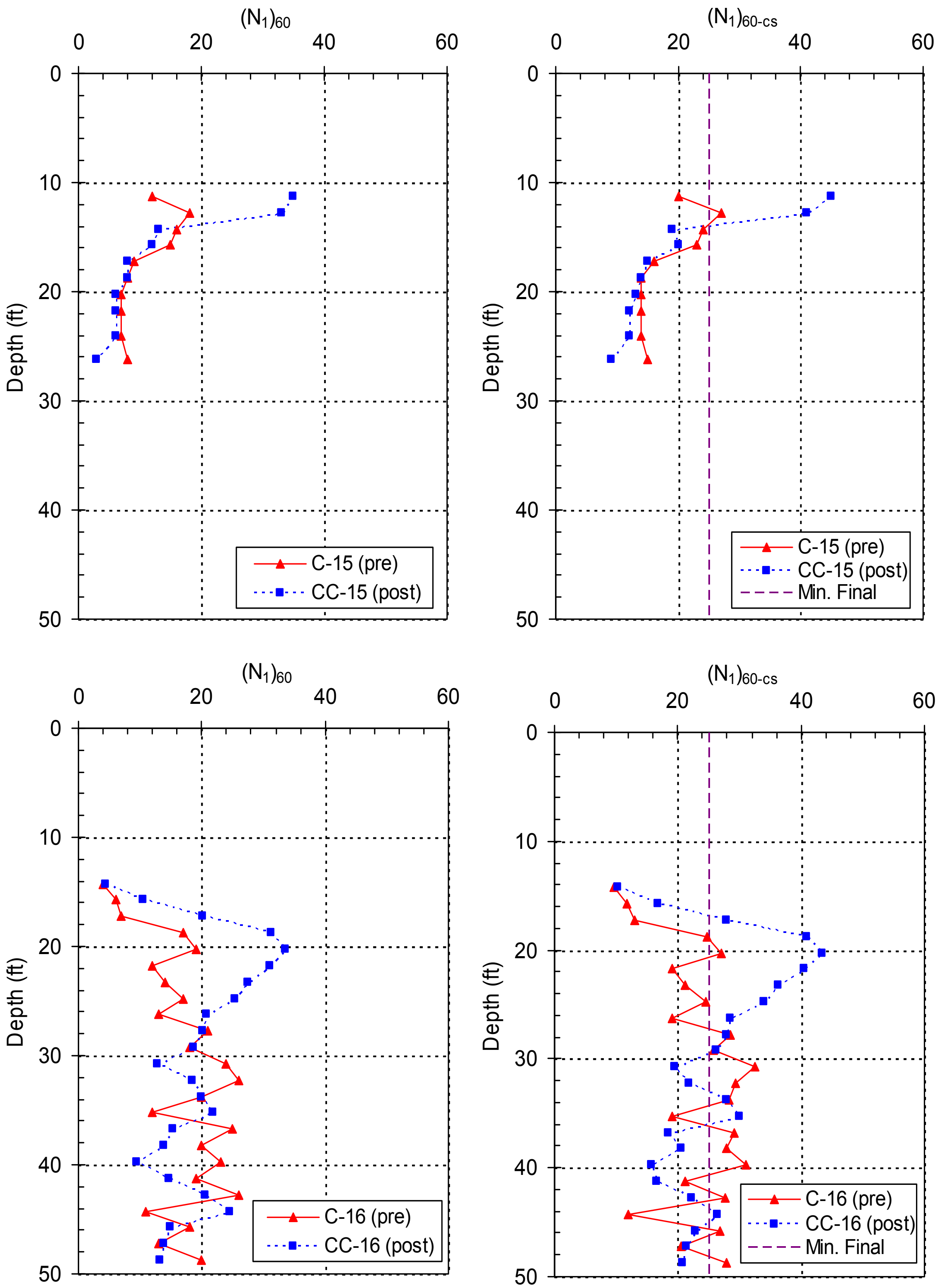


Figure 13-8 SPT $(N_1)_{60}$ and $(N_1)_{60\text{-cs}}$ test results for direct comparison data from 9.3% A_r treatment sections 15 and 16 at the Cherry Hill Bridge case history. The minimum final clean sand blow count criterion for acceptance used at the site is indicated with a dashed line in the clean sand plots.

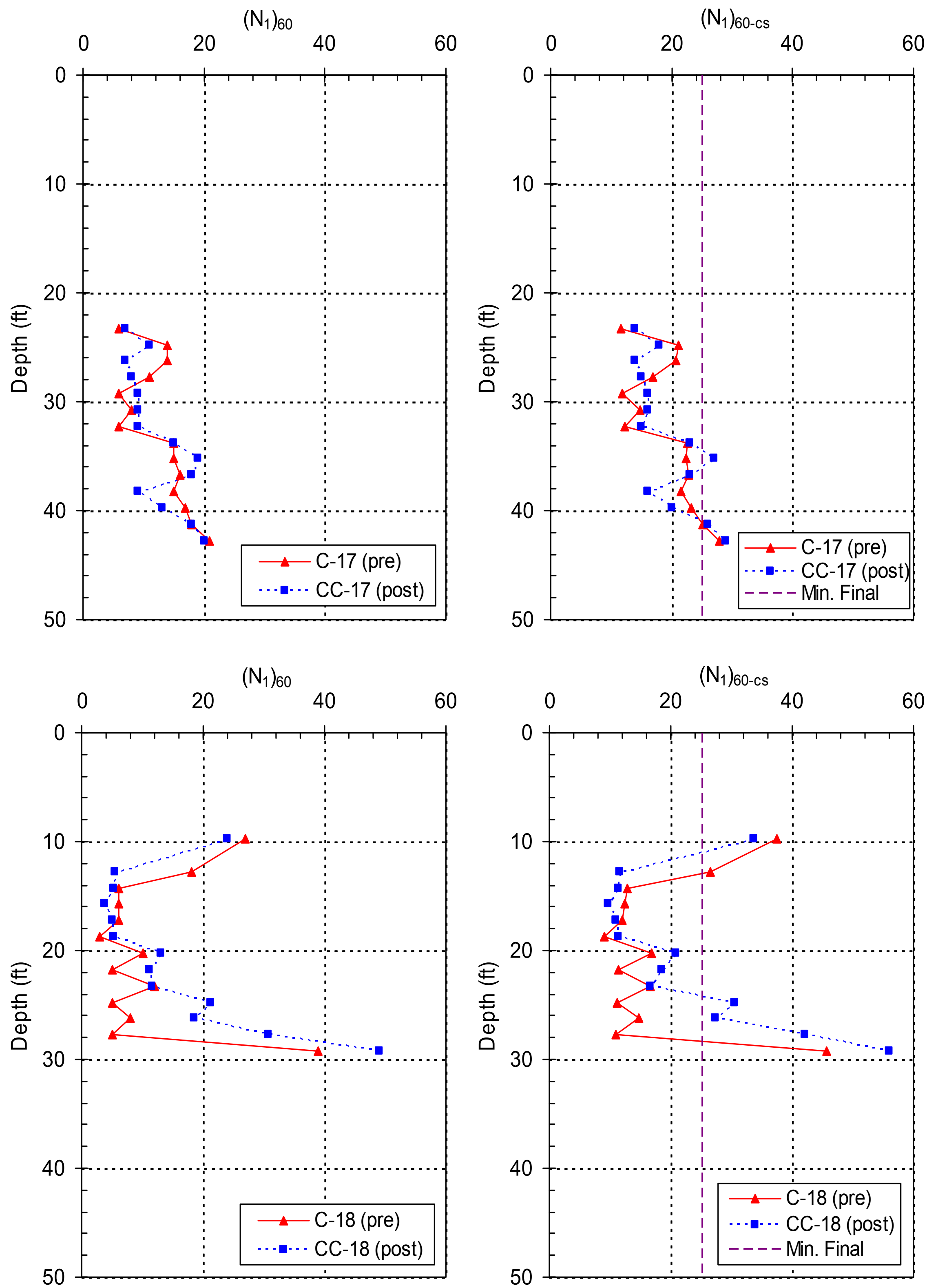


Figure 13-9 SPT $(N_1)_{60}$ and $(N_1)_{60\text{-cs}}$ test results for direct comparison data from 9.3% A_r treatment sections 17 and 18 at the Cherry Hill Bridge case history. The minimum final clean sand blow count criterion for acceptance used at the site is indicated with a dashed line in the clean sand plots.

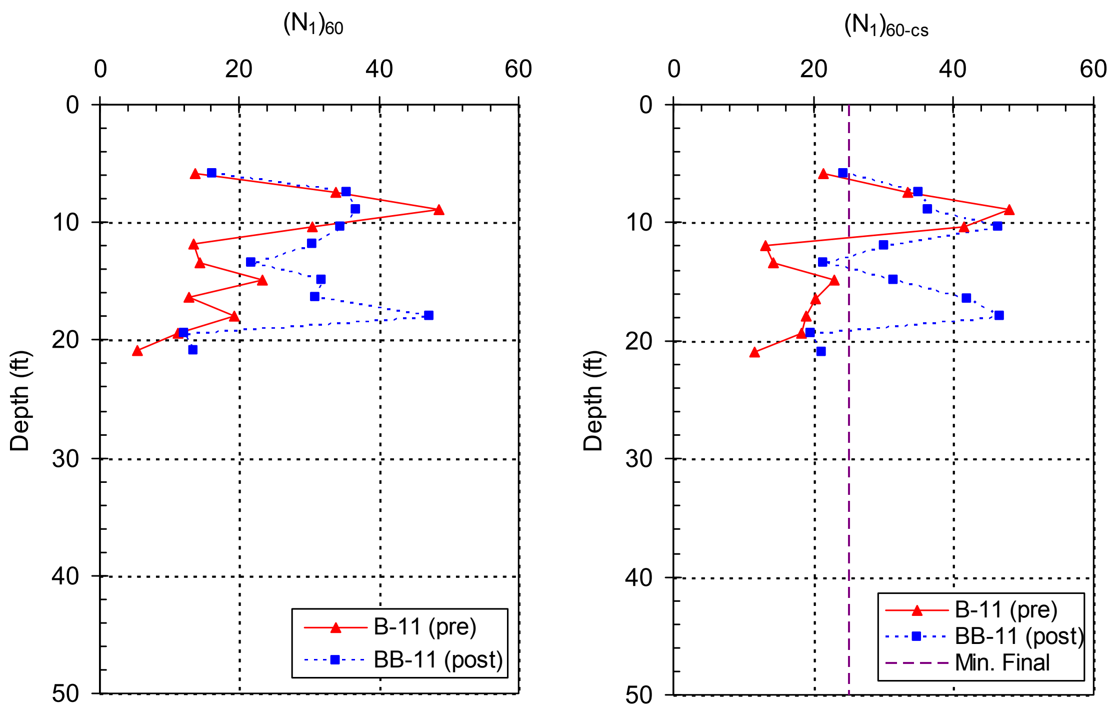


Figure 13-10 SPT $(N_1)_{60}$ and $(N_1)_{60\text{-}cs}$ test results for direct comparison data from18.6% A_r treatment section 11 at the Cherry Hill Bridge case history. The minimum final clean sand blow count criterion for acceptance used at the site is indicated with a dashed line in the clean sand plots.