Analysis of Aggregate Pier Systems for Stabilization of Subgrade Settlement

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16. Abstract

Every year, ODOT undertakes numerous pavement patching/resurfacing projects to repair pavement distress and structural failure due to soft and/or organic soils constituting the subgrade. Other than the temporary solution of patching/resurfacing, removal of weak soils and replacement with new suitable engineered fill can be used for permanent remediation. However, when problem soils are relatively deep this method can be too costly. There are various vertical column support methods used for civil engineering structures when soils are not strong enough to support the structure. Although some of these methods are utilized by several state DOTs to remediate settlement problems of existing roadways, they were not used in Ohio for this purpose. This study investigated the applicability of various vertical column support systems to improve subgrade and reduce settlements for existing roadways in Ohio. A decision matrix has been developed to identify the feasible methods. Two sites with ongoing subgrade settlements have been investigated through detailed subsurface investigations for possible implementation. Several technically feasible vertical column support methods have been identified for these sites. Lifetime cost-benefit analysis performed show that although there are some upfront costs associated with these remediation methods, the vertical column support ground improvement methods are significantly more cost effective compared to the current practice of patching/resurfacing temporary alternative during the project's lifetime, and can result in significant cost savings to ODOT and roadway users.

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LIST OF ABBREVIATIONS

AADT = Annual average daily traffic
AASHTO = American Association of State Highway and Transportation Officials
ASTM = American Society for Testing and Materials
CG = Compaction grouting
CMC = Controlled modulus columns
CPT = Cone penetration test
DOT = Department of Transportation
DSM = Deep soil mixing
EPA = Environmental Protection Agency
FHWA = Federal Highway Administration
IRI = International roughness index (mm/m)
JG = Jet grouting
LL = Liquid limit
LOI = Loss on ignition test
NCHRP = National Cooperative Highway Research Program
NHTSA = National Highway Traffic Safety Administration
NSC = National Safety Council
ODOT = Ohio Department of Transportation
PCI = Pavement condition index
PCR = Pavement condition rating
PI = Plasticity index
PL = Plastic limit
PSI = Present serviceability index
RAP = Rammed aggregate piers
SBT = Soil behavior type from CPT
SHRP = Strategic Highway Research Program
SPT = Standard penetration test
SPT-N = Standard penetration test blow count
SPT-N_{60} = Corrected standard penetration test blow count
STA = Stark County
SUM = Summit County
USCS = Unified soil classification system
VCC = Vibro concrete columns
VSC = Vibro-replacement stone columns
ΔIRI = International roughness index difference (in mm/m)
q_u = Unconfined compressive strength
w = Moisture content

CHAPTER 1. INTRODUCTION

1.1 Statement of the Problem

Every year, patching and resurfacing projects are undertaken to repair pavement distress and structural failure due to problematic subgrade soils. The request for proposals (RFP) issued by the Ohio Department of Transportation (ODOT) for this research project stated that "*Every year, patching/resurfacing projects are undertaken by ODOT to repair pavement distress/structural failure due to soft and/or organic soils constituting subgrade*." These conditions seriously impact roadway function and safety, and create substantial costs to remediate the problems caused by subgrade settlement.

Many of the subgrade settlement issues occur as a result of very soft to soft cohesive soils, very loose to loose granular soils, saturated soils, and/or organic soils, all of which yield low strength and subgrade support characteristics. Settlements of these soils may exist throughout the year, but may also occur following saturation during the spring and autumn rainfall. The settlement problem can usually be addressed by using conventional remediation methods or by using vertical column support methods.

Conventional methods to remediate subgrade settlements caused by problem soils include removal of weak soils and replacement with new suitable engineered fill, near-surface chemical stabilization such as lime or cement, or preloading/surcharging with or without wick drains. However, when problem soils are relatively deep, or long term settlement tolerances are low, these conventional methods can sometimes prove ineffective or too costly. New technologies and extended application of old technologies has led to some relatively limited use of vertical column support methods for the remediation of roadway subgrade settlements. The use of vertical support columns can be employed either as a deep foundation system or as a ground improvement technique.

Vertical column support systems create relatively stiff column elements which allow for reduced loads on the surrounding weaker soils and improved support of the overlying roadway systems. Vertical column composition varies greatly and includes concrete, grout, stone/aggregate, sand, or soil-cement inclusions within the existing soil matrix. Depending on soil conditions some of the methods may require geosynthetic encasement. Some methods not only create strong vertical column elements at the locations they are installed but also improve the surrounding weak soils through a number of differing techniques. As a result, the current vertical column support methods can generally be grouped into two distinct subsets: ground improvement and deep foundations. Examining the available vertical column support methods and identifying the cost effective methods which provide increased subgrade support and decreased settlements will assist ODOT by improving roadway safety, reducing future pavement rehabilitation projects, lowering repair costs, as well as lowering the overall cost to the road users and society as a whole.

1.2 Research Objectives

The vertical column support systems used as a ground improvement method along with the surrounding soils behave as a composite foundation system, and the columns carry more loads than the surrounding soil due to their greater stiffness. For an existing roadway system, the installation of vertical column systems results in a large portion of the subgrade, pavement, and traffic loads being carried by the columns due to their higher stiffness and therefore reducing the amount of pressure on the soil matrix resulting in reduced soil settlements. The installation of vertical column systems such as aggregate piers not only provides highly stiff elements that have high angle of internal friction, but also causes high lateral stress development in the soil matrix around the piers due to the lateral densification of the soils during the installation of piers. These composite foundation systems increase the bearing capacity under the roadway pavement, resulting in reduced roadway settlement and pavement distress/structural failures. Depending on the type of the aggregates used in the piers, the columns can also increase the permeability of soils below the roadways and can facilitate faster dissipation of excess pore water pressures.

The main objectives of this research project are to evaluate the applicability and use of various vertical column support systems to improve the subgrade support and reduce settlements for existing roadways in Ohio, to compare the vertical column treatment systems to conventional soft subgrade treatment techniques, to perform a cost-benefit analysis of methods that can be used for remediation, and ultimately to determine cost effective means to reduce pavement distress that can be utilized for Ohio's problematic soils.

1.3 Background and Significance of Work

In addition to the proven deep foundation, such as pre-cast piles and drilled shafts, there are various ground improvement techniques that can be used to improve the bearing capacity and reduce roadway settlements when weak soils are present at a project site. Some of these ground improvement methods include soil-cement columns, compaction grouting, vibroflotation, vibro-replacement stone columns, and rammed aggregate piers.

Deep foundation methods such as pre-cast concrete piles, steel piles and drilled shafts have been used for a long time and have proven to be effective in reducing settlement of structures. However, local constructability requirements and their costs can prevent deep foundations from being a favorable alternative for some projects. Due to the cost and time savings, ground improvement methods are being utilized more and more in transportation projects. Some of the ground modification methods have been used for some time and their behavior is well documented. Vertical column support methods, such as rammed aggregate piers and stone columns, are relatively new methods used in transportation projects to improve bearing capacity and reduce soil settlements.

The use of vertical column support systems for the remediation of existing roadways that are exhibiting settlement problems is a limited technology in Ohio. There are several vertical column support systems available and have been used for structures and embankments. Some of the vertical column support systems available are rammed aggregate piers, auger cast piers, vibroflotation, vibro-replacement stone columns, sand columns, deep soil mixing, soil jetting, compaction grouting, controlled modulus columns, vibro concrete columns, vibro-dry concrete columns, auger cast-in-place piles, pre-cast piles, concrete solid piles, and concrete pipe piles.

It is important to understand the engineering fundamentals, construction techniques, processes, and costs involved with each method to be able to assess their applicability, effectiveness, advantages, and disadvantages for the projects they are considered. The evaluation of each method should focus on a complete system, including the characterization of the site's soil and groundwater conditions, evaluation of vertical column alternatives, design of column and load transfer mechanism, and pavement design when the vertical columns are used. For example, for vibro-compaction techniques detailed soil gradation information is critical to the design of such methods, as minor changes in soil gradation characteristics could affect method feasibility. Furthermore, the in-situ soil testing method used (e.g., standard penetration, cone penetration, and pressuremeter) need to correspond to the technique used for performance verification of the ground improvement technique, as the test data obtained during design will be the baseline to which the improved ground will be compared. On the other hand, some of the methods, such as driven piles, may not be feasible due to the construction vibration and noise constraints depending on the project location. Therefore, the evaluation and selection of method(s) for the subgrade settlement remediation should be conducted carefully and thoroughly considering the site conditions as well as the advantages and limitations of each possible method. As mentioned previously, examining the available vertical column support methods while identifying the cost effective methods which provide increased subgrade support and decreased settlements will assist ODOT by improving roadway safety, reducing future pavement rehabilitation projects, lowering repair costs, as well as lowering the overall cost to the road users and society as a whole.

1.4 Research Methodology and Work Plan

The research project was proposed to be achieved through the following five phases:

- Phase 1 – Literature review,
- Phase 2 – Decision tree and site selection,
- Phase 3 – Implementation plan,
- Phase 4 – Field work/construction, and
- Phase 5 – Monitoring, numerical modeling, assessment, and report.

<u>Phase 1 – Literature review:</u> There are various vertical column support methods used to remediate and/or prevent the soils settlement of structures. Several components are important in assessing the applicability and effectiveness of these vertical column support methods. Although these methods are regularly used for new roadway construction and other structures, currently they are not common for the remediation of subgrade settlement of existing roadways. Therefore, it is important to evaluate each method considered for this project carefully and understand their advantages, limitations, and construction processes, which will be achieved through the literature review phase. Survey of other state DOTs' experiences with subgrade soil settlements and use of vertical column support methods for the existing roadways will be conducted and analyzed.

Phase 2 – Decision tree and site selection: Using the information gathered during literature review phase, critical aspects and components of vertical column support methods for the successful implementation in transportation projects will be identified. A decision matrix to be used for the selection of applicable vertical column support system(s) will be developed based on these critical aspects and components identified.

As part of this phase, several sites experiencing ongoing settlement problems will be reviewed for potential implementation of vertical column support method(s) in the field. Subsurface investigations will be conducted at the site(s) deemed appropriate for possible implementation. Once the investigations are completed, vertical column support method(s) applicable at these site(s) will be selected using the decision tree developed. Finally, a detailed lifetime cost-benefit analysis will be performed to determine the cost effectiveness of the vertical column support methods identified as feasible methods at the sites investigated.

The following three phases (Phases 3, 4, and 5) proposed for the research are related to the implementation of the method(s) selected. Continuation of the project with implementation phase was in discretion of ODOT based on the results of cost-benefit analysis.

Phase 3 – Implementation plan: The work proposed for this phase included preparing instrumentation and monitoring plans, field and load testing plans, and preliminary drawings and specifications.

Phase 4 – Field work/construction: The main focus of the research team during the construction phase was proposed to be monitoring the installation of vertical column supports and instrumentation, taking the initial/baseline readings of the instrumentation, and monitoring the in-situ field and load tests.

Phase 5 – Monitoring, numerical modeling, final assessment, and report: The proposed work for this phase included monitoring the instrumentation installed during the construction, performing numerical analyses to model the behavior of installed vertical column support method(s), assessing both the performance of the method(s) installed using the data collected in the field and the numerical modeling/analyses performed. The project would be concluded with a final report comprising the summary of the detailed literature review, decision tree, subsurface investigations data, laboratory test results, construction drawings for test sites, data gathered during construction, numerical analysis results, post-construction monitoring data collected until the end of the project, and the assessment of the results and data collected.

1.5 Report Outline

This report consists of seven chapters. This chapter, Chapter 1, contains introductory information. Chapter 2 presents the information on vertical column support methods considered for this research project. Chapter 2 also contains a summary of the survey data collected from other state DOTs on their experiences with subgrade settlement problems under existing roadways and the use of vertical column support methods in their states. Chapter 3 introduces the decision matrix developed that can be used to identify potential vertical column support methods for a given project site in Ohio. Chapter 4 presents the subsurface investigations conducted at two potential implementation sites (SUM-224-13.14 and STA-44-18.23 sites). Subsurface investigations included drilling and sampling, field testing, and laboratory testing. Chapter 5 presents the analysis of data collected from subsurface investigations and the selection of potential vertical column support method(s) for the two sites using the decision matrix developed. Chapter 6 describes lifetime cost-benefit analysis components for roadway remediation projects, develops a general model for performing the cost-benefit analysis, and analyzes lifetime costs and benefits of remediation for the two sites considered. The final chapter, Chapter 7, presents the summary of work done, conclusions, and recommendations.

CHAPTER 2. LITERATURE REVIEW

2.1 Introduction

The primary objectives of this research project are to evaluate the applicability of the various vertical column support methods for subgrade support and settlement reduction, compare the vertical column support systems to conventional treatments, and determine the most cost effective means to reduce pavement distress caused by Ohio's problematic soils. The objective of the literature review was to gather and review published information on the current vertical column support systems used for subgrade support and to reduce settlements of existing roadways.

A specific search was also performed to identify other research studies performed on the use of vertical column support systems for the remediation of existing roadways exhibiting subgrade settlements. No other similar research studies were located. Some previous research studies on the use of vertical column support systems used to support embankments for new roadways, road widening, and bridge approaches were reviewed. Although these research projects were not for the remediation of existing roadway settlements, they did include valuable information on the vertical column support methods and their applicability for various soil conditions.

This chapter presents the findings of the literature review conducted on the vertical column support methods considered in this research project for the remediation of existing roadway settlements. The methods considered are discussed under two main groups; ground improvement methods and deep foundation systems. A summary of the survey conducted on other state DOTs' experiences with subgrade soil settlements under roadways and with their use of vertical column support methods are also provided later in this chapter.

2.2 Ground Improvement Methods

Where deep foundations rely on transmitting structural loads to suitable soils or bedrock, ground improvement methods can also increase the strength parameters and decrease future settlement of the in-situ soils. Many ground improvement methods not only improve the in-situ soils, but also create strong vertical column elements out of aggregate materials (stone), sand, grout, or soil-cement columns. Some ground improvement methods such as vibroflotation, vibro concrete columns, deep soil mixing, soil jetting, compaction grouting, controlled modulus columns and sand columns have been used for some time for various type of structures. Other methods such as aggregate piers (including rammed aggregate piers and vibro-replacement stone columns) are relatively new methods for ground improvement in the transportation industry and the construction industry as a whole.

The primary appeal of ground improvement methods is the vertical column elements can typically be terminated at relatively shallower depths compared to deep foundations since the surrounding poor soils are improved through their installation process, as shown in Figure 1.

Generally speaking, terminating at shallower depths allows ground improvement methods to be done more quickly and more cost-effectively than deep foundations. The benefit of shallower ground improvement methods is even more pronounced when the structural loads to be supported are low to moderate in size.

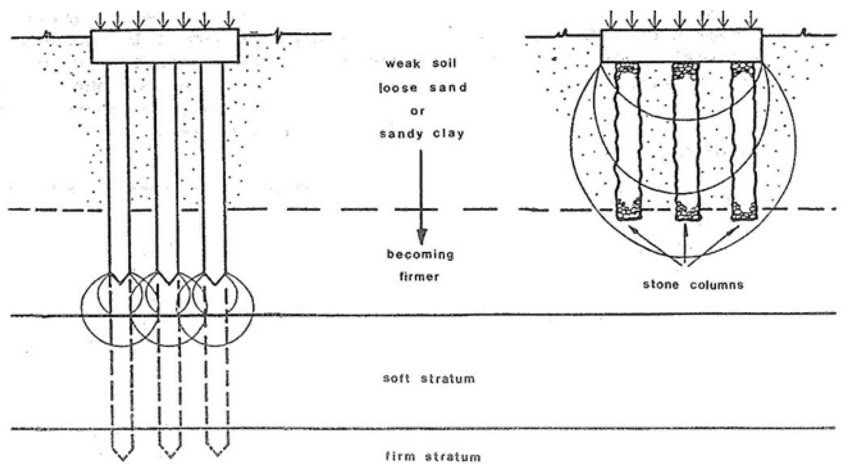


Figure 1. Comparison of deep foundations (piling) and ground improvement (stone columns) (Glover 1985)

Several ground improvement methods are considered during this study for possible use in the stabilization of subgrade settlement of existing roadways. The methods considered include vibroflotation, vibro-replacement stone columns, vibro concrete columns, deep soil mixing, jet-grouting (soil jetting), compaction grouting, controlled modulus columns, sand columns, and rammed aggregate piers. The overview, construction process, fundamentals, techniques, advantages, and limitations of each method are discussed in the following.

2.2.1 Vibroflotation

Vibroflotation (also known as vibro compaction) can trace its origins to the 1930s in Germany with the first working example implemented around late 1930s. Within the United States, vibroflotation has been in use since late 1940s. The method was envisioned as a solution to improve clean granular materials via compaction or densification either above or below the water table by inserting a horizontally-vibrating probe into the soil. Over time, improvements have been made to the system primarily focusing on the use of larger, more powerful vibratory equipment which results in achieving higher relative densities to larger radial distances. More powerful vibratory equipment also allows for more efficient implementation and generally lower cost associated with larger spacing between improved locations.

Construction Process, Fundamentals, and Techniques:

The vibroflotation process involves inserting a horizontally-vibrating probe into clean granular soil. A brief overview of the construction process is shown in Figure 2. The probe or vibroflot is supported by a crane and extended to the design depth under the weight of the vibration equipment, jetting fluid forced from the bottom of the probe, and vibration if necessary. Depths of approximately 3 to 15 m (10 to 50 ft) are commonly achieved; however, some equipment can extend to depths up to 36 m (120 ft) below the ground surface. Next, the vibrator is activated and slowly raised to the surface creating a cylinder of improved soil. Water and/or additional granular material can be fed to the bottom of the vibration equipment to aid in the compaction process or maintain the surface elevations. The radial compaction of the surrounding soil relies on the temporary elimination of contact between the sand particles and the permanent rearrangement into a denser state with the aid of gravity (Figure 3).

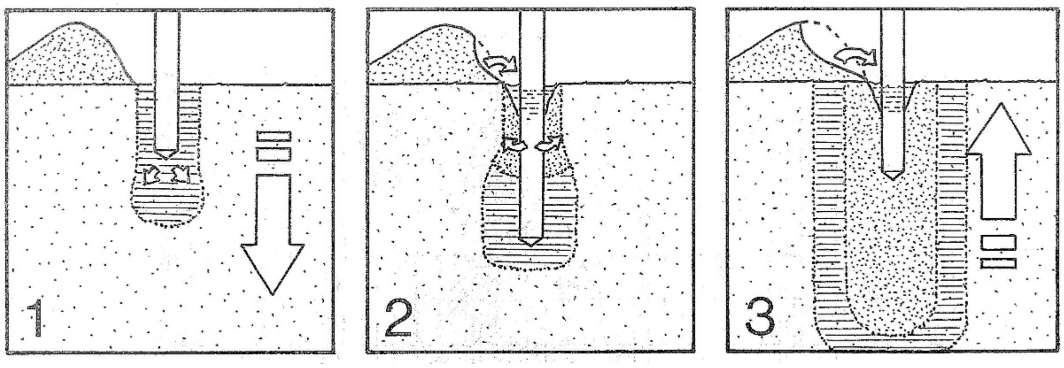


Figure 2. Vibroflotation process (1) Insertion of the vibrator (2) Initiation of vibration and addition of fill material (optional) (3) Vibration continues while slowly raised to surface to form densified cylinder (Glover 1985)

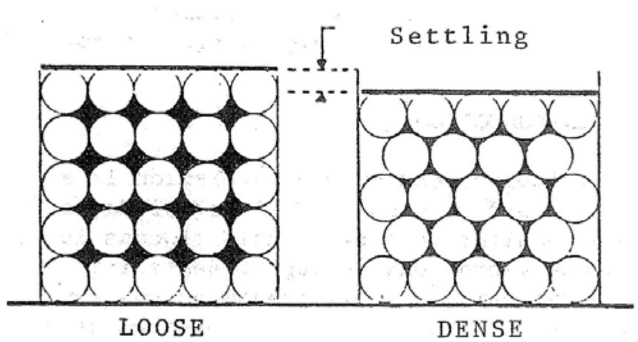


Figure 3. Soil densification and settlement after vibroflotation (Besancon and Pertusier 1985)

The compaction is obtained only at the elevation of the vibrator head and relies on the rapid transmission of pore water pressure to facilitate the "flotation" of the sand particles, even when the process is performed beneath the groundwater level. Significant silt and clay contents reduce the densification achieved by eliminating the rapid transmission of pore water pressures and dampening the vibratory forces. Therefore, for maximum ground improvement sands should have silt content no greater than 12 to 15 percent and clay content no greater than 2 to 3 percent

(Glover 1985). Figure 4 shows the effectiveness of vibroflotation method based on grain size analysis. In the figure, Zone A represents materials having excellent potential for densification, Zone B ideal for sands with particle size distributions within this range, and Zone C represents soils which are generally not suitable for vibroflotation (Glover 1985). Soydemir et al. (1997) stated that saturated sands with fine contents less than 25 percent and clay contents below 2 percent will yield densification enhancement by vibratory ground improvement methods. In addition to the particle size criteria, the soils must have high enough friction angle to transfer the shear waves generated during vibration. For sites with soils outside the aforementioned parameters, the vibro-replacement stone column technique is appropriate. Due to the variability in soil types, equipment specifications, and desired ground improvement at each site, the spacing of the probe locations is unique to each individual site. However, typical spacing ranges from approximately 1.5 to 3.6 m (5 to 12 ft) (Wightman 1991). Oftentimes, standard penetration (SPT) or cone penetration (CPT) tests are performed before and after the vibroflotation process to confirm the design assumptions and ground improvement criteria.

Advantages:

The primary advantage of the vibroflotation process stems from the ability to simply improve the existing ground rather than rely on the addition of grout, stone, or concrete columns to support structural loads. The relatively fast process allows for large areas to be improved in a quite short period of time. The presence of groundwater does not diminish the compaction ability of vibroflotation since the vibratory forces are transmitted through the water to the surrounding soil. Sites containing clean sands meeting the requirements outlined above can generally achieve relative densities greater than 90% (Glover 1985).

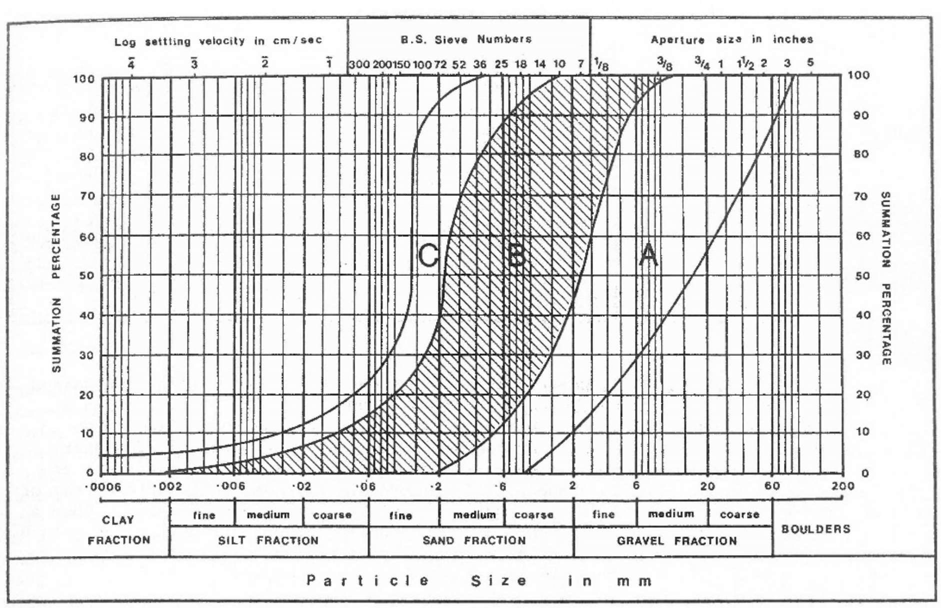


Figure 4. Particle size analysis ranges for vibroflotation process: Zone A-Excellent potential, Zone B-ideal for sands, and Zone C-generally not suitable for vibroflotation (Glover 1985)

Limitations:

Careful pre-construction particle size analysis of the in situ soils must be performed since the amount of densification and radial distances of improvement are primarily a factor of the soil type. As a result of the soil type limitations of this method, the vibro-replacement stone column and vibro concrete column methods were developed. These methods, which are discussed later, have increased the number of applicable sites for the vibration-based processes. Dense concentrations of boulders or presence of weakly cemented granular soils may also prohibit the effectiveness of compaction using vibroflotation (Glover 1985).

2.2.2 Vibro-Replacement Stone Columns

An extension of the aforementioned vibroflotation (or vibro compaction) method is the vibro-replacement stone column method. Since the true vibroflotation method is primarily applicable to a narrow number of sites containing clean sands, the vibro stone column and vibro concrete column methods were later developed to expand the applicability of the method for other soil conditions. The vibro-replacement method is also known as the vibro-stone column, vibro-displacement stone column, or vibro aggregate pier method. Vibroflotation simply improves the surrounding granular soil, whereas the vibro-replacement method laterally compacts the surrounding clean granular soil and constructs a stiff vertical stone column element. The first rudimentary stone column method for ground improvement can be traced to France in the 1800s. The modern techniques and equipment, as the vibro-replacement stone column known today, have been used in Europe since the 1950s and later in the United States since the 1970s. A significant amount of research has been performed related to the seismic benefits of vibro-replacement stone columns and reducing liquefaction potential in earthquake prone locations and sites. In addition, numerous studies have been conducted on the increased bearing capacity, settlement reduction, slope stability improvements, time rate of settlement increases, and time-related construction advantages of this method for ground improvement (Mitchell and Huber 1983, Swenson et al. 1995, Davis and Roux 1997, Saxena and Hussin 1997, Somasundaram et al. 1997, Soydemir et al. 1997, Ausilio and Conte 2007, Guetif et al. 2007, Han et al. 2007, Blackburn et al. 2010).

Construction Process, Fundamentals, and Techniques:

The vibro-replacement stone column construction process begins with the insertion of a crane supported vibratory probe into the ground by means of the weight of the equipment, and/or water jetting and vibration. In some vibro stone column processes, pre-augering of the shafts can also be performed to accelerate the construction process and increase the diameter of the stone column. Stone or aggregate backfill material is added from the surface to the bottom of the probe via the cavity created by the probe or feeder tubes extending to the probe tip. After a single lift of stone material has been added, the horizontal vibrator is reinserted into the stone lift multiple times. The vibration compacts the stone mass and also laterally stresses the surrounding soil. Successive lifts ranging from 0.3 to 1.2 m (1 to 4 ft) in thickness are placed, vibrated/compacted and continued up to the ground surface. Stone columns typically terminate after either penetrating through the soft compressible silt/clay layer with the tips embedded in stiff load bearing material, or can terminate with the ends of the stone columns embedded in the silt/clay layer which is referred to as floating (Shahu and Reddy 2012). The lateral resistance of the surrounding soil and the vibration process combine to create a dense vertical stone column element with typical assumed friction angles on the order of 35 to 45 degrees (Barksdale and Bachus 1983). McCabe and Egan (2010) analyzed over 20 stone column installations within primarily cohesive soils and concluded that the typically assumed friction angle of 40 degrees provides a conservative value for the bottom feed practice. Figure 5 shows the schematic of vibro-replacement stone column construction.

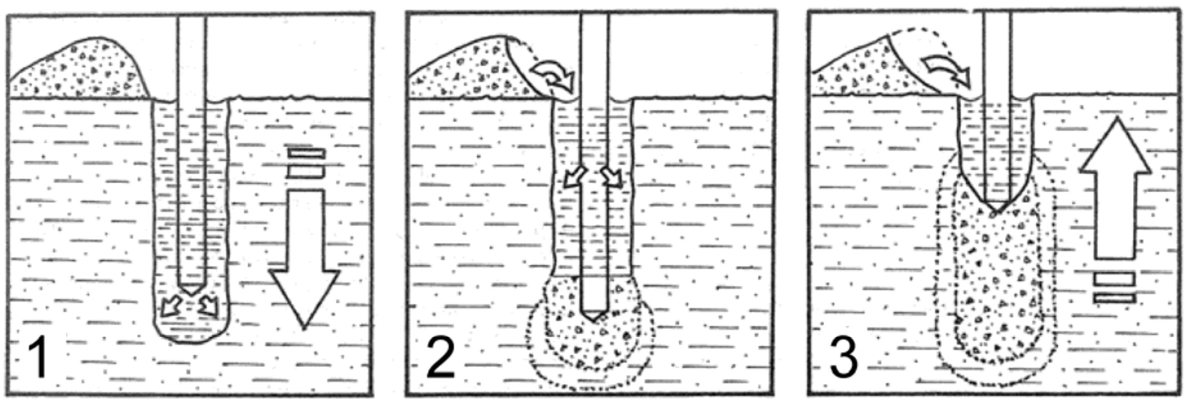


Figure 5. Vibro-replacement stone column process (1) Insertion of the vibrator (2) Initiation of vibration and addition of successive lifts of stone backfill material (3) Filling/compaction continues to the surface forming compacted stone column (Glover 1985)

Stone columns can also be partially or fully encased in a geotextile material. A study by Dash and Bora (2013) resulted in non-encased stone columns providing an improvement in bearing capacity of 3.5 times the original conditions, the partially encased columns providing a bearing capacity improvement of 5 times the original capacity, and the fully encased columns providing a bearing capacity improvement of 3 times the original capacity. The partially encased columns were only on the upper 60% of the column. Figure 6 shows the behavior of stone columns with partial, full and without geotextile encasement reported by Glover (1985). The study results show that the fully encased and non-encased columns provide comparable improvement in bearing capacity. The partial encasement shows a clear advantage over the full and non-encased columns in regards to the improvement of bearing capacity.

When vibro-replacement is performed on sites with primarily granular material, significant relative density improvement of the surrounding soil can be achieved. Although some reports suggested that clayey soil or sands with fine contents greater than 15 percent between compaction points were not greatly improved (Glover 1985, Chen and Bailey 2004), other tests show that significant improvement can be achieved owing to the lateral pre-stressing and increased capacity for dissipation of pore water pressure of the cohesive soil (Ausilio and Conte 2007). In either case, a composite layer method is used to evaluate the parameters of the combined stone column and surrounding weaker soil system. The stone columns have higher shear strength and stiffness characteristics than the surrounding soil, therefore when settlement of the composite layer begins the stresses are naturally transmitted to the stronger stone columns. In addition to the modulus values of the stone column, the area replacement ratio is a key component of the improved ground (Priebe and Grundbau 1995). The area replacement ratio is calculated by dividing the cross sectional areas of the total number of stone columns by the total cross sectional area of the site to be improved. Common area replacement ratios range from 15 to 35 percent, but the ratio is site specific depending on the soil and structural loading conditions. Figure 7 shows the effect of area ratio, calculated by dividing the total cross sectional area by the cross sectional area of the stone columns, on the ground improvement presented as improvement factor, *n* (Priebe 1995). As the area replacement ratio increases, i.e. the area ratio decreases, the improvement factor increases as shown in Figure 7.

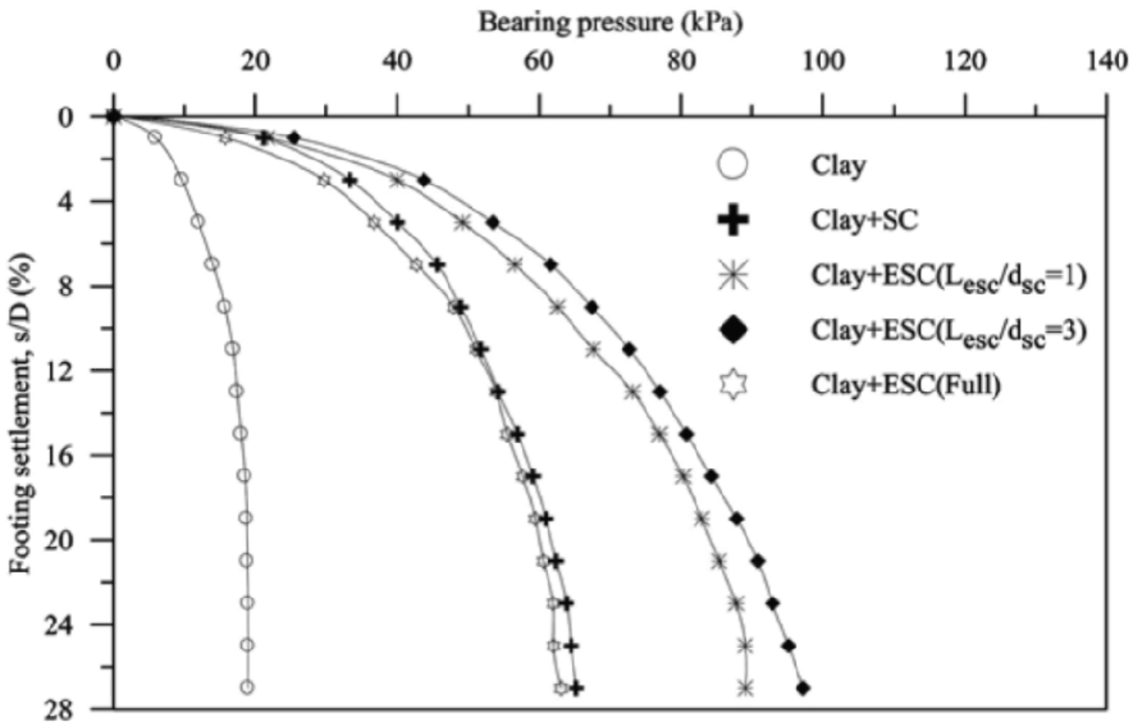


Figure 6. Influence of geosynthetic encasement on the performance of stone columns floating in soft clay (Glover 1985) (Note: “Clay+SC” for no encasement, “Clay+ESC(L_{esc}/d_{sc})” for partial encasement, and “Clay+ESC(Full)” for full encasement)

Advantages:

The vibro-replacement stone column method has been proven on numerous project sites and conditions and has gained widespread acceptance in the United States (Munfakh et al. 1984, Nayak 1985, Mitchell and Huber 1985). Transportation related projects have used stone columns as embankment fill support for highways, interchanges, and bridge approaches as well as ground improvement for a hospitality station and box culvert (Barksdale and Bachus 1983). The method not only efficiently installs strong stone column elements throughout the site, some improvement of the surrounding ground can also be anticipated depending on the soil conditions. If freedraining aggregate materials are used to construct the stone columns, they can also provide quick dissipation of excess pore water pressure in fine-grained soils. The drainage path lengths are reduced resulting in reduced primary consolidation settlement times. The vibro-replacement stone columns were reported to be more cost effective than piles and drilled piers for a 150,000 square foot department store located in Cypress, California for static settlement reduction (Lopez and Shao 2007). Davis and Roux (1997) reported significant cost savings in soft clays to limit the differential settlement of a water storage tank in Los Angeles, California. Costs for stone column installation measured 43 percent of the cost of lime-columns, 43 percent of jet grouting, 38 percent of compaction grouting, 24 percent of removal and recompacting, 21 percent of driven piles and grade beams, and 17 percent of drilled piles and grade beams.

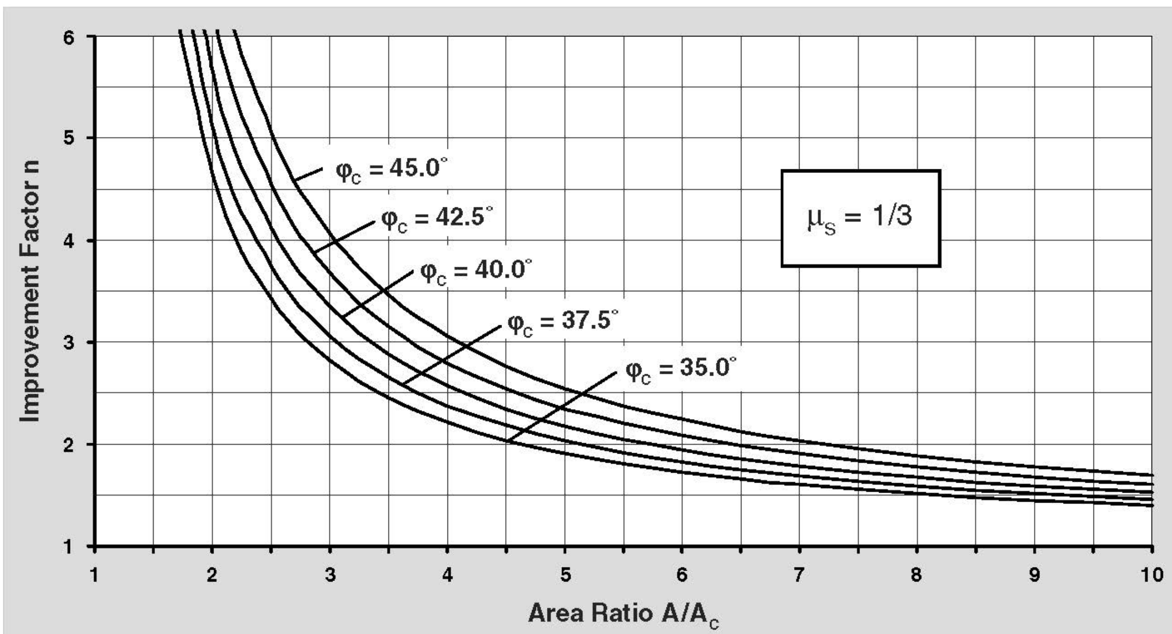


Figure 7. Design chart for vibro-replacement stone columns (Priebe 1995)

Limitations:

As with most ground improvement or deep foundation methods it is critical to implement a vibro-replacement ground improvement for appropriate site conditions. When the site soils contain more than 15 to 25 percent fines or greater than 2 percent clay, the density of the soil between stone columns is not greatly improved (Soydemir et al. 1997, Chen and Bailey 2004). Caution should be used when vibro stone columns are used in very soft, compressible soil, and/or decomposable organic materials, because of the lack of lateral support provided by these material types (Barksdale and Bachus 1983). Also, if a dense overlying stratum is not present, close quality control should be followed to ensure heaving and/or radial cracking does not occur. In one instance ground heave of 1 m (3 ft) was experienced during the installation of stone columns in compressible clay and silt for an embankment support in Iowa (White et al. 2002). If organic other compressible soils will be encountered, consideration should be given to using the vibro concrete column method discussed below. It is also necessary to perform extensive pre and post stone column construction testing to confirm that the assumed benchmarks have been achieved.

2.2.3 Vibro Concrete Columns

The vibro concrete column method was derived from the vibroflotation and vibro-replacement stone column methods. It was developed to adapt the vibro technique to sites where very soft and compressible cohesive soils or organic soils could not provide the lateral support necessary to adequately densify the stone column during installation or for long term support. Originally developed in 1976, vibro concrete columns offer an alternative to conventional ground improvement methods as well as deep foundation methods such as piles or piers. The development of the use of concrete in the vibrocompaction process provides numerous other applications that were not suitable prior to its development. Numerous transportation-related projects have successfully utilized vibro concrete columns for construction including embankments over organic silt and peat, bridge approach fills through existing fills, loose sands, and organic deposits (Mankbadi et al. 2004, Zamiskie et al. 2004).

Construction Process, Fundamentals, and Techniques:

As in the vibro-replacement stone column process, a crane supported vibratory probe is inserted into the ground under the weight of the equipment and/or vibration forces. The vibrator extends to a suitable end bearing soil layer, or to a loose granular soil layer that can be compacted using the vibratory process in order to create a suitable end bearing. The vibro concrete column process offers the additional advantage of creating an enlarged base of concrete which is created by inserting the concrete under pressure via the bottom feed vibrator. A conventional plastic concrete mixture can be used, or a dry concrete mixture can be utilized which is known as the dry vibro concrete method. After the initial lift of concrete is inserted, measuring approximately 0.6 m (2 ft) in thickness, the probe is reinserted into the concrete lift and the vibration process is initiated to force the plastic concrete or dry cement and stone material radially outward, compacting or laterally prestressing the surrounding soil. The vibratory equipment is subsequently raised to the surface in a slow consistent process while the concrete is inserted under high pressure creating a consistent concrete column up to the ground surface. Figure 8 shows the schematic of vibro concrete column installation process.

The three fundamental benefits of vibro concrete columns are the ability to improve granular soil layers when encountered, while carrying loads through very soft or organic soil layers via the concrete column, and having the ability to easily enlarge the base or near surface column without additional special equipment. The dry vibro concrete column process, which uses a combination of dry cement and aggregate, provides superior compaction of the surrounding soil matrix when compared to the traditional vibro concrete columns with plastic concrete mixtures. The vibro concrete columns combine these benefits into one process which can be conducted quickly on projects. The design components, i.e., skin friction and end bearing, used to analyze the resultant concrete columns are similar to conventional piles. When the loading will be placed over a large area it is sometimes beneficial to construct either a conventional pile cap or a load transfer platform to help evenly distribute the loads and reduce differential settlements that can result between the stiff concrete columns and relatively weak surrounding soil.

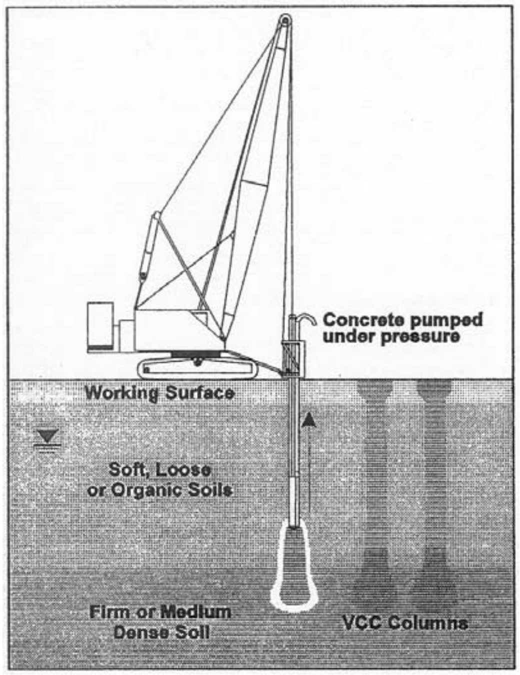


Figure 8. Vibro concrete column installation (Schaefer et al. 1997)

Advantages:

Vibro concrete columns allow for a strong pile-type column to be constructed in the same ground improvement vibratory process of vibro-stone columns. They can be utilized on project sites with compressible very soft clay or organic soil where other similar vibratory methods were previously not suitable. When the dry vibro concrete column process is performed, densification and lateral prestressing of the surrounding soil is accomplished similar to vibro-stone column methods. Unlike with traditional piles or piers, a granular layer can be improved to be a suitable end bearing stratum. The vibro concrete column method also allows for enlarged column sections without any additional or specialized equipment, which is especially beneficial near the surface where differential settlements are a concern.

Limitations:

The adverse effect of a strong vertical concrete element is that differential settlements can occur at the surface due to the different support characteristics when compared to the surrounding very soft soil. This consequence is amplified since the surrounding very soft or organic soils are not appreciably improved by the vibration approach. As a result, it may be necessary to construct a traditional pile cap or other type of load transfer platform (Mankbadi et al. 2008) which will carry the loading to the structurally superior columns. Camp and Siegel

(2007) reported on the failure of a vibro concrete column supported embankment, when up to 50 mm (2 in.) differential settlements were experienced at the ground surface due to an under-designed load transfer platform. The failure resulted in a complete reconstruction of the embankment with a more traditional pile-supported structural slab design.

2.2.4 Deep Soil Mixing

The adaptation of lime or cement stabilization of soft subgrade soils has gained widespread acceptance throughout the construction industry, the transportation industry, as well as within the state of Ohio. The deep mixing method utilizes the same benefits of cementitious additives to subgrade soils, and changes the incorporation method from a near surface alternative to a vertical or "deep" alternative. The term deep mixing is used to encompass numerous terminologies such as deep soil mixing, deep jet mixing, and deep mixing method. The nomenclature deep mixing applies to deep ground improvement using mechanical shafts or augers. Although its origins can be traced to the United States in 1954, research and development of the modern methods on deep mixing originated in Japan around 1967 as a potential method to stabilize soft or loose marine soils below the water level (Schaefer et al. 1997). Large scale testing and construction continued around 1974-1976 and coincided with use in the United States and Europe (primarily Scandinavia) during the 1970's. While originally envisioned for marine based projects, the methods were quickly deemed to be suitable for use on land and proved to be a quick and efficient way to stabilize large areas of soft soil.

Archeewa et al. (2011) researched the use of the deep soil mixing method to reduce bridge approach settlements due to soft clay foundation soils. Up to 8.0 m (26 ft) high new embankment was supported by 1.2 m (3.9 ft) diameter, 8.3 m (27 ft) long deep soil mixing columns. The use of deep soil mixing columns successfully limited the settlement of the new bridge approach fill to approximately 3.0 mm (0.12 in).

Construction Process, Fundamentals, and Techniques:

The deep mixing process can be executed with either mixing paddles, partial augers, or other mixing apparatus. Different equipment can have varying numbers of mixing shafts (Figure 9) for more efficient mass stabilization or accurate stabilization, depending on the project needs. The area of each shaft typically overlaps the adjacent shaft and subsequent strokes of the panels can be overlapped as well to create a combined stabilized mass. The addition of a lime or cement in dry or wet slurry form (Liu et al. 2007) generally produces a soil-cement column element with greater strength, lower permeability, and decreased compressibility. The deep wet mixing method utilizes a cement or lime slurry, whereas the dry jet mixing method transmits a dry cement or lime powder to the soil by air pressure. Deep mixing column diameters typically range from 0.6 to 1.5 m (2 to 5 ft) and can be extended to depths over 30 m (100 ft) (Bruce 2000) and can even be performed underwater from barge platforms.

The end result of the deep mixing process is numerous soil-cement columns with improved strength and settlement characteristics. The deep mixing process also can potentially improve drainage properties in clays. In one case the hydraulic conductivity of a marine clay was increased 100 times its original state (Shen et al. 2008). The ground surrounding the columns are not affected, however, it is possible to overlap the columns in order to create solid mass stabilized areas. The type and quantity of chemical added is dependent on the soil types with lime-cement mixtures being the most common. The type of chemical chosen can also vary based on the primary component of the soil whether peat, other organic soil, soft silt/clay, or predominantly coarse grained material. The deep soil mixing construction process is shown in Figure 10.

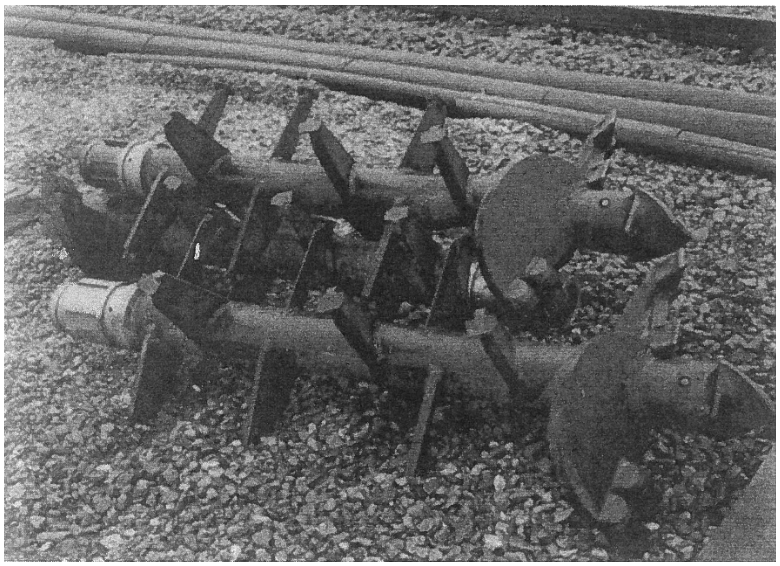


Figure 9. Deep mixing tool (Vriend et al. 2001)

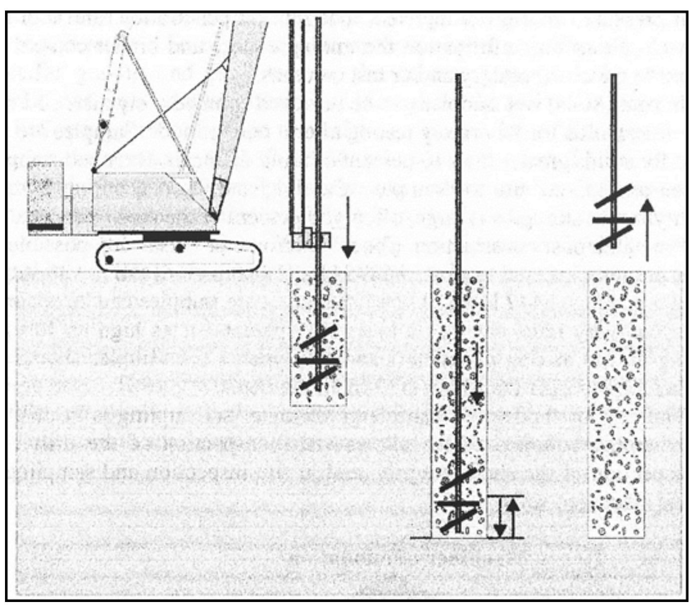


Figure 10. Deep soil mixing process (Burke et al. 2001)

Advantages:

Deep soil mixing can be effective for soils above or below the groundwater table, and a wide range of soil types such as organics, fine-grained, and coarse grained materials can be improved. Single soil-cement columns can be constructed in a grid pattern, or overlapping panels of multiple columns can be performed to suit the needs of the project. Appropriate equipment selection can result in little to no spoils to remove from the process and vibrations are limited due to the nature of the mixing action. Since lime and cement additives are commonly used for numerous other applications, the products are usually easily available. Dasenbrock (2004, 2005) reported cost savings of approximately 12 percent (or $500,000) for deep mixing method columns when compared to driven piles and associated mat foundation for embankment and approach fill support at a new interchange project near the Minneapolis/St. Paul area. Deep soil mixing columns are very well established in practice and have been constructed on transportation projects throughout the United States including bridge approaches and abutments, tunnel support, dewatering applications and test embankments with MSE walls (Lambrechts and Roy 1997, Stewart et al. 2004, Miki and Nozu 2004, O'Rourke and McGinn 2004, Meyersohn 2007, Olsson et al. 2009, Jameson et al. 2010, Archeewa et al. 2011, Yang et al. 2011). The Scandinavian countries of Sweden and Norway have also used deep soil mixing on transportation related projects such as roadway embankments, railway embankments (Esrig et al. 2003), and other trial embankments with successful performance in limiting settlements (Jelisic and Leppanen 2003).

Limitations:

Although the deep mixing method has been utilized successfully on numerous projects throughout the United States, most of the specialized deep mixing rigs and apparatus are located on the east and west coasts. The limited availability and resultant high mobilization costs could prove costly when smaller stabilized areas are required (Bruce 2000). Very dense soil, very stiff soil, or soils containing boulders can pose problems for the deep mixing rigs. Rigorous testing programs must also be executed during and after installation in order to confirm the properties of the improved soil-cement columns (Puppala et al. 2004). Deep mixing methods also may require a curing time to allow the improved soil to reach its desired strength. One study conducted where slurry deep soil mixing columns were installed in a marine clay, required 10 days of curing just to return to the clay to its original strength (Shen et al. 2008). The clay continued to gain strength up to 70 days at which it had increased in strength 50% of its original strength. Since soil types/properties, water contents, and organic contents affect strength and settlement characteristics, varying performance can be experienced when compared to the well-known properties of a steel/concrete pier/pile or grouted inclusion.

2.2.5 Jet Grouting (Soil Jetting)

Jet grouting, also known as soil jetting, creates a similar stiff soil column as in the deep soil mixing method, however the two ground improvement methods are substantially different in most other aspects. The term jetting is used because the system relies on hydraulic energy to force grouting fluid/air/water through small nozzles near the end of the apparatus. Jet grouting was conceived in Japan in the early 1970's and its use has since expanded throughout most of the world and in the United States specifically since the 1980's. It has been used for numerous applications including excavation support, underpinning, piling, anchoring, tunnel stabilization, groundwater control, ground stabilization, and environmental applications (Gallavresi 1992, Byle and Haider 1998, Pinto et al. 2003).

Construction Process, Fundamentals, and Techniques:

Jet grouting equipment ranges in size from large crane supported systems to small electric units which allow for the numerous applications and uses. The jet grout columns can be installed at an angle to reach locations that are not accessible to other methods. The jet grouting process begins with a drill bit extended to the design depth via traditional rotary methods. Once the design depth is encountered, single (grout), double (grout and air), or triple systems (grout, air, and water) (Figure 11) are injected at high pressure through the nozzles near the end of the drill bit and either rotated or held stationary while the apparatus is slowly raised to the surface (Figure 12). A portion of the eroded soil is carried to the surface through the annulus of the drill bit, while the remaining soil is mixed with the grouting fluid to create the soil-grout column element. The resultant soil-grout column element is commonly referred to as a soilcrete element. Lift speed, rotational speed, and the slurry makeup are vital aspects to monitor and maintain during the installation procedures. Expected soilcrete column diameters vary based upon soil types and specific method used, but generally range from 0.5 to 5 m (1.5 to 16 ft).

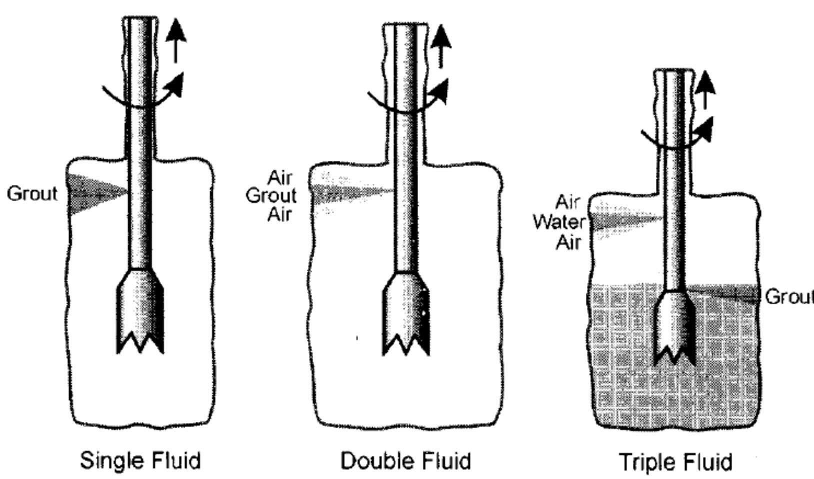


Figure 11. Basic jet grouting systems (Burke et al. 2000)

Care must be taken to maintain and monitor the pressures during installation. When they are not properly maintained, excessive pressures can lead to hydro-fracturing of soft soil. In addition to creating a flawed soilcrete element, hydrofracturing can cause ground heave at the surface near the area of improvement. The parameters of the soilcrete element are usually tested with wet grab samples shortly after installation and/or core samples after the soilcrete has been allowed to properly cure. The compressive strength of the soilcrete column element varies by soil type, chemical type, and addition rate but is generally displayed in Figure 13.

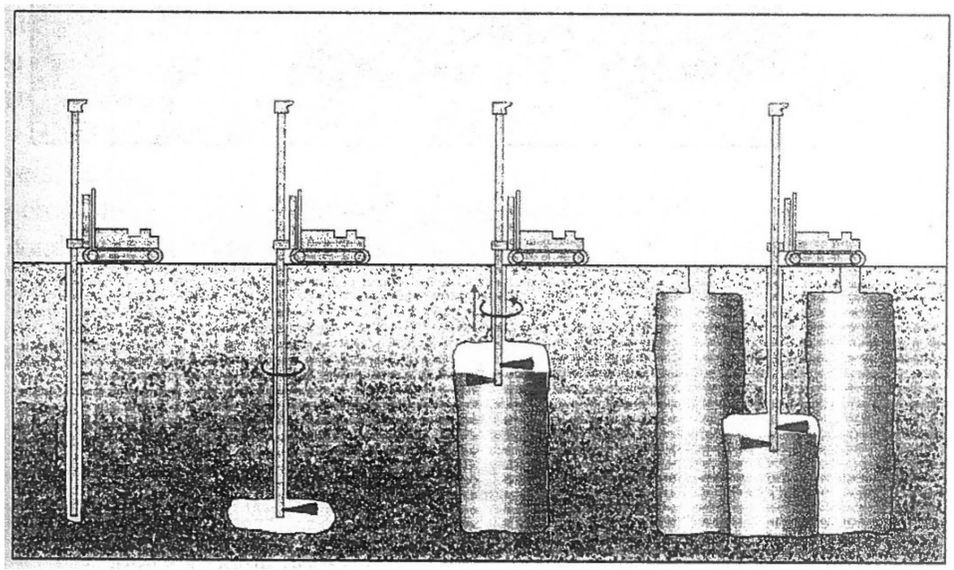


Figure 12. Jet grouting process (Burke 2004)

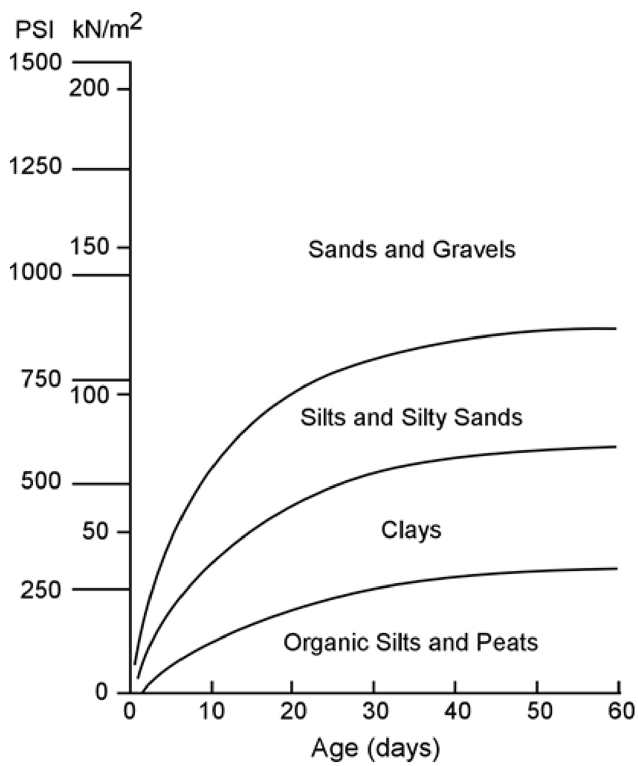


Figure 13. Typical soilcrete strengths by soil type (reproduced from Burke 2004)

Advantages:

The range of equipment sizes and variability of the construction process allow jet grouting to be used in a variety of conditions and projects. The ability to angle the equipment and lack of large vibratory forces in the process also allow for use nearby existing structures where vibrations could be of concern. Jet grouting is also applicable to all soil types including gravel, sand, silt, clay, and organics (Collotta et al. 2004). In addition to the creation of high strength soilcrete columns, Ho et al. (2001) reported strength gain in soft marine clays extending several meters beyond the limits of the treated zone due to the generation of heat from the soil-cement hydration process. The flexibility of jet grouting also allows for large diameter soilcrete elements to be constructed and even increased in size near the ground surface to assist in transferring the loads to the stiff elements, i.e. to act as a load transfer platform.

Jet grouting is an established technology that has been successfully used on numerous projects, some of which are referenced below. Chen et al. (2011) reported the use of jet grouted columns to support new interstate highway embankment loads in silty clays, high plasticity clays, and organic high plasticity clays in Hawaii. Jet grouting has also been successfully used to reduce settlement and increase support for a temporary railroad bridge abutment, as part of the Massachusetts Central Artery/Tunnel project (Maswoswe and Druss 2001). The opposite bridge abutment and four piers were supported by conventional drilled shafts, as originally designed. Refusal was encountered upon boulders within the upper fill materials at certain location during drilled shaft installation. Due to significant schedule constraints, jet grouting was selected since the system was already being used on other portions of the project. Where the drilled piers extended to bedrock at depths of approximately 33 m (110 ft), the soilcrete columns were terminated upon much shallower clay soil at a depth of 13 m (42 ft). The aforementioned fill and underlying organics were jet grouted and successfully limited settlement below the allowable design limit of 15 mm (0.6 in.) (Figure 14).

Limitations:

Erosion of the underlying soil is one of the key components to the soil jetting process. Clays with high cohesion characteristics can resist the erosion forces and clog the spoil returns, resulting in variable pressures, grout slurry quality, and element geometry. On the other end of the particle size spectrum, the up-hole velocities of the process are generally not capable of ejecting particles larger than sand size. The potential for heave at the ground surface can also be of concern, if there are nearby structures which are sensitive to heaving forces. Variability in soil strata can cause inconsistency in the soilcrete quality and geometry, therefore careful quality control (or acceptance of a variable end product) should be planned for jet grouting operations in changing soils.

2.2.6 Compaction Grouting

Compaction grouting has been used in the United States since the 1950's, and has gained widespread use and acceptance during that time based upon successful performance on numerous projects. There are various uses for compaction grouting such as conventional increase in bearing capacity and settlement control (Chastanet and Blakita 1992, El-Kelesh and Matsui 2002, Kummerer et al. 2003), karst formation stabilization (Warner et al. 2003), mitigation of liquefaction potential (Ivanetich et al. 2000), retaining wall repair (Byle 1992), and lifting of structures (Boghart et al. 2003, Strauss et al. 2004).

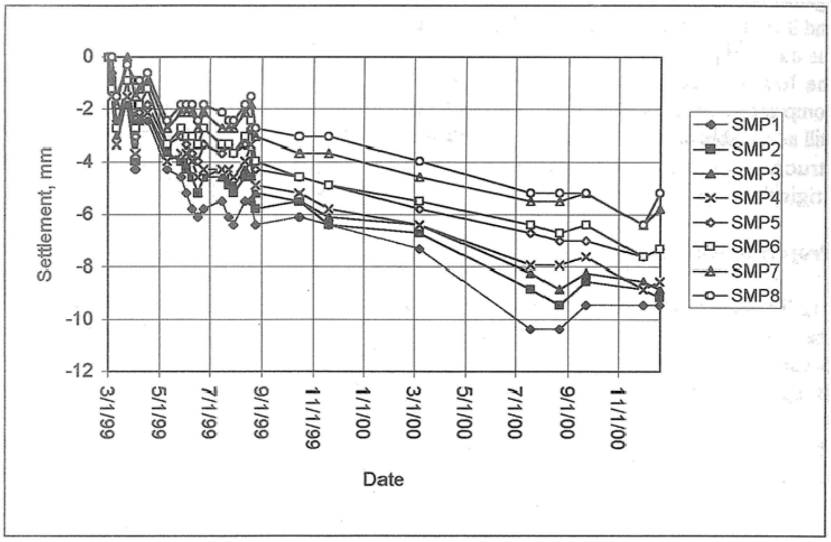


Figure 14. Settlement measurements of temporary railroad bridge abutment (Maswoswe and Druss 2001)

Construction Process, Fundamentals, and Techniques:

Compaction grouting was defined as “Grout injected with less than 25 mm (1 in.) slump. Normally a soil-cement with sufficient silt sizes to provide plasticity together with sufficient sand sizes to develop internal friction. The grout does not enter the soil pores, but remains in a homogeneous mass that gives controlled displacement to compact loose soils, gives controlled displacement for lifting structures, or both” (Brown and Warner 1973). The process begins with a casing that is drilled or driven to the desired depth. The grout is injected under high pressure until the refusal criteria have been met such as grout volume, injection pressure, or ground heave. The compaction grouting process is shown in Figure 15.

Both “top down” and “bottom up” methods can be used depending on the site conditions and design. The casing is extracted/lowered to the next lift, generally measuring less than 2.1 m (7 ft), and the above steps are repeated until the upper/lower limit of the treatment zone is achieved. While the grout intervals form bulbs in weaker soil, they have been shown to be cylindrical in shape in uniform soil conditions. The resultant composite soil-column system is improved with the inclusion of grout columns and compacted soft or loose soil layers surrounding the grouted column elements. El-Kalesh et al. (2012) reported significant increases in SPT N-values where compaction grouting was used in fine sand, silty sand, and sandy silt for field testing of ground improvement at the Tokyo International Airport (Figure 16).

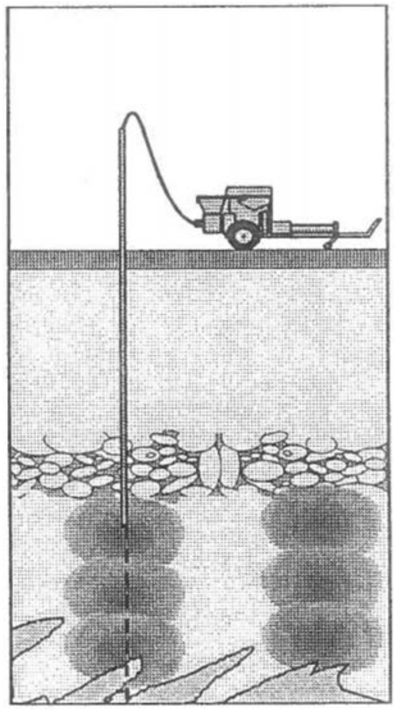


Figure 15. Compaction grout bulb construction (Schaefer et al. 1997)

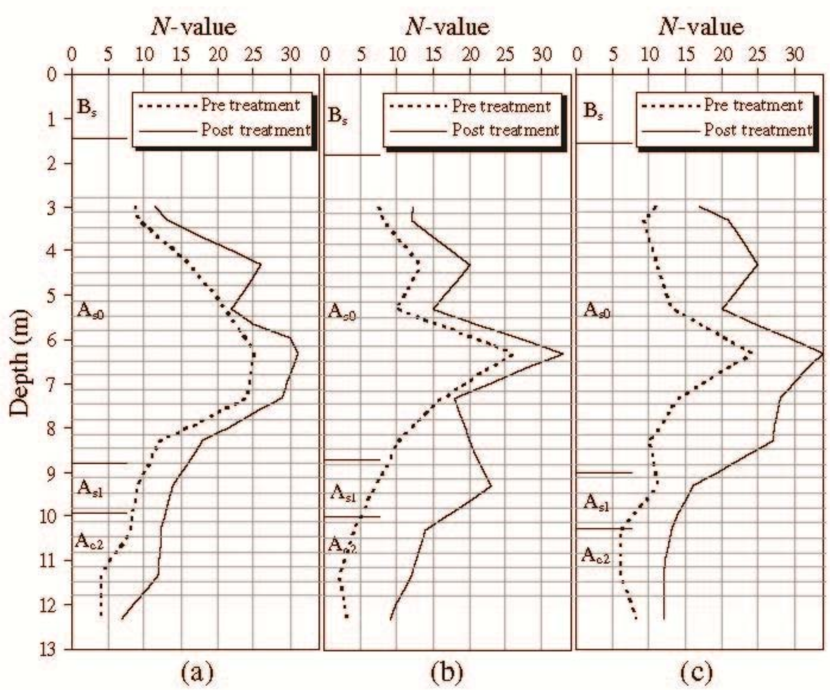


Figure 16. SPT-N values of SPT tests for a compaction grouting field test (El-Kalesh et al. 2012)

Conventional design approaches conservatively assume the volume of injected grout is directly related to the volume of void reduction in the surrounding soil. Since the grout compresses the soil laterally, the grout injection rate must be controlled in order not to exceed the soil's ability to dissipate the resultant increase in pore water pressure. The control of the injection rate is most important in soil with high fine contents and soil below the groundwater table. The high pressure operation also relies on overburden soil pressure to resist the upward forces acting within the grout bulb. As a result, compaction grouting near the ground surface can prove problematic, and sometimes result in upward ground heave. The hydro fracturing and resultant ground heave can prove beneficial for raising settled structures and reducing future settlements, when performed in a controlled manner.

Advantages:

Compaction grouting can be extended to depths measuring up to 120 m (400 ft) below the ground surface, which exceeds the capabilities of most other ground improvement methods. The method of ground improvement in the surrounding soil is based upon lateral spreading of the highly pressurized grout. Therefore, the compaction can be achieved in both granular and some cohesive soils. Since the vibrations are limited, this method is also a popular choice for use around existing structures. Sophisticated methods of monitoring and implementation also allow for controlled conditions for raising existing structures and customizing the vertical column element geometries as needed. In addition, the equipment to perform compaction grouting is readily available and is generally cost effective when compared to the mobilization of heavy equipment required for other ground improvement or deep foundation methods. Although the actual dollar figures were not provided, Oakland and Bachand (2003) reported that compaction grouting was significantly less expensive than piles, soil mixing, and jet grouting for ground improvement during the expansion of a water treatment plant in South Carolina.

Limitations:

Abundant research has not been performed on compaction grouting and most design methods are largely empirical. Although some successful implementations have been performed and documented, the design of compaction grouted projects relies largely on experience and requires careful monitoring and extensive testing before, during, and after construction. Some ground heaving can be experienced when the installation methods and procedures are not customized to the in-situ soils, or where the soils vary greatly with depth. Compaction grouting is especially susceptible to ground heaving near the ground surface where large overburden pressures are not present to resist the grouting pressures. High fine-grained soil contents can lead to hydro fracturing under the high pressures, and conversely large coarse-grained soils with high void contents can allow the grout to fill the pores instead of densifying the surrounding soil. Brown and Warner (1973) and Ivanetich et al. (2000) reported that, nominal ground improvement was achieved in the upper soils, specifically in the upper 3.0 m (10 ft), due to the limited overburden pressures and only moderate increases in density were achieved in fine grained soils.

2.2.7 Controlled Modulus Columns

Controlled modulus columns are a relatively new method of ground improvement. The process results in a composite system of improved ground. The system was developed in France in the 1990s and has been used in Europe since that time to support relatively lightly loaded structures such as railway and road embankments (Plomteux et al. 2004, Lacazedieu et al. 2005).

Pearlman and Porbaha (2006) conducted a study on the design and monitoring of an embankment on controlled modulus columns. The project involved the construction of a new railway embankment with up to approximately 7.5 m (25 ft) of new earth fill. The problem soils at the site consisted of soft clay and peat which extended to depths ranging from about 6 to 11 m (20 to 36 ft). Controlled modulus column elements were installed to depths of 10 to 11 m (33 to 36 ft) with a design area replacement ratio of 13.9 percent. The finite element model prepared prior to construction predicted settlements on the order of 32 to 34 mm (1.26 to 1.34 in); however, actual settlement measurements recorded under design loads were 10 mm (0.39 in), or approximately one third of the original estimate.

Construction Process, Fundamentals, and Techniques:

Controlled modulus columns offer another ground improvement alternative to a similar set of fundamental concepts. Extending loads to deeper underlying dense or hard soil layers while improving the upper soft or loose layer through lateral displacement and compaction are concepts also included in the vibro-replacement stone column, vibro concrete column, and compaction grouting methods discussed previously. However, the key differentiating factor with controlled modulus columns is the way in which lateral displacement and compaction is achieved. Other methods rely on horizontal vibration, compaction, or high grout pressures during placement, whereas controlled modulus columns achieve the lateral displacement through the specially designed auger and initial augering process. Figure 17 shows the schematic of controlled modulus column installation process.

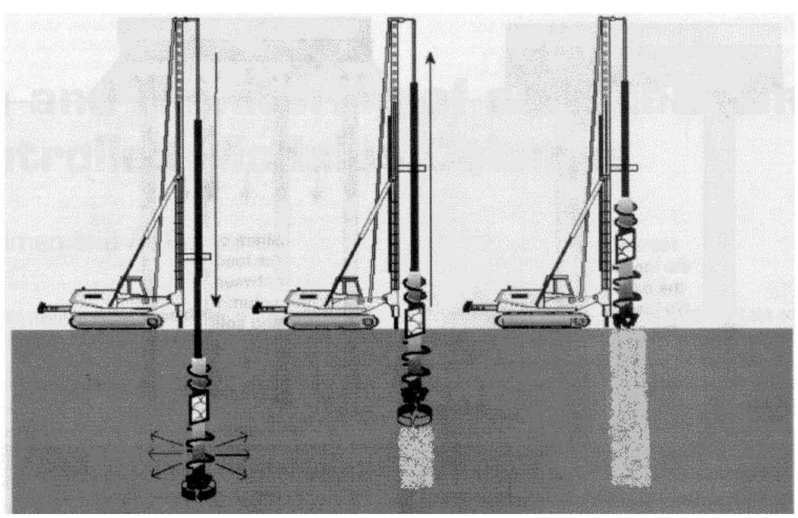


Figure 17. Controlled modulus column installation (Pearlman and Porbaha 2006)

The controlled modulus column process is initiated when the specially designed auger is extended with high torque and high downward forces to the design depth or until predetermined drilling criteria, such as drilling torque, is achieved. The grouting process is then started through the drill shafts to the bottom of the hollow auger where the grout-cement mixture is placed at a relatively low pressure, less than 1,000 kPa (145 psi). The grout placement is continued while the augers are raised to the surface and the upper auger flights, which are designed in the reverse direction, prevent the surrounding soil from losing the recently achieved compaction. It is also important to note that the surrounding soil is not mixed with the grout column element, rather it is laterally displaced during the downstroke and maintained in place during the upstroke or grouting process by the specially designed augers. Column diameters typically range from 300 to 500 mm (12 to 20 in.), installed at a center to center spacing of 1.2 to 3.0 m (4 to 10 ft), and can be extended to depths of approximately 23 m (75 ft) below the ground surface.

Advantages:

Controlled modulus columns can be implemented on a wide range of project sites with varying soil conditions. Very soft to soft silt/clay soils, organic soils, and loose fine sands can all be improved with controlled modulus columns even when they extend below the groundwater table. The process creates no spoils and almost no vibration which is helpful for environmentally sensitive sites or where construction occurs nearby existing vibration sensitive structures. The resulting composite ground system of grout column inclusions and improved surrounding soil offers the load capacity benefits of traditional piles/piers while improving the surrounding soil at the same time. Documented successful implementation was also performed by Pearlman and Porbaha (2006) who performed a study of controlled modulus columns to support an access road embankment constructed over very soft clay and peat soils. The controlled modulus columns were selected over a combined vertical wick drain and surcharge solution to reduce predicted settlement due to time constraints and over stone columns due to lower long term settlement predictions and potential stone column bulging problems in the very soft clay and peat layers. Post construction load tests confirmed the columns limited settlements to 10 mm (0.4 in.) under the design loads.

Limitations:

Some case studies have been performed on controlled modulus column sites; however, since the process is relatively new the amount of research information and documentation of proven success is not as extensive as other methods. The stiff grout column elements surrounded by the soft compressible soil require the construction of a load transfer platform, typically with layers of select fill and geogrid.

2.2.8 Sand Columns

The general term “sand column” encompasses several specific methods of ground improvement including vertical sand drains, sand compaction piles, and geotextile encased sand columns. Originating in the early 1900’s as a method for increasing drainage and decreasing settlement times, modifications and improvements of the sand column method, such as encasing in geotextiles, has led to a wide range of applications and functions. The sand compaction pile method was developed in Japan in 1956 as a treatment for soft marine soils and has continued to be developed and researched in Japan and other Asian countries (Yea and Kim 2010). Sand compaction piles have not seen widespread use in the United States to date, most likely due to the development of vibro-replacement stone columns and rammed aggregate piers.

Construction Process, Fundamentals, and Techniques:

Sand columns are most frequently used in applications with very soft silt/clay or organic soil where consolidation of the layers are a function of moisture content and the dissipation of pore water pressures. When placed in a grid pattern within a highly compressible soil, the length of the drainage path can be greatly reduced and provides some consolidation advantages. However, Sasaki (1985) summarized reports by others concluding that the installation of vertical sand drains were barely effective in accelerating settlement with the quantifiable values ranging from 2 to 4 percent improvement, and also stated that the primary benefit came from strengthening of the embankments rather than any drainage enhancement. These limited advantages were improved with the development of other ground improvement methods. For example, the sand compaction pile method combines the decreased drainage path distance with some of the benefits of vibro-replacement stone columns. The encasement of sand columns with geotextiles allows for higher structural loads to be carried by the sand columns and transmitted to underlying layers with greater strengths.

There are several sand column installation methods and techniques. Basic sand columns can be installed via the replacement method whereby very soft soils are removed by augering and replaced with the sand column inclusion. Sand compaction piles are placed by a specially designed vibratory or driven probe which is inserted into the ground. The probe carries the sand to the tip with water or air pressures and laterally compacts the resultant sand column in lifts with horizontal vibration forces. Since geotextile encased sand columns are placed in very soft cohesive or organic soils they can be placed by displacement, replacement, or even vibratory pile-type installation procedures.

Advantages:

Sand columns are best suited for very soft silt/clay, organic, or peat conditions and can be used above or below the groundwater level. Installation of sand columns allows for increased time rate consolidation of cohesive or organic soil, provides enhanced drainage, and can function as strong column elements transmitting loads to underlying stronger soil layers (Yoshitomi et al. 2007). Sand and geotextile materials are generally accessible and familiar to the construction industry.

Limitations:

Sand columns are generally not applicable to coarse grained soil conditions and are limited in scope to sites with very soft to soft clay/silt or organic soils. While sand columns have been shown to provide improved lateral spreading support on sites with organic soil, the improvement in consolidation characteristics is somewhat limited (Sasaki 1985). The sand compaction pile method can provide greater ground improvement characteristics than traditional vertical sand columns; however, their use has been largely limited to Japan and other Asian countries limiting the accessibility of this method in the United States. Geotextile encased sand columns are a relatively new method of ground improvement which greatly limits the amount of research performed and the number of successful case histories available. The primary limitation to the sand column methods in the United States is the widespread use of vibro-replacement stone columns and rammed aggregate piers which can provide similar drainage enhancements while providing much stronger vertical stone column elements for superior load support and settlement reduction.

2.2.9 Rammed Aggregate Piers

Rammed aggregate piers were developed in the 1980's in the United States as a method of ground improvement for lightly to moderately loaded structures. Over time, successful project performance and increased acceptance has led to use on a wide variety of project types and loading conditions including retail/commercial projects (Sheu et al. 2007), earth embankments (White and Suleiman 2004, Morales et al. 2011), mechanically stabilized earth walls (Thompson et al. 2009), retaining walls (Wong et al. 2004), railroad embankments (Carchedi et al. 2006), slope stabilization (Parra et al. 2007), and box culverts (White and Hoevelkamp 2004). The unique site improvement characteristics and numerous applications lie with the specific installation process and techniques.

Aggregate piers are often designed to deform in bulging near the top of the pier. Earlier studies show that rammed aggregate piers with length to diameter ratios greater than 3.5 are more likely to deform in bulging, whereas tip deformation is more likely to occur with smaller ratios (Wissmann et al. 2000). Pitt et al. (2003) completed a research report evaluating the use of rammed aggregate piers in transportation related construction. The study focused on three projects including the support of new embankment fill, bridge approach fill, and a combination of box culvert and embankment fill. Problem soils consisted of fill and soft cohesive soil extending to depths ranging from 4 to 14 m (13.1 to 45.9 m). The implementation of rammed aggregate piers successfully limited settlement of the embankment and structures. In the first case study the prior installation of stone columns allowed for a comparative analysis between the two methods. In addition, full-scale load tests performed show that rammed aggregate piers have stiffness 5 to 10 times greater than vibro-replacement stone columns in the same soil as shown in Figure 18.

Construction Process, Fundamentals, and Techniques:

The installation process begins with the augering of the cavity for rammed aggregate pier installation. The cavity can typically be drilled to depths up to 9.0 m (30 ft) below the ground surface at diameters ranging from approximately 600 to 900 mm (24 to 36 in.). A bottom bulb is formed by placing a poorly graded crushed gravel material into the cavity and compacting with the specially designed beveled tamper which vibrates with downward vertical force but also laterally displaces the stone. The process is continued in approximate 300 mm (1.0 ft) lifts with the continued use of open graded material, or more commonly, well graded highway aggregate base type material. Once the ground surface is reached a bulbous shaped vertical aggregate column extends to the desired depth. The schematic of the rammed aggregate pier installation process is shown in Figure 19.

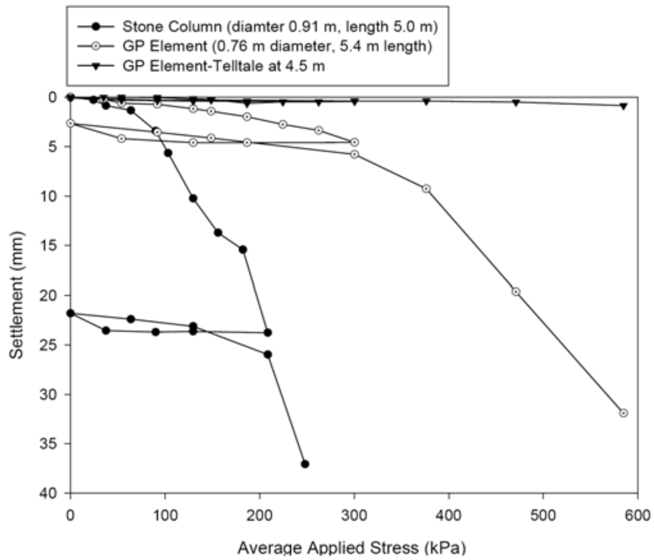


Figure 18. Comparative stress-deformation plot for rammed aggregate piers (GP elements) and stone columns (Pitt et al. 2003)

In addition to the inclusion of an aggregate column, the surrounding soil is displaced laterally and compacted by the outward angle of the beveled tamper forcing the aggregate against the soil. When open graded stone is used, the piers act as a drainage path for rapid dissipation of excess pore water pressure. The lateral pre-stressing of the soil matrix is effective in reducing future settlements (Handy 2001) and the aggregate piers have been shown to have very high angles of internal friction, between 48 and 52 degrees (Pitt et al. 2003), due to the high level of compaction achieved. As a result, the improved composite system allows for the piers to be terminated within soft compressible soils as opposed to extending to underlying stiff or dense soils as required in some of the other methods.

The resulting system of aggregate piers and improved soil are viewed as a composite system, with a composite friction angle, ϕ_{comp}, and composite soil cohesion, c_{comp}, as shown in the below equations (Barksdale and Bachus 1983):

$$\phi_{\text{comp}} = \arctan[R_a \tan\phi_g + (1 - R_a) \tan\phi_m] \quad (1)$$

$$c_{\text{comp}} = (1 - R_a)\, c_m \quad (2)$$

where R_a is the area replacement ratio, ϕ_g is the rammed aggregate pier friction angle, ϕ_m is the matrix soil friction angle, and c_m is the cohesion value of the matrix soil. Since the rammed aggregate piers are constructed of granular material, there is no cohesion input for the rammed aggregate pier within the composite cohesion equation.

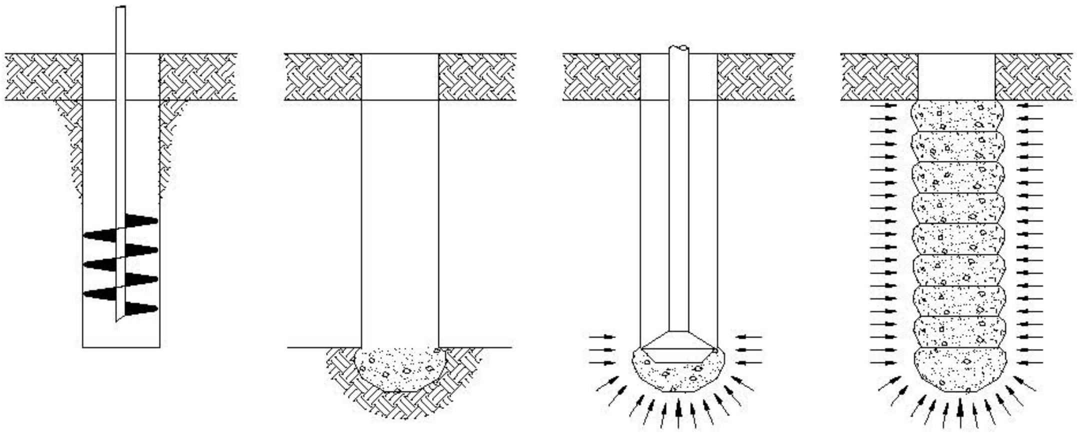


Figure 19. Rammed aggregate pier installation: Augering of cavity, insertion of aggregate, tamping/compaction, and placing and compacting aggregate in lifts (Pitt et al. 2003)

In regards to limiting settlement of existing roadways, the settlement of the upper rammed aggregate pier composite zone is limited by introducing a stiffer vertical element within the weaker soil. Therefore the settlement in the reinforced zone can be calculated utilizing a spring analogy based upon the area replacement ratio, R_a, beneath the footing, the stiffness ratio of rammed aggregate pier element to surrounding soil, R_s, the rammed aggregate pier stiffness modulus, k_g, and the bearing pressure q. The following equation (Lawton and Fox 1994) is used to determine settlement, s, in the upper zone:

$$s = [q\, R_s / (R_a\, R_s + 1 - R_a)]/k_g \tag{3}$$

While the aforementioned settlement equation would not directly apply to a roadway or embankment loading situation, the principles of the spring analogy within the upper zone would still confirm a decrease in settlement with the installation of rammed aggregate piers.

Advantages:

The flexibility and widespread applicability of rammed aggregate piers provides a strong background for many soil types and project uses. Rammed aggregate piers have been proven successful in increasing bearing capacities and controlling settlements in soft silt/clays, organic soils, and granular soils. The ability to use well graded crushed aggregate or poorly graded stone within the column allows for use where groundwater or drainage issues are a concern. In addition to reducing drainage path lengths, Handy and White (2006) reported that the extremely high lateral stresses created by the installation of rammed aggregate piers create radial cracking and wedging through which the high pore water pressures rapidly dissipate. Morales et al. (2011) reported the results of a full scale trail embankment and showed that the time to 90 percent consolidation settlement was reduced to 4.7 weeks with rammed aggregate piers, where timeframes of 6 to 8 months were estimated for other ground improvement methods. Fox and Edil (2000) summarized three case histories of the use of rammed aggregate piers to successfully limit settlements where peat or organic soils were present. In addition to the documented successful performance, the cost for the use of rammed aggregate piers was estimated to be 20 to 30 percent of the cost of deep foundations for the case histories summarized by Fox and Edil (2000).

Since the local state's highway aggregate base material is typically specified for use, it is almost always ensured that there will be easy and economical access to the required materials. Since the aggregate piers are not required to extend to suitable end bearing material, decreased lengths of installation and superior ground improvement characteristics allow for greater cost effectiveness. Rammed aggregate piers are also frequently used to support lightly loaded slabs-on-grade (Miller et al. 2007) without the need for structural slab design and construction. Proven performance, extensive research, and documentation of rammed aggregate pier projects provide an extensive knowledge base and reassurance of successful performance when properly applied. In comparison tests, rammed aggregate piers have been shown to have stiffness 5 to 10 times greater and settlements significantly less than vibro-replacement stone columns when exposed to the same loads (Pitt et al. 2003).

Limitations:

When project sites with clean sands are encountered, rammed aggregate piers can be used to provide stiff stone column inclusions within loose layers (Shields et al. 2004). However, rammed aggregate piers cannot improve densities in clean sands to large radial distances as with vibratory methods.

2.3 Deep Foundation Systems

Deep foundations are a proven and reliable method for reducing the settlement of structures by transmitting the structural loads to suitable underlying soils where inadequate bearing soils are encountered near the ground surface. Some of the axial loads are supported by skin friction between the deep foundation element and the surrounding soil, and the remaining loads are supported by end bearing of the pile/pier within very stiff to hard cohesive soils, dense granular soil, or sound bedrock. Deep foundations are grouped into two primary groups, piles and shafts (or piers). Generally speaking, piles are driven into the ground to a specified depth or refusal criteria, whereas shafts are constructed by first removing the soil to create a void and subsequently filling the void with grout or concrete. Sometimes the terms pile and shaft are intermixed based upon the slenderness ratio (length to diameter ratio). For example, auger cast-in-place foundation elements are commonly referred to as auger cast-in-place piles (ACIP) since they are generally smaller in diameter compared to their driller shaft counterpart. Auger cast-in-place piles, drilled shafts, precast concrete piles, and steel piles are four of the most common deep foundation methods.

2.3.1 Auger Cast-In-Place Piles

Auger cast-in-place piles are also referred to other names such as auger cast piers, ACIP piles, drilled displacement piles, or continuous flight auger piles. The use of auger cast piles in the United States has largely been focused in the commercial/private sector such as power plants (Pegues et al. 2007), however, they are gaining greater use as they become better understood and provide another viable alternative for deep foundations. Brown et al. (2007) reported transportation related use for sound walls, bridge piers and abutments, retaining structures, and pile supported embankments.

Construction Process, Fundamentals, and Techniques:

As indicated in the aforementioned names for the process, hollow continuous flight augers are both rotated and vertically forced into the ground in a single continuous process. The augers are subsequently raised to the ground surface with little to no rotation while grout is immediately inserted into the cavity under pressure through the hollow auger. It is extremely important to monitor and control the grout pressure and grout take of the pile to prevent necking of the pile element. Grout materials are usually high in cement content, are limited to sand sized aggregate and contain admixtures to aid in the pumping process. Auger cast-in-place piles diameters typically range from 0.3 to 0.9 m (12 to 36 in.) in the United States with depths extending up to 30 m (100 ft), or greater with specialized equipment. Full depth reinforcement can be constructed, but generally the reinforcement is limited to the upper portion of the pile. Figure 20 shows the equipment of auger cast-in-place pile installation.

Figure 20. Auger cast-in-place pile installation (Brown et al. 2007)

Auger cast-in-place piles are typically extended to hard/dense bearing soil or sound bedrock in order to support the required loads. Significant skin friction is also a benefit of the process which has allowed for limited use for settlement reduction when extending to bedrock was not cost feasible (Srinivasan et al. 2011).

Advantages:

Auger cast-in-place piles offer several advantages over other deep foundation methods including limited noise and vibration impacts to surrounding areas, especially compared to driven piles. Auger cast-in-place piles offer efficiency of installation since the augering and grouting operations are performed in a single process and there is no need to cutoff or splice the piling. The augers also function as temporary casing during installation, preventing cave-ins and eliminating the need for cumbersome casing or slurry. Auger cast-in-place piles are well suited for medium to stiff clays/silts, cemented sands or weak limestone, medium dense to dense sands, and silty sands. Auger cast-in-place piles can also provide some ground improvement by laterally displacing and densifying the surrounding soil. Brown et al. (2007) briefly summarized several other studies where shallow continuous flight auger piles were used in conjunction with pile caps to support embankments and reduce excessive settlements.

Limitations:

Since auger cast-in-place piles have primarily been used for commercial construction projects to date in the United States, there are limited studies and case histories for transportation projects. More importantly, very soft soils or loose sands (especially under groundwater) can cause difficulties for auger cast-in-place pile installation and quality control (Brown et al. 2007). When compared to drilled shafts, auger cast-in-place piles are generally smaller in diameter and are not extended to the same depths due to the higher torque requirements of the installation process. Continuous flight auger piles are only capable of penetrating very weak rock, whereas drilled shafts are commonly socketed into rock or other strong bearing materials. The augering process also creates soil spoils which must be disposed of or wasted on site. Costs are relatively high when compared to the ground improvement type alternatives, however, the costs are much lower than the drilled shaft and driven pile methods discussed below.

2.3.2 Drilled Shafts (Drilled Piers)

Although they are more commonly referred to as drilled shafts or piers, there are numerous other terms including caissons, cast-in-drilled-hole piles, and bored piles. The smaller cylindrical shafts recognized today can be traced back to the early 1900's where they were oftentimes excavated by hand. The function of drilled shafts is to extend large structural loads from the ground surface to deep suitable strong soil or sound rock. Brown et al. (2010) reported extensive transportation related uses such as bridge foundations, sound walls, retaining walls, signs, and even lighting structures.

Construction Process, Fundamentals, and Techniques:

The construction of a drilled shaft involves drilling a hole, typically measuring between 1.0 to 3.6 m (3 to 12 ft) in diameter, into the underlying deep strong soil layer or rock formation. The shaft is usually extended several feet into the bearing material, also known as a socket. During the drilling process the hole must remain stable and can be accomplished without any casing material if the soil conditions allow, or with casing if caving soil and/or groundwater are encountered. After the bearing material is inspected, concrete and associated reinforcing steel "cage" is placed into the drilled shaft to create a reinforced concrete pier extending to strong bearing soil/rock. The construction phases of drilled shafts are presented in Figure 21. Depths extending up to 60 m (200 ft) are relatively common for drilled shafts, but even deeper depths can be constructed with appropriate equipment.

Advantages:

Drilled shafts provide the same low vibration advantages similar to the auger cast-in-place piles since the process is executed through drilling rather than driving forces. In stiff to hard clay or rock formations drilled shaft construction is very straightforward and efficient since the sidewalls of the shaft foundation remain intact. Apart from some drilling difficulties in certain soil types, drilled shafts can be successfully implemented in all soil types provided suitable bearing material is encountered. Another very important advantage is the ability to visually inspect the bearing soil/rock prior to concrete placement to ensure the strata is consistent with the design.

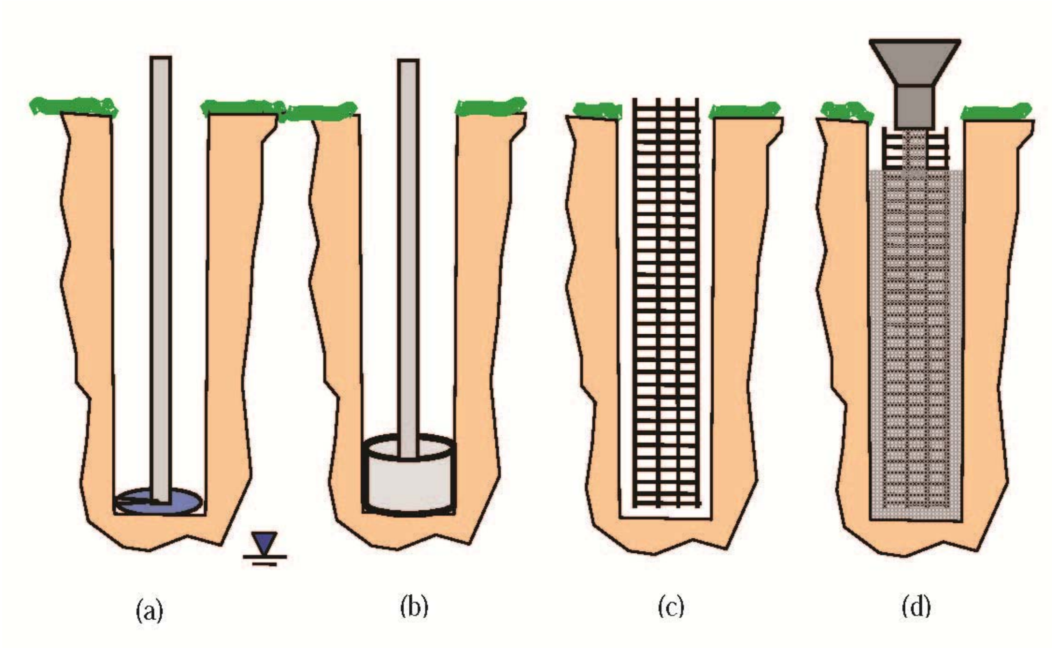


Figure 21. Drilled shaft installation (a) Drill hole (b) Clean bearing surface (c) Install reinforcing steel (d) Place concrete (Brown et al. 2010)

Limitations:

When groundwater or caving soils are encountered, the construction of drilled shafts is slowed by the installation of casing. Large diameter drilled shafts are usually designed for the end bearing, and therefore suitable bearing material must be encountered. When the suitable end bearing soil layers are not present at relatively shallow depths, then the drilled shafts are designed for skin friction which requires long shaft lengths to provide enough skin friction. The need to extend to suitable bearing material can lead to higher costs compared to other methods that can be terminated at shallower depths. The installation of drilled shafts takes significantly more time than the ground improvement methods and auger cast-in-place piles discussed.

2.3.3 Precast Concrete Piles

Driven piles have been used for a very long time in one of many forms and millions of driven piles are installed throughout the world every year. Several types of piles exist including wood, steel, concrete, and composite piles combining multiple materials. The focus of this section, precast concrete piles come in several shapes including square, cylindrical, and hollow/pipe designs. Precast concrete piles can be simply reinforced, or more commonly in the United States, prestressed.

Construction Process, Fundamentals, and Techniques:

Precast concrete piles, as the name describes, are constructed at a casting yard and transported to the project site. A crane is used to support the pile driving hammer which is usually powered by diesel fuel, compressed air, or hydraulic fluid. Precast concrete piles are described as “friction” piles due to the high levels of friction between the rough concrete surface and displaced soil. Pipe piles compound this effect since the skin friction is generated on the outside of the pile as well as the interior surface of the pile, until the pile plugs, at which time it behaves like a closed-end pile. The piles are terminated when refusal is encountered upon a hard or dense end bearing material or when sufficient skin friction has been developed. The refusal of a pile is typically defined by the number of blows by the driving hammer per length driven. Typical reinforced concrete pile lengths measure 9 to 15 m (30 to 50 ft) whereas prestressed piles measure 15 to 40 m (50 to 130 ft). Common widths or diameters of square/cylindrical precast concrete piles range from 255 to 915 mm (10 to 36 in.) and precast concrete pipe pile outside diameters typically range from 915 to 1,675 mm (36 to 66 in.) (Hannigan et al. 1998 and 2006).

Advantages:

Because of the long history of precast concrete pile use, there is extensive documentation and history of proven, successful performance. More specifically, transportation projects have long favored the use of driven piles because of the performance base criteria that can be specified. The concrete construction makes precast piles corrosion resistant and suitable for use regardless of groundwater conditions. Reinforced or prestressed concrete materials are also suitable for high axial loads.

Limitations:

As with other deep foundation methods, relatively high costs are a significant disadvantage to precast concrete piles. The precast concrete piles are also susceptible to damage during handling and installation. Because of the extremely high strengths and axial stiffness, some type of pile cap or load transfer platform is required to transfer surface loads directly to the pile elements. Installation can take significantly more time than ground improvement methods and the auger cast-in-place pile deep foundation method. In addition, the pile driving process is usually noisy process and causes vibrations which may result in disturbances to the neighboring structures and occupants of these structures.

2.3.4 Steel Piles

Another type of driven pile is the steel pile, which consists of either H-sections or pipe (cylindrical) piles. Steel H-pile flange and web thicknesses are typically the same and cross sections generally range from 200 to 360 mm (8 to 14 inches). Steel pipe piles are comprised of seamless, welded, or spiral welded steel pipes with common diameters ranging from 200 to 1220 mm (8 to 48 inches). Pipe piles can have open or closed ends, and can be left unfilled or filled with concrete.

Construction Process, Fundamentals, and Techniques:

As with other driven piles, steel piles are installed via a crane supported pile driving hammer which is usually powered by diesel fuel, compressed air, or hydraulic fluid. Steel piles can be designed as "friction" piles, end bearing piles, or a combination of both friction and end bearing. The piles are terminated when refusal is encountered upon a hard or dense end bearing material or when sufficient skin friction has been developed. The refusal of a pile is typically defined by the number of blows by the driving hammer per length driven. Driving shoes can be utilized to protect the pile sections during installation. For pipe piles a flat plate or conical point can be utilized depending on the anticipated driving conditions. Typical steel pile lengths measure 5 to 40 m (15 to 130 ft) and the pile sections can be easily spliced together when greater pile lengths are required (Hannigan et al. 1998 and 2006).

Advantages:

Because of the extensive history of the use of driven steel piles, there is widespread documentation of proven, successful performance. More importantly, transportation related projects have preferred the use of driven piles because of the performance based criteria that can be specified. H-piles and open ended pipe piles can be utilized to more easily penetrate through dense granular soils and very stiff to hard cohesive soils. The ability to easily cut or splice piles together allows for use on projects with highly variable soil layers and anticipated termination depths. Closed end pipe piles can also be filled with concrete to increase axial load carrying capacity.

Limitations:

As with other deep foundation methods, relatively high costs are a significant disadvantage of steel piles. Steel pipe piles are also susceptible to damage during installation. Additionally, pile driving is a noise intensive process and causes vibrations which may result in disturbances to nearby structures and occupants of these structures. H-piles also have the inclination to stray from the intended path when below ground obstructions are encountered.

2.4 Analysis and Summary of Vertical Column Support Methods

There are several critical components that play a role in determining the effectiveness of a vertical column support system for a specific project. Some of these critical components are soil type, groundwater conditions, time limitations, and costs. The evaluation process for the selection of a vertical column support method should take into account not only these critical components but also other components, such as pavement conditions, load transfer platform, and availability of these methods based on geographical location of a project site. The site location is important, because the majority of these methods require specialty geotechnical engineering contractors. The evaluation process will likely not lead to only one single optimal solution. While some methods can be eliminated, others can be well suited for certain site conditions or other aspects of the project. The principal aspects to be evaluated during the selection process are soil type, groundwater conditions, replacement of exiting roadway surface pavement, time constraints/considerations, and cost. It is expected that the use of a vertical column support system will be necessitated by the presence of problem soils beneath the existing pavements. The general description "problem soils" can be more precisely described as soft clay/silt soils, loose granular soils, and organic soils which are oftentimes exacerbated by being in a saturated state due to either heavy rainfall or the presence of groundwater. These poor soil conditions can be commonly found in lake deposited soils which are primarily fine grained silts and clays and frequently contain organic soils such as peat. Based on ODOT's experience, poor soils in Ohio most often occur in glacial interlobate moraine, kettles, and pro-glacial lake (glaciolacustrine) deposits. The selection of the right method(s) suitable for soil types present in the subgrade is paramount for the effectiveness of any method used for the remediation of settlements. Therefore, methods not suitable for the soils present at the site should be eliminated during the decision process. Table 1 displays the general applicability of each vertical support column method by problematic soil types.

Table 1. Applicability of vertical column support method by soil type

Vertical Column Support Method		**Gravels**	**Sands**	**Silts**	**Clays**	**Organics**
Ground Improvement	Vibroflotation	X	X			
	Vibro-replacement stone columns	X	X	X	X	
	Vibro concrete columns	X	X	X	X	X
	Deep soil mixing		X	X	X	X
	Jet grouting		X	X	X	X
	Compaction grouting		X	X		
	Controlled modulus columns	X	X	X	X	X
	Sand columns			X	X	X
	Rammed aggregate piers	X	X	X	X	X
Deep Foundation	Auger cast-in-place piles	X	X	X	X	X
	Drilled shafts	X	X	X	X	X
	Precast concrete piles	X	X	X	X	X
	Steel piles	X	X	X	X	X

Groundwater conditions should also be evaluated as part of the selection process. Unlike the frequently used remove and replace method, most vertical column support methods are not adversely affected by the presence of groundwater. In fact, sites with high groundwater tables are well suited for ground improvement or deep foundations because they can be installed from the ground surface and do not require dewatering. The greatest effect on vertical column support methods is the construction process when groundwater is anticipated. Casing may be required for methods that rely on the cavity sidewalls to remain stable and intact during construction such as rammed aggregate piers, drilled shafts, or predrilled vibro-replacement stone columns. High pressure methods such as jet grouting and compaction grouting can be performed below the groundwater table, however, careful quality monitoring and control must be performed to prevent hydrofracturing and heave within water bearing soft or loose soil strata. In addition to the groundwater considerations, analysis of long term drainage conditions is also important. Vibro-replacement stone columns, sand columns, and rammed aggregate piers can provide improved drainage for saturated silt/clay soils by reducing the drainage path lengths. The reduced drainage path length substantially accelerates consolidation settlements, reducing the time needed to complete the consolidation process. This could help the number of future pavement repair/patching projects for existing roadways. Table 2 briefly summarizes the general groundwater and drainage considerations for each vertical column support method.

Table 2. Vertical column support method groundwater and drainage considerations

Vertical Column Support Method		**Groundwater Considerations**	**Drainage Considerations**
Ground Improvement	Vibroflotation	None	No improvement
	Vibro-replacement stone columns	None	Enhanced drainage
	Vibro concrete columns	None	No improvement
	Deep soil mixing	None	No improvement
	Jet grouting	Construction QC	No improvement
	Compaction grouting	Construction QC	No improvement
	Controlled modulus columns	None	No improvement
	Sand columns	None	Enhanced drainage
	Rammed aggregate piers	Construction (Casing)	Enhanced drainage
Deep Foundation	Auger cast-in-place piles	None	No improvement
	Drilled shafts	Construction (Casing)	No improvement
	Precast concrete piles	None	No improvement
	Steel piles	None	No Improvement

When settlement repair operations need to be performed through the surface of an existing pavement, limiting the size of equipment penetrating the pavement can offer benefits to the project. Jet grouting and compaction grouting can limit the ground/pavement surface disturbance due to the relatively small diameters of the drilling, casing, and jetting equipment used compared to the other methods. They also offer the advantage of angled installation to extend beneath existing structures. The relative surface disturbance of each method is listed in Table 3.

Table 3. Relative pavement surface disturbance by vertical column support method

Vertical Column Support Method		**Relative Surface Disturbance**
Ground Improvement	Vibroflotation	Moderate
	Vibro-replacement stone columns	Moderate to High
	Vibro concrete columns	Moderate to High
	Deep soil mixing	High
	Jet grouting	Low
	Compaction grouting	Low
	Controlled modulus columns	Moderate
	Sand columns	Moderate
	Rammed aggregate piers	Moderate
Deep Foundation	Auger cast-in-place piles	Moderate to High
	Drilled shafts	High
	Precast concrete piles	High
	Steel piles	Low to Moderate

The duration of road closure to remediate the subgrade settlement and to repair the roadway pavement is another critical component that needs to be considered during the evaluation process. Not only the installation duration but also the curing time should be considered for the methods which utilize cementitious materials. Admixtures or materials accelerating the cure time can be used, but any allowed cure time will result in additional duration when compared to the methods which use solely sand or stone elements.

The success of any vertical column support method will certainly be measured by its ability to reduce settlements but also by its cost effectiveness when compared to the other vertical column support methods and traditional remediation methods. The relative installation cost of each method is displayed in Table 4. Consideration must also be given to the potential for differential settlements at the pavement surface when very stiff vertical columns are installed within weak soils. In their normal applications, deep foundations are structurally connected at the top with pile caps that transmit all the loads from structure to the piles or piers. Some ground improvement methods such as vibro-concrete columns and compaction grouting produce concrete/grout columns that behave as piles or piers after installation. Concrete or grout columns may need to incorporate a structural slab or other similar design to transfer the loads to very strong columns and prevent differential settlements at the surface. In other somewhat similar vertical column supported earth embankment applications, load transfer platforms constructed out of layered select fill and geogrid materials have been used to control differential settlements. Other relatively weaker column elements such as those constructed from stone or weak soil-cement mixtures may be able to prevent differential settlements with lesser load transfer platforms or even none at all. The requirement of a pile cap or load transfer platform could introduce additional expenses to methods which are already relatively costly.

Table 4. Relative cost and construction duration for vertical column support methods

Vertical Column Support Method		**Relative Cost**	**Relative Construction Duration**
Ground Improvement	Vibroflotation	Low	Short
	Vibro-replacement stone columns	Low	Short
	Vibro concrete columns	Moderate	Medium
	Deep soil mixing	Moderate to High	Medium
	Jet grouting	Moderate to High	Medium
	Compaction grouting	Low to Moderate	Medium
	Controlled modulus columns	Moderate	Medium
	Sand columns	Low to Moderate	Short
	Rammed aggregate piers	Low	Short
Deep Foundation	Auger cast-in-place piles	Moderate to High	Medium
	Drilled shafts	High	Long
	Precast concrete piles	High	Medium to Long
	Steel piles	High	Medium to Long

2.5 Survey of Other State DOTs

As part of this research project, a survey of the other states' Departments of Transportation (DOTs) has been conducted. The main objectives of the survey were to solicit information and feedback on the:

1. Severity of the pavement distress/failure due to subgrade soil settlements in other states across the nation;
2. Methods used to remediate subgrade settlement problems; and
3. Other states' experiences with the vertical column support systems evaluated.

A four-page questionnaire has been prepared to solicit information and feedback. All fifty state DOTs have been contacted for the survey. A copy of the survey form sent to the state DOTs is included at the end of this chapter. Thirty-five (35) state DOTs have responded to the survey. While thirty-three (33) of the DOTs completed and returned the survey, one of the DOTs indicated that they sent the survey but was never received, and another one indicated that they will not participate in the survey. The DOTs survey participation rate was 66%. Some states have participated in the survey with multiple responses coming from different divisions of the same agency, such as geotechnical and materials divisions separately. Because of this reason there were actually a total of thirty-eight (38) completed surveys received, although thirty-three (33) of the state DOTs have participated in the survey. An overview of the survey participation by state DOTs is given in Table 5. The analysis of the survey responses is summarized and presented in the following.

Table 5. Overview of the survey participation by the state DOTs

States participated:	Alabama	Illinois	Nevada
	Alaska	Indiana	New Hampshire
	Arizona	Iowa	New Mexico
	Arkansas	Kentucky	New York
	California	Louisiana	North Dakota
	Colorado	Maryland	South Carolina
	Connecticut	Michigan	South Dakota
	Delaware	Minnesota	Texas
	Florida	Missouri	Utah
	Georgia	Montana	Wisconsin
	Hawaii	Nebraska	Wyoming
States did not participate:	Idaho	North Carolina	Vermont
	Kansas	Oklahoma	Virginia
	Maine	Oregon	Washington
	Massachusetts	Pennsylvania	West Virginia
	Mississippi	Rhode Island	
	New Jersey	Tennessee	

2.5.1 Severity of Subgrade Settlements

The survey results indicate that pavement distress/failure due to subgrade settlement is affecting roadways across the nation. Only a very low percentage of respondents (25%) indicated that subgrade settlement is a "rare" occurrence in their state. A total of 75% of the responses indicated medium or high frequency of occurrence for pavement distress due to subgrade settlement. Figure 22 shows the distribution of pavement distress/failure frequency due to subgrade soil settlements reported by the state DOTs.

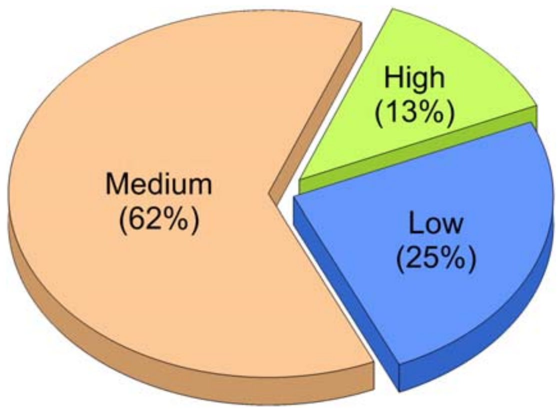


Figure 22. Frequency of pavement distress/failure experienced by the state DOTs due to subgrade soil settlements

The survey results also show that when the subgrade settlements are occurring due to problematic soils and causing distress on the roadway, remediating the settlement problem with patching is not a permanent solution. The overwhelming majority (88%) of the DOTs responding to the survey indicated that the roadway continues to show distress after patching due to subgrade settlement and require additional pavement surface patching later. The number of repeated pavement surface patching reported was five (5) times, on average, and as much as ten (10) times reported by one of the state DOTs.

2.5.2 Problematic Soil Types

The survey results showed that soil types causing subgrade settlement across the nation include soft cohesive soils, loose granular soils, fills, organic soils, and saturated soils. Although soft cohesive soil is the most common soil type causing pavement subgrade settlement problems in most states as shown in Figure 23, the majority of DOTs indicated that they deal with several types of soils causing subgrade settlement and pavement distress.

2.5.3 Use of Traditional versus New Remediation Methods

One of the traditional methods used to remediate pavement settlements when unsuitable soils constitute the subgrade is the "remove and replace" method. This method includes removal of the problem soils and replacing them with more suitable soils that can carry the pavement and traffic loads without excessive settlements. This method is most applicable when the problem soils are not very deep. As the depth of unsuitable soils increases, this method starts to become prohibitively expensive. ODOT uses the "remove and replace" method for depths up to approximately 1.5 m (5 ft). Figure 24 presents the other states' practice for the "remove and replace" method. As shown in the figure, the majority of the states (73%) limit the "remove and replace" method within the top 1.2 m (4 ft) depth.

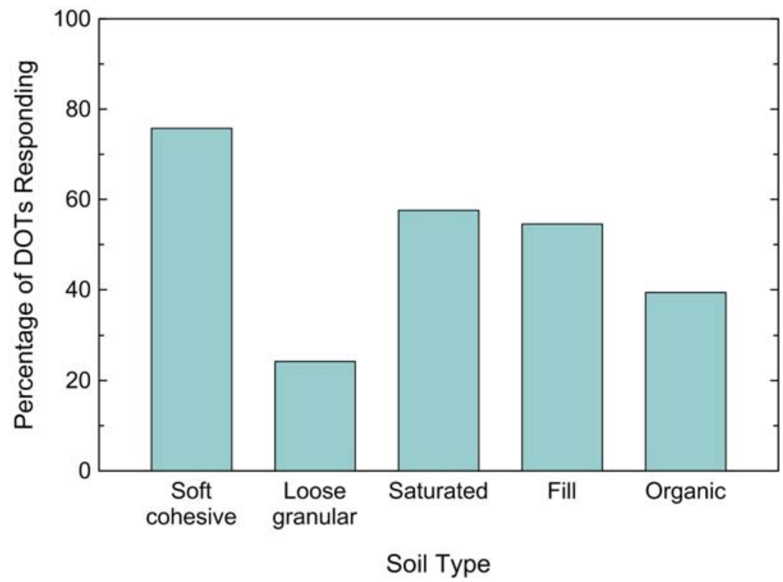


Figure 23. Common soil types causing subgrade settlement and pavement distress nationwide

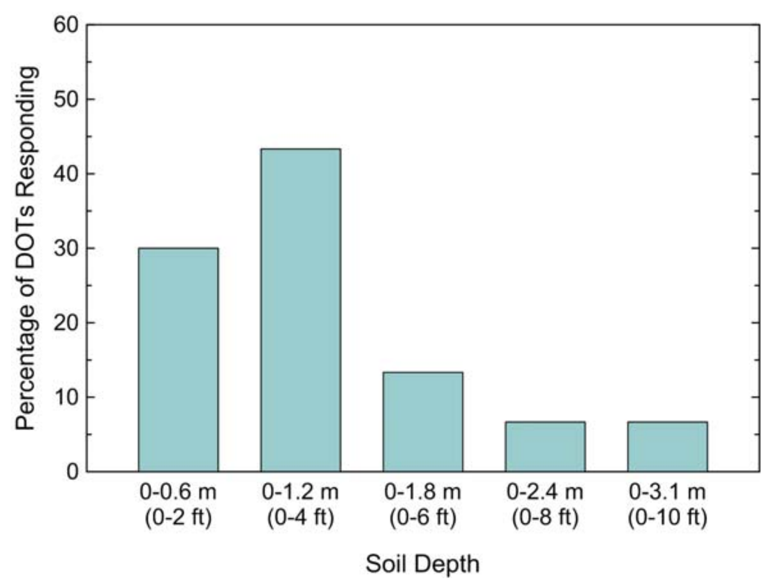


Figure 24. Depth range where "Remove & Replace" method used for remediation by the state DOTs

As mentioned previously, the survey also solicited information on other methods used by the state DOTs. Figure 25 shows that the remove and replace method is used by all of the states who responded to the question. Chemical stabilization methods (such as lime, lime kiln dust, and cement) are used by 41% of the states and 69% of the states have used other methods, including vertical column support methods, to remediate subgrade settlements of existing roadways in their states (Figure 25).

The percent usage of these three main remediation method categories shown in Figure 25 also significantly varied among the states. While some states rely only on the "remove and replace" method for the remediation of subgrade settlement problems, some states heavily rely on "other methods", i.e., non-traditional methods, for remediation (as much as 95% of the time). Figure 26 shows, on average, the percent usage distribution of the three main remediation categories by state DOTs across the nation. As shown in the figure, on average, "remove and replace" is used in 57% of the projects, chemical stabilization is used in 26% of the projects, and non-traditional methods are used in 42% of the projects to remediate the subgrade settlement of roadways. It should be noted that the total does not equal to one hundred percent, because the responses of all states are averaged separately for each category.

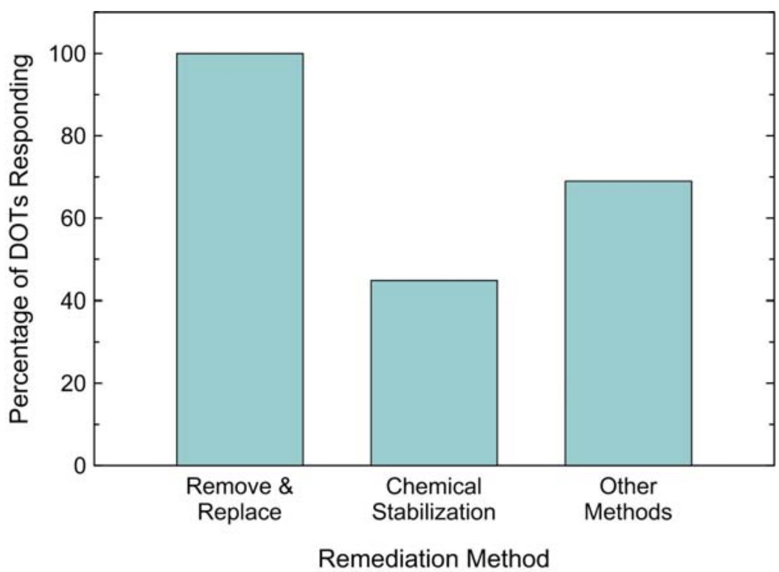


Figure 25. Use of traditional and new remediation methods (under Other Methods) by the state DOTs

2.5.4 Vertical Column Support Methods Used by Other State DOTs

Figure 27 presents the survey results for the vertical column support methods used by other state DOTs. As mentioned previously, vertical column support methods include both ground improvement and deep foundation methods. The ground improvement and deep foundation methods are grouped and presented separately in Figure 27. The figure shows that the majority of the vertical column methods considered in this research project have been used by some of the other states to remediate pavement subgrade settlements. Two of the ground improvement methods, vibro-dry concrete columns and vibroflotation (vibro compaction) methods, have not been used for roadway subgrade settlement remediation by any of the states who responded to the survey. All four deep foundation systems considered in this study have been utilized by various states.

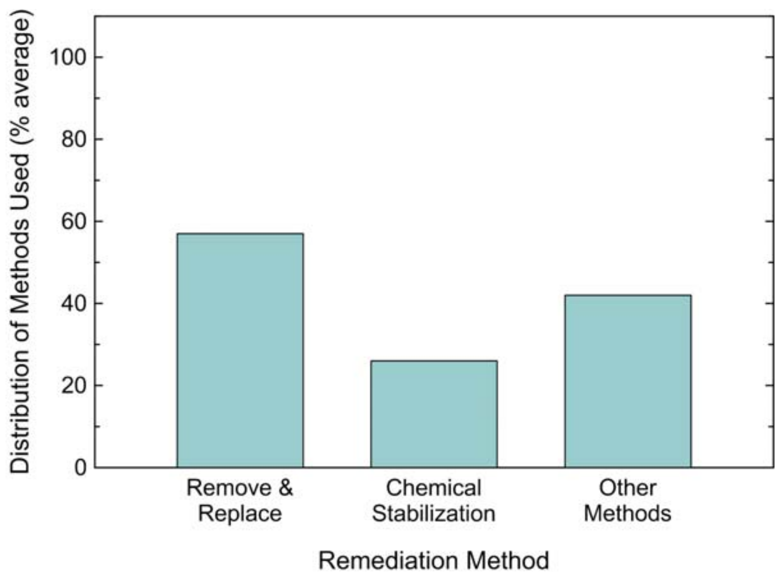


Figure 26. Percent utilization of traditional versus new remediation methods (under Other Methods), on average, by the state DOTs

Aggregate piers method is the most commonly used vertical column support method among those being evaluated. Aggregate piers were used by 24% of the DOTs responding. The usage of compaction grouting and stone columns followed the aggregate piers. Each method is used by 18% of the DOTs responding. Among the deep foundation methods, steel piling was the most commonly used method by the DOTs to remediate subgrade settlements. Among the DOTs responding, 20% used steel piles. In these cases, relatively thick soft soil deposits required the use of deep foundations and the steel pile foundation option was selected due to relatively quick installation and low cost compared to other deep foundation alternatives.

The survey results also indicated that while some states use only one vertical column support system exclusively for all their projects, some states utilize multiple vertical column support systems, as much as eight different types, to remediate pavement distress/failure due to subgrade soil settlements.

The survey results also show that the state DOTs consider time and cost advantages in selecting a specific vertical column support method over other possible methods. On the other hand, the DOTs responding indicated the cost as the main reason for eliminating some of the vertical column support methods. There was one case, where a vertical column support method (ground improvement method) was implemented but failed to remediate the settlements, and therefore another vertical column support method (deep foundation system) was used for remediation. For this case, very soft and thick soil deposits as well as the vertical columns installed as ground improvement not being installed deep enough were indicated as the causes for the failure of the initially installed ground improvement method.

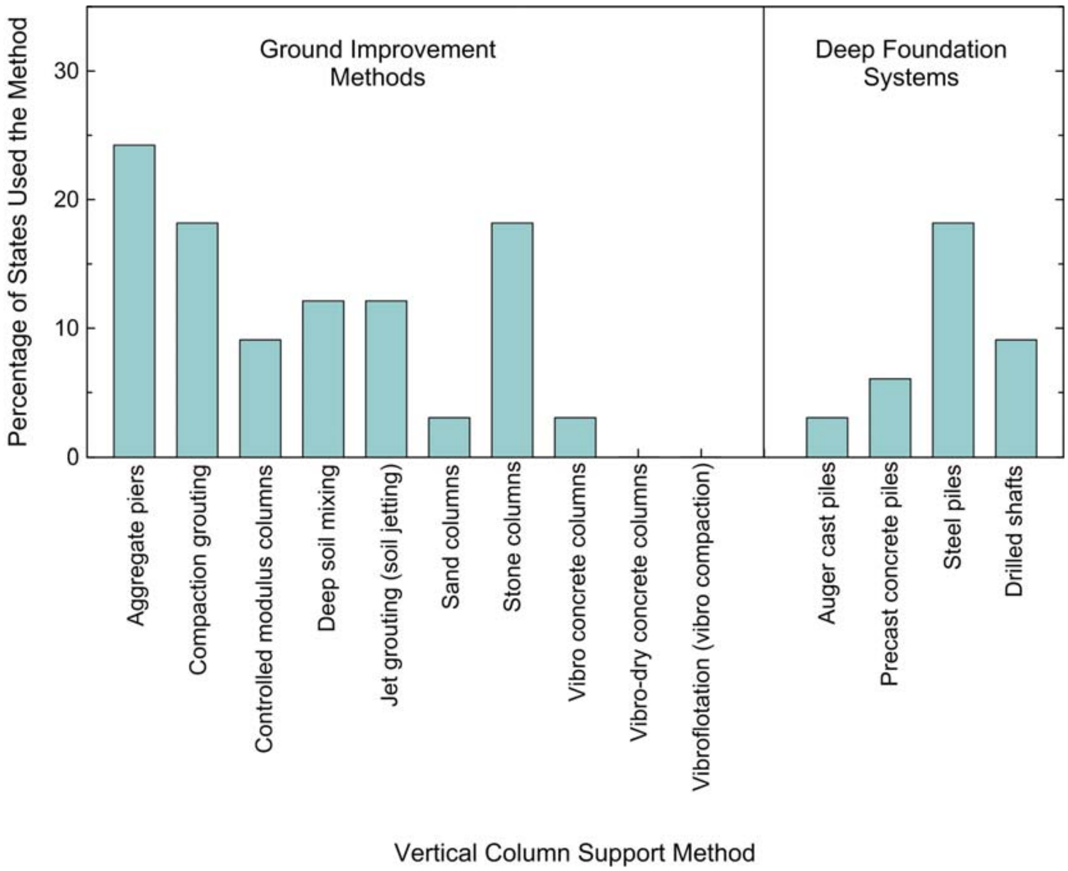


Figure 27. Vertical column support methods used by the state DOTs to remediate subgrade settlement of existing roadways

2.5.5 Survey Form

The survey form sent to other state DOTs to gather information is provided in the following four pages.

UNIVERSITY of DAYTON

Survey of Remediation Methods for Pavement Distress/Failure Due to Subgrade Soil Settlements

Agency : ____

Name : ______________________ **Title :** ______________________

Phone : ______________________ **Email :** ______________________

Please click or check (with X) the boxes to the following questions and fill out the blanks. We appreciate your feedback and input concerning your experiences with remediation methods for pavement distress/failure due to subgrade soil settlement used in your State.

1. How often does pavement distress/failure due to subgrade soil settlement occur in your State?

☐ Rarely ☐ Occasionally ☐ Frequently

2. What are the most common causes of roadway subgrade settlement in your State? (Please check all that apply)

☐ Soft cohesive soils ☐ Loose granular soils ☐ Saturated soils

☐ Fills ☐ Organic soils ☐ Other: ____________

3. The pavement patching/resurfacing projects undertaken by your DOT to repair pavement distress/structural failure due to subgrade soil settlement are (approximately):

Number of projects : ____________ projects / year

Total cost of these repairs : ____________ $ / year

Percentage of DOT budget : ____________ %

4. What is the percentage of these pavement distress/structural failures remediated by relatively significant subgrade work (such as remove & replace and ground improvement)?

Percentage by the number of repair projects:

☐ < 20% ☐ 20-40% ☐ 40-60% ☐ 60-80% ☐ > 80%

Percentage by the total repair costs:

☐ < 20% ☐ 20-40% ☐ 40-60% ☐ 60-80% ☐ > 80%

5. Do the roadways remediated by pavement surface patching usually continue to show distress due to subgrade soil settlement and require additional pavement surface patching(s) later?

☐ No ☐ Yes (as many as _____ times resurface patching has been done)

6. How much does a roadway usually settle before a relatively significant subgrade work (such as remove & replace or ground improvement) is utilized for remediation?

☐ 2 inches ☐ 4 inches ☐ 6 inches ☐ 9 inches ☐ > 12 inches

7. **What is the depth typically considered for the "removal and replacement" of problematic subgrade soils causing pavement settlement (versus options such as ground modification)?**

☐ 0-2 feet ☐ 0-4 feet ☐ 0-6 feet ☐ 0-8 feet ☐ 0-10 feet

8. **Which of the following method(s) do you utilize to remediate subgrade settlement of your State's roadways (and its percent utilization of all methods used)?**

☐ Remove & replace (________ %) ☐ Chemical stabilization (________ %) ☐ Others (________ %)

9. **Please check the vertical column support methods that have been used by your DOT to remediate subgrade settlement of existing roadways? Please also indicate the number of projects and post-construction performance for the methods implemented.**

	No. of Projects	Overall post-construction performance: Very good	Good	Fair	Poor
Ground improvement:					
☐ Aggregate piers	______	☐	☐	☐	☐
☐ Compaction grouting	______	☐	☐	☐	☐
☐ Controlled modulus columns	______	☐	☐	☐	☐
☐ Deep soil mixing	______	☐	☐	☐	☐
☐ Jet grouting (soil jetting)	______	☐	☐	☐	☐
☐ Sand columns	______	☐	☐	☐	☐
☐ Stone columns	______	☐	☐	☐	☐
☐ Vibro concrete columns	______	☐	☐	☐	☐
☐ Vibro-dry concrete columns	______	☐	☐	☐	☐
☐ Vibroflotation (vibro compaction)	______	☐	☐	☐	☐
☐ Other: ____________________	______	☐	☐	☐	☐
Deep foundations:					
☐ Auger cast piles	______	☐	☐	☐	☐
☐ Precast concrete piles	______	☐	☐	☐	☐
☐ Steel piles	______	☐	☐	☐	☐
☐ Drilled shafts	______	☐	☐	☐	☐
☐ Other: ____________________	______	☐	☐	☐	☐

10. Was the vertical column support method selected (if applicable) over a conventional method (such as remove and replace) due to reduced: a) construction time, b) construction/remediation cost, and/or c) road/lane closure duration during construction benefit(s)? (Please check all that apply)

	Time	Cost	Road closure	Other reason(s)
Ground improvement:				
Aggregate piers	☐	☐	☐	____________
Compaction grouting	☐	☐	☐	____________
Controlled modulus columns	☐	☐	☐	____________
Deep soil mixing	☐	☐	☐	____________
Jet grouting (soil jetting)	☐	☐	☐	____________
Sand columns	☐	☐	☐	____________
Stone columns	☐	☐	☐	____________
Vibro concrete columns	☐	☐	☐	____________
Vibro-dry concrete columns	☐	☐	☐	____________
Vibroflotation (vibro compaction)	☐	☐	☐	____________
Other: ____________	☐	☐	☐	____________
Deep foundations:				
Auger cast piles	☐	☐	☐	____________
Precast concrete piles	☐	☐	☐	____________
Steel piles	☐	☐	☐	____________
Drilled shafts	☐	☐	☐	____________
Other: ____________	☐	☐	☐	____________

11. What is/are the preferred method(s) in your State for remediating subgrade settlements of existing roadways? (Please check all that apply)

☐ Remove & replace

☐ Chemical stabilization (e.g., lime, lime kiln dust, and cement)

☐ Ground improvement method (please refer to the list above)
- The most preferred/used method is : ____________________
- The second most preferred/used method is : ____________________

☐ Deep foundation elements (please refer to the list above)
- The most preferred/used method is : ____________________
- The second most preferred/used method is : ____________________

12. **Please check the vertical column support methods that have been eliminated from consideration to remediate subgrade settlement of existing roadways due to poor performance, high costs, time constraints, and/or availability of contractors/equipment in your State?**

	Performance	Costs	Time	Availability	Other(s)
Ground improvement:					
☐ Aggregate piers	☐	☐	☐	☐	________
☐ Compaction grouting	☐	☐	☐	☐	________
☐ Controlled modulus columns	☐	☐	☐	☐	________
☐ Deep soil mixing	☐	☐	☐	☐	________
☐ Jet grouting (soil jetting)	☐	☐	☐	☐	________
☐ Sand columns	☐	☐	☐	☐	________
☐ Stone columns	☐	☐	☐	☐	________
☐ Vibro concrete columns	☐	☐	☐	☐	________
☐ Vibro-dry concrete columns	☐	☐	☐	☐	________
☐ Vibroflotation (vibro compaction)	☐	☐	☐	☐	________
☐ Other: ____________	☐	☐	☐	☐	________
Deep foundations:					
☐ Auger cast piles	☐	☐	☐	☐	________
☐ Precast concrete piles	☐	☐	☐	☐	________
☐ Steel piles	☐	☐	☐	☐	________
☐ Drilled shafts	☐	☐	☐	☐	________
☐ Other: ____________	☐	☐	☐	☐	________

13. **Please provide any additional comments and feedback that you may think would be useful on the use of vertical column support methods for roadway settlements:**

____
____
____
____
____
____
____
____

We thank you very much in advance for your input and help with our research project. Please return your survey responses either by emailing to bilgin@udayton.edu or by mailing to Dr. Ömer Bilgin, P.E., Department of Civil Engineering, University of Dayton, 300 College Park, Dayton, Ohio 45469-0243.

2.6 Conclusions

The vertical column support methods have been extensively and successfully used to remediate and/or prevent soil settlements for various civil engineering structures over the years. Various transportation related projects such as bridges, culverts, sound walls, and new embankments have used vertical column support systems for some time for increasing bearing capacity and reducing settlement where poor soils are present.

In addition to evaluating the applicability of each method by soil type, it is also important to understand the engineering fundamentals of the processes involved. In overall terms, deep foundations transmit structural loads from the surface to underlying strong soil or rock through strong grout, concrete, or steel column type elements. The inclusion of these column elements do not substantially improve the weak soils, rather they bypass the weak soil layers and transmit the loads directly to relatively deep soil/rock layers which are capable of carrying the loads. Deep foundations are typically the method of choice when loads are relatively high, soils are very weak, and/or weak soil deposits are very deep. Ground improvement methods are gaining popularity because they also strengthen the surrounding weak soil and create a composite soil/column system of improved ground. The improved composite system generally allows the vertical column component of ground improvement systems to be terminated at shallower depths than the piles or piers of deep foundations. Ground improvement methods generally excel in low to moderate loading conditions and can be created from stone, concrete, soil-cement mixtures, grout, or sand. The aforementioned ground improvement characteristics make these methods particularly appealing for use under existing roadways to increase subgrade support and reduce settlements.

The survey of other state DOTs show that pavement distress and structural failure of existing roadways due to settlement of subgrade soils occur in various states across the nation. Patching and resurfacing projects are undertaken to repair pavement distress and structural failure due to problematic subgrade soils, but most of the time this approach only provides a temporary solution and does not remediate the problem permanently. The survey results indicated that repeated patching and resurfacing work is usually needed in most of the problem locations. Mainly because, the settlements not only continue at these locations, but may also accelerate due to the additional loads introduced from the patching and resurfacing.

The use of vertical column support systems for the remediation of existing roadways exhibiting excessive total and/or differential settlement is a limited methodology in Ohio at this time. However, the survey of other state DOTs conducted indicated that many of these methods have also been successfully utilized to remediate subgrade settlements of existing roadways.

Identifying the cost effective vertical column support methods which provide increased subgrade support and decreased settlements can assist ODOT by improving roadway safety, reducing future pavement rehabilitation projects, lowering repair costs, as well as lowering the overall cost to the road users and society as a whole.

CHAPTER 3. DECISION MATRIX

3.1 Introduction

Phase 2 of the proposed project included development of a decision matrix, or decision tree, to be used by ODOT to identify potential vertical column support method(s) feasible for the remediation of subgrade soil settlements experienced under existing roadways. Based on the findings during Phase 1 – Literature Review, critical aspects and components of the vertical column support methods investigated for successful implementation in transportation projects were identified.

Because of the number of vertical column support methods investigated and their extensive use in civil engineering projects, there is a significant amount of literature on these methods. After an extensive review of the literature, limiting factors were identified for each of the methods to act as a basis for elimination from the matrix. Using these limiting factors, a decision matrix or decision tree was developed to aide in identifying the technically suitable vertical column support ground improvement method(s) for a roadway having subgrade settlement problems. Due to the number of methods available/considered for the project and the factors affecting the methods' feasibility for a given project, the decision matrix was based on method elimination approach rather than a selection process.

Identifying possible remediation methods for any project depends on both technical capabilities/limitations and cost effectiveness of each method. The methodology used in formulating the decision matrix and the decision matrix developed in this section is based on the technical capabilities and the limitations of the vertical column support methods investigated during the project. The cost-benefit analysis will be discussed later in the report.

The deep foundation systems were also considered during the research as discussed earlier in the report. Based on the literature review, discussions with contractors and ODOT engineers, and the research team's own experience it was readily apparent that a deep foundation system would be technically feasible and capable of remediating the subgrade settlement problem of existing roadways. However, deep foundation methods would be a prohibitively expensive solution compared to the ground improvement methods. One of the main goals of this research project was identifying cost-effective methods; therefore, the deep foundation systems were eliminated from the decision matrix and from further consideration.

3.2 Critical Components of Decision Matrix

Each vertical column support method has its own advantages and limitations. The following parameters affect the technical feasibility of vertical column support method and determined to be critical in developing the decision matrix:

- Problem soil type
- Top depth of problem soil

- Bottom depth of problem soil
- Presence of groundwater
- Angled installation necessity, i.e. existing pavement has to be preserved
- Possibility of installations from centerlines
- Hard/dense soil overlying problem soils
- Presence of nearby vibration sensitive structures
- Permission to remove existing pavement and subgrade for load transfer platform
- Time availability for pre-loading for consolidation

In addition, if soils overlying the problem soils have boulders, cobbles, and/or large debris, some of the methods require predrilling or coring to penetrate through these materials, and this additional work would increase the installation costs of vertical support column method. For the remaining methods, the installation duration would also increase while passing through these obstacles resulting in increased remediation costs. Therefore, in this study and in developing the decision matrix, soils that may overly the problem soils are assumed to be free of boulders, cobbles, and/or large debris.

Some of the vertical column support methods also improve the drainage conditions at the site that would help with accelerating the continuing consolidation process. However, considering that these methods are considered for the existing roadways already in service for some period of time and there is no increase in roadway embankment, it is not anticipated that improved drainage conditions would be a significant factor for the roadway remediation. Therefore, the improved drainage conditions are not considered in the decision matrix. It should be noted however that the improvement drainage can provide a path for excess pore pressures that may built up during the installation process of some methods to dissipate.

The implementation of sand columns along with preloading can be a cost effective solution when used to accelerate the consolidation process and preconsolidate clay and organic soils. For the application to existing roadways, both preloading and waiting for consolidation are not realistic. Therefore, sand columns are eliminated from further consideration as one of the vertical column support ground improvement method alternatives for the remediation of existing roadways settlements.

3.3 Decision Matrix Development

Over time, different vertical column support or ground improvement methods have been developed and tailored to improve specific soil types and conditions. As a result, the type of problem soil is the first consideration for the decision tree. Fine grained soils located within glacial interlobate moraine, kettles, and pro-glacial lake (glaciolacustrine) deposits are the predominant soil types causing settlement of Ohio's existing roadways. These fine grained problem soils can be more specifically classified as organic soil, clay, silt, or a combination thereof. Organics, clays, and silts are compressible when subject to new loads, such as embankments and roadways, and the consolidation of these fine grained soils takes place over extended periods of time.

The research project initially started considering all problematic soils types, including the loose cohesionless soils, causing subgrade settlement problems for existing roadways. However, it has been determined that fine grained cohesive soils are the predominant problematic soil types causing roadway settlements and pavement distress. ODOT has also indicated that soft cohesive soils and organic soils are the ones mainly causing problems and requiring repair work. After the discussions with ODOT at early stages of the project, a decision was made to consider only soft cohesive soils and organic soils for the project in evaluating the vertical column support methods.

Because of the number of vertical column support methods covered and the complexity of each method considered, the research team has initially decided to develop the decision matrix using spreadsheets. The decision matrix has evolved as the project progressed and more input parameters were added to the spreadsheet. The evaluation of the sites for possible implementation was also performed using the draft spreadsheet decision matrix which was still a work-in-progress. Later during the research a simpler version of the decision matrix which is in a more traditional flow chart format was developed. Since it is a single page flow chart and does not require a computer, unlike the spreadsheet version to run the decision matrix, it is more user friendly and can be used by ODOT in future projects. Both versions of the decision matrix development are briefly explained in the following.

3.3.1 Decision Matrix – Spreadsheet Version

The development of the decision matrix had been initially started as spreadsheet programming. This decision was made mainly because it would provide flexibility to modify and add additional criteria to the matrix for the method selection/elimination as the research progresses. The site-specific data are entered by the users on an input sheet to be used for the assessment of methods. A copy of a sample spreadsheet input page is shown in Table 6. Using the input data, the spreadsheet consisted of three main steps in evaluating and determining the feasible vertical column support ground improvement methods:

Step 1: Evaluating the feasibility of remediation from the sides of a roadway without damaging the existing pavement. Most of the methods require large holes on the pavement for the application of methods. Jet grouting and compaction grouting can be performed through relatively small holes on the pavement which can be patched after the installation. All the other methods require large disturbance to the pavement surface and the replacement of the whole pavement would be required after the treatment. If the methods can be installed from the sides of a roadway without disturbing the existing roadway, then the additional costs to repair the pavement can be avoided. This would also avoid or reduce the traffic delays and accident risks during construction. Jet grouting, compaction grouting, and deep soil mixing are the methods that can be installed at an angle and reach under the roadway from the sides. Methods are capable of reaching only certain distances/depths, therefore, the width of the roadway, depth and thickness of the problem soils could limit the use of possible methods for angled installation. The spreadsheet would check the methods to assess their applicability from the sides of the roadway.

Step 2: Assessing all the methods investigated to identify vertical column support method(s) technically feasible based on the site specific information, such as soil type, problem soil depth, and thickness, provided by the user on input sheet shown in Table 6.

Step 3: Performing preliminary rough cost estimate for ground improvement methods on a cost estimating spreadsheet using the resources from Strategic Highway Research Program (SHRP 2) by the FHWA, and transferring the data to the decision matrix spreadsheet. A sample input data and cost estimate for a sample method using the SHRP2 resources is given in Figure 28.

The outcome of the assessment performed by the decision matrix spreadsheet program is presented on an output sheet as shown in Figure 29. The column under a method name is automatically blocked out when the decision matrix analysis assesses the method as unsuitable for the site. Some of the data collected on an input sheet is used not for the method selection/elimination process but rather they are used to alert the users on some issues by providing comments on the output sheet. As an example, “Construction QC must account for groundwater” comment is shown in Figure 29 for a couple of the methods to point out/alert users that those methods are technically feasible for the site, but the presence of groundwater may require additional attention during the construction. For the methods that produce heavy vibrations, like any of the vibro- methods or rammed aggregate piers, specified QA and QC should be followed to prevent damage to nearby structures. For vibro-stone columns, if there is no hard dense layer overlying problem soil, construction QC needs to be followed to prevent ground heaving and radial cracking at the surface.

Once the technically feasible methods are identified for a specific site using the decision matrix spreadsheet, the preliminary cost estimates are obtained by the user from the SHRP2 resources and the cost is transferred to the decision matrix output sheet to have all the results in one location.

Table 6. Decision matrix sample spreadsheet input

Item No	Question / Input	Data
1	Problem soil type	Organics
2	Second problem soil type	Clays
3	Third problem soil type	Gravels
4	What is top depth of problem soil	< 3 m (< 10 ft)
5	What is bottom depth of problem soil	9 - 14 m (30 to 45 ft)
6	Groundwater present within improvement depths	Yes
7	Does the soil overlying the problem soils have boulders, cobbles, hard/dense layers overlying problems soils	Large Debris / Cobbles
8	Is angled installation required	No
9	Does the system need to improve drainage	No
10	Are there nearby structures that are susceptible to damage from vibrations	No
11	Can the pavement section be removed & reconstructed with Load Transfer Platform	Yes
12	Can relatively small (<12 inch) diameter penetrations be cored through centerlines	Yes
13	Can work be performed from the sides of the roadway (within right-of-way)	Yes
14	Can the project be delayed while pre-loading and consolidation occurs (i.e., months)	No

Notes to User:

A. This estimating tool is provided as a means to perform an initial project scoping estimate. Use for any other purpose is strongly discouraged. The accuracy and reliability of the estimated costs are highly dependent upon the user inputs, care should be taken to adjust inputs for specific project characteristics. The user assumes all risks associated with the cost estimates produced by this estimating tool.

B. Guidance on unit cost ranges and potential impacts on cost is provided in the cost information summary for each technology. Users are responsible for determining appropriate unit costs.

C. Cells highlighted require user input.

D. Cells with numbers in italic are automatically calculated, but may be manually overridden by the user.

1. Calculate the Surface Area Where Deep Mixing is to be Installed

Length (ft):	1,000
Width (ft):	40
Area (ft^2):	*40,000*

2. Estimate the Total Quantity of Deep Mixing to be Installed

Design output information required - Preliminary grid spacing and average depth of installation are necessary for this step

Estimated Longitudinal Grid Spacing (ft):	3.00
Estimated Transverse Grid Spacing (ft):	3.00
Number of Deep Mixing Columns to be Installed:	*4,792*
Average Depth of Deep Mixing Column Installation (ft):	20
Diameter of Deep Mixing Columns (ft):	3
Total Quantity of Deep Mixing Columns (yd^3):	*25,091*

3. If Needed, Estimate the Materials Required for an Initial Working Platform

Length (ft):	1,000
Width (ft):	40
Quantity of Geosynthetic for a Working Platform (yd^2):	*4,444*
Optional, Thickness of Granular Layer for Working Platform (in):	12
Optional, Estimated Density of Granular Material for Working Platform (lb/ft^3):	120
Total Quantity of Granular Material for Working Platform (ton):	*2,400*

4. Estimate Additional Embankment Due to Settlement

Design output information required - Estimated amount of settlement is needed

Estimated Depth of Settlement (ft):	3
Total Quantity of Embankment Material (yd^3):	*4,444*

5. Estimated Cost of Deep Mixing Methods - Refer to Cost Information Summary for Typical Unit Cost Ranges and Impacts

	Unit Cost	Quantity	Cost
Optional, Geosynthetic for Working Platform (yd^2):	$ 2.75	*4,444*	$ 12,222
Optional, Granular Material for Working Platform (ton):	$ 7.50	*2,400*	$ 18,000
Deep Mixed Columns ($/yd^3$):	$ 8.00	*25,091*	$ 200,731
Mobilization (lump sum):	$ 50,000.00	*1*	$ 50,000
Additional Embankment Due to Settlement ($/yd^3$):	$ 4.00	*4,444*	$ 17,778
Estimated Total Cost of Deep Mixing Methods:			**$ 298,731**
Estimated Unit Cost of Deep Mixing Methods for Area Treated ($/ft^2$):			**$ 7.47**

Figure 28. Conceptual cost estimating tool (for deep soil mixing method) used in decision matrix (reproduced from FHWA SHRP2 tools)

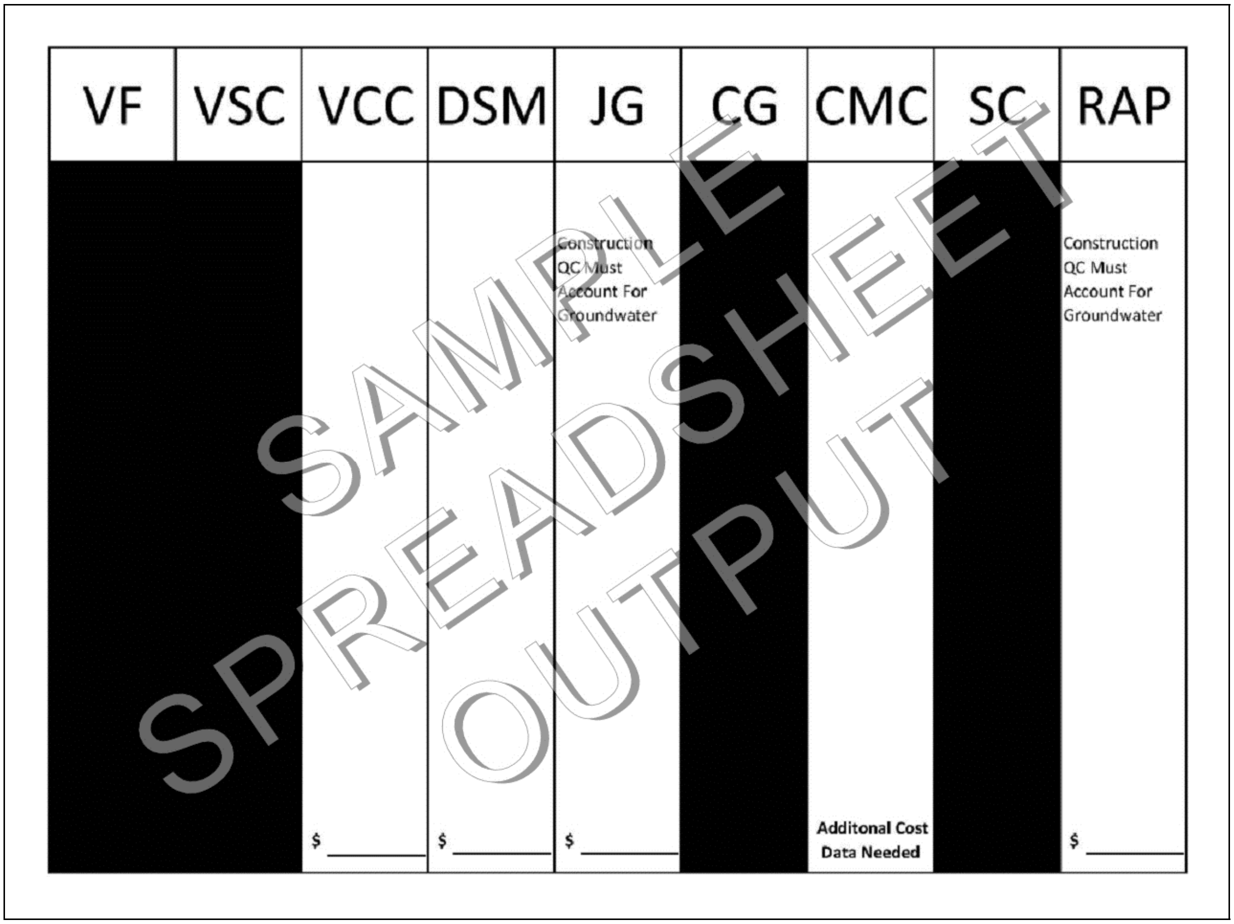


Figure 29. Sample output of decision matrix spreadsheet program

3.3.2 Decision Matrix – Flow Chart Version

Since the ground improvement methods have been created and tailored to improve specific soil types, the type of problem soil is the first consideration for the decision tree. Soil type eliminates methods which are not suitable for the specific type of soils encountered at the problem site. After evaluating the suitability of ground improvement methods based upon the soil type or types, the next steps in the decision tree focus on eliminating the remaining methods.

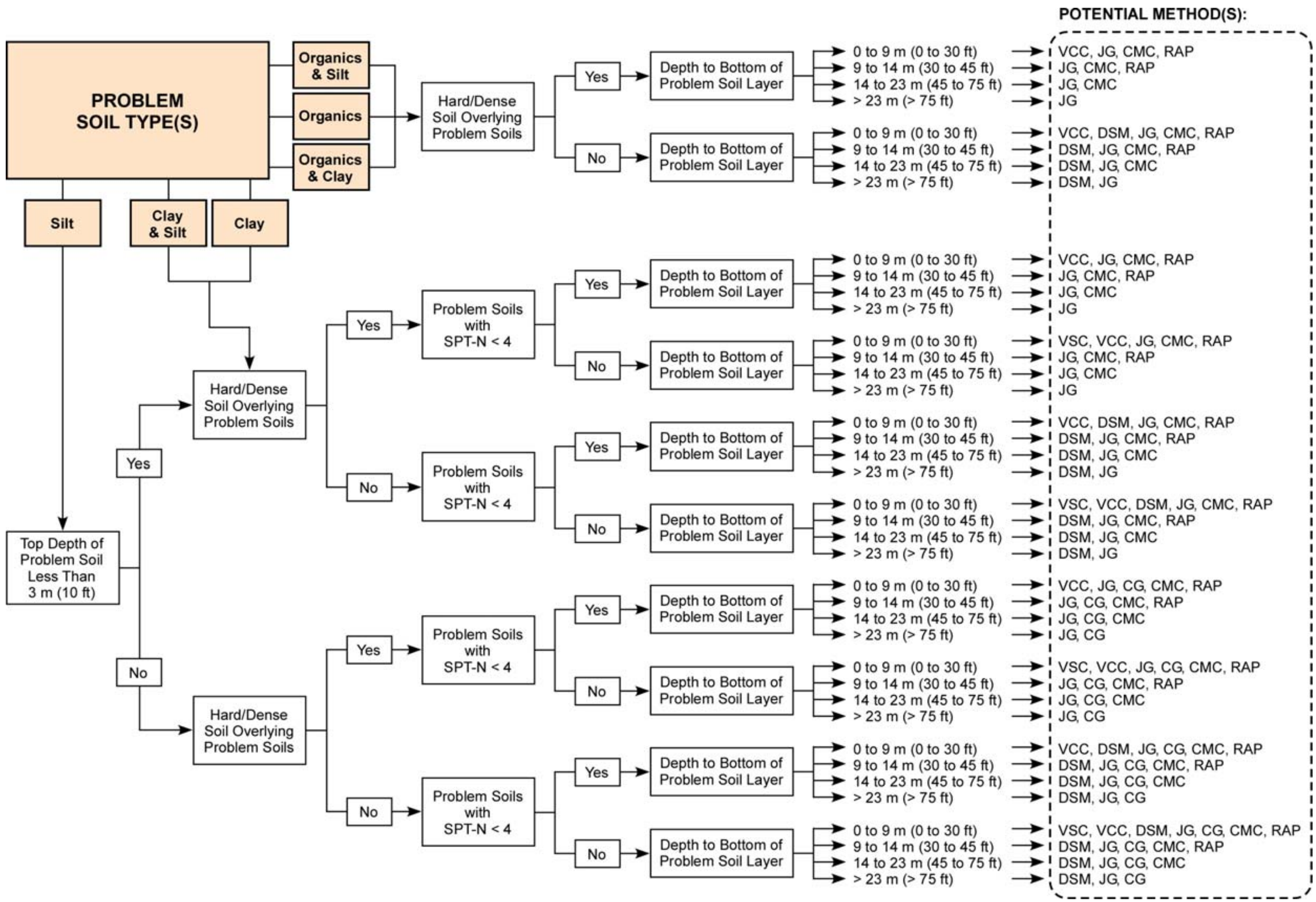
Since compaction grouting can be used to improve liquefiable soils, such as the silty sands and silts commonly encountered in Ohio's glacial interlobate moraine, kettles, and pro-glacial lake (glaciolacustrine) deposits, the second step of the flowchart considers the elimination of the compaction grouting method. Since compaction grouting uses high pressures within the grout column, substantial overburden pressure is required to allow these pressures to develop. Otherwise, heaving of the surface soils will result, and the surrounding soils within the upper three meters (ten feet) will not be significantly improved. Therefore, if problem soils are located within the upper three meters (ten feet) of the site, the soils within this zone will not be improved, and compaction grouting is not a suitable option.

Depending on the type of vertical column method, very hard or dense soils can be impenetrable to the ground improvement equipment. For example, deep soil mixing equipment consists of large diameter mixing implements that can encounter refusal on large gravel, cobbles, or boulders, on even hard and very dense soil layers. As a result, the next step in the decision tree evaluates the suitability of deep soil mixing due to the presence of overlying very dense granular or hard cohesive soil. Alternatively, jet grouting uses a much smaller diameter piece of equipment which can more readily extend through these very hard or dense zones. Controlled modulus columns, rammed aggregate piers, vibro-replacement stone columns, and vibro concrete columns utilize powerful equipment and/or vibration to penetrate the very hard or dense soil layers.

Among many other distinctions, vibro-replacement stone columns are more limited than rammed aggregate piers in application within very soft soils with SPT-N values less than 4. The next step of the decision tree eliminates vibro-replacement stone columns from consideration if the problem soils contain SPT-N values less than 4. Although somewhat similar, rammed aggregate piers can be utilized in these soft problem soils due to the unique ramming equipment and greater lateral consolidation. Where even poorer soils are present and lateral support is of concern, the rammed aggregate pier construction technique can be modified to provide the required lateral constraints. Furthermore, when the organic soils are the problem soils to be improved, the vibro-replacement stone columns are eliminated since they are not effective in these soils.

In addition to the aforementioned critical components, the bottom depth of the problem soil is the final step in the suitability evaluation of the decision tree. Based upon the commonly available achievable depths of the ground improvement equipment, each option is eliminated when the problem soil depth exceeds this value. While some methods offer newer/more specialized equipment capable of achieving deeper depths, the most commonly referenced maximum depths were utilized for the decision tree. While specialty equipment may be available, the cost implications will likely preclude the use of the ground improvement method requiring specialized equipment.

The decision matrix developed that can be used by ODOT to identify potential vertical column support ground improvement methods is given in Figure 30. It should be noted that this flow chart does not consider the angled installation from the sides of the roadway, since the reach of ground improvement methods under the roadway will be limited.



Method symbols: VSC=Vibro-replacement stone columns, VCC=Vibro concrete columns, DSM=Deep soil mixing, JG=Jet grouting, CG=Compaction grouting, CMC=Controlled modulus columns, and RAP=Rammed aggregate piers

Figure 30. Decision matrix developed for vertical column ground improvement method selection

3.4 Summary and Conclusions

The critical components of the decision matrix have been identified mainly during the literature review. The development of the decision matrix has initially started using spreadsheets. The draft version of this decision matrix spreadsheets was used during the project to assess the feasibility of vertical support column methods for the sites evaluated for implementation. Later, a more traditional flow chart version of the decision matrix was developed for the purpose of this report and for ODOT to use in preliminary evaluation of the vertical column support methods for future projects. Both the flow chart and the draft spreadsheet version of the decision matrix have been presented.

The decision matrix developed considers only the ground improvement alternatives of the vertical column support systems. The deep foundation systems, at least some of them, would be technically feasible to remediate the existing roadway settlement problems for most situation. However, they would be prohibitively expensive alternative. Therefore, the deep foundation systems were not considered in the development of decision matrix developed.

CHAPTER 4. SUBSURFACE INVESTIGATIONS

4.1 Introduction

Several sites experiencing subgrade settlement problems have been considered for possible implementation of vertical column support method(s) in the field. The following nine sites have been considered by ODOT for this purpose:

- SUM-77-18.32
- SUM-224-13.14
- STA-44-18.23
- SUM-93-7.86
- STA-77-08.30
- GEA-168-07.60
- SUM-8-07.60
- WYA-53-16.90
- SUM-271-10.22

ODOT ruled out six of these sites and selected three of them, SUM-77-18.32, SUM-224-13.14, and STA-44-18.23, for further assessment for this research project, i.e., evaluating the sites for subsurface investigations and possible implementation of vertical column support method(s). After the review of available documents, SUM-77-18.32 site had also been eliminated from further consideration by ODOT for not being a typical roadway settlement problem intended for this research project.

Conducting a survey of all ODOT districts has been considered to get detailed information on the pavement subgrade settlement problems that they are experiencing in their own districts, learn about their approach to remediate these problems, and find out if there are any sites in their districts that may be suitable for possible implementation. ODOT indicated that the sites identified in District 4 would be enough to choose from for the implementation of some of the vertical column support methods.

Finally, the remaining two sites having subgrade settlement and pavement distress proceeded to subsurface investigations and assessment for possible field implementation of vertical column support method(s). These two sites were:

- SUM-224-13.14 (will be referred to as SUM-224 site)
- STA-44-18.23 (will be referred to as STA-44 site)

Both sites were located in the northeastern part of the State of Ohio, in close proximity to the City of Akron. The locations of the two sites are shown in Figure 31.

In order to assess the feasibility of different vertical column support methods for possible implementation at the selected sites, extensive subsurface investigations for detailed site characterization were needed. Subsurface investigations, including drilling and sampling, cone penetration (CPT) soundings, pressuremeter tests, and laboratory testing, were conducted for the two sites selected by ODOT for this project. Overview of the SUM-224 and STA-44 sites and the subsurface investigations performed at these sites are explained in this chapter.

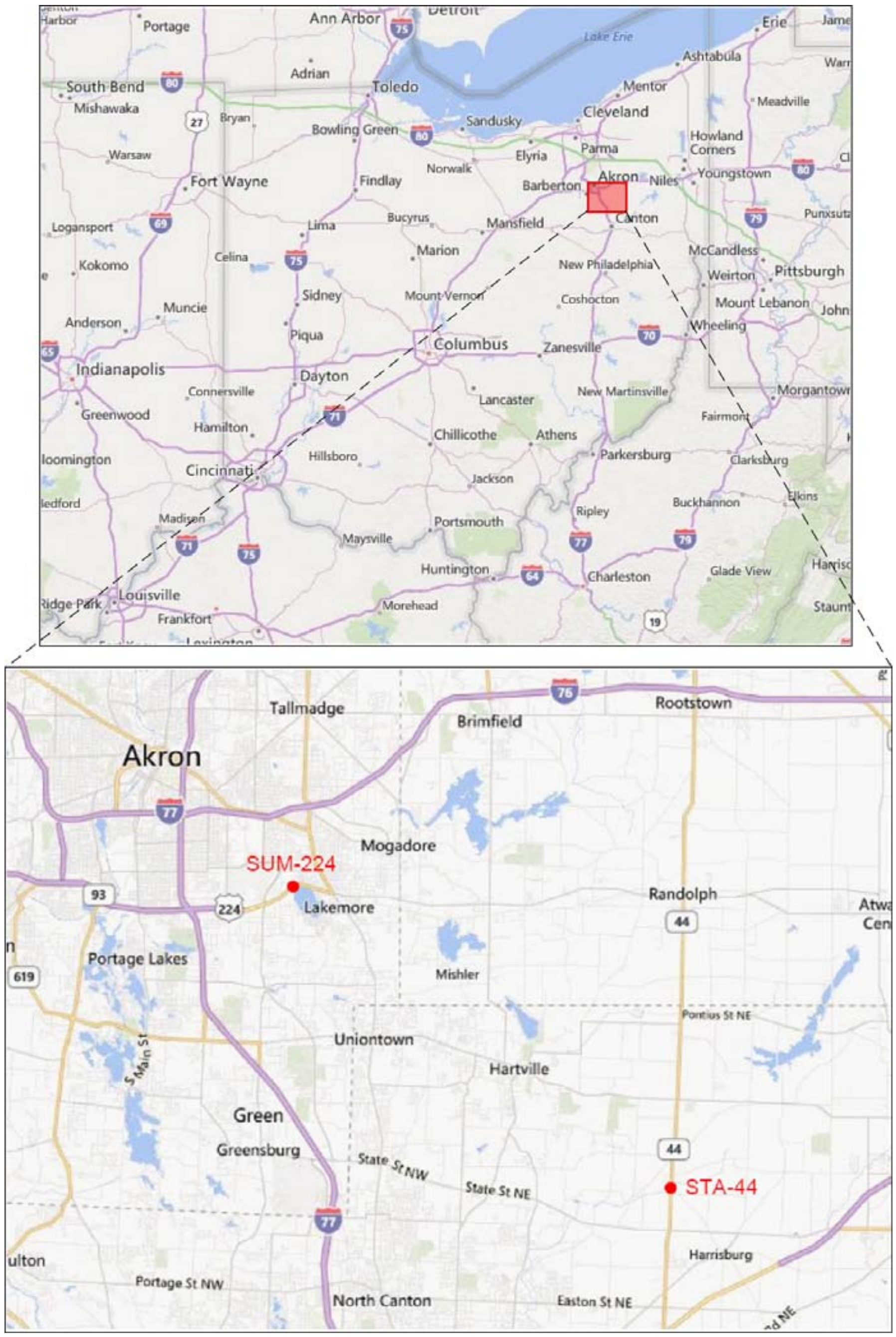


Figure 31. Location of SUM-224 and STA-44 sites

4.2 SUM-224 Site

4.2.1 Site History and Background

The site (SUM-224-13.14) is located in Summit County on State Route 224 in Springfield Township, southeast of Akron, Ohio. The roadway is adjacent to a lake, Springfield Lake, as shown in Figure 32. The roadway was originally built as a two-lane roadway and it was extended to four lanes in 1993. A city engineer involved with the extension project indicated that lightweight fill (Elastizell engineered fill) was used as part of the subgrade to reduce loads on soft soils below. In addition, it was also noted by the city engineer that light poles were not installed on the north side of the roadway, due to an inadequate bearing capacity for the foundations.

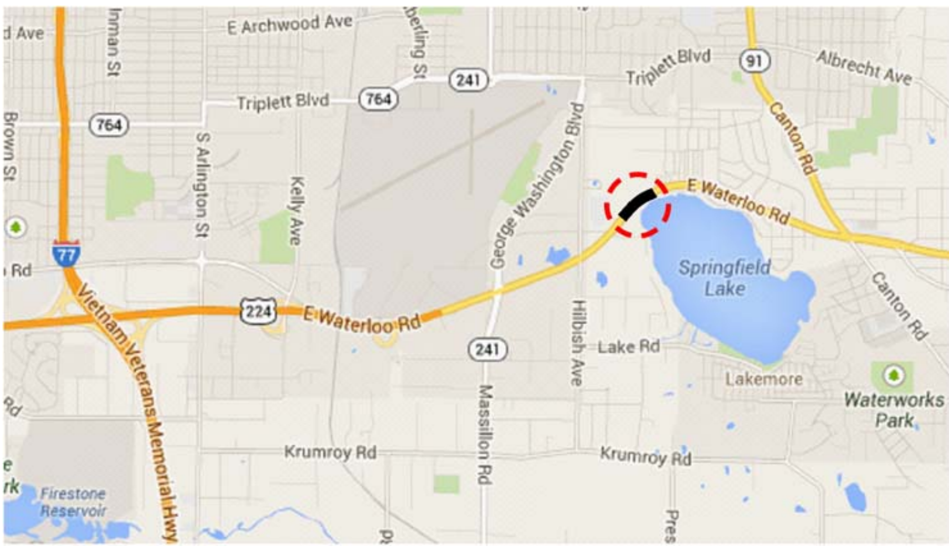


Figure 32. SUM-224 site map

After completion of the extension project in 1993, an approximately 180 m (600 ft) long section of the roadway, especially on the westbound lanes, continued to settle causing significantly poor pavement conditions with dips, cracks, and large potholes. Several repair projects, involving patching and complete lane resurfacing, have been done on this problem section of the roadway. The whole roadway section was resurfaced approximately five years ago.

The problem section of the roadway has continued to settle after the resurfacing was completed. This is probably due to the additional pavement thickness needed to level the previously settled roadway. ODOT engineers became more alert about the continued settlements once the settlements reached a level such that the curb on the north side had sunk below the existing roadway grade. The roadway has a curve at this location and the outer edge of the curve is on the north side of the roadway. The super-elevation on the north side of the roadway was also lost due to the settlements. Some of the pictures taken at the site showing some of the settlement problems are presented in Figure 33.

Figure 33. Pavement settlement, lost super-elevation, sunk curb, and cracked pavement problems at SUM-224 site

4.2.2 Overview of Field Investigations

Detailed subsurface investigations were conducted along the approximately 180 m (600 ft) long section of the roadway having ongoing settlement problems. The field work of the investigations was conducted over a period of 19 non-consecutive days. The field work included drilling, standard penetration testing (SPT), soil sampling with split-spoon and Shelby tubes, cone penetration testing (CPT), and pressuremeter testing. The dates of field work performed are summarized in Table 7. In addition, one monitoring well and one inclinometer casing were installed during the field work. The field work performed at the site was monitored full-time by the research team, because of the complexity of soil conditions at the site requiring instant decisions, the importance of the investigations performed, and the sensitivity of the data and samples collected.

Table 7. Field work breakdown by test and dates at SUM-224 site

Work Type	Date of Investigations	Number of Days
Drilling (SPT and sampling)	Aug 20, 2013 – Aug 23, 2013	4
	Aug 26, 2013 – Aug 29, 2013	4
	Subtotal:	**8**
Pressuremeter	Aug 30, 2013	1
	Sep 3, 2013	1
	Oct 7, 2013 – Oct 8, 2013	2
	Subtotal:	**4**
CPT	Sep 23, 2013 – Sep 26, 2013	4
	Oct 7, 2013 – Oct 9, 2013	3
	Subtotal:	**7**
	TOTAL:	**19**

Figure 34 shows the proposed drilling and CPT sounding locations for the field work. In the figure, the locations starting with letter “B” show the soil boring locations and the location designations starting with letter “C” show the locations of CPT soundings. The letter “I” at the end of boring B-012-0-13 indicates that the inclinometer installation was planned at this location. Similarly, the letter “M” at the end of boring B-016-0-13 indicates that the groundwater monitoring well installation was planned at this location.

While the boreholes were drilled at the proposed locations shown in Figure 34, there were a few changes at the CPT sounding locations due to the obstructions above the ground surface, hindering the ability of rig to center and level at the planned location, or below the ground surface, preventing tests from advancing to the desired depths. The changes to the proposed locations are listed in the following:

- C-001-0-13 was relocated NW approximately 4 m (13 ft) to the shoulder

- C-006-0-13 was relocated SW approximately 1.5 m (5 ft)
- Additional CPT (named C-009-1-13) was performed 1.5 m (5 ft) NE of C-009-0-13
- Additional CPT (named C-010-1-13) was performed 9 m (30 ft) SW of C-010-0-13
- C-019-0-13 was relocated SW approximately 2.1 m (7 ft)

After the completion of subsurface investigations at the site, surveying of all investigation locations was going to be performed by ODOT District 4. However, the surveying at the site was not conducted, because of the difficult winter and weather conditions, and the traffic maintenance needed during the surveying. Therefore, the boring locations presented in this report and its attachments are approximate locations and should be treated as such.

Figure 34. Proposed drilling and CPT sounding locations for SUM-224 site

4.2.3 Drilling and Sampling

The drilling and sampling was performed by Bowser-Morner, Inc. of Dayton, Ohio. Two drill rigs, a truck mounted and an ATV rig (Figure 35), were utilized during the first two days of drilling. Only one rig, the ATV rig, was used for the remaining work after the third day of field work. The drilling was performed using 8.25 cm (3.25 in.) diameter hollow stem augers. At the monitoring well and inclinometer installation locations 10.80 cm (4.25 in.) diameter augers were utilized.

Figure 35. Truck mounted and ATV rigs used at SUM-224 site

The preliminarily borehole depths proposed at the beginning of investigations and the actual depths drilled at the site are given in Table 8. Initially, the termination depth for each borehole was specified to be the proposed depth or the depth at which a minimum of 6.1 m (20 ft) thick of reasonable SPT blow count material is encountered, whichever is deeper. A reasonable SPT blow count material for this investigation was defined as either granular soils with the SPT-N number greater than 30 or cohesive soils with the SPT-N number greater than 10. The reason for specifying the 6.1 m (20 ft) of reasonable material was to ensure an adequate amount of dense/stiff strata was available for vertical column support methods that require terminating in such soils, in case they would be the most feasible methods for possible implementation. However, on the first two boreholes, the high blow count material was not encountered until much deeper than expected. With schedule and budget considerations, it was decided that the initial parameters would only need to be met on four of the eight boreholes covering the area at the site. For the other four boreholes, the drilling would be terminated after one suitable SPT-N number is measured after the planned depth is reached. The boreholes in which 6.1 m (20 ft) of good blow count material was sampled include: B-007-0-13, B-008-0-13, B-015-0-13, and B-016-0-13.

Table 8. Borehole depths at SUM-224 site

Borehole ID	Proposed Depth (m / ft)	Drilled Depth (m / ft)
B-004-0-13	18.3 / 60	22.9 / 75
B-007-0-13	21.3 / 70	38.1 / 125
B-008-0-13	24.4 / 80	32.5 / 106
B-012-0-13	24.4 / 80	27.7 / 91
B-013-0-13	21.3 / 70	25.9 / 85
B-015-0-13	21.3 / 70	30.5 / 100
B-016-0-13	21.3 / 70	30.5 / 100
B-020-0-13	18.3 / 60	19.8 / 65
TOTAL:	**171 / 560**	**228 / 747**

Standard penetration tests (SPT) were performed and split-spoon samples were taken at 0.76 m (2.5 ft) increments for the first 3.05 m (10 ft) of each borehole and at 1.52 m (5 ft) increments from depths of 3.05 m (10 ft) to the bottom of each borehole. A total of 165 split-spoon samples were taken at this site. Undisturbed samples using Shelby tube samples were taken at selected depths from three different boreholes. A total of 10 undisturbed samples were taken at this site. Table 9 provides a summary of the number of samples collected from each borehole, as well as the depths at which the undisturbed samples were taken.

4.2.4 Pressuremeter Tests

Pressuremeter tests were conducted at four different locations nearby boreholes B-015-0-13, B-016-0-13, and B-020-0-13. There were two locations near borehole B-020-0-13. The pressuremeter testing locations were placed 2.1 to 3 m (7 to 10 ft) away from the corresponding borehole locations. The tests were conducted in the holes drilled specifically for the pressuremeter testing. Bowser-Morner drilled the holes and geologists from ODOT's Office of Geotechnical Engineering performed the tests using the pressuremeter equipment owned by ODOT. The equipment used is shown in Figure 36.

A total of 17 pressuremeter tests were attempted at this site and nine of them were completed successfully. The remaining eight tests were not successfully completed due to various reasons. The majority of the tests failed due to either sands heaving into the auger, and not allowing the pressuremeter probe to be inserted, or because the hole would not stay open due to the non-cohesive nature of the soil. One of the tests at this site failed due to mechanical issues with the pressuremeter probe, and one test failed due to the pressuremeter probe not being extended past the auger into the soil completely. Table 10 gives the boreholes and depths at which the pressuremeter tests were conducted.

Table 9. Number of soil samples collected at SUM-224 site

Borehole ID	Split-spoon Samples	Shelby Tube Samples	Undisturbed Sample Depths (m / ft)
B-004-0-13	17	-	-
B-007-0-13	27	-	-
B-008-0-13	23	3	5.0 / 16.5 27.9 / 91.5 32.2 / 105.5
B-012-0-13	20	6	3.5 / 11.5 6.6 / 21.5 9.6 / 31.5 12.6 / 41.5 15.7 / 51.5 18.7 / 61.5
B-013-0-13	19	-	-
B-015-0-13	22	-	-
B-016-0-13	22	-	-
B-020-0-13	15	1	8.1 / 26.5
TOTAL:	**165**	**10**	-

Table 10. Pressuremeter testing summary at SUM-224 site

Borehole ID	Number of Tests Attempted	Number of Successful Tests	Test Depths (m / ft)
B-015-1-13	5	2	10.7 / 35 13.7 / 45
B-016-1-13	5	3	9.1 / 30 12.2 / 40 23.5 / 77
B-020-1-13	2	1	6.1 / 20
B-020-2-13	5	3	3.7 / 12 9.1 / 30 19.5 / 64
TOTAL:	**17**	**9**	-

(a) Pressuremeter control panel (b) Pressuremeter probe

Figure 36. Pressuremeter test equipment

4.2.5 Cone Penetration Tests (CPT)

Cone penetration tests (CPT) were conducted at 17 different locations at this site. The tests were conducted by ODOT's Office of Geotechnical Engineering using ODOT's CPT rig (Figure 37). Each CPT test was run until the tip resistance reached 25 MPa (3.63 ksi) or higher, or the cone inclination angle experienced a jump past six degrees or gradually reached an inclination angle of seven degrees. These parameter limits for termination were set to prevent damaging the CPT equipment.

Figure 37. ODOT's CPT rig used for cone penetration testing

Depths of CPT termination at this site ranged from 5.0 m (16.4 ft) to 36.3 m (119.09 ft). Each CPT location was pre-drilled approximately 2.1 m (7 ft) by Bowser-Morner to penetrate a hard sub-base layer present under some parts of the roadway. However, in some locations the cone hit refusal at depths shallower than 3.5 m (11.55 ft) and the holes needed to be pre-drilled deeper a second time. This was the case at C-006-0-13, C-017-0-13, and C-019-0-13. At the C-017-0-13 location, the hole needed to be pre-drilled a total of three times. When attempting to pre-drill C-006-0-13 a second time, the drillers were unable to penetrate the hard sub-base with their augers so the location was moved approximately 1.5 m (5 ft) in the southwest direction.

During the testing at the C-001-0-13, C-009-0-13, and C-010-0-13 locations, the cone passed the hard sub-base layer, but did not penetrate deep enough to provide the desired information about deeper soils. Because of this, it was decided to pre-drill new holes nearby the original locations and attempt to test again.

Dissipation tests were also conducted in conjunction with the CPT soundings. These tests measure the rate at which the excess pore pressure dissipates from the soil. A total of 14 different dissipation tests were attempted/conducted at varying depths at eight different CPT locations. A summary of CPT and dissipation testing is presented in Table 11.

Table 11. Cone penetration testing summary at SUM-224 site

Test ID	Attempt Number	Depth of Penetration (m / ft)	Number of Dissipation Tests	Dissipation Test Depths (m / ft)
C-001-0-13	1	14.46 / 47.44	-	-
C-001-1-13	1	15.10 / 49.54	-	-
C-002-0-13	1	18.34 / 60.16	-	-
C-003-0-13	1	26.42 / 86.66	-	-
C-005-0-13	1	36.30 / 119.09	1	13.7 / 44.9
C-006-0-13	1	2.16 / 7.08	-	-
C-006-1-13	1	25.30 / 83.01	-	-
C-009-0-13	1	9.08 / 29.78	1	7.0 / 23
C-009-1-13	1	30.56 / 100.24	1	21.5 / 70.6
C-010-0-13	1	7.00 / 22.96	-	-
C-010-0-13	2	5.00 / 16.4	-	-
C-010-1-13	1	29.30 / 96.10	1	15.6 / 51.2
C-011-0-13	1	18.52 / 60.75	2	8.5 / 28 17.7 / 58
C-014-0-13	1	20.58 / 67.50	3	10.7 / 35 13.7 / 45 18.3 / 60
C-017-0-13	1	2.98 / 9.77	-	-
C-017-0-13	2	3.52 / 11.55	-	-
C-017-0-13	3	25.02 / 82.09	3	12.2 / 40 17.7 / 58 23.5 / 77
C-018-0-13	1	20.68 / 67.83	-	-
C-019-0-13	1	2.12 / 6.95	-	-
C-019-1-13	1	2.00 / 6.76	-	-
C-019-1-13	2	14.34 / 47.04	2	8.4 / 27.5 14.3 / 47
TOTAL:	**21**	**328.78 / 1,078.67**	**14**	-

4.2.6 Instrumentation

One monitoring well to observe the groundwater levels at the site and one inclinometer casing to detect any soil movements were installed in two of the boreholes. The monitoring well was installed in borehole B-016-0-13 after the completion of drilling and it was 21.3 m (70 ft) deep. The inclinometer casing was installed in borehole B-012-0-13 after the completion of drilling and it was 27.1 m (89 ft) deep.

4.2.7 Laboratory Testing

An extensive laboratory testing program was conducted on the soil samples collected during drilling operations at the SUM-224 site. The extensive testing was needed for detailed subsurface characterization to assess the feasibility of the vertical column support methods investigated, to analyze costs and benefits of feasible methods, and to be used for the design of vertical column supports during the implementation phase.

All the laboratory tests presented in this report were performed by Bowser-Morner, Inc. of Dayton, Ohio in their AASHTO accredited and ODOT approved laboratories. The laboratory tests conducted are summarized in the following.

<u>Tests performed on samples collected by split-spoon sampler (i.e., disturbed samples):</u>

- Fifty-seven (57) unified soil classification (USCS) and complete ODOT soil classification tests were performed in accordance with ASTM D422, D2216, D2487, D4318, D3282, and ODOT specifications.

- Seven (7) grain-size analysis tests were performed in accordance with ASTM D422 and D2216.

- One-hundred-thirty-five (135) moisture-content determinations were made in accordance with ASTM D2216.

- Twenty-four (24) moisture, ash and organic matter of peat and other organic soils loss on ignition (LOI) tests were performed in accordance with ASTM D2974. These tests were performed to determine the quantity of organic matter.

<u>Tests performed on samples collected by Shelby tubes (i.e., undisturbed samples):</u>

- For each of the ten (10) undisturbed samples collected, a unified soil classification and ODOT soil classification test were performed in accordance with ASTM D422, D2216, D2487, D4318, D3282, and ODOT specifications.

- Seven (7) unconfined compressive tests were performed. The unconfined compressive tests were performed in accordance with the ASTM D2166 specifications. The samples were loaded at a constant axial strain rate of 1% per minute. The samples were loaded to failure or 20% strain, whichever happened first.

- Two (2) unconsolidated-undrained (UU) triaxial tests were performed. These tests were performed in accordance with ASTM D2850. The samples were loaded at a constant axial strain rate of 1% per minute. The samples were loaded to failure or 20% strain, whichever happened first.

- Three (3) consolidation tests were performed. These tests were performed in accordance with the ASTM D2435.

- Of the ten undisturbed samples collected, six (6) samples underwent specific-gravity (G_s) tests which were performed in accordance with ASTM D854. The specific gravity values were used to determine the void ratio of the soils, which is a required parameter for the other undisturbed sample tests.

A summary of all the laboratory tests performed on soil samples collected from SUM-224 site is provided in Table 12.

Table 12. Summary of laboratory testing for SUM-224 site

Type of Testing	**Number of Tests**
USCS / Complete ODOT classification	57
Grain size analysis	7
Moisture content	135
LOI	24
Specific gravity	6
Unconfined compression	7
Triaxial	2
Consolidation	3

4.3 STA-44 Site

4.3.1 Site History and Background

The site (STA-44-18.23) is located in Stark County on State Route 44 in Marlboro Township, southeast of Akron, Ohio. It is also to the southeast of the SUM-224 site. STA-44 site map is provided in Figure 38. The two lane roadway was originally built in 1924. There is almost no shoulder on either side of the roadway at this location. A major embankment stabilization project was conducted just north of the site around 2006.

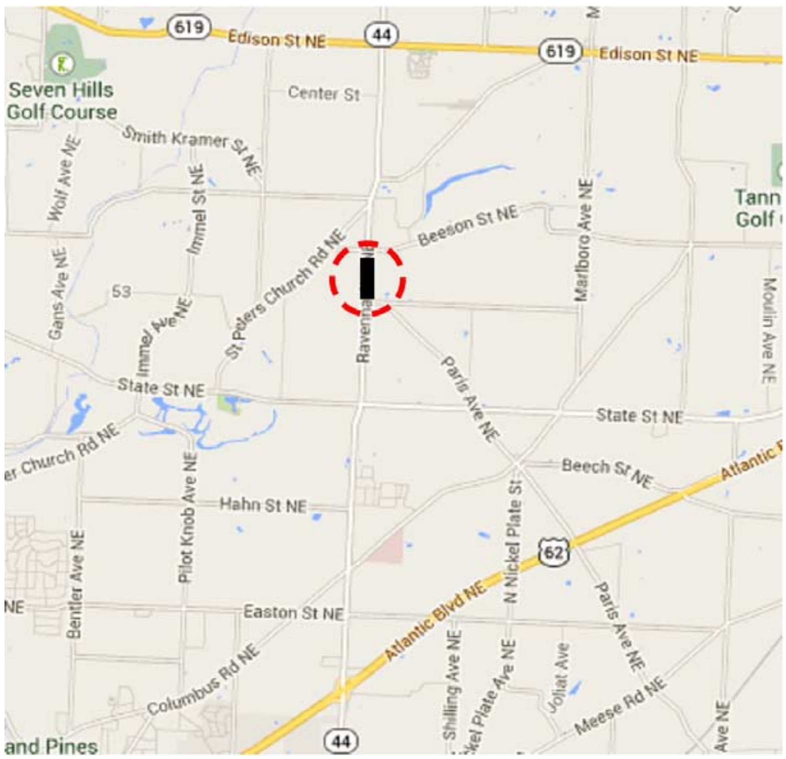


Figure 38. STA-44 site map

An approximately 240 m (800 ft) long section of the roadway has been experiencing subgrade settlements causing pavement distress. The pavement distress was more severe along an approximately 180 m (600 ft) long section of the roadway with several settlement dips along the roadway, cracks on the pavement, movements and separations on the shoulders, and various size potholes. The sporadic asphalt patching work previously completed on the roadway was also visually apparent. The communications with the residents in the area and old photos of the roadway available online indicated that there was some berm stabilization work conducted in the past along this problem section of the roadway. Some of the pictures taken at the site showing settlement problems are presented in Figure 39.

Figure 39. Pavement distress, settlement dips, cracked and patched pavement problems at STA-44 site

4.3.2 Overview of Field Investigations

Detailed subsurface investigation was conducted along an approximately 300 m (1,000 ft) long section of the roadway having ongoing settlement problems. The field work of the investigation was conducted over a period of 13 non-consecutive days. The field work included drilling, standard penetration testing (SPT), soil sampling with split-spoon and Shelby tubes, cone penetration testing (CPT), and pressuremeter testing. The dates of the field work performed are summarized in Table 13. Two inclinometer casings were installed at the site during field work. In addition, two monitoring wells were also planned to be installed at the site, however, they could not be installed due to an artesian aquifer encountered at various depths across the site. The field work performed at the site was monitored full-time by the research team, because of the complexity of soil conditions at the site requiring instant decisions, the importance of the investigations performed, and the sensitivity of the data and samples collected.

Table 13. Field work breakdown by test and dates at STA-44 site

Work Type	Date of Investigations	Number of Days
Drilling (SPT and sampling)	Sep 4, 2013 – Sep 6, 2013	3
	Sep 9, 2013 – Sep 11, 2013	3
	Subtotal:	**6**
Pressuremeter	Sep 12, 2013	1
	Oct 9, 2013 – Oct 10, 2013	2
	Subtotal:	**3**
CPT	Sep 16, 2013 – Sep 19, 2013	4
	TOTAL:	**13**

Figure 40 shows the proposed drilling and CPT sounding locations for the field work. In the figure, the locations starting with letter “B” show the soil boring locations and the location designations starting with letter “C” shows the location of the CPT soundings. The letter “I” at the end of borings B-006-0-13 and B-007-0-13 indicates that the inclinometer installations were planned at these locations. Similarly, the letter “M” at the end of borings B-001-0-13 and B-009-0-13 indicates that the groundwater monitoring well installations were planned at these locations. Almost all of the drilling and CPT locations shown in Figure 34 have shifted approximately 0.6 to 0.9 m (2 to 3 ft) away from the shoulders towards the center of the road to allow space for setting up the rigs on the roadway.

The research team initially planned some of the drilling and CPT locations in the fields adjacent to the roadway to determine the extent of problematic soils causing settlements and to help with the design of vertical column support methods for implementation. However, ODOT District 4 requested to stay within the right-of-way for all explorations. Therefore, the drilling and CPT locations were located on the roadway as shown in Figure 40.

After the completion of subsurface investigations at the site, surveying of all investigation locations was going to be performed by ODOT District 4. However, the surveying at the site was not conducted, because of the difficult winter and weather conditions, and the traffic maintenance needed during the surveying. Therefore, the boring locations presented in this report and its attachments are approximate locations and should be treated as such.

4.3.3 Drilling and Sampling

The drilling and sampling was performed by Bowser-Morner, Inc. of Dayton, Ohio using an ATV rig. The drilling was performed using 8.25 cm (3.25 in.) diameter hollow stem augers. At the monitoring well and inclinometer installation locations 10.80 cm (4.25 in.) diameter augers were utilized.

The preliminarily borehole depths proposed at the beginning of the investigations and the actual depths drilled at the site are given in Table 14. Initially, the termination depth for each borehole was specified to be the proposed depth or the depth at which a minimum of 6.1 m (20 ft) thick of reasonable SPT blow count material is encountered, whichever was deeper. A reasonable SPT blow count material for this investigation was defined as either granular soil with the SPT-N number greater than 30 or cohesive soils with the SPT-N number greater than 10. The reason for specifying the 6.1 m (20 ft) thick reasonable material was to ensure an adequate amount of dense/stiff strata was available for vertical column support methods that require terminating in such, in case they would be the most feasible methods for possible implementation.

Table 14. Borehole depths at STA-44 site

Borehole ID	Proposed Depth (m / ft)	Drilled Depth (m / ft)
B-001-0-13	18.3 / 60	15.2 / 50
B-004-0-13	18.3 / 60	21.8 / 71.5
B-006-0-13	18.3 / 60	21.5 / 70.5
B-007-0-13	18.3 / 60	24.1 / 79
B-009-0-13	18.3 / 60	22.9 / 75
B-013-0-13	18.3 / 60	18.3 / 60
B-016-0-13	18.3 / 60	19.8 / 65
TOTAL:	**128 / 420**	**144 / 471**

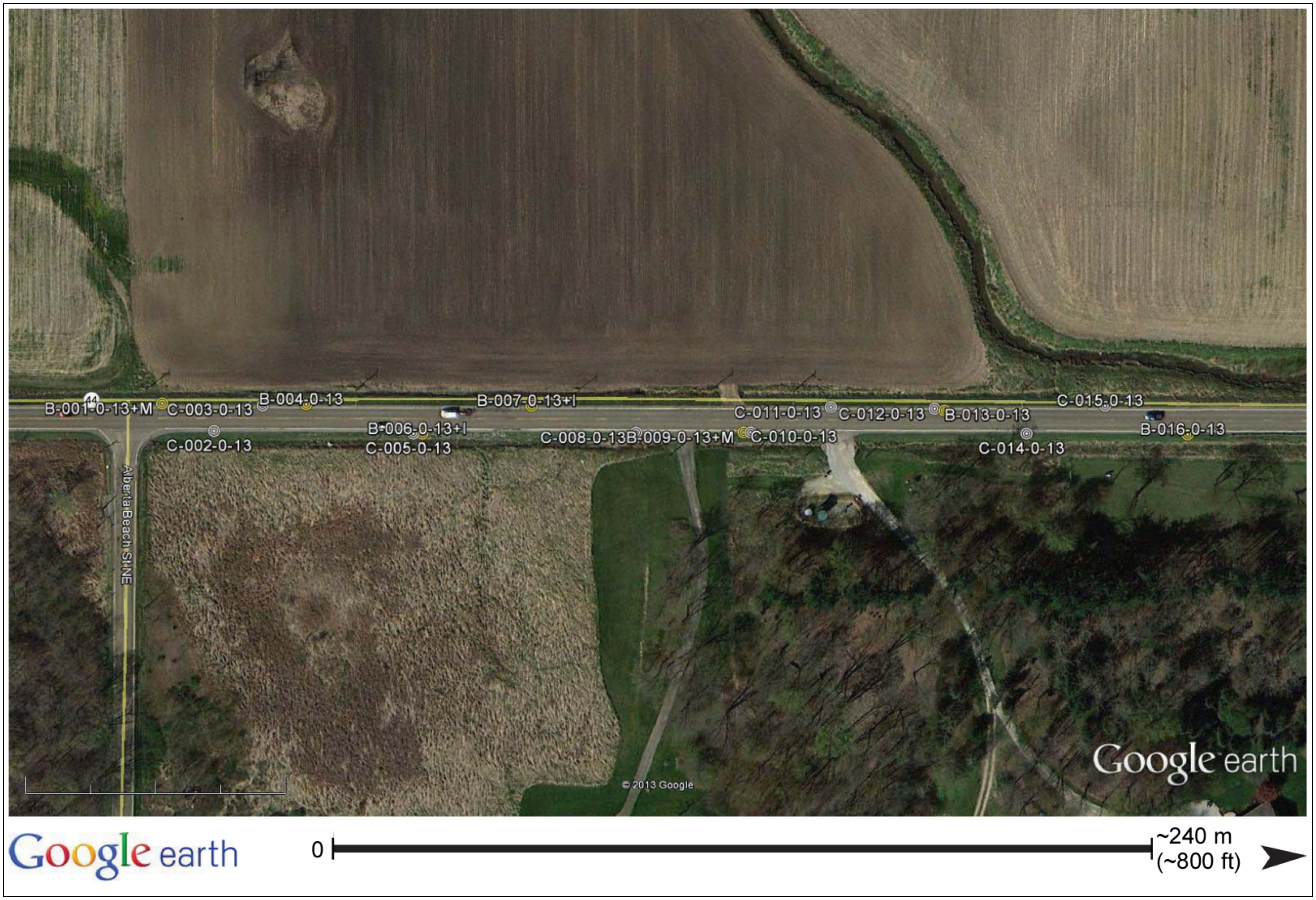


Figure 40. Proposed drilling and CPT sounding locations for STA-44 site

Standard penetration tests (SPT) were performed and split-spoon samples were taken at 0.76 m (2.5 ft) increments for the first 3.05 m (10 ft) of each borehole and at 1.52 m (5 ft) increments from depths of 3.05 m (10 ft) to the bottom of each borehole. A total of 109 split-spoon samples were taken at this site. Undisturbed samples using Shelby tube samplers were taken at selected depths from three different boreholes. A total of 13 undisturbed samples were taken at this site. Table 15 provides a summary of the number of samples collected from each borehole, as well as the depths at which the undisturbed samples were taken.

Table 15. Number of soil samples collected at STA-44 site

Borehole ID	Split-spoon Samples	Shelby Tube Samples	Undisturbed Sample Depths (m / ft)
B-001-0-13	12	4	5.0 / 16.5 8.1 / 26.5 12.6 / 41.5 14.2 / 46.5
B-004-0-13	17	-	-
B-006-0-13	16	6	3.5 / 11.5 6.5 / 21.5 9.6 / 31.5 12.6 / 41.5 15.7 / 51.5 18.7 / 61.5
B-007-0-13	19	-	-
B-009-0-13	16	-	-
B-013-0-13	14	3	5.0 / 16.5 9.6 / 31.5 14.2 / 46.5
B-016-0-13	15	-	-
TOTAL:	**109**	**13**	-

4.3.4 Pressuremeter Tests

Pressuremeter tests were conducted at three different locations nearby boreholes B-013-0-13 and B-006-0-13. There were two testing locations near borehole B-013-0-13. The pressuremeter testing locations were placed 2.1 to 3 m (7 to 10 ft) from corresponding borehole locations. The tests were conducted in the holes drilled specifically for the pressuremeter testing.

Bowser-Morner drilled the holes and the geologists from ODOT's Office of Geotechnical Engineering performed the tests using the pressuremeter equipment owned by ODOT.

A total of 12 pressuremeter tests were attempted at this site and nine of them were completed successfully. Two of the tests that were not successfully completed due to sands heaving into the auger, not allowing the pressuremeter probe to be inserted, or because the hole would not stay open due to the non-cohesive nature of the soil. One of the tests failed due to large incremental pressures causing the soil to fail too quickly without collecting enough data for the analysis of test results. Table 16 gives the boreholes and depths at which the pressuremeter tests were conducted.

Table 16. Pressuremeter testing summary at STA-44 site

Borehole ID	Number of Tests Attempted	Number of Successful Tests	Test Depths (m / ft)
B-006-1-13	5	5	3.7 / 12 6.1 / 20 8.2 / 27 10.7 / 35 13.7 / 45
B-013-1-13	2	1	6.4 / 21
B-013-2-13	5	3	4.6 / 15 9.4 / 31 12.2 / 40
TOTAL:	**12**	**9**	-

4.3.5 Cone Penetration Tests (CPT)

Cone penetration tests (CPT) were conducted at nine different locations at this site. The tests were conducted by ODOT's Office of Geotechnical Engineering using ODOT's CPT rig. Each CPT test was run until the tip resistance reached 25 MPa (3.63 ksi) or higher, or the cone inclination angle experienced a jump past six degrees or gradually reached an inclination angle of seven degrees. These parameter limits for termination were set to prevent damaging the CPT equipment.

Depths of termination at this site ranged from 10.4 m (34.2 ft) to 24.8 m (81.5 ft). Each CPT location was pre-drilled approximately 2.1 m (7 ft) by Bowser-Morner to penetrate any hard sub-base layer that may be present under the roadway. However, at the C-012-0-13 location the cone hit refusal at a shallow depth of 2.24 m (7.35 ft). Then the CPT rig was moved slightly and the test was restarted.

Dissipation tests were also conducted in conjunction with the CPT soundings. These tests measure the rate at which the excess pore pressure dissipates from the soil. A total of eight different dissipation tests were attempted/conducted at varying depths at five different CPT locations. During these dissipation tests extremely high pore pressures were measured at certain depths which could possibly be due to the artesian conditions in the area. A summary of CPT and dissipation tests performed is presented in Table 17.

Table 17. Cone penetration testing summary at STA-44 site

Test ID	Attempt Number	Depth of Penetration (m / ft)	Number of Dissipation Tests	Dissipation Test Depths (m / ft)
C-002-0-13	1	19.14 / 62.78	-	-
C-003-0-13	1	14.88 / 48.81	-	-
C-005-0-13	1	17.36 / 56.96	3	3.7 / 12.1 9.7 / 31.8 15.5 / 50.9
C-008-0-13	1	18.34 / 60.16	1	6.2 / 20.3
C-010-0-13	1	18.90 / 62.01	-	-
C-011-0-13	1	10.44 / 34.24	-	-
C-012-0-13	1	2.24 / 7.35	-	-
C-012-0-13	2	18 / 59.04	1	6.4 / 21
C-014-0-13	1	21.62 / 70.91	1	20.2 / 66.2
C-015-0-13	1	24.84 / 81.48	2	16.3 / 53.5 24.6 / 80.7
TOTAL:	**10**	**165.76 / 543.83**	**8**	**-**

4.3.6 Instrumentation

Two inclinometer casings to observe any soil movements were installed in boreholes B-006-0-13 and B-007-0-13 after the completion of drilling at these locations. The casing installed was 19.8 m (65 ft) deep in B-006-0-13 and 23.5 m (77 ft) deep in B-007-0-13.

Two monitoring wells initially planned for the site could not be installed during the drilling operations due to artesian aquifer encountered at various depths across the site. The research team later proposed to install monitoring wells on the side of the roadway and developed drawings for the installation. Due to the ODOT District 4’s request of staying within the right-of-way, which is very narrow at the site, for any drilling/installations and related safety concerns of monitoring wells being close to the roadway, no monitoring wells were installed at the STA-44 site.

4.3.7 Artesian Conditions

An artesian aquifer was encountered at the STA-44 site at a number of drilling and CPT locations. In some of the boreholes the artesian water would come up through the hollow stem of the auger as well as from outside the auger at ground level as shown in Figure 41.

Figure 41. Artesian aquifer encountered at STA-44 site

Similarly, during the CPT soundings the water would come out of the hole at the road surface after the cone and rods were pulled out completely. During the CPT soundings, the pore pressures at certain locations far exceeded the hydrostatic pressure for that depth and it was difficult to determine whether these high pressures were due to the artesian conditions or due to the excess pore pressures that had not yet dissipated. Table 18 lists the locations and depths at which the augers or CPT cone were when the artesian water began to flow out of the hole at the road surface.

4.3.8 Laboratory Testing

An extensive laboratory testing program was conducted on the soil samples collected during the drilling operations at the STA-44 site. The extensive testing was needed for detailed subsurface characterization to assess the feasibility of vertical column support methods investigated, to analyze costs and benefits of feasible methods, and to be used for the design of vertical column supports during the implementation phase.

Table 18. Depths of artesian water encountered at STA-44 site

Location	Depth (m / ft)
B-001-0-13	14.6 / 48.0
B-004-0-13	20.9 / 68.5
B-006-0-13	21.5 / 70.5
B-007-0-13	18.1 / 59.5
B-009-0-13	20.9 / 68.5
C-003-0-13	14.9 / 48.8
C-005-0-13	18.3 / 60.0

All the laboratory tests presented in this report were performed by Bowser-Morner, Inc. of Dayton, Ohio in their AASHTO accredited and ODOT approved laboratories. The laboratory tests conducted are summarized in the following.

Tests performed on samples collected by split-spoon sampler (i.e., disturbed samples):

- Fifty-three (53) unified soil classification (USCS) and complete ODOT soil classification tests were performed in accordance with ASTM D422, D2216, D2487, D4318, D3282, and ODOT specifications.

- Two (2) grain-size analysis tests were performed in accordance with ASTM D422 and D2216.

- Fifty (50) moisture-content determinations were made in accordance with ASTM D2216.

- Ten (10) moisture, ash and organic matter of peat and other organic soils loss on ignition (LOI) tests were performed in accordance with ASTM D2974. These tests were performed to determine the quantity of organic matter.

Tests performed on samples collected by Shelby tubes (i.e., undisturbed samples):

- For each of the thirteen (13) undisturbed samples collected, a unified soil classification and ODOT soil classification test were performed in accordance with ASTM D422, D2216, D2487, D4318, D3282, and ODOT specifications.

- Seven (7) unconfined compressive tests were performed. The unconfined compressive tests were performed in accordance with the ASTM D2166 specifications. The samples were loaded at a constant axial strain rate of 1% per minute. The samples were loaded to failure or 20% strain, whichever happened first.

- Two (2) unconsolidated-undrained (UU) triaxial tests were performed. These tests were performed in accordance with ASTM D2850. The samples were loaded at a constant axial strain rate of 1% per minute. The samples were loaded to failure or 20% strain, whichever happened first.

- One (1) consolidated-undrained (CU) triaxial test was performed. This test was performed in accordance with ASTM D4767. The samples were loaded at a constant axial strain rate of 0.0417% per minute. The sample was loaded to failure or 20% strain, whichever happened first.

- Three (3) consolidation tests were performed. These tests were performed in accordance with the ASTM D2435.

- Of the thirteen undisturbed samples collected, ten (10) samples underwent specific-gravity (G_s) tests which were performed in accordance with ASTM D854. The specific gravity values were used to determine void ratio of the soils, which is a required parameter for the other undisturbed sample tests.

A summary of all the laboratory tests performed on soil samples collected from STA-44 site is provided in Table 19.

Table 19. Summary of laboratory testing for STA-44 site

Type of Testing	**Number of Tests**
USCS / Complete ODOT classification	53
Grain size analysis	2
Moisture content	50
LOI	10
Specific gravity	10
Unconfined compression	7
Triaxial (UU)	2
Triaxial (CU)	1
Consolidation	3

4.4 Summary and Conclusions

Extensive subsurface investigations were conducted for the two sites, SUM-224 and STA-44, which were assessed for implementation of feasible vertical column support systems. Subsurface investigations, including drilling and sampling, cone penetration (CPT) soundings, pressuremeter tests, and laboratory testing, were conducted for the two sites selected by ODOT for this project. Overview of the SUM-224 and STA-44 sites and the subsurface investigations performed at these sites were explained in this chapter. The results obtained from the subsurface investigations are presented in the following chapter during the data analysis and selection of feasible methods for the sites.

CHAPTER 5. DATA ANALYSIS AND IDENTIFICATION OF METHODS

5.1 Introduction

As explained in the previous chapter, extensive subsurface investigations, involving drilling and sampling, CPT soundings, pressuremeter tests, and laboratory testing have been conducted for the SUM-224 and STA-44 sites for the evaluation of these sites to identify feasible vertical column support methods for implementation. Significant amount of data were collected from these investigations to obtain key information about the subgrade characteristics in order to identify the most suitable vertical column support method(s) for implementation at each site. In addition, the data collected would be used by the contractors to design the vertical column support method(s) selected for implementation.

Data collected from field investigations and laboratory testing are presented and analyzed to characterize the subsurface conditions at the two sites being investigated. The decision matrix developed, with the analysis results and site conditions, will then be utilized to identify the feasible vertical column support method(s) for possible implementation at these sites. The data collected, analysis of the data, and the method selection using the decision matrix developed are presented in this chapter. The data and analysis for the SUM-224 site are presented first, followed by the data and analysis for the STA-44 site. In the later part of the chapter, the identification of the feasible methods for the SUM-224 site and then for the STA-44 site are presented.

5.2 SUM-224 Site Data and Analysis

The data collected and the analysis of data for the SUM-224 are presented in the following. The data collected both during the investigations in the field (SPT and CPT tests) and in the laboratory are discussed in the following.

5.2.1 Standard Penetration Testing (SPT)

Standard penetration testing (SPT) with split-spoon sampling was conducted at the site in eight borehole locations as previously mentioned, generating 165 SPT data. The SPT-N numbers obtained in the field were later converted to SPT-N_{60} values. The SPT-N number provides a good indication of how relatively dense/stiff or loose/soft the soil is at the testing depth. The SPT-N_{60} is a corrected SPT-N value that accounts for efficiency of the drilling and sampling apparatus and procedures. Close attention should be paid to the SPT results as a large object, such as a boulder or cobble, blocking the spoon may result in an unusually high and inaccurate SPT-N number. On the other hand, heaving sands can make it difficult to properly sample the soil and often times result in falsely low SPT-N numbers. Figure 42 and Figure 43 presents the SPT-N and SPT-N_{60} data, respectively, varying with depth at each borehole location for the SUM-224 site.

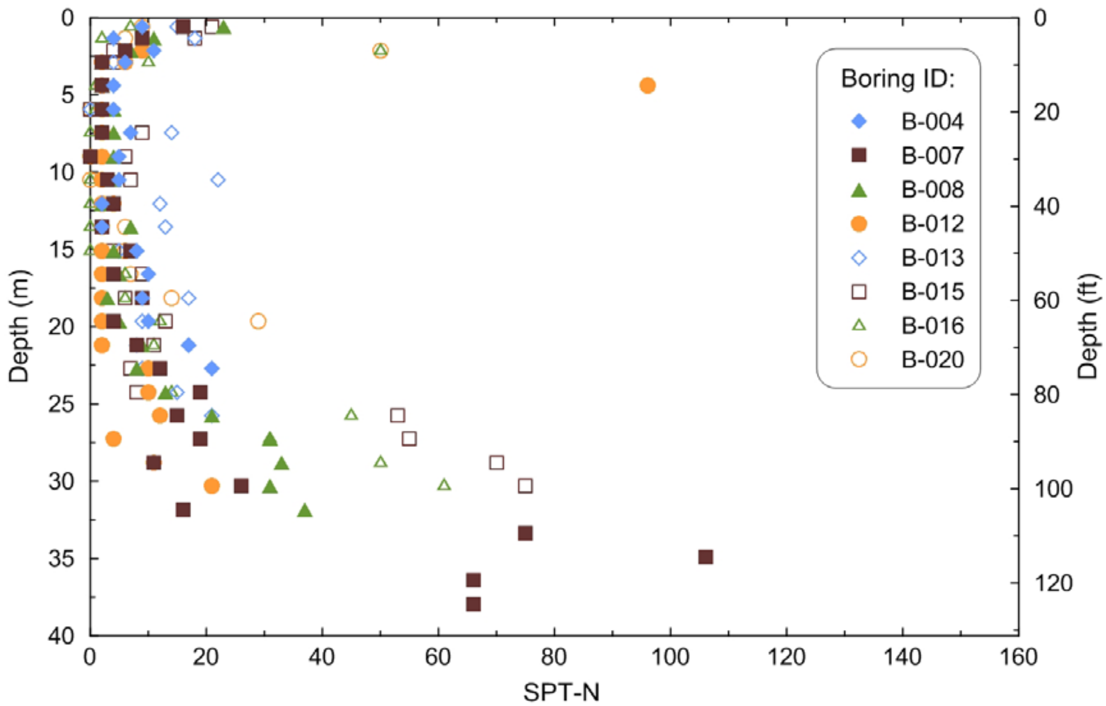


Figure 42. SUM-224 site SPT-N values

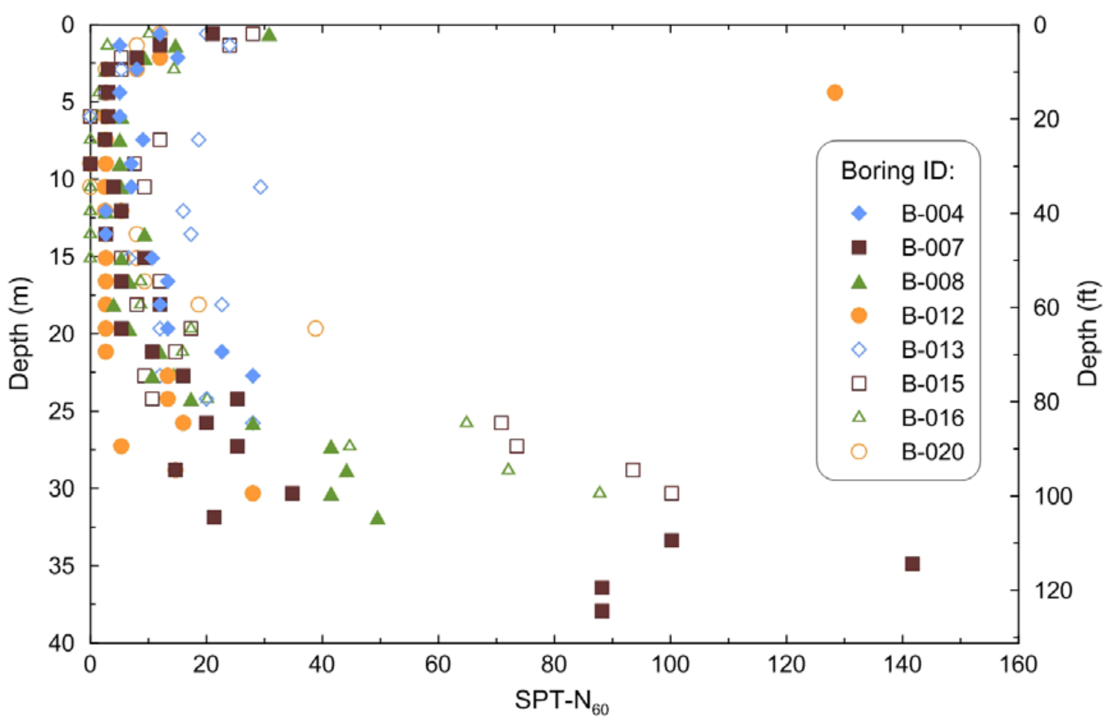


Figure 43. SUM-224 site SPT-N_{60} values

As shown in Figure 42 and Figure 43, consistently low SPT blow counts were recorded at depths from 3 m (10 ft), just past the stiff base and concrete sub-base layers, to about 18 m (60 ft) at the SUM-224 site. The thick low blow count material was especially present at borehole locations of B-012-0-13 and B-16-0-13. The blow counts began to increase at depths starting from 18 m (60 ft), indicating the presence of more reasonable soils. The blow counts indicated very stiff soils and possible bedrock in some locations at depths ranging from 24 to 38 m (80 to 125 ft).

5.2.2 Cone Penetration Testing (CPT)

Cone penetration testing was conducted at 16 different locations as previously discussed. In conjunction with the CPT soundings, dissipation tests were performed at various depths to determine the soil's consolidation behavior. During the tests, cone tip pressure, sleeve friction, and pore pressure were measured by the instrumentation installed in the CPT probe and automatically recorded by the data acquisition system connected to the CPT rig. Using the tip pressure and sleeve friction measured, friction ratio, ratio of the sleeve friction to the tip resistance, was also calculated and recorded. Higher tip pressure, lower sleeve friction, and no excess pore pressure build up are all typically indicative of non-cohesive soils. On the other hand, lower tip pressure, higher sleeve friction, and excess pore pressure build up are all typically indicative of cohesive soils.

The ODOT Geotechnical Engineering Office has prepared a draft report for the CPT soundings they have performed at the SUM-224 site. The subsurface investigation locations were not surveyed since the project did not have the implementation phase. Therefore, the draft CPT report was never finalized. Due to the size of the report and since it was not finalized, the whole draft CPT report is not provided with this report. However, sample pages from the draft report for the CPT soundings performed at the C-018-0-13 location are provided in Figure 44 and Figure 45.

Figure 44 shows the cone resistance, sleeve friction, and pore pressure values collected during the testing and presents their variations with depth. The figure shows that very low tip resistance was encountered until depths of 12 m (40 ft). Some fluctuations in the resistance have been observed between 12 to 19 m (40 to 62 ft). Below 19 m (62 ft), higher tip resistance and sleeve friction have been measured, indicating reasonable soils at these CPT locations. Figure 45 shows the estimated soil behavior types using the measured tip resistance and friction values through the testing depths at the C-018-0-13 location.

Three SPT and CPT locations at the SUM-224 site were paired during the subsurface investigations, i.e., located next to each other, to analyze/compare findings and develop correlations if needed. The SPT and CPT pairs were: B-008-0-13 and C-009-0-13, B-015-0-13 & C-014-0-13, and B-016-0-13 & C-017-0-13. One of the parameters estimated from the CPT measurements using correlations is SPT-N_{60} values, which were provided in the draft report prepared by ODOT. The SPT-N_{60} values calculated from the field SPT-N numbers and the SPT-N_{60} values estimated from CPT tests measurements are presented in Figure 46 for two of the SPT & CPT pairs. The figure shows that the results are overall in good agreement. Very low numbers from CPT soundings at very shallow depths are because holes were pre-drilled so that the CPTs would be able to pass any hard layers present below the pavement. As shown in Figure 46, the CPT tests capture the SPT locations where the heaving sands possibly occurred during drilling, causing inaccurately lower SPT blow counts, as previously mentioned.

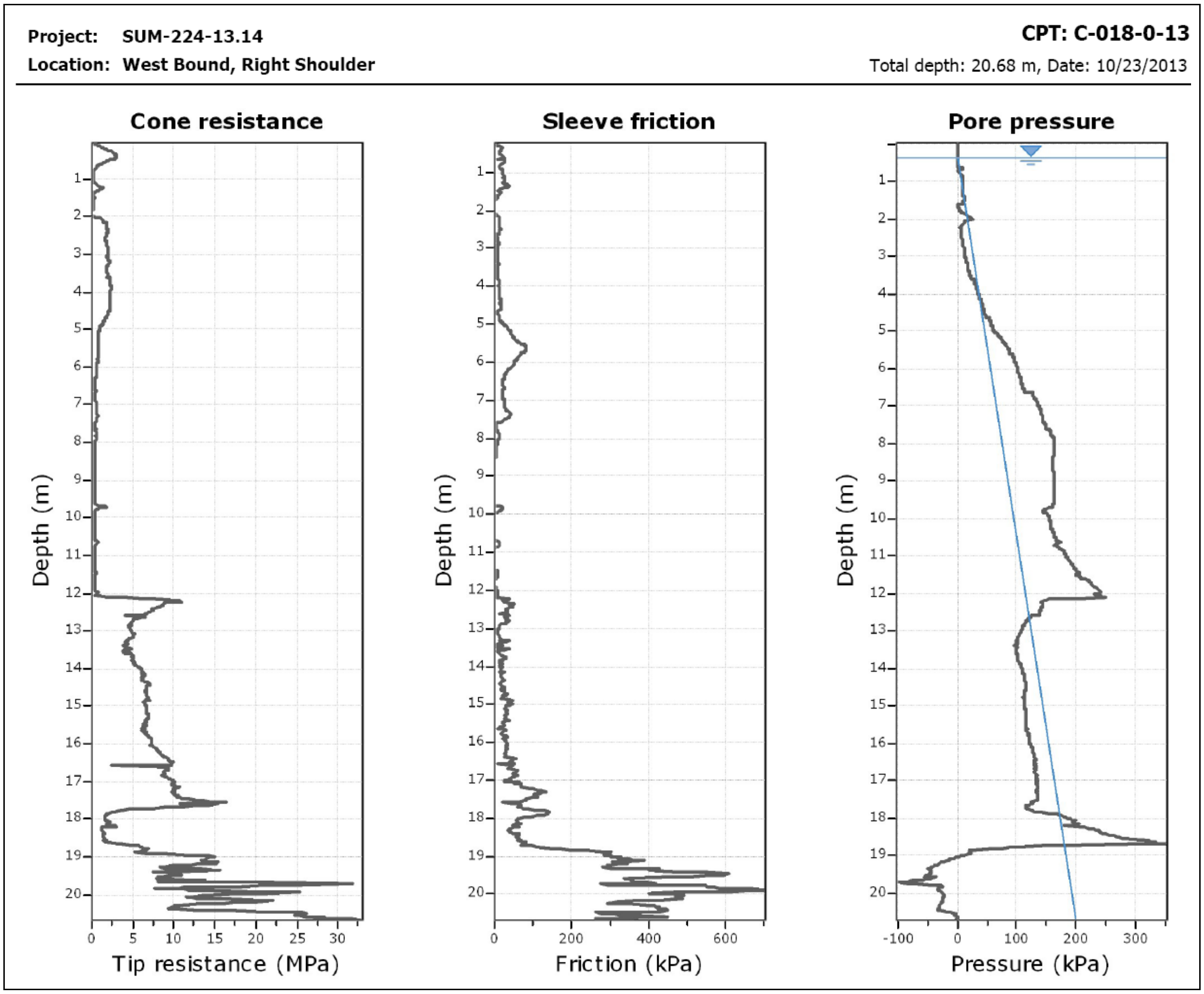


Figure 44. SUM-224 site CPT sounding data collected at C-018-0-13

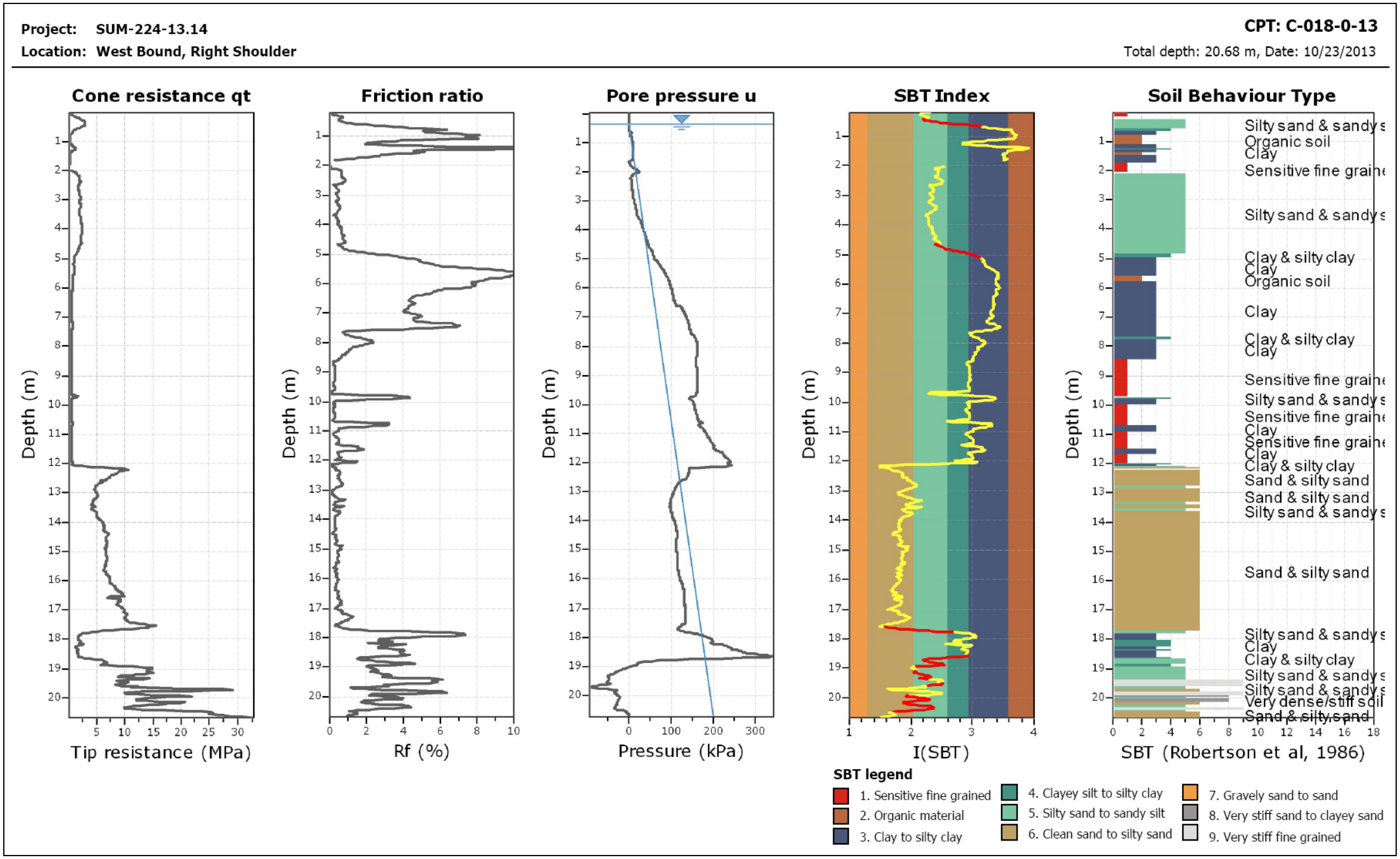
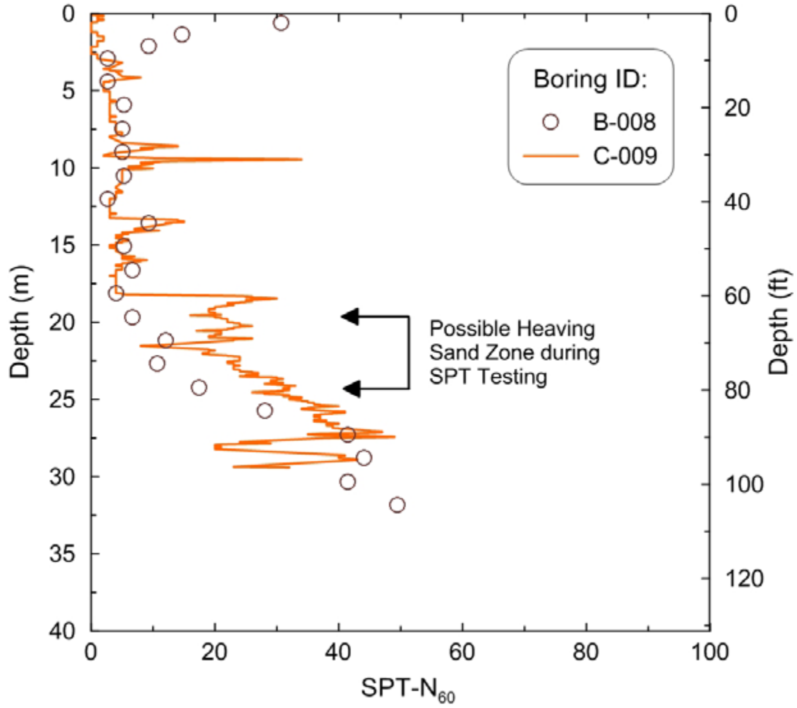


Figure 45. SUM-224 site CPT sounding data analysis/interpretation at C-018-0-13



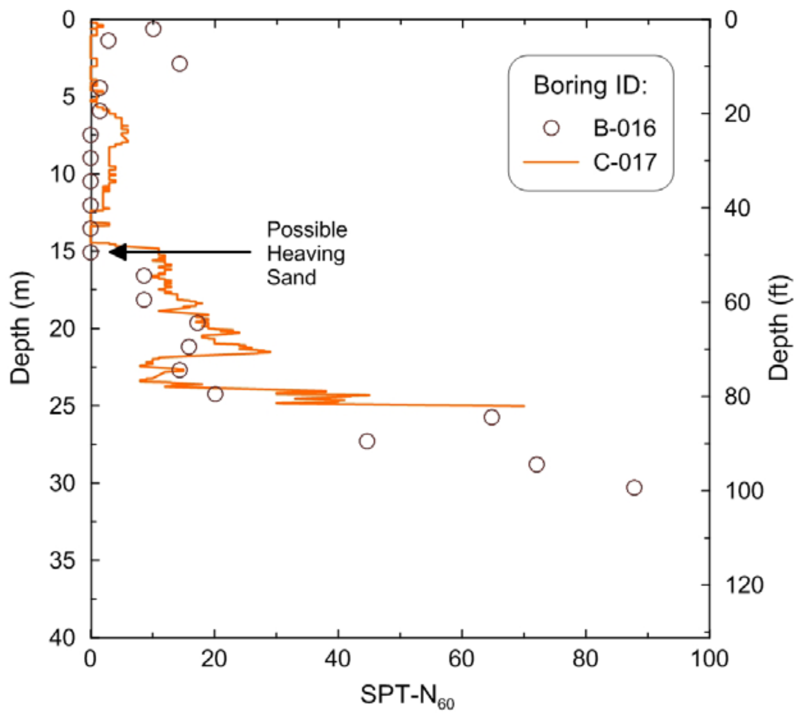


Figure 46. Comparative analysis of SPT-N_{60} values obtained from SPT and CPT tests at SUM-224 site

5.2.3 Instrumentation Reading

One monitoring well and one inclinometer casing were installed at the site. The data collected for these installations included only the baseline readings taken at the end of the subsurface investigation field work. At the time of the baseline reading, the groundwater water level was 0.66 m (26 in) below the pavement surface in the monitoring well at the SUM-224 site.

At the direction of ODOT no additional instrumentation readings were taken at the site. Because only a baseline reading was taken from the inclinometer installation, there is no data to present or to assess if there are any ground movements at the site.

5.2.4 Laboratory Testing

The data interpreted from laboratory testing comes from a variety of tests including: Moisture content tests, Atterberg limit tests, organic content loss on ignition (LOI) tests, unconfined compressive strength tests, triaxial tests, and consolidation tests. The test results show that the soil conditions at the site are quite complex and soil properties changes significantly, both across the site and with depth. The variation of several soil properties with depth at each borehole location at the SUM-224 site are presented in Figure 47 through Figure 52, as listed in the following:

- Figure 47 shows the moisture content values,
- Figure 48 shows the liquid limit values,
- Figure 49 shows the plastic limit values,
- Figure 50 shows the plasticity index values,
- Figure 51 shows the loss on ignition values, and
- Figure 52 shows the unconfined compressive strength.

The problem soils and their extent at the SUM-224 site were easily identifiable by their very high moisture contents (Figure 47) and organic content (from loss on ignition tests) (Figure 51) indicating soft cohesive and organic soils. These were the primary problematic soils causing subgrade settlements resulting in pavement distress at the site. Moisture contents as high as 403% and loss on ignition (LOI) values of as high as 89% were measured at this site as shown in Figure 47 and Figure 51.

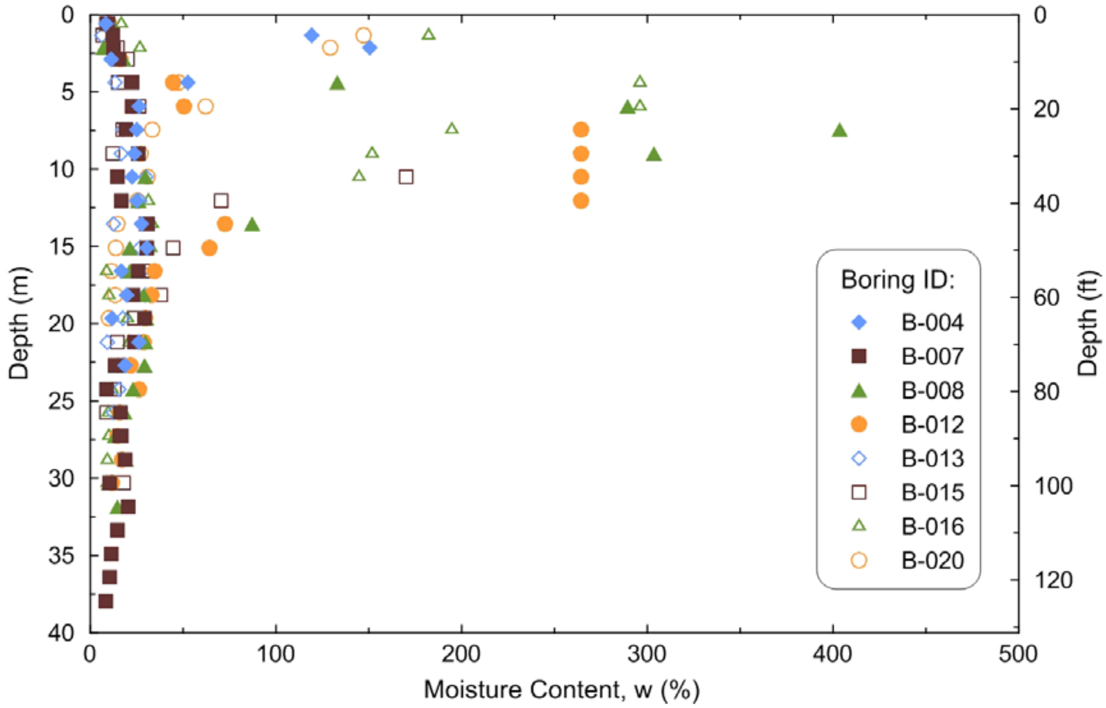


Figure 47. SUM-224 site moisture content values

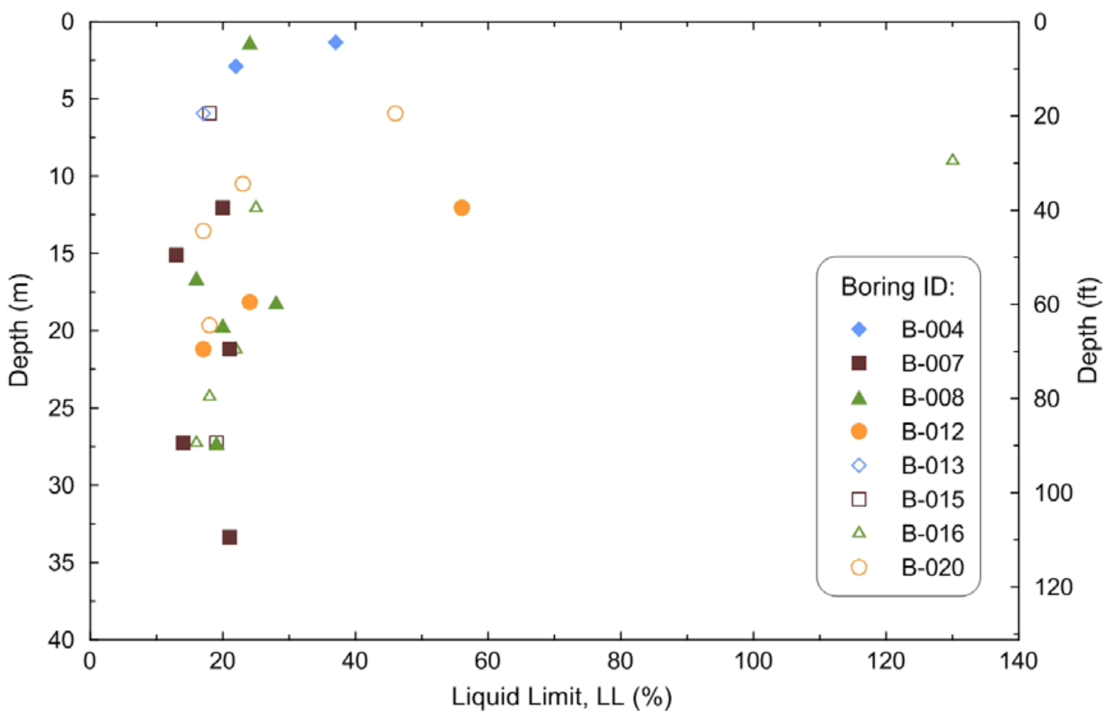


Figure 48. SUM-224 site liquid limit values

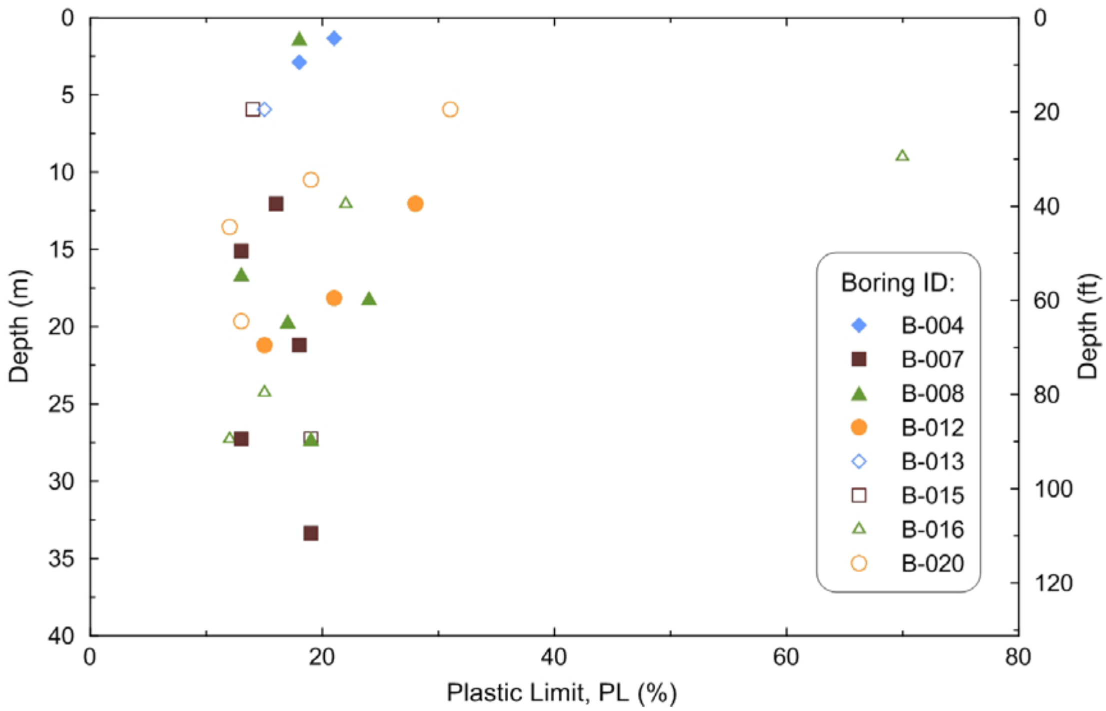


Figure 49. SUM-224 site plastic limit values

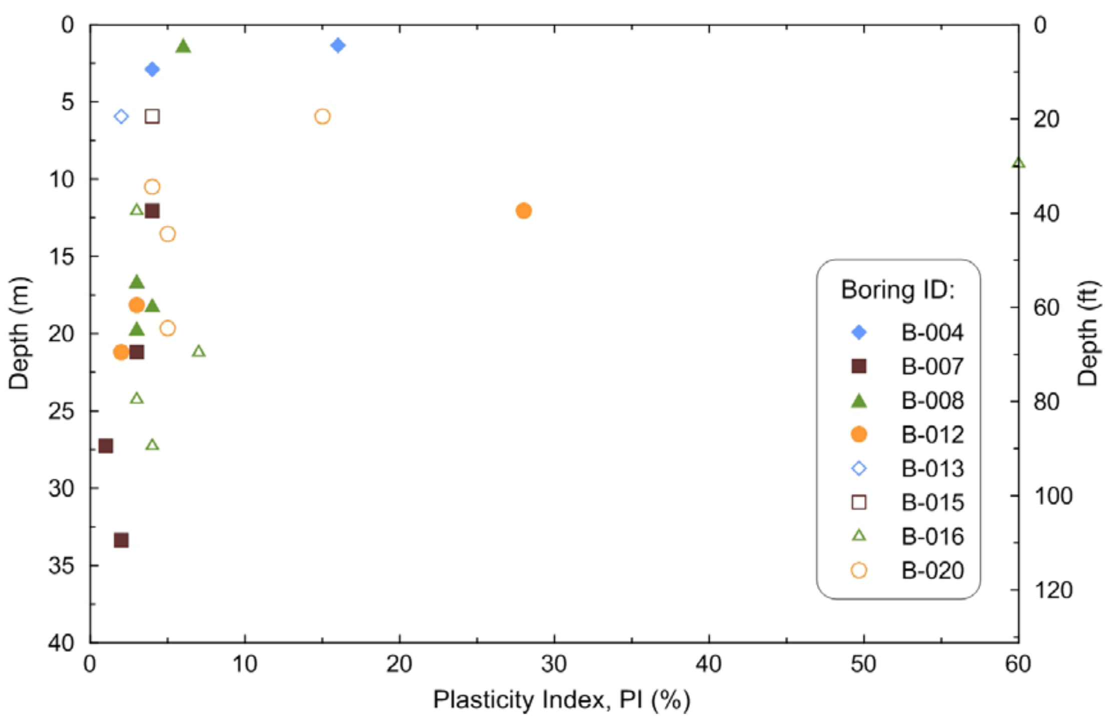


Figure 50. SUM-224 site plasticity index values

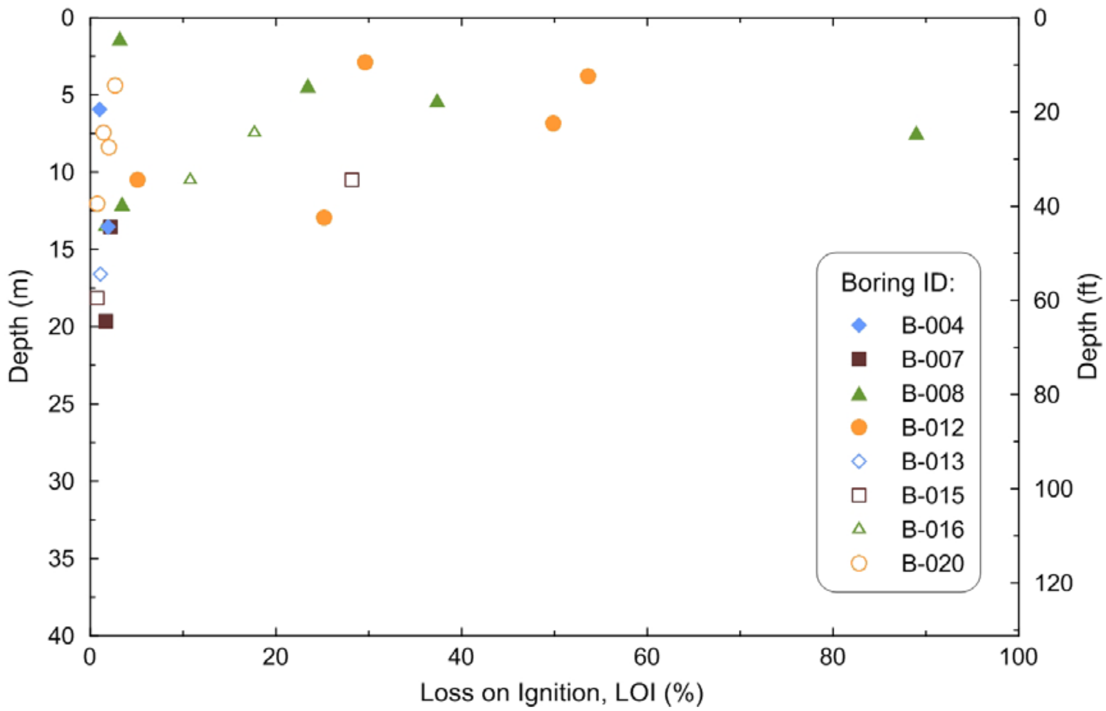


Figure 51. SUM-224 site loss on ignition values

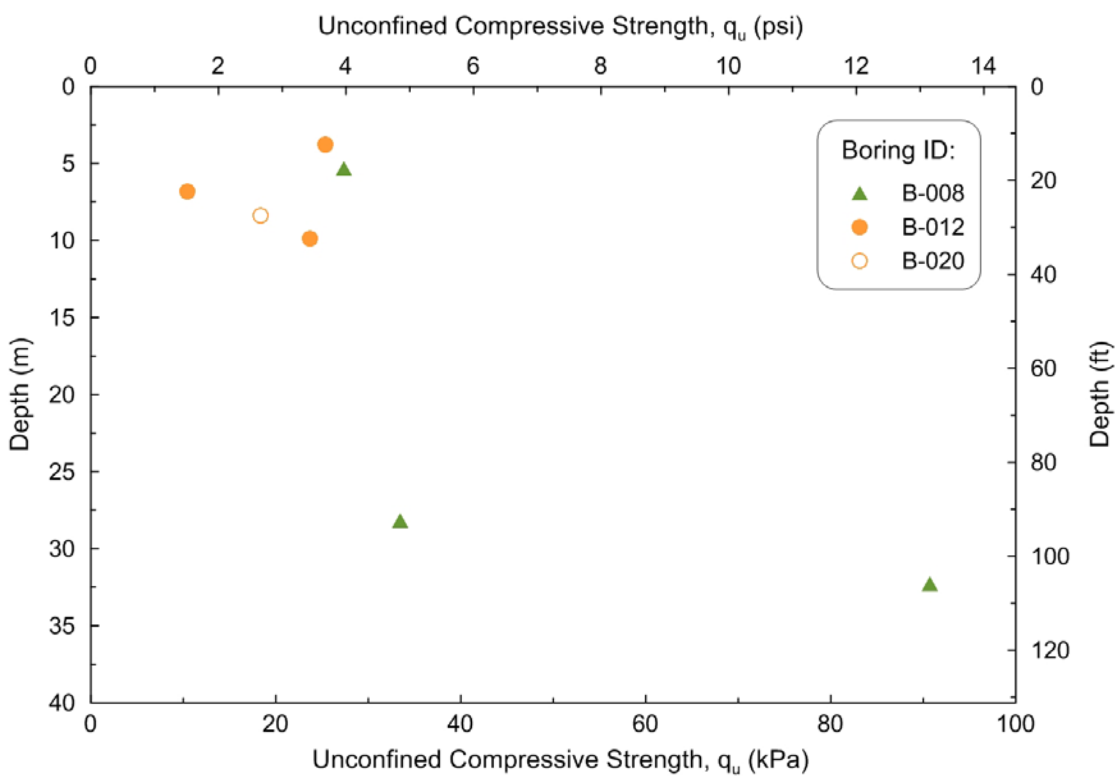


Figure 52. SUM-224 site unconfined compressive strength values

5.3 STA-44 Site Data and Analysis

The data collected and the analysis of data for the STA-44 site are presented in the following. The data collected both during the investigations in the field (SPT and CPT tests) and in the laboratory are discussed in the following.

5.3.1 Standard Penetration Testing (SPT)

Standard penetration testing (SPT) with split-spoon sampling was conducted at the site in seven borehole locations as previously mentioned, generating 109 SPT data points. The SPT-N numbers obtained in the field were later converted to SPT-N_{60} values. The SPT-N number provides a good indication of relatively how dense/stiff or loose/soft the soil is at the testing depth. The SPT-N_{60} is a corrected SPT-N value that accounts for efficiency of the drilling and sampling apparatus and procedures. Figure 53 and Figure 54 presents the SPT-N and SPT-N_{60} data, respectively, varying with depth at each borehole location for the STA-44 site.

Figure 53 and Figure 54 show very low SPT-N blow count material was encountered starting just below the pavement to about 10 m (33 ft) in some of the borings. At some locations towards the northern part of the site, specifically boreholes B-013-0-13 and B-016-0-13, acceptable blow counts were encountered around 4.6 m (15 ft). However, for the rest of the site acceptable blow counts were encountered at depths ranging from 10 to 20 m (33 to 65 ft). Below 20 m (65 ft) depth, the blow counts indicated very dense strata, as well as possible bedrock in some locations.

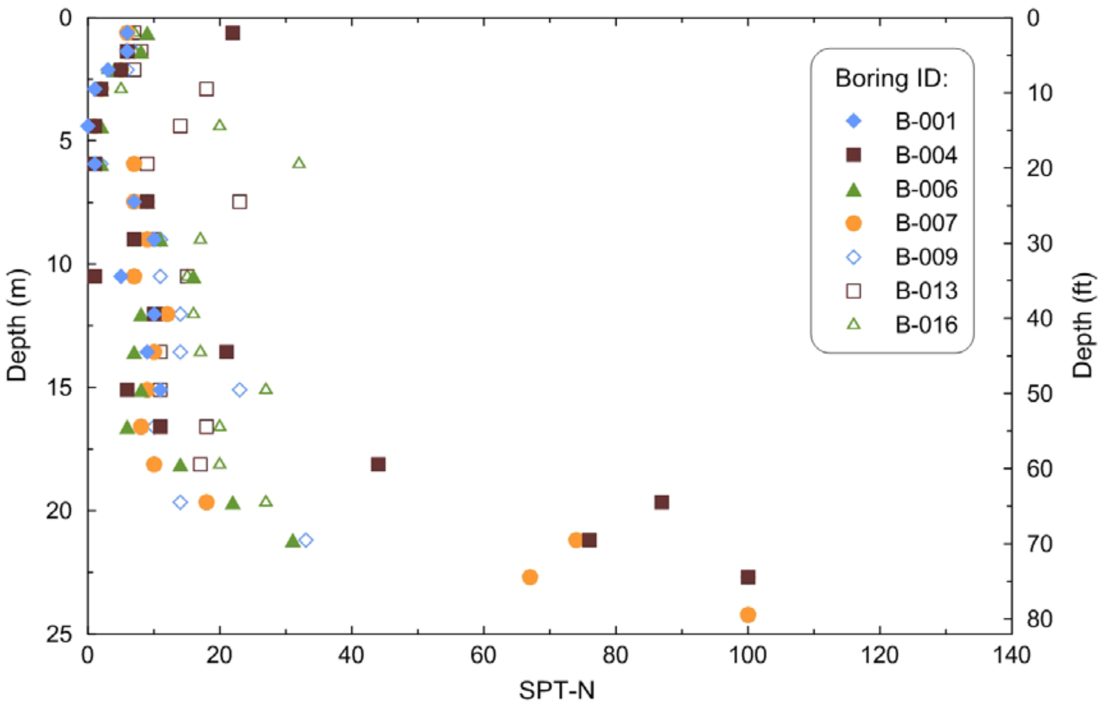


Figure 53. STA-44 site SPT-N values

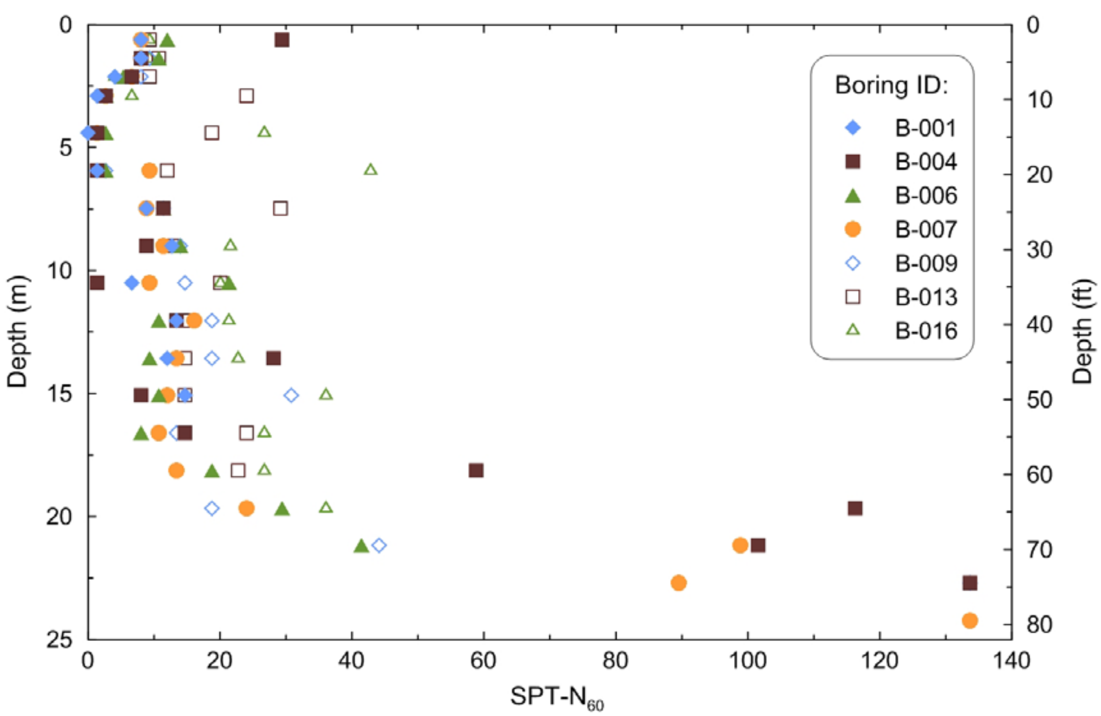


Figure 54. STA-44 site SPT-N_{60} values

5.3.2 Cone Penetration Testing (CPT)

Cone penetration testing was conducted at nine different locations. In conjunction with the CPT soundings, dissipation tests were performed at various depths to monitor the soil's consolidation behavior. During the tests, cone tip pressure, sleeve friction, and pore pressure were measured by the instrumentation installed in the CPT probe and automatically recorded by the data acquisition system that it is connected to inside the CPT rig. Using the tip pressure and sleeve friction measured, friction ratio, ratio of sleeve friction to tip resistance, was also calculated and recorded.

The ODOT Geotechnical Engineering Office has prepared a draft report for the CPT soundings they have performed at the STA-44 site. The subsurface investigation locations were not surveyed since the project did not have the implementation phase. Therefore, the draft CPT report was never finalized. Due to the size of the report and since it was not finalized, the whole draft CPT report is not provided with this report. However, sample pages from the draft report for the CPT soundings performed at the C-002-0-13 location are provided in Figure 55 and Figure 56.

Figure 55 shows the cone resistance, sleeve friction, and pore pressure values collected during the test and presents their variations with depth. The figure shows that very low tip resistance was encountered until depths of 10 m (33 ft). Some fluctuations with a relatively higher resistance have been observed between 10 to 19 m (33 to 62 ft). At 19 m (62 ft) a material with very high tip resistance and sleeve friction have been encountered. Figure 56 shows the estimated soil behavior types using the measured tip resistance and friction values through the testing depths at the C-002-0-13 location.

Three SPT and CPT locations at STA-44 site were paired during the subsurface investigations, i.e., located next to each other, to analyze/compare findings and develop correlations if needed. The SPT and CPT pairs were: B-006-0-13 & C-005-0-13, B-009-0-13 & C-010-0-13, and B-013-0-13 & C-012-0-13. One of the parameters estimated from the CPT measurements using correlations is SPT-N_{60} values, which were provided in the draft report prepared by ODOT. The SPT-N_{60} values calculated from the field SPT-N numbers and the SPT-N_{60} values estimated from CPT tests measurements are presented in Figure 57 for two of the SPT & CPT pairs. The figure shows that the results are overall in good agreements, especially for the B-009-0-13 and C-010-0-13 pair. Relatively high values from SPT tests in B-006-0-13 between 7 and 11 m (23 to 36 ft) could be due to the presence of trace gravel encountered at these depths. Gravel presence can cause higher resistance to split-spoon's penetration resulting in falsely higher blow counts as mentioned previously. Very low numbers from CPT soundings at very shallow depths are because the holes were pre-drilled so that the CPTs would be able to pass any hard layers present below the pavement.

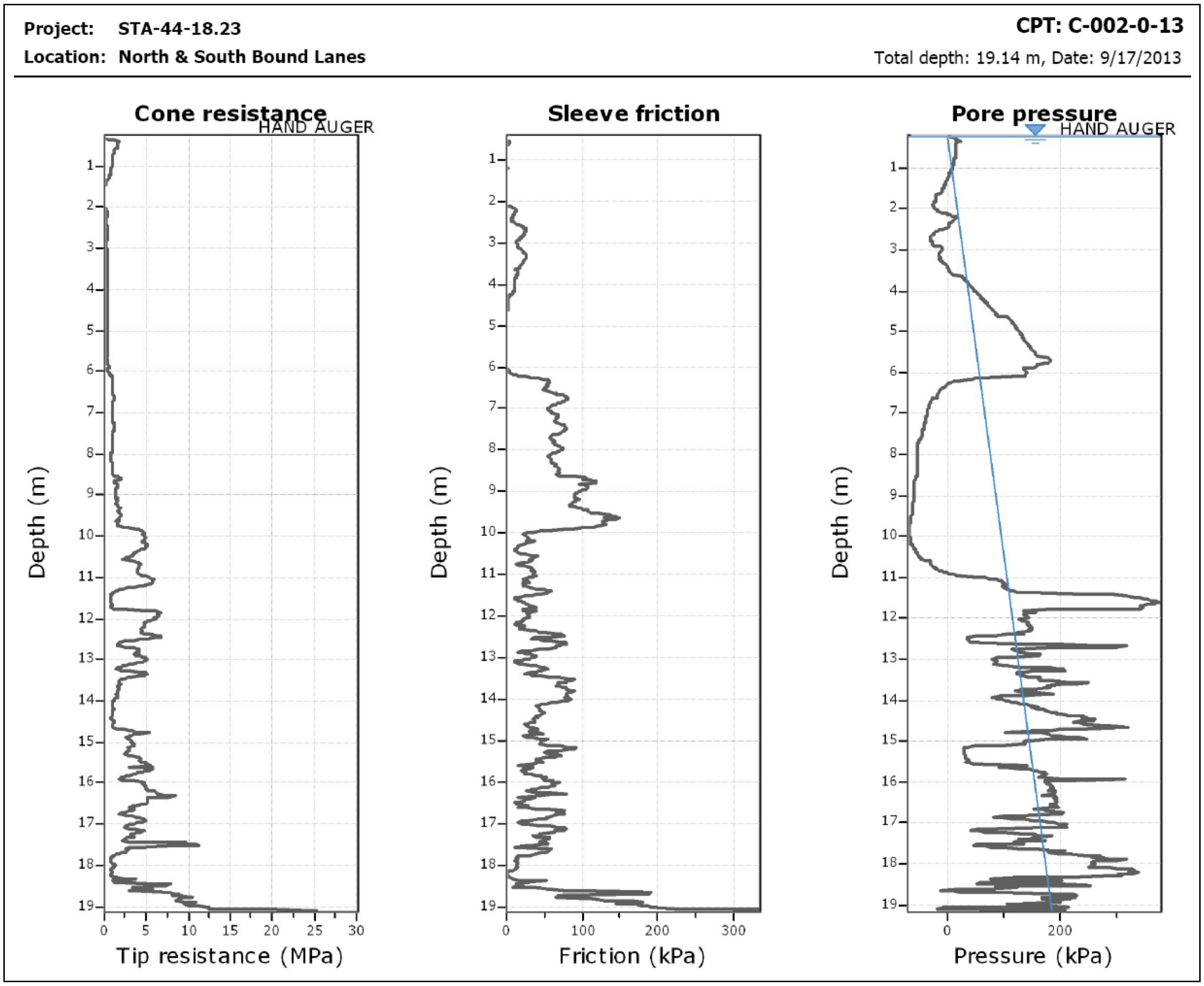


Figure 55. STA-44 site CPT sounding data collected at C-002-0-13

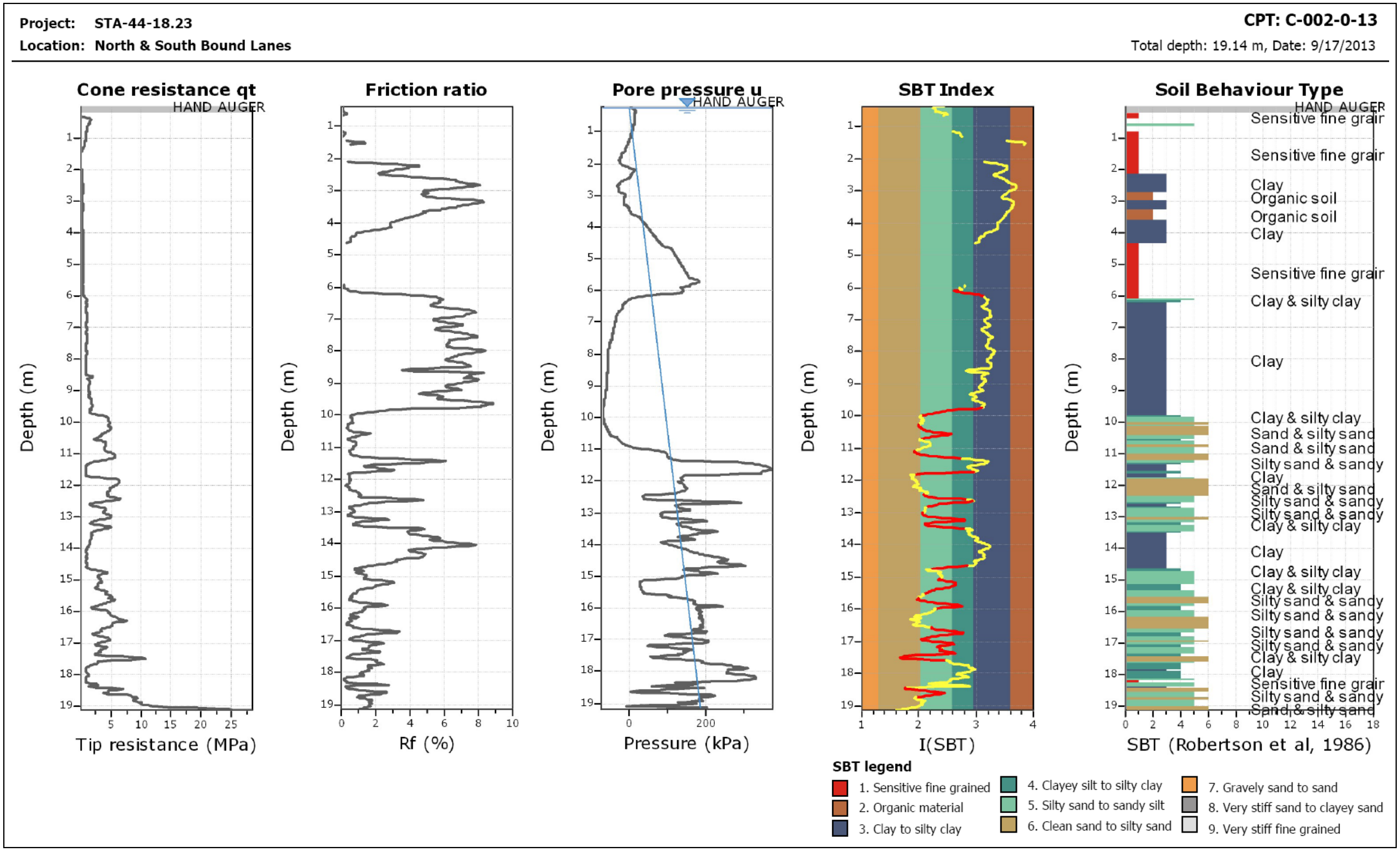
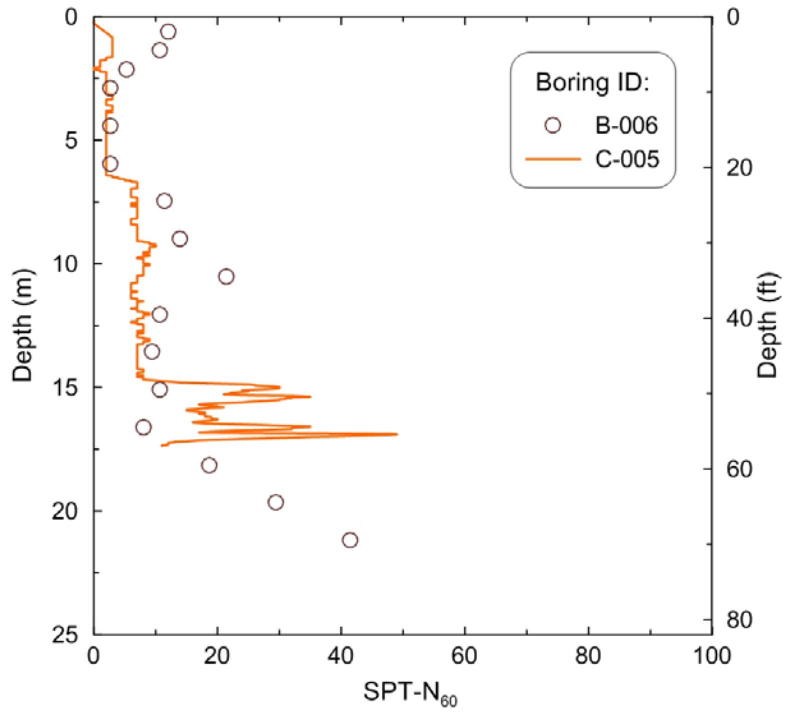


Figure 56. STA-44 site CPT sounding data analysis/interpretation at C-002-0-13



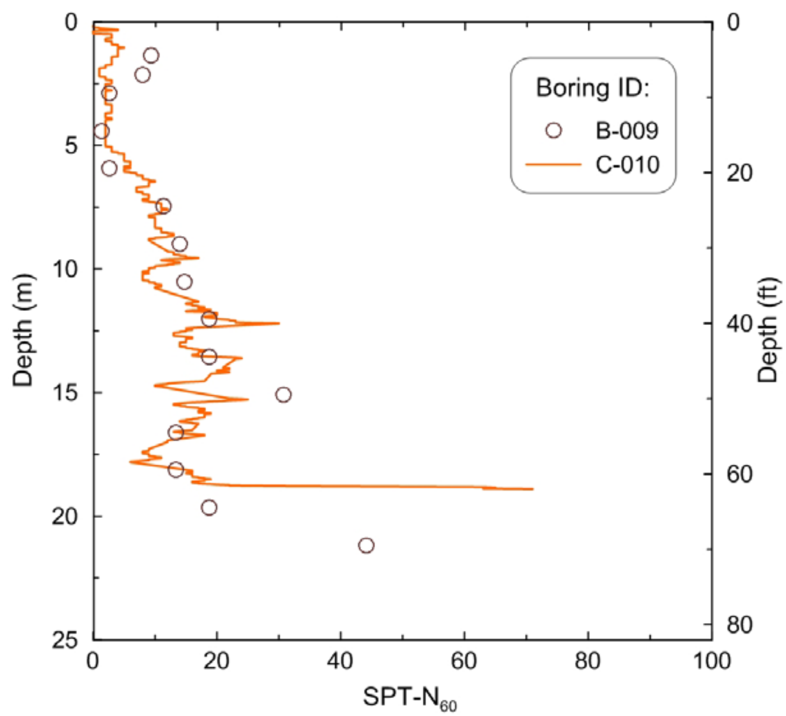


Figure 57. Comparative analysis of SPT-N_{60} values obtained from SPT and CPT tests at STA-44 site

5.3.3 Instrumentation Reading

Two inclinometer casings were installed at the site. The two monitoring wells planned were unable to be installed at the STA-44 site due to the presence of an artesian aquifer near the depths of installation. The data collected for these installations included only the baseline readings taken at the end of the subsurface investigation field work. At the direction of ODOT no additional instrumentation readings were taken at the site. Because only a baseline reading was taken from the inclinometer installation, there is no data to present or to assess if there are any ground movements at the site.

5.3.4 Laboratory Testing

The data interpreted from laboratory testing comes from a variety of tests including: Moisture content tests, Atterberg limit tests, organic content loss on ignition (LOI) tests, unconfined compressive strength tests, triaxial tests, and consolidation tests. The test results show that the soil conditions at the site are quite complex and soil properties change significantly, both across the site and with depth. The variation of several soil properties with depth at each borehole location at the STA-44 site are presented in Figure 58 through Figure 63, as listed in the following:

- Figure 58 shows the moisture content values,
- Figure 59 shows the liquid limit values,
- Figure 60 shows the plastic limit values,
- Figure 61 shows the plasticity index values,
- Figure 62 shows the loss on ignition values, and
- Figure 63 shows the unconfined compressive strength.

The problem soils and their extent at the STA-44 site were easily identifiable by their high moisture contents (Figure 58) and organic content (from loss on ignition tests) (Figure 62) indicating soft cohesive and organic soils. These were the primary problematic soils causing subgrade settlements resulting in pavement distress at the site. Moisture contents as high as 265% and loss on ignition (LOI) values of as high as 53% were measured at this site as shown in Figure 58 and Figure 62.

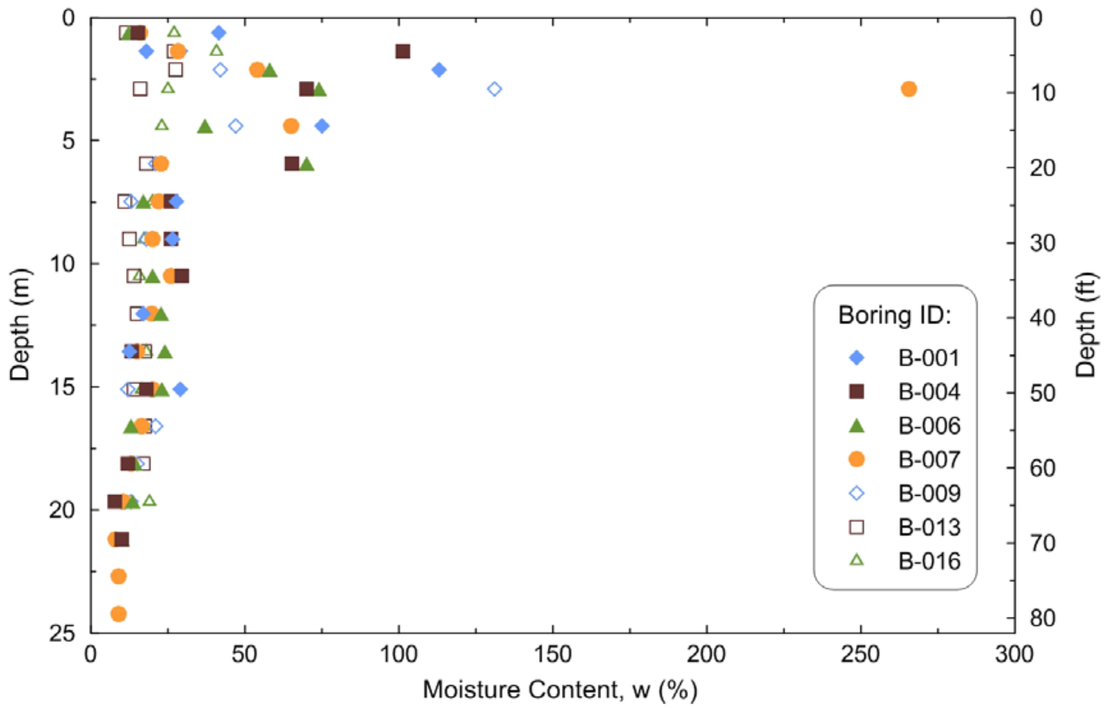


Figure 58. STA-44 site moisture content values

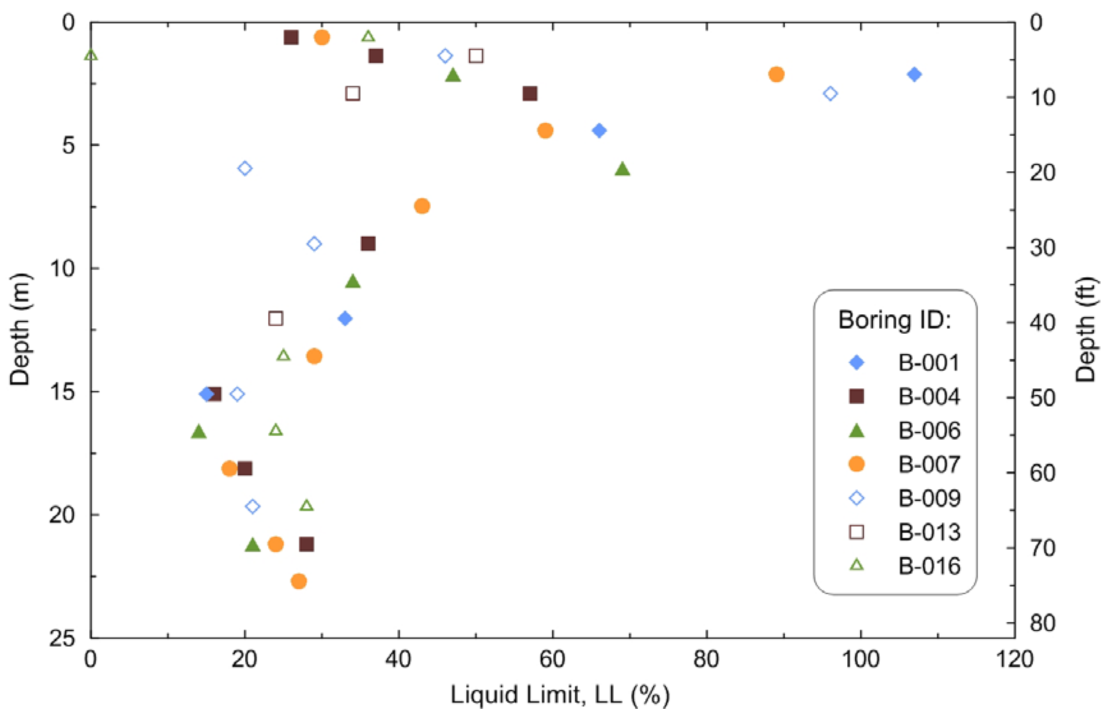


Figure 59. STA-44 site liquid limit values

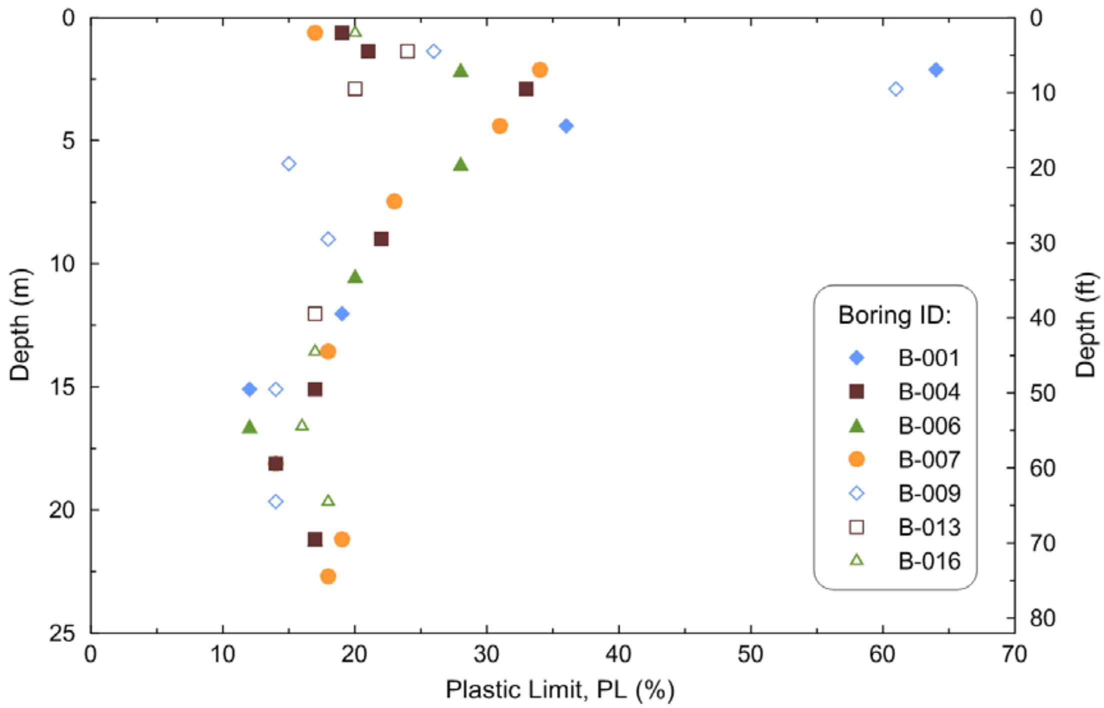


Figure 60. STA-44 site plastic limit values

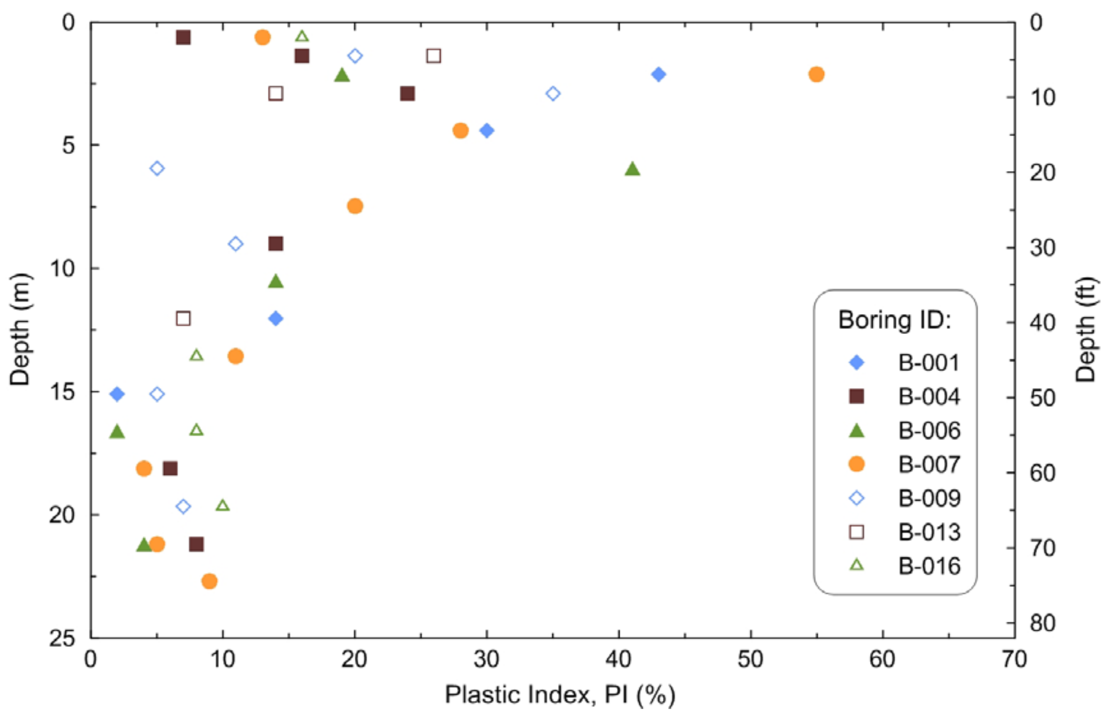


Figure 61. STA-44 site plasticity index values

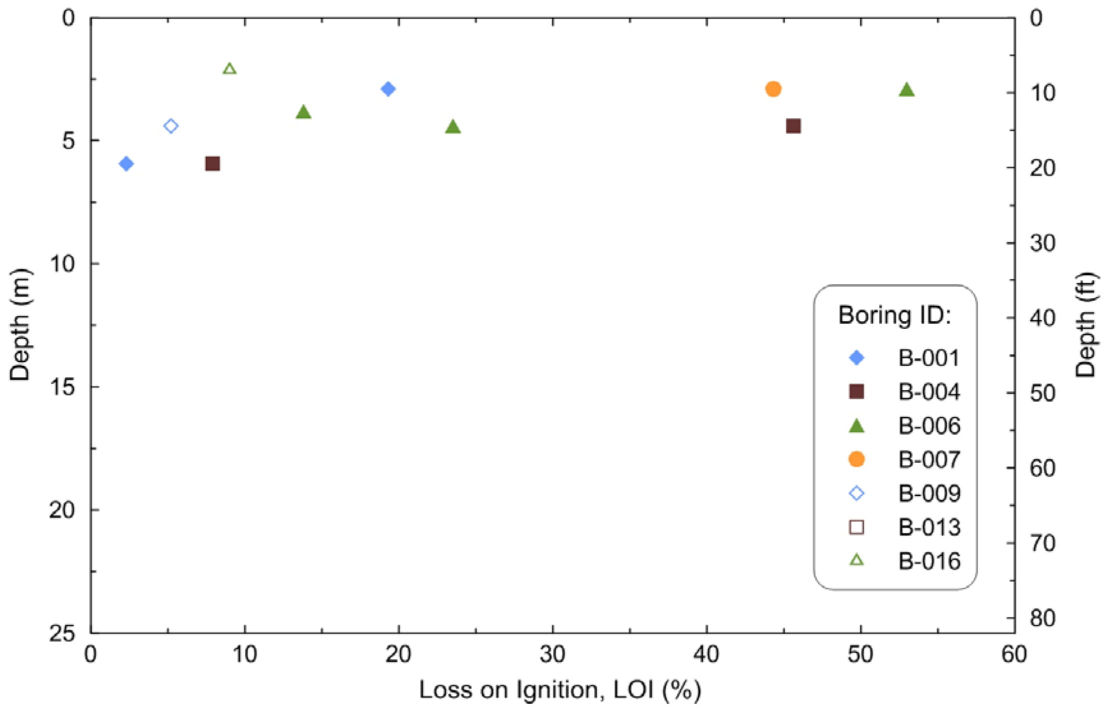


Figure 62. STA-44 site loss on ignition values

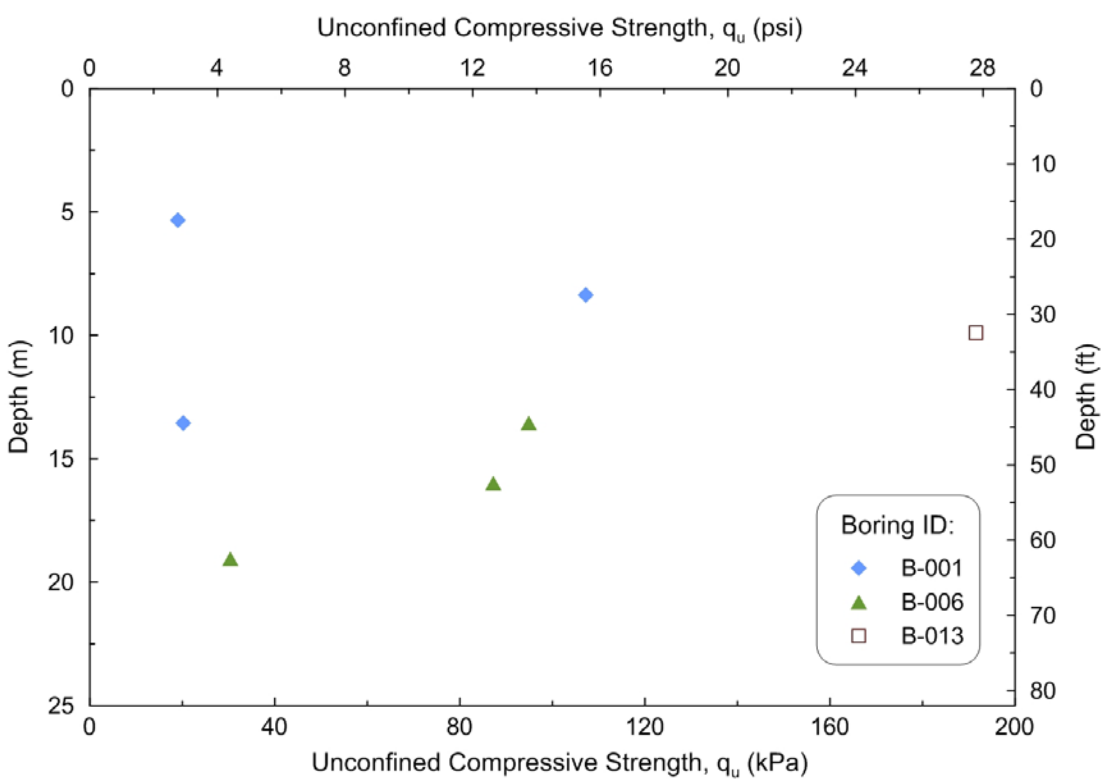


Figure 63. STA-44 site unconfined compressive strength values

5.4 Identification of Feasible Methods for the Sites Investigated

After collecting the data needed and performing the analysis, the decision matrix developed during this project was used to identify feasible vertical column support methods. As previously mentioned, vertical column support ground improvement systems were considered in the development of the decision matrix among the two main groups of methods: ground improvement and deep foundations. The vertical column support deep foundation systems (at least some of them) were considered to be applicable to most problematic soil conditions, however they would be prohibitively expensive to remediate the subgrade settlement problems of existing roadways.

The literature review, critical components of vertical column support methods, and decision matrix development process proved that the problematic soil type and their extent below the ground surface are the two most important parameters in identifying the feasible methods. Because, some methods are not effective in all soil types and each method has its own limitations for the application depths.

5.4.1 Methods for SUM-224 Site

Figure 64 through Figure 66 show the soil profiles and properties at the borehole locations at the SUM-224 site. The information on the figures includes soil types, SPT-N_{60} values, moisture content measurements, and loss on ignition data. While Figure 64 shows the soil profile across the site, Figure 65 and Figure 66 show the soil profiles on the north and south sides of the site, respectively. The soil profiles show that the presence of problem soils is wider and thicker on the northern side of the roadway.

Applying the decision matrix developed during this project to the soil conditions at the site, resulted in three technically feasible vertical column support ground improvement methods for the remediation of ongoing settlement problems at the SUM-224 site. The methods identified are (not in any particular order):

- Jet grouting,
- Deep soil mixing, and
- Controlled modulus columns.

5.4.2 Methods for STA-44 Site

Figure 67 shows the soil profile and properties at the borehole locations at the STA-44 site. Similar to the previous site, the information on the figure includes soil types, SPT-N_{60} values, moisture content measurements, and loss on ignition data. The soil profile shows that problem soils are more present on the south side of the site.

Applying the decision matrix developed to the soil conditions at the site, resulted in four technically feasible vertical column support ground improvement methods for the remediation of ongoing settlement problems at the STA-44 site. The methods identified are (not in any particular order):

- Jet grouting,
- Deep soil mixing,
- Controlled modulus columns, and
- Rammed aggregate piers.

5.5 Summary and Conclusions

Data collected from the field investigations and laboratory testing are presented and analyzed to characterize the subsurface conditions at the SUM-224 and STA-44 sites. The decision matrix developed was utilized to identify technically feasible vertical column support ground improvement methods for the remediation of ongoing settlement problems at these sites. The decision matrix analysis showed that there are three potential methods for the SUM-224 site and four potential methods for the STA-44 site.

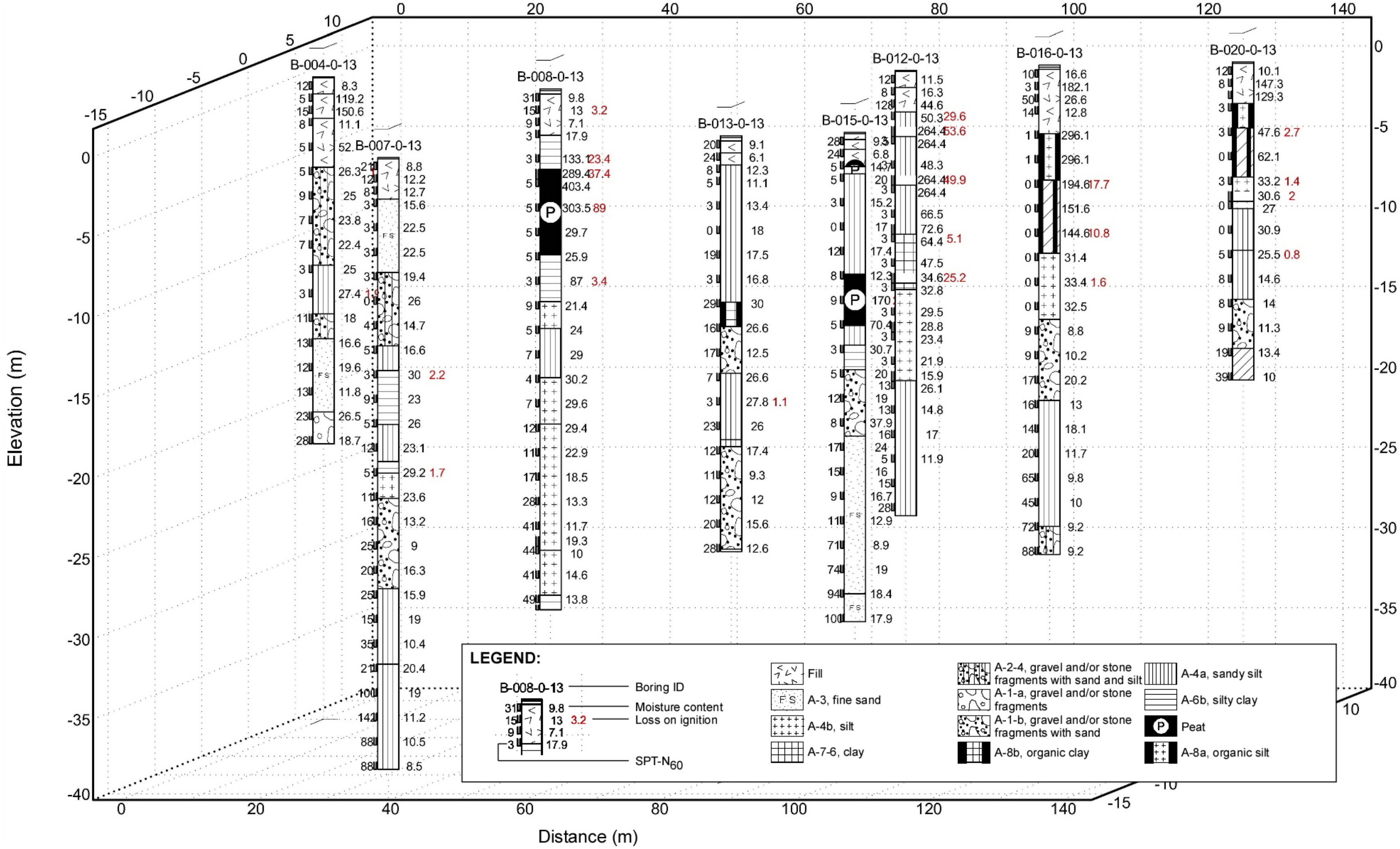


Figure 64. SUM-224 site 3D subsurface soil profile

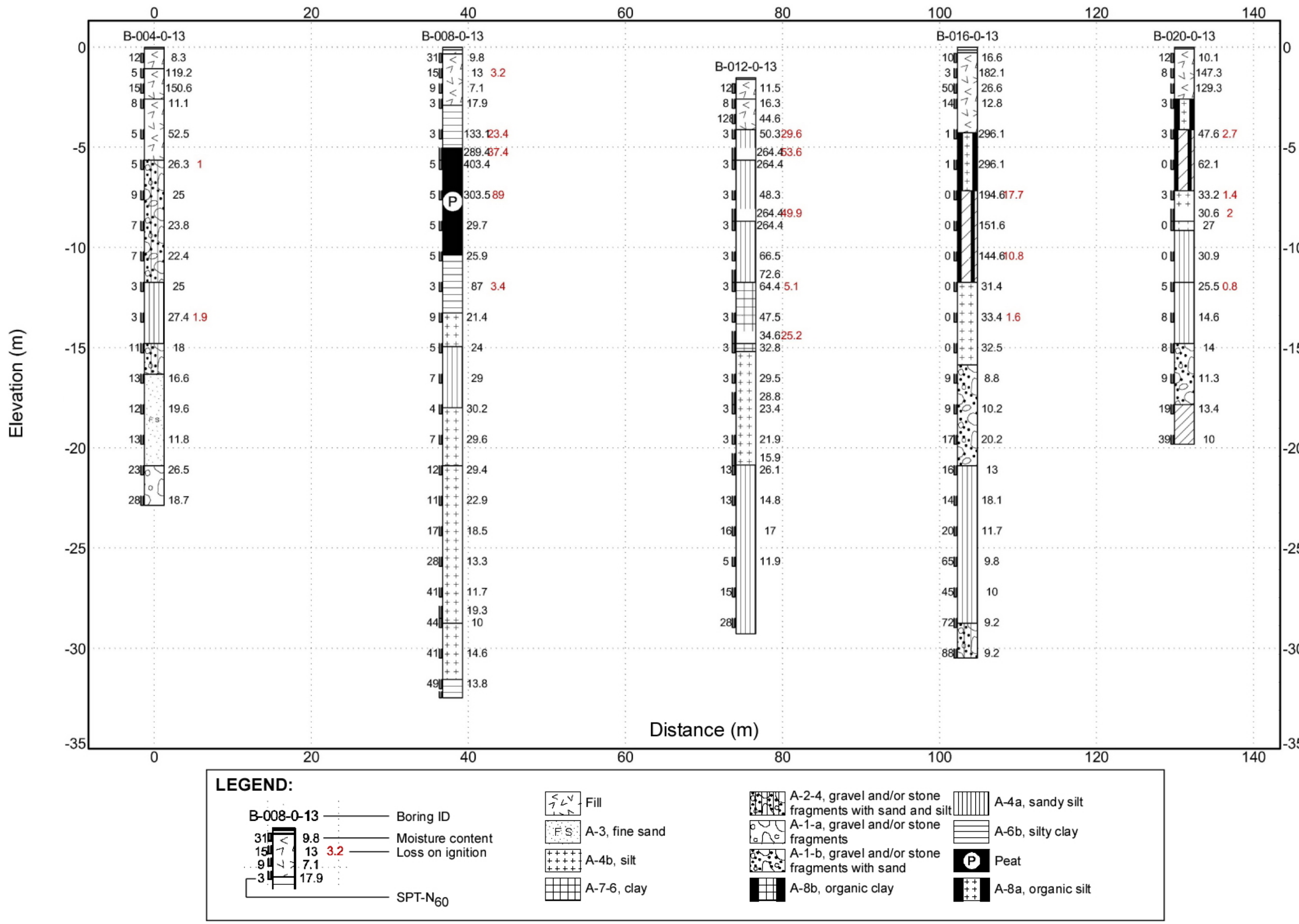


Figure 65. SUM-224 site subsurface soil profile on the north side

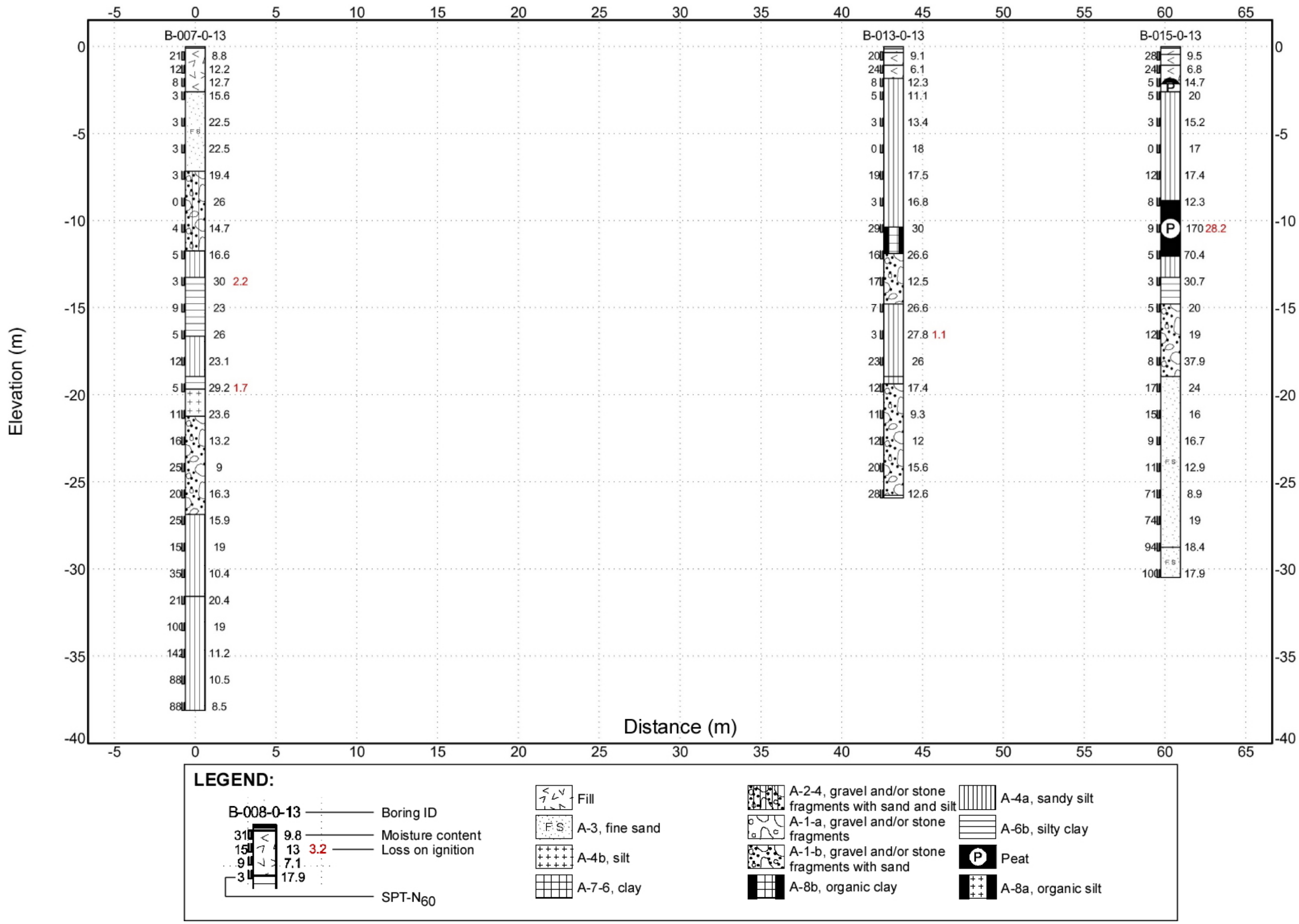


Figure 66. SUM-224 site subsurface soil profile on the south side

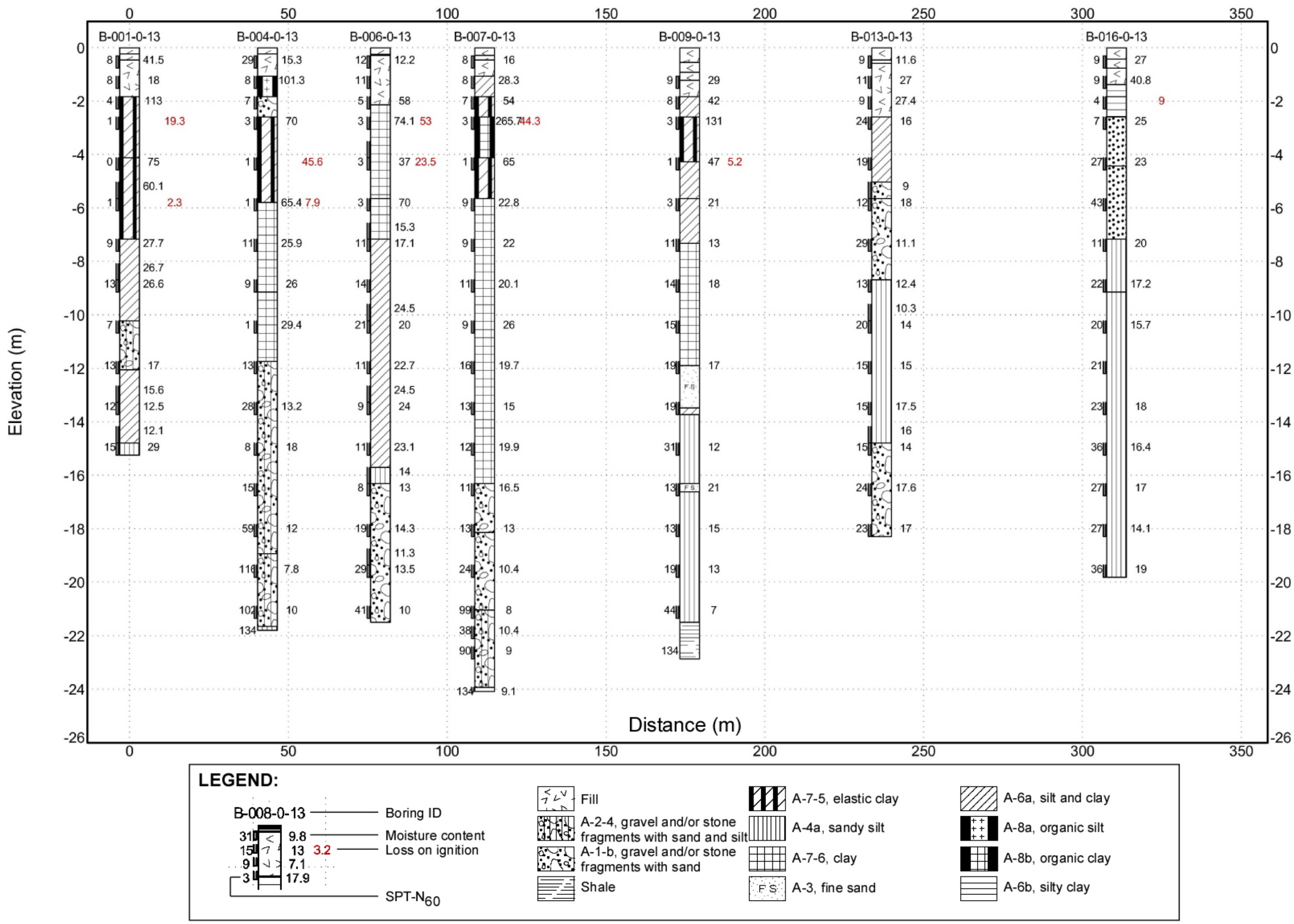


Figure 67. STA-44 site subsurface soil profile

CHAPTER 6. LIFETIME COST-BENEFIT ANALYSIS

6.1 Introduction

A lifetime cost-benefit analysis was conducted to evaluate the cost efficiency of using various vertical column support systems as means of remediation for subgrade settlement of existing roadways. The cost-benefit analysis of utilizing vertical column support methods then was compared to ODOT's current practice in dealing with these existing roadway settlement problems which is pavement patching/resurfacing.

Two alternatives and costs associated with each alternative are considered for the purpose of cost-benefit analysis:

- Alternative 1: This is a temporary solution, and also ODOT's current practice, in remediating the pavement distress caused by subgrade soil settlements, which is patching and resurfacing.
- Alternative 2: This is a permanent solution, which involves utilization of vertical column support ground improvement methods, in remediating the issues caused by the subgrade settlements.

An overview of the costs associated with the effects of pavement condition on vehicle operating costs is also provided in this chapter. The permanent remediation of ongoing settlement problems of an existing roadway using vertical column support methods would improve the pavement conditions and reduce the costs associated with using the roadway in poor conditions. The subgrade settlements causing pavement distress and failure produce poor conditions for roadway users even with patching and resurfacing, which are a temporary remediation. In many cases patching and resurfacing would cause additional loads on problematic soils due to the additional asphalt which needs to be placed to compensate for the settlements and to bring the roadway to its original elevation.

Following the review of costs associated with both alternatives, the lifetime cost-benefit analysis for the SUM-224 and STA-44 sites investigated for possible implementation of vertical column support methods is presented for the two alternatives discussed. The deep foundation systems are typically much more costly alternatives compared to the ground improvement methods. For this reason, the deep foundation alternatives of vertical column support systems are excluded from the cost-benefit analysis.

In addition, a design chart was developed during this project for the lifetime cost-benefit analysis to be used by ODOT in future projects and the procedure for using the design chart is discussed later in the chapter.

6.2 Overview of Cost Components

There are three main cost components associated with roadway projects and for the two alternatives considered in this project. These cost components are:

- Construction costs,
- Road user costs, and
- Societal costs.

Both alternatives include all three cost components. The comparison of relative construction costs (i.e. patching/resurfacing for Alternative 1 and ground improvement construction for Alternative 2) depend on the method implemented and the amount of patching/resurfacing done. However, the road user costs and societal costs are always higher for the Alternative 1, due to relatively poor pavement conditions which exist on the roadways as a result of this alternative.

These three cost components listed above and the effect of pavement condition on the costs involved is discussed in the following.

6.3 Construction Costs

The construction costs encompass all of the expenses involved for both alternatives that have to do with work being done on the site during its lifetime. The construction costs associated with both alternatives are shown in Figure 68.

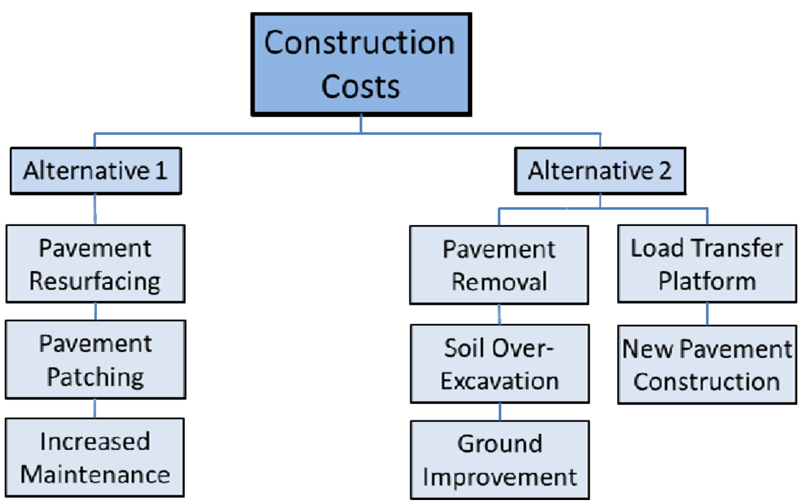


Figure 68. Construction costs for Alternatives 1 (patching/resurfacing) and Alternative 2 (ground improvement)

Both alternatives will have maintenance and regular pavement resurfacing projects done throughout their lifetime. The regular pavement wear and tear is not included in the cost analysis, since it will occur in both alternatives. The costs listed under Alternative 1 in Figure 68 are the additional costs of these items with respect to Alternative 2 due to the pavement distress and failure caused by the settlement of problematic subgrade soils. It is readily apparent as observed at the two sites investigated for this project that the roadway having subgrade settlement problems will have more repair, resurfacing, and maintenance.

The construction cost items that should be considered for Alternative 1, therefore, are the additional expenses of repair work with respect to a roadway with no soil settlement problems. These costs may be difficult to identify, unless there is a very good and detailed record-keeping program. The costs should include not only the materials for the work done in-house but all the labor hours spent by all the personnel involved, e.g., engineers, technicians, pavement crew, traffic control, coordination, etc. It was difficult to identify a typical value for the additional money spent by ODOT on annual maintenance solely due to the settlement problems.

The construction costs for Alternative 2 include the removal of existing pavement, soil over-excavation for a load transfer platform, installation of ground improvement, construction of the load transfer platform, and placing new pavement. Alternative 2 provides a completely new roadway section at the problem site. Preliminary cost estimates for the methods can be obtained using the FHWA SHRP2 cost data calculation sheets as mentioned previously during the discussion of the decision matrix development section.

6.4 Road User Costs

The road user costs are the costs incurred by the public, i.e. users of the roadway. As reported by the National Cooperative Highway Research Program (NCHRP) Report 720 (Chatti and Zaabar 2012) published by the Transportation Research Board, the road user costs include vehicle operating costs, travel time delay costs, safety and accident costs, comfort and convenience cost, and environmental impact costs as shown in Figure 69. In this research, "environmental impacts" costs have been considered separate from the road user costs as a main cost category under "societal costs" because these environmental impacts affect the whole society, not only the roadway users. The road user costs reviewed and considered in this study are provided in Figure 70. As shown in the figure, the four main components of road user costs evaluated in this study are the costs associated with the vehicle operation, travel time delay, safety and accident, and comfort and convenience, which are summarized in the following.

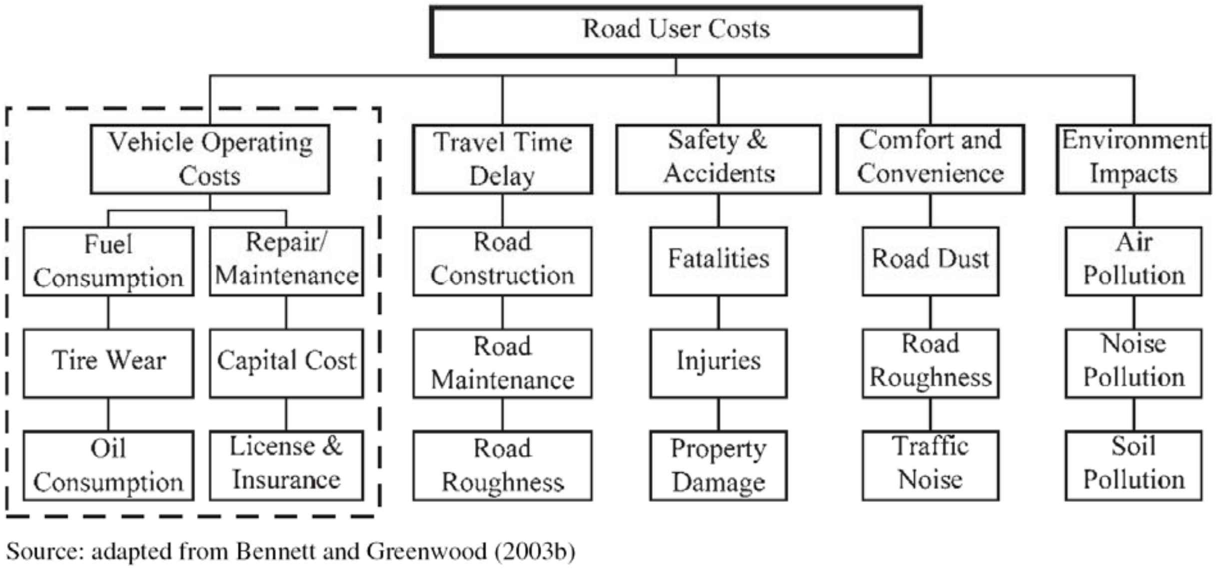


Figure 69. Road user cost components discussed in NCHRP Report 720
(after Chatti and Zaabar 2012)

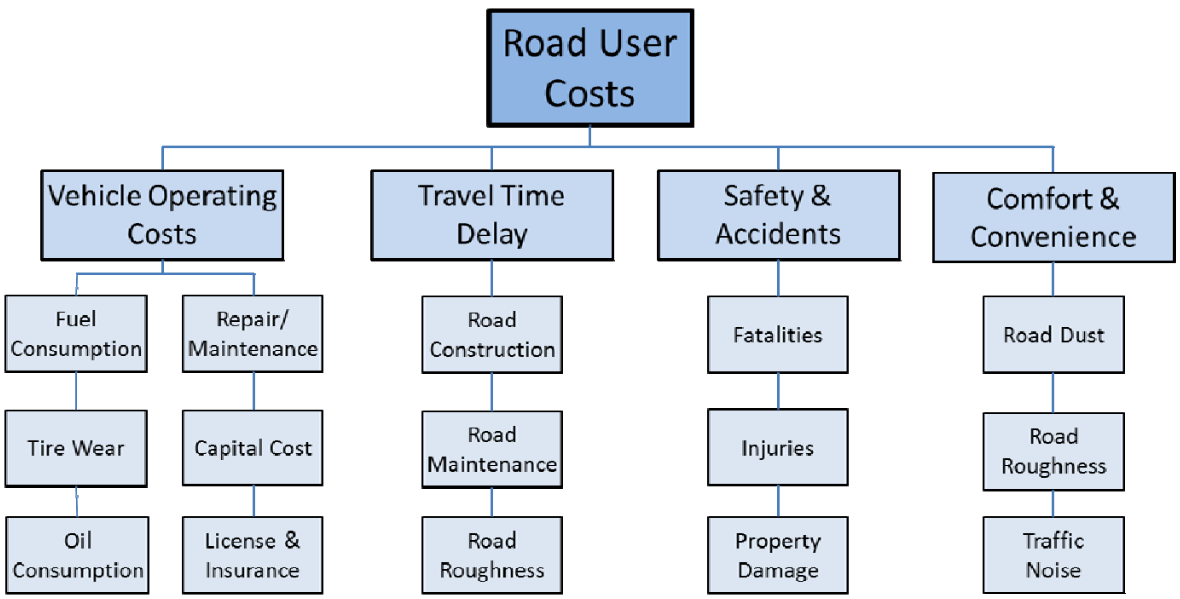


Figure 70. Road user cost categories and subcategories

6.4.1 Vehicle Operating Costs

Vehicle operating costs are made up of fuel consumption, tire wear, oil consumption, repair/maintenance, capital, and license & insurance costs. The roadway conditions which would be affected by the two alternatives considered affect some of the road user cost items significantly. The items affected the most are the fuel consumption, tire wear, and repair/maintenance costs. When considering the two alternatives, the capital, license, and insurance costs are not affected by the roadway conditions and the oil consumption is only affected marginally. Therefore, the three most significant contributors (i.e., fuel consumption, tire wear, and repair/maintenance costs) are considered and quantified for the cost-benefit analysis.

Alternative 1 will result in roadway conditions that have much worse average lifetime conditions than Alternative 2, so this difference will affect the vehicle operating costs. A research conducted by Chatti and Zaabar (2012) on behalf of the National Cooperative Highway Research Program investigated the effects of pavement roughness on fuel usage, tire wear, and vehicle maintenance. The NCHRP Report 720 presenting the findings of that research explains that as the condition of the roadway deteriorates and the roughness of the road increases, the amount of rolling resistance and friction increases. This increase in rolling resistance and friction causes the tires to wear faster, requires the engine to do more work, and overall creates more vibrations and torque on the vehicle. Chatti and Zaabar (2012) generated a spreadsheet program accompanying NCHRP Report 720 to calculate these vehicle operating costs by considering numerous factors including the road condition or “roughness”. Other inputs for this software include texture depth, percent grade, super-elevation, pavement type, speed, temperature, vehicle size distribution, daily traffic volume or roadway length, and vehicle kilometers (or miles) traveled for the section of the roadway in consideration. For this analysis, a grade and super-elevation are assumed to be zero at both sites, since neither one has significant grade or super-elevation. Figure 71 shows the input screen for the NCHRP Report 720 software.

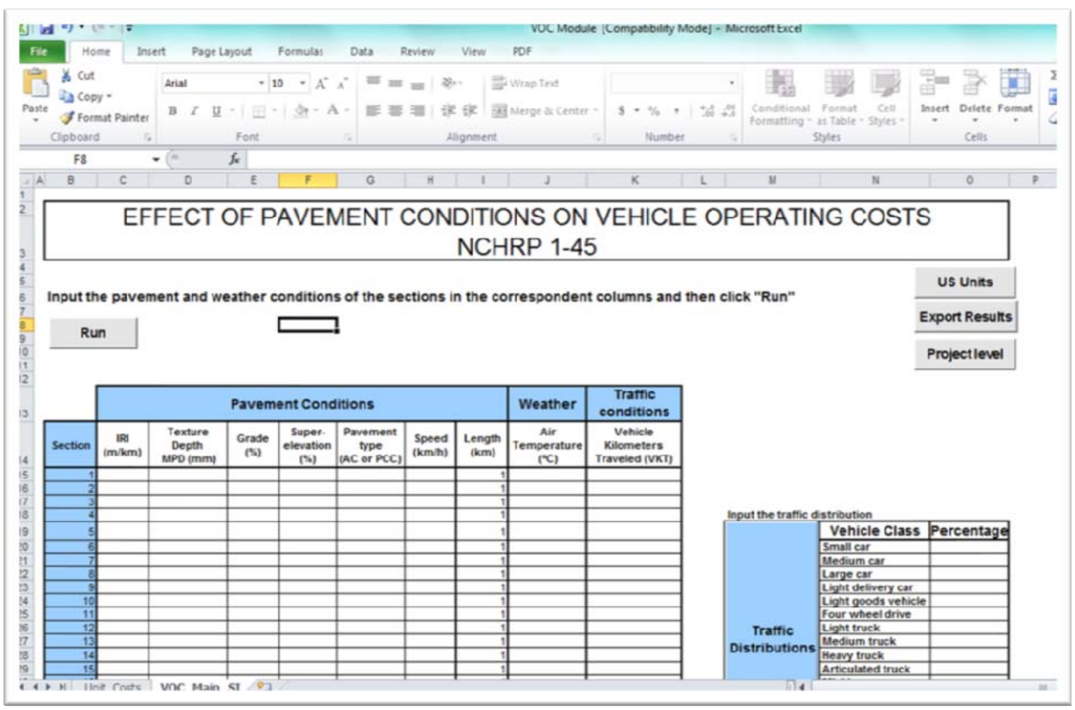
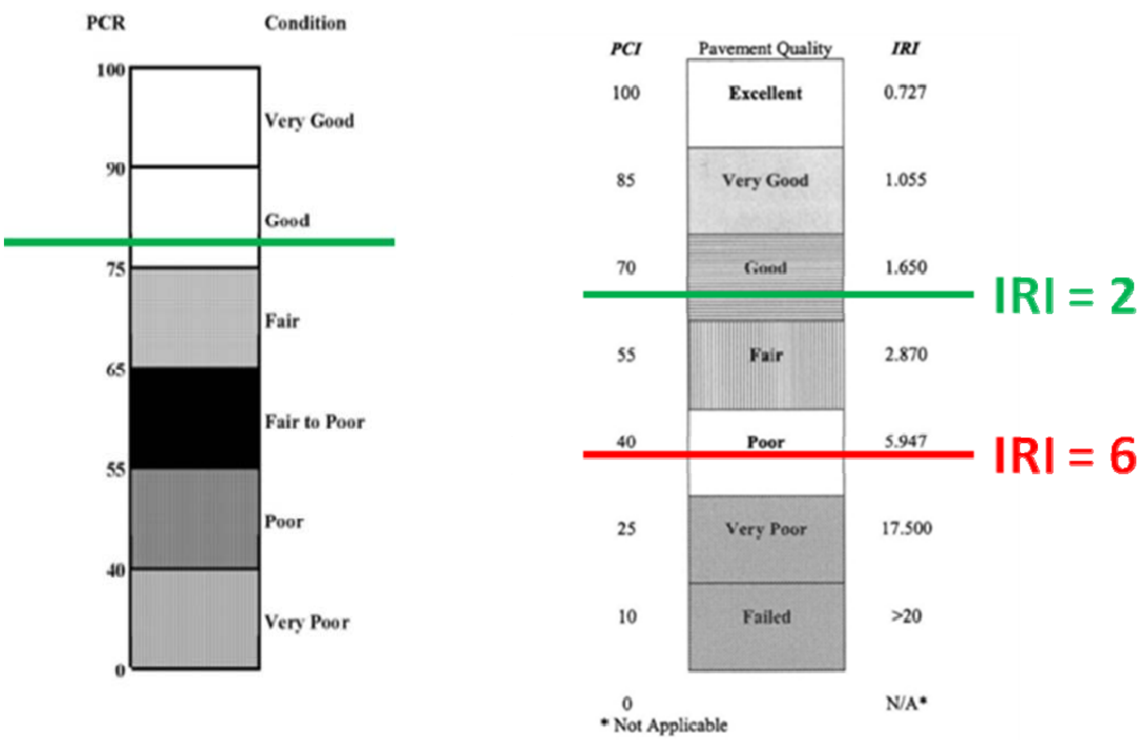


Figure 71. Vehicle operating costs calculator (NCHRP Report 720 companion software)

International Roughness Index (IRI):

The value used in the NCHRP Report 720 companion software to enumerate the road condition is the measurement of International Roughness Index (IRI). The IRI was developed by World Bank and it is a standardized roughness measurement used to define the characteristics of a longitudinal profile for a traveled wheel path. The IRI is a filtered ratio of a vehicle's accumulated suspension motion in millimeters (or in inches) divided by the distance traveled by the vehicle in meters (or in miles). In this study millimeter/meter (mm/m) is used as unit for the IRI. The IRI is used throughout the world as a system for measuring road condition. IRI is also used by many state DOTs to determine roadway condition.

ODOT uses a different system, pavement condition rating (PCR), to measure the condition of their roadways. Therefore, a correlation was needed between IRI and ODOT's PCR values in order to apply NCHRP Report 720 companion software to Ohio roads that have been rated. Correlations were made between the rating systems by matching the written descriptions of the two rating systems to the measurement range associated with each rating system. Figure 72 shows a side by side comparison of the different pavement rating systems given by two different sources that show written descriptions.



PCR Manual (ODOT 2006) Park et al. (2007)

Figure 72. IRI and PCR comparison and correlation

After relating the two systems to each other, a range of IRI values were used for the cost analysis. The IRI values are determined to be ranging between an IRI of two indicating good roadway conditions, and IRI of six, indicating poor roadway conditions. The actual vehicle operating costs from the varying IRI values add up to a large sum of money especially over the lifetime of a roadway with traffic growth factored in. Because of this, instead of including the vehicle operating costs from both alternatives in the cost comparison, only the difference in vehicle operating costs between the Alternative 1 and Alternative 2 solutions will be used. Alternative 2 will assume good roadway conditions due to the ground improvement with an IRI value of two and Alternative 1 will always assume a higher average lifetime IRI. This is due to the fact that Alternative 1 does not permanently remediate the settlement issues and therefore will have higher IRI values. The difference in the vehicle operating costs due to the higher IRI values of Alternative 1 will be the costs added to Alternative 1 in the cost-benefit analysis.

Vehicle Operating Cost Calculations:

Vehicle operating costs are determined for IRI values of two to six. The two alternatives are assigned an IRI value and the difference between the two vehicle operating costs is found and is added into the overall cost of Alternative 1. Alternative 2 is assumed to have an overall lifetime average condition of an IRI of two because once the ground improvement method is implemented the roads will then be assumed to have an average Ohio road rating (actually it will be better than an average, but this value is used to be conservative). According to Ohio's recently collected PCR data, the average PCR rating of Ohio roadways is "good". This pavement condition description corresponds to an IRI value of approximately two, therefore an IRI value of two will always be assumed for Alternative 2. This means that Alternative 1 will have a range of IRI values of three to six. This method finds the additional vehicle operating costs caused by the difference in average pavement condition, ΔIRI value of one, two, three, and four. It should be noted that IRI values of both Alternative 1 and Alternative 2 will fluctuate between the patching and resurfacing works, but the difference in IRI values considered in the analysis are the average values through the lifetime for the roadway.

6.4.2 Travel Time Delay

The travel time delay costs are the costs that accumulate during any delay from the time value of people involved and things affected by the delay. In road work, a delay is usually a result of a work zone or a detour. There can also be a travel time delay due to the condition of the road, but this has such a small effect on the travel time, the actual delay costs would be negligible. When a person driving on the road is delayed, it costs them money because of the lost potential labor hours. The FHWA's Work Zone Road User Cost Manual (FHWA, 2011) covers all the delay costs from work zones and provides example calculations. The delay costs calculated include personal travel time value, work travel time value, truck travel time, vehicle-depreciation, freight inventory delay, vehicle operation during delay, and emission cost due to a delay.

For the cost-benefit analysis in this project, work zone delay costs were considered, however because of the number of unpredictable variables needed for the calculations, and comparable work zone delay conditions between the two alternatives are predicted to occur, the work zone delay costs were eliminated from further consideration and from the overall cost comparisons.

6.4.3 Safety and Accidents Costs

This cost is based on the statistical probability of a car accident occurring on a roadway and costs associated with that accident. A method outlined in the FHWA Work Zone Road User Costs (2011) manual for calculating the crash costs was used to calculate the cost of safety and accidents risk in this analysis. The method used in the manual identifies the crash rate of three types of accidents at a specific site based on the probability of accidents. The three types of accidents that are assigned a crash rate are accidents resulting in fatality, injury, and property damage only. An amplification factor defined as the "crash modification factor" is applied to the crash rates to adjust for the increased risks. For the analysis performed in this study a "crash modification factor" will be applied to adjust for the increased risk caused by worsened pavement conditions.

A study conducted by Chan et al. (2010) found a strong correlation between the roadway condition and the frequency of accidents. The study shows that the frequency of accidents increases as the pavement conditions worsen. Chan et al. (2010) used several parameters for the pavement condition one of which was present serviceability index (PSI). PSI is scaled from 0 to 5, with 5 corresponding to the best pavement conditions. Figure 73 shows how the accident frequency increases as the pavement conditions worsen for a given annual average daily traffic (AADT). Since the cost values are based on IRI values, IRI values corresponding to the PSI values are also presented in Figure 73. The correlations shown in Figure 73 are used to determine percent increase in crash rates due to the decrease in pavement conditions for the changing IRI values.

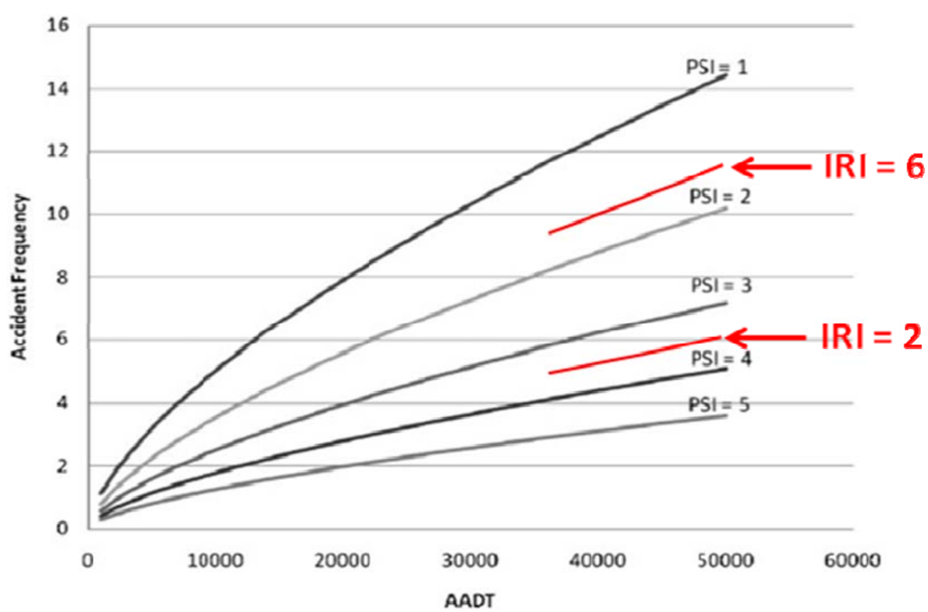


Figure 73. Effect of pavement condition on accident frequency (modified from Chan et al. 2010)

Once a crash rate amplification factor is found for the difference in road condition and is applied to the crash rate, the adjusted crash rate is then multiplied by the volume of the traffic traveling the road each year to get a number of accidents at the site each year resulting in an injury, fatality and property damage only. The number of accidents of each type is then multiplied by the monetary value given to each type of accident.

The FHWA Work Zone Road User Cost manual (FHWA 2011) uses two different types of values for accidents; human capital costs and comprehensive costs. Human capital costs are given by the National Highway Traffic Safety Administration (NHTSA) and the National Safety Council (NSC) bulletins, and are defined as "those 'hard dollar' costs related directly to the crash such as property damage, medical care, compensations and legal costs." The comprehensive costs come from the USDOT estimate of the economic value of a statistical life and the FHWA Technical Advisory (T 7570.2), "Motor Vehicle Accident Costs", and include "the intangible nonmonetary losses or consequences to individual, families and the society, in addition to the human capital costs" (Mallela and Sadasivam 2011). The values given in this manual were given in 2001 dollars. The values were converted to 2014 dollars using the Consumer Price Index conversion for the analysis in this project. Table 20 shows the 2001 human capital cost and comprehensive cost values from the FHWA road user cost manual, and also the adjusted 2014 values.

Table 20. Monetary values of crashes (after Mallela and Sadasivam 2011)

	Speed ≥ 80 km/hr (50 mi/hr)			**Speed ≤ 72 km/hr (45 mi/hr)**		
	Fatality	**Injury**	**Property Damage Only**	**Fatality**	**Injury**	**Property Damage Only**
Human Capital Value (2001)	1,277,640	52,569	6,497	1,117,167	36,604	6,291
Comprehensive Value (2001)	4,106,620	98,752	7,800	3,622,179	60,900	7,068
Human Capital Value (2014)	1,646,110	67,730	8,371	1,439,357	47,161	8,105
Comprehensive Value (2014)	5,290,964	127,232	10,050	4,666,811	78,463	9,106

Note: All values are in US dollars.

Similar to the vehicle operating costs calculations, a range of IRI values are used in the analysis to find a difference between accident risk costs at a range of roadway conditions. Again, Alternative 1 assumes the higher IRI value because the settlement issues that persist in Alternative 1 will continue to prevent the roadway from maintaining good conditions like an average roadway.

6.4.4 Comfort & Convenience Costs

For the cost-benefit analysis in this project, comfort & convenience costs were not considered because these are relatively minor cost items and there are a number of unpredictable variables needed for the calculations.

6.5 Societal Costs

The environmental impact (or societal) costs affect the society as a whole. However, it is difficult to identify to what extent they are affecting society and what the real long term costs will be since some of the long term effects of certain practices are still unknown. The main types of environmental impacts are air, soil, and water pollution. It is important not to ignore these impacts, as they could play a more significant role in the future. For this project, the potential future impacts of either soil or water pollution do not have a money value associated with them. The analysis does consider the cost of the impact that air pollution has on society. Using the NCHRP Report 720 companion software, the difference in fuel usage between different IRI values was determined to identify the quantity of vehicle emissions associated with that amount of fuel usage. Once the quantity of additional vehicle emissions is determined, they are multiplied by the monetary values assigned to each type of emission.

For the analysis in this project, Alternative 1 was again assigned the higher IRI value to represent the lifetime overall rougher pavement. The values for quantities of vehicle emissions given off per fuel usage are obtained from the EPA Office of Transportation and Air Quality's Emission Facts (EPA 2000). These quantities for passenger cars are presented in Table 21.

Table 21. Average emissions and fuel consumption for passenger cars (EPA 2000)

Pollutant/Fuel	Emission & Fuel Consumption Rates (per mile driven)	Calculation	Annual Emission & Fuel Consumption
VOC	1.034 grams (g)	(1.034 g/mi) x (12,000 mi/yr) x (1 lb/454 g)	27.33 lb
THC	1.077 g	(1.077 g/mi) x (12,000 mi/yr) x (1 lb/454 g)	28.47 lb
CO	9.400 g	(9.400 g/mi) x (12,000 mi/yr) x (1 lb/454 g)	248.46 lb
NOx	0.693 g	(0.693 g/mi) x (12,000 mi/yr) x (1 lb/454 g)	18.32 lb
PM_{10}	0.0044 g	(0.0044 g/mi) x (12,000 mi/yr) x (1 lb/454 g)	0.12 lb
$PM_{2.5}$	0.0041 g	(0.0041 g/mi) x (12,000 mi/yr) x (1 lb/454 g)	0.11 lb
CO_2	368.4 g	(368.4 g/mi) x (12,000 mi/yr) x (1 lb/454 g)	9,737.44 lb
Gasoline Consumption	0.04149 gallons (gal)	(12,000 mi/yr) / (24.1 mi/gal)	497.93 gal

Once the additional vehicle emissions resulting from Alternative 1 are found, the emission cost of the alternative is found by multiplying by those additional emission amounts by monetary values. The monetary values used in this analysis are taken from the emission cost calculation section of the FHWA Work Zone Road User Cost manual (FHWA 2011). These values were provided as 2010 California urban dollars, so for this analysis the values were converted to 2014 Ohio urban dollars using the Consumer Price Index conversion. Table 22 provides the original calculated and converted values for dollar amounts for emissions.

Table 22. Calculated and converted monetary values of emissions

	CO_2	VOC	CO	NOx	PM10
CA Urban 2010	$ 37.00	$ 1,140.00	$ 70.00	$ 16,300.00	$ 131,800.00
Cleveland/Akron 2014	$ 36.08	$ 1,111.75	$ 68.26	$ 15,896.04	$ 128,533.64

6.6 Cost-Benefit Analysis of Project Sites

Using the cost items and approach discussed above the lifetime cost-benefit analysis for the SUM-224 and STA-44 sites are performed and presented in this section. Cost items for the Alternative 1 and Alternative 2 will be as follow:

- Alternative 1 cost categories:
  - Construction costs (additional repair and maintenance costs due to the subgrade settlements)
  - Road user costs (additional road user costs compared to Alternative 2 due to the poor pavement conditions caused by subgrade settlements)
  - Societal costs (additional societal costs compared to Alternative 2 due to the poor pavement conditions caused by subgrade settlements)

- Alternative 2 cost categories:
  - Construction costs (ground improvement, load transfer platform, and new pavement)

The road user and societal costs for the Alternative 2 are not included, because Alternative 1 only includes the additional road user costs and societal costs compared to Alternative 2 due to the poor pavement conditions.

Insufficient information was able to be obtained on the maintenance history of these specific sites, and hardly any information was found indicating how often maintenance has been required at these sites in the past, or how much is usually spent on maintenance at these sites. This inhibits realistic maintenance costs from being used in the analysis because the actual costs, the type of work, and the frequencies of this work are unknown. ODOT manual states that routine maintenance is performed whenever a road might need it, but stated that the cost is usually ignored due to a “lack of dependable data” (ODOT Division of Pavement Engineering, 1999). This agrees with the information provided by ODOT District 4 regarding the routine maintenance; that is, detailed record is not kept of how much minor work is performed, how often it’s performed, and how much it costs each time at a specific site. However, it is safe to assume that the routine maintenance will cost more for a road with subgrade settlement issues. Even though there was no specific information about how often routine maintenance was performed at these sites, all the visible patches on the roadway and the varying thicknesses of the asphalt encountered in the field during subsurface investigations clearly indicate that additional patchwork had been done. For example, Figure 74 shows the thickness of asphalt at each borehole location at the STA-44 site. As shown in the figure, asphalt thickness of up to 559 mm (22 in) was encountered at the site. In addition, there was an attempt to stabilize the embankment on the east side, adjacent to the wetland at the STA-44 site, as previously mentioned.

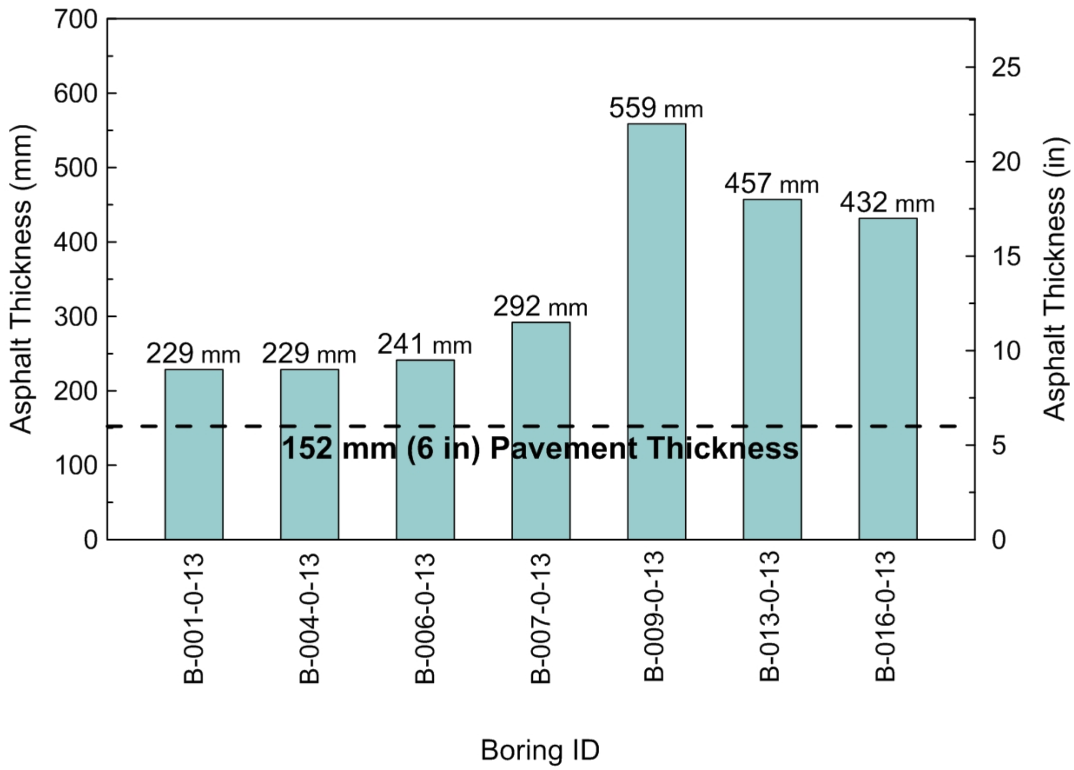


Figure 74. Thickness of asphalt at boring locations at STA-44 site indicative of repeated patching

In the absence of the specific cost data either from being nonexistent or from not being able to be located, for the purpose of the cost-benefit analysis of these two sites being investigated, no construction costs (additional repair and maintenance costs due to the subgrade settlements) are assumed for the Alternative 1. Only the road user costs and societal costs (additional costs compared to Alternative 2 due to the poor pavement conditions caused by subgrade settlements) will be considered and compared to the Alternative 2 construction costs.

6.6.1 SUM-224 Site Cost-Benefit Analysis

Traffic Volume:

Based on ODOT traffic survey reports, the current average daily traffic volume at the SUM-224 site is 22,400, with 20,890 cars and 1,510 trucks. Using the last ten year's traffic volume data for the SUM-224 site and the trend line analysis outlined in the Ohio Certified Traffic Manual (2007), the traffic volume growth projections were performed to estimate the future traffic volumes. These estimated volumes are used to determine some of the road user and societal costs. Figure 75 shows the projected total number of vehicle volume that will be using the SUM-224 site, which is experiencing ongoing settlement problems, over the next 100 years.

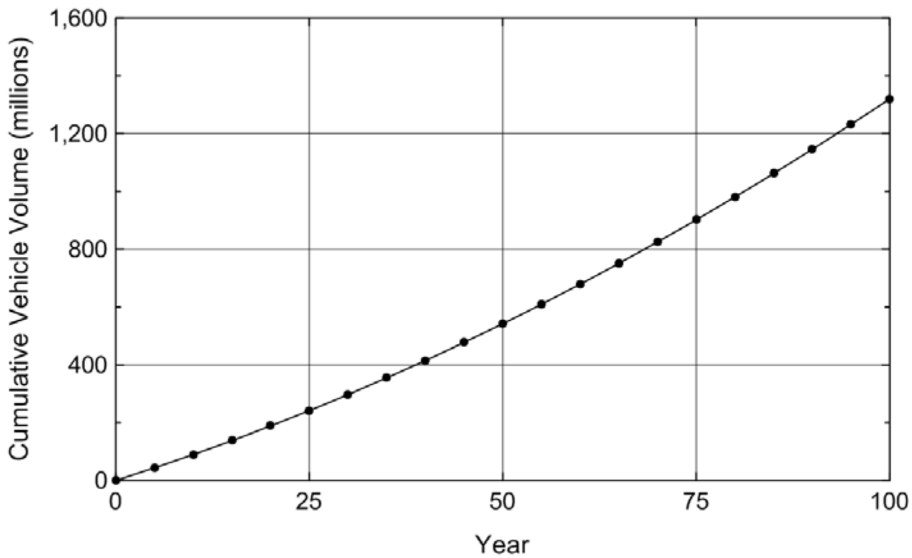


Figure 75. SUM-224 site total number of vehicles over lifetime

Construction Costs:

Construction costs for Alternative 1, as previously mentioned, are assumed to be zero for the SUM-224 site. Construction costs for Alternative 2 include the removal of existing pavement, soil over-excavation for a load transfer platform, installation of ground improvement, construction of the load transfer platform, and placing new pavement.

Based on the analysis using the decision tree developed, there were three technically feasible methods applicable at the SUM-224 site. These methods were jet grouting, deep soil mixing, and controlled modulus columns. Analysis results showed that all three methods could successfully be implemented at the site in order to stop or limit the subgrade settlement of the roadway. The construction costs estimates for the three alternatives were determined based on the SHRP2 cost estimation data and discussions with the specialty contractors that implement these methods. The average unit prices provided in the SHRP2 data sheets were used for the

SHRP2 estimates. SHRP2 did not have a cost estimation tool for the controlled modulus columns method, therefore only the contractor's estimate is used for this method. Table 23 provides the preliminary cost estimates for the installation of ground improvement methods only for the three technically applicable methods. Table 24 shows the preliminary cost estimates for all the cost items included in Alternative 2 mentioned earlier.

The construction costs listed in Table 24 show that for Alternative 2 controlled modulus columns is the most cost effective method for the SUM-224 site based on the preliminary cost estimates.

Road User and Societal Costs:

Road user costs (vehicle operating costs and safety/accidents costs) and societal costs (emission costs) are the costs items for Alternative 1 since the additional road user costs compared to Alternative 2 due to the poor pavement conditions caused by subgrade settlements are calculated.

Figure 76 provides the additional costs of Alternative 1 for various average IRI differences compared to Alternative 2 during its lifetime at the SUM-224 site. Figure 76(a) presents the vehicle operating costs, Figure 76(b) presents the safety and accident costs, and Figure 76(c) presents the emission costs associated with the poor pavement conditions. As shown in the figure, the safety and accidents cost is the largest cost item among the three contributors, because of the consequences of accidents. The total of all three cost items are given in Figure 77. These graphs factor in the predicted future traffic growth using linear trend-line analysis as discussed earlier.

Table 23. SUM-224 site Alternative 2 estimated construction costs (ground improvement only)

	Construction Costs ($)	
Method	**Average SHRP2**	**Contractor**
Deep Soil Mixing	2,028,154	2,450,000
Jet Grouting	11,770,883	6,100,000
Controlled Modulus Columns	N/A	1,750,000

Table 24. SUM-224 site Alternative 2 all estimated construction costs

	Construction Costs ($)	
Method	**Average SHRP2**	**Contractor**
Deep Soil Mixing	2,242,874	2,990,000
Jet Grouting	12,215,483	6,640,000
Controlled Modulus Columns	N/A	2,290,000

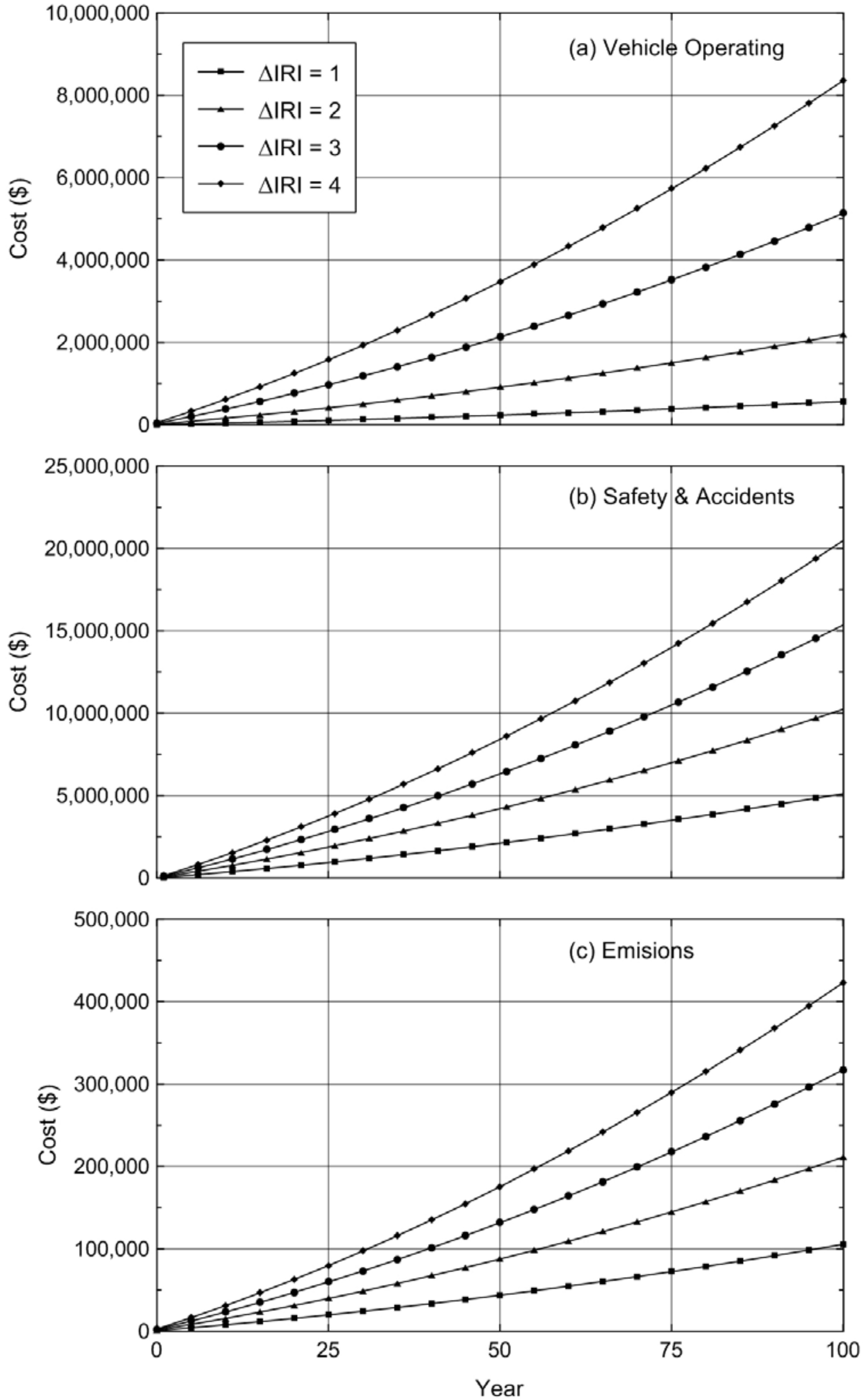


Figure 76. SUM-224 site estimated road user and societal costs: (a) vehicle operating costs, (b) safety & accidents costs, and (c) emissions

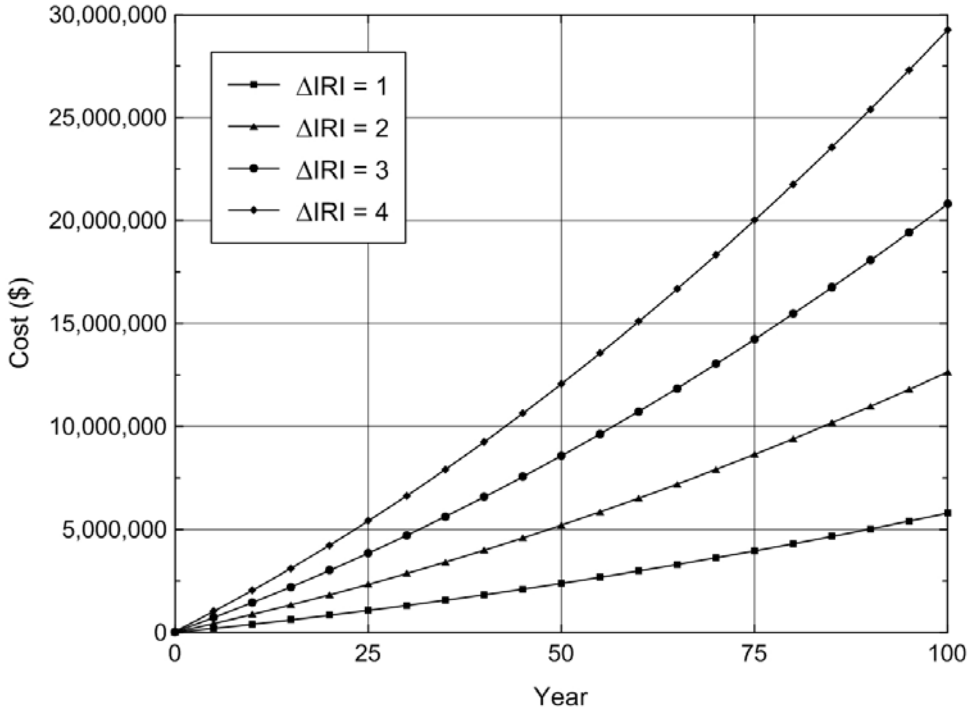


Figure 77. SUM-224 site estimated total road user and societal costs

The road user and societal costs for Alternative 2 are zero for the SUM-224 site, since only the additional costs associated with the relatively poor roadway conditions are considered in the Alternative 1 cost calculations.

Costs Comparison of Alternatives:

As presented under construction costs, the most cost effective feasible vertical column support method for the SUM-224 site is controlled modulus columns and all the costs for the remediation (Alternative 2) are estimated to be $2,290,000, as it was shown in Table 24. This cost of Alternative 2 is compared to the total costs of Alternative 1, which were presented in Figure 77, for the lifetime cost-benefit analysis and to determine the more cost effective method among the two alternatives being evaluated. The cost comparison is presented in Figure 78. In the figure, solid lines present costs of Alternative 1 for different IRI differentials between Alternative 1 and Alternative 2, while the dashed line presents the costs of Alternative 2.

The IRI values for the site were not measured, however based on the site conditions and its history, it is conservatively estimated that there will be an IRI difference of two, ΔIRI=2, on average between the Alterative 1 and Alternative 2 throughout the lifetime of this roadway section. For the estimated ΔIRI of two, the Alternative 2 would be the more cost effective alternative in 25 years. Any design life more than 25 years, which is the case for all roadways and ground improvement methods, Alternative 2 would result in cost savings to ODOT and their clients, the public. Even with a ΔIRI of one, the return of investment period is 49 years which is still much less than the design life of roadways and the ground improvement methods.

It should be noted that the costs for Alternative 1 did not include any additional repair/resurfacing expenses associated with the poor roadway conditions caused by the subgrade soil settlements. It has been noted that these expenses occur, however they were not included in the costs since there was not sufficient record related to these expenses. The inclusion of these expenses would result in the return of investment periods being much shorter.

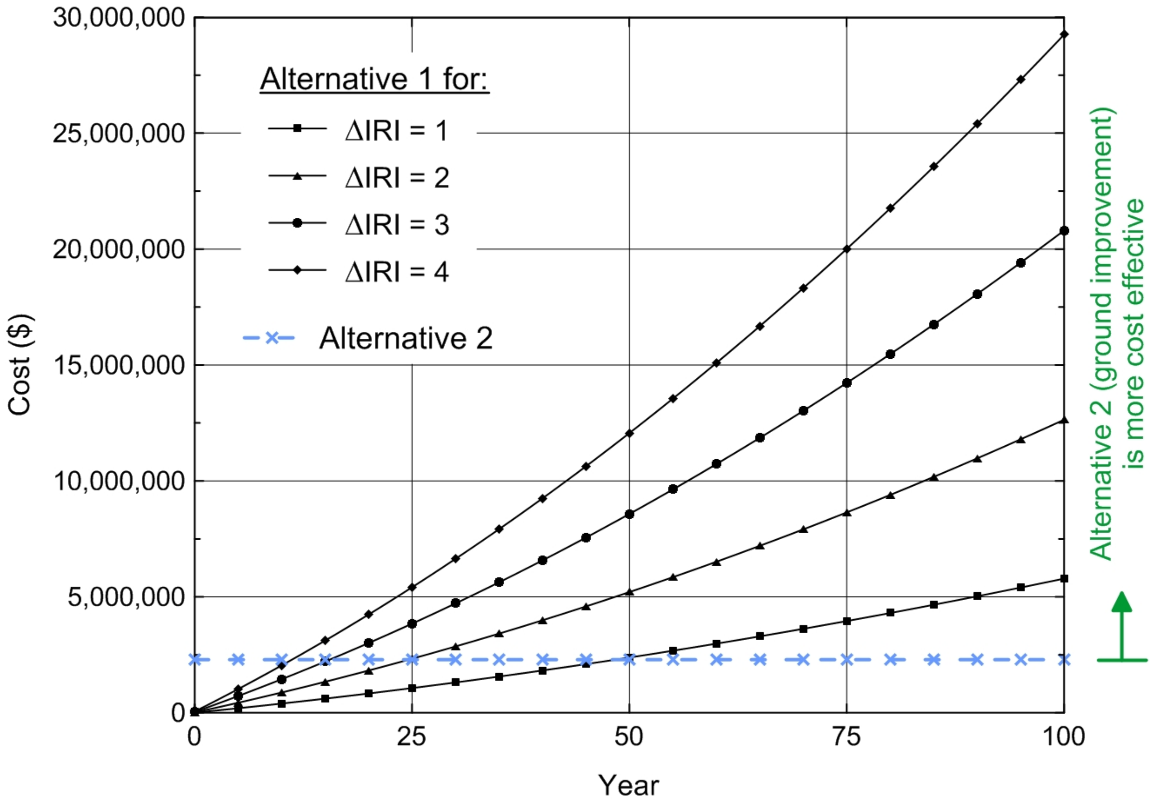


Figure 78. SUM-224 site cost comparisons of Alternative 1 and Alternative 2

6.6.2 STA-44 Site Cost-Benefit Analysis

Traffic Volume:

Based on ODOT traffic survey reports the current average daily traffic volume at the STA-44 site is 6,950, with 6,590 cars and 360 trucks. Using the last ten year's traffic volume data for the STA-44 site and the trend line analysis outlined in the Ohio Certified Traffic Manual (2007), the traffic volume growth projections were performed to estimate the future traffic volumes. These estimated volumes are used to determine some of the road user and societal costs. Figure 79 shows the projected total vehicle volume that will be using the STA-44 site, which is experiencing ongoing settlement problems, over the next 100 years.

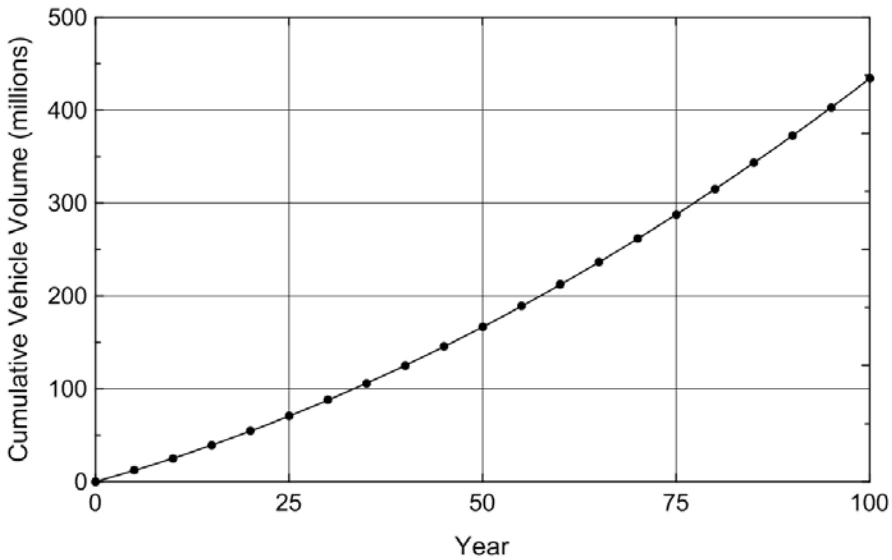


Figure 79. STA-44 site total number of vehicles over lifetime

Construction Costs:

Construction costs for Alternative 1 as previously mentioned are assumed to be zero for the STA-44 site.

Construction costs for Alternative 2 include the removal of existing pavement, soil over-excavation for a load transfer platform, installation of ground improvement, construction of the load transfer platform, and placing new pavement.

Based on the analysis using the decision tree developed, there were four technically feasible methods applicable at the STA-44 site. These methods were jet grouting, deep soil mixing, controlled modulus columns, and rammed aggregate piers. Analysis results showed that all four methods could successfully be implemented at the site in order to stop or limit the subgrade settlement of the roadway. The construction cost estimates for the four alternatives were determined based on the SHRP2 cost estimation data and discussions with the specialty contractors that implement these methods. The average unit prices provided in the SHRP2 data sheets were used for the SHRP2 estimates. SHRP2 did not have a cost estimation tool for the controlled modulus columns method, therefore only the contractors estimate is used for this method. Table 25 provides the preliminary cost estimates for the installation of ground improvement methods only for the four technically applicable methods. Table 26 shows the preliminary cost estimates for all the cost items included in Alternative 2 mentioned earlier.

Table 25. STA-44 site Alternative 2 estimated construction costs (ground improvement only)

	Construction Costs ($)	
Method	**Average SHRP2**	**Contractor**
Deep Soil Mixing	787,103	680,000
Jet Grouting	3,304,167	1,700,000
Controlled Modulus Columns	N/A	556,000
Rammed Aggregate Piers	789,700	638,750

Table 26. STA-44 site Alternative 2 all estimated construction costs

	Construction Costs ($)	
Method	**Average SHRP2**	**Contractor**
Deep Soil Mixing	981,616	916,250
Jet Grouting	3,498,680	1,936,250
Controlled Modulus Columns	N/A	792,250
Rammed Aggregate Piers	883,640	875,000

The construction costs listed in Table 26 show that for Alternative 2 controlled modulus columns is the most cost effective method for the STA-44 site based on the preliminary cost estimates. It should be noted that rammed aggregate pier costs are based upon a site specific estimate by the contractor, while all others are based upon general contractor provided parameters.

Road User and Societal Costs:

Road user costs (vehicle operating costs and safety/accidents costs) and societal costs (emission costs) are the costs items for Alternative 1 since additional road user costs compared to Alternative 2 due to the poor pavement conditions caused by subgrade settlements are calculated.

Figure 80 provides the additional costs of Alternative 1 for various average IRI differences compared to Alternative 2 during its lifetime at the STA-44 site. Figure 80(a) presents the vehicle operating costs, Figure 80(b) presents the safety and accident costs, and Figure 80(c) presents the emission costs associated with the poor pavement conditions. As shown in the figure, the safety and accident costs is the largest cost item among the three contributors, because of the consequences of accidents. The total of all three cost items are given in Figure 81. These graphs factor in the predicted future traffic growth using linear trend-line analysis as discussed earlier.

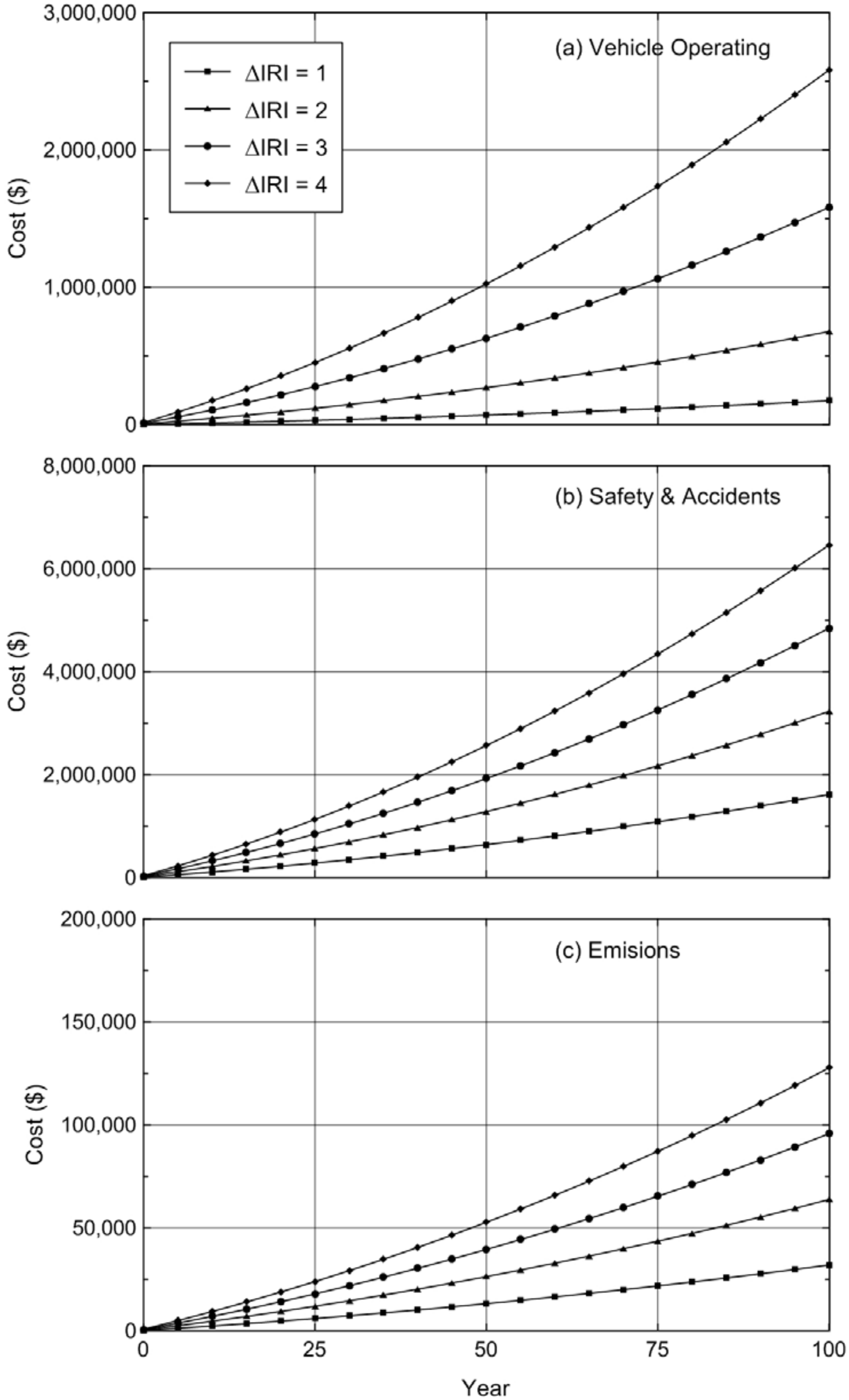


Figure 80. STA-44 site estimated road user and societal costs: (a) vehicle operating costs, (b) safety & accidents costs, and (c) emissions

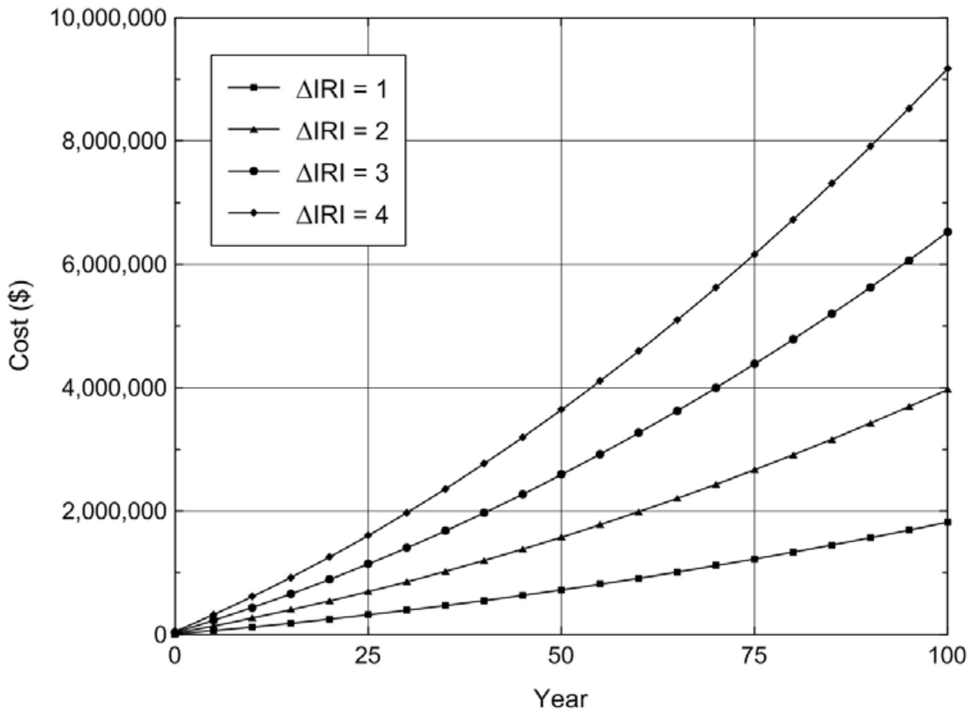


Figure 81. STA-44 site estimated total road user and societal costs

Similar to the SUM-224 site, the road user and societal costs for Alternative 2 are zero for the STA-44 site, since only the additional costs associated with the relatively poor roadway conditions are considered in the Alternative 1 cost calculations.

Costs Comparison of Alternatives:

As presented under construction costs, the most cost effective feasible vertical column support method for the STA-44 site is controlled modulus columns and all the costs for the remediation (Alternative 2) is estimated to be $875,000, as it was shown in Table 26. This cost of Alternative 2 is compared to the total costs of Alternative 1, which were presented in Figure 81, for the lifetime cost-benefit analysis and to determine the more cost effective method among the two alternatives being evaluated. The cost comparison is presented in Figure 82. In the figure, solid lines present costs of Alternative 1 for different IRI differentials between Alternative 1 and Alternative 2, while the dashed line presents the costs of Alternative 2.

The IRI values for the site were not measured, however based on the site conditions and its history, it is conservatively estimated that there will be an IRI difference of two, ΔIRI=2, on average between Alterative 1 and Alternative 2 throughout the lifetime of this roadway section. For the estimated ΔIRI of two, Alternative 2 would be the more cost effective alternative in 29 years. Any design life more than 29 years, which is the case for all roadways and ground improvement methods, Alternative 2 would result in cost savings to ODOT and the public. Even with a ΔIRI of one, the return of investment period is 54 years which is still much less than the typical design life of roadways and the ground improvement methods.

It should be noted that the costs for Alternative 1 did not include any additional repair/resurfacing expenses associated with the poor roadway conditions caused by the subgrade soil settlements. It has been noted that these expenses occur, however they were not included in the costs since there was not sufficient record related to these expenses. The inclusion of these expenses would result in the return of investment periods being much shorter.

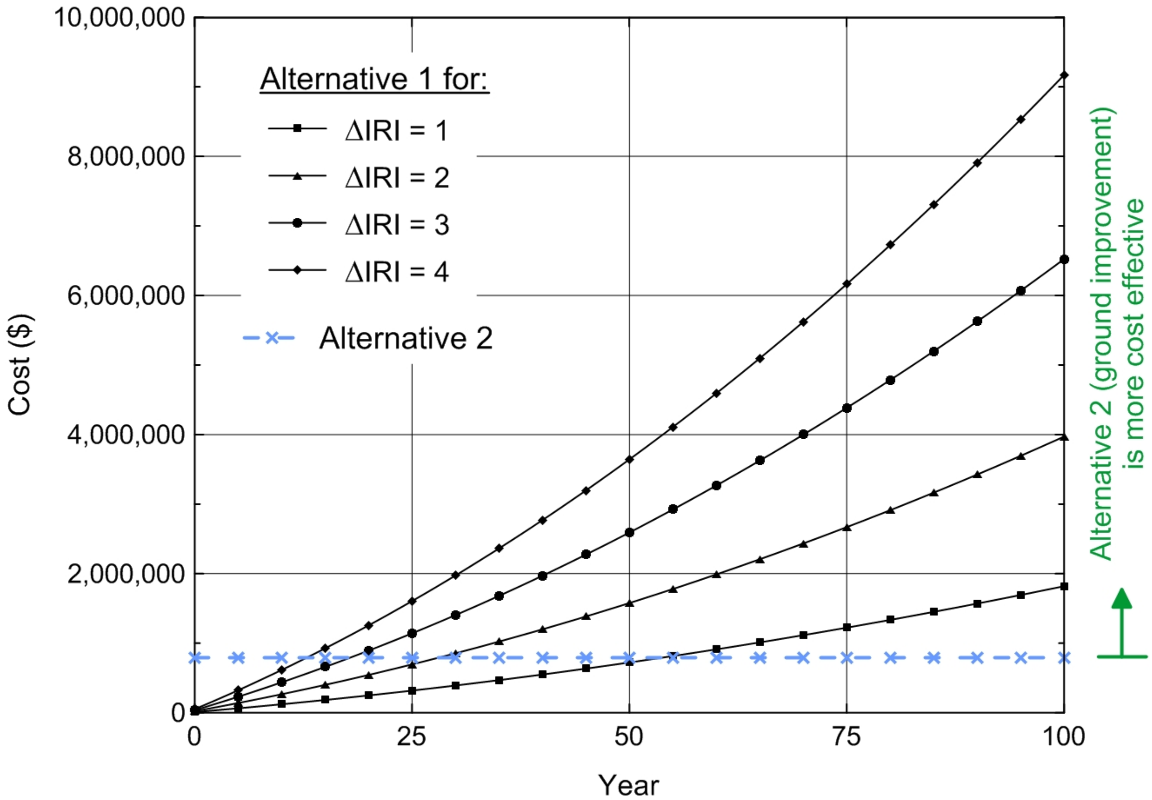


Figure 82. STA-44 site cost comparisons of Alternative 1 and Alternative 2

6.7 Design Charts for Lifetime Cost-Benefit Analysis

The calculations for the road user and societal costs can be very extensive and include many factors such as site conditions, traffic volume, traffic growth, average lifetime pavement condition, and increased maintenances costs. Because of this, lifetime cost-benefit analysis design charts were developed to provide a tool for evaluating the cost-effectiveness of an alternative. These costs were found for varying conditions and consolidated to present in a user-friendly and easy-to-use format.

The charts were developed based on the differing average lifetime IRI values, ΔIRI, similar to the process used for the SUM-224 and STA-44 sites. The costs presented in the design charts are per foot length of problem roadway section and per 1,000 vehicles average daily traffic volume on that roadway section. Two design charts were developed based on the speed limits of the roadway section: one for speed limits less than 80 km/hr (50 mi/hr) and another one for speed limits equal to or more than 80 km/hr (50 mi/hr). The lifetime cost-benefit analysis design charts developed are given in Figure 83.

The design charts given in Figure 83 are based on constant traffic volume and do not take into account traffic volume growth that will occur during the service life of the roadway. Because the traffic volume growth predictions are site specific, the predictions should be performed specifically for each project using linear trend-line analysis as discussed earlier and outlined in the Ohio Certified Traffic Manual (2007). Therefore, an adjustment factor should be applied to the estimated costs obtained from the design charts in Figure 83. In the absence of the traffic data for linear trend-line analysis for growth or for quick preliminary estimate a 50% increase in the costs can be conservatively used as the traffic volume growth adjustment factor. The 50% is recommended based on the analysis performed for the sites investigated in this project.

The cost estimate obtained from the design charts (plus the traffic volume growth adjustment) are the costs of Alternative 1 (i.e. no ground improvement method for remediation). These costs do not include any additional repair and maintenance costs due to the subgrade settlements. For a realistic and more accurate cost-benefit analysis, these costs should also be estimated for the duration of service life of the roadway and should be added to the costs obtained using the design charts in Figure 83, with the growth adjustment applied. Then these costs should be compared to the Alternative 2 costs, which is the application of the vertical column support method with the construction of a new road for the problem section of the roadway.

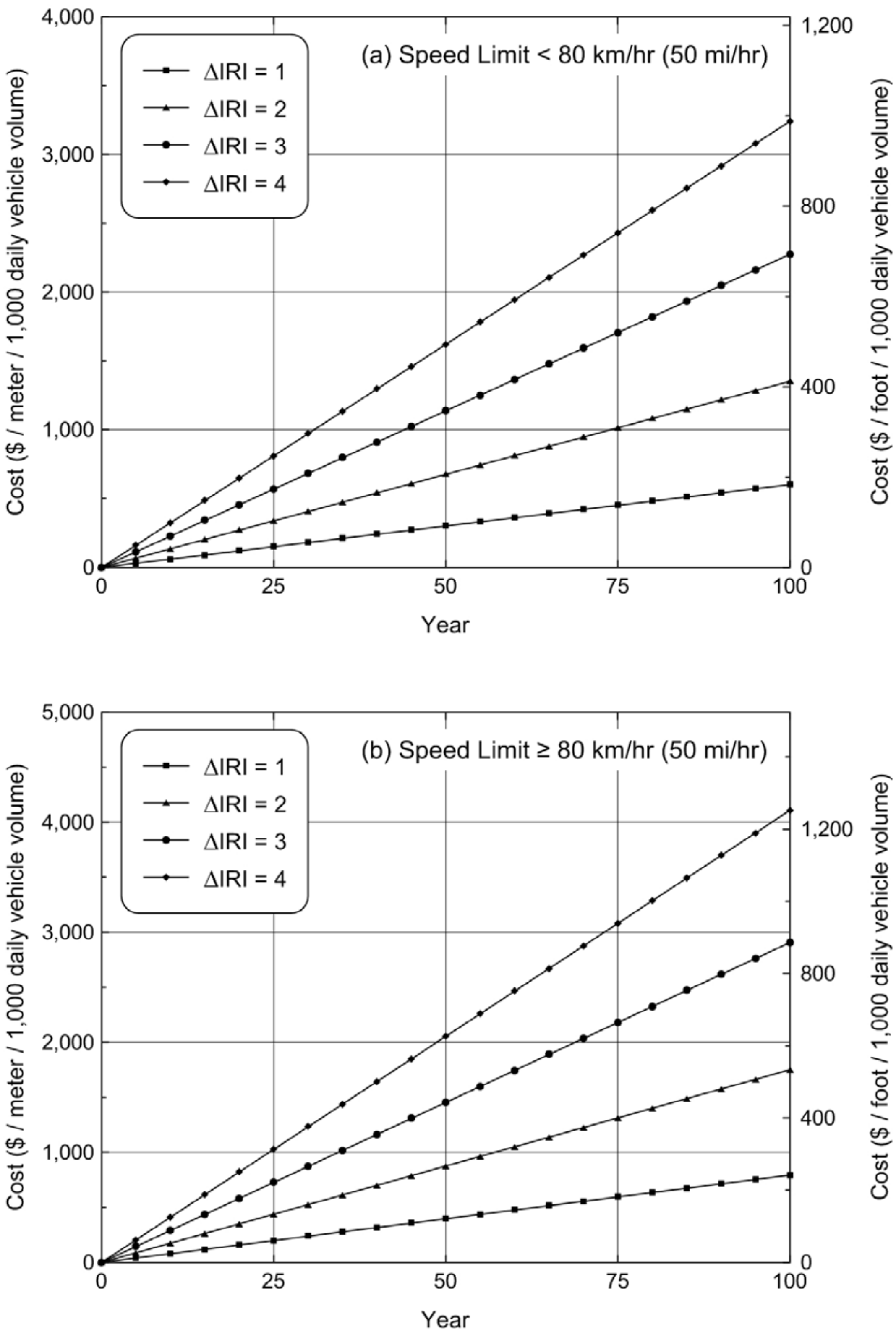


Figure 83. Design charts for lifetime cost-benefit analysis: (a) Speed limit < 80 km/hr (50 mi/hr) and (b) Speed limit ≥ 80 km/hr (50 mi/hr)

6.8 Summary and Conclusions

An overview of the costs associated with the effects of pavement condition on road user costs and societal costs was provided in this chapter. The permanent remediation of ongoing settlement problems of an existing roadway using vertical column support methods would significantly improve the pavement conditions and reduce the costs associated with poor roadway conditions. This would significantly reduce the costs associated with safety and accidents, which is the major cost contributor.

The cost-benefit analysis of two alternatives, Alternative 1 and Alternative 2 were discussed. Alternative 1 is a temporary solution, and also ODOT's current practice in remediating the pavement distress caused by subgrade soil settlements, which is patching and resurfacing. Alternative 2 is a permanent solution, which involves utilization of vertical column support ground improvement methods, in remediating the issues caused by the subgrade settlements. A comparative cost-benefit analysis for the two sites, SUM-224 and STA-44 sites, analyzed during this project for possible implementation of vertical support column methods was performed and the results were presented.

The cost-benefit analysis results show that although there are some initial costs associated with the construction and installation of ground improvement methods, Alternative 2 is definitely a more cost effective alternative when the lifetime of the roadway is considered. This is valid for both SUM-224 and STA-44 sites. The cost savings actually would be higher for Alternative 2, because the costs of additional patching/resurfacing repair work done at these sites were not included in the costs analysis. Unfortunately, there were no records for this kind of repair work done in house, although the subsurface investigations encountered very thick pavement at some of the boring locations indicating multiple pavement repair projects were done.

Design charts for the lifetime cost-benefit analysis have also been developed for estimating preliminary costs in future projects for Alternative 1 and then comparing those to the cost estimates for Alternative 2.

CHAPTER 7. CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

Every year pavement patching/resurfacing projects are undertaken by ODOT to repair pavement distress/structural failure due to soft and/or organic soils constituting the subgrade. These conditions seriously impact roadway function and safety, and create substantial costs to remediate the problems caused by subgrade settlement.

There are various vertical column support methods used for civil engineering structures when soils are not strong enough to support the structure. These methods are grouped in two main categories: ground improvement methods and deep foundation systems. Many of the vertical column support methods are also commonly used in transportation projects. Although some of these methods are utilized by several state DOTs to remediate settlement problems of existing roadways, they are not very commonly used for this application and they are definitely a new concept for ODOT, since they have never been used for an existing roadway in Ohio.

The main objectives of this research project were to evaluate the applicability and use of various vertical column support systems to improve the subgrade support and reduce settlements for existing roadways in Ohio, to perform a cost-benefit analysis of methods that can be used for remediation, and ultimately to determine cost effective means to reduce pavement distress that can be utilized for Ohio's problematic soils.

The extensive literature review indicated that there are methods suitable for different soil types. ODOT experiences problems with soft cohesive and organic soils causing subgrade settlements resulting in pavement distress and failure. A survey of other state DOTs indicated that many states have subgrade soil settlement problems under existing roadways and soft cohesive and organic soils are also the main problem soil types in their states. There are vertical column support methods that work well in these types of problem soils. The deep foundation systems usually are technically applicable to most site conditions, however they are prohibitively expensive alternatives compared to the ground improvement methods. Therefore, the project concentrated on the ground improvement methods.

After determining the critical components of various methods, a general decision matrix was developed to be used in identifying the technically feasible vertical column support methods to remediate the subgrade settlements. The decision matrix was initially started as a spreadsheet program. Later, the more user friendly and easy to use traditional flow chart version was developed that can be used to identify technically feasible methods for any project site having soft cohesive and/or organics as primary problem soil type.

Two sites have been identified by ODOT to conduct subsurface investigations to assess the suitability of the sites for possible implementation of some of the vertical column support methods. Subsurface investigations, including drilling and sampling, standard penetration testing, cone penetration testing, pressuremeter testing, and laboratory testing were conducted for detailed site characterization and to obtain soil properties for the design of vertical column support methods during the implantation phase. Very soft and thick soil layers were encountered at the sites investigated, especially at the SUM-224 site.

Based on the results of the subsurface investigations and the decision matrix developed, the vertical column support methods technically applicable at the sites have been identified as:

- SUM-224 site:
  - Jet grouting,
  - Deep soil mixing, and
  - Controlled modulus columns.

- STA-44 site:
  - Jet grouting,
  - Deep soil mixing,
  - Controlled modulus columns, and
  - Rammed aggregate piers.

The overview of cost components involved to perform a lifetime cost-benefit analysis has been discussed. Earlier studies showed the pavement conditions affect the costs associated with the roadway related to the road user and societal costs. The biggest cost item was found to be the safety & accident related costs. The previous studies also showed that as the pavement conditions deteriorate the frequency of accidents increase.

One of the important expense categories for the cost benefit analysis is the additional patching/resurfacing costs required to repair damage caused by the subgrade soil settlements. Therefore, keeping good records of all the material and labor costs associated with the repair work performed on the roadways having subgrade settlement problems is very important for the lifetime cost-benefit analysis to evaluate cost effectiveness of permanent remediation methods.

Design charts to be used for the lifetime cost-benefit analysis of future projects have been developed to estimate preliminary costs associated with ODOT's current practice of patching/resurfacing of roadway sections showing pavement distress due to the subgrade soil settlements.

The research findings indicate that:

- Some of the vertical column support methods could be used to remediate subgrade soil settlements caused by soft cohesive and organic soils in Ohio,
- Multiple technically feasible methods are available for the remediation of the two sites investigated for possible implementation, and
- The application of the ground improvement methods is more cost effective alternative compared to ODOT's current practice of patching/resurfacing.

Based on the results of the lifetime cost-benefit analyses performed for the SUM-224 and STA-44 sites considered for possible implementation work, it was decided not to pursue the implementation phase of the project. Although the lifetime cost-benefit analysis showed that the benefits outweighs the cost, the initial construction costs and funds needed for the vertical column support methods as the reason for not pursuing the implementation phase of the project.

7.2 Recommendations

Identifying other sites where the problem soils are not as deep as the two sites being investigated during this project can be considered for possible implementation in the future. Shallow problematic soils would result in lower ground improvement installation costs which could be more reasonable as initial costs to pursue the implementation of vertical column support method(s).

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