Develop Settlement Criteria and Design Approach for Embankments and Retaining Walls Built on Compressible Soils

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16. Abstract

The overarching objective of this project is to identify settlement criteria for embankments and retaining walls and develop a systematic soil improvement design tool to facilitate the method selection and construction time and cost estimation. There is a consensus that the post-construction settlement has profound impacts on the service conditions of roadways and bridge approaches. However, there is no widely accepted settlement criterion in the nation, so each states practice differently. This project surveyed 49 state DOTs on their settlement criteria based on their zoning of bridge approach embankments. 23 out of 49 state DOTs responded to the survey and provided their information on settlement requirement and zoning. The collected data indicate that most state DOTs have their own settlement requirements for embankments, particularly, for the embankment supporting the bridge approaches. However, the requirement varies significantly. In addition, the zoning criteria used by these responding states are also drastically different and some state DOTs do not use zoning, instead, they use one criterion for its embankments regardless of its distance from the bridge. Among the states that have no existing statewide settlement requirement, they either allow each district to use its own criteria or allow the responsible project engineer to determine the allowable settlement based on the project condition. Considering that fact, an calculation tool was developed, which allows users to assess the construction time and cost at a user-specified allowable settlement. The developed calculation tool was validated by using extensive published field data. By using the tool, the user can assess multiple soil improvement methods and then select the best one or multiple technologies to achieve a balance between cost and construction time.

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EXECUTIVE SUMMARY

Excessive embankment or retaining wall settlements are the cause of many bridge and roadway distresses. It has been reported that more than 25% of the bridges in the US have approach slab problems, many of which are relevant to intolerable approach settlements or bumps. Most of state DOTs acknowledge the importance of controlling post-construction settlements of embankments and retaining walls. However, there is no widely accepted settlement criteria to guide the practice.

This project conducted a survey and collected responses from state DOTs regarding their criteria of embankment and retaining wall settlements. Out of the 49 state DOTs that we solicited responses from, 22 of them were able to send back responses on time, which reflects a responding rate of 45%. The data indicate that most of state DOTs have settlement requirement for their embankments, particularly, for the embankment supporting the bridge approach. However, the requirement varies significantly and can be divided into categories: (1) the embankment supporting bridge approach needs to settle nearly the same as the bridge; (2) the embankment supporting bridge approach can settle 0.5 or 1 inch more than the bridge; and (3) the allowable embankment settlement depends on the length of the bridge approach. Among the states that have no existing statewide settlement requirement, they either allow each district to use its own criteria or allow the engineer to determine the allowable settlement based on the project.

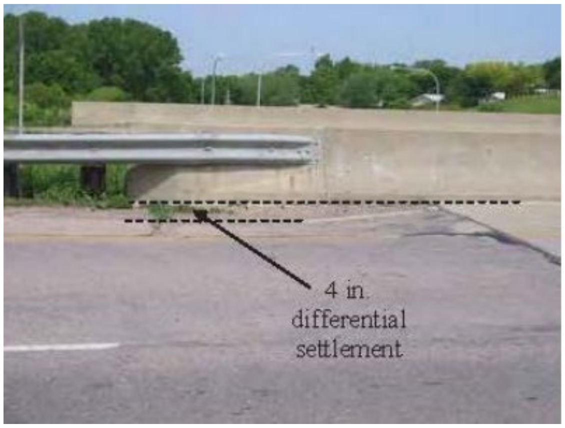
Based on the survey outcome, a two-step design procedure was developed with associated analysis tools to facilitate the implementation: preliminary selection charts and detailed calculator. For the preliminary stage, various cost-time charts were developed based on different Texas prototype settings for different degrees of settlement reductions. These charts can be used by designers to determine possible candidate of soil improvement methods for settlement mitigation based on the allowable project duration and budget. Thereafter, a calculation tool, i.e., a calculator, will allow users to perform detailed soil improvement design based on site-specific data and allowable settlement. The implementation of this procedure will help users maintain consistent in their design to achieve the balance between construction time and cost. The calculation tool includes modules to calculate settlements and installation parameters for popular soil improvement methods, such as preloading with wick drains, stone columns, rammed aggregate columns, deep soil mixing, etc. The tool was calibrated using published data.

CHAPTER 1 INTRODUCTION

1.1 STATEMENT OF PROBLEM

Embankments have been widely used to support roadways and railroads since 1980s as the urbanization and transportation needs have forced the use of sites that have soft and compressible soils (Leroueil 1994; Leroueil et al. 1990). It is estimated that most of the newly built highway embankments are over highly compressible soil in the coastal or delta regions worldwide because densest population are in these regions, which has the greatest increase of demand on transportation (Vipulanandan et al. 2009). However, embankment failure, particularly excessive settlement, has become a salient issue in many areas, causing riding discomfort or even transportation disruption, as shown in Figure 1-1. In general, the settlement will be noticeable when the differential settlement is 0.5 inches or more (Wahls 1990) and may become problematic when the settlement exceeds 1 inch (Zaman et al. 1991).

(a) (b)



(c) (d)

Figure 1-1. Excessive settlement of embankment: (a) railway embankment (Koseki et al. 2012), (b) highway embankment (Courtesy of Jie Han), (c) bridge approach settlement (courtesy of Iowa DOT), and (d) significant differential settlement at bridge approach (courtesy of Ohio DOT).

A statistical survey completed in Japan found that embankment failure, particularly, excessive settlement, occurs much more often than other types of earthen work failures, as shown in Figure 1-2 that illustrates the frequency of each failure type among all failure cases. In the figure, the total frequency of all types of failure exceeded 100% because some failure case involved more than one type of failure modes. Even though the data are derived from railroad embankments, it is representative of the issue we are facing with highway embankments. From their experience, WSDOT warned that the issue of excessive embankment settlement surpassed other issues of embankments, such as stability or erosion. They pointed out that post-construction settlement caused damage to structures and utilities located within the embankment or adjacent to the embankment. They also suggested that the embankment settlement near an abutment created an unwanted dip in the roadway surface, or downdrag on structures (WSDOT 2013).

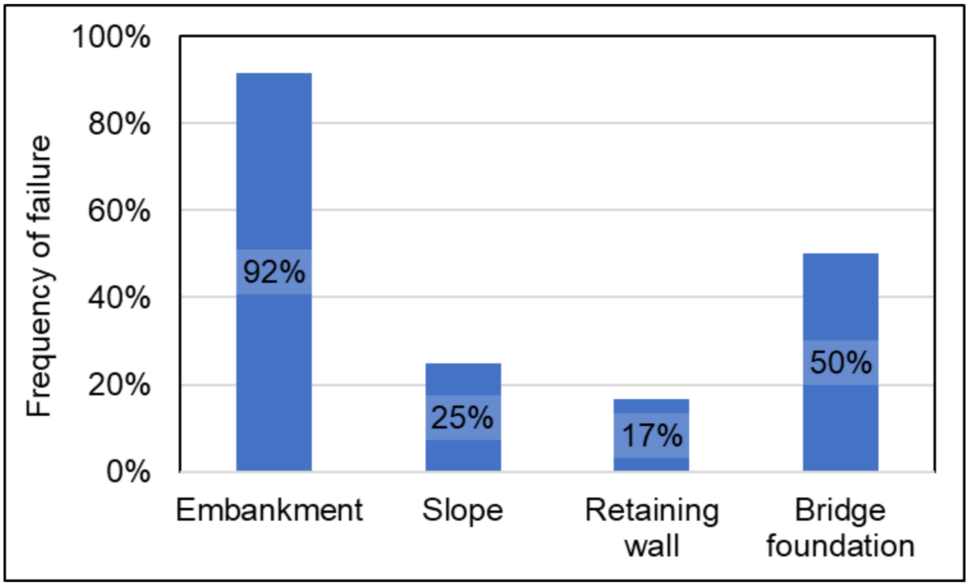


Figure 1-2*.* Occurrence of embankment failure (modified based on data in Koseki et al. (2012)).

These issues have a good reflection in TxDOT experience. TxDOT Project 0-6716, even though focused on the excessive movement of MSE wall, pointed out the excessive settlement of subsurface could be an important factor that led to unexpected MSE wall lateral movements. As a matter of fact, China estimated that about 30% of their pavement distresses were associated with intolerable embankment settlements. Although the U.S. does not have a nationwide statistical data of the occurrence of excessive embankment or retaining wall settlements, Ha et al. (2002) reported that 25.4% of TxDOT bridges had settlement problems. Briaud et al. (1997) stated that 25% of the bridges in the US have bridge approach slab problems and many of them are affected by approach settlements or bumps. Many other state DOTs acknowledged the importance of the issue and emphasized it in their specifications or manuals to provide guidance for site investigation, design, and quality assurance (for example, FLDOT (2000), LADOTD (2016), WSDOT

(2013)). Even though there is a census on the importance of controlling post-construction settlements of embankments and retaining walls that support roadways, there is no widely accepted criteria for allowable settlements. Equally important, the settlement criteria greatly impact the cost and construction time. As a result, lacking settlement criteria makes soil improvement practice difficult and makes construction time and cost estimation challenging.

Extensive studies have been conducted to investigate the mitigation of settlements of embankments and retaining walls approaching bridges. SHRP2 (The second Strategic Highway Research Program) sponsored a study titled "*Geotechnical Solutions for Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform*" (Schaefer and Berg 2012). The main objective of the project was to develop a soil improvement selection platform to facilitate design and construction of embankments and other working platforms. The direct outcome of the project is the birth of an online soil improvement selection platform named ***GeoTechTools*** (https://www.geoinstitute.org/geotechtools/login), which is free for any user. It covers 47 soil improvement methods, which are nearly all the methods in use all over the world. Among these methods, nearly 10 of them are ranked as routinely used by all DOTs, including lightweight filling, preloading with or without wick drains (i.e., PVDs), dynamic compaction, aggregate ramped columns, jet grouting, etc. In practice, these soil improvement methods are often used in different combinations to achieve the goal (Han 2015; Schaefer et al. 2017). Even though ***GeoTechTools*** is a comprehensive platform, it primarily provides conceptual guidance for soil improvement method selection. The users still need to rely on their own analyses to quantitatively compare different methods to identify the best to fit into their schedule and budget.

In summary, current practice needs a guidance on settlement criteria to ensure the performance of embankments and retaining walls, which should be incorporated into a systematic analysis tool to assist the design.

1.2 OBJECTIVES AND TASKS OF THIS PROJECT

Considering the state-of-the-art of practice, the overarching objective of this study is to establish a settlement criterion to ensure consistent practice and develop an analysis tool to assist the implementation.

To fulfil the objective, the following tasks were performed:

- Conduct a comprehensive literature review on soil improvement methods that mitigate settlements of embankments and retaining walls;
- Perform a survey to collect information about the settlement control practice in state DOTs;
- Develop and calibrate a calculation tool, which can be used to analyze and compare different soil improvement methods; and
- Incorporate cost estimates into the analysis.

Upon the completion of the above-itemized tasks, a detailed user instruction is developed to help users make full use of the tool developed.

CHAPTER 2 LITERATURE REVIEW

2.1 SETTLEMENT MITIGATION METHODS

2.1.1 Preloading with and without wick drain

When soft soil with high ground water table is present, preloading with or without wick drain often is the first method to be considered because it is very effective and inexpensive (Bergado et al. 1996; Bergado and Patawaran 2000). The basic concept of preloading is to reduce void ratio of soft soils through consolidation by applying surcharge for a certain time period and then removing it after reaching a consolidation goal. It includes fill preloading and vacuum preloading, and these two methods can be separately or combined. Preloading is cost effective to modify saturated, low strength and highly compressible cohesive soils, which has been commonly used in highways, airports, land reclamations, storage tanks, and buildings (Han 2015). The advantages of preloading include the following: (1) cost effective and long successful history; and (2) less needed construction space. The main limitation is the long-time waiting period for construction. Many studies reported a reduction of more than 70% post-construction settlement because of preloading (Bo and Choa 2003; Chu et al. 2009).

Even though preloading with wick drain (PVD) can significantly shorten the time for consolidation, it often still consumes considerable amount of construction time. For example, Indraratna et al. (2018) reported a comparison case based on field data, in which preloading with wick drain shortened the primary consolidation time from more than 20 years to a few months. Long et al. (2014) reported a case that 90% primary consolidation was completed within 8 months with wick drains spaced at 3 ft. Similarly, Voottipruex et al. (2014a) compared a case history and claimed that, for a 10 ft high embankment with wick drain spaced at nearly 3 ft, 8-12 months would be needed to complete more than 80% consolidation. The settlement due to preloading can be calculated using the following method:

$$S_{ct} = U_t S_c \tag{2-1}$$

where S_{ct} is the settlement at time t; S_c is the final primary consolidation settlement of the foundation; U_t is the degree of consolidation.

The primary and secondary consolidation settlements are classic problems of soil mechanics, which have matured solutions for preloading without wick drain. Computer programs, such as SAF-1 (Prototype Engineering Inc. 1993) or EMBANK (FHWA 1992), are also available; however, they may not be usable on some computers because they are not compatible with latest OS due to lack of maintenance. Alternatively, customized spreadsheets can be easily developed to meet the needs. In TxDOT sponsored project 0-5530, Vipulanandan et al. (2009) recommended to use compression index (C_c), recompression index (C_r), and coefficient of consolidation (C_v) based on the in-situ stress levels to make the calculation accurate.

Once PVDs (wick drains) are installed, they will induce radial flow to shorten the drainage pathway for water dissipation. As a result, the drainage will include both vertical drain to the top/bottom surface and radial drain to the wick drains, as shown in Figure 2-1. Barron first proposed an analytic solution and then Hansbo improved it by considering the disturbance of wick drainage installation (Barron 1948; Hansbo 1981a). The so-called "Barron-Hansbo's solution" shown in Figure 2-2 has been used by most of the state DOTs to estimate the rate of consolidation for preloading with wick drain. In practice, wick drains are assumed to be flexible and have no stiffness. Therefore, inclusion of wick drains does not change the total consolidation settlement but only changes the rate of consolidation. The existing method to calculate the total consolidation settlement is still valid. If vertical consolidation is also significant compared with radial drain caused by PVDs, the overall consolidation degree should be the combination of radial flow and vertical flow $U_t = 1 - (1 - U_v)(1 - U_h)$ where U_h is the degree of consolidation in the radial direction. Per Santi and Elifrits (2001), the installation rate of PVDs is approximately 60 ft/day.

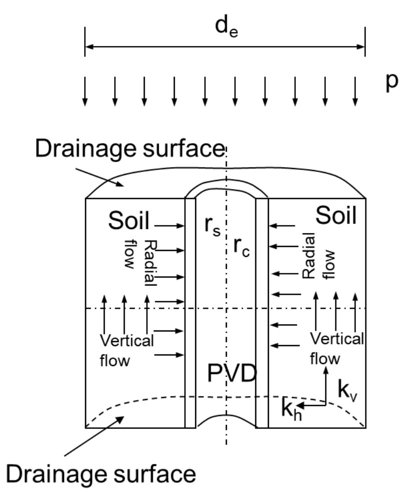


Figure 2-1. Drainage with wick drain.

$$t = \frac{d_e^2}{8c_h}\left[F(n) + F_s + F_r\right]\ln\left[\frac{1}{1-\overline{U_h}}\right]$$

t = Time to achieve design degree of consolidation

$\overline{U_h}$ = Design average degree of horizontal consolidation

d_e = Equivalent well diameter

$F(n)$ = Drain spacing factor

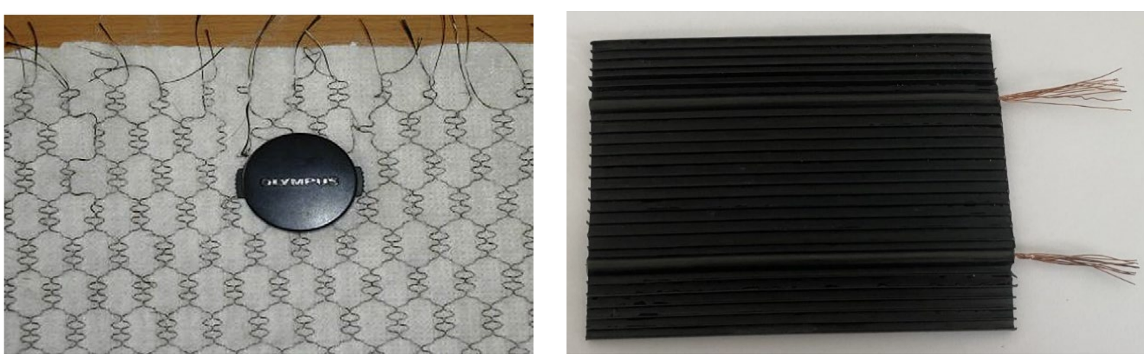
F_s = Soil disturbance factor

F_r = Well resistance factor (typically ignored)

c_h = Horizontal coefficient of consolidation

Figure 2-2. Barron-Hansbo solution.

Without more explanation, the preloading with wick drains can dramatically reduce consolidation time but such time reduction may not be enough for some cases. Therefore, other techniques have to be used jointly to further improve efficiency, such as vacuum preloading and air-boosted preloading, which can further improve preloading efficiency for 10 to 20% (Shen et al. 2015; Yan and Chu 2003). The vacuum preloading also has a limitation to maintain the vacuum pressure to 1.67 ksf and potential tension cracks. In recent years, electro-osmotic dewatering has been incorporated into wick drains to accelerate drainage. Electro-osmosis is a phenomenon that electrical voltage drives water flow in a direction due to the dipolar nature of water molecules. Under the sustained voltage, water is subjected to additional gradient that significantly speeds up water flow (Jones 1996). This concept was not used in preloading until electrically conductive materials were added to wick drains. Jones et al. (2011) simply attached metal wires to the interior of the wick drain filter jackets (sometimes called "filter sleeve") to provide electric conductivity, as shown in Figure 2-3 (a). In contrast, Zhuang et al. (2012) added conductive powder into polymer to make conductive core and also embedded copper wire to boost conductivity as shown in Figure 2-3 (b). In a reported land reclamation project, preloading with conductive wick drain was able to improve consolidation rate by nearly 100% compared with preloading with regular wick drain (Huang et al. 2022). Another advantage of the conductive wick drain is that it can remove water from unsaturated soils.



(a) (b)

Figure 2-3. Electrically conductive filter jacket or core: (a) conductive jacket (Jones et al. 2011), and (b) conductive core (Zhuang et al. 2012).

2.1.2 Deep foundations

Deep foundations are defined as developing resistance at depths greater than 5 times the diameter of the foundation (Department of the Navy 1982). Figure 2-4 shows the typical occasions that deep foundations are used (Hannigan et al. 2016). The allowable settlement is normally limited to the amount required to develop the resistance of deep foundation elements and the group settlement is typically used in engineering practice.

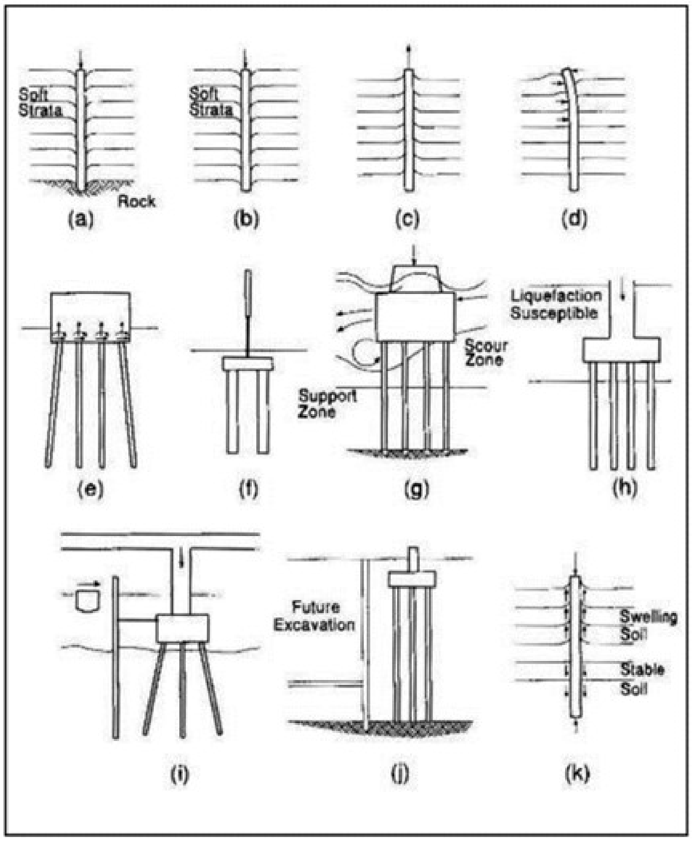


Figure 2-4. *Occasions to use deep foundation (Department of the Navy 1982).*

In most recent two decades, piles are used to cater the need of rapid embankment construction over soft soil (Collin et al. 2005; Han and Gabr 2002). Piles, acting as competent elements, take most of the embankment load with assistance of soil arching and transfer it to deep soil layers, as shown in Figure 2-5 (Filz et al. 2012; van Eekelen et al. 2012). In addition, geosynthetic can be used as a basal reinforcement to bridge over piles to further improve the load transfer efficiency, as shown in Figure 2-6. Under the overburden load over it, the geosynthetic layer is stretched and sagged, developing tension to prevent further downward movement of soil mass over it. With the assistance of geosynthetics, the piles/columns can be further spaced apart to save construction time and budget. Some reported cases increased the column spacing by using multiple geosynthetic layers.

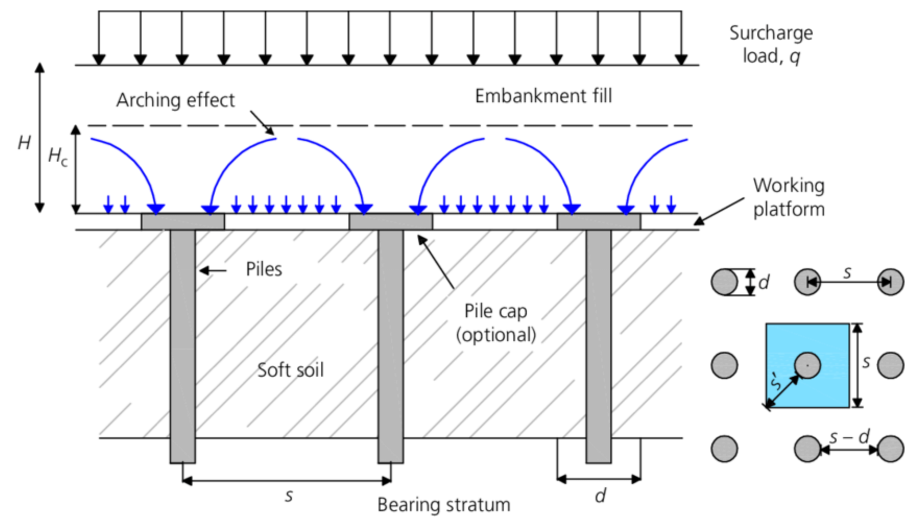


Figure 2-5. Load transfer mechanism of column supported embankment (Fagundes et al. 2015).

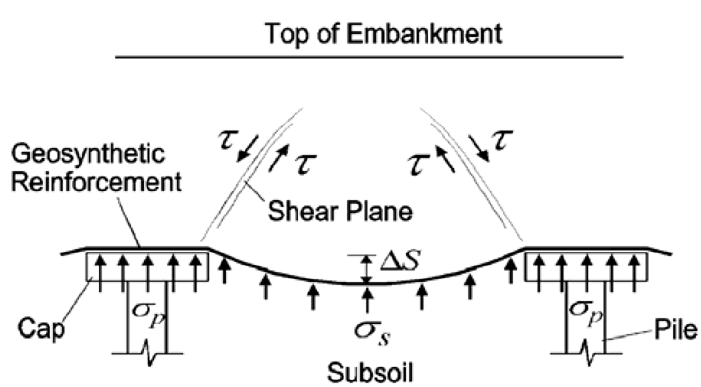


Figure 2-6. Inclusion of geosynthetic layer to further improve load transfer (Ali and Al-Samaraee 2013).

So far, column/pile supported embankment with or without geosynthetic as basal reinforcement have been appeared in construction of many major interstate highways such as 1-95, I-35 and I-7. The outstanding advantage of pile/column supported embankment is that no or very limited waiting time for consolidation occurring is needed because the majority of the load will be carried by piles or columns. Other studies also showed that by including piles/columns, the deep-seated slope failure can be prevented, which is very important for high fill embankments. Collin et al. (2005) claimed that pile/column supported embankments may be the one to solve the time constraints of embankment construction. TxDOT also had a few successful experiences for such project, for example, DFW Connector project in SH 114−121 corridor.

Driven piles are very adaptable and can be installed to accommodate compression, tension, or lateral loads, with specifications set according to the needs of the structure, budget and soil conditions. Driven piles are particularly good in fine-grained soils or soil particles can be displaced. The advantages of using driven piles are: (1) piles can be prefabricated which allows efficient installation; (2) does not need removal of soils; and (3) high lateral and bending resistances. The disadvantages include: (1) heavy equipment and noisy on site; (2) not suitable when soil has poor drainage; and (3) not suitable with adjacent structures. PDCA (2018) reported the unit price of driven pile is $51 per linear foot. Per PDPI (2013), the driven pile support cost is $20/ton-$30/ton which can be calculated using the pile cost divided by the allowable single pile load. WKG[2] (2013) proposed the following formula to predict the driven pile cost:

$$PC = 90.66PL^{-0.402} \tag{2-2}$$

where PC is the pile support cost (dollars/allowable ton); PL is the allowable pile load.

The installation rate of driven piles depends on hammer type, soil type, pile type and size, and groundwater. Caltrans (2015) concluded that the average speed of hammers is 40 to 60 blows per minute which could drive the pile into the ground more than 10 ft.

Drilled shaft foundation is referred to as cast-in-place deep foundation elements which is constructed in drilled holes to place reinforcing steel and concrete. Several other types of deep foundations are also used in transportation projects including micropiles, and continuous flight auger piles which are typically installed in a group to support load. However, drilled shaft is typically in a large size to carry load as a single pile. Drilled shaft foundations are formed by excavating a hole with a typical diameter of 3 to 12 feet and placing concrete and rebar cage. The length of drilled shafts can be up to 200 ft in the U.S. which can be even up to 300 ft or more (Brown et al. 2018). Drilled shafts can be installed in a variety of soil and rock profiles when there is a hard bearing layer. Drilled shafts have the following advantages: (1) easy construction on cohesive soils and most rocks; (2) suitable to a wide range of ground conditions; (3) have high axial load capacity and excellent strength in flexure; and (4) low noise and vibration. However, drilled shafts also have limitations: (1) construction is sensitive to groundwater; (2) load testing is challenging and expensive; (3) single shaft foundation lacks redundancy; and (4) not be efficient for thick soft soils. Mathias and Cribbs (1998) limits the settlement of drilled shafts to 5% of the pile diameter which was based on the research done by Bernal and Reese (1983). The unit cost of drilled shafts varies in a very wide range (Brown et al. 2018).

Micropiles is also known as mini-piles and root piles which are deep foundation elements constructed using high-strength, small-diameter steel casing and/or threaded bars (Keller, https://www.keller-na.com/expertise/techniques/micropiles). Micropiles are widely used to provide structural support to structures, underpin foundations, enhance mass stability, and transfer loads. The micropile casing generally has a diameter in the range of 3 to 10 inches. Typically, the casing is advanced to the design depth using a drilling technique. Reinforcing steel, typically an all-thread bar is inserted into the casing. High-strength cement grout is then pumped into the casing. The casing may extend to the full depth or end above the bond zone with the reinforcing bar extending to the full depth. The advantages of micropiles include: (1) small and lightweight; (2) cost effective; (3) low level of noise and vibration; and (4) convenience of installation. The limitation of micropiles is the high cost compared to other piling method (Sabatini et al. 2005). The settlement of micropile group can also be calculated using the equivalent footing method

Continuous flight auger (CFA) piles are constructed by rotating a hollow stem continuous flight auger into the soil to a designed depth. Concrete or grout is pumped through the hollow stem, maintaining static head pressure, to fill the cylindrical cavity created as the auger is slowly removed (Brown et al. 2007). Typically, a minimum reinforcing steel is placed after the hole is filled with concrete or grout. CFA piles work well in the following types of soils: (1) medium to very stiff clay soils; (2) cemented sand or weak limestones; (3) residual soils; (4) medium dense to sense silty sands and well-graded sands; and (5) rock overlain by stiff or cemented deposits. CFA piles can be problematic in the following types of soils: (1) very soft soils; (2) loose sands or very clean uniformly graded sands under groundwater; (3) geologic formation containing voids, water pools or lenses of very soft soils, and/or flowing water; (4) hard soil or rock overlain by soft soil or loose, granular soil; (5) sand-bearing stratum underlying stiff clay; (6) highly variable ground conditions; (7) conditions requiring penetration of very hard strata; (8) ground conditions requiring uncommonly long piles; and (9) ground condition with deep scour or liquefiable sand layers. CFA has the following advantages: (1) minimal levels of vibration, lower noise; and (2) suitable for tension loads. Limitations of CFA piles include: (1) steel casing is need for underwater construction; (2) issues with removal and disposal of contaminated soils; and (3) the risk of in situ concrete drying out due to the leakage of sand to affect the installation of steel cages.

2.1.3 Lightweight concrete

Lightweight concrete is a mixture made with lightweight coarse aggregates such as shale, clay, or slate, which give it its characteristic low density. Structural lightweight concrete has an in-place density of 90 to 115 lb/ft^3, whereas the density of regular weight concrete ranges from 140 to 150 lb/ft^3. There are three types of lightweight concrete: (1) lightweight aggregate concrete; (2) aerated, cellular foamed or gas concrete; and (3) no-fines concrete. The advantages of lightweight concrete include: (1) minimize the dead load and enhance workability; (2) decrease thermal conductivity; (3) stronger and durable; (4) high resilience to freezing and thawing compared to regular concrete; and (5) adopt industrial wastes. Remund (2017) used cellular concrete as backfill materials and found out that it not only reduced the settlement but also reduced the lateral earth pressure.

The limitations of lightweight concrete include: (1) sensitive to amount of water in the mix; (2) placement and finishing are difficult; (3) the mixing period is longer than regular concrete; and (4) porous and has a low resistance. The application of lightweight concrete is also environmentally friendly, in that the lightweight concrete fill provide 130 yd^3 per delivered load of dry cement which is 10 to 15 yd^3 per load for soil and granular fill. The reduced trucking significantly reduces the greenhouse gas emission and traffic congestion Taylor and Halsted (2021). Lightweight is widely used to treat the bridge approach settlement problem to minimize the differential settlement. The net load design method is typically used for lightweight concrete design which is intended to pre-excavate soils to reach a zero net load to achieve a zero settlement. The lightweight concrete takes about 6 months to dry.

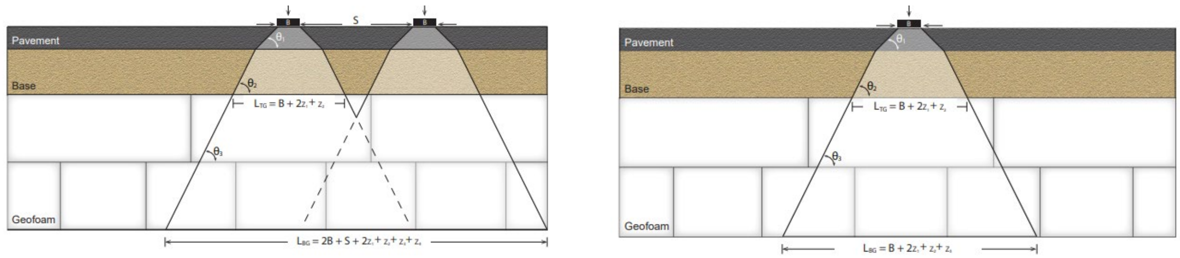
2.1.4 Geofoams

Geofoam has a little bit longer history than others as a lightweight backfill material for embankments and retaining walls. The iconic project, i.e., I-15 reconstruction in Utah, has been widely studied to signify its success. According to various reports, geofoam not only reduced the settlement and improved slope stability but also effectively reduced the stress on utility lines (Bartlett et al. 2001; Bartlett et al. 2000). Compared with other lightweight materials, geofoam has the lowest density, as shown in Table 2-1; therefore, engineers shall consider its applicability at different sites.

Table 2-1. Comparison of different lightweight fill materials

	Cellular concrete	Expanded clay shale	Geofoam
Unit weight (lb/ft^3)	40 – 100	35 - 65	1 - 20
Construction method	Mixing and pumping	Backfilling and compacting	Stacking and alignment

Geofoam is expanded polystyrene (EPS) or extruded polystyrene (XPS) manufactured into large lightweight blocks. The blocks vary in size but are often 6.6 ft × 2.5 ft × 2.5 ft. The primary function of geofoam is to provide a lightweight void fill below a highway, bridge approach, embankment or parking lot. EPS Geofoam minimizes settlement on underground utilities. EPS geofoam is manufactured in various unit weights that typically range from about 0.7 to 2.85 lb/ft^3. Geofoam design loads are recommended to not exceed the compressive resistance at 1% capacity (EPS Industry Alliance 2012). It is suitable for the following conditions: (1) made for project locations with weak soil; (2) perfect for projects with short timelines; (3) lower cost; and (4) provides soil frost protection. ASTM D7180 specifies all the requirements for use of geofoam in geotechnical projects. The advantages of geofoam include: (1) low density with high strength; (2) predictable behavior; (3) not polluting surrounding soil and can be reused; and (4) decreases construction time. The limitations of geofoam are: (1) fire hazards; (2) vulnerable to petroleum solvents; (3) buoyancy; and (4) susceptible to insect damage. The load distribution diagrams in Figure 2-7 should be used to calculate the maximum stress in geofoam caused by live load. The sum of maximum stress caused by the live load and the dead load should be lower than the compressive strength of geofoam at 1% strain. Stark et al. (2004) suggested the total settlement of a geofoam embankment should have five components: immediate settlement of the geofoam; immediate settlement of foundation soil; primary consolidation of foundation soil; secondary consolidation of foundation soil; and creep of geofoam.



(a) Transverse load distribution (b) Longitudinal load distribution

Figure 2-7. Load distribution in geofoam under live load (Stark et al. 2004).

In addition to lightweight concrete and geofoam, other types of lightweight materials have used to reduce embankment settlement, because they result in minimal disturbance to subsurface soil. Puppala et al. (2013) reported using expanded clay shale (ECS) to build SH 360 in Arlington, Texas and estimated the settlement would be reduced by 2/3 compared with the embankment using regular fill materials.

2.1.5 Deep mixing

The deep mixing methods mixes the native soil with hardening agent (for example, cement, lime, and slag) using augers to increase the bearing capacity of soft soils, which include wet and dry methods (Han 2015). The wet method uses the binder in a slurry form while the dry method uses the binder in a dry powder form. Deep mixing method is typically used for soft cohesive soils with the pH value greater than 5 and organic content less than 6% (Elias et al. 2006). The treatment depth can reach up to 200 ft. It widely used to support superstructures, containment remediation, and liquefaction mitigation (Han et al. 2007). The advantages of deep mixing include: (1) applicable to most soil types; (2) fast installation and low noise level; (3) can be used as water barrier and earth retaining at the same time; and (4) less spoil soil. But deep mixing also has the following limitations: (1) high transportation cost; (2) not guaranteed quality; and (3) no quality control standards.

2.1.6 Aggregate columns

Aggregate columns are columns of compacted stone installed in groups in poor soil to increase bearing pressure and mitigate settlement under structural footings. Aggregated columns are suitable for cohesive soils with undrained shear strength higher than 300 psf (SPT N = 3) (Han 2015). This technology has been successfully used for buildings, industrial facilities, storage tanks, embankments, bridge abutments, roadway widening and utilities and pipelines. The advantages of aggregate columns include: (1) long history of successful record; (2) fast installation and cost effective; and (3) could provide short drainage path to accelerate the consolidation. The limitations of aggregate columns are: (1) not suitable for very soft soils (undrained shear strength lower than 300 psf) due to bulging at low confining pressure near ground surface; (2) rammed aggerate columns cannot be installed in clean sands with a high groundwater table with a potential of liquefaction; and (3) installation is difficult in rocks and boulders.

There are three settlement calculation methods for aggregate columns including (Han 2010): (1) stress reduction method (Aboshi 1979); (2) improvement factor method (Priebe 1995); (3) elastic-plastic method (Castro and Sagaseta 2009; Pulko and Majes 2005), and (4) equivalent modulus method (Horikoshi and Randolph 1999; Poulos 2001), all of which will be discussed in detail in CHAPTER 4.

The determination of consolidation rate is more challenging as rigid elements (aggregate columns, deep mixing) will inevitably accelerate the rate of consolidation due to the stress transfer (Han and Ye 2001). When an embankment is built, excessive pore water pressure is generated. As time goes by, pore water pressure dissipates gradually due to drainage, which makes soil settle more than the rigid inclusions. The differential settlement between soils and rigid inclusions induces soil arching that transfers load from soils to rigid inclusions, which further reduces excessive pore wate pressure in soil. Such phenomenon is called “stress-transfer-induced pore water dissipation”, which depends on the stiffness ratio between the soil and the rigid inclusion and can be very significant. As a result, regardless types of rigid elements, the key issue is to determine the stress redistribution so that the consolidation rate can be calculated and, consequently, the post-construction settlement can be accurately estimated.

2.1.7 Summary

Above-discussion has made it clear that each soil improvement method has its advantages and disadvantages. For example, preloading is cheaper but needs longer waiting time while pile/column-support can be faster but is more costly. Therefore, engineers need to balance construction time and cost requirement to select the most suitable one to meet the settlement. Farnsworth et al. (2008) manifested such conclusion by comparing MSE wall constructed over soft lacustrine soils. Three different methods were used at different segments for the project: (1) one-stage MSE wall supported by lime-cement columns; (2) geofoam embankment with tilt-up panel fascia walls; and (3) two-stage MSE wall with wick drain and preloading. It was found out that using geofoam was the fastest one, while preloading with wick drain was the cheapest one but had the longest construction time. Lime-cement column was the most expensive one and did not perform well. The data presented by Farnsworth et al. (2008) also enlightens us that combined use of different soil improvement methods may be the approach to achieve balanced cost and construction time. Ye et al. (2006) combined deep soil mixing columns and preloading with wick drain to assist embankment construction over soft soil. With preloading and wick drain, the length of deep soil mixing columns was able to reduce by half. And with the inclusion of deep soil mixing column, the construction time was limited to less than one year.

2.2 SETTLEMENT DESIGN CRITERIA

Determining the allowable settlement for embankments and retaining walls is not an easy task as it needs to consider the structure it supports as well as the structures it is adjacent to. Kelly et al. (2015) described the settlement criteria used in Australia, in which an embankment was divided into 5 zones, i.e., Zones 1A, 1B, 2A, 2B, and 3, as shown in Figure 2-8 and post-construction settlement criteria were set for each zone:

- Zone 1: post-construction settlement limit of 2 inches in 40 years,
- Zone 2: as a transition zone, the settlement shall be meet the requirement of 0.5% maximum change in grade between Zones 1 and 3, and
- Zone 3: allow to deform up to 8 inches prior to intervention to reconstruct the pavement back to level.

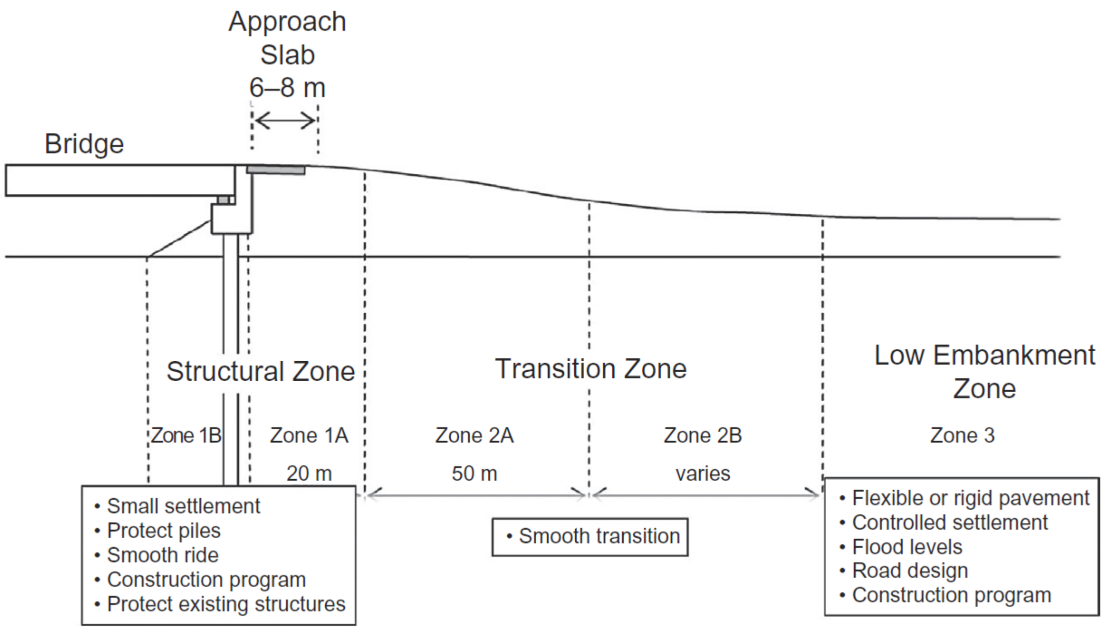


Figure 2-8. Embankment zoning to control settlement (Kelly et al. 2015).

Similar approach was adopted by Canada as described in a case history by Sangiuliano et al. (2016), in which the embankment was divided into 4 zones based on its distance from a bridge abutment and allowable settlement for each zone was specified in Table 2-2. The settlement criteria of Australia and Canada is consistent with the claim of many engineers that different soil improvement methods or setups could be considered according to its distance from a bridge abutment to achieve the best outcome, as shown in Figure 2-9.

Table 2-2. Embankment zoning and allow settlements (summarized from Sangiuliano et al. (2016))

	Zone 1	Zone 2	Zone 3	Zone 4
Distance from abutment (ft)	0 – 65	65 – 165	165 – 250	> 250
Allowable total settlement (inches)	1	2	4	8
Allowable differential settlement (inches)	0.6	1	1.6	1.6

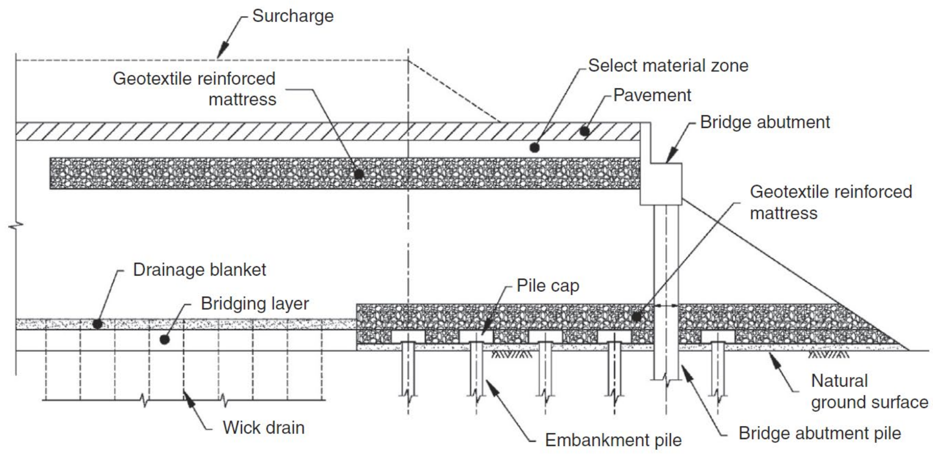


Figure 2-9. Selection of soil improvement methods based on zoning (Hsi and Martin 2015).

However, there is no widely accepted zoning and settlement criteria in the U.S. Some state DOTs may have rough criteria used for preliminary soil improvement method screening. For example, LADOTD uses the post-construction settlement at 6 months as a threshold to decide the improvement methods:

- < 1 inch, use a geosynthetic load transfer platform or lengthen the bridge. No other measured is needed; and
- > 1 inch, use preloading or wick drain or combination of preloading and wick drain.

However, AASHTO and FHWA specify the allowable settlement for MSE walls (AASHTO 2016; Berg et al. 2007), as listed in Table 2-3. For square panel, if the joint width is less than ¾ inch, the allowable differential settlement is stricter to preclude panel cracking; for example, it should be less than 1/200 for ½ inch joint width and it should be less than 1/300 for ¼ inch joint width.

Table 2-3. Allowable settlements of MSE wall

	Square panel (¾-inch joint width)		Full height panels	Drycast facing	Welded wire facings
	Area < 30 ft^2	30 ft^2 <Area < 75 ft^2			
Allowable differential settlement	1/100	1/200	1/500	1/200	1/50

CHAPTER 3 SURVEY OF STATE DOTS PRACTICE

Since there are no settlement criteria for embankments and retaining walls by TxDOT, a survey was conducted to collect information about embankment and retaining wall settlement criteria used by other State DOTs. The survey form is included in Appendix – A: SURVEY FORM. The survey is intended to collect the zoning and settlement criteria used for flexible and rigid pavements and bridge approach. The survey was first distributed through AASHTO representative of TxDOT to other state DOTs in late October 2022 or early November 2022. And then we reached out to the DOTs that did not respond to the survey to collect their responses. Most of the survey data was documented in the survey form except Alabama and Florida which were interviewed through phone.

3.1 SURVEY RESULT SUMMARY

3.1.1 Responding summary

Through various channels, we were able to collect survey from 22 state DOTs as indicated in Table 3-1 and Figure 3-1. The geographic distribution of the responding states is shown in Figure 3-2.

Table 3-1. Summary of responding state DOTs (green cell indicates the state DOT responded to the survey)

Responding state		No responding state	
AL|Alabama	LA|Louisiana	AZ|Arizona	NV|Nevada
AK|Alaska	MT|Montana	DE|Delaware	NH|New Hampshire
AR|Arkansas	NE|Nebraska	HI|Hawaii	NM|New Mexico
CA|California	NJ|New Jersey	ID|Idaho	NC|North Carolina
CO|Colorado	NY|New York	IL|Illinois	OR|Oregon
CT|Connecticut	ND|North Dakota	KY|Kentucky	PA|Pennsylvania
FL|Florida	OH|Ohio	ME|Maine	RI|Rhode Island

GA|Georgia	OK|Oklahoma	MD|Maryland	TX|Texas
IN|Indiana	SC|South Carolina	MA|Massachusetts	UT|Utah
IA|Iowa	SD|South Dakota	MI|Michigan	VT|Vermont
KS|Kansas	TN|Tennessee	MN|Minnesota	WA|Washington
	VA|Virginia	MS|Mississippi	WV|West Virginia
		MO|Missouri	WI|Wisconsin
			WY|Wyoming

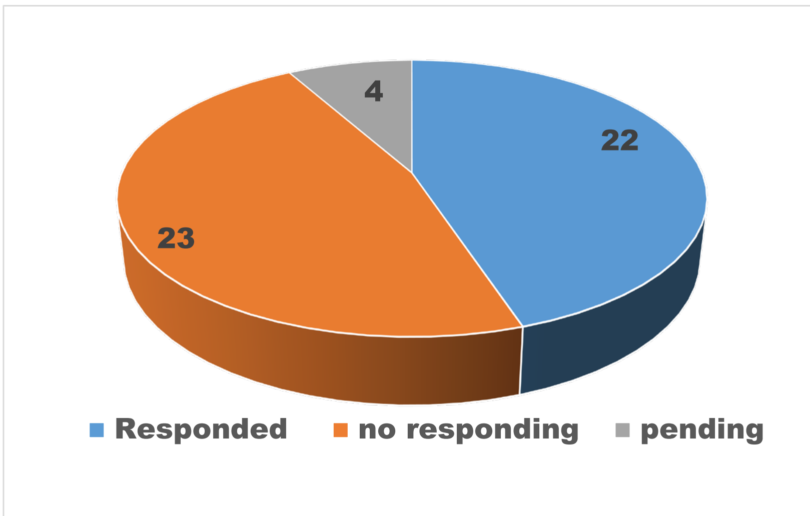


Figure 3-1. Survey responding summary.

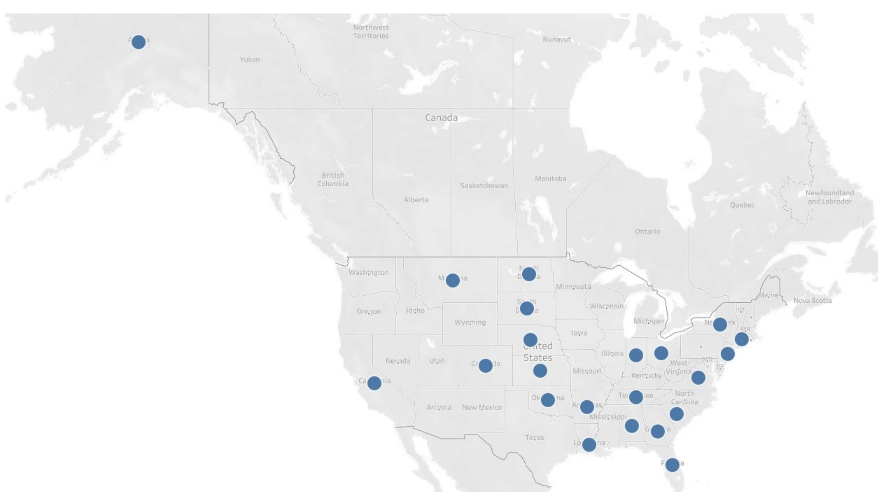


Figure 3-2. Responding state distribution.

3.1.2 Synopsis of the responses from each responding DOT

The responses of each responding state DOT are listed herein. Most of the responses directly reflect the information provided in the survey form; however, for some DOTs, we followed up with the responders to obtain further clarifications. The data presented in this section reflect our best understanding of the responses, which are summarized in terms of zoning and settlement criteria.

Alaska

- Three zones: Zone 1 – 0 – 20 ft; Zone II – 20 – 50 ft; Zone III - > 50 ft
- Settlement criteria: 1 inch for all zones

Alabama

- Single zone/no zoning: Zone I – within 50 ft of a bridge
- Settlement criteria: Zone I – 0.5 inches

Arkansas

- Single zone/no zoning
- Settlement criteria: 2 – 3 inches of total settlement (the differential settlement between a bridge abutment and the embankment within 100 feet of the bridge abutment to be less than 2 inches)

California

- Two zones: Zone 1 – approach slab length; Zone II – the rest
- Settlement criteria: Zone I – 1 inch; Zone II – no requirement

Colorado

- No zoning
- Settlement criteria: no statewide criteria and evaluate project by project; but definitely no more than 2 inches

Connecticut

- Two zones: Zone 1 – 0 – 16 ft (typical approach slab length); Zone II – 16 – 100 ft;
- Settlement criteria: Zone I – 1 inch; Zone II – 2 inches

Florida

- Single zone/no zoning
- Settlement criteria: no written criteria for the settlement. But in general, it is required that the settlement of a bridge approach is compatible with the bridge. That means the settlement of the bridge approach embankment settlement should be nearly the same with that of the bridge

Georgia

- No zoning
- Settlement criteria: no statewide criteria and evaluate project by project

Indiana

- Single zone
- Settlement criteria: 1 inch

Iowa

- Two zones: Zone 1 – close to abutment; Zone II – the rest
- Settlement criteria: Zone I – 0.4 inch; Zone II – 1 inch

Kansas

- Two zones: Zone I – within 100 ft of the bridge abutment; Zone II – the remaining length of embankment
- Settlement criteria: Zone I – 0.5 inches; Zone II – no requirement

Louisiana

- Single zone/no zoning
- Settlement criteria: 1 inch of post construction embankment settlement. The differential settlement is limited to 1/300 both within the fill and across fill/structure interfaces.

Montana

- Single zone/no zoning
- Settlement criteria: no criteria

Nebraska

- Two zones: Zone I – 0 – 21 ft, the length of any part of the structure (approach slab, wing walls, etc.); Zone II – the remaining
- Settlement criteria: Zone I – 0; Zone II – 0.5 inch

New Jersey

- Single zone/no zoning
- Settlement criteria: AASHTO settlement criteria, i.e., the embankment needs to settle compatibly with the bridge

New York

- Single zones
- Settlement criteria: the design focuses on minimizing differential settlement of the approach slab. But there is no general settlement criteria and allowable settlement in general is project-specific and needs to consider traffic staging, schedule, available means and methods. But as to design-built projects, the entire project to be limited to 1"/100' longitudinally and 0.5"/10' transversely over 50 years post-construction.

North Dakota

- Single zone
- Settlement criteria: 1 inch.

Ohio

- Two zones: Zone I – the length of any part of the structure (approach slab, wing walls, etc.); Zone II – the remaining length of approach embankment
- Settlement criteria: Zone I – the embankment needs to settle together with the bridge, which depends on span and approach slab lengths; Zone II - final grading

and paving is held until settlement is "complete" (90% total settlement), or less than 1/8" of settlement is measured between two readings taken one-week apart.

Oklahoma

- Two zones: Zone I – the length of approach slab; Zone II – the remaining length of approach embankment
- Settlement criteria: do not have written criteria but in general 1 inch is used

South Carolina

- More than four zone: Zone I – from bridge to the end of the approach slab; Zones II, III, … - every 500 feet or less depending on soil conditions
- Settlement criteria: Zone I – 0.05*Lslab (note: Lslab is slab length in feet and the amount of settlement determined is in inches); Zone II – vary depending on soil condition

South Dakota

- Single zone/no zoning
- Settlement criteria: no criteria

Tennessee

- Single zone
- Settlement criteria: 1 inch

Virginia

- Two zones: Zone I – 100 ft from bridge abutment; Zone II – the remaining length of approach embankment
- Settlement criteria: Zone – I: 1 inch; Zone – II: 2 inches.

3.2 CONCLUSIONS

The survey data show that the practice of the state DOTs varies significantly. The key findings of the survey are:

- Georgia, South Dakota and Montana do not have settlement criteria. The remaining 19 states have some criteria even though the criteria may not appear in any written document, for example, Arkansas, Oklahoma.
- Florida, New Jersey and Ohio do not have a fixed value for allowable settlement but require the embankment settlement to be compatible with the bridge settlement so that to control the differential settlement between bridge and approach.
- Among the ten states having zoning, six use the approach slab as Zone 1 and the remaining four use the distance from the bridge to define their Zone 1.
- Among the ten states having specific settlement criteria, 0.5 inches or 1 inch is used for Zone 1 expect for South Carolina which uses 0.05*Lslab (note: Lslab is slab length in feet and the amount of settlement determined is in inches) to determine allowable settlement.

CHAPTER 4 CREATING TIME-COST CHARTS FOR DIFFERENT SOIL IMPROVEMENT METHODS

4.1 INTRODUCTION

Embankment is an essential earth work to support roadways and other transportation infrastructures. Its design, construction, and maintenance have direct impacts on the serviceability of roadways and bridges. Thus, the second Strategic Highway Research Program Project Number R02 (SHRP 2 R02) - *Geotechnical Solutions for Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform,* developed an online soil improvement method selection tool to facilitate the transportation engineering community, which is called “GeoTechTools” (Schaefer and Berg 2012). This online selection tools initially encompassed 47 soil improvement methods and have been experienced a few modifications including combining similar methods and adding new methods. Based on the field applications, the following methods are among the most frequently used ones to deal with soft soils (Han 2015; Schaefer et al. 2017):

- Preloading with wick drain,
- Lightweight fill,
- Deep compaction,
- Aggregate columns,
- Column/pile,
- Deep mixing, and
- Rigid inclusion.

Due to the grout-ability issue, most of the grouting technology cannot be used in soft clay except for jet grouting. In recent years, rigid inclusion has gained significant attention as it provides a most robust improvement method compared to existing ones (Larisch et al. 2015). In general, rigid inclusion refers to the stiff inclusion into the soil, which has a stiffness 100 – 1,000 times higher than the surrounding natural ground. In such a definition, many soil improvement may be classified as rigid inclusion, such as deep soil mixing, vibro concrete columns, jet grouting, etc. However, to eliminate the vagueness, rigid inclusion nowadays primarily refers to drilled displacement concrete columns. According to the recent data, the advantage of rigid inclusions is its quick installation and good settlement reduction; however, it is generally more expensive than other methods. Therefore, it may become an appealing alternative if the soft soil goes beyond 35 ft deep or the embankment exceeds 20 ft wide. Many innovative developments (shown in Figure 4-1) in drilling tools have incentivized the application of rigid inclusions worldwide. Rather than piling, rigid inclusion typically works together with a thick load transfer platform to assist distributing the load more effectively among soil and rigid inclusion as shown in Figure 4-2 (Cacciola et al. 2018). In contrast, the pile is designed to carry 100% of the load.

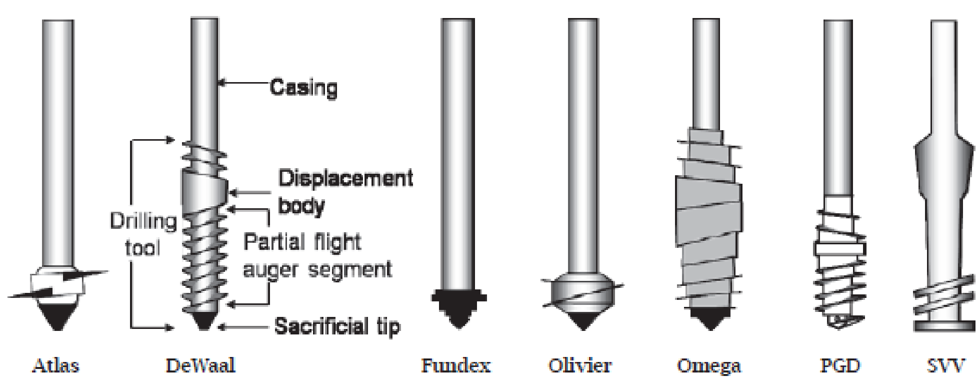


Figure 4-1. Drilling tools for rigid inclusions (Basu et al. 2010).

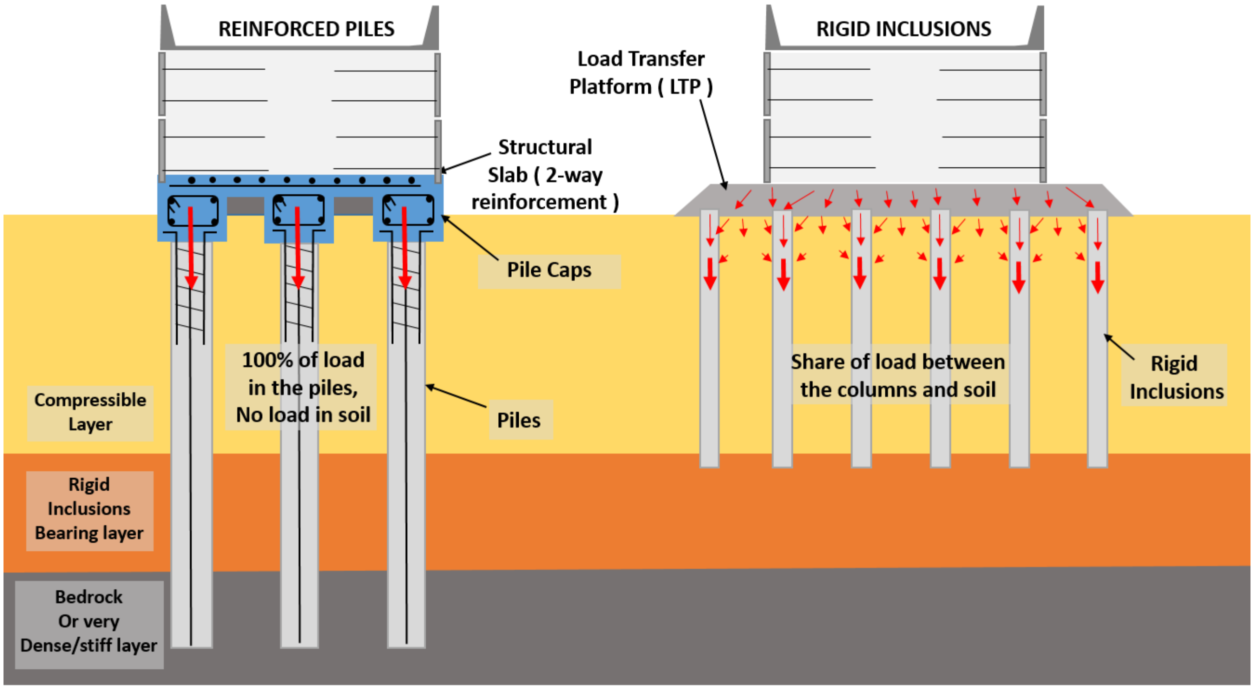


Figure 4-2. Rigid inclusion vs. piling to support embankment (Siddiqui et al. 2017).

Considering the state-of-the-art of practice, this project focused on preloading with and without wick drains, lightweight fill, stone columns, rammed aggregate columns, deep mixing and piles. The design methods for these soil improvements are summarized hereafter, which are used to develop time-cost charts for preliminary method selection.

4.2 SETTLEMENT ESTIMATION OF DIFFERENT SOIL IMPROVEMENT METHODS

4.2.1 Preloading with and without wick drain

Preloading with or without wick drain is one of the most effective and most commonly used method to reduce post-construction settlement of embankments (Bergado et al. 1996; Bergado and Patawaran 2000). The fundamental concept of preloading is to remove water from inter-particle voids by applying surcharge for a certain period and then removing it after reaching a consolidation goal. The inclusion of wick drain (PVD), which is usually spaced at 3 – 6 ft, can shorten the consolidation time from years to months

(Indraratna et al. 2018; Long et al. 2014). The settlement due to preloading can be calculated using the following method:

$$S_{ct} = US_c \tag{4-1}$$

where S_{ct} is the settlement at time t; S_c is the total primary consolidation settlement of the foundation; U is the degree of consolidation.

For the preloading without wick drain, the degree of consolidation is calculated based on Terzaghi's one-dimensional consolidation theory. The degree of consolidation (U) is calculated as:

$$T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2,\ U = 0 - 60\%$$
$$T_v = 1.781 - 0.933\log(100 - u\%),\ U > 60\% \tag{4-2}$$

where $T_v = \frac{c_{vt}}{H_{dr}^2}$; C_v is the coefficient of consolidation; H_{dr} is the longest drainage distance, if the clay layer is two-way drainage, H_{dr} equals half of the clay layer thickness; t is the consolidation time.

Once PVDs (wick drains) are installed, they will induce radial flows to shorten the drainage pathway for water dissipation. In these situations, the dominant drain is in radial direction. Barron first proposed an analytic solution and then Hansbo improved it by considering the disturbance of wick drainage installation (Barron 1948; Hansbo 1981a), which is "Barron-Hansbo's solution" and has been used widely to estimate the rate of consolidation for preloading with wick drains, as shown in Eq. 4-3.

$$t = \frac{d_w^2}{8c_h}[F(n) + F_s + F_r]\ln\left[\frac{1}{1-U}\right] \tag{4-3}$$

where U is the degree of consolidation; d_w is the equivalent well diameter=2(a+b)/π; a and b are the wick drain width and thickness, respectively; F(n) is the drain spacing factor; F_s is the soil disturbance factor; F_r is the well resistance factor (typically ignored); C_h is the horizontal coefficient of consolidation.

If vertical consolidation is also significant compared with radial drain caused by PVDs, the overall degree of consolidation should be the combination of radial flow and vertical flow as shown in Eq. 4-4 below:

$$U_t = 1 - (1 - U_v)(1 - U_h) \tag{4-4}$$

where U_v and U_h are the degree of consolidation in the vertical and radial directions, respectively.

4.2.2 Lightweight fill

Lightweight fill usually has a unit weight that is a fraction of regular backfill materials such as crushed stone or soil. Thus, the overburden stress of subgrade soil can be significantly reduced if lightweight materials are used to replace regular backfill materials. According to existing data, cellular concrete, geofoam and expanded clay shale have been successfully used for embankment applications (Bartlett et al. 2001; Bartlett et al. 2000; Puppala et al. 2013; Stark et al. 2004; Taylor and Halsted 2021). Table 4-1 lists the unit weights of the three different lightweight fill materials. Geofoam's unit weight can be as low as 1% of soil, making it most effective to reduce over burden stress. The existing consolidation theory is still applicable to calculate the settlements.

Table 4-1. Comparison of different lightweight fill materials

	Cellular concrete	Expanded clay shale	Geofoam
Unit weight (lb/ft^3)	40 – 100	35 - 65	1 - 20
Construction method	Mixing and pumping	Backfilling and compacting	Stacking and alignment

4.2.3 Deep mixing column

The stress reduction method can be used to calculate the settlement for end-bearing deep-mixing columns:

$$S' = \frac{S}{1 + a_s(n-1)} \tag{4-5}$$

where S' is the settlement of soil after treated by deep mixing; S is the settlement of soil without treatment; a_s is the area replacement ratio of deep mixing columns; and n is the stress concentration ratio which can be determined based on the modulus ratio of deep mixing column to the soil (E_c/E_s) as shown in Figure 4-3 (Jiang et al. 2013b). It is suggested by Han (2015) that it should be cautious to use a stress concentration ratio greater than 20 for deep mixing columns.

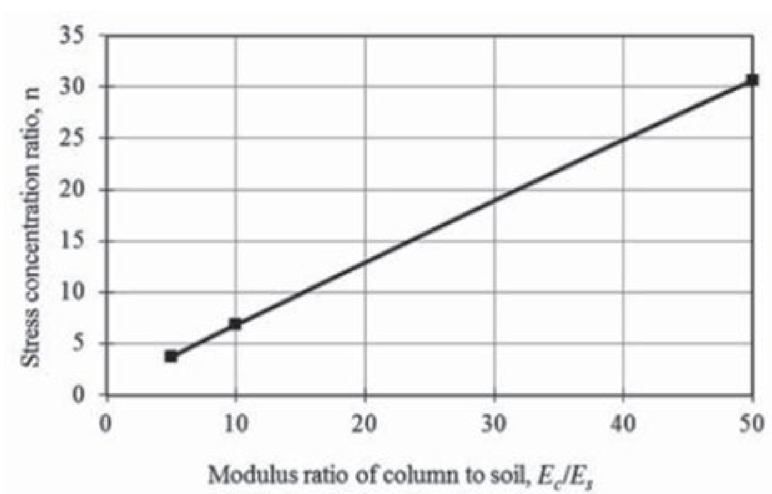


Figure 4-3. Stress concentration ratio vs. modulus ratio of column to soil.

Han et al. (2009) suggested that the piled-raft method could be used to calculate the settlement of deep mixing treat soft foundations. This method converts the piled raft foundation into equivalent pier to calculate the piled raft settlement, as shown in Figure 4-4. This approach can be used for other rigid inclusions, such as aggregate columns and sand columns.

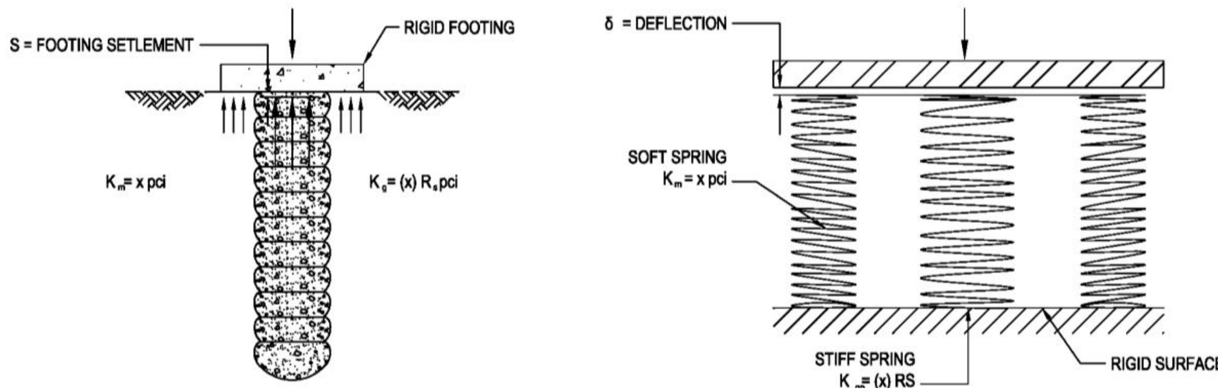


Figure 4-4. Equivalent composite foundation for settlement calculation (Sangiuliano et al. 2016).

4.2.4 Aggregate columns

4.2.4.1 General design concepts for settlement reduction

The aggregate columns mainly consist of rammed aggregate piers and stone columns (Schaefer et al. 2017). The installation of aggregate columns introduces more competent elements to support an embankment. Based on the scenario, three different settlement calculation models were proposed: (1) stress reduction factor model (Aboshi 1979); (2) improvement factor model (Priebe 1995); (3) elastic-plastic method (Han 2015), and (4) equivalent pier model (Castro and Sagaseta 2009; Pulko and Majes 2005).

Stress reduction method: Since the aggregate column has a much higher stiffness, it will share more load from compressible soil. Through various experimental data, Barksdale and Bachus (1983) proposed that stress concentration ratio is approximately linear to the modulus ratio of column to soil, as shown in Figure 4-5. (Han 2010) independently developed a theoretical relationship between modulus ratio and stress concentration ratio, as shown in Eq. 4-6, which is consistent with what was proposed by Barksdale and Bachus (1983). Furthermore, Han (2015) suggested the stress concentration ratio (n) should be limited to 5, concerning the bulging of stone columns, which is also marked in Figure 4-5.

$$n = 1 + 0.217(\frac{E_c}{E_s} - 1) \tag{4-6}$$

where E_c and E_s are the elastic modulus of column and soil, respectively.

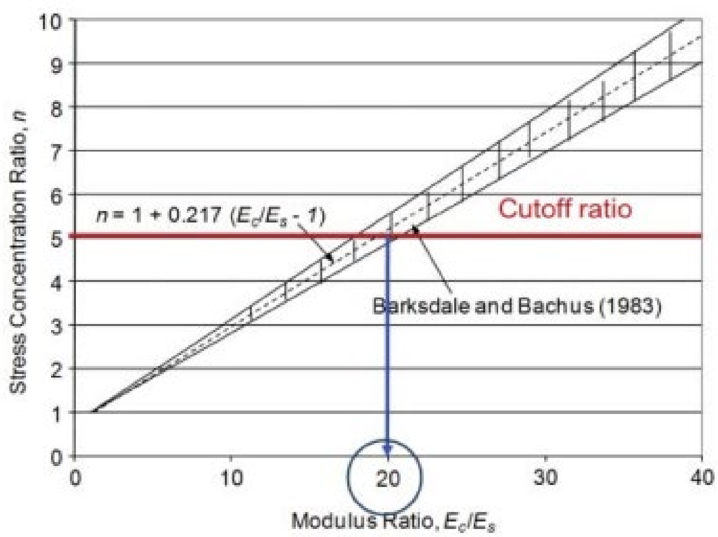


Figure 4-5. Stress concentration ratio vs. modulus ratio E_c/E_s.

With the determined stress concentration ratio, the settlement of aggregate column-soil composite can be derived from Eq. 4-5.

Even though the concept of stress reduction method is simple, the estimation of stress concentration relies on accurate estimation of soil and column modulus, which often are not available in the soil report. The vast majority of field data indicated that stress concentration ratio of stone columns varies from 2 to 5; as a result, the design usually uses 3 or 4 (Schaefer et al. 2017). As to rammed aggregate piers, the field often suggested a range of 5 to 10 and design often used 6. GeoPier claimed a stress concentration ratio higher than 20 (Schaefer et al. 2017).

Improvement factor method: Priebe (1995) suggested that the settlement with columns is a fraction of settlement without columns as indicated in Eq. 4-7. He further provided two approaches to define the improvement factor (I_f): (1) it can be calculated as a function of area replacement ratio and friction angle, as shown in Eq. 4-8; (2) it can be determined from a design chart, as shown in Figure 4-6.

$$S' = \frac{S}{I_f} \tag{4-7}$$

$$I_f = 1 + a_s\left[\frac{5-a_s}{4(1-a_s)\,tan^2\left(45^o-\frac{\phi}{2}\right)} - 1\right] \tag{4-8}$$

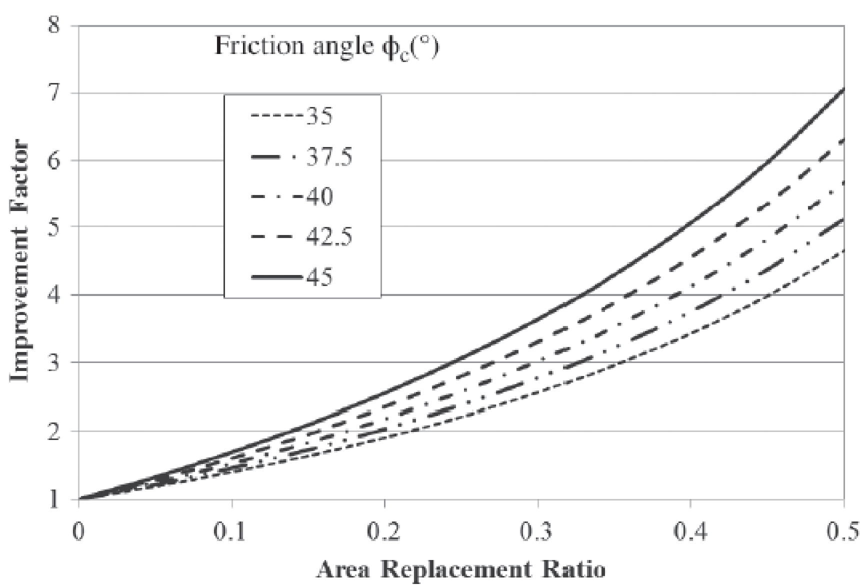


Figure 4-6. Improvement factor (I_f) vs. area replacement ratio (a_s).

Elastic-Plastic Method: This method is based on elastic-plastic constitutive models (Han 2015). The soft soil is assumed to be linearly elastic while aggregate columns are assumed to be linearly elastic-perfectly plastic following the Mohr-Coulomb failure criterion with a constant dilatancy angle. The plasticity starts with the upper portion of the column and extends to the whole length of the column. This method is too complicated, so it is not often used in practice.

Equivalent modulus method: Considering equal deformation situation under a raft, Horikoshi and Randolph (1999) and Poulos (2001) deemed that the columns and soil can be equivalent to a pier, as shown in Figure 4-7. The equivalent modulus of the pier is the area weighted average of soil and columns, as shown in Eq. 4-9. In reality, equivalent modulus method is a simpler format of equivalent composite foundation concept, as it assumes the materials are linear elastic, which makes it more suitable for concrete columns than aggregate columns.

$$E_{eq} = E_s + (E_c - E_s)\frac{A_{tc}}{A_f} \tag{4-9}$$

where E_{eq}, E_c, and E_s are the equivalent, column, and soil modulus, respectively; A_{tc} is the summation of cross-section areas of all individual columns; A_g is the total area of columns and soil.

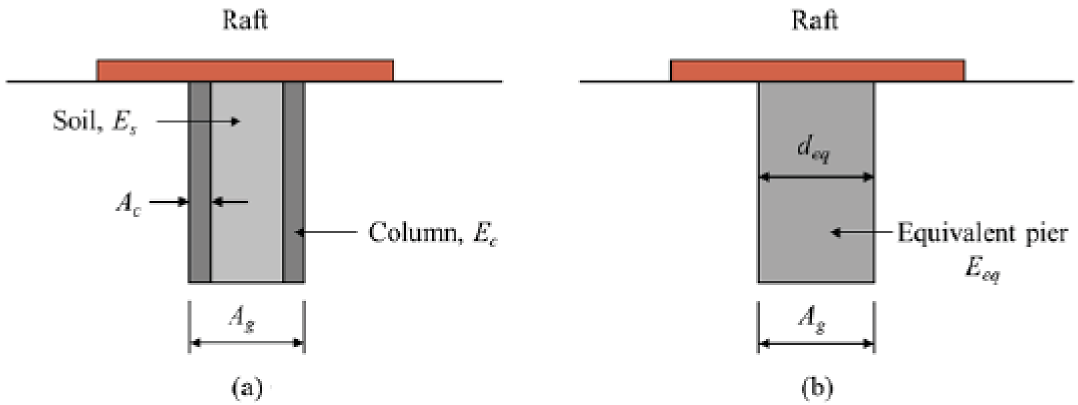


Figure 4-7. Equivalent pier: (a) soil-column composite; and (b) equivalent pier (Han 2015)

For rammed aggregate columns, instead of equivalent modulus, stiffness modulus is used to calculate the settlement. That rammed piers could be treated as elastic springs, and the settlement can simply be calculated from the compression of the rammed piers, as shown in Eq. 4-10.

$$S' = \frac{q_c}{K_g} \tag{4-10}$$

where q_c is the load on rammed pier, which is calculated from stress concentration ratio and area replacement ratio; k_g is the rammed pier stiffness modulus. Rammed aggregate pier stiffness modulus ranges from 75 to 360 pci for support of rigid footings and should be lower for embankment applications. It is common to use 65 pci for embankments.

Among these methods discussed, the stress reduction ratio and improvement factor methods have been widely used and well received by the practice (Han 2015; McCabe and Egan 2010).

Consolidation of aggregate columns: A unique feature of aggregate columns is that they are highly permeable and can serve as a similar function of wick drain (Han 2010; Han and Ye 2001). When an embankment is built, excess pore water pressure is generated. As time goes by, pore water pressure dissipates gradually due to drainage, which makes soil settle more than the columns. The differential settlement between soils and columns induces soil arching that transfers load from soils to rigid inclusions, which further reduces excess pore water pressure in soil. Such phenomenon is called "stress-transfer-induced pore water dissipation", which depends on the stiffness ratio between the soil and the columns and can be very significant. Considering the deformation compatibility, Han and Ye (2001) derived a closed-form solution to estimate the consolidation due to inclusion of stone columns. The solution accounted for both vertical and radial drain based on Terzaghi's 1D and Barron's radial consolidation. Thus, it can use existing Terzaghi and Barron solutions to calculate U_v and U_h, respectively, and then use Eq. 4-4 to calculate overall consolidation expect that the coefficient of consolidation needs to consider stress concentration ratio. The proposed formulate to calculate coefficient of consolidation in vertical and radical directions as shown in Eqs. 4-11 and 4-12.

$$c_r' = c_r(1 + n\frac{1}{N^2-1}) \tag{4-11}$$

$$c_v' = c_v(1 + n\frac{1}{N^2-1}) \tag{4-12}$$

where N is the diameter ratio, i.e., the ratio of column diameter to equivalent tributary area diameter, C_r and C_v are the radical and vertical coefficient of soil, respectively.

A design chart was developed to assist the estimation of consolidation associated with stone columns, as shown in Figure 4-8.

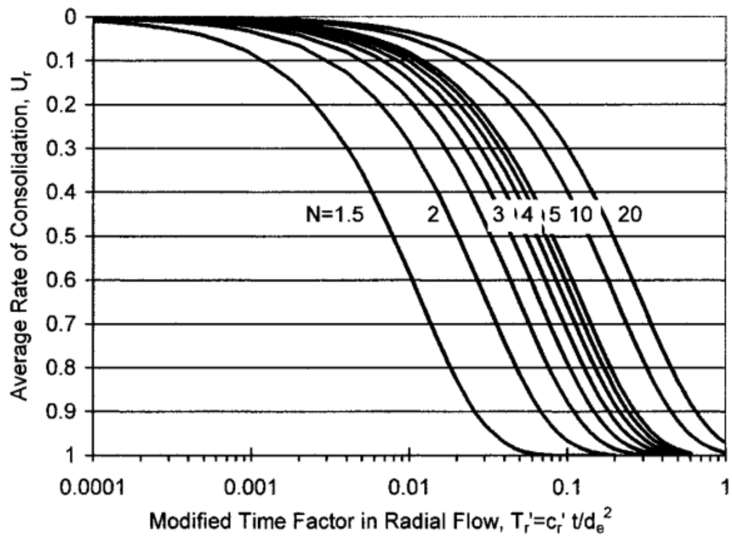


Figure 4-8. Rate of consolidation in radial flow for stone columns (Han and Ye 2001).

4.2.4.2 Design considerations

Even though the materials used for rammed piers and stone columns are similar, their properties are different due to different construction methods used to build them. Stone columns are constructed by down-hole vibratory methods. Basically, vibro forces are used to create a hole and then backfill materials are introduced into the hole to form a dense, tightly interlocked mass of aggregate, which is significantly stiffer than surrounding soil (Barksdale 1987; Barksdale and Bachus 1983). In contrast, rammed aggregate piers are installed by drilling 18- to 36-inch diameter holes into the subsurface soils and ramming lifts of well-graded aggregate within the holes to form stiff, high-density aggregate columns. Due to the construction differences, rammed columns are stiffer than stone columns. However, their design procedures (shown in Figure 4-9) are similar except for the selection of stress concentration ratio according to their stiffness. The first design parameter to be selected is the area replacement ratio, which is defined as the ratio of the cross-section of a column to the tributary area of the column. With a selected area replacement ratio and stress concentration ratio, the settlement can be calculated according to Eq. 4-5 if the stress reduction ratio method is used. If the settlement meets the requirements, the column diameter and spacing can be calculated according to Eq. 4-13.

$$a_s = \frac{A_c}{A_e} = C(\frac{d_c}{s})^2 \tag{4-13}$$

where a_s is the area replacement ratio; A_c is the cross-section area of the column; A_e is the tributary area of the column; d_c is the diameter of a column, s is the center-to-center spacing between columns, C is the pattern constant, depending on the arrangement pattern of the columns.

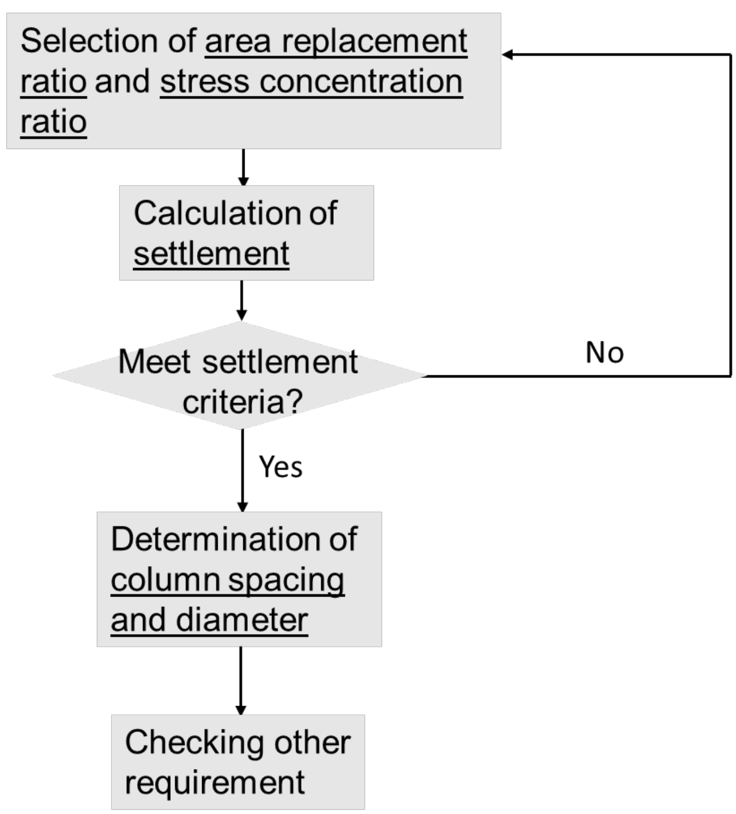


Figure 4-9. Aggregate column design flow chart

4.2.5 Rigid inclusions

As previously discussed, rigid inclusion in general refers to the stiff inclusion into the soil, which has a stiffness 100 – 1,000 times higher than the surrounding soil. Therefore, under this broad definition, rigid inclusion should include deep soil mixing, vibro concrete columns, jet grouting, and various piles. However, nowadays rigid inclusion often exclusively to drilled displacement concrete columns. Due to its similarity to deep soil mixing and piles, the previously discussed methods to calculate settlement of deep soil mixing and piles can be used for rigid inclusion. Han (2020) suggested that rigid inclusions are much stiffer than aggregate columns, so they are typically spaced further. The large spacing may cause differential settlement at the top of the embankment if the embankment height is limited. He recommended using a parabolic curve to calculate the settlement above soil and rigid inclusion.

4.2.6 Piles, and drilled shaft

There are many methods that can be used to calculate the settlement of piles supporting embankments, which include equivalent piers, Meyerhof method based on SPT and CPT, equivalent footing, etc. (Han 2015; Schaefer et al. 2017).

Equivalent pier method: As previously discussed, competent element and surrounding soil can be converted into an equivalent pier as shown in Figure 4-7, which is more suitable for concrete columns than aggregate columns. The Poulos (2007) used several methods to calculate the settlement of driven piles and compared with the measured results. He found out that equivalent pier method is more practical for driven pile groups.

Meyerhof method: Meyerhof (1976) developed a method to calculate the driven pile group settlement in cohesionless soils using SPT and CPT value as shown in Eqs. 4-14, 4-15, and 4-16, which was commented as a conservative method by Cheney and Chassie (2002).

$$S = \frac{4 p_f I_f \sqrt{B}}{N_{1(60)}} \text{ (Homogeneous sand)} \tag{4-14}$$

$$S = \frac{8 p_f I_f \sqrt{B}}{N_{1(60)}} \text{ (Silty sand)} \tag{4-15}$$

where $I_f = 1 - \frac{D'}{8B} \geq 0.5$, S is estimated total settlement (inches); p_f is design foundation pressure (ksf); B is width of pile group (ft); $N_{1(60)}$ is average corrected SPT N value within a depth B below pile toe; D' is two thirds of pile embedded length (ft); and I_f is influence factor for group embedment.

$$S = \frac{p_f I_f B}{2 q_{ca}} \tag{4-16}$$

where q_{ca} is average cone tip resistance within depth of B below the pile toe (ksf); other parameters have the same physical meanings as the ones in Eqs. 4-14 and 4-15.

Equivalent footing method: Terzaghi et al. (1996) proposed the equivalent footing concept to predict the group settlement in cohesive soils, which was modified in Cheney and Chassie (2002) and adopted in AASHTO (2016) as shown in Figure 4-10.

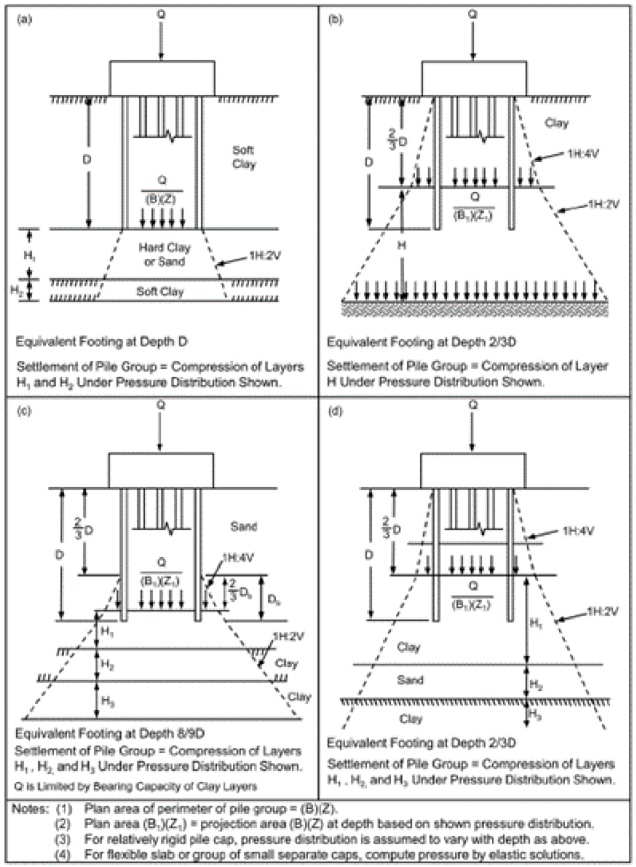


Figure 4-10. Stress distribution below pile group (AASHTO 2016).

The above discussed methods can be used for pile/drilled shaft supported embankments. Sometimes, geosynthetics layers can be used as basal reinforcement so that the piles/shafts can be spaced further apart, as shown in Figure 2-6. As a result, the differential settlement between the soil and pile/drilled shaft may be reflected to the crest of the embankment, causing surface differential settlement. In such applications, soil arching and tensioned membrane effect need to be taken into account to estimate the possible differential settlement at the embankment crest. A more common practice is to limit the pile/drilled shaft spacing so that the differential settlement is negligible, for example, the spacing cannot be more than 70% of the embankment height.

4.2.7 *Applicability of different methods*

Each soil improvement method has its own advantages and disadvantages and its applicable conditions. This section summarizes the applicability of these methods discussed previously, which can be used as references when selecting appropriate method(s).

Table 4-2. Applicability of different methods (Basu et al. 2010; Bergado et al. 1996; Brown 2005; Bruce 2013; Schaefer et al. 2017)

Methods	Applicable soil conditions	Applicable depth (ft)	Geometry
Preloading with wick drain	All soils but most effective in soft clays	Up to 200 ft	PVD is typically 4 inches wide by ⅛ to ⅜ inches thick
Lightweight fill	All soils but primarily for clay	N/A	N/A
Aggregate column	All soils, particularly for soft to very soft clays	Up to 35	Stone columns up to 36 inches in diameter but mostly 24 inches; rammed pier 18 to 36 inches in diameter, but mostly 24 inches
Deep soil mixing	Primarily clays	Up to 130	Up to 84 inches in diameter but mostly 24 inches
Rigid inclusion	All soils		Mostly 6 to 12 inches in diameter
Piles	All soils	> 100	Varies

4.3 CASE HISTORY REVIEW AND REASSESSMENT AND PROTOTYPE DEVELOPMENT

A few case histories have been provided by TxDOT, which includes ongoing and completed soil improvement projects of TxDOT. These case histories provide extensive information of detailed soil improvement design as well as soil conditions at the sites. THe provided case histories are summarized in Table 4-3 and are reviewed in detail in this section.

Table 4-3. Case history summary

Project ID	Project description	Improvement method(s)
0200-16-020	US-69 Jefferson County at SH 73	Wick Drains and Rigid Inclusions
0739-02-162	IH 10 Jefferson County	Wick Drains and Pile Supported Embankment
0009-12-219	IH 30 Rockwall County	Stone Columns
0196-03-268	IH 35E Lowest Stemmons, Dallas County	Stone Columns
0500-03-107	IH 45 Harris County	Stone Columns and CSB foundation replacement
0500-01-117	N/A (no information)	Stone Columns and pile supported embankment with Load Transfer Platform
0500-04-105	IH 45 Galveston	Unreinforced shafts as rigid inclusions

0200-16-020: The soil improvement is part of the US 69 Improvement Project, which is located at SH 73 at Jefferson County, as shown in Figure 4-11. The objectives of the project involve:

- Reconfiguring the interchange from the cloverleaf configuration to a turbine (spiral) configuration
- Adding direct connectors
- Adding main lane improvements
- Improving frontage road and ramps
- Adding retaining walls and culverts
- Widening and replacing bridges

The project site, located southeast of Beaumont near Port Arthur, has soft to medium stiff clay to approximately 80 ft and some places have clay up to 140 ft. The ground water is typically within 2 ft from the ground surface. Embankments with heights up to 11 ft are to be built to support roadways and bridge approach slabs. The estimated settlement of embankment significantly exceeded the allowable values, which is listed in Table 4-4. To ensure the serviceability, soil improvement methods are used to bring down the settlement to no more than 2 inches. The estimated time for consolidation ranges from 11 to 19 months as indicated in Table 4-4. Due to the existence of very soft clay up to 10 feet at site, in addition to excessive settlement, the stability of the embankment slope is also a concern at some locations. Therefore, different approaches are adopted to address the settlement and stability issues, which are elaborated below:

- *Case 1*: only wick drains are used to accelerate the consolidation, as shown in Figure 4-12. The wick drains of 60 ft in depth are spaced at 4 ft both underneath the embankment and beyond the toe of the embankment as illustrated in the figure. Such treatment is mostly applied to embankment with a height no more than 7 ft;
- *Cases 2 and 3*: the wick drain layout is similar to Case 1. The major difference is that one or two rows of rigid inclusions are used beyond the toe of the embankment slope, as shown in Figure 4-13, to improve the stability of the slope. Due to the existence of rigid inclusion, the wick drains beyond the toe of the embankment slope are spaced at 6 ft.

Table 4-4. Estimated settlement for unimproved and improved soil (from TxDOT project file).

DIRECT CONNECTOR	ABUTMENT No.	MAXIMUM ESTIMATED SETTLEMENT WITHOUT GROUND IMPROVEMENT (inch)	TARGET SETTLEMENT AFTER GROUND IMPROVEMENT (inch)	ESTIMATED PRELOADING TIME REQUIRED WITH WICK DRAINS (months)
DC 73WB-69SB (RED CONNECTOR)	1	17.2	15.2	18-20
	24	10.9	8.9	14-16
DC 69SB-73EB (BLUE CONNECTOR)	1	18.0	16.0	11-13
	24	13.6	11.6	11-13
DC 73EB-69NB (GREEN CONNECTOR)	1	12.2	10.2	10-12
	24	13.6	11.6	12-14
DC 69NB-73WB (YELLOW CONNECTOR)	1	11.5	9.5	17-19
	23	15.2	13.2	14-16

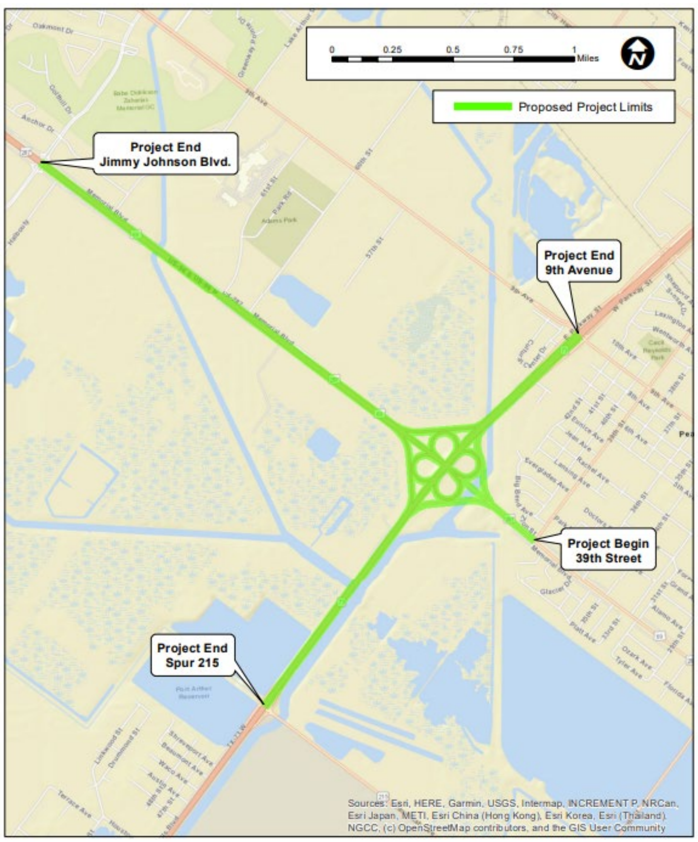


Figure 4-11. Project location map – Project-0200-16-020.

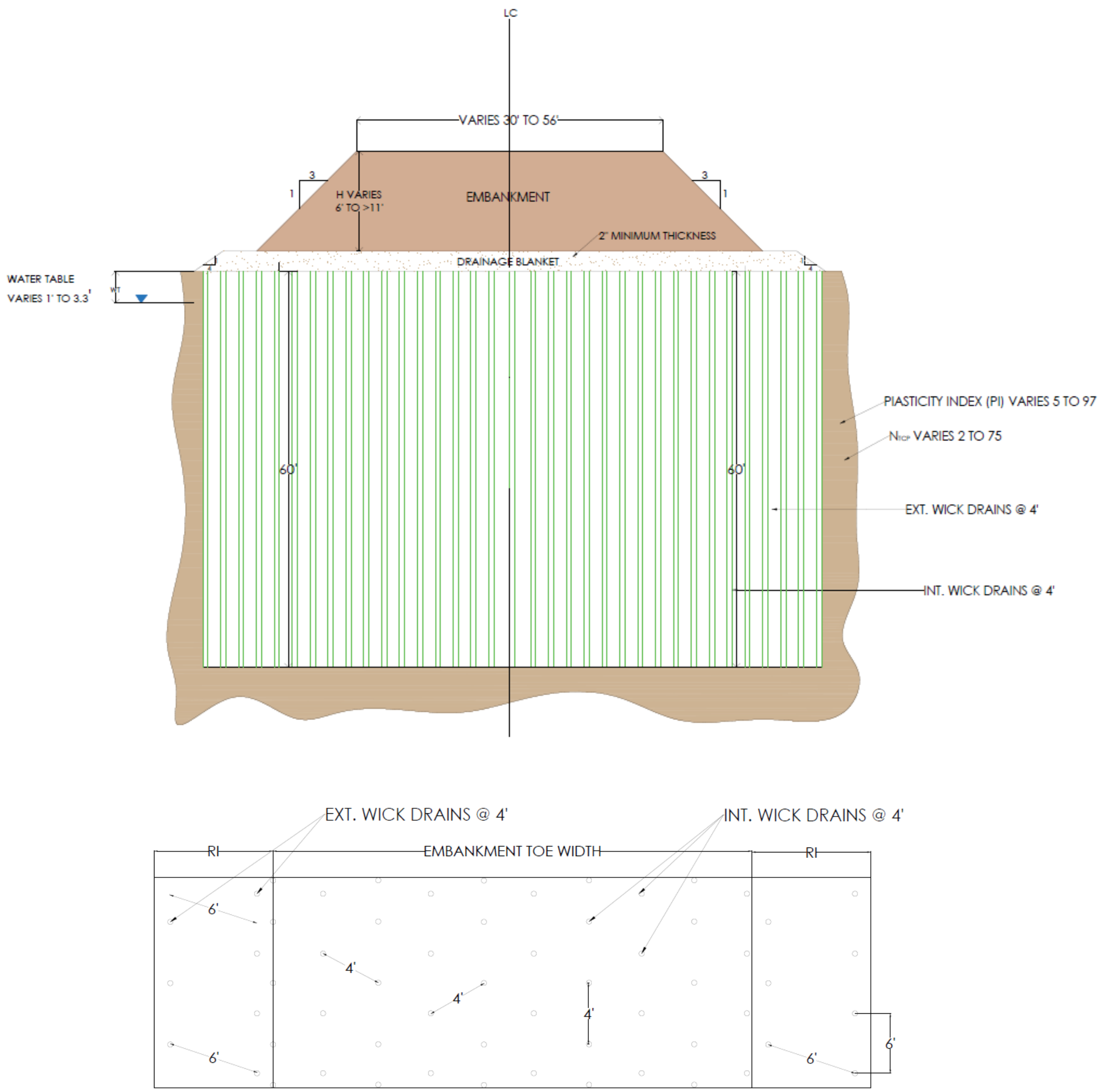


Figure 4-12. Typical cross-section of embankments with wick drains.

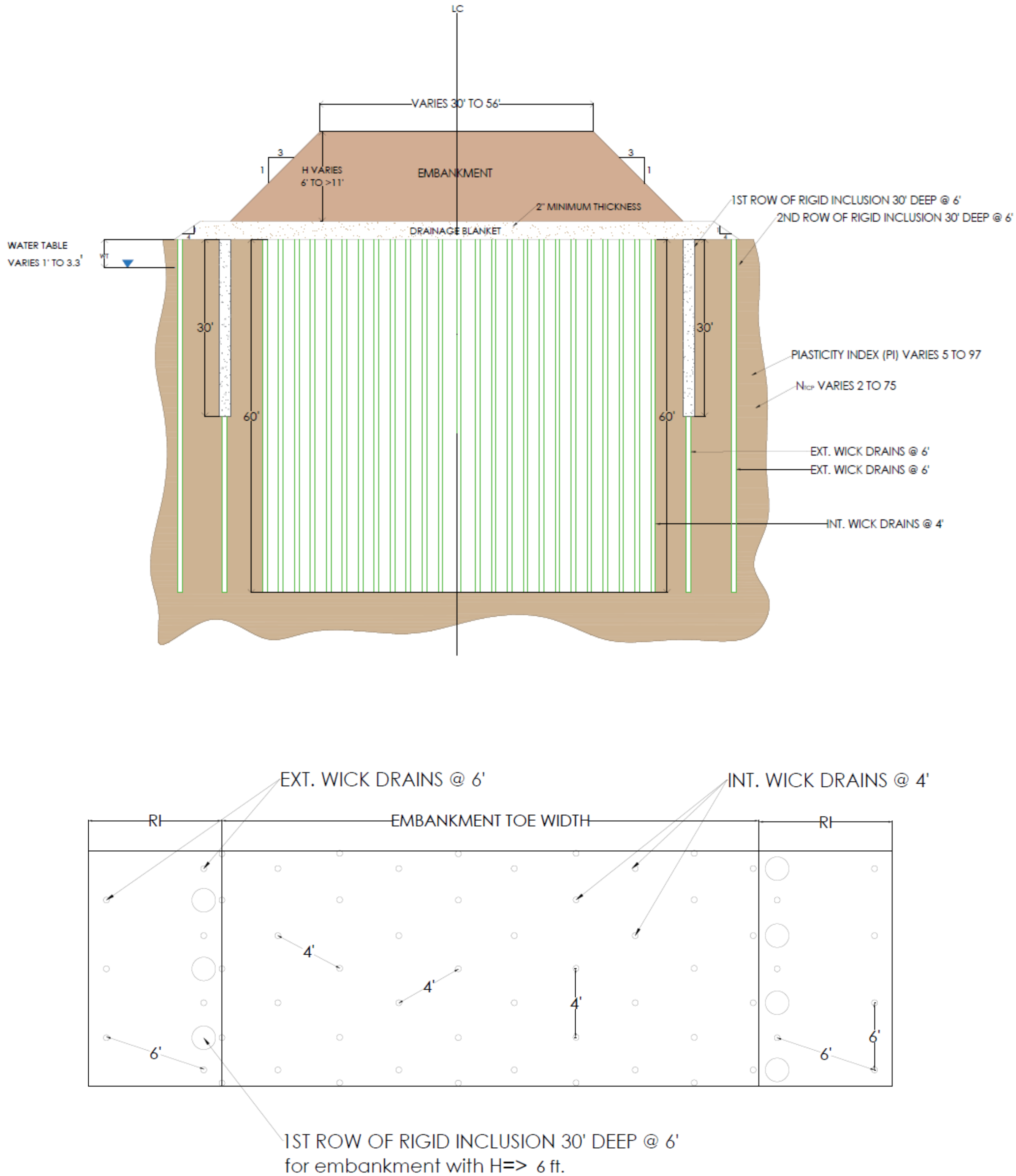


Figure 4-13. Typical cross-section of embankments with wick drains and rigid inclusions.

0739-02-162: The project is located in Beaumont District of TxDOT, as shown in Figure 4-14, which involves widening IH 10 from approximately FM365 to Walden Rd. The major objectives of the project are:

- Widening existing four lanes to six lanes
- Constructing frontage roads
- Constructing retaining walls
- Constructing bridges

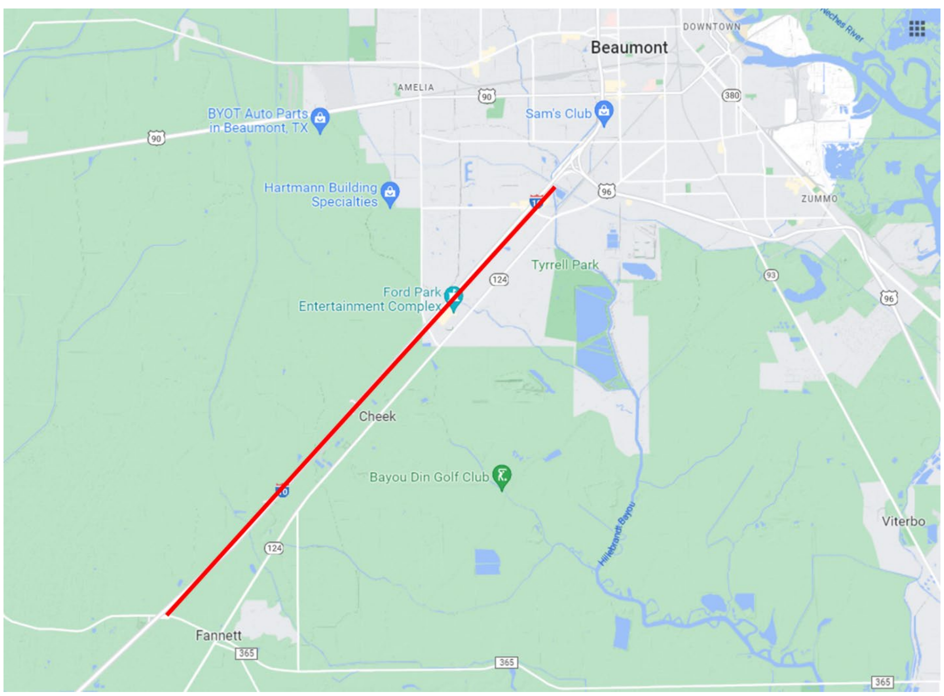
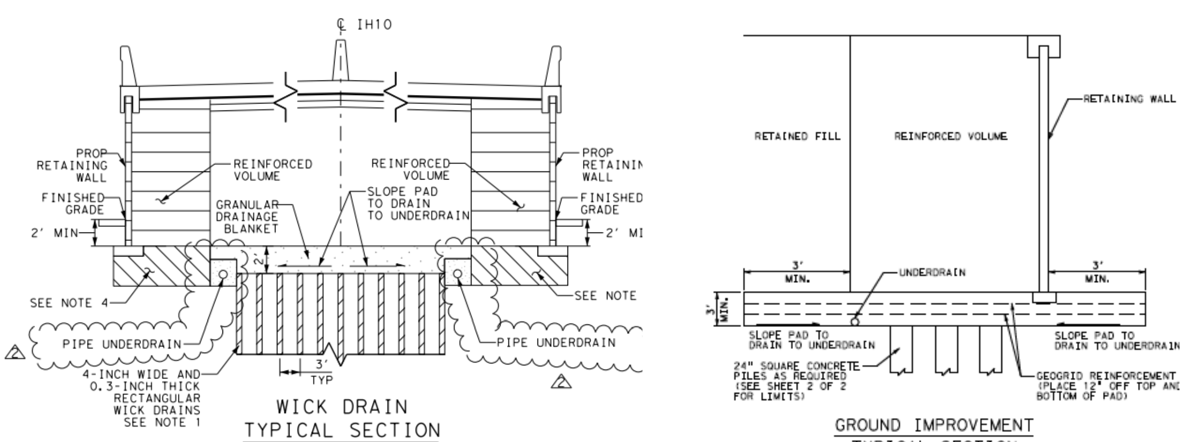


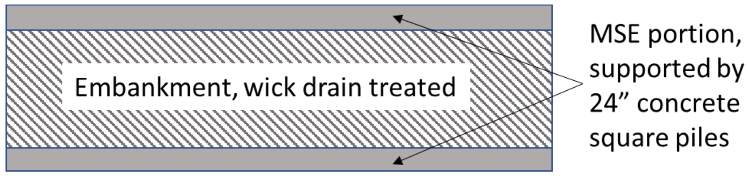
Figure 4-14. Project location map – Project-0739-02-162.

The in-site soil primarily consists of medium to stiff clay with possible sand layers. The borings were terminated at a depth of 40’ but the clay soil was expected to extend into a much great depth. The ground water was encountered at depth of 18-25’ during site exploration but it was expected to be much shallower during some time of a year. To support the widened roadway, MSE walls were built to retain soil behind the bridge abutment, as shown in Figure 4-15. To mitigate the settlement, wick drains and piles were used underneath the embankment fill and MSE walls, respectively, as shown in Figure 4-15. The MSE wall in general had a height of 5’ to 24.5’, as shown in Figure 4-16.

For the embankment located at the north of the bridge (i.e., the bridge over Inva Canal), the wick drains were spaced at 4’ and had penetration depths of 75’, 65’ and 40’, respectively, depending on the embankment heights, as shown in Figure 4-16. The MSE walls on both sides of the roadways were supported by two or three rows of 24” square concrete piles with a penetration depth of 34’, as shown in Figure 4-17. As for the south side of the bridge, the treatment was similar to the north side of the bridge, except for arrangement of the wick drain and piles. Specifically, the penetration depths of wick drains were 95’, 80’, 65’ or 40’, depending on the embankment heights (shown in Figure 4-18), while the penetration depth of concrete piles was 31’ (Figure 4-19). Due to the existence of MSE walls, the settlement criteria are assumed to 1/00 according to FHWA design manual.



(a) (b)



(c)

Figure 4-15. Typical cross-sections: (a) wick drains; (b) pile supported embankment; and (c) plan view of the ground improvement underneath embankment and MSE. wall

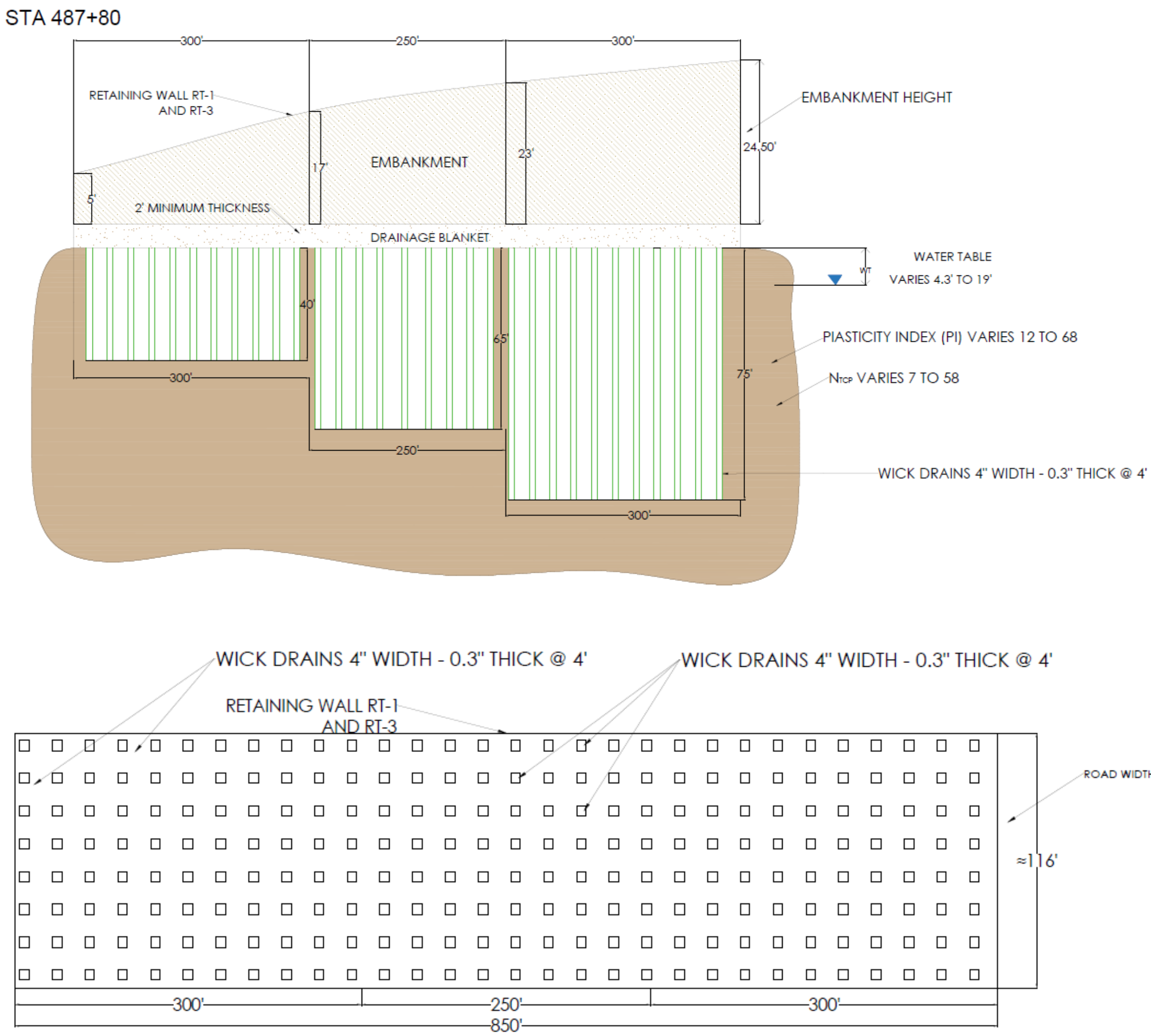


Figure 4-16. Typical cross-section of embankments with wick drains (north of bridge).

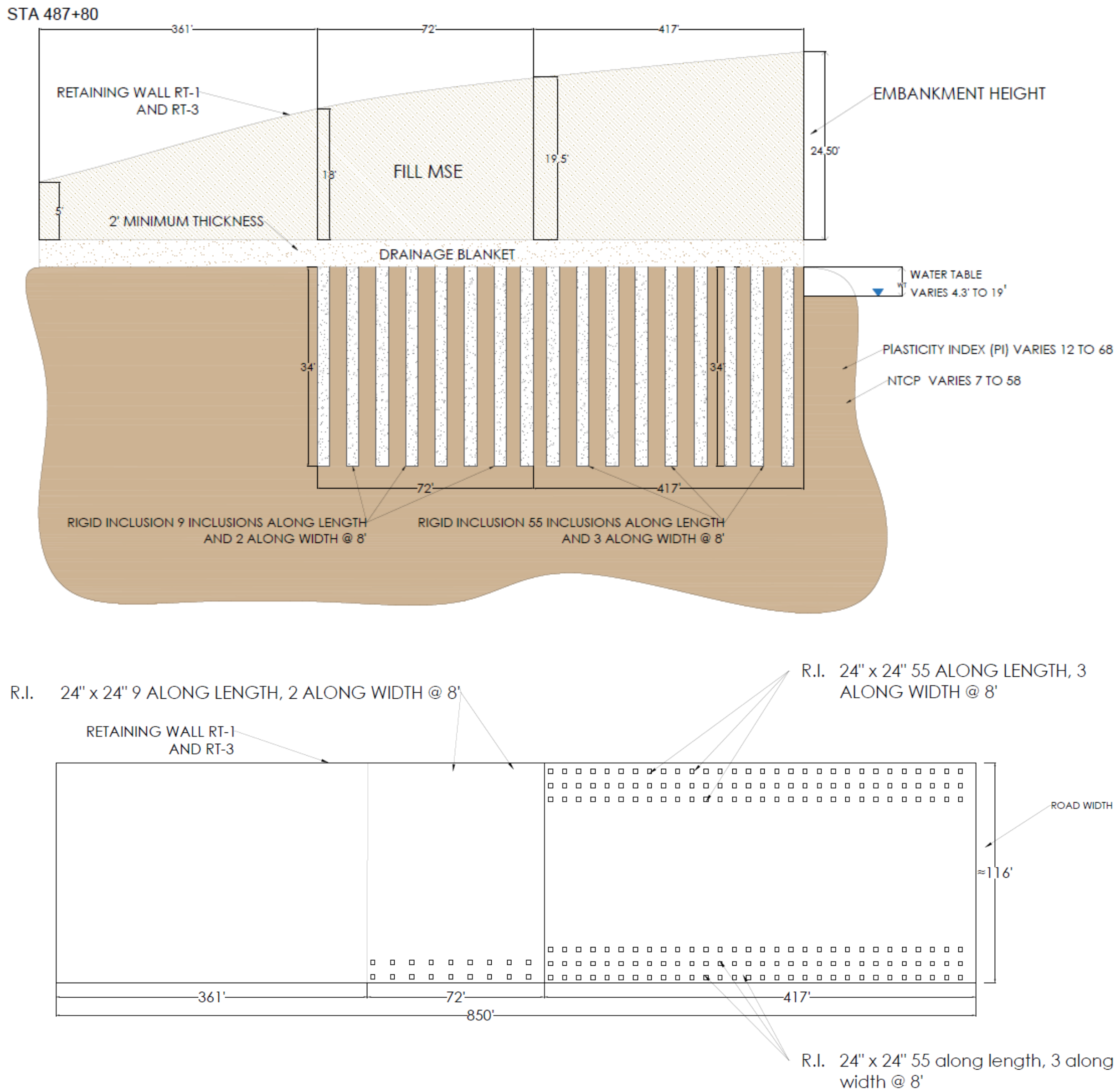


Figure 4-17. Typical cross-section of embankments with piles (north of bridge).

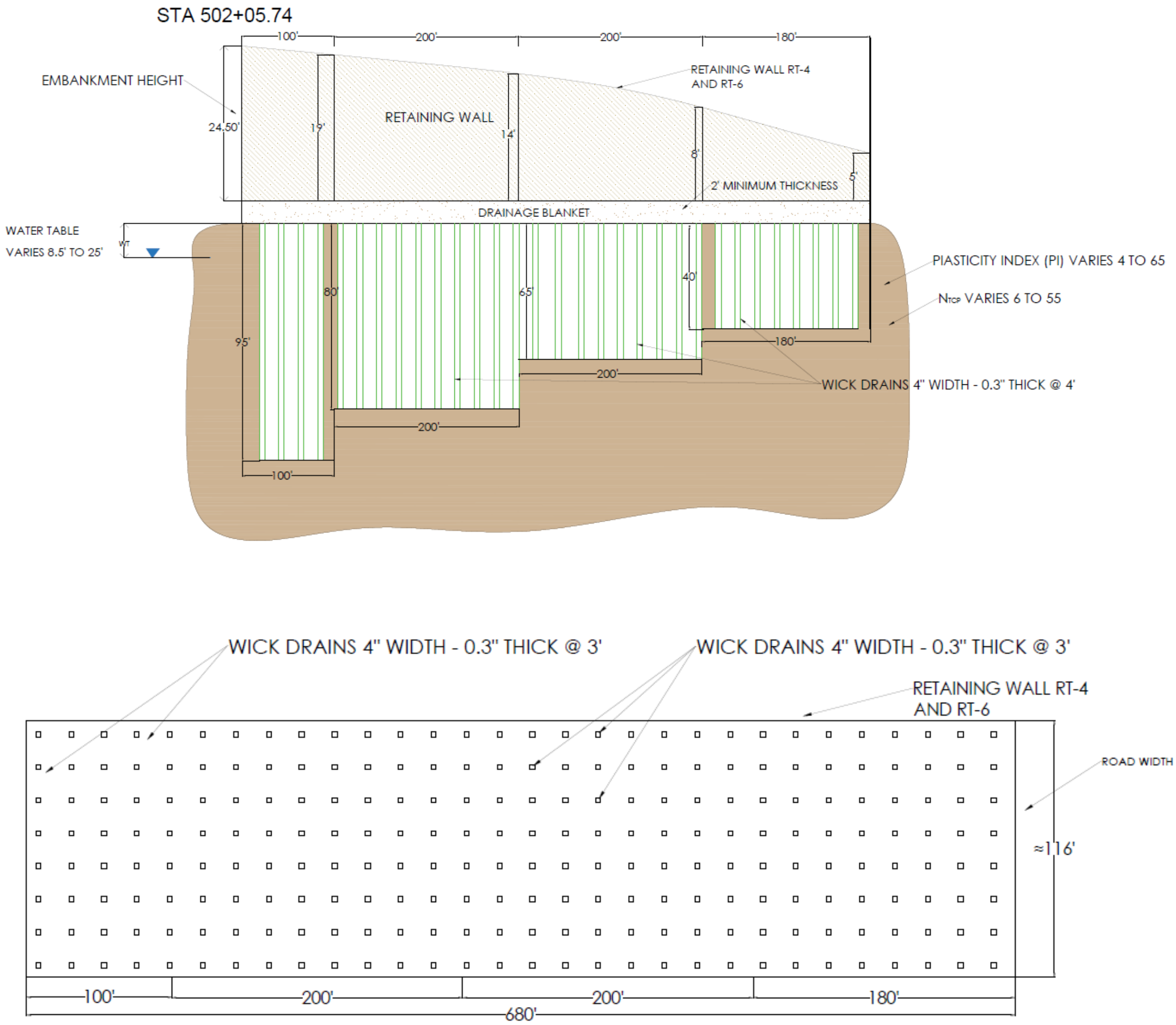


Figure 4-18. Typical cross-section of embankments with wick drains (south of bridge).

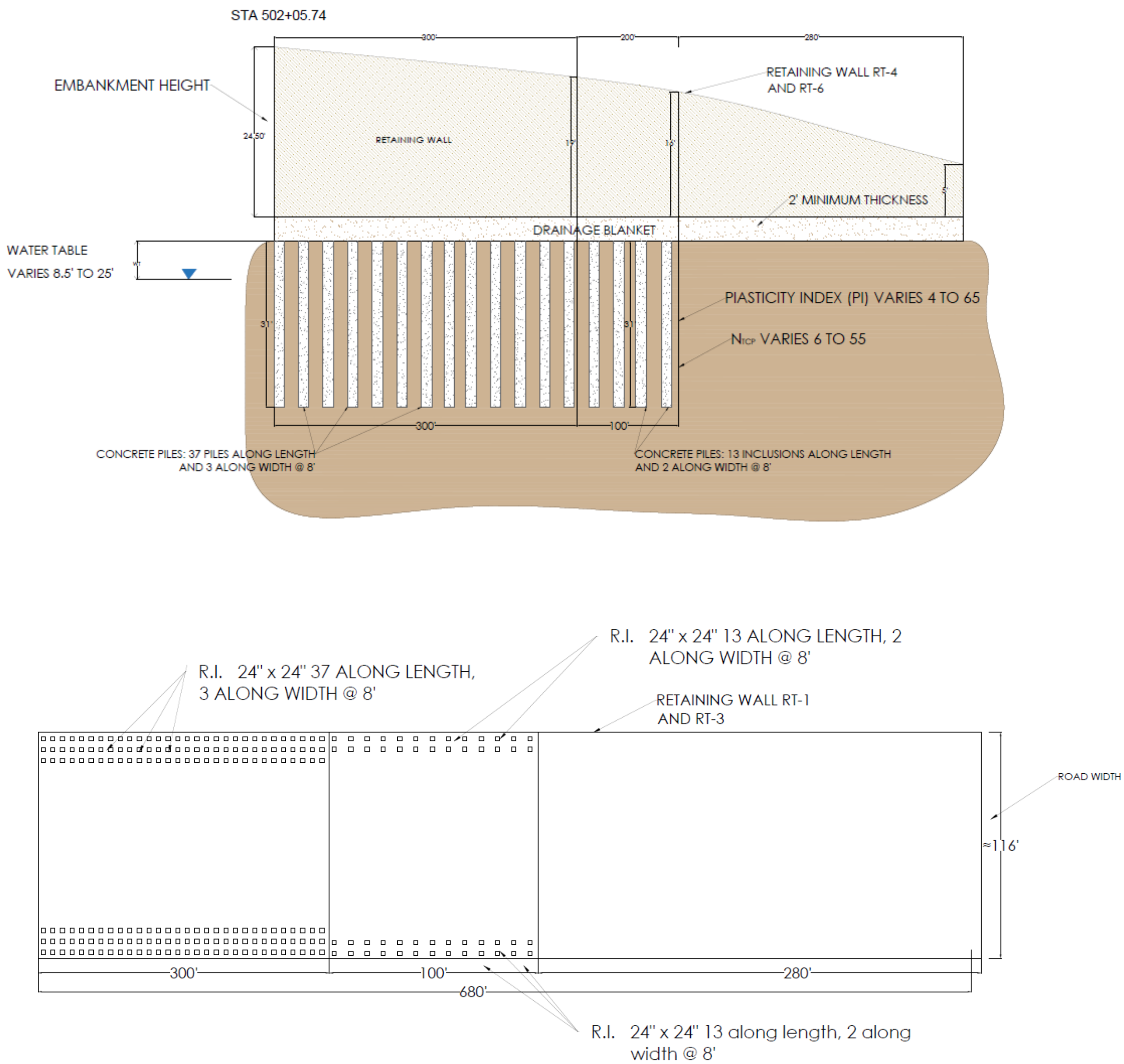


Figure 4-19. Typical cross-section of embankments with piles (south of bridge).

0009-12-219: This project is located in Rockwall County of Dallas District of TxDOT, as shown in Figure 4-20, which involves widening four main lanes to six main lanes for IH 30, widening frontage road, reconstructing existing interchange, etc. The borings of this project are not available; therefore, the soil conditions are not known. Stone columns and rammed aggregate columns are used for this project to support the embankment and

MSE walls. In general, there are two typical layouts of the stone columns/rammed aggregate columns, which shown in Figure 4-21 and Figure 4-22, respectively. For these two typical layouts, the MSE wall heights are similar but the spacing and required columns bearing capacity are different. For both layouts, the columns have a diameter of 30” and are installed in a triangular pattern as shown in the figures. However, due to lack of soil conditions, no more information can be derived on why two different column spacings are used.

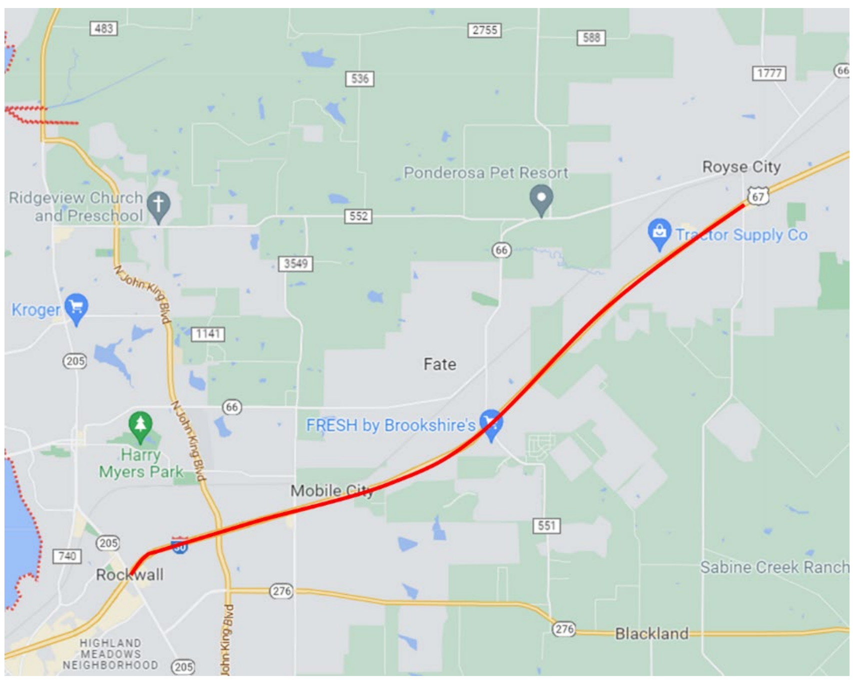


Figure 4-20. Project location map – Project-0009-12-219.

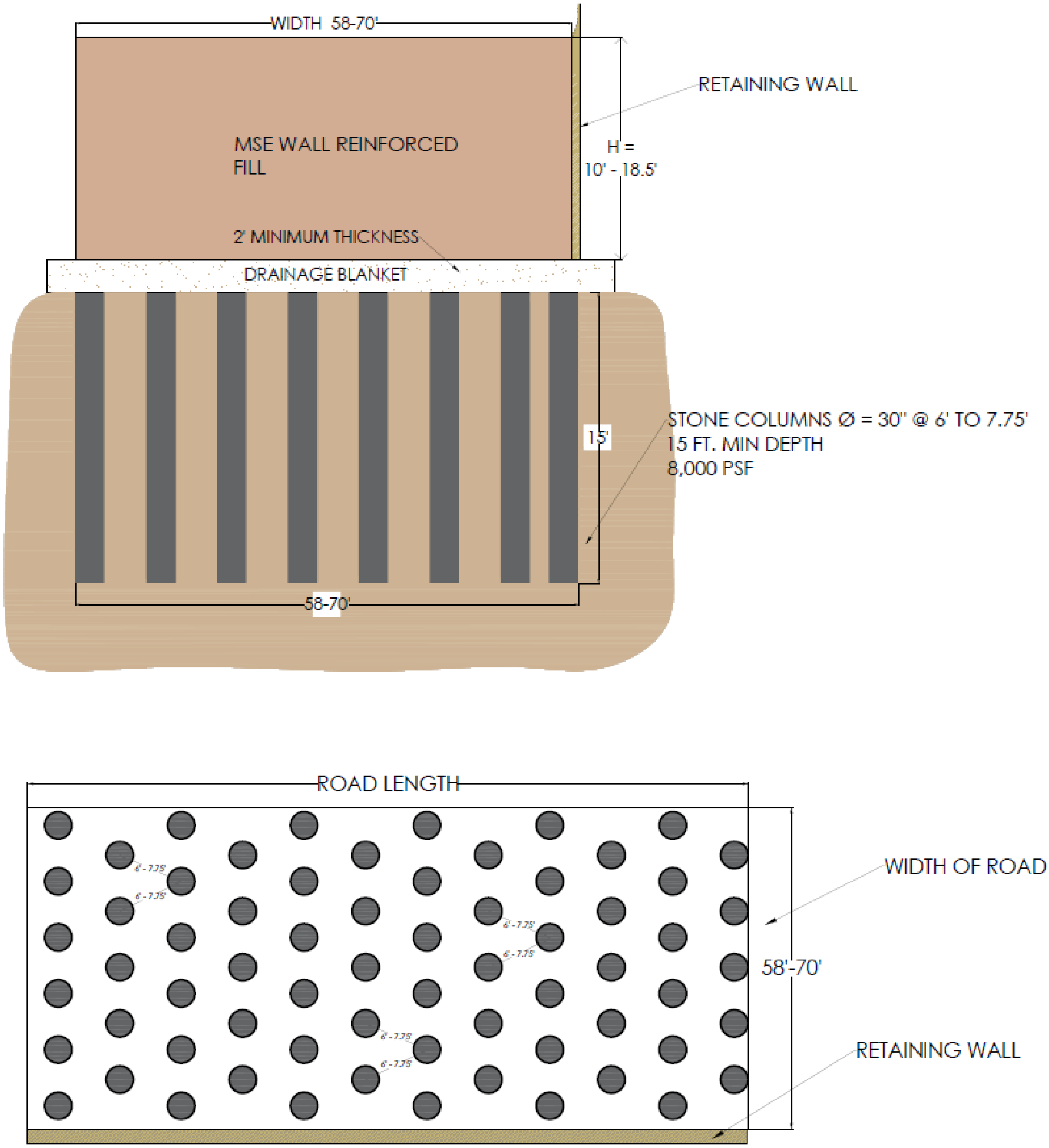


Figure 4-21. Cross-section and layout of stone columns/rammed aggregate columns (1st typical section)

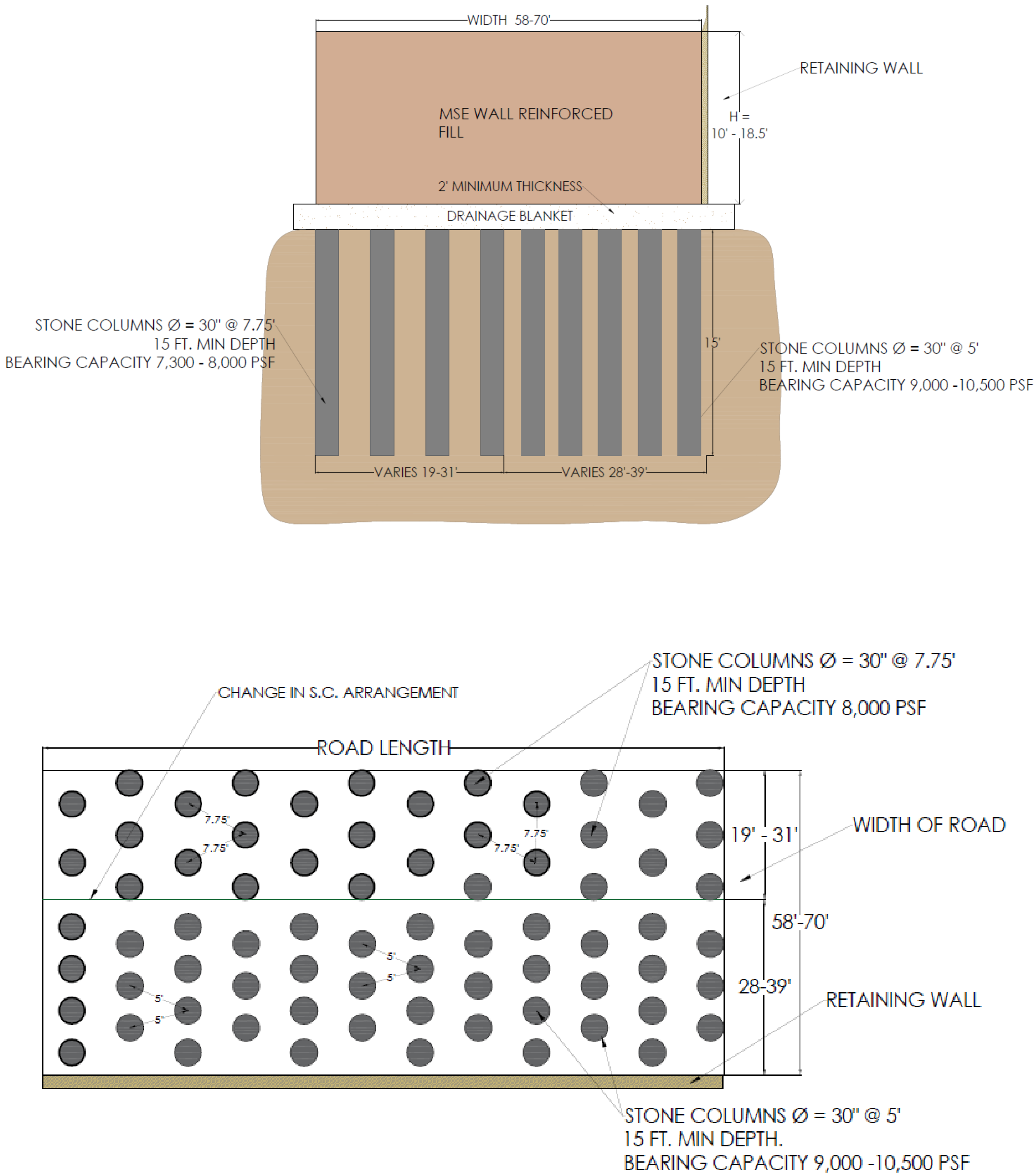


Figure 4-22. Cross-section and layout of stone columns/rammed aggregate columns (2nd typical section)

0196-03-268: This project is located in Dallas County, which involves the full reconstruction and widening of I-35E from approximately the Reunion District to Oak Lawn Ave, as shown in Figure 4-23. The objectives of this project are:

- The general-purpose lanes will be widened from six to eight lanes
- Continuous frontage roads will be constructed
- Improvements at numerous intersections within the project limits
- Reconstruct the two existing "grandfathered" toll managed lanes

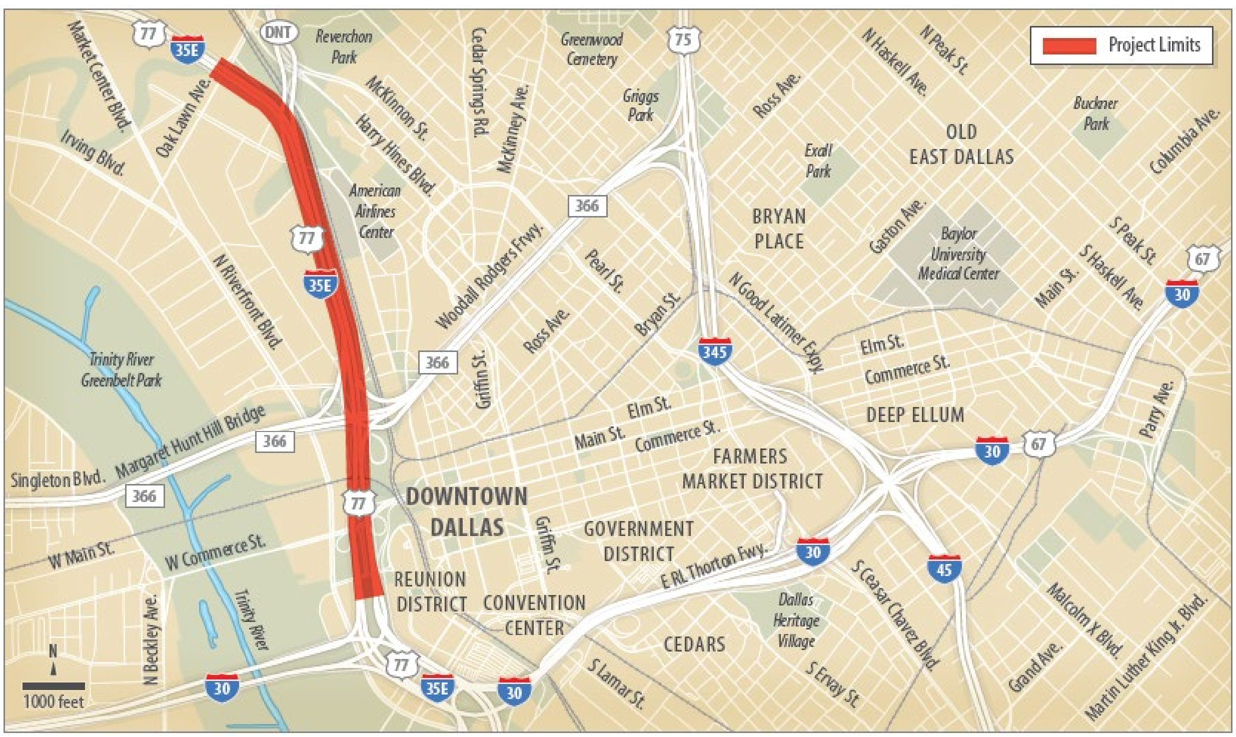
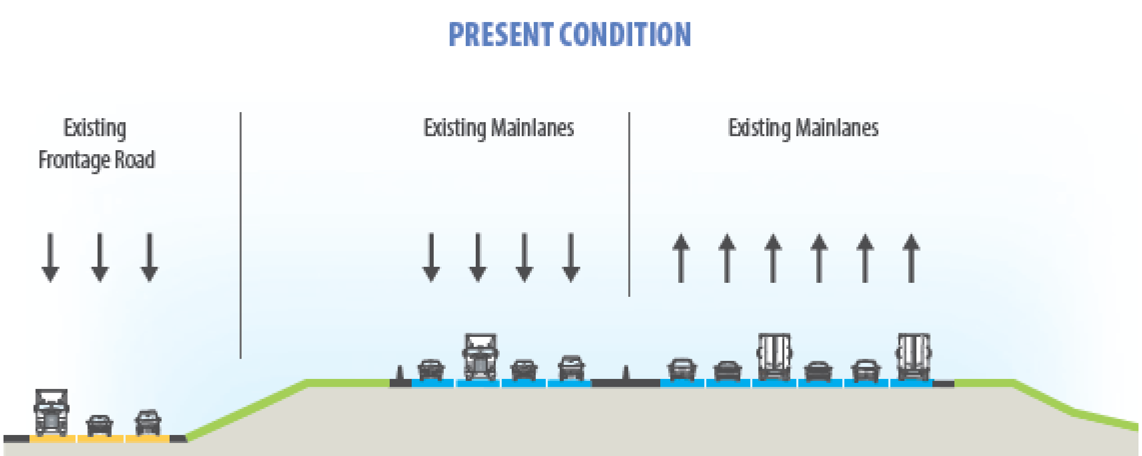
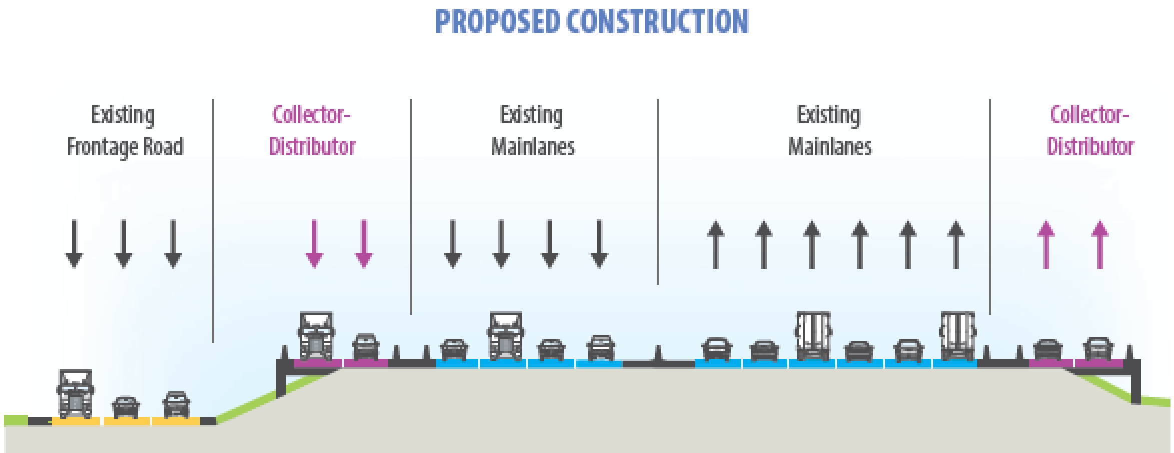


Figure 4-23. Project location map – Project-0196-03-268.

To support the widening, MSE walls are to be built, as shown in Figure 4-24. The heights of the MSE wall to be built vary from 10 ft to over 20 ft. At some locations, the part of the slope has to be removed and provide space for frontage road widening. As a result, a soil nail wall is to be built to retain the embankment and an MSE wall is to be built to provide space for the widening of the main lanes. Since the soil nail wall does not impose additional load to the existing ground, the construction of MSE wall creates additional settlements. The site investigation started from the pavement of IH 35 and the frontage roads. The boring often went to as deep as 50 ft but, excluding the thickness the existing embankment, the penetration in general ended at 30 ft to 35 ft below the original ground surface. No ground water was reported in the site exploration. Soft clay soil was reported in shallow depth and high-quality strata, including dense sand, weathered limestone and shale, was encountered in rather shallow depth and often in a range of 20' to 30' ft below the surface. To mitigate the settlements, stone columns are to be installed to support the MSE walls with a height of ~20 ft. According to the subsurface soil and retaining soil conditions, the MSE walls are to support by stone columns with varying length, as shown in the four figurations in Figure 4-25. The diameter of the stone columns is 30 in. and spaced at 5.5 to 7 ft in a triangular pattern as shown in Figure 4-26.

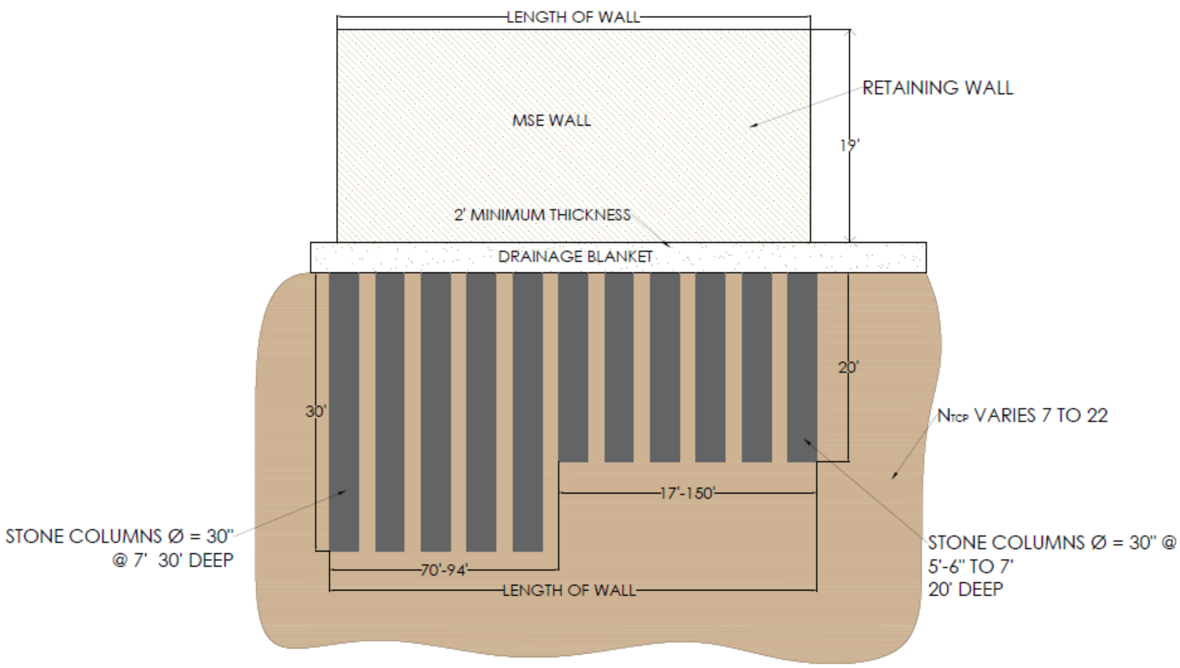


(a)

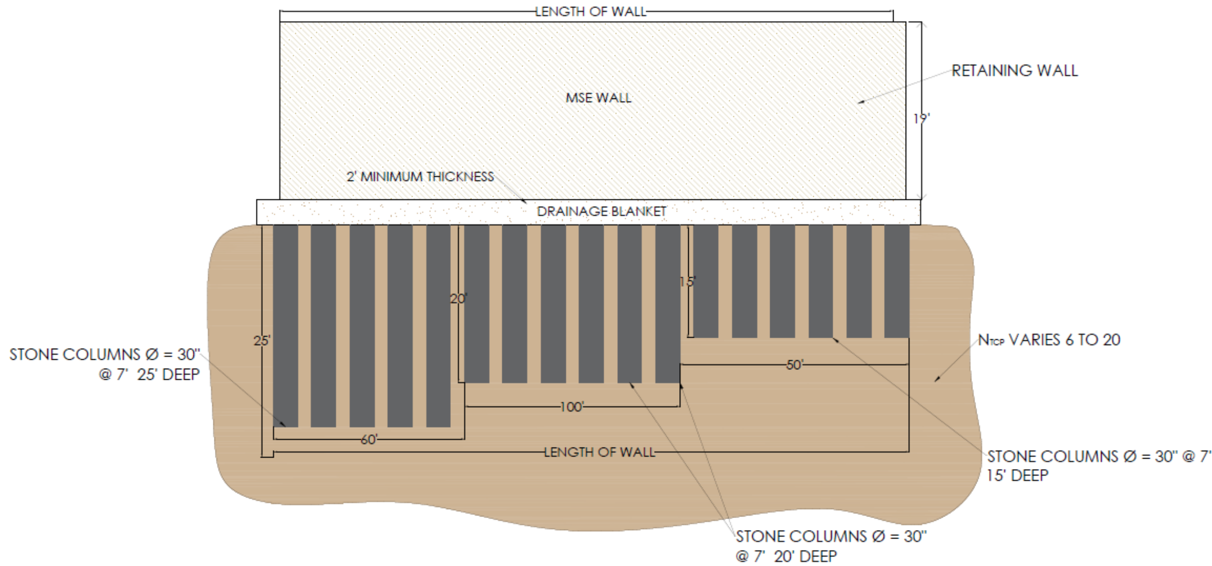


(b)

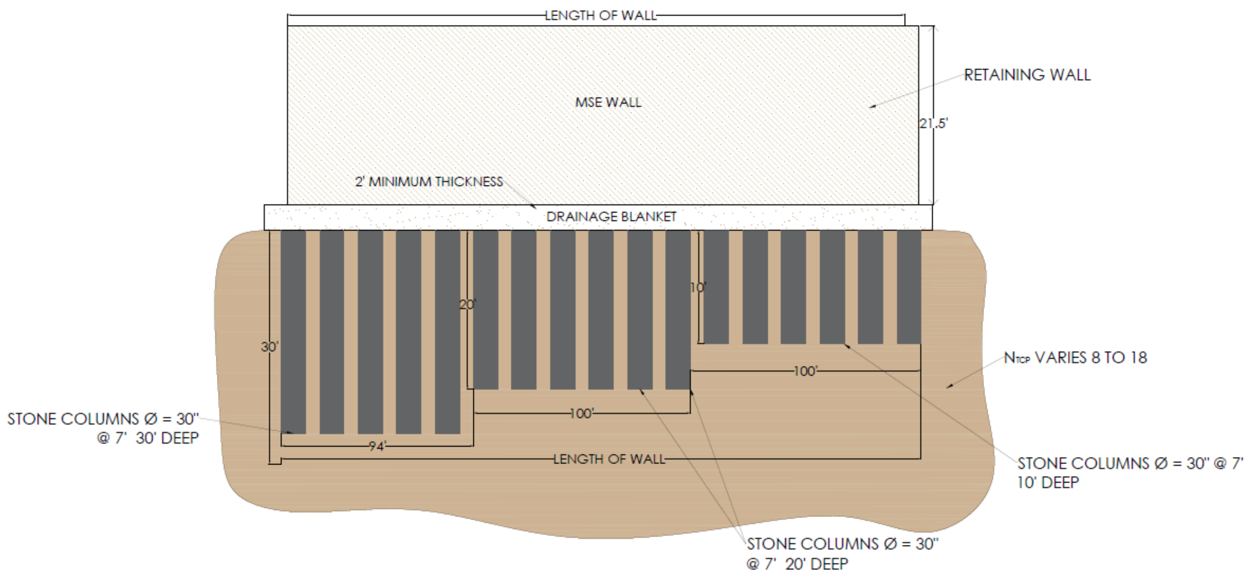
Figure 4-24. Project overview: (a) existing condition; and (b) proposed construction.



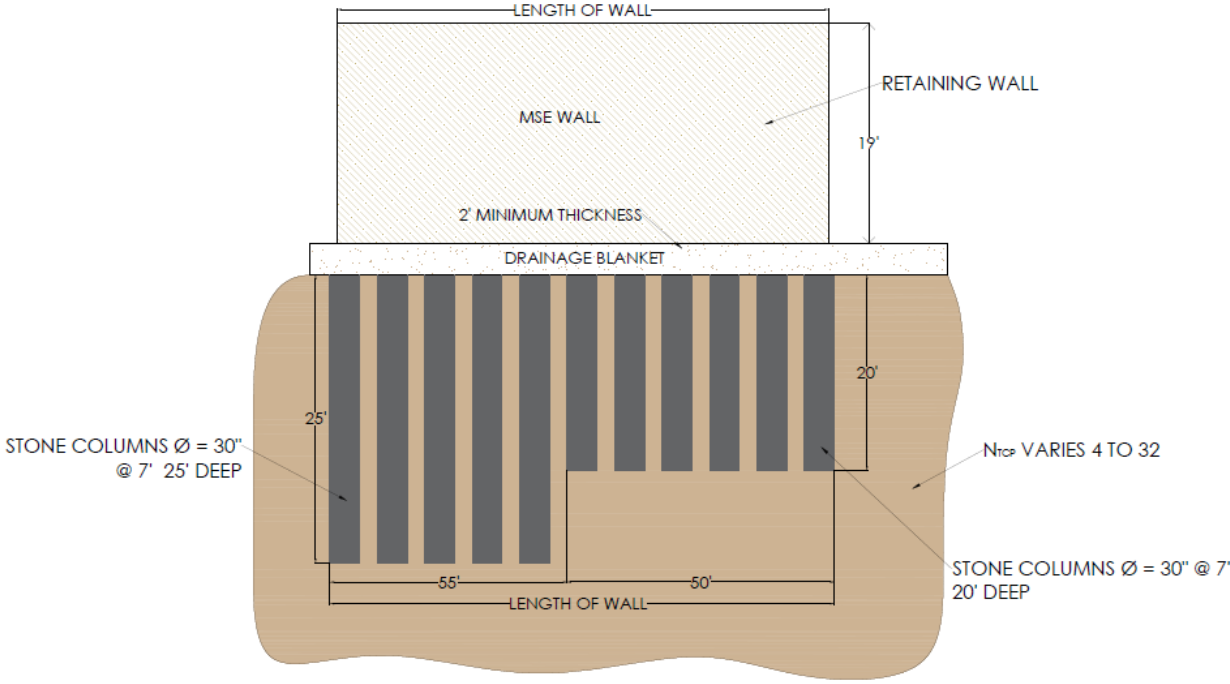
(a)


LENGTH OF WALL
MSE WALL
RETAINING WALL
19'
2' MINIMUM THICKNESS
DRAINAGE BLANKET
20'
15'
25'
STONE COLUMNS Ø = 30"
@ 7' 25' DEEP
NTCP VARIES 6 TO 20
50'
100'
60'
LENGTH OF WALL
STONE COLUMNS Ø = 30" @ 7'
15' DEEP
STONE COLUMNS Ø = 30"
@ 7' 20' DEEP

(b)


LENGTH OF WALL
MSE WALL
RETAINING WALL
21.5'
2' MINIMUM THICKNESS
DRAINAGE BLANKET
20'
10'
30'
NTCP VARIES 8 TO 18
100'
STONE COLUMNS Ø = 30"
@ 7' 30' DEEP
100'
94'
LENGTH OF WALL
STONE COLUMNS Ø = 30" @ 7'
10' DEEP
STONE COLUMNS Ø = 30"
@ 7' 20' DEEP

(c)



(d)

Figure 4-25. Cross-section of MSE wall with stone column support: (a) 30 and 20 ft column combination, (b) 25, 20 and 15 ft column combination, (c) 30, 20 and 10 ft column combination, and (d) 25 and 20 ft column combination.

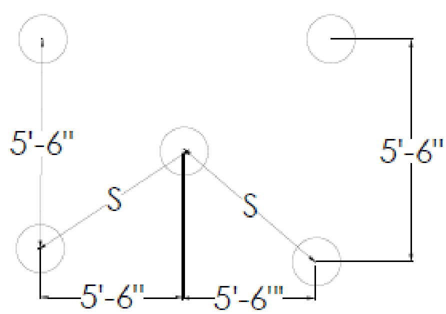


Figure 4-26. Stone column installation pattern

0500-03-107: This project is located in Harris County of Texas, which involves widening IH 45, as shown Figure 4-27. The objectives of the project involve:

- Widening IH 45 to 10 main lanes,
- Widening frontage roads to 3 lanes
- Constructing 2 HOV
- Constructing concrete pavement and retaining walls

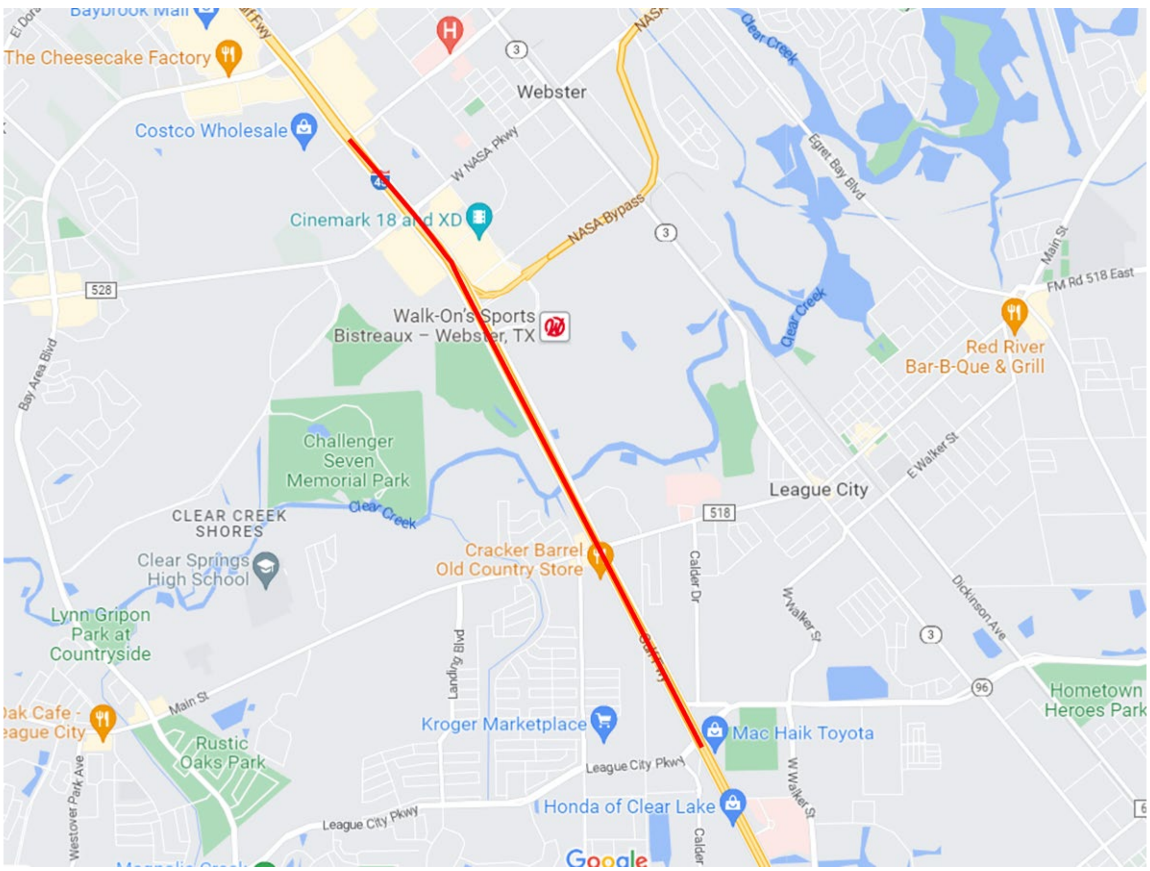


Figure 4-27. Project location map – Project-0500-03-107.

The situation of this project is quite similar to Project 0196-03-268, which has been described in detail previously and shown in Figure 4-24. The existing embankment was widened to support the new roadways. The soil boring indicated that the soil conditions were quite well. Due to the loading of existing embankment, the soil underneath ranged from stiff to hard clay and there was no soft clay. Ground water table was encountered at varying depths from as shallow as 4.5 ft to as deep as 16 ft. Most of locations showed a ground water table depth around 13 ft. MSE walls were built to support the widened roadways. The MSE wall heights ranged from as low as around 5 ft and as tall as nearly 20 ft. For the MSE wall less than 10 ft in height, the over-excavation and replacement with high-quality fill was used to provide competent support for the MSE wall. For MSE wall higher than 10 ft, stone columns with a diameter of 36 in. were installed at 12 ft spacing in a square pattern to provide support, as marked in Figure 4-28. The penetration depth of the stone columns was 12 ft.

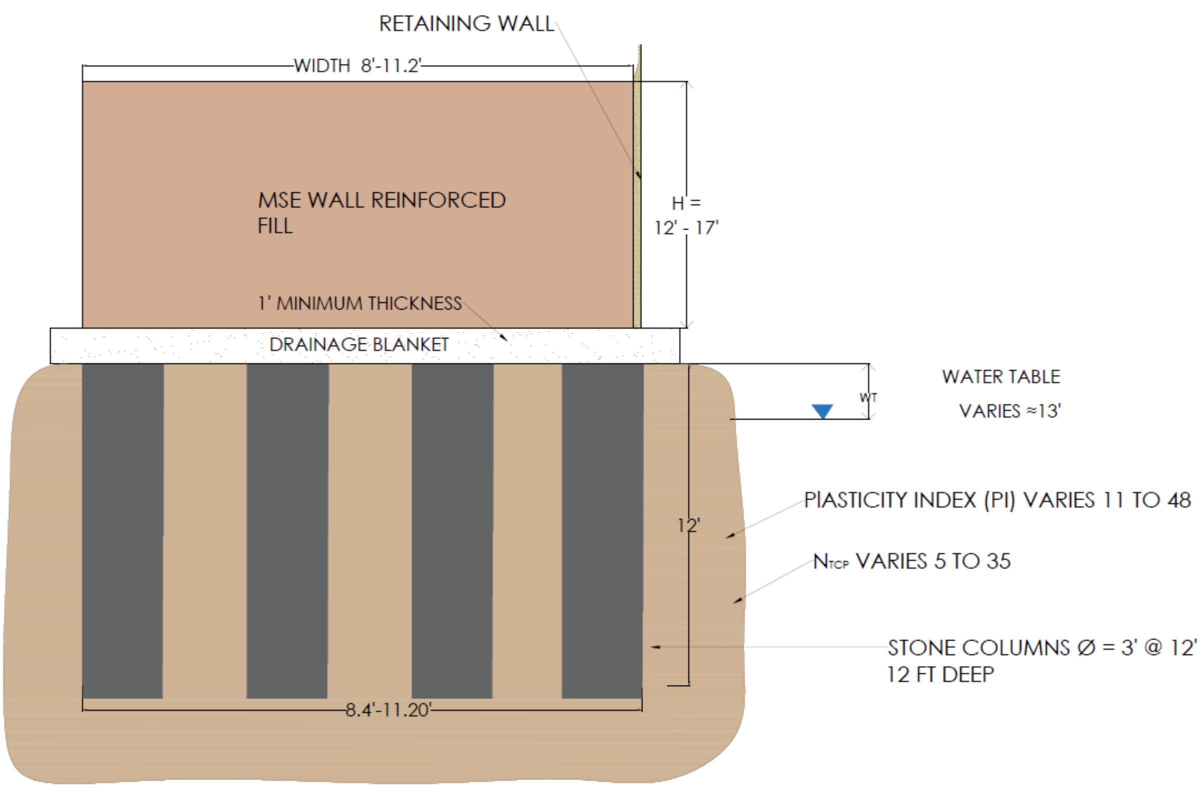


Figure 4-28. Cross-section of a typical section – Project-0500-03-107.

0500-01-117: N/A

0500-04-105: This project is located in Galveston County of Texas, which involves widening IH 45, as shown in Figure 4-29. The objectives of the project involve:

- Widening IH 45 to 8 main lanes
- Widening frontage roads to 2 lanes
- Constructing concrete pavement and retaining walls
- Construction bridges and storm drain

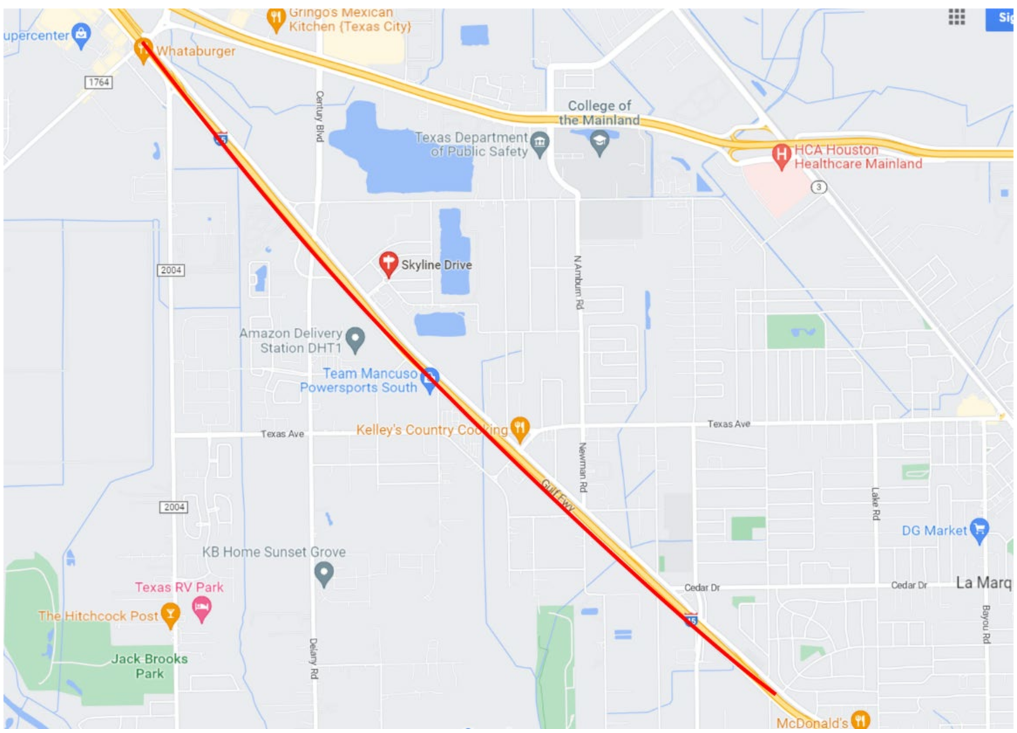
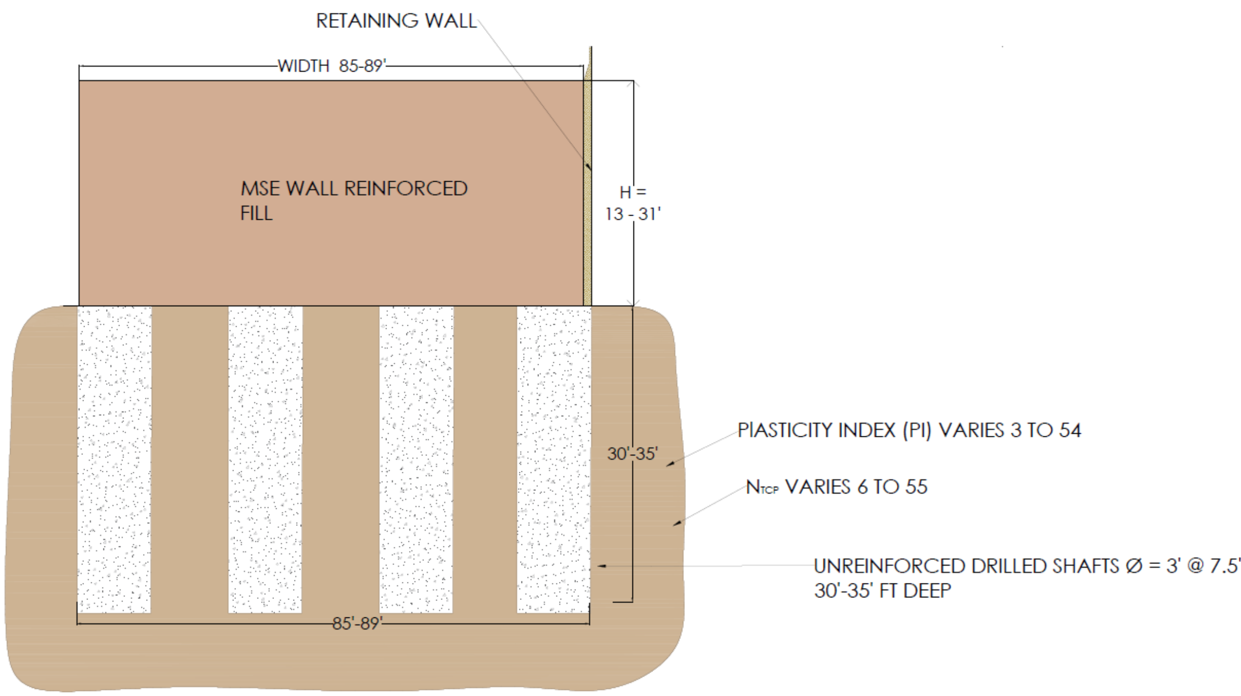


Figure 4-29. Project location map – Project-0500-04-105.

This project is very similar to Projects 0196-03-268 and 0500-03-107. The stone columns with 12 ft penetration were installed to support the MSE walls, as shown in Figure 4-30.

The ground water depth was unclear as some borings encountered the ground water table and others did not.



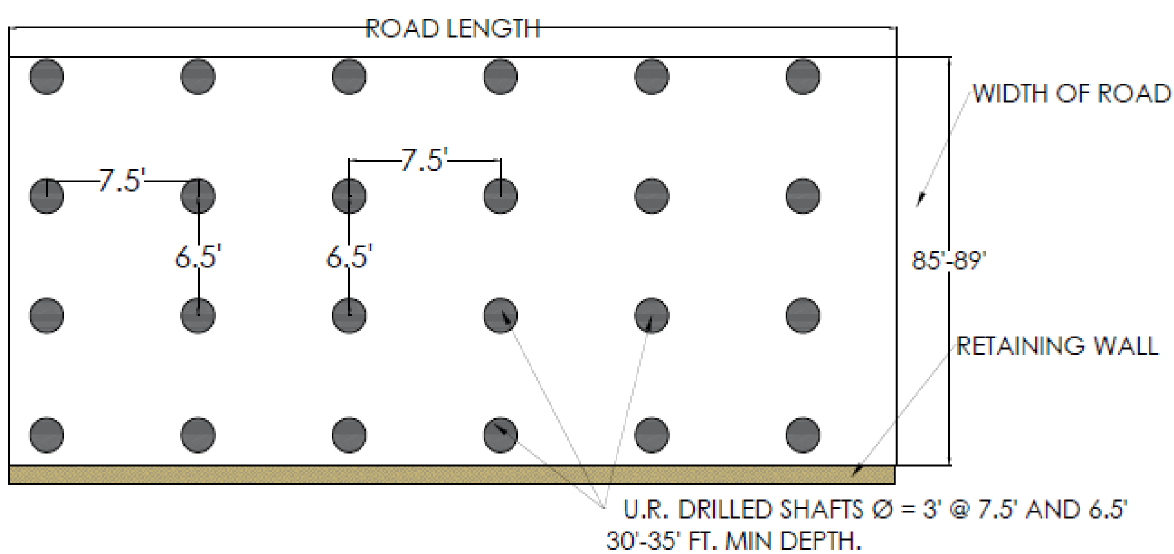


Figure 4-30. Cross-section of a typical section – Project-0500-04-105.

4.4 TIME-COST CHARTS AND DISCUSSIONS

To facilitate preliminary settlement mitigation method selection, a series of time-cost charts are developed, so the user can use them to identify possible methods for their sites before more detailed analysis and comparison.

4.4.1 Prototype development

Based on the review and assessment of the case histories, the possible soil conditions, and embankment/MSE wall dimensions are summarized in Table 4-5 and Figure 4-31. Even though only 6 case histories were reviewed, they could be representative of Texas situations.

Table 4-5. Summary of soil and embankment conditions based on case histories

Category	Item	Maximum possible value
Soil	Texas cone penetrometer (TCP)	58
	Plasticity index (PI)	97
	Treated depth (ft)	90
Embankment/MSE wall	Width (ft)	140
	Height (ft)	25
	Settlement criteria	2” (embankment); 1” (MSE) wall

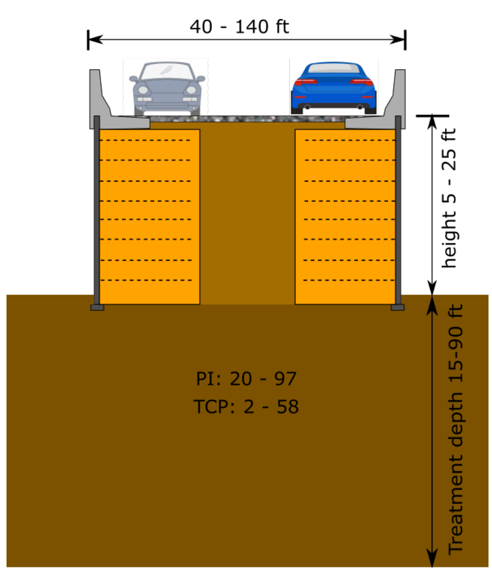


Figure 4-31. Prototype of embankment/MSE wall for this task.

In addition, the following assumptions are adopted to develop the construction time and cost charts:

- The total area to be treated is 40,000 ft^2 (this number is based on 200 × 200 ft^2),
- The soil depth is assumed to be 30 ft,
- Stone columns and rammed aggregate columns are limited to 30 ft in length,
- This task only assesses individual soil improvement method, and no combination is considered; thus, the consolidation associated with stone columns and deep mixing is not accounted for,
- Construction time and cost of different soil improvement are calculated based on the unit cost and productivity information published in various sources, which is shown in Table 4-8, and may needed to be updated to reflect the best information of Texas.

Table 4-6. Construction rate and unit cost of different methods (Bruce 2013; Schaefer et al. 2017).

Method	Unit cost	Construction rate
Wick drain	$1-4/ft	20,000 ft/day
Stone column	$30-50/ft	600 ft/day
Rammed aggregate column	$35-60/ft	400 ft/day
Deep mixing	$80-110/yd^3	500 ft/day
Driven pile	$200-400/yd^3	400 ft/day

To make the analysis result more applicable, instead of calculating specific settlement, this analysis focuses on the degree of settlement reduction defined as the percentage of reduction of total settlement without soil improvement. Starting from 40% settlement reduction with respect to the untreated soil, the analysis quantifies the needed time and associated cost for these methods to achieve up to 90% settlement reduction. Even though this study assumes 40,000 ft^2 of treatment area and 30 ft treatment depth, the results can be converted to the sites with different dimensions by accounting for the scale factor that is based on the total soil volume ratio of the prototype to the site to be calculated.

4.4.2 Design parameters

The soil improvement design is mainly based on the possible information provided by the boring logs, which consists of TCP blow counts, liquid limit, plastic limit, and plasticity index. The other needed information should be derived or correlated from the available information on boring logs, i.e., TCP blow counts and Atterberg limits, which are summarized in Table 4-6 and shown in Figure 4-32.

Table 4-7. Correlations used to derive needed design parameters

Derived information	Correlation	Reference(s)
SPT blow count (N)	$N_{SPT}=0.7N_{TCP}$ (fine grained soil) $N_{SPT}=0.5N_{TCP}$ (coarse grained soil)	Touma and Reese (1972); Lawson et al. (2018)
Undrained shear strength (S_u)	$S_u=N_{TCP}/55$ (tsf) (for CH soil) $S_u=N_{TCP}/45$ (tsf) (for CL soil)	Vipulanandan et al. (2008)
Soil modulus (E)	$E = K_c \times S_u$ (Kc - correlation factor, shown in Figure 4-32)	Mansour et al. (2010)
Compression index (C_c)	$C_c=PI/74$	Kulhawy and Mayne (1990)
Swell index (C_s)	$C_s=PI/370$	
Coefficient of consolidation (C_v)	$C_v = \frac{97.808 \times 10^{-7}(1.192 + ACT^{-1})^{6.998}(4.135LL+1)^{4.29}}{PI(2.03LL+1.192+ACT^{-1})^{7.998}}$ (ft²/s) $C_v = 1.202PI^{-3.1025}$ (in²/s)	Carrier (1985) and Solanki (2011)
Overconsolidation ratio (OCR)	$OCR = 0.58\frac{N_{60}p_a}{\sigma_o'}$	Kulhawy and Mayne (1990)

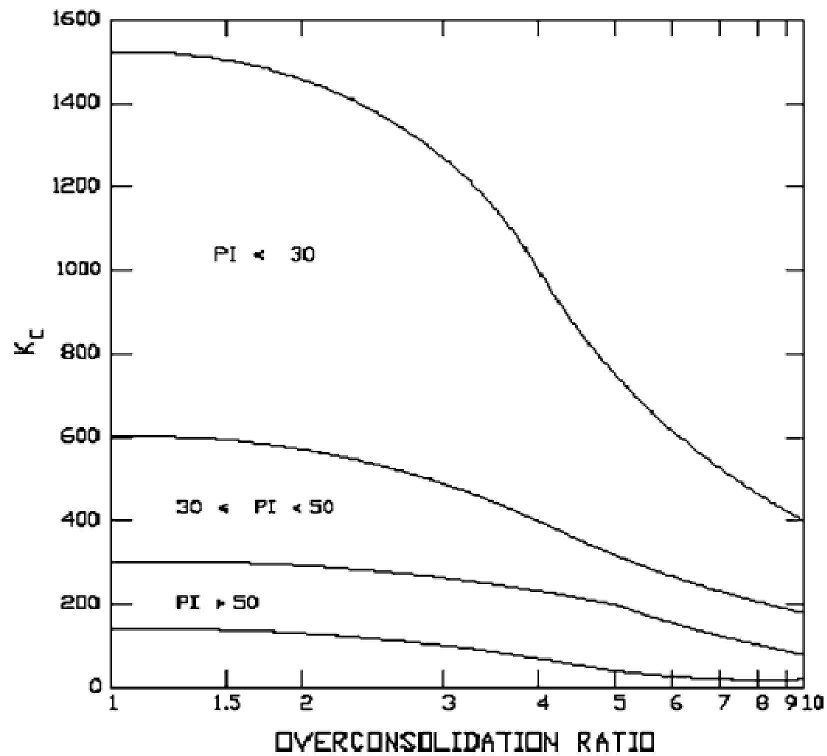


Figure 4-32. Correlation factor for soil modulus (after Joint Department of Army and Air Force, USA, TM-5-818-1)

To avoid using too much project specific information and make the yielded results more applicable, the PI of soil is used in this analysis as a variable, which ranges from 20 to 70 in this study. The basic parameters used for the design of each of above-listed methods are summarized in Table 4-7.

Table 4-8. Design parameter for each method

Method	Design parameter	Remarks
Wick drain	4" width, 0.3" thickness, 4' spacing	Typical dimensions
Stone column	30" diameter, modulus = 7,000 psi stress concentration = 4, length < 35'	Ambily and Gandhi (2007); Schaefer et al. (2017)
Rammed aggregate column	30" diameter, stiffness modulus = 65 pci, stress concentration = 3, length < 35', stress concentration ratio = 6	Schaefer et al. (2017)

Deep mixing columns	30” diameter, unconfined strength (q_u) = 145 psi (soft clay); = 290 psi (stiff and hard clay), modulus = $300q_u$	(Han 2015); Bruce et al. (2013)
Piles	24” diameter reinforcement concrete	

4.4.3 Major results and discussions

Each of the methods is analyzed by assuming 40%, 50%, 55%, 60%, 70%, 80%, and 90% of settlement reduction or up to its possible maximum reduction, which is summarized Table 4-9. Except lightweight fill, the remaining other methods reduce the settlement by introducing more competent elements into soil. Since the lightweight fill depends on the dimensions of the embankment, it is not included in the cost analysis of this study. As shown in Table 4-9, stone column, rammed aggregate column and deep soil mixing column are limited to maximum of 55%, 60% and 90% settlement reduction due to the fact that their area replacement ratio cannot be more than 0.4.

Table 4-9. Applicability of different methods

	<50 %	**50 %**	**55 %**	**60 %**	**70 %**	**80 %**	**90 %**	**>90 %**
Lightweight fill	√							
Stone column	√	√	√					
Rammed aggregate column	√	√	√	√				
Deep soil mixing	√	√	√	√	√	√	√	
Pile	√	√	√	√	√	√	√	√
Wick drain	√	√	√	√	√	√	√	√

Figure 4-33 and Figure 4-35 **-** Figure 4-38 present the required duration and associated cost for different methods to achieve different degrees of settlement reduction. The cost of each method covers a range to reflect the unit price variation shown in Table 4-6. The duration of the wick drain only includes the time for consolidation but does not include the duration for wick drain installation because compared to consolidation time the installation time can be ignored. All the results assume that the soil PI is 20, which is a common value in Texas soil in Beaumont, Houston, Dallas, Fort Worth, Austin, and San Antonio areas. The change of PI does not impact the design of other methods except wick drain. The PI is strongly associated with soil permeability and consequently the coefficient of consolidation (C_v). Thus, the higher the soil's PI, the longer it takes to achieve the desired consolidation. The effect of PI on consolidation is presented in Figure 4-34. To facilitate the preliminary selection, the cost and duration of all the methods at different degrees of settlement reduction are organized into a design chart as shown in Figure 4-39. The top of the chart contains the duration information, and the bottom of the chart contains the cost information. Again, the numbers of this figure are based on 40,000 ft^2 site with 30 ft deep of soil, i.e., 1,200,000 ft^3 (i.e., 40,000 ft^2×30 ft). It can be proportionally scaled up or scaled down based on the total volume of soil to be treated. The only exception is the duration needed for wick drain. Since the needed time for consolidation depends on the spacing of the wick drain but not the depth of the wick drains and the installation time of wick drain is negligible. More similar design charts considering different soil PIs are provided in Appendix A of this report.

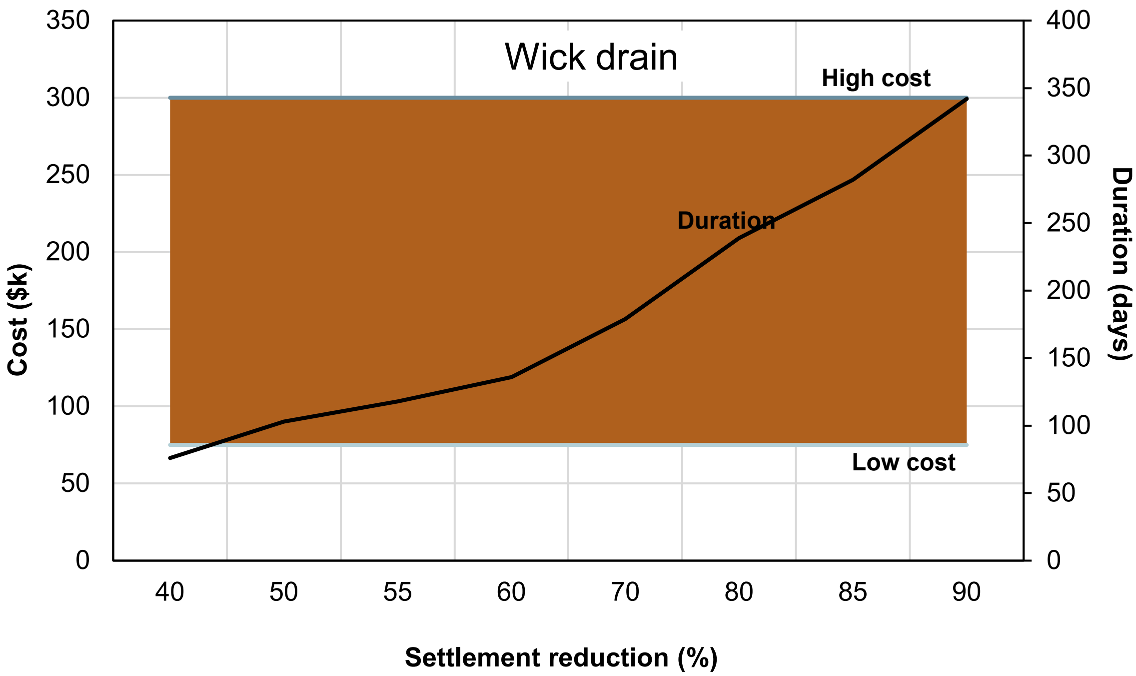


Figure 4-33. Wick drain cost and required duration to achieve design settlement (PI=20)

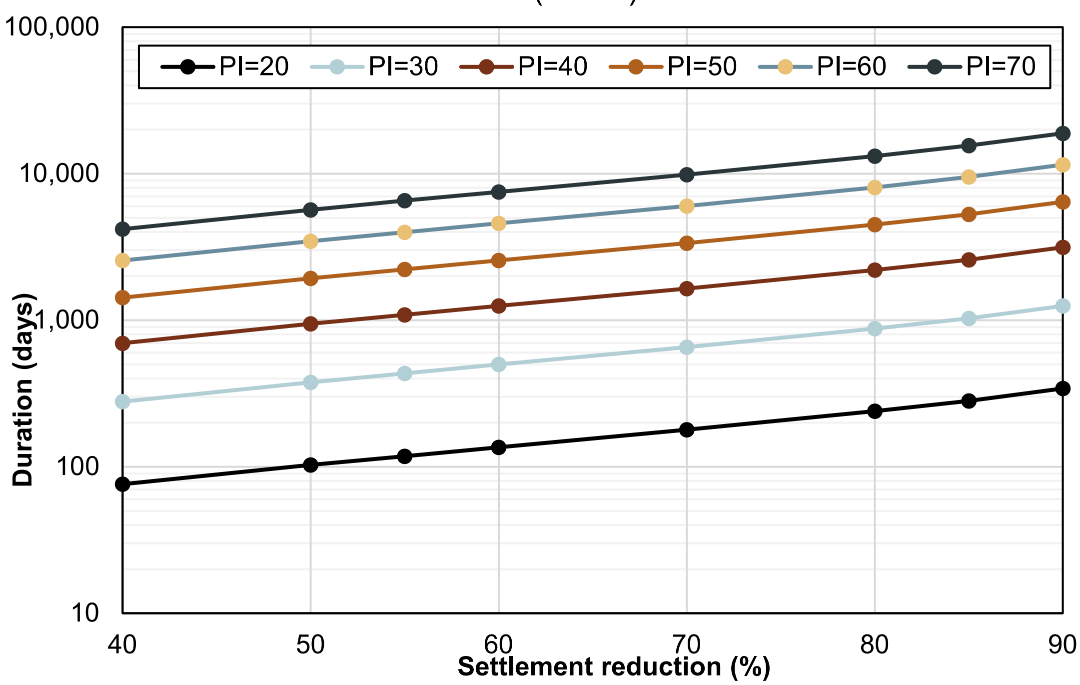


Figure 4-34. Wick drain required duration to achieve design settlement for different PIs

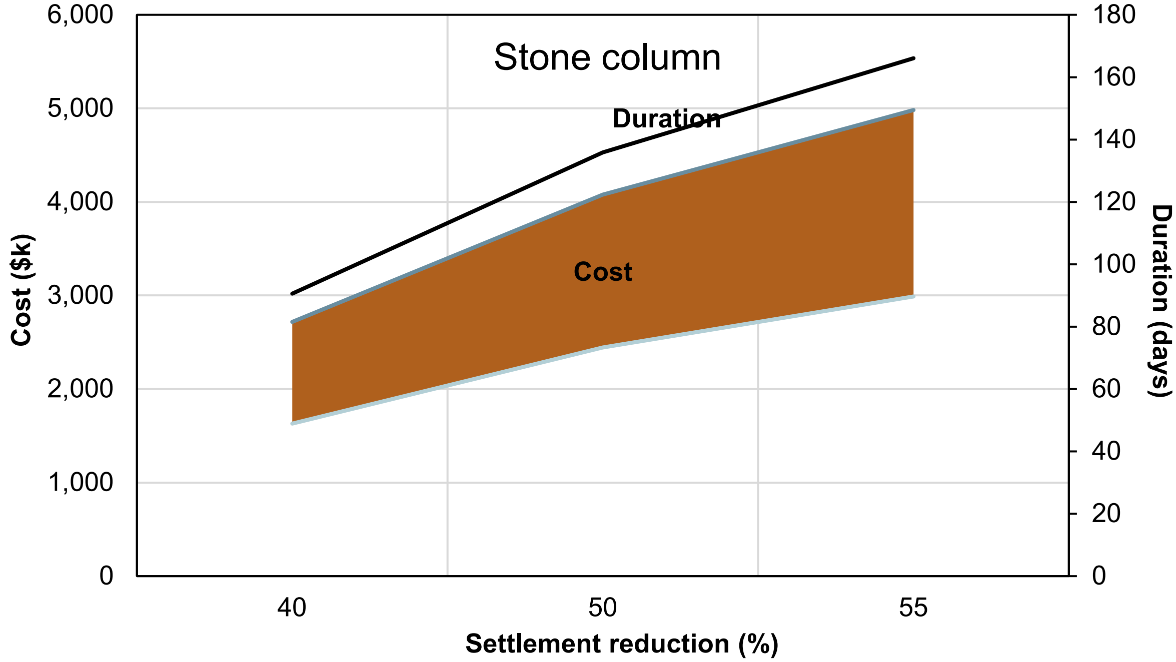


Figure 4-35. Stone column cost and required duration to achieve design settlement

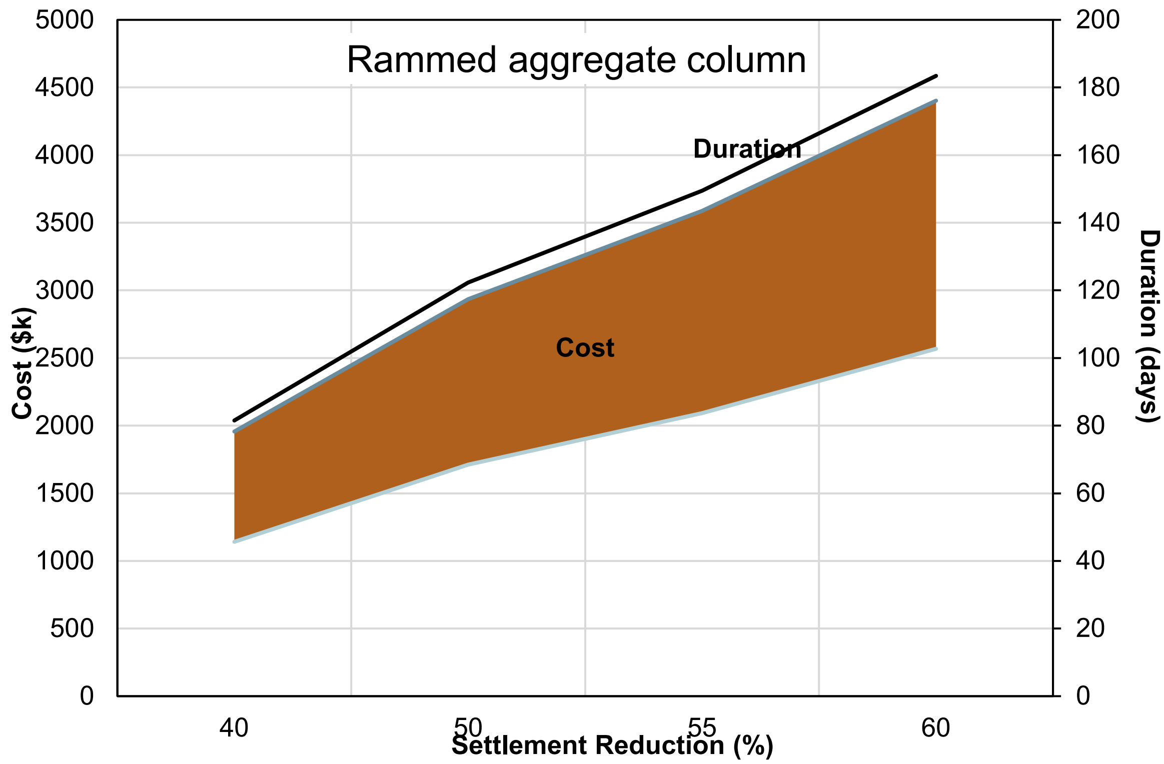


Figure 4-36. Rammed aggregate column cost and required duration to achieve design settlement

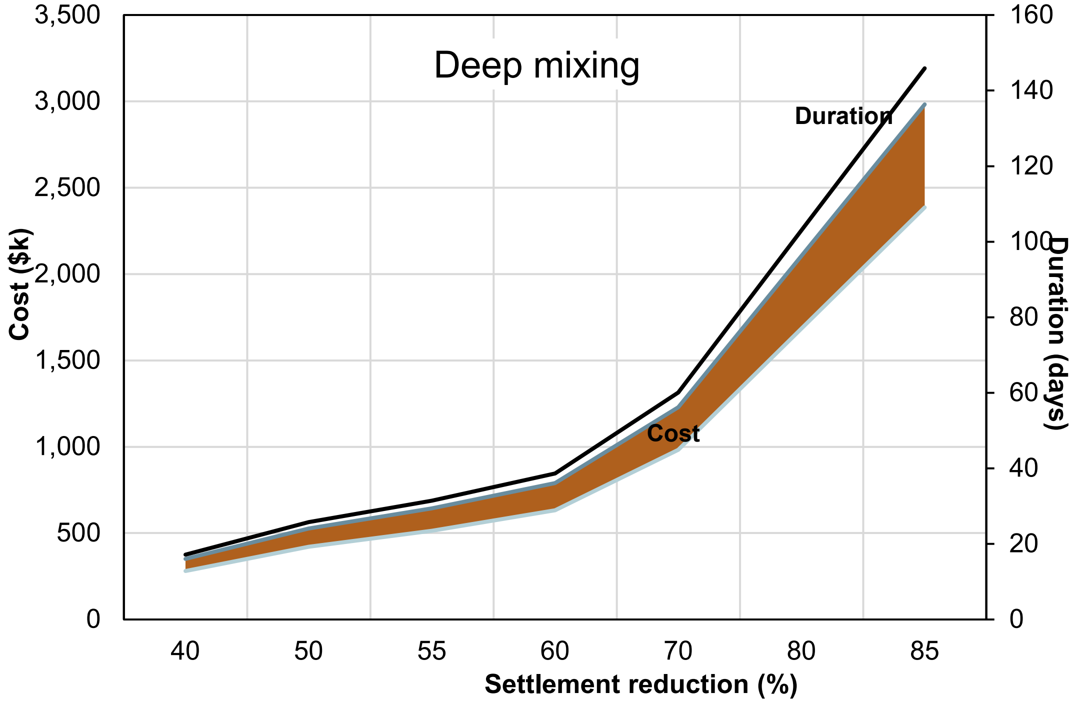


Figure 4-37. Deep soil mixing column cost and required duration to achieve design settlement

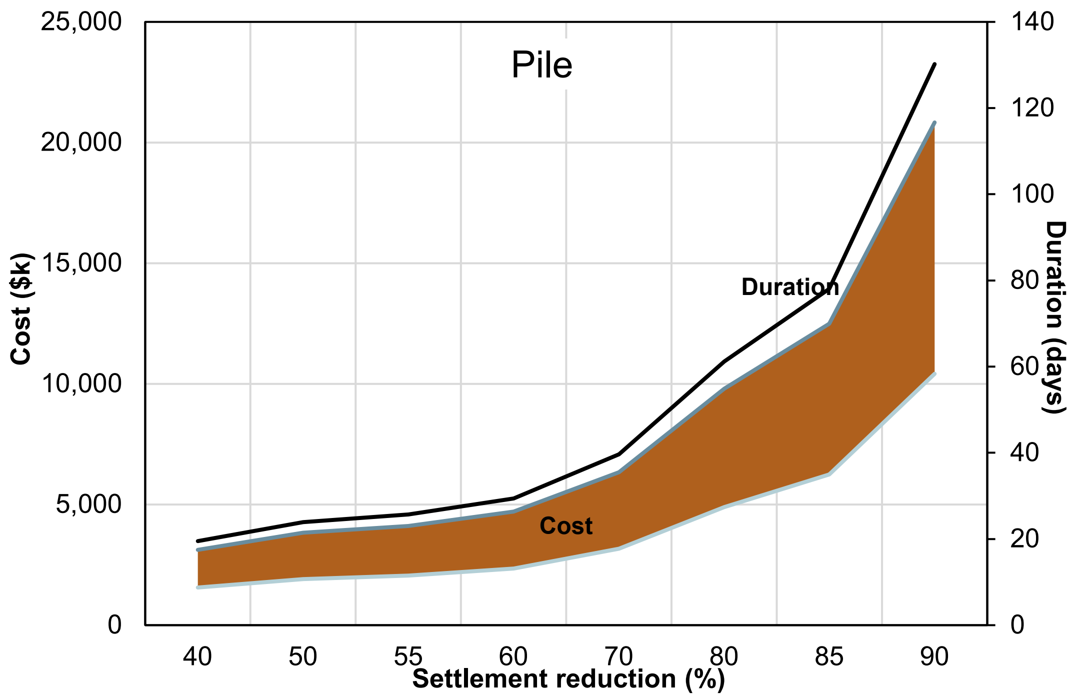


Figure 4-38. Pile cost and required duration to achieve design settlement

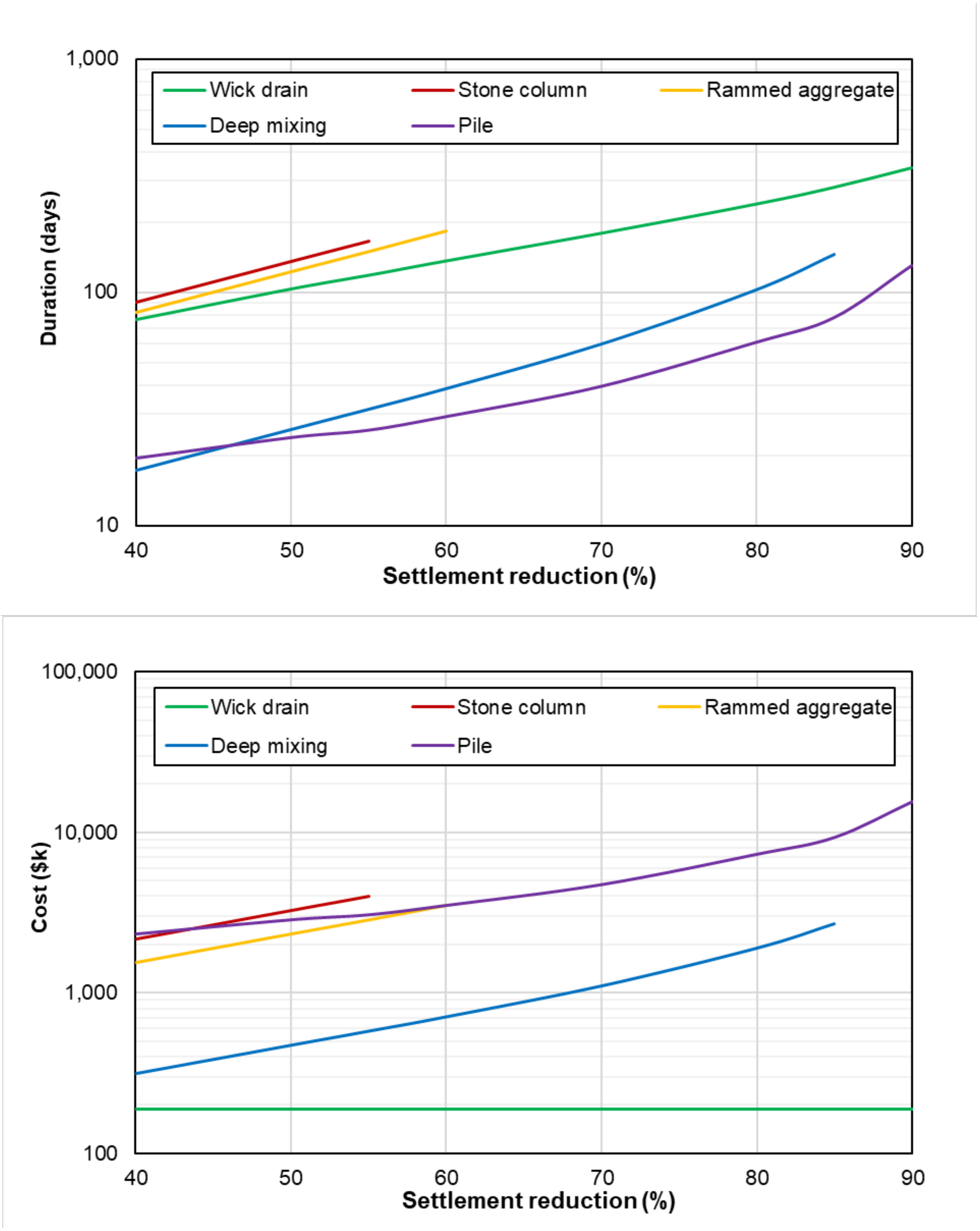


Figure 4-39. Preliminary selection chart of different soil improvement methods

4.5 DESIGN EXAMPLES

This design example is used to demonstrate how to use the developed design charts to choose the suitable soil improvement method. The below scenario is assumed:

- The untreated soil is expected to settle 4 in. after construction,
- The soil needs to support an embankment, and the post-construction settlement should be no more than 2 in. for post-construction settlement (2/4 = 50% settlement reduction),
- The time is limited to around 50 days.

Based on the above information, we shall start from top portion of the chart (Figure 4-40), which is the duration of each method. The design procedure is listed as follows:

(1) A horizontal line ① is drawn from 50 days until it intersects with the vertical line for 50% settlement reduction ($\frac{2''}{4''} \times 100\% = 50\%$).

(2) Then we find out that only deep mixing and pile need fewer than 50 days.

(3) A vertical line ② and horizontal line ③ will help us to determine the construction periods are 23 and 25 days for pile and deep mixing, respectively.

(4) The vertical line ④ brings us to the cost chart, from which you find the costs are \$500k and \$2,000k for deep mixing and pile foundation, respectively using horizontal lines ⑤. Figure 4-40 applies for soil PI is equal to 20. The design charts from PI=20, 30, 40, 50, 60, and 70 are provided in Appendix – A of this report.

If cost is more important than schedule (i.e., budget is limited), the design should start from the cost chart to determine the acceptable soil improvement method(s) and then move up to the duration chart to determine the corresponding construction duration(s) of the acceptable method(s).

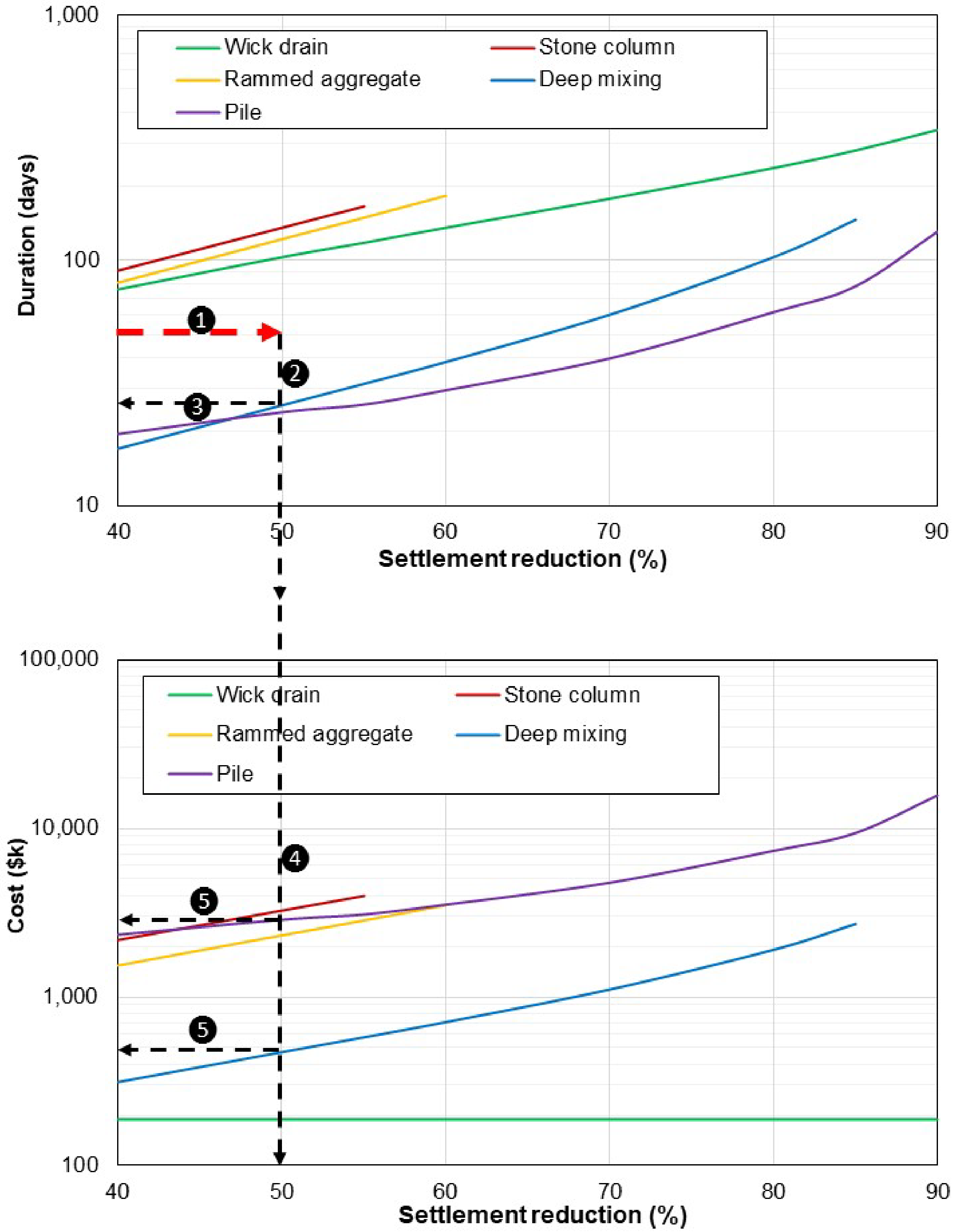


Figure 4-40. Design procedure illustration.

CHAPTER 5 CALCULATION TOOL AND DESIGN PROCEDURE

5.1 INTRODUCTION

Upon using the time-cost charts, the selection of soil improvement can be narrowed down into one or two choices. Then a systematic calculation and analysis tool will be needed to help designers for the final design and cost estimate. For such reason, an excel-based calculation tool that calculates the total settlement and determines the key design parameters is developed. The tool can evaluate commonly used soil improvement methods, which include preloading with/without wick drains, stone and rammed columns, deep mixing, lightweight fill, and piles. It has the capability of determining installation parameters (for example, diameter, depth, spacing, etc.) and construction cost for the preliminarily selected soil improvement method(s) for the given time set by users. In addition, this tool allows users to access a combination of the soil improvement methods. The general workflow of the calculation tool is shown in Figure 5-1.

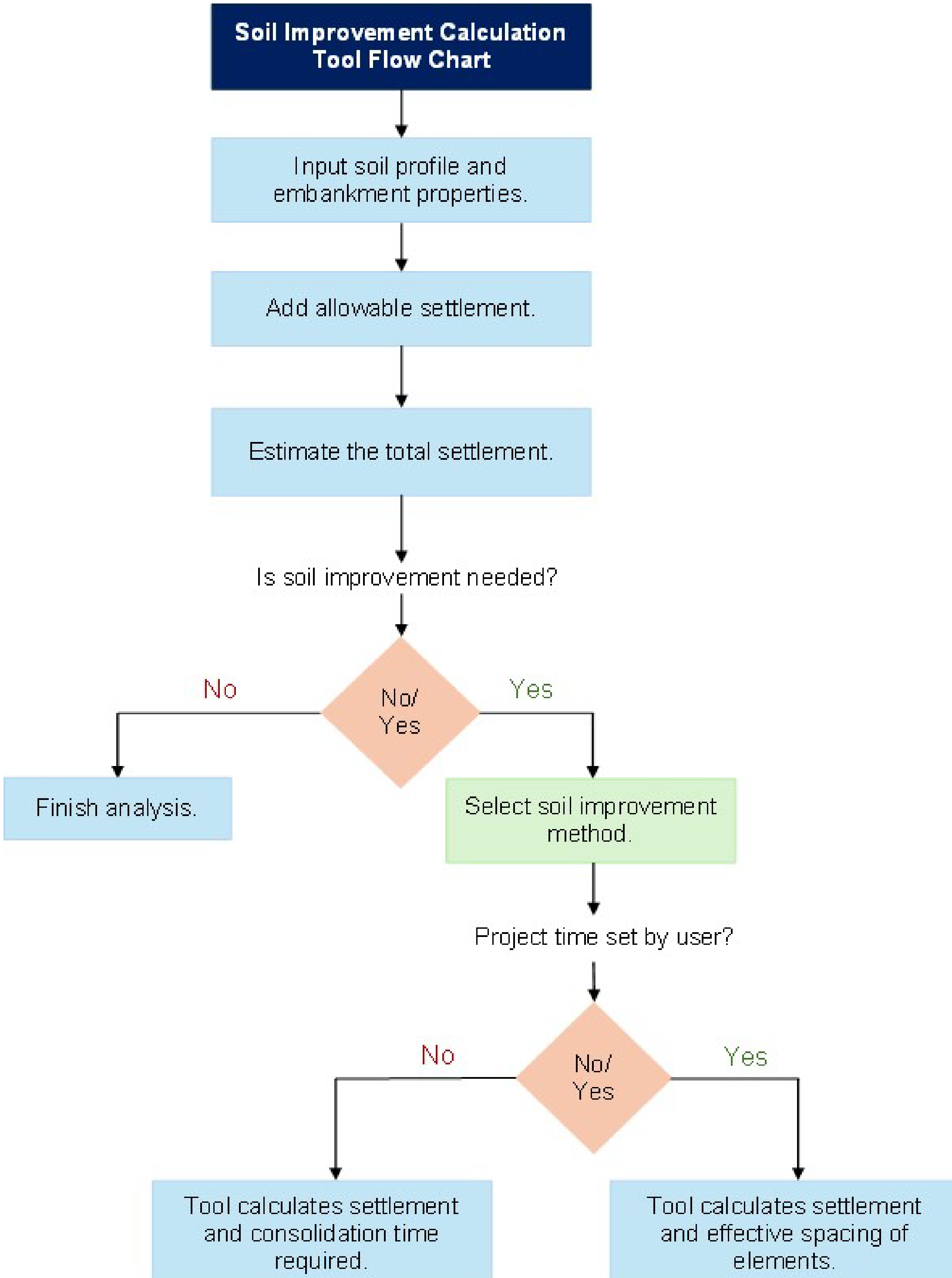


Figure 5-1. Ground improvement calculation tool general flow chart.

5.2 SETTLEMENT ANALYSIS DESIGN CONSIDERATIONS

The calculation tool has been developed and calibrated based on several design procedures and considerations. Table 5-1 exhibits parameters, correlations and values employed in the algorithm. Some of the values presented are completely integrated into the algorithm, and others are employed to estimate various soil parameters in case of their absence.

Table 5-1. Design considerations.

Derived information	Correlation	Reference(s)
Compression index (C_c)	C_c=PI/74	(F.H. Kulhawy 1990)
Swell index (C_s)	C_s=PI/370	
Pre-consolidation Pressure (σ_c)	$\sigma_c = 0.47 N_{60} Pa$	(F.H. Kulhawy 1990)
Over consolidation ratio (OCR)	$OCR = \frac{\sigma_c}{\sigma_o'}$	(Ladd et al. 1977)
Settlement tolerance for results convergence	± 0.2 inches	
Wick drains		
Equivalent diameter of a PVD (d_c)	$d_c = (b + t)/2$	(Rixner et al. 1986)
Equivalent diameter (d_e)	$d_e = 1.06dc \quad triangular\ pattern$ $d_e = 1.13dc \quad square\ pattern$	(Han 2015)
Coefficient of radial consolidation (C_h)	$C_h = 1.5Cv$	
Horizontal permeability (k_r)	$k_r = \frac{Ch}{Cv} k_v$	
Diameter of smear zone (d_s)	$d_s = (1.5 - 3)dc$ $d_s = 2dc \ (used)$	(Hansbo 1981b)
Permeability of smear zone (k_s)	$k_s = \frac{1}{\lambda} kr \quad \lambda = (2 - 6)$ $k_s = \frac{1}{\lambda} kr \quad \lambda = 4 \ (used)$	(Han 2015)
Stone (SC) and Rammed Aggregate Columns (RAC)		
Equivalent diameter (d_e) (SC, RAC, deep mixing)	$d_e = 0.907dc\ triangular\ pattern$ $d_e = 0.785dc \quad square\ pattern$	(Han 2015)

Area replacement ratio (a_s)	$a_s = (0.1 - 0.4)$	(Han 2015; Vernon R Schaefer 2016a)
Stress concentration ratio (n)	$n = (2\ to\ 5\ )\quad SC, \quad n = 5\ (used)$ $n = (5\ to\ 10\ )RAC, \quad n = 7\ (used)$	(Han 2015; Vernon R Schaefer 2016a)
Permeability of smear zone (k_s)	$k_s = \frac{1}{\lambda} kr \quad \lambda = (2 - 6)$ $k_s = \frac{1}{\lambda} kr \quad \lambda = 5\ \ (used)$	(Han 2015)
Deep Mixing		
Stress concentration ratio and column to soil elastic modulus ratio	$\left(\frac{n}{Ec/Es}\right) = 0.625$	(Jiang et al. 2013a)

5.3 USER INTERFACE OF THE SOIL IMPROVEMENT CONSOLIDATION SETTLEMENT TOOL

5.3.1 Start page

Figure 5-2 shows the general view of the start page. The Consolidation Settlement Tool Start Page is divided into two sections. Section 1 is the Soil Parameters Input section while Section 2 is the Settlement Results section.

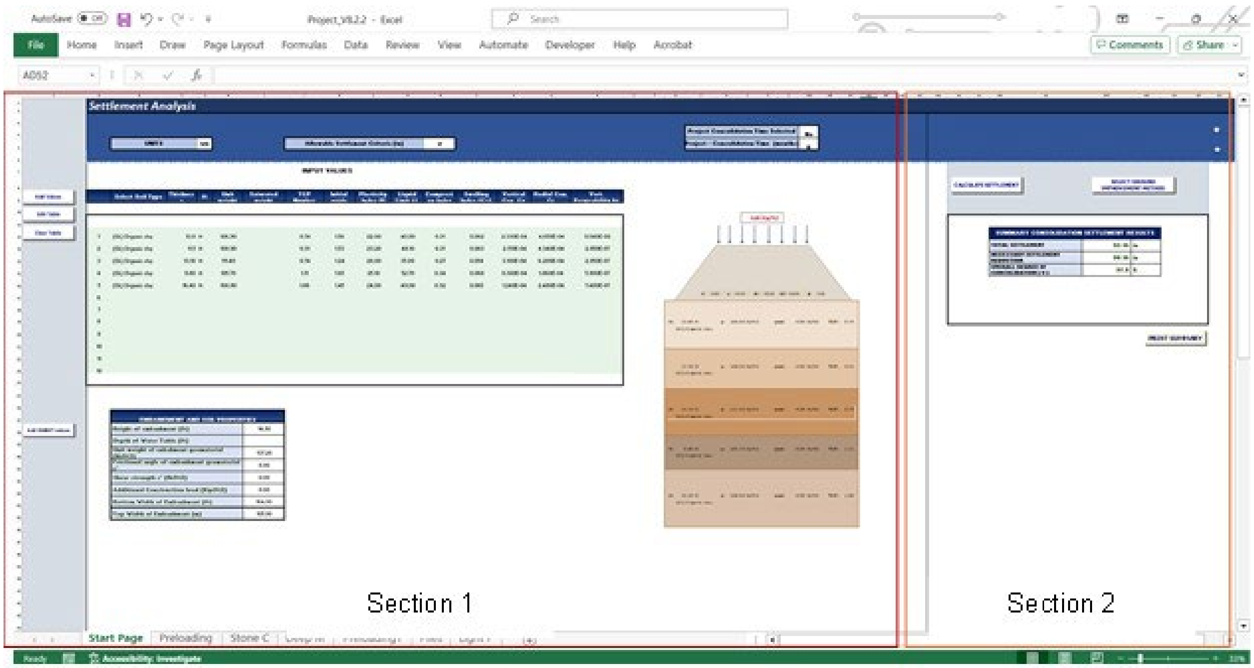


Figure 5-2. The start page.

5.3.1.1 Section 1: Soil Parameters Input.

The soil parameters input section located on the ***Start Page*** is where measurement units, allowable settlement criteria, properties of the soil profile, and embankment properties are input by users, and are visualized graphically. Figure 5-3 shows the input section (i.e., Section 1) on the ***Start Page,*** which are numbered from 1 to 11 in the figure for an easy illustration purpose and are discussed in order below:

1. The ***UNITS*** cell shows the measurement units system of the current project.
2. ***Allowable Settlement Criteria*** cells display the total settlement allowed for the current project.

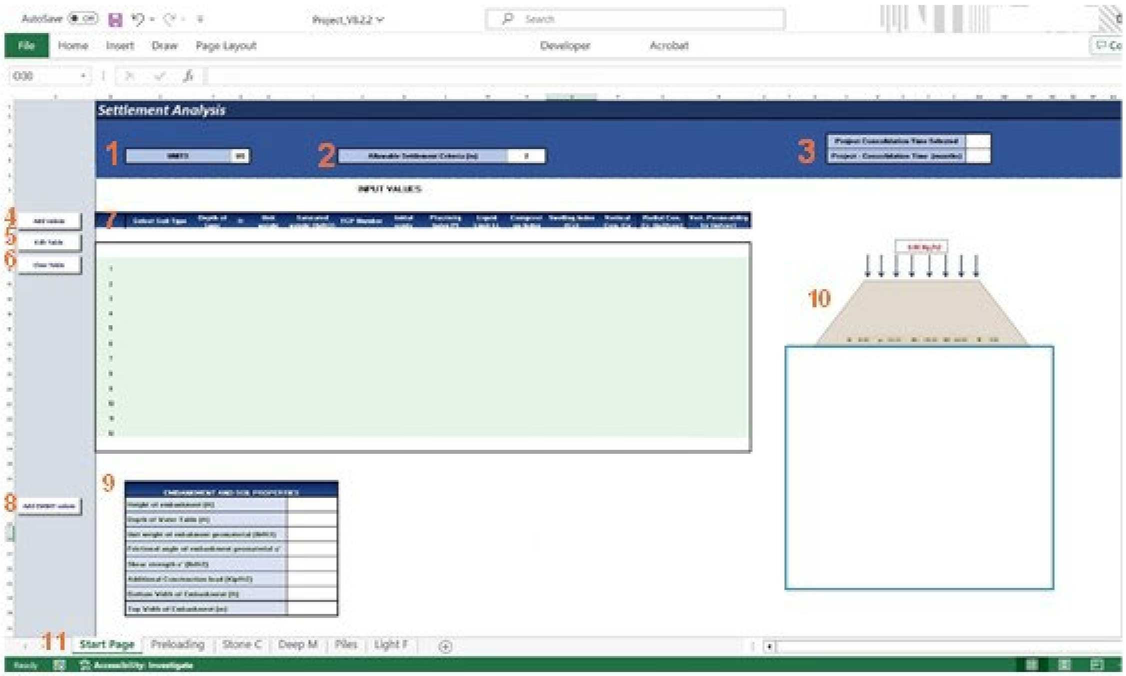


Figure 5-3. Soil improvement calculation Tool *Start Page*.

3. The ***Project Consolidation Time Selected*** is a Yes/No cell that shows either if users decide to set the time of the project or not. ***Project-Consolidation time*** field indicates the duration, in months, needed to reach consolidation settlement using ground improvement method. Based on the (Yes/No) decision, two cases can be derived. **Case I:** time set **by users** for the soil improvement analysis and the tool will determine installation details, so the total time is within the time given by the user. **Case II:** if initial value shown will be of **zero (0),** the tool will calculate the time needed during the analysis. Figure 5-4 shows an augmented view of the consolidation settlement time display for case II.

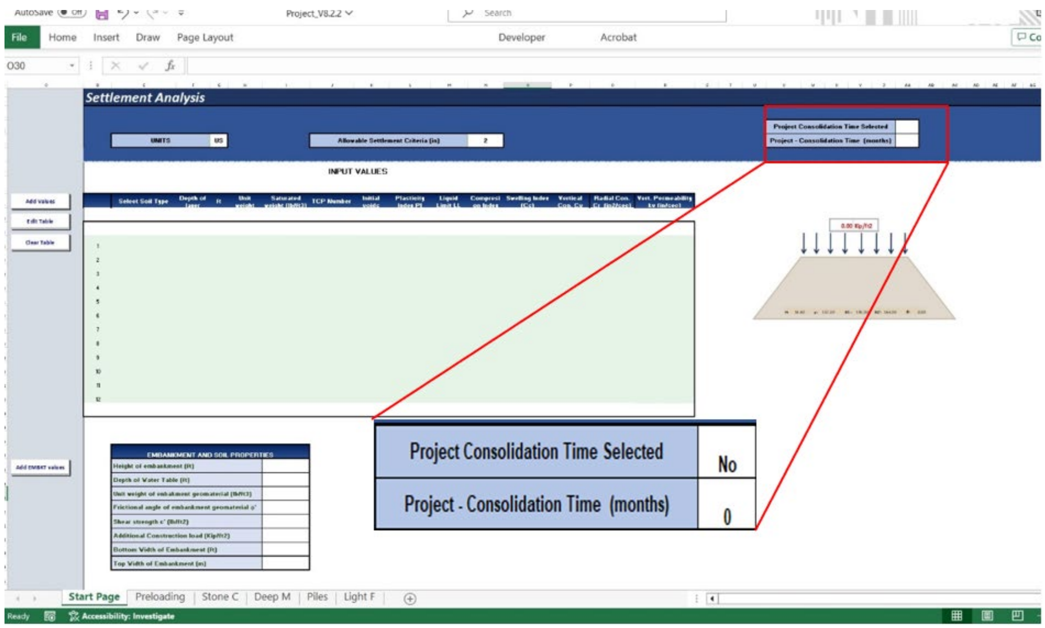


Figure 5-4. Consolidation settlement time.

4. ***The Add Values*** button is used to add the properties of the soil profile to the project. The user can add up to 12 layers of soil.
5. ***The Edit Table*** button is to edit the soil profile previously entered by the user.
6. ***Clear Table*** button clears off all the values that are currently set for the soil profile and embankment properties.
7. ***Soil profile Table*** displays the information of the soil profile after values are added with ***The Add Values*** button.
8. ***Add EMBKT Values*** button opens a user form where embankment parameters should be added.
9. ***The embankment and backfill properties table*** display the values of the embankment set by users.
10. ***Cross-section graphically displays*** the embankment and the soil profile. Figure 5-2 shows what the page looks like after data inputs are completed.

11. These taps allow user to switch other module of the worksheets, which are available for this tool.

5.3.1.2 Section 2: Settlement and soil improvement analysis

In Section 2, users can start the settlement analysis and subsequently select any ground improvement method if needed. Figure 5-5 shows Section 2 on the ***Start Page*** and each part of it is numbered and discussed here briefly.

1. ***CALCULATE SETTLEMENT*** button is used to run the settlement analysis based on the given allowable settlement in Section 1. At this stage, soil improvement has not been considered, namely, the tool calculates the pre-improvement total settlement. This settlement value will allow users to decide if soil improvement is needed.

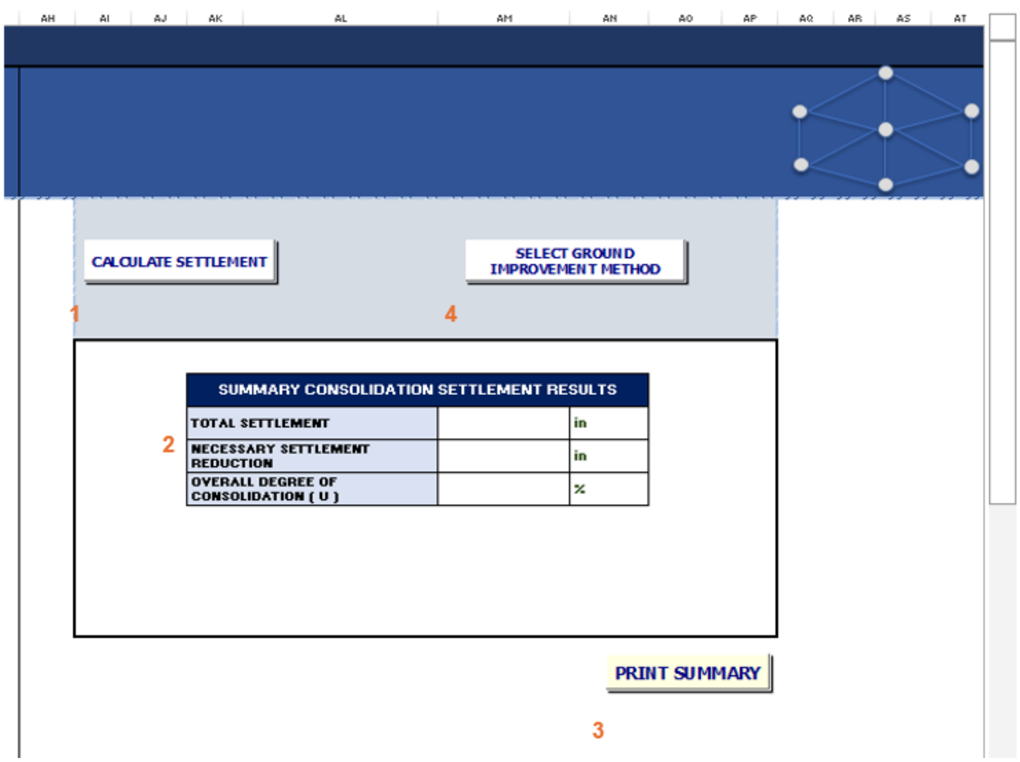


Figure 5-5. Start Page. Settlement and soil improvement analysis section.

2. ***Summary consolidations settlement results.*** The summary table presents the settlement results before ground improvement of the given soil profile.
3. ***Print Summary button*** allows users to print the settlement results.
4. ***SELECT GROUND IMPROVEMENT METHOD.*** If the calculated settlement exceeds the allowable value, the users can choose to continue the analysis by selecting a ground improvement method to mitigate the excessive settlement. Once this button is clicked, a dropdown menu will appear, and the user can choose a soil improvement method to proceed the analysis.

5.3.2 Soil improvement method interface (preloading and wick drains)

Preloading with wick drain, stone columns, rammed aggregate columns, deep mixing columns, piles, and lightweight fill are the soil improvement methods included in this tool. Even though the selected soil improvement method varies, execution of analysis is similar for all soil improvement methods included in this tool. First, once a soil improvement is selected, the calculation of the settlement for the selected soil improvement will be activated. Then the tool will take necessary soil properties and embankment information from the start page to perform calculations. Finally, results will be presented in a table and a graph. The interface of every method is similar in appearance, and hence, in Figure 5-6, is shown the results page of preloading with wick drains, for demonstration purpose.

1. ***Project-Consolidation time (months).*** Once the analysis is complete, the required time of the project to complete the consolidation settlement is shown in this box.

2. ***Method employed.*** Shows the soil improvement method in which the analysis was executed.
3. ***Installation pattern*** is a diagram that shows the installation pattern chosen by users or set by the default options of the tool (triangle pattern), as shown in Figure 5-6. In addition, the spacing of the soil improvement elements, which could be wick drain, stone columns, aggregate columns, piles, depending on the selected soil improvement method.

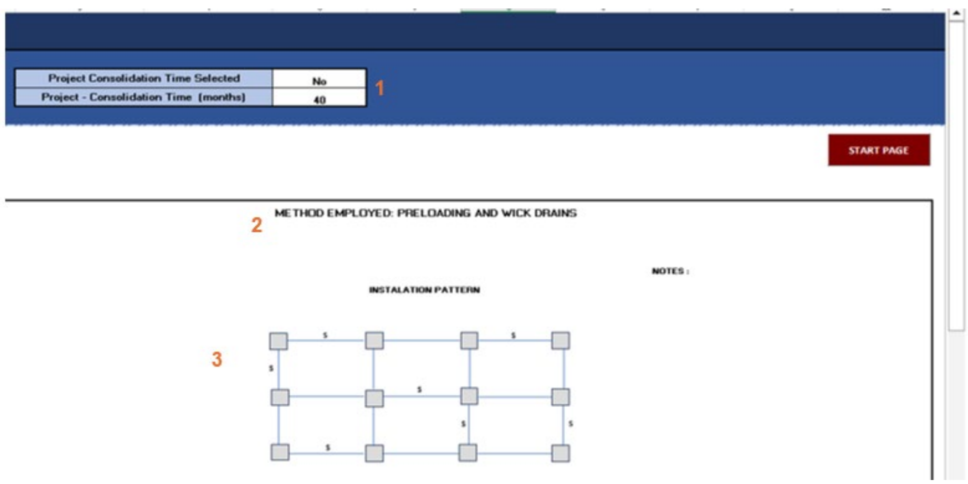


Figure 5-6. Soil improvement method results interface. Installation pattern diagram.

4. ***Time vs. Consolidation Settlement chart and table.*** In this chart and table as shown in Figure 5-7, the values of consolidation settlement with respect to the time are presented after completion of the analysis. The users can utilize the curve to gain additional information, such as the progress of settlement with time, etc.

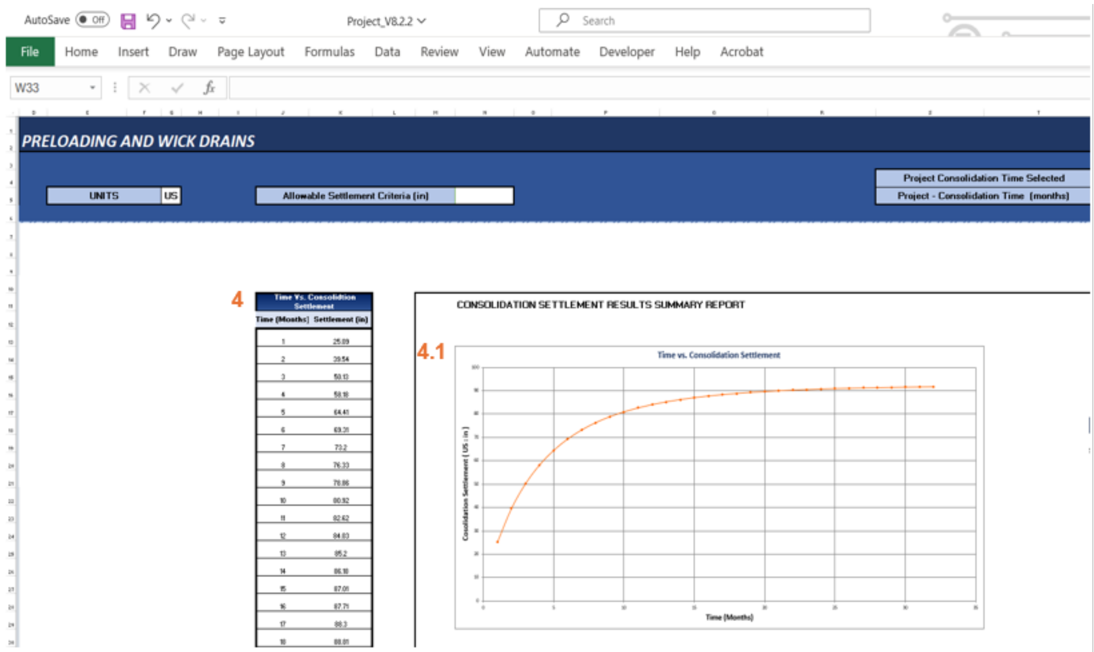


Figure 5-7. Soil improvement method result interface.

5. ***Section 5*** summarizes important settlement information such as total settlement, settlement achieve by the soil improvement method, and the overall degree of consolidation, as shown in Figure 5-8.

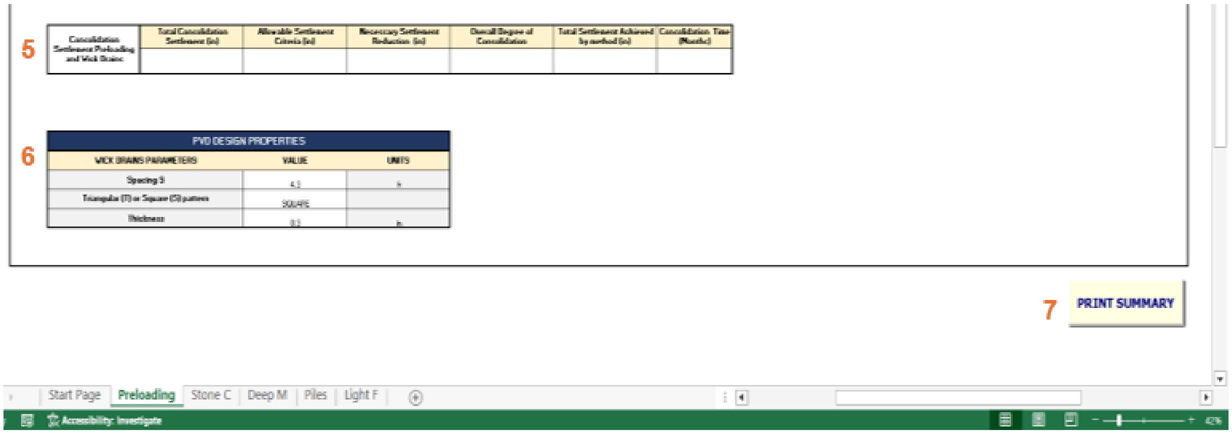


Figure 5-8. Soil improvement method result interface and design parameters.

6. ***Design Properties.*** In this section is summarized the design parameters of the soil improvement elements.
7. ***Print Summary button*** allows users to print the settlement results after ground improvement method has been selected.

5.4 USING THE SOIL IMPROVEMENT CONSOLIDATION SETTLEMENT TOOL.

This section aims to explain the process on how to use the calculation tool to estimate the total settlement of the foundation soil. An example is used to explain the procedure step-by-step.

5.4.1 Analysis example

For this example, the ***total settlement*** is calculated following by the ***consolidation settlement time required*** using the calculation tool. The ground improvement method employed: ***preloading and wick drains***. The allowable settlement is two inches (2”).

Properties and design parameters of PVDs are shown in Table 5-2. An embankment with a bottom width of 165 feet and slope of 2:1, 14.8 feet tall, unit weight γ = 137.2 lb/ft^3 is to be constructed above the soil profile shown in Table 5-3. No Ground water table was observed. Data are taken from Shang et al. (1998).

Table 5-2. PVDs design properties.

PVDs Design Properties	
Width (inches)	4
Thickness (inches)	0.20
Spacing (feet)	4.3
Installation Pattern	Square

Compression index (C_c), swelling index (C_s) were estimated with the equation proposed by (F.H. Kulhawy 1990). In addition, Coefficient of vertical consolidation (C_v) was calculated with values of volume coefficient of compressibility (m_v), and vertical permeability (k_v). The coefficient of horizontal consolidation C_h = 2C_v.

Table 5-3. Properties of soil profile.

Soil Type	Thickness (ft)	Unit weight (lb/ft3)	TCP	Voids ratio (eo)	PI	LL	Cc	Cs	Vertical Con. Cv (in2/sec)	Radial Con. Cr (in2/sec)	Vert. Permeability kv (in/sec)
(OL) Organic clay	11.5	108.9	0.31	1.53	23.2	48.1	0.31	0.063	2.17E-04	4.34E-04	2.95E-07
(OL) Organic clay	13.1	111.4	0.74	1.24	20.0	35.0	0.27	0.054	3.10E-04	6.20E-04	2.95E-07
(OL) Organic clay	9.8	105.7	1.11	1.65	25.1	52.7	0.34	0.068	9.30E-05	1.86E-04	5.90E-07
(OL) Organic clay	16.4	108.9	1.86	1.45	24.0	49.0	0.32	0.065	1.24E-04	2.48E-04	7.48E-07

5.4.1.1 Entering soil profile properties and selecting the units.

1. Click on the ***Add Values button*** (see Figure 5-9) to start entering the soil profile properties and to set the units system. A dialog box will open asking the number of soil layers that compose the soil profile (Figure 5-10).

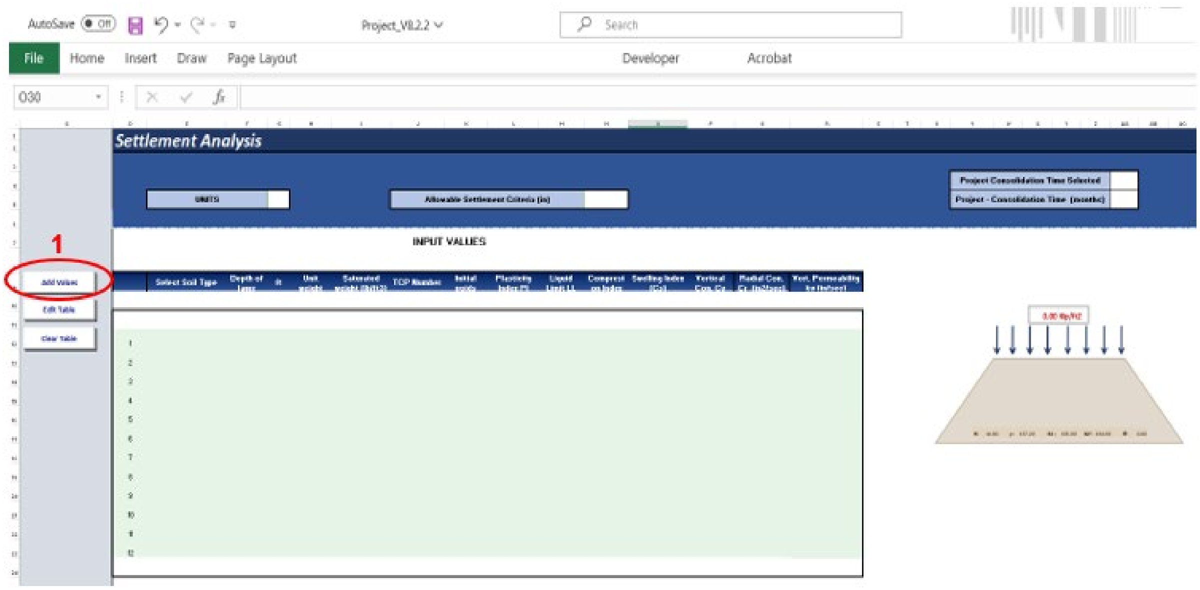


Figure 5-9. Entering soil profile parameters.

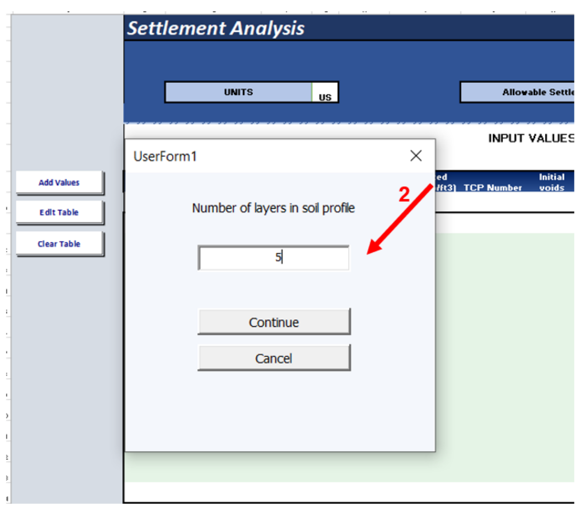


Figure 5-10. Entering number of soil layers.

- Note: in case the soil profile table is not empty (from a previous analysis), click on the ***clear table*** button. This action will clear all values from the tables.

2. Insert the number of soil layers within the soil profile and press continue. The tool allows users to enter a maximum of twelve (12) layers. **Enter five (5) as the number of soil layers in soil profile** (Figure 5-11).
3. ***In the Insert Soil Profile Data*** dialog box (Figure 5-11), select the unit system from the drop-down menu (US, SI). **US units' system for this example**.

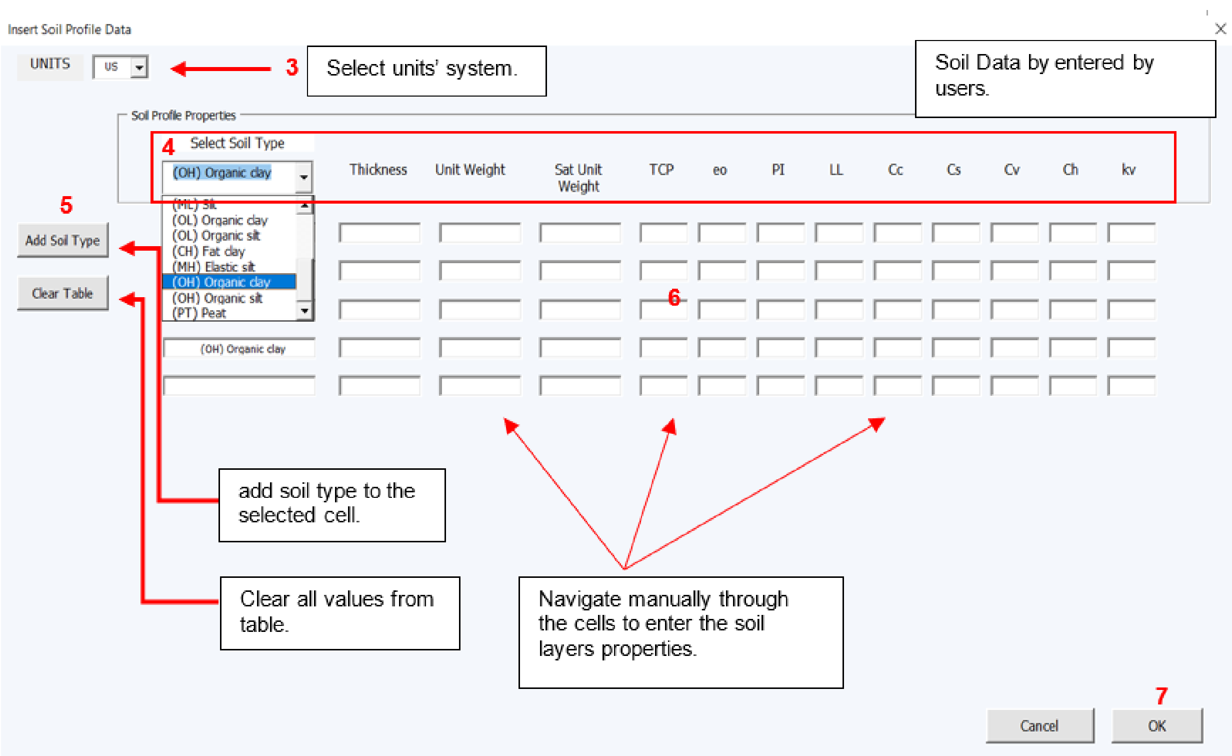


Figure 5-11. Soil parameters input form.

4. ***From the Soil Type*** drop-down menu, select the type of soil for each layer and then click on the cell (first column) of the layer the type of soil will be added.
5. After selecting the cell is the soil type to be added, click on the ***Add Soil Type button.*** This action will add the description of the soil type to the desired cell.

6. **Proceed to enter the soil profile information presented in Table 5-3** and navigate through the cells and by pressing tab or using the keyboard arrows to add the respecting values for each layer. Figure 5-12 shows the entered soil data based on Table 5-3.

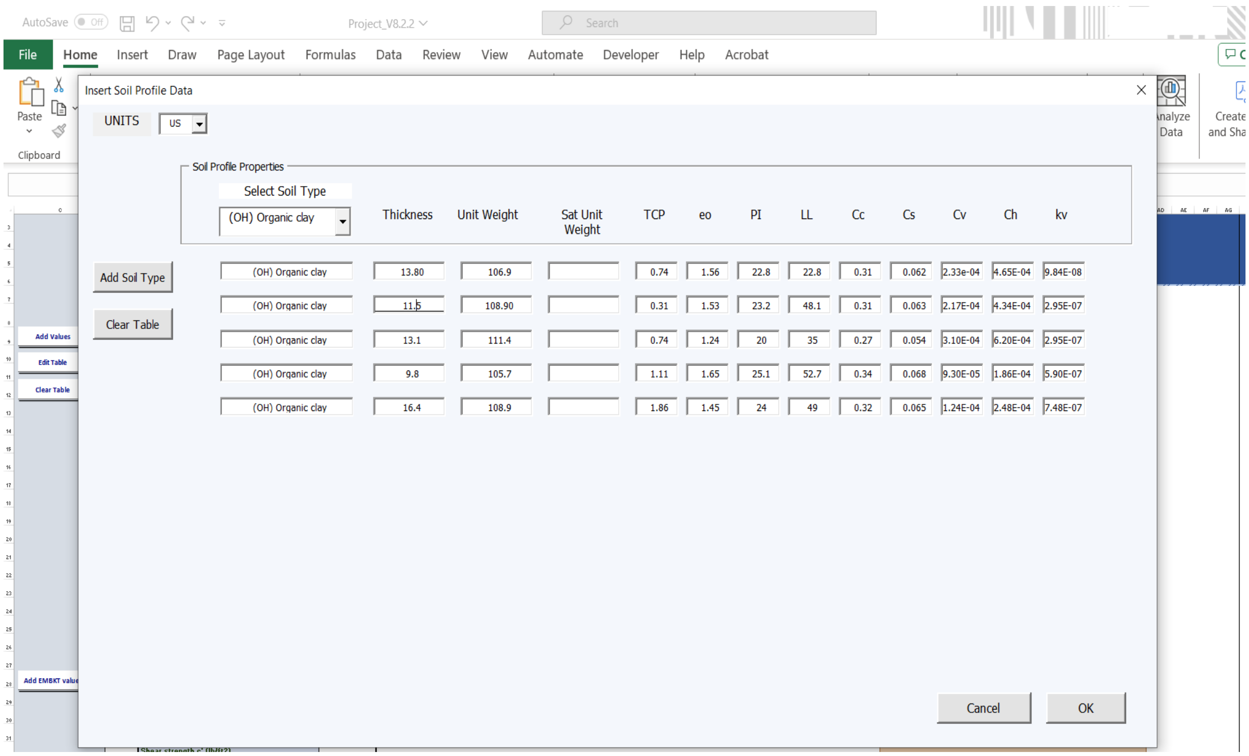


Figure 5-12. Analysis procedure: soil properties input form.

7. Once all soil data is entered, press continue. A pop-up message (Figure 5-13) will appear indicating the tool will estimate the values for horizontal coefficient of consolidation (Ch) as 1.5 times the coefficient of vertical consolidation (Cv), (C_h = 1.5C_v) (Vernon R. Schaefer, 2016) in case values for C_h has not been entered.

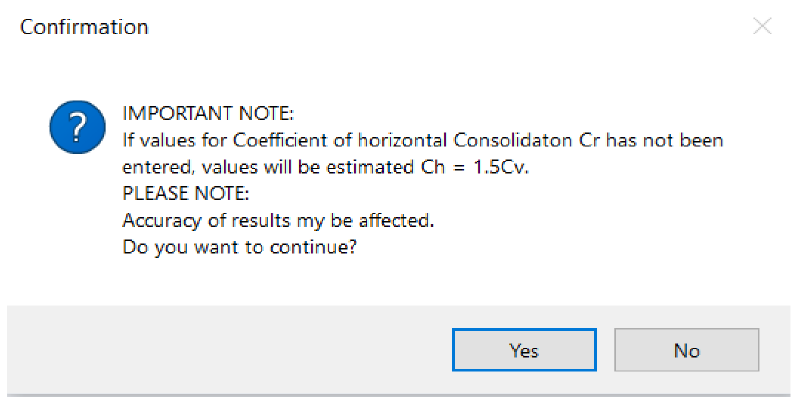


Figure 5-13. Coefficient of radial consolidation: Pop-up warning.

8. Press ***Yes*** to continue. All data added should be visible in the in the ***Soil profile table*** of the start page and the soil profile Figure 5-14 will show every layer with its properties. In addition, the compression index (Cc) and the swelling index (Cs) will be estimated with equation proposed by Kulhawy and Mayne, 1990 (see Table 5-1) in the event users do not provide these values.

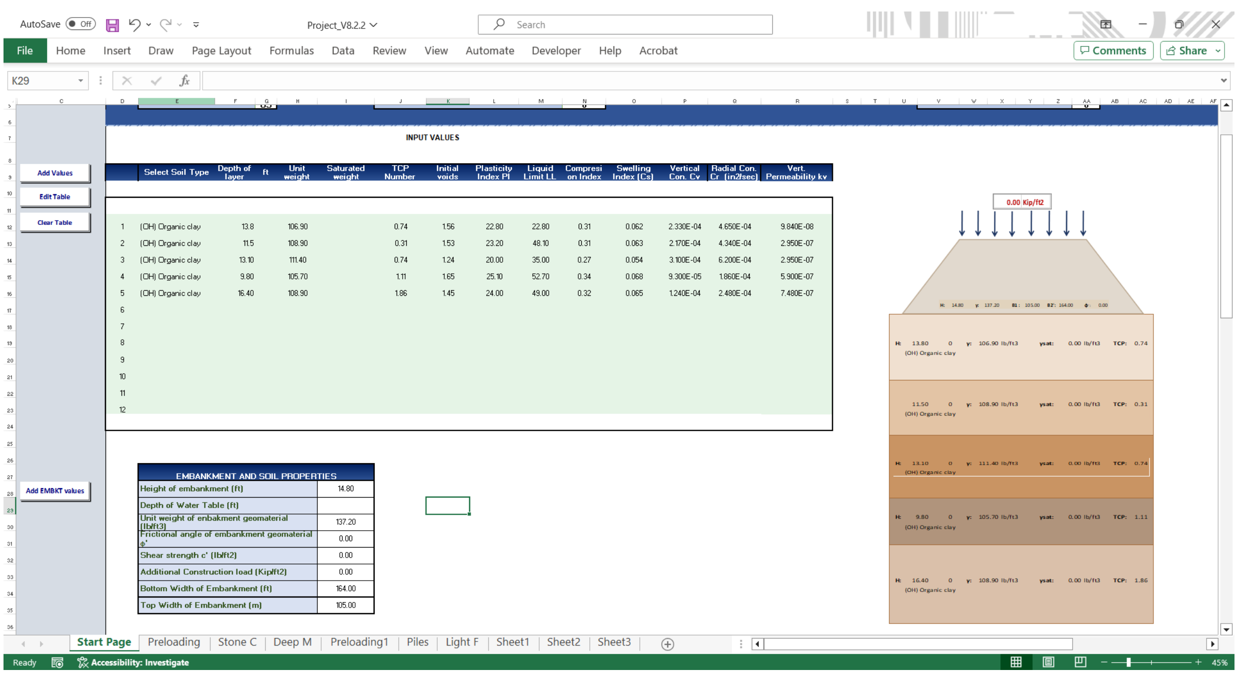


Figure 5-14. Analysis procedure: soil profile properties table and diagram.

5.4.1.2 Entering embankment properties.

1. Press the ***Add EMNKT values*** button. An embankment properties dialog box, shown in Figure 5-15 where the properties of the embankment must be entered, will be displayed. For the settlement analysis, friction angle and shear strength are not required. In addition, if a retaining wall is to be analyzed, the same value for the top and bottom width can be entered or with of the wall could be added either in the bottom or top width of the embankment field.

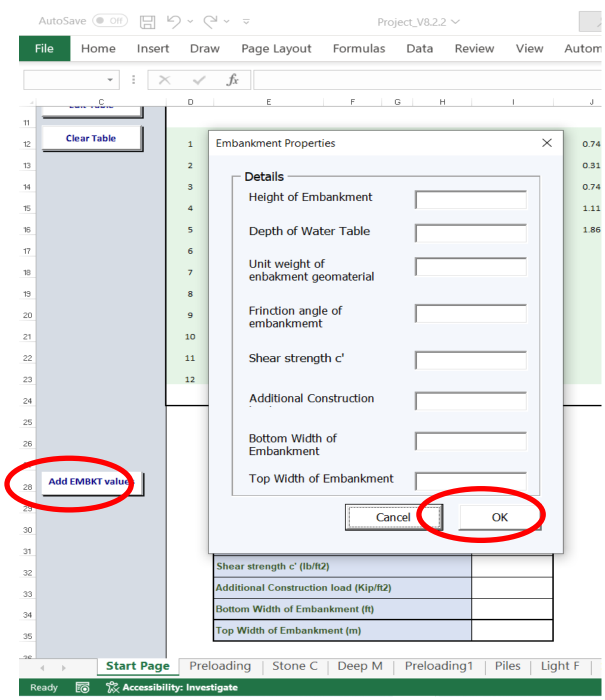


Figure 5-15. Embankment properties input form.

2. **Add the embankment backfill properties**. As explained earlier, no ground water table was encountered for this example, hence **leave the water table box empty as shown in** Figure 5-16. **If zero is entered instead, the tool would take it as if the ground water table were at the top of the soil profile.**

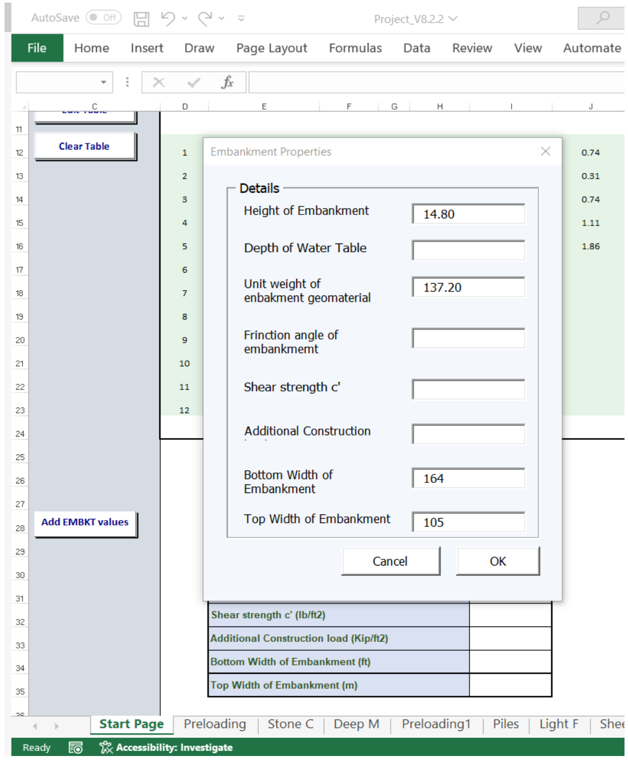


Figure 5-16. Embankment properties.

3. Once the values are added, press ***OK*** to continue. The soil profile table and graphic representation of the soil profile will appear as shown in Figure 5-14, and it will show the entered data. Once checked all values are correct the settlement analysis can be started.

5.4.1.3 Running Settlement Analysis.

Once the required soil profile and embankment properties information has been entered, the calculator is ready to start the settlement analysis. In the second section of the ***Start Page*** (Figure 5-17).

1. Click on the ***CALCULATE SETTLEMENT*** button and a new dialog box will appear. This dialog box will require users to enter the allowable settlement for the structure analyzed.
2. Enter the allowable settlement for the project **(2 inches**) and click on ***RUN.***

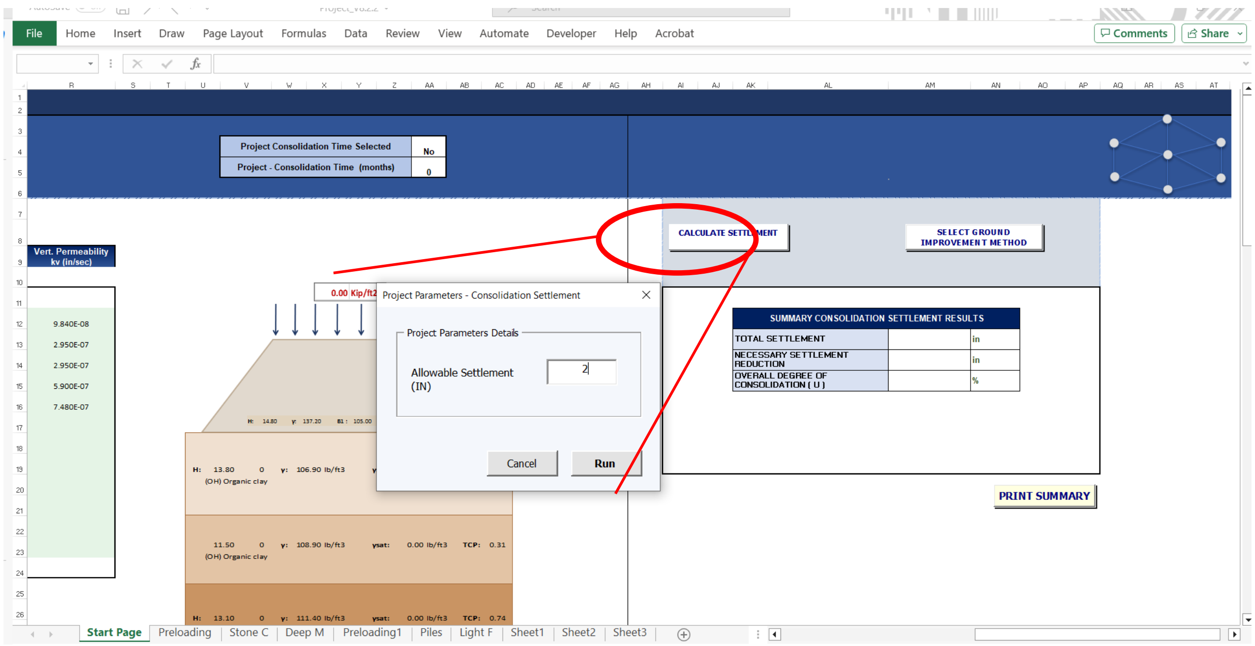


Figure 5-17. Allowable settlement and total settlement analysis.

3. Once the analysis is completed a new message will appear on the screen (Figure 5-18) indicating the total settlement, necessary settlement reduction, and the overall degree of consolidation. Click ***OK*** to continue. In addition, these values will be available in the ***Summary consolidation settlement results.*** For this example, a total settlement of **92.16** inches was obtained and a necessary settlement reduction of **90.16** inches representing a settlement reduction of **97.8 %.**

- Table results can be easily printed by clicking on the ***Print summary*** button.

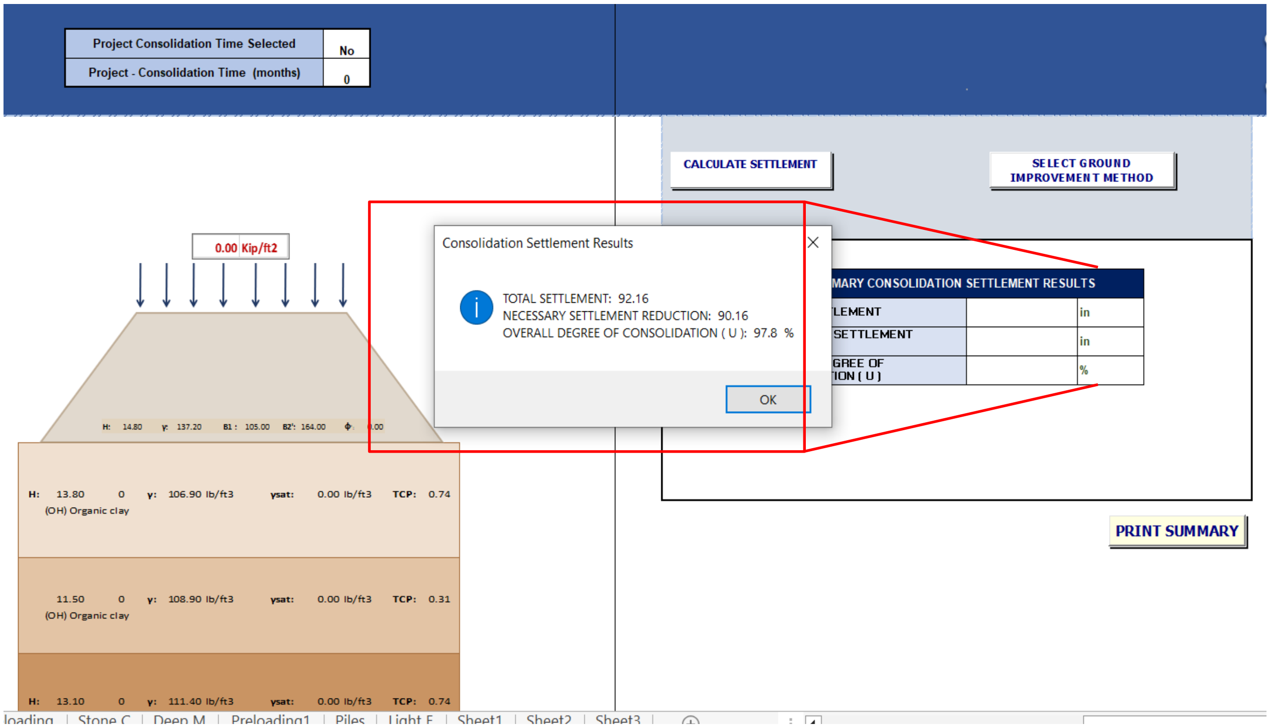


Figure 5-18. Total settlement analysis results window.

5.4.1.4 Soil Improvement Settlement Analysis.

Soil improvement settlement analysis, as explained in Section 5.3.1.2, is executed from the Start Page and it will redirect users to the soil improvement method they select from the menu. Once the 5.4.1.3 is finished and the settlement analysis without soil improvement is complete. The user can start analysis with soil improvement. The following procedure explains *Preloading and Wick Drains* method. Each method follows a similar process with slight differences depending on the design parameters users must enter. The procedure is as follows:

1. Click on ***the Select ground improvement method*** button. This button will automatically display a ***Ground Improvement*** dialog box (Figure 5-19).

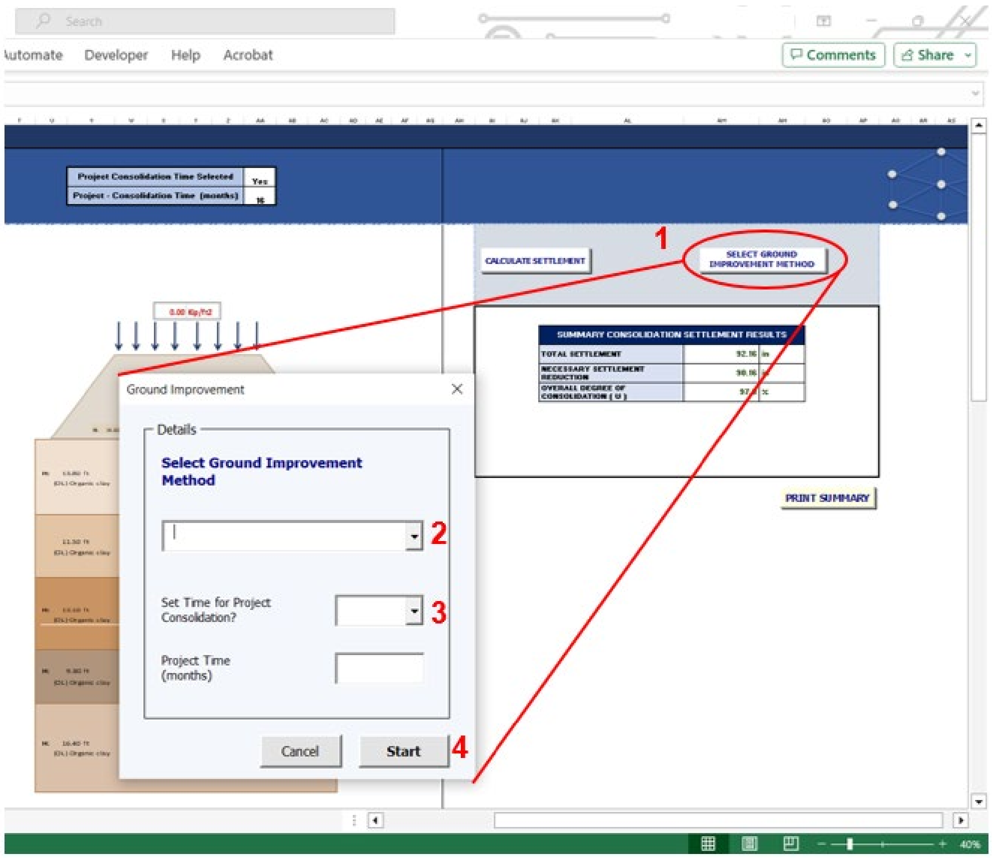


Figure 5-19. Soil improvement method and project time selection.

2. Select the soil improvement method from the dropdown menu **(Preloading and wick drains for this example)**.

- Select (Yes/No) depending on if the time is to be set by the user or not. If the time is to be set by the user, **select Yes.** The tool will estimate the optimal spacing of the soil improvement elements and the settlement. If otherwise, **select No** and the tool will calculate the consolidation time required and the settlement. **In this example, it will ask the tool to estimate the required and the settlement achieved by the method**. Therefore, Yes is selected from the menu and the ***<u>Project Time</u>*** box is left in blank (Figure 5-20).

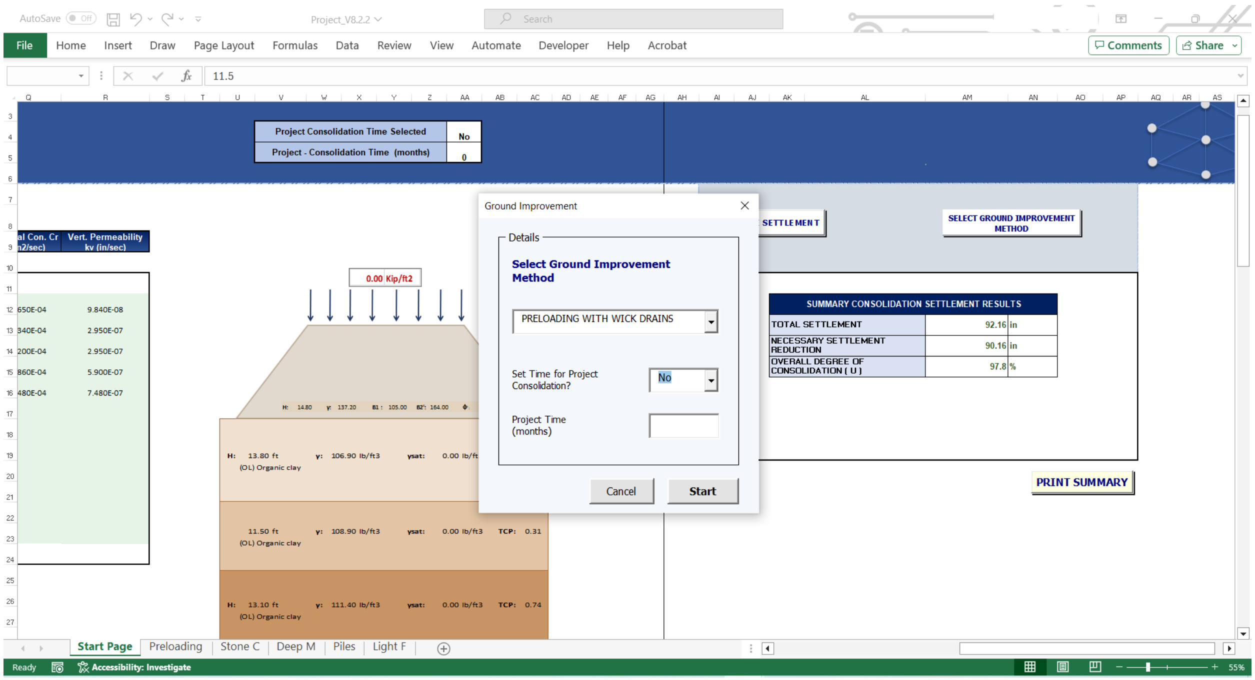


Figure 5-20. Analysis procedure: GI Soil improvement method selection.

- Note: In case the project time is set to ***yes,*** but the last box is left blank, a warning will appear requesting users to enter the time (Figure 5-21).
-

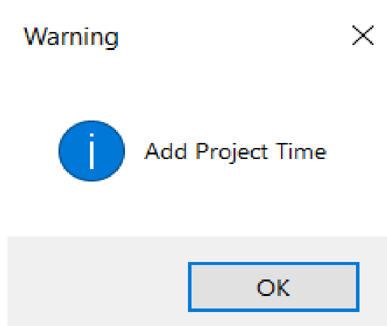


Figure 5-21. Project time warning message.

3. Click on ***Start*** to begin the soil improvement settlement analysis. The tool will automatically activate the results page of the soil improvement method selected.

5.4.1.5 Soil improvement analysis page (Case: preloading and wick drains).

In the previous section it was explained how to start the consolidation settlement analysis employing a soil improvement method. This section aims to illustrate the additional steps to complete the analysis. Once on the worksheet of soil improvement method selected, a user form will show up and require some design parameters (user defined).

Depending on the selection made in Step 1 of Section 5.4.1.4 Soil Improvement Settlement Analysis., two cases are expected. **First case**: consolidation time and settlement are to be estimated by the tool based on the input wick drain spacing. **Second case:** installation spacing of the soil improvement elements and settlement are to be estimated based on the project's timeline.

1. Enter the PVD's thickness. The tool takes three-eight (3/8") inch as default thickness if not input by the user.
2. Spacing. **First case**: it is required to be entered by users. However, four (4') feet is set as default value, in case spacing is not entered. The **second case** spacing input is not available (see Figure 5-22). The tool estimates the spacing in multiples of 0.5 feet (e.g., 3.0', 3.5, 4.0') with a lower limit of one foot (1') and upper limit of six feet (6') (Vernon R Schaefer 2016a) so the prescribed project timeline can be satisfied.

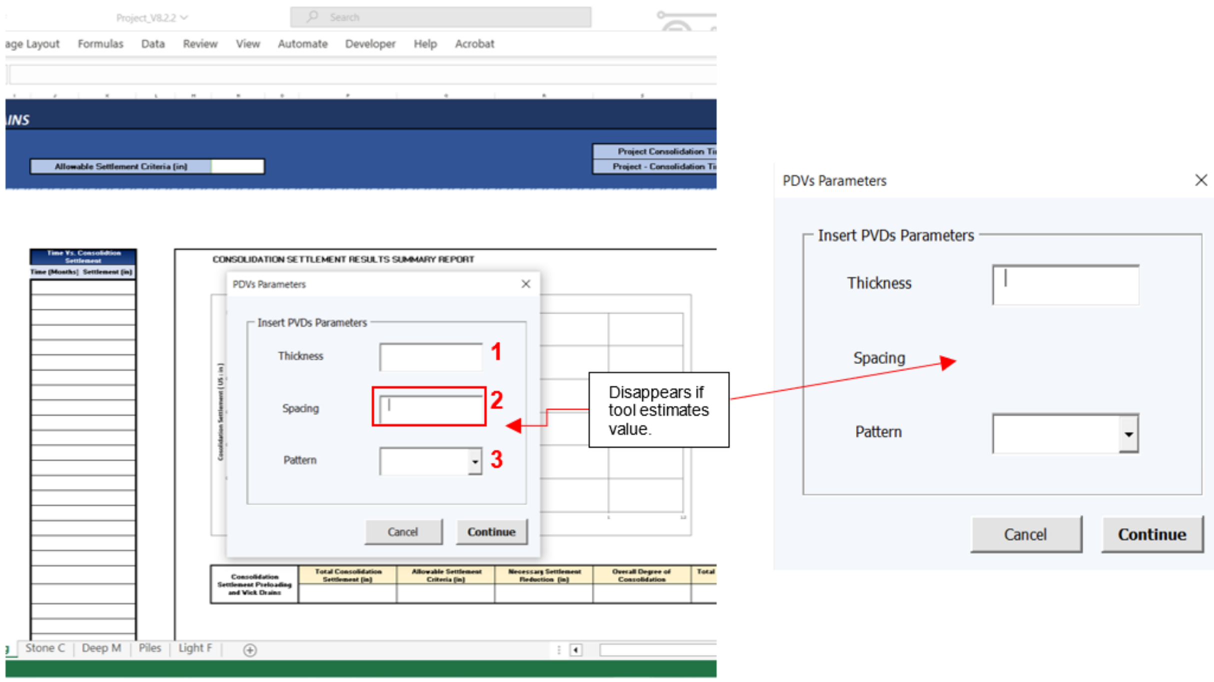


Figure 5-22. Wick drains design parameters user form.

3. From the drop-down menu, choose the installation pattern (triangular/square). The calculation tool sets the pattern as equilateral triangular by default whenever the box is left blank.

- **Enter 0.378 (3/8") inches for thickness, spacing = 4.3' and square pattern values for the example (Figure 5-23).**

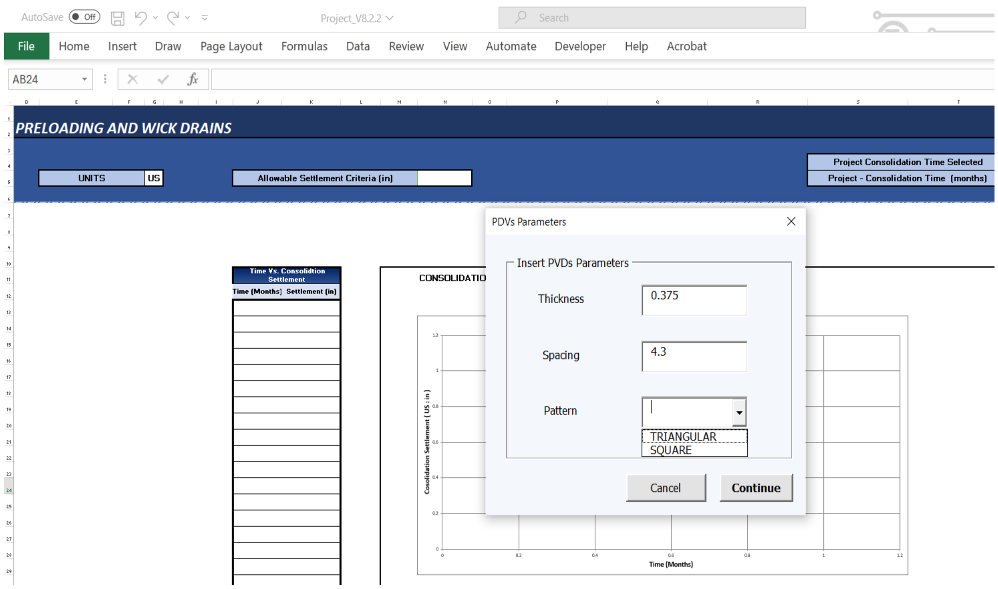
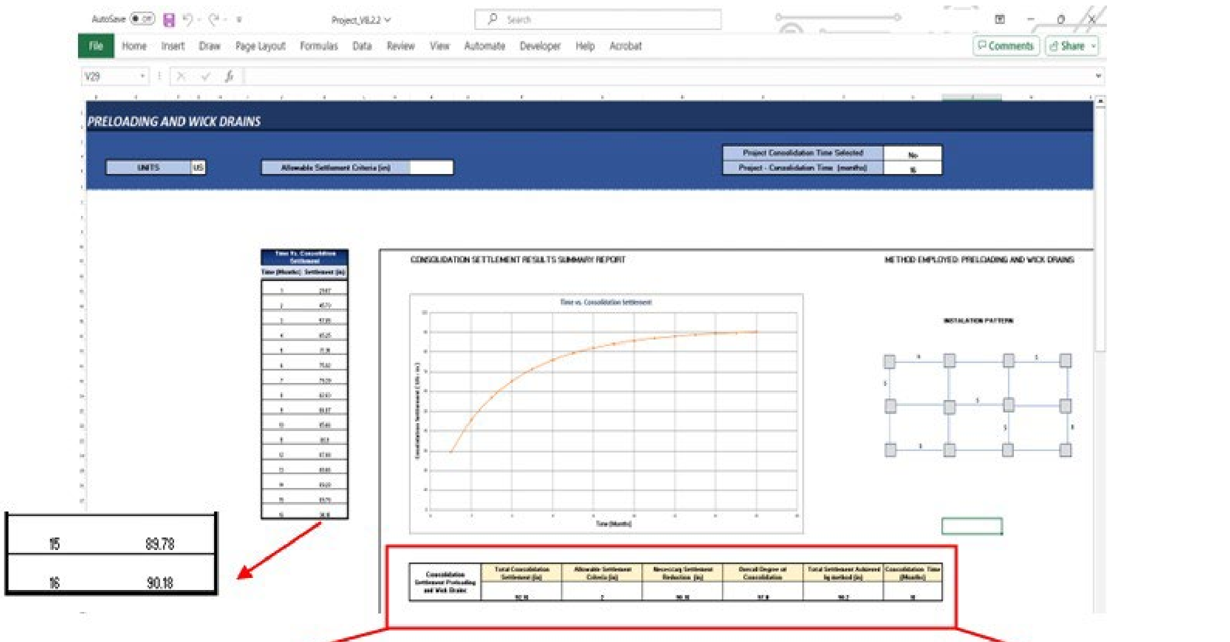


Figure 5-23. Analysis procedure: wick drains.

4. Click continue to start the analysis. Values and results will be displayed as shown in Figure 5-24. After the analysis has been completed, the time required to complete a **100%** of the necessary settlement reduction (90.18” ≈ 90.20”) is **16 months** (**see Figure 5-24**).



Consolidation Settlement Preloading and Wick Drains	Total Consolidation Settlement (in)	Allowable Settlement Criteria (in)	Necessary Settlement Reduction (in)	Overall Degree of Consolidation	Total Settlement Achieved by method (in)	Consolidation Time (Months)
	92.16	2	90.16	97.8	90.2	16

PVD DESIGN PROPERTIES		
WICK DRAINS PARAMETERS	VALUE	UNITS
Spacing S	4.3	ft
Triangular (T) or Square (S) pattern	SQUARE	
Thickness	0.375	in

Figure 5-24. Analysis procedure: preloading and wick drains settlement results.

5.4.2 Other soil improvement methods

The Soil Improvement Settlement Analysis Tool requires users to input design parameters based on the differences among all methods, for example, stress concentration ratio (n) for stone columns and rammed aggregate, column to soil Elastic modulus ratio (Ec/Es) for deep mixing columns, etc. Two examples of other soil improvement methods input details are explained below.

5.4.2.1 Stone columns and rammed aggregate columns

The ***stress reduction method*** has been employed to develop the settlement analysis of Stone (SC), Rammed Aggregate Column (RAC), and deep mixing. In addition, the analysis of SC and RAC can be carried out from the same work page. From a drop-down menu, the type of aggregate column is to be selected (Figure 5-25). Figure 5-26 shows a dialog box in which the stress concentration ratio n must be entered. Otherwise, the tool runs the analysis with values of n = 3, n = 7, for stone columns and rammed aggregate columns, respectively (Vernon R Schaefer 2016a).

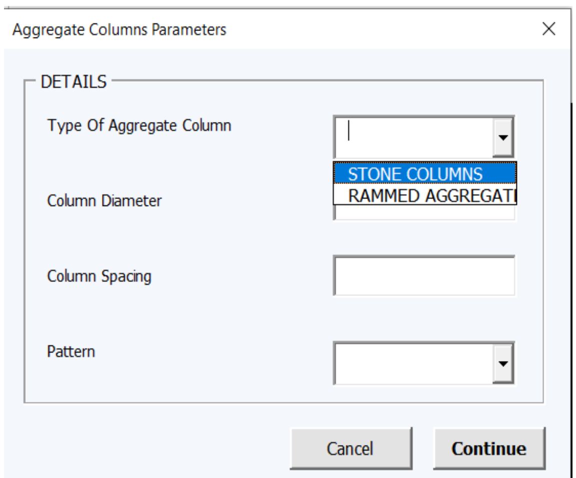


Figure 5-25. Stone columns design parameters form.

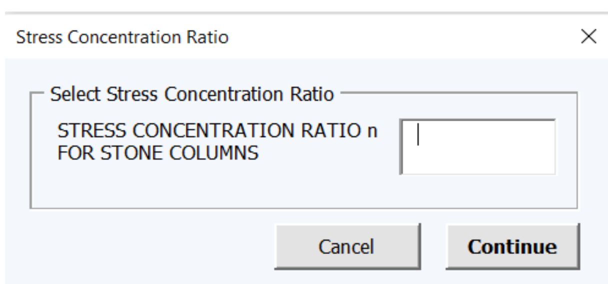


Figure 5-26. Aggregate columns stress concentration ratio user form.

5.4.2.2 Deep Mixing.

Figure 5-27 shows the design parameters for deep mixing. Stress concentration ratio can be inputted, and the tool will approximate the column to soil Elastic modulus ratio (Ec/Es) with a linear relationship (Shang et al. 1998) and vice versa. Moreover, the tool considers diameter of twenty-four (24) inches, triangular installation pattern as default values, which can be overridden by the values provided by the user.

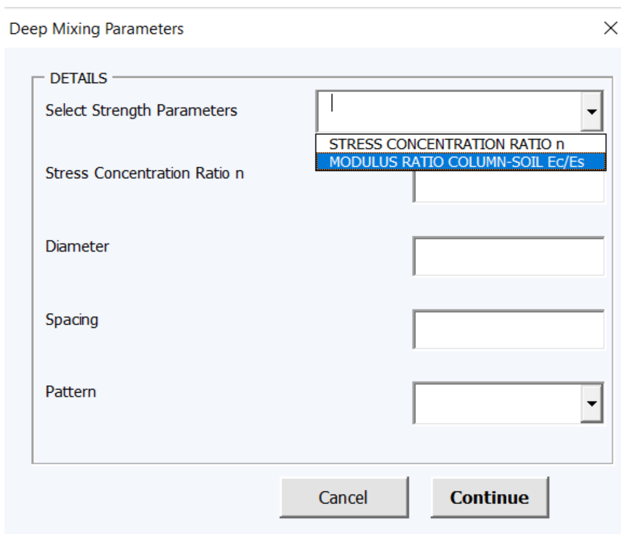


Figure 5-27. Deep mixing design parameters form.

5.5 COST ESTIMATION

The last step using the calculation tool is the estimation of the final project cost based on the soil improvement design, as shown in the workflow below (Figure 5-28).

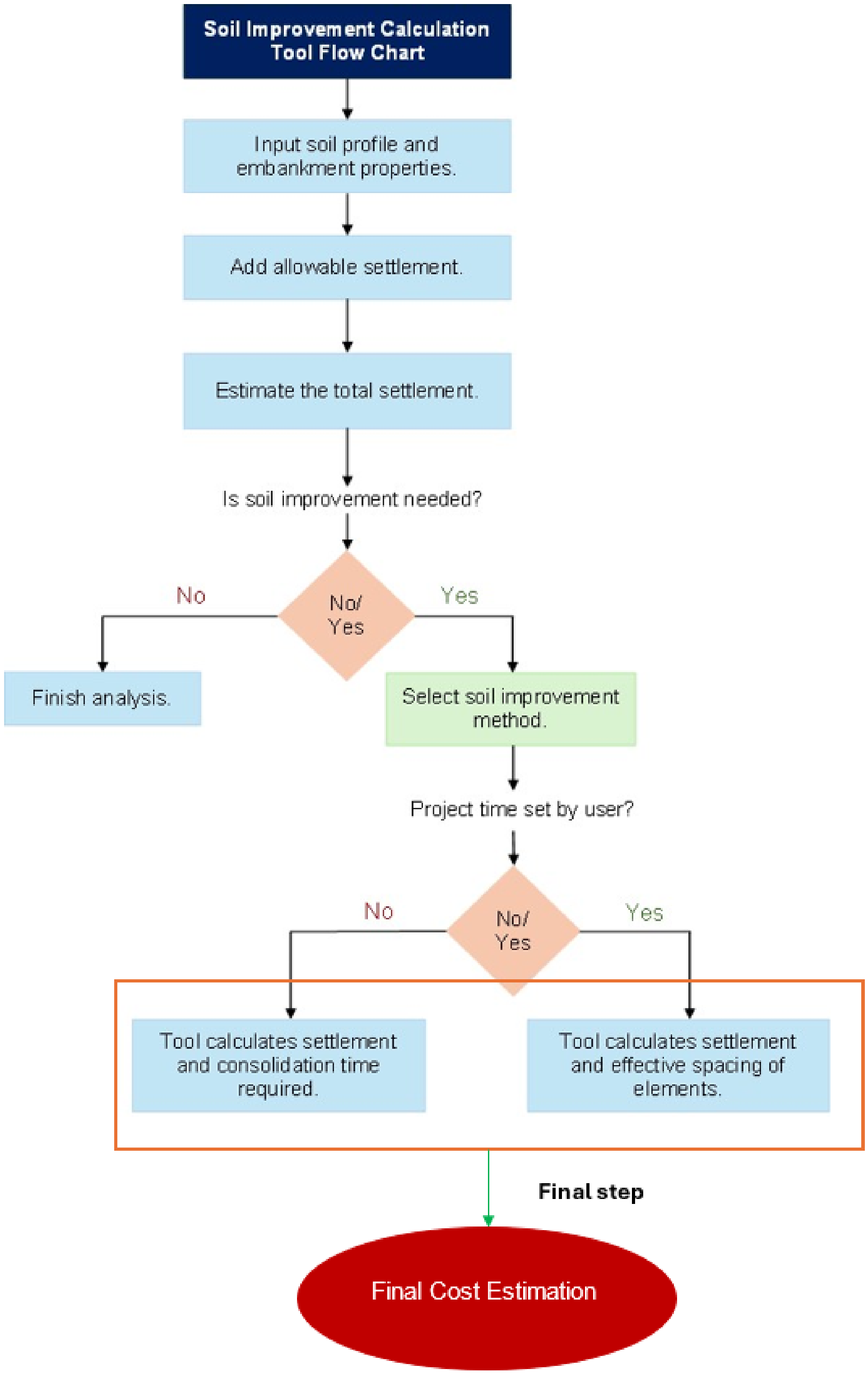


Figure 5-28. General workflow.

The cost estimation uses the basic principle to calculate the cost, i.e., the cost equals quantity times unit cost. The calculation tool will estimate total quantity (the number of elements), namely, the total linear feet in for wick drains and stone columns, and the total cubic yard for deep mixing, light weight fill, and piles. When more than one soil improvement method is used, it will also calculate the respective amounts for combination methods.

The cost estimate section can be found in the bottom right corner of every soil improvement method's sheet as shown in Figure 5-29. To generate a cost estimate, click on the "Estimate cost" button, and a prompt message will appear (see Figure 5-30).

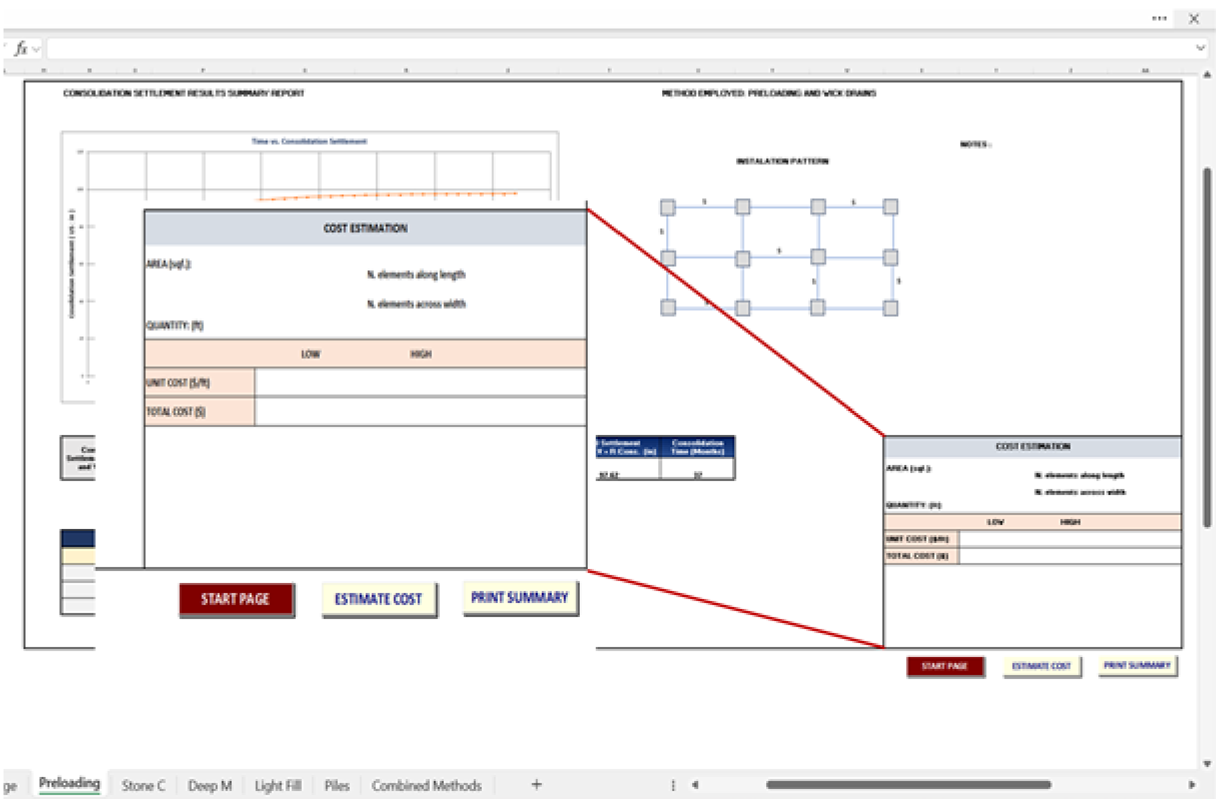


Figure 5-29. Cost estimation section.

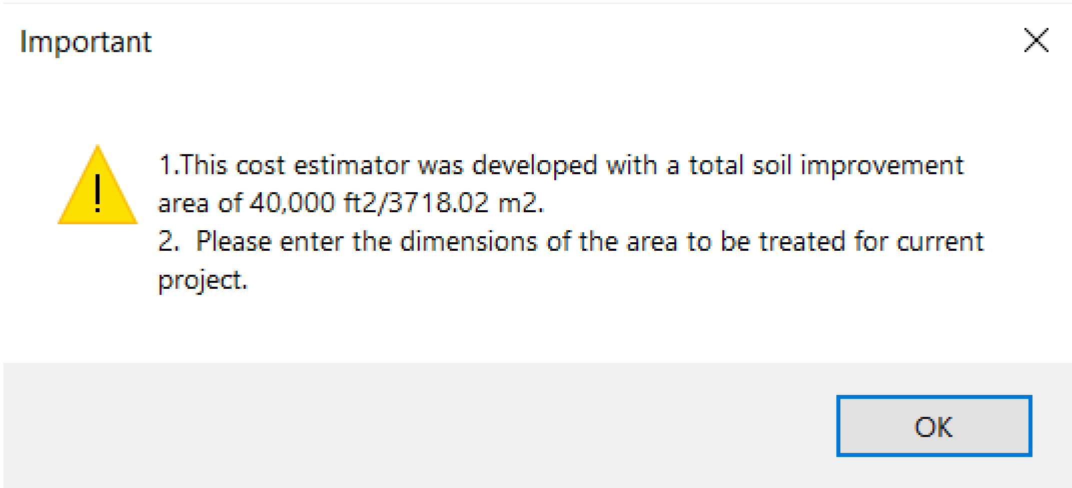


Figure 5-30. Prompt message.

The tool will prompt users to input the dimensions of the area to be treated. If users do not enter these parameters, the tool will automatically estimate the area based on the bottom width of the embankment provided in the first stage of the settlement analysis, using a default length of 200 ft. The depth of wick drains, and deep mixing elements is estimated based on the assumption that they will be installed throughout the entire soft soil strata being analyzed. For aggregate columns and piles, the installation depth is provided by users during the analysis. For lightweight fill, given that the height of the fill decreases as the slope decreases with length of the embankment; to estimate an approximate cost, the tool will ask for the average height of the fill.

Following the example presented in time-cast charts for wick drains, the tool will ask for two costs, as shown in Figure 5-31:

- High unit cost
- Low unit cost

For this example, enter a low unit cost of $1.00/ft and a high unit cost of $4.00ft/ft. The total length of the area to be treated (installation of the wick drains) is 500 ft. Note that the width will be left empty. As explained above, the tool will use the value of the bottom with of the embankment, which is 164 ft for this example. Given that costs are variable, the tool cannot assume any unit cost in the estimation process. Therefore, it is crucial to input these values for the calculation.

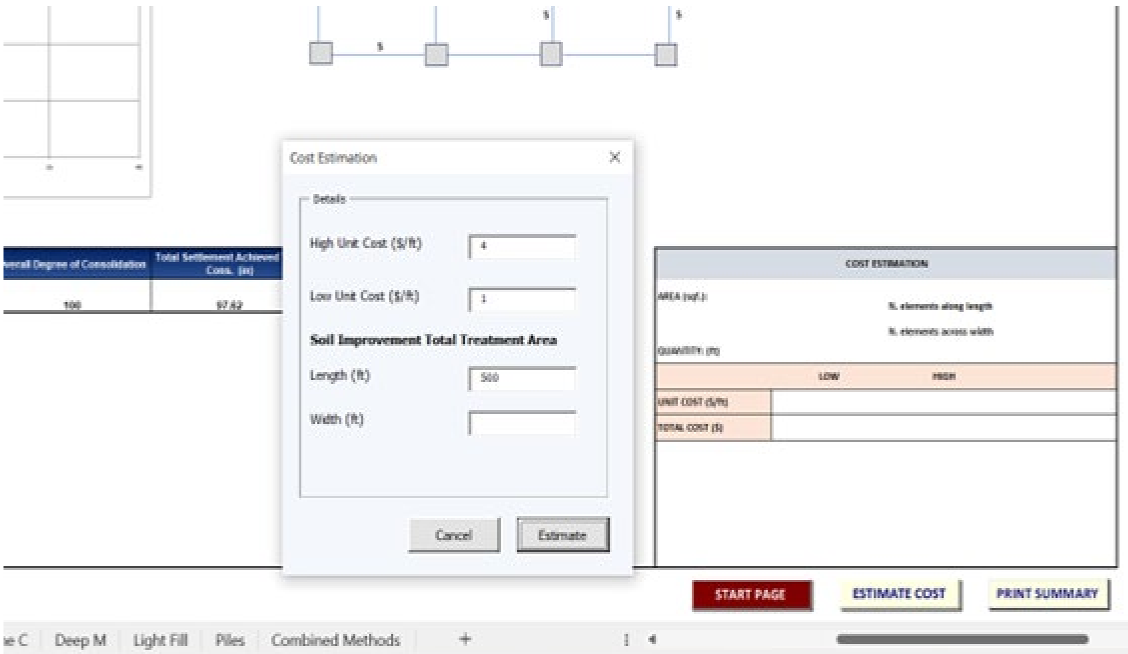


Figure 5-31. Unit cost and area of improvement form.

Click on “Estimate” and the tool will show the following results, as shown in Figure 5-32:

1. Total area
2. Total quantity
3. Number of elements along length
4. Number of elements across width
5. Low and High Unit cost
6. Total High and low cost.

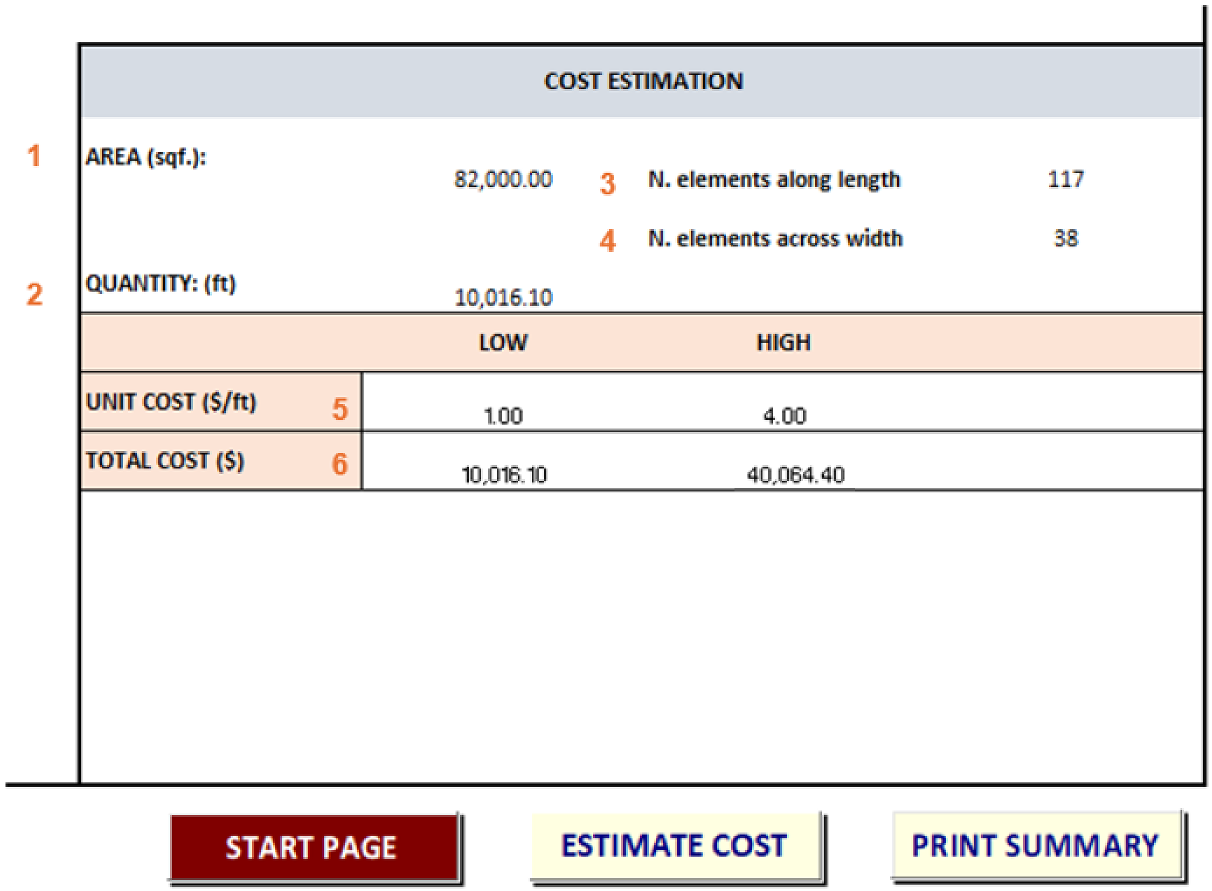


Figure 5-32. Final cost estimation results.

In summary, the wick drains are to be installed in a square pattern, spaced at 4.25 ft to achieve a consolidation settlement of approximately 90.20 inches in 16 months. The number of elements to be installed is 117 along the length and 38 across the width of the embankment.

CHAPTER 6 VALIDATIONS

6.1 CASES HISTORIES USED IN THE CALIBRATION-VALIDATION.

The goal of validation was to test the overall efficiency and accuracy of the programming code for each soil improvement method developed in this project. At this stage, several design parameters embedded within the code were tested and validated using cases histories, thereby proving consistency with existing observations.

For this calibration process, several case histories were utilized, which were collected from existing literature databases. The selection of the case histories was based on different sets of data containing soil properties, soil improvement designs, and settlement results of improved versus unimproved soil. Table 6-1 presents a summary of the case histories employed in the validation process.

Table 6-1 Summary case histories.

No.1	Case History	Improvement method(s)
1	Compressibility and Flow Parameters from PVD Improved Soft Bangkok Clay	Wick Drains
2	Back-analyses of flow parameters of PVD improved soft Bangkok clay with and without vacuum preloading from settlement data and numerical analysis	Wick Drains
3	Behavior of thick marine deposits subjected to vacuum combined with surcharge preloading	Wick Drains
4	Vacuum preloading consolidation of reclaimed land: a case study	Wick Drains
5	Ground Modification Methods Reference Manual- VII Chapter 5. Stone Columns example	Stone Columns
6	New Analytical Approach for Predicting Horizontal Displacement of Stone Columns	Stone Columns

7	Performance of a mechanically stabilized earth retaining wall built on soft clay foundation improved by rammed aggregate piers in a trial embankment	Rammed Aggregate Columns
8	Performance Monitoring of Rammed Aggregate Piers (RAPs)	Rammed Aggregate Columns
9	Ground Modification Methods Reference Manual VII Chapter 5. RAC example.	Rammed Aggregate Columns
10	Numerical Modeling of an Embankment over Soft Ground Improved with Deep Cement Mixed Columns: Case History	Deep Mixing
11	Deep soil mixing used to reduce embankment settlement	Deep Mixing
12	Deep soil mixing design under seismic conditions - a case study	Deep Mixing
13	Settlement Mitigation Using Light Weight Fill Embankment Systems	Lightweight Fills
14	Case Study of EPS Geofoam Lightweight Fill for Settlement Control at Bridge Approach Embankment	Lightweight Fills
15	Settlement of group of Pile example. Foundation Engineering	Pile Supported Embankment
16	Analyses of a pile-supported embankment over soft clay: Full-scale experiment, analytical and numerical approaches	Pile Supported Embankment
17	Performance of Pile-Supported Embankment over Soft Soil: Full-Scale Experiment	Pile Supported Embankment

6.2 CALIBRATION DETAILS AND COMPARISON

This section presents the details of the validation, which include each case history, a brief description and a summary of the extracted data, as well as a comparison of the results obtained with the calculation tool. The user can independently verify the result.

<u>Case 1:</u> Compressibility and Flow Parameters from PVD Improved Soft Bangkok Clay (Bergado et al. 1996)

Three full scale embankments (TS-1, TS-2, TS-3) were constructed on very soft clay. To accelerate the consolidation and improve the clay soil, wick drains were installed at three different spacings for TS-1, TS-2, and TS-3 at 5, 4 and 3 ft respectively. Figure 6-1 shows the cross-section of the embankment and the scheme of the wick drains installed, while Table 6-2 summarizes the soil properties, PVD design, and embankment properties as well as the comparison of the results from the case study and those obtained with the calculation tool. The total settlement from both the case study and the tool are in close agreement, with values of 66.91 and 69.48 inches respectively. In addition, the settlement resulted after the installation of the wick drains show similar results for the three embankments.

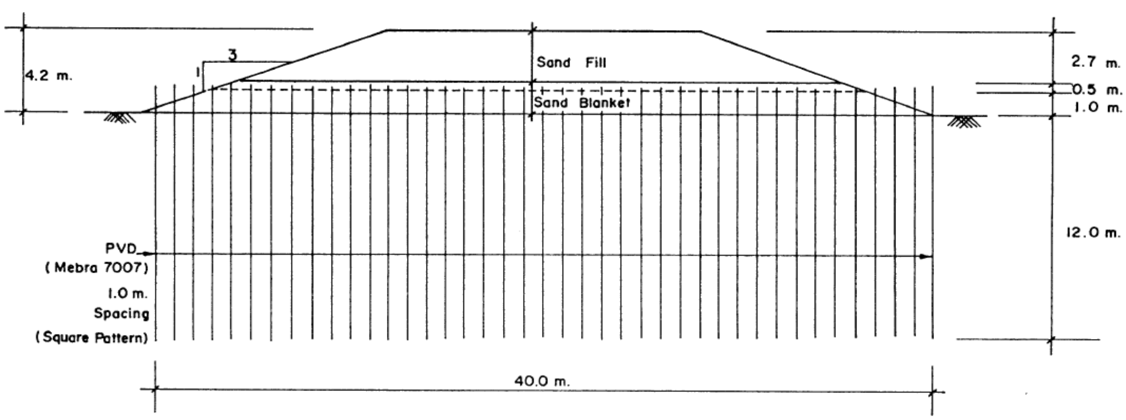


Figure 6-1. PVD installation scheme (Bergado et al. 1996).

Table 6-2 Calibration parameters and results summary (Case 1).

Calibration and Test Summary Sheet			
Paper Title:	Compressibility and Flow Parameters from PVD Improved Soft Bangkok Clay	UNITS:	US

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	PI	LL	Cc	Cs	Cv (in2/s)	Cr (in2/s) TS1 /TS3	Cr (in2/s) TS2	kv (in/s)
weather crust	6.6	1.06	107.8	117.8	1.35	35	80	0.82	0.08	5.4E-05	2.1E-04	2.0E-04	3.2E-08
CH	19.7	0.85		88.5	2.48	58	98	1.39	0.14	3.9E-05	2.1E-04	2.0E-04	2.3E-08
CH	6.6	1.38		95.0	1.18	60	100	0.76	0.08	5.2E-05	2.1E-04	2.0E-04	3.1E-08
CH	19.7	2.13		100.0	1.46	43	78	0.86	0.09	6.5E-05	2.1E-04	2.0E-04	3.8E-08

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)
13.8	114.58	1.64	131.2	48.4

Soil Improvement elements Design

Embankment	Spacing (ft)	Pattern	Thikness (in)
TS-1	5	square	0.375
TS-2	4	square	
TS-3	3.3	square	

Calibration Results Summary Sheet

Total Settlement

Description	Case history (in)	Calculated by the Tool (in)
TS-1	66.91	**69.48**
TS-2	66.91	**69.48**
TS-3	66.91	**69.48**

Settlement achieved with Soil Improvement

Description	Case history (in)	Consolidation Time Case history (months)	Calculated by the Tool (in)	Time by the Tool (months)
TS-1	49.2	13.8	**49.96**	**14**
TS-2	57.1	13.8	**58.79**	**14**
TS-3	62.4	14.3	**65.16**	**14**

***Case 2:* Back-analyses of flow parameters of PVD improved soft Bangkok clay with and without vacuum preloading from settlement data and numerical analysis (Voottipruex et al. 2014b)**

Conventional PVD and vacuum were utilized to improve a soft clay soil foundation for Third Runways expansion at Suvarnabhumi International airport in Thailand. Since the PVD details were missing in the literature, various simulations using different PVDs were made and compared with the observational data. Table 6-3 lists the ones that exhibited closest results with the obsrvational data.

Table 6-3 Calibration parameters and results summary (Case 2).

Calibration and Test Summary Sheet

Paper Title: Back-analyses of flow parameters of PVD improved soft Bangkok clay with and without vacuum preloading from settlement data and numerical analysis **UNITS:** **US**

Soil Profile

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	PI	LL	Cc	Cs	Cv (in2/s)	Cr (in2/s) SP-W5-023T	Cr (in2/s) SP-W5-021T	kv (in/s)
(CH) Fat clay	6.60	1.06	107.80	117.80	1.35	60.00	20.00	0.82	0.08	5.4E-05	1.2E-04	1.3E-04	3.2E-08
(CH) Fat clay	9.80	0.79		87.90	2.52	52.00	14.00	1.41	0.14	3.9E-05	1.2E-04	1.3E-04	2.3E-08
(CH) Fat clay	16.40	1.28		89.20	2.44	50.00	12.00	1.44	0.14	3.9E-05	1.2E-04	1.3E-04	2.3E-08
(CH) Fat clay	9.80	1.49		95.60	1.18	62.00	21.00	0.87	0.09	6.5E-05	1.2E-04	1.3E-04	3.8E-08
(CH) Fat clay	6.60	2.13		100.00	1.46	49.00	11.00	0.74	0.07	6.5E-05	1.2E-04	1.3E-04	3.8E-08

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)
12.5	114.58	1.64	284	216.5

Soil Improvement elements Design

Embankment	Spacing (ft)	Pattern	Thikness (in)
1	2.8	triangular	0.2

Calibration Results Summary Sheet

Total Settlement

Description	Case history (in)	Calculated by the Tool (in)
Predicted (SP-W5-023T)	70.0	**68.12**
Predicted (SP-W5-021T)	58.0	**68.12**

Settlement achieved with Soil Improvement

Description	Case history (in)	Consolidation Time Case history (months)	Calculated by the Tool (in)	Time by the Tool (months)
Predicted (SP-W5-023T)	70.0	12.3	**61.5**	**12**
Observed (SP-W5-023T)	57.1	12.3	**61.5**	**12**
Predicted (SP-W5-021T)	51.2	12.3	**62.55**	**12**
Observed (SP-W5-021T)	56.7	12.3	**62.55**	**12**

<u>Case 3:</u> Behavior of thick marine deposits subjected to vacuum combined with surcharge preloading (Zhang et al. 2021) .

This case history involved a marine deposit improved by vacuum preloading and wick drains. The land reclamation project in coastal China involved reclaiming land from slurry dredge from the seabed. Due to the poor properties of the marine mud, soil treatment was necessary. An observation-test embankment, treated with PVDs, was constructed to simulate the total pressure from vacuum and surcharge preloading on the soil. The validation results are presented in Table 6-4.

Table 6-4. Calibration parameters and results summary (Case 3).

Calibration and Test Summary Sheet

Paper Title: Behaviour of thick marine deposits subjected to vacuum combined with surcharge preloading **UNITS:** US

Soil Profile

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	PI	LL	Cc	Cs	Cv (in2/s)	Cr = Cv (in2/s)	Cr = 1.5 Cv (in2/s)	kv (in/s)
WEATHER	6.56	1.06	107.80	117.80	1.35	35.00	80.00	0.82	0.08	5.4E-05	5.4E-05	8.1E-05	9.8E-08
CH	19.68	0.85		88.45	2.48	58.00	98.00	1.39	0.14	3.9E-05	3.9E-05	5.9E-05	3.0E-07
CH	6.56	1.38		95.00	1.18	60.00	100.00	0.76	0.08	5.2E-05	5.2E-05	7.8E-05	3.5E-07

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)
11.5	114.7		754	720

Soil Improvement elements Design

Embankment	Spacing (ft)	Pattern	Thikness (in)
SI	2.6	triangular	0.2
SII	2.6	triangular	0.2

Calibration Results Summary Sheet

Total Settlement

Description	Case History Hyperbolic method (in)	Case History TS Asakoas method (in)	Calculated by the Tool (in)
SI (predicted)	52.3	50.7	**49.56**
SII (predicted)	61.0	59.8	**49.56**

Settlement achieved with Soil Improvement

Description	Case history (in)	Consolidation Time Case history (months)	Calculated by the Toot Cr = 1.5 Cv (in)	Time Tool (months)
SI	52.0	6	**47.95**	**6**
SII	59.0	6	**48**	**6**

Note: The settlement obtained in the tool reflects the general soil profile from the case study.
Specific soil properties for each embankment not available.

<u>Case 4:</u> Vacuum preloading consolidation of reclaimed land: a case study (Shang et al. 1998)

This is another land reclamation project, located in Xingang Port, Tianjing China. Vacuum preloading and PVDs were used to improve an area of 4,800,000 ft^2 of reclaimed land for pier construction. Four control embankments of 164 x 164 ft^2 each, were constructed on PVD improved soil with a surcharge fill equivalent to 2.03 kips/ft^2 (97 kPa) to simulate the total pressure. Figure 6-2 shows the distribution of the testing area. The results obtained from the tool are compared to those where the surcharge fill was the highest (2.03 ksf), as shown in Table 6-5.

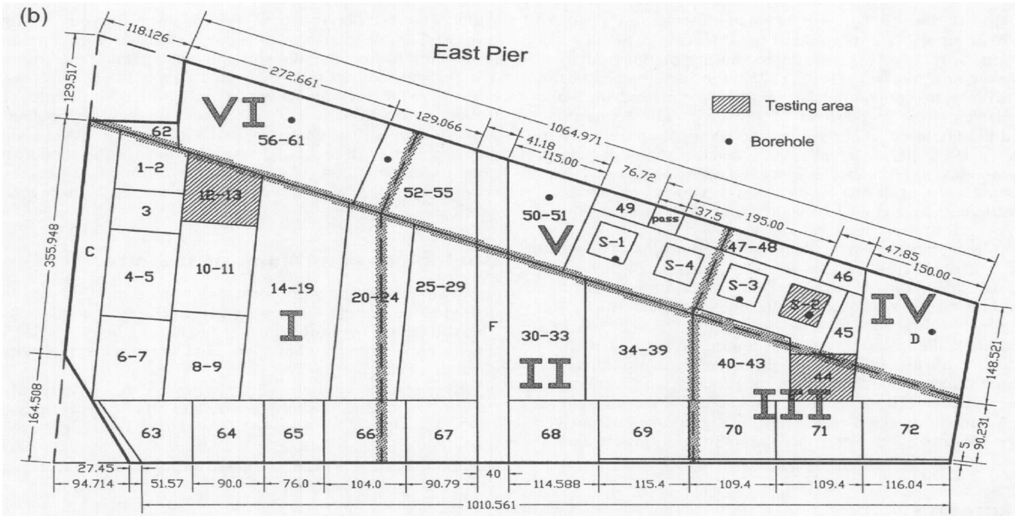


Figure 6-2. Plan view of port and control sections (Shang et al. 1998).

Table 6-5. Calibration Parameters and Results Summary (Case 4)..

Calibration and Test Summary Sheet

Paper Title: Vacuum preloading consolidation of reclaimed land: a case study — UNITS: US

Soil Profile

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	PI	LL	Cc	Cs	Cv (in2/s)	Cr = 2Cv in2/s	kv (in/s)
CH	13.78	0.10	106.95		1.56	40.80	40.80	0.31	0.062	2.3E-04	4.7E-04	3.2E-08
PT	11.48	0.52	108.86		1.53	48.10	48.10	0.31	0.063	2.2E-04	4.3E-04	2.3E-08
OL	13.12	0.50	111.40		1.24	35.00	35.00	0.27	0.054	3.1E-04	6.2E-04	2.3E-08
OL	9.84	0.87	105.67		1.65	52.70	52.70	0.34	0.068	9.3E-05	1.9E-04	3.8E-08
OL	16.40	1.31	108.86		1.45	49.00	49.00	0.32	0.065	1.2E-04	2.5E-04	3.8E-08

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)
14.8	137.2		164	105

Soil Improvement elements Design

Embankment	Spacing (ft)	Pattern	Thikness (in)
1	4.26	square	0.19

Calibration Results Summary Sheet

Total Settlement

Description	Case history (in)	Calculated by the Tool (in)
Test embankment	90.528	97.8

Settlement achieved with Soil Improvement

Description	Case history (in)	Consolidation Time Case history (months)	Calculated by the Tool (in)	Time by the Tool (months)
Test embankment	47.2	4	66.83	4
Test embankment	90.5	12.3	91.8	12

Note: in this case soil profile was in a under consolidated stated (i.e., soil is still going under consolidation process due to its own weight)

<u>Case 5:</u> Ground Modification Methods Reference Manual- VII Chapter 5. Stone Columns example (Vernon R Schaefer 2016a).

In the example referenced from the FHWA Ground Improvement manual, an estimation of the total settlement of foundation soil under an embankment and the settlement of the composite foundation (stone-column) is presented. One of the main goals of stone columns is to reduce the settlement by increasing the stiffness of soil-aggregate column matrix by transferring more pressure to the stiffer elements, the example focuses on the settlement of the foundation after the soil improvement. Table 6-6 shows the settlement of the composite foundation and the consolidation time obtained with tool. Note that C_v and other consolidation parameters were estimated based on correlation as they were missing from the literature. It is noteworthy that the correlation is incorporated in the calculation tool.

Table 6-6. Calibration parameters and results summary (Case 5).

Calibration and Test Summary Sheet

Paper Title: Ground Modification Methods Reference Manual- VII Chapter 5. Stone Columns example   UNITS: US

Soil Profile

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	PI	LL	Cc	Cs	Cv (in2/s)	Cr = 2Cv in2/s	kv (in/day)	
Lean Clay	50.00	1.46		120.00	0.60			0.20		5.4E-05		1.27	

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)
14.8	137.2		164	105

Soil Improvement elements Design

Embankment	Spacing (ft)	Diameter (in)	C. length (ft)	Pattern	Stress ratio (n)
1	5.7	36 in	50	Triangular	5

Calibration Results Summary Sheet

Total Settlement

Description	Case history (in)	Calculated by the Tool (in)
Embankment 1	27	**25.08**

Settlement of composite foundation

Description	Case history (in)	Consolidation Time Case history (months)	Calculated by the Tool (in)	Time by the Tool (months)
1	10.0	N/A	**12.49**	**2**

Note: Diference is because in the example the pressure at the layer = γH. The tool distrutes that pressure by pressure distribution under an emabnkment theory.

<u>Case 6:</u> New Analytical Approach for Predicting Horizontal Displacement of Stone Columns (Chan and Poon 2015).

This case history involved the expansion of Kooragang Coal Terminal, a coal export terminal located in Koorang Island in New South Wales, Australia. A portion of this island is situated over reclaimed swap land underlain by soft soil. The main goal of the project was to expand the terminal to increase coal export capacity. Figure 6-3 shows the proposed expansion.

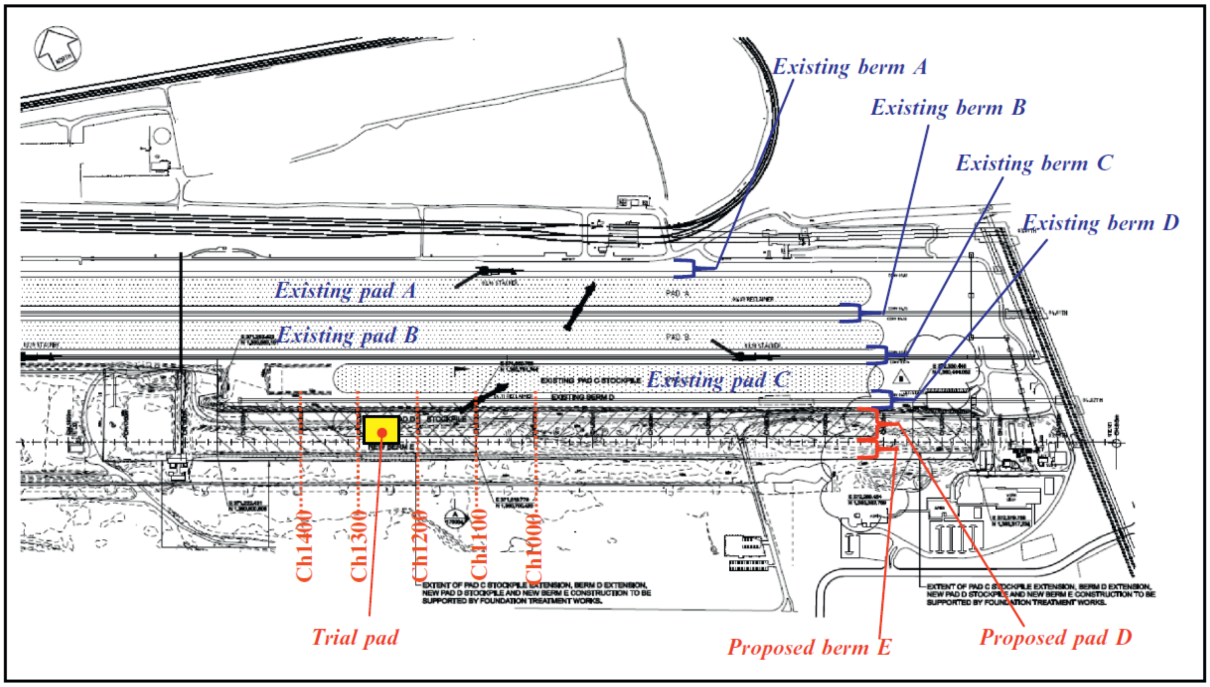


Figure 6-3. Plan view of proposed expansion (Chan and Poon 2015).

The site consisted of a very soft clay approximately 15 ft (4 m) thick. Above this soft clay, there is sand fill, and below the clay, it is a dense sand clay. Stones columns were installed throughout the entire soft layer up the denser soil. The settlement was reduced by approximately 1 ft. Table 6-7 summarizes the properties and settlement results.

Table 6-7. Calibration parameters and results summary (Case 6).

Calibration and Test Summary Sheet

Paper Title:	New Analytical Approach for Predicting Horizontal Displacement of Stone Columns	UNITS:	US

Soil Profile

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	PI	LL	Cc	Cs	Cv (ft2/day)	Cr = 1.5 Cv (ft2/day)	Cr = 2 Cv (ft2/day)	kv (ft/day)
SP	4.92	21.00	127.32										
SP	4.92	21.00	120.95										
CH	13.12	1.93	98.67		1.77	54.00	80.00	0.92	0.10	0.032	0.048	0.065	9.84E-03

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)
16.4	127.32		131.2	65.6

Soil Improvement elements Design

Embankment	Spacing (ft)	Diameter (in)	C. length (ft)	Pattern	Stress ratio (n)
1	9.02	47.2	23.0	square	5

Calibration Results Summary Sheet

Total Settlement

Description	Case history (in)	Calculated by the Tool (in)
Embankment 1	Not Specified	**19.68**

Settlement of composite foundation

Description	Case history (in)	Consolidation Time Case history (months)	Calculated by the Tool (in)	Time by the Tool (months)
Observed Settlement after 1 month	6.69	1	**5.90 (Cr = 1.5Cv)**	**1**
Predicted	7.08	1	**6.69 (Cr = 2Cv)**	**1**

<u>Case 7:</u> Performance of a mechanically stabilized earth retaining wall built on soft clay foundation improved by rammed aggregate piers in a trial embankment (Abdullah 2022).

A full-scale trial embankment supported by rammed aggregate piers (RAC) was constructed to evaluate a proposed soil improvement method for a railway project in the Philippines. The primary goal was to mitigate consolidation settlement in a very soft clay between stiffer soils. The clay layer is underlain by a stiff sandy silt and overlain by stiff clay. RAPs were installed to a depth of 46 ft and spaced at 5 ft. Results obtained from the calculation tool were compared to the observed settlement data at approximately 30 days, which was the maximum observation period in the study, which are summarized in Table 6-8.

Table 6-8. Calibration parameters and results summary (Case 7).

Calibration and Test Summary Sheet

Paper Title: Performance of a mechanically stabilized earth retaining wall built on soft clay foundation improved by rammed aggregate piers in a trial embankment **UNITS:** US

Soil Profile

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	PI	LL	Cc	Cs	Cv (ft2/day)	Cr (ft2/day)	kv (ft/day)
Stiff clay brown	16.40	5.75	114.59		0.96			0.42	0.96	0.171		9.84E-03
Very soft grayish clay	6.56	7.37	89.12		1.09			0.31	1.09	0.171		9.84E-03
Stiff sandy silt	6.56	10.00	108.22									
Very loose silty sand	22.96	4.66	108.22		2.55			1.10	2.55	0.388		9.84E-03
Hard clay	16.40	30.00	133.69									
Medium dense sand	4.0	30.00	133.7									

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)
16.4	119.68		49.2	26.9

Soil Improvement elements Design

Embankment	Spacing (ft)	Diameter (in)	C. length (ft)	Pattern	Stress ratio (n)
1	4.92	23.62	45.92	triangular	7

Calibration Results Summary Sheet

Total Settlement

Description	Case history (in)	Calculated by the Tool (in)
Embankment 1	Not Specified	**16.14**

Settlement of composite foundation

Description	Case history (in)	Consolidation Time Case history (months)	Calculated by the Tool (in)	Time by the Tool (months)
Embankment 1	0.79	1	**0.79**	**1**

<u>Case 8:</u> Performance Monitoring of Rammed Aggregate Piers (RAPs) (KURT BAL 2021).

A RAP system was conducted at a wastewater facility and monitored in Turkey. A full-scale test embankment was constructed over improved soil with properties listed in Table 6-9. Piers were installed in a square pattern to a depth of 50 ft. The settlement observed in the study (9.45 – 12.2 in) is slightly lower than the estimated with the settlement tool (14.17 in). This discrepancy may be attributable to the estimated stress concentration ratio (n) by the calculation tool, as the value of n was not provided in the literature.

Table 6-9. Calibration parameters and results summary (Case 8).

Calibration and Test Summary Sheet

Paper Title: Performance Monitoring of Rammed Aggregate Piers (RAPs) **UNITS:** US

Soil Profile

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	PI	LL	Cc	Cs	Cv (ft2/day)	Cr = 2Cv	kv(ft/day)
silty clay	13.12	0.48		117.13	0.85	47.00	63.00	0.22		0.32	0.65	2.27E-02
Silty Sand	19.68	10.00		117.13								
silty clay	16.40	2.42		117.13	0.85	30.00	48.00	0.11		0.32	0.65	2.27E-02
silty clay 1	98.40	5.48		114.59	1.10			0.27	0.05	0.32	0.65	2.27E-02
Silty clay 2	98.40	21.50		114.59	0.85			0.13	0.03	0.32	0.65	2.27E-02

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)
20.01	115.22	2.3	120.7	65.6

Soil Improvement elements Design

Embankment	Spacing (ft)	Diameter (in)	C. length (ft)	Pattern	Stress ratio (n)
1	4.59	19.68	49.2	square	

Calibration Results Summary Sheet

Total Settlement

Description	Case history (in)	Calculated by the Tool (in)
Test Embankment	7.87-31.48	**29.91**

Settlement of composite foundation

Description	Case history (in)	Consolidation Time Case history (months)	Calculated by the Tool (in)	Time by the Tool (months)
Test Embankment	9.45-12.20	1.7	**14.17**	**1**

Note: In this case Cr was supposed to be twice Cv and Cv was equal for all soil layers.

<u>Case 9:</u> Ground Modification Methods Reference Manual VII Chapter 5. RAC example (Vernon R Schaefer 2016b).

In the example consulted from the FHWA Ground Improvement Manual, like the example of stone columns, an estimation of the settlement before and after soil improvement is presented. The results obtained with the tool agree with those in the literature, as shown in Table 6-10.

Table 6-10. Calibration parameters and results summary (Case 9).

Calibration and Test Summary Sheet

Paper Title:	Ground Modification Methods Reference Manual VII Chapter 5. RAC example	UNITS:	US

Soil Profile

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	PI	LL	Cc	Cs	Cv (ft2/day)	Cr (ft2/day)	kv (in/day)
Lean Clay	15.00	0.44		120.00	0.70			0.25		0.10	0.20	1.27

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)
20	125	0	100	40

Soil Improvement elements Design

Embankment	Spacing (ft)	Diameter (in)	C. length (ft)	Pattern	Stress ratio (n)
20	5	33	15	square	6

Calibration Results Summary Sheet

Total Settlement

Description	Case history (in)	Calculated by the Tool (in)
Test Embankment	22	**21.96**

Settlement of composite foundation

Description	Case history (in)	Consolidation Time Case history (months)	Calculated by the Tool (in)	Time by the Tool (months)
Test Embankment	0.68		**0.73**	**1**

<u>Case 10:</u> Numerical Modeling of an Embankment over Soft Ground Improved with Deep Cement Mixed Columns: Case History (Yapage et al. 2014).

This case history describes an embankment constructed for the Ballina Bypass section of the Pacific Highway located in Australia. The settlement analysis of the embankment constructed over a soft clay deposit improved soil with deep mixed columns is shown. Figure 6-4 shows a plan view of the installation pattern. Additionally, Table 6-11 presents the soil properties and compares the settlement obtained from the tool with that observed from the case study.

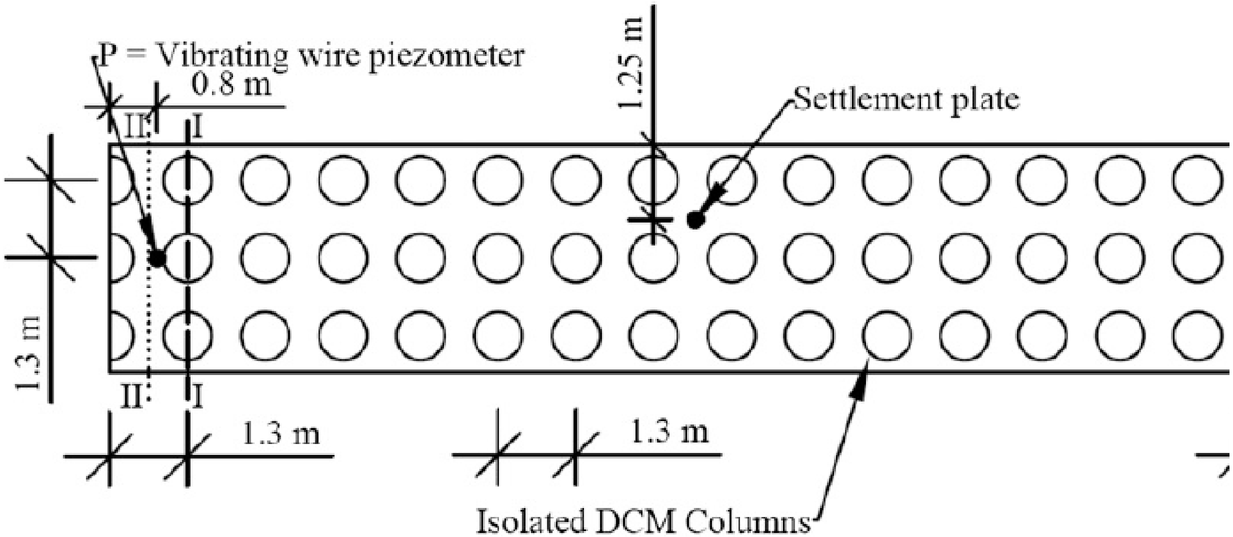


Figure 6-4. Installation scheme of deep mixed elements(Yapage et al. 2014).

Table 6-11. Calibration parameters and results summary (Case 10).

Calibration and Test Summary Sheet

Paper Title:	Numerical Modeling of an Embankment over Soft Ground Improved with Deep Cement Mixed Columns: Case History	**UNITS:**	**US**

Soil Profile

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	PI	LL	Cc	Cs	Cv (m2/day)	Cr	kv(m/s)
Firm clay	1.64	5.89		114.59	2.00			0.80	0.08	0.02		9.10E-08
soft clay	26.24	0.72		92.31	3.00			1.30	0.10	0.02		6.10E-08
Silty sand	16.40	20.00		114.59								

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)
18.27	120.95	0	135.23	62.16

Soil Improvement elements Design

Embankment	Spacing (ft)	Diameter (in)	C. length (ft)	Pattern	Stress ratio (n)
1	4.26	31.49		square	6.5

Calibration Results Summary Sheet

Total Settlement

Description	Case history (in)	Calculated by the Tool (in)
Embankment	N/A	**61.40**

Settlement of composite foundation

Description	Case history (in)	Consolidation Time Case history (months)	Calculated by the Tool (in)	Time by the Tool (months)
Embankment	15.74	13.3	**18.50**	**13**

Case 11: ***Deep soil mixing used to reduce embankment settlement (Bergado et al. 2000).***

The project rehabilitated a section of the Bangna-Bangpakong Highway in Thailand. The primary objective was to ensure slope stability and to reduce severe differential settlements by improving the soft soil under this major road. Deep mixed elements were installed to a depth of 53 ft, which penetrated the total depth of the soft strata. The settlement results from the calculation tool agree with the values from the published results, as shown Table 6-12.

Table 6-12. Calibration parameters and results summary (Case 11).

Calibration and Test Summary Sheet

Paper Title:	Deep soil mixing used to reduce embankment settlement	**UNITS:**	**US**

Soil Profile

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	Cc/(1+e0	Cs/(1+e0	Cc	Cs	Cv (ft2/day)	Cr (ft2/day)	kv(ft/day)
Soft clay	9.84	0.85		111.41	0.90	0.30	0.03	0.57	0.06	7.4E-02	0.15	9.8E-03
Soft clay	19.68	0.85		89.12	1.00	0.45	0.05	0.90	0.09	5.9E-02	0.12	9.8E-03
Soft clay	16.40	1.42		92.31	0.80	0.40	0.04	0.72	0.07	5.9E-02	0.12	9.8E-03
Soft clay	6.56	1.82		92.31	0.80	0.35	0.04	0.63	0.06	5.9E-02	0.12	9.8E-03
Soft clay	6.56	2.67		98.67	1.20	0.30	0.03	0.66	0.07	7.4E-02	0.15	9.8E-03
Medium-stiff clay	4.92	3.36		105.04	1.20	0.25	0.03	0.55	0.06	7.4E-02	0.15	9.8E-03

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)
8.2	120.95	4.26	72.16	39.36
9.84	120.95	4.26	78.72	39.36

Soil Improvement elements Design

Embankment	Spacing (ft)	Diameter (in)	C. length (ft)	Pattern	Ec/Es
1	4.92	23.62	52.48	square	11.2
2	4.92	23.62	52.48	square	11.2

Calibration Results Summary Sheet

Total Settlement

Description	Case history (in)	Calculated by the Tool (in)
Embankment 1	53.14	**54.32**
Embankment 1	63.8	**71.24**

Settlement of composite foundation

Description	Case history (in)	Consolidation Time Case history (months)	Calculated by the Tool (in)	Time by the Tool (months)
Settlement observation were between 5.90-27.55 but the most recorded was between 5.90-13.78	5.90-13.78	12.0	**13.78**	**12**

Note: for this project the projected settlement over a period of 25 years was expected to be 25.58 in. The tool estimnated a settlement of 0.79 31.09 in over a period of 17 years.
Note 2: Values for Cc, Cs were calculated from Recompression ratio (Cs/(1+e0) and Compression ratio (Cc/(1+eo).

<u>Case 12:</u> Deep soil mixing design under seismic conditions - a case study (Akçakal et al. 2019).

Deep mixing elements were installed under the structure of the Turkmenbashi International Seaport in Turkmenistan. The soft soil ranged between 20 and 33 ft from the seabed level. The project aimed to mitigate excessive settlement of the soft clay and to analyze the foundation’s response under seismic conditions. Post-construction consolidation settlement was limited to 2”, which was achieved. In the study, an additional load ranging from 1.25 to 2.5 ksf was applied simulate working conditions. The settlement obtained with the tool was 3.2” for the 2.5 ksf and 1.6” for 1.25 ksf. While the tool’s results, shown in Table 6-13, agree with the value from the study when applying the load of 1.25 ksf, the study did not specify whether the value of 1.36” obtained represents an average, lower or upper bound of the load applied.

Table 6-13. Calibration parameters and results summary (Case 12).

Calibration and Test Summary Sheet

Paper Title: Deep soil mixing design under seismic conditions - a case study **UNITS:** US

Soil Profile

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	PI	LL	Cc	Cs	Cv (ft2/day)	Cr (ft2/day)	kv(ft/day)
Sand Fill	8.27			127.32								
Very Soft Clay	19.68	1.00		117.77	0.80			0.15	0.03	0.26		2.6E-02
Clay 1	21.25	6.00		120.95	0.80			0.15	0.03	0.26		2.6E-02

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)	Additional load (Kip/ft2)
19.68	127.32	3.28	21.32	21.32	1.25-2.51

Soil Improvement elements Design

Embankment	Spacing (ft)	Diameter (in)	C. length (ft)	Pattern	Ec/Es
1	4.26	31.49	49.2	square	13.33

Calibration Results Summary Sheet

Total Settlement

Description		Case history (in)	Calculated by the Tool (in)
1		Not specified	10.63

Settlement of composite foundation

Description	Case history (in)	Consolidation Time Case history (months)	Calculated by the Tool (in)	Time by the Tool (months)
Tool settlement under 120 kpa	1.34	N/A	3.15	12
Tool settlement under 60 kpa		N/A	1.57	10

<u>Case 13:</u> Settlement Mitigation Using Lightweight Fill Embankment Systems (Yenigalla et al. 2013).

During the construction of a new bridge along State Highway 360 (SH360) in Arlington, the south end embankment approach of the bridge was chosen as a testing embankment to use Expanded Clay Shale (ESC) as lightweight fill material. Figure 6-5 shows the south approach embankment of the highway. An additional construction load of 1.67 ksf was included in the analysis. Settlement results obtained from the tool indicate higher values compared to those observed in the case history. The tool estimated settlements of approximately 6 and 1.56 inches for ESC with and without additional load, respectively (shown in Table 6-14). However, based on the case history, the total settlement observed was 1.7 inches for the additional load case.

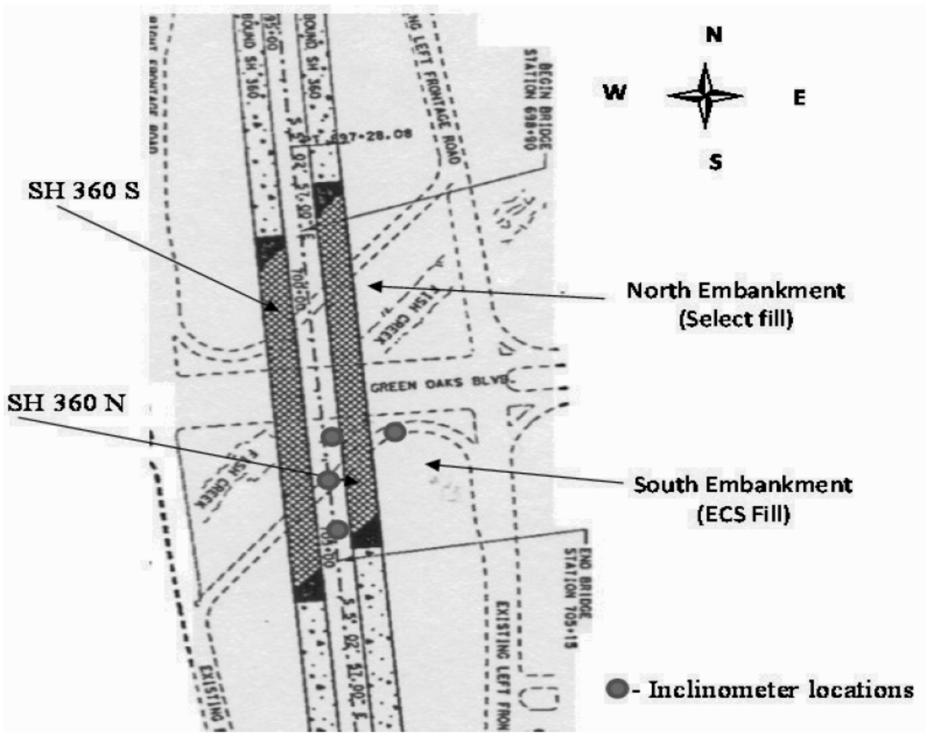


Figure 6-5. Embankment with ECS in SH 360 Arlignton, Texas (Anand J. Puppala 2013).

Table 6-14. Calibration parameters and results summary (Case 13).

Calibration and Test Summary Sheet

Paper Title: Settlement Mitigation Using Light Weight Fill Embankment Systems **UNITS:** **US**

Soil Profile

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	PI	LL	Cc	Cs	Cv(in2/s)	Cr (ft2/day)	kv(ft/day)
Soft clay 2	16.00	2.20	91.00		0.80			0.34	0.023	5.4E-05		2.88E-05
well-graded sand	10.00		96.70									

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)	Addionat traffic load (Kl/ft2)
30	94.2	N/A	170	50	1.67

Soil Improvement elements Design

Fill Type	Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)
ESC	30	39.8	N/A	170	50

Calibration Results Summary Sheet

Total Settlement

Description	Case history (in)	Calculated by the Tool (in) w/o additional load	Calculated by the Tool (in) w additional load
Expanded clay shale	1.7	**1.56**	**6**
Regular fill	7.50	**9.36**	**13.08**

<u>Case 14:</u> Case Study of EPS Geofoam Lightweight Fill for Settlement Control at Bridge Approach Embankment (Anand J. Puppala 2013).

Geofoam Lightweight fill was used to mitigate the excessive settlement on the bridge approach at Maine Turnpike Beech Ridge Road overpass. The goal was to reduce a settlement from approximately 12 inches in a soft silty clay overlaying a silty sand. The settlement criterion was to limit differential settlements between the bridge and the approach embankment to 6 inches or less. The height of the geofoam fill was 14 ft, adjusted to include a two-inch layer of sand fill for geofoam protection. The results from the tool indicated a greater settlement compared to the one observed in the study, as shown in Table 6-15.

Table 6-15. Calibration parameters and results summary (Case 14).

Calibration and Test Summary Sheet

Paper Title: Case Study of EPS Geofoam Lightweight Fill for Settlement Control at Bridge Approach Embankment **UNITS:** **US**

Soil Profile

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	Cs/(1+e0	Cc/(1+e0	Cc	Cs	Cv(in2/s)	Cr (ft2/day)	kv(ft/day)
Fill	17.00		120.00									
Silty Clay	61.00	1.87		118.00	1.20	0.02	0.15	0.33	0.04	0.13		2.9E-05
Silty sand	10.00											

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)	Addionat traffic load (Kl/ft2)
16	120	17	120	36	1.28

Soil Improvement elements Design

Fill Type	Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)
Geofoam	14	1.8	17	120	36

Calibration Results Summary Sheet

Total Settlement

Description	Case history (in)	Calculated by the Tool (in) w additional load
Total estimated settlement with regular fill	12	**19.2**

Settlement of Light weight fill

Description	Case history (in)	Consolidation Time Case history (years)	Calculated by the Tool (in)	Time by the Tool (years)
Total settlement observed and estimated in the case history ranges 0.1-9.5 The settlement (1.2) is from the last observation and reading taken in the case history.	1.2	1.25	**2.18**	**1.25**

<u>Case 15:</u> Settlement of group of pile example. Foundation Engineering (Braja M. Das 2019)

A case history using a group of piles to mitigate settlement of clay soil was used to validate the pile module of the calculation tool. The results demonstrate great agreement between the theoretical and calculated values, as shown in Table 6-16.

Table 6-16. Calibration parameters and results summary (Case 15).

Calibration and Test Summary Sheet												
Paper Title:	Settlement of group of Pile example. Foundation Engineering							UNITS:	US			
Soil Profile												
Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	PI	LL	Cc	Cs	Cv (ft2/day)	Cr (ft2/day)	kv(ft/day)
Sand	6.56	2.10	c									
Clay	52.48	2.09		114.59	0.82	12.00		0.30	0.03			
Clay	13.12	3.86		120.32	0.70	12.00		0.20	0.02			
Clay	6.56	4.45		120.95	0.75	12.00		0.25	0.03			

Embankment/Retaining Wall Parameters				
Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)
47.56	120.95	6.56	7.22	7.22

Soil Improvement elements Design					
Embankment	Spacing (ft)	Diameter (in)	Pattern	Leght (depth) of Pile (ft)	length of group (ft)
1	3.28	19.7	square	54.08	10.82

Calibration Results Summary Sheet

Total Settlement		
Description	Case history (in)	Calculated by the Tool (in)
Embankment 1	Not specified	0.51

Settlement of the pile foudation				
Description	Case history (in)	Consolidation Time Case history (years)	Calculated by the Tool (in)	Time by the Tool (years)
	0.18	not specified	0.18	65

<u>Case 16:</u> Analyses of a pile-supported embankment over soft clay: Full-scale experiment, analytical and numerical approaches (Nunez et al. 2013).

A full-scale embankment test was conducted over a pile foundation at the Chelles test site in France. The primary objective of the test was to analyze and compare the performance of the pile-improved foundation using a numerical analysis versus observational data. The embankment was 16 ft in height and was constructed over soft alluvial soil. Soil properties are shown in Table 6-17. The settlement estimated with the tool agrees with the settlement obtained from the analysis and observation data from the study.

Table 6-17 Calibration parameters and results summary (Case 16).

Calibration and Test Summary Sheet

Paper Title:	Analyses of a pile-supported embankment over soft clay: Full-scale experiment, analytical and numerical approaches	**UNITS:**	**SI**

Soil Profile

Soil Type	Thickness (ft)	SPT	Unit weight (lb/ft3)	Sat U. weight (lb/ft3)	eo	PI	LL	Cc	Cs	Cv (ft2/day)	Cr (ft/day)	kv(ft/day)
Silty Clay	5.58	0.64	127.32		1.00			0.20	0.03	0.595		
Clay	1.97	1.00		95.49	1.70			0.54	0.05	0.595		
Sandy Clays	13.78	1.30		127.32	0.70			0.10	0.01	0.595		
Sandy clays	6.56	1.97		127.32	0.60			0.13	0.01	0.595		
Sand and gravel	16.40	2.73		127.32								

Embankment/Retaining Wall Parameters

Height (ft)	U weight (lb/ft3)	G. water depth (ft)	Bottom Width (ft)	Top Width (ft)	Additional load (Kpa)
16.4	121.59	6.56	78.72	26.24	

Soil Improvement elements Design

Embankment	Spacing (ft)	Diameter (in)	Pattern	Leght (depth) of Pile (ft)	length of group (ft)
1	6.56	23.62	square	27.55	91.84

Calibration Results Summary Sheet

Total Settlement

Description	Case history (in)	Calculated by the Tool (in)
Embankment 1	10.43	**11.81**

Settlement of the pile foudation

Description	Case history (in)	Consolidation Time Case history (years)	Calculated by the Tool (in)	Time by the Tool (years)
The range shows the measures taken in different settlement plates	1.18-3.35	not specified	1.69	20

6.3 CONCLUSION

The calculation tool developed in this project can analyze various soil conditions and accurately estimate consolidation time and settlement, as shown in Figure 6-6, Figure 6-7, and Figure 6-8. Particularly, when sufficient soil properties data is available. However, if some soil properties are missing, the correlations incorporated in the tool can approximately estimate the missing soil parameters, but the accuracy of the results may be impacted. Therefore, to obtain accurate estimates on consolidation time and settlement, it is recommended that soil be tested, and soil properties be obtained.

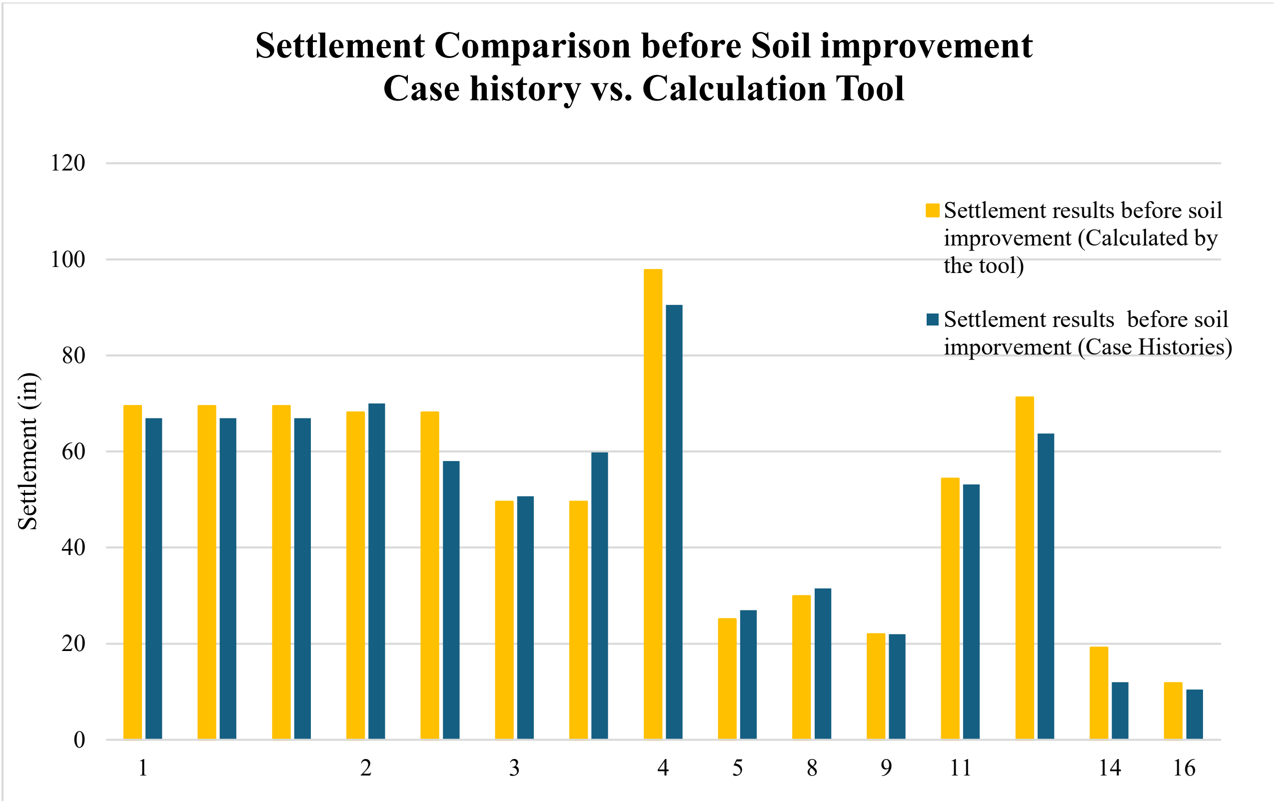


Figure 6-6. Settlement comparison before soil treatment case histories vs. calculation tool.

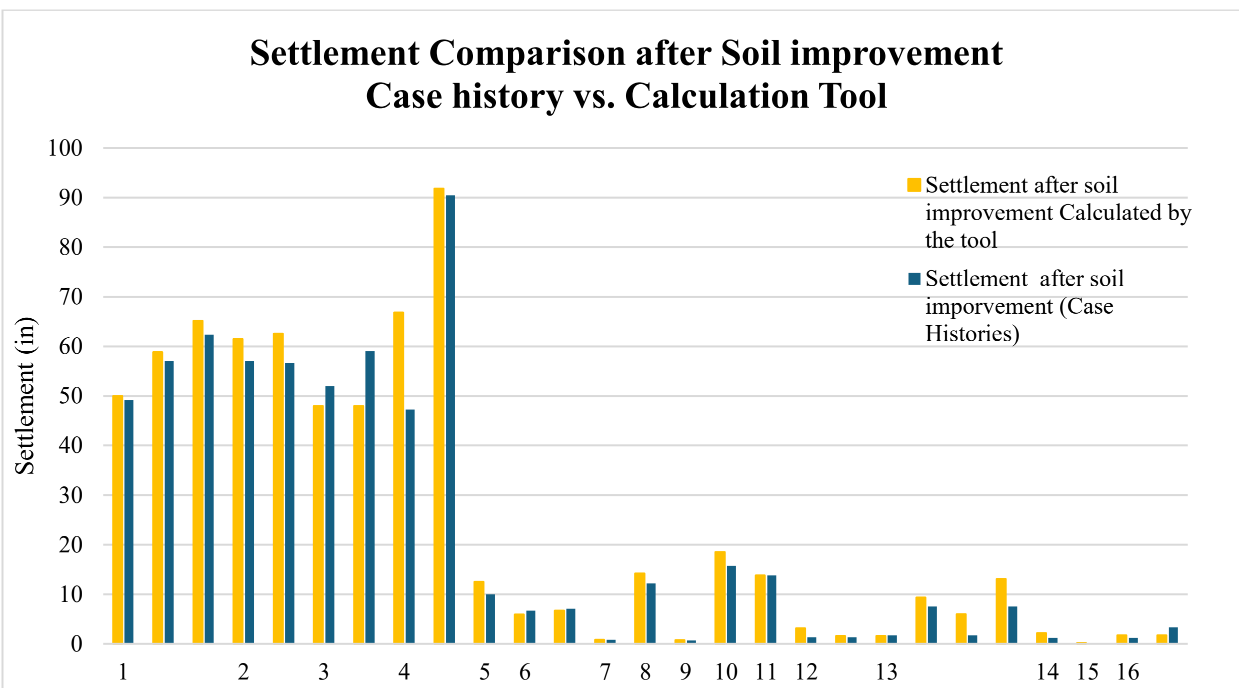


Figure 6-7. Settlement comparison after soil treatment case histories vs. calculation tool.

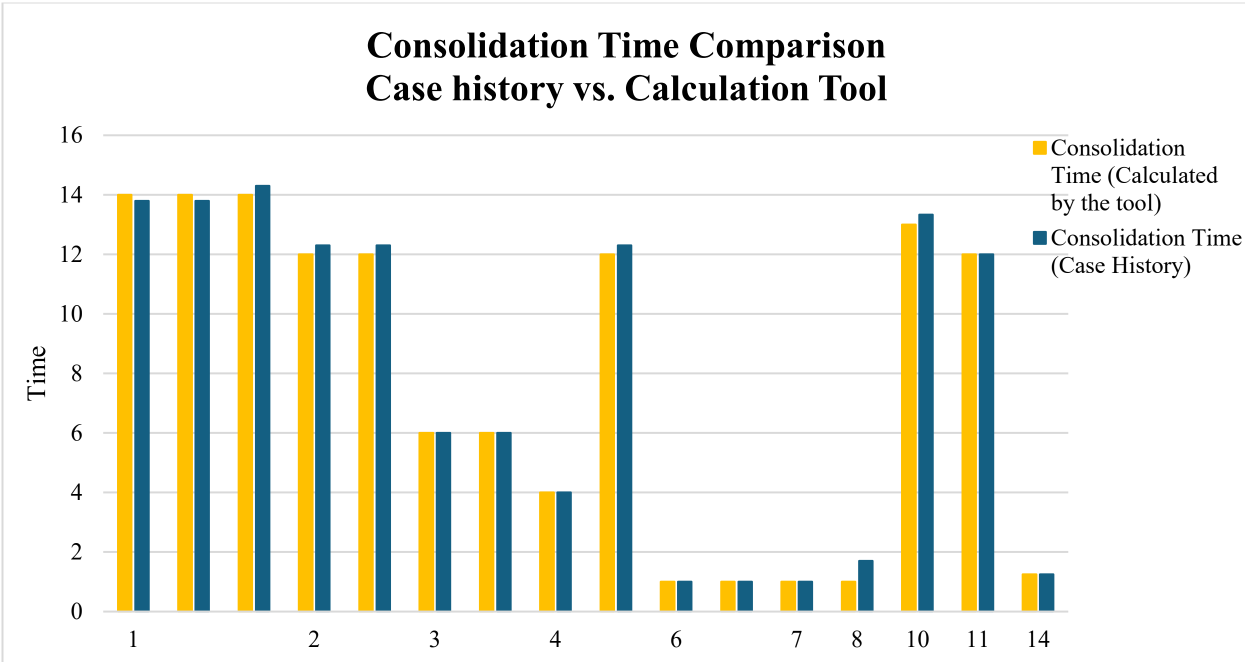


Figure 6-8. Comparison of consolidation time required case histories vs. calculation tool.

CHAPTER 7 CONCLUSIONS

This project received substantial support from many state DOTs as the topic is also an interest to them. Upon completion of this project, the following conclusions can be drawn:

- State DOTs practice dramatically different in terms of zoning and settlement criteria for their embankments and retaining walls that support roadways.
- Georgia, South Dakota and Montana do not have settlement criteria. The remaining 19 states have some criteria even though the criteria may not appear in any written document; for example, Arkansas, Oklahoma.
- Florida, New Jersey and Ohio do not have a fixed value for allowable settlement but require the embankment settlement to be compatible with the bridge settlement so that the differential settlement between bridge and approach is insignificant.
- Among the ten states having zoning, six use the approach slab as Zone 1 and the remaining four use the distance from the bridge to define their Zone 1.
- Among the ten states having specific settlement criteria, 0.5 inches or 1 inch is used for Zone 1 expect for South Carolina which uses 0.05*Lslab (note: Lslab is slab length in feet and the amount of settlement determined is in inches) to determine allowable settlement.

It is recommended that state DOT should consider the project cost and construction time when specifying the settlement criteria.

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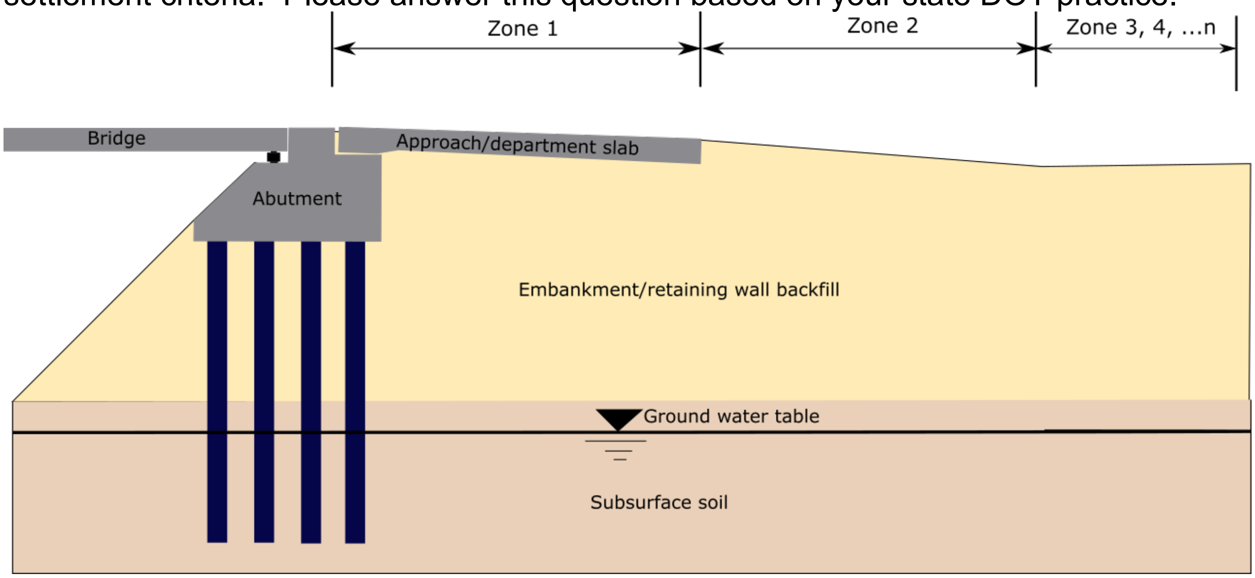
APPENDIX – A: SURVEY FORM

Participant information			
Name		**State DOT**	
Email		**Phone**	

Question 1: Many DOTs divide an embankment into different zones based on its distance from a bridge abutment (as shown in the figure below) and specify the settlement criteria. Please answer this question based on your state DOT practice.



Please answer the following questions by referring to figure above. (Please assume flexible pavement for this question.)

Choice of zoning scenario at your DOT	
Single zone ☐ Two zones ☐ Three zones ☐ Four zones ☐ More than four zones ☐	
Zoning criteria and settlement criteria (Please answer the questions based on your choice of zoning)	
Single zone	Allowable settlement _______________ inches
Two zones	Zone 1: Range _______ ft; Allowable settlement _________ inches
	Zone 2: Range _______ ft; Allowable settlement _________ inches
Three zones	Zone 1: Range _______ ft; Allowable settlement _________ inches

	Zone 2: Range ________ ft; Allowable settlement _________ inches
	Zone 3: Range ________ ft; Allowable settlement _________ inches
Four zones	Zone 1: Range ________ ft; Allowable settlement _________ inches
	Zone 2: Range ________ ft; Allowable settlement _________ inches
	Zone 3: Range ________ ft; Allowable settlement _________ inches
	Zone 4: Range ________ ft; Allowable settlement _________ inches
More than four zones	Please specify your zoning criteria and settlement criteria:

Question 2: Do you use the same criteria for rigid and flexible pavements?

Yes ☐; No ☐

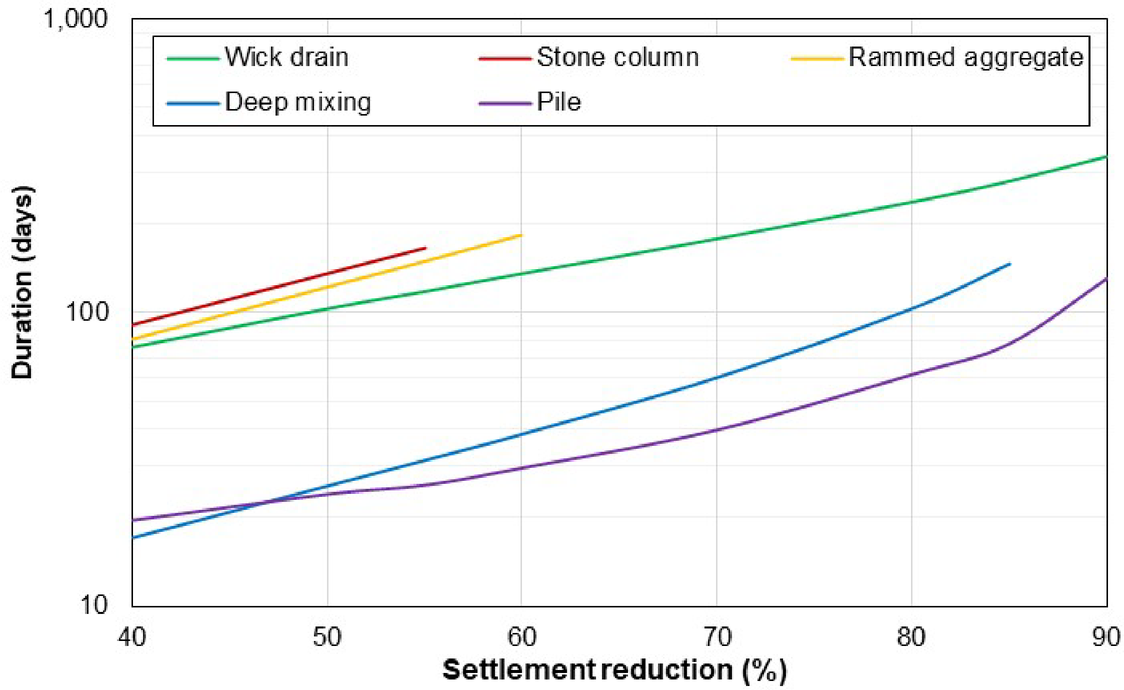
Question 3: If you answer "No" to Question 2, please complete the form below by assuming rigid pavement.

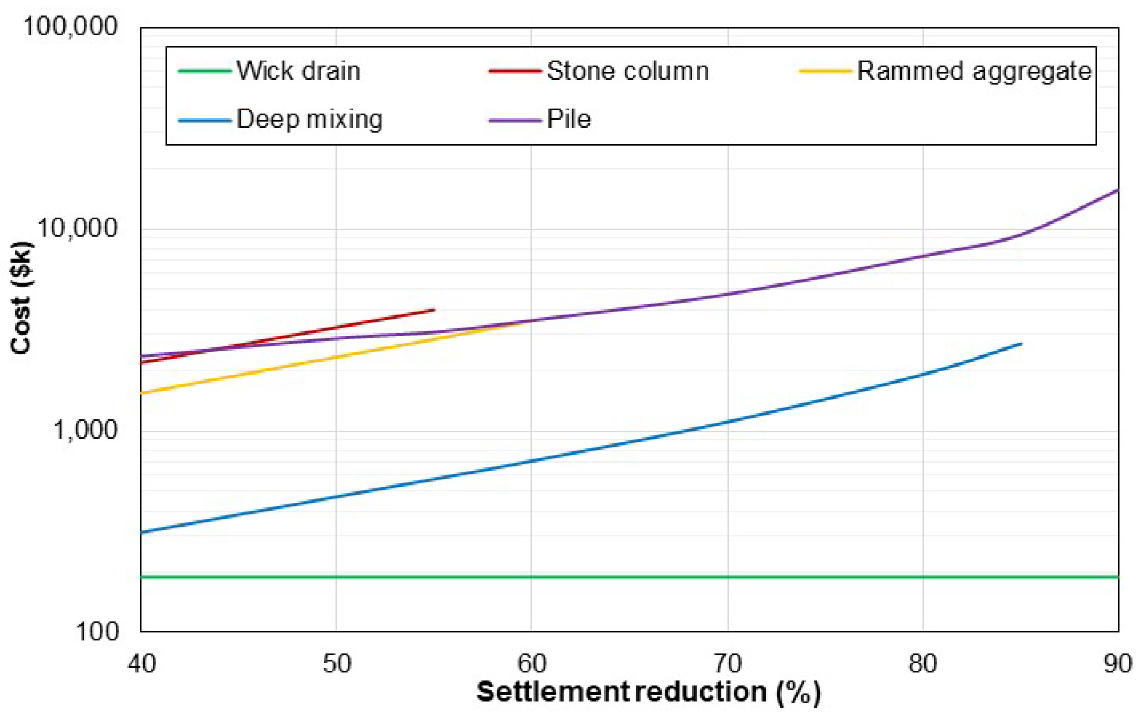
Choice of zoning scenario at your DOT	
Single zone ☐ Two zones ☐ Three zones ☐ Four zones ☐ More than four zones ☐	
Zoning criteria and settlement criteria (Please answer the questions based on your choice of zoning)	
Single zone	Allowable settlement ________________ inches
Two zones	Zone 1: Range ________ ft; Allowable settlement _________ inches
	Zone 2: Range ________ ft; Allowable settlement _________ inches
Three zones	Zone 1: Range ________ ft; Allowable settlement _________ inches

	Zone 2: Range _______ ft; Allowable settlement ________ inches
	Zone 3: Range _______ ft; Allowable settlement ________ inches
Four zones	Zone 1: Range _______ ft; Allowable settlement ________ inches
	Zone 2: Range _______ ft; Allowable settlement ________ inches
	Zone 3: Range _______ ft; Allowable settlement ________ inches
	Zone 4: Range _______ ft; Allowable settlement ________ inches
More than four zones	Please specify your zoning criteria and settlement criteria:

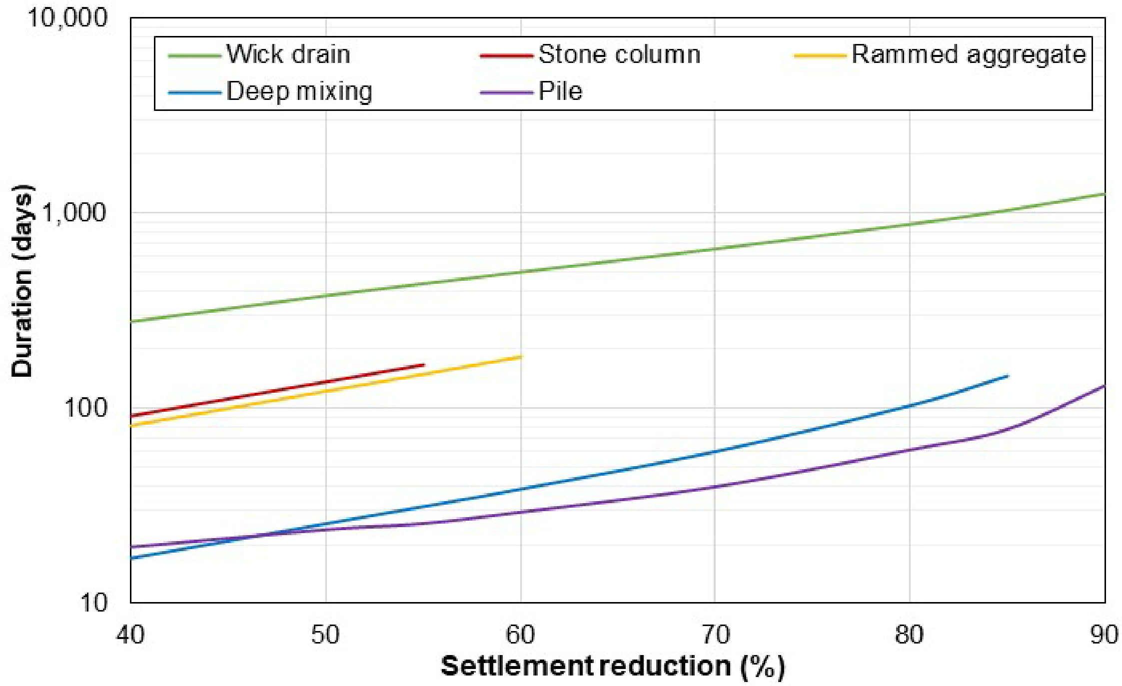
<u>Question 4</u>:
If you would like to provide additional information, please provide it below:

APPENDIX – B: DESIGN CHARTS





<u>Figure A-1</u>: Cost and duration of different methods (PI = 20)



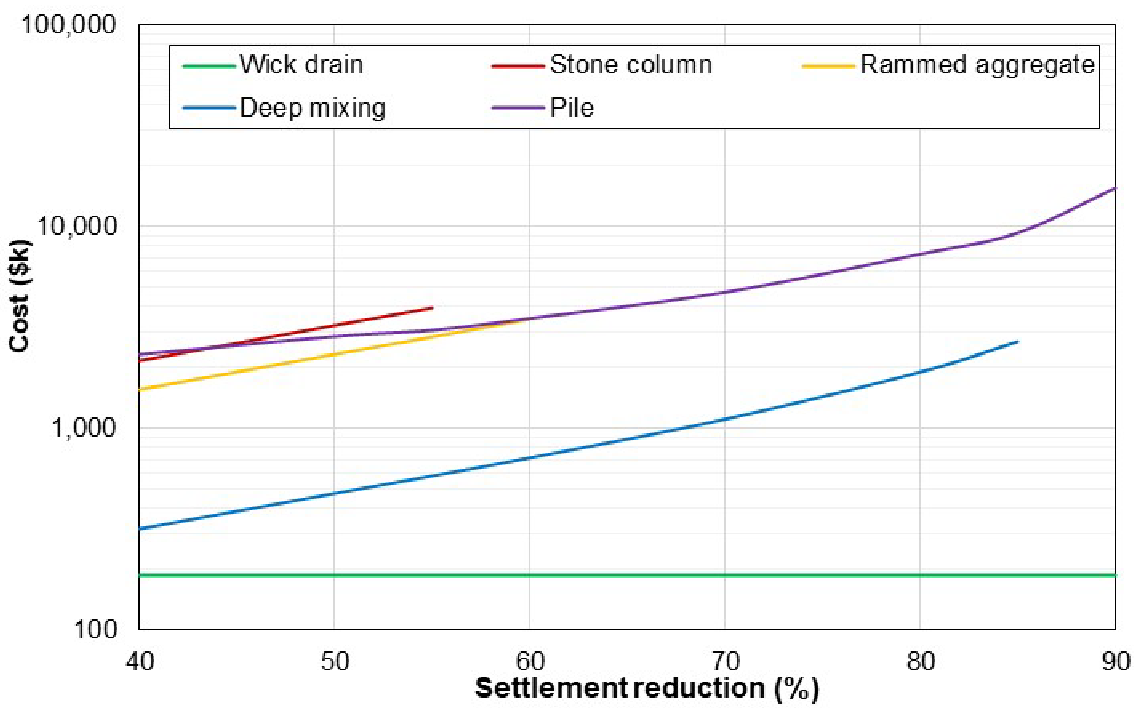
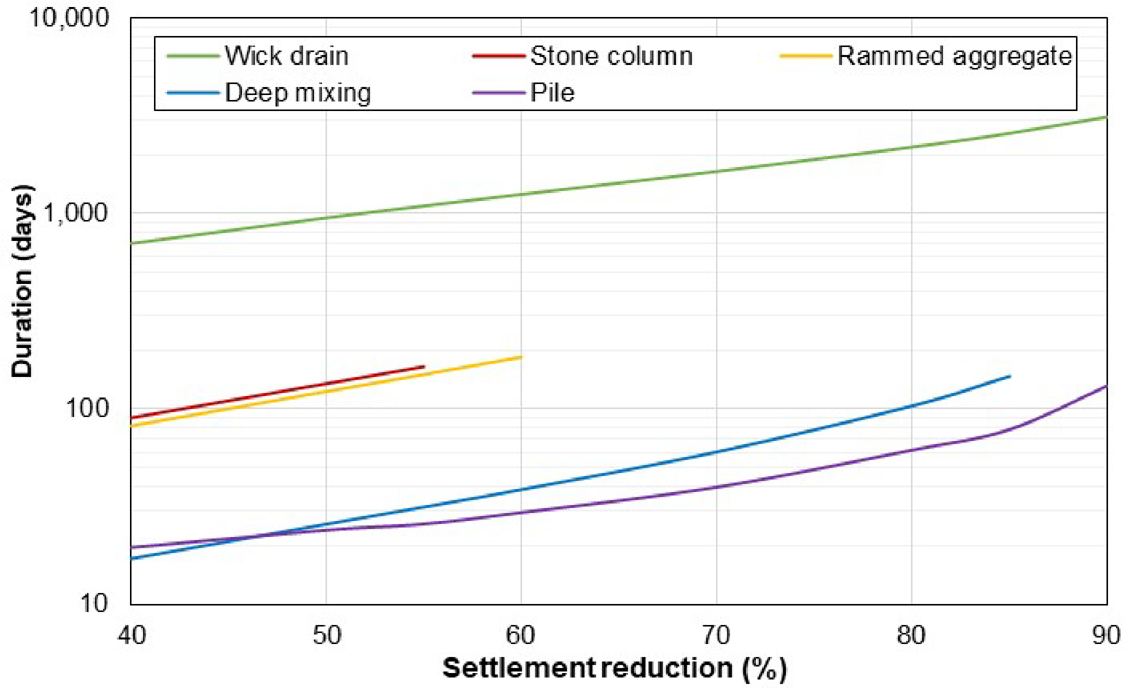
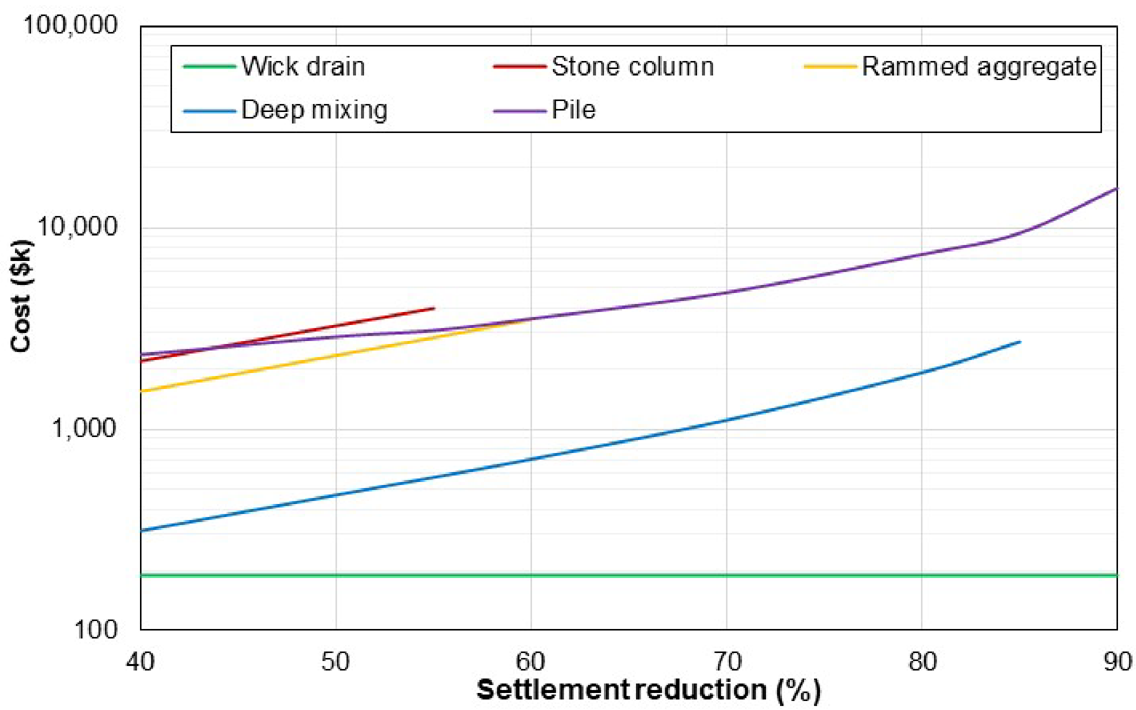


Figure A-2: Cost and duration of different methods (PI = 30)





<u>Figure A-3</u>: Cost and duration of different methods (PI = 40)

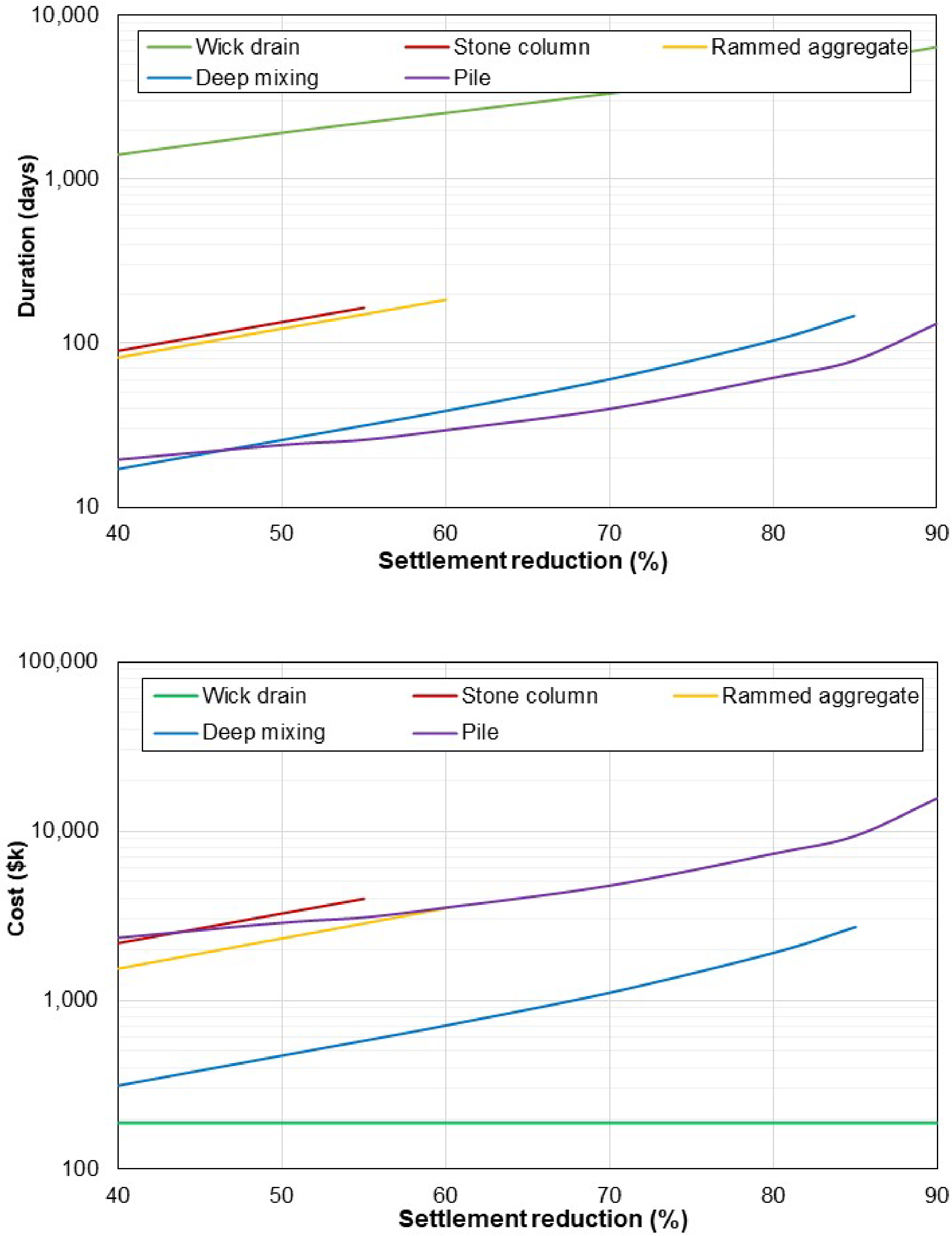
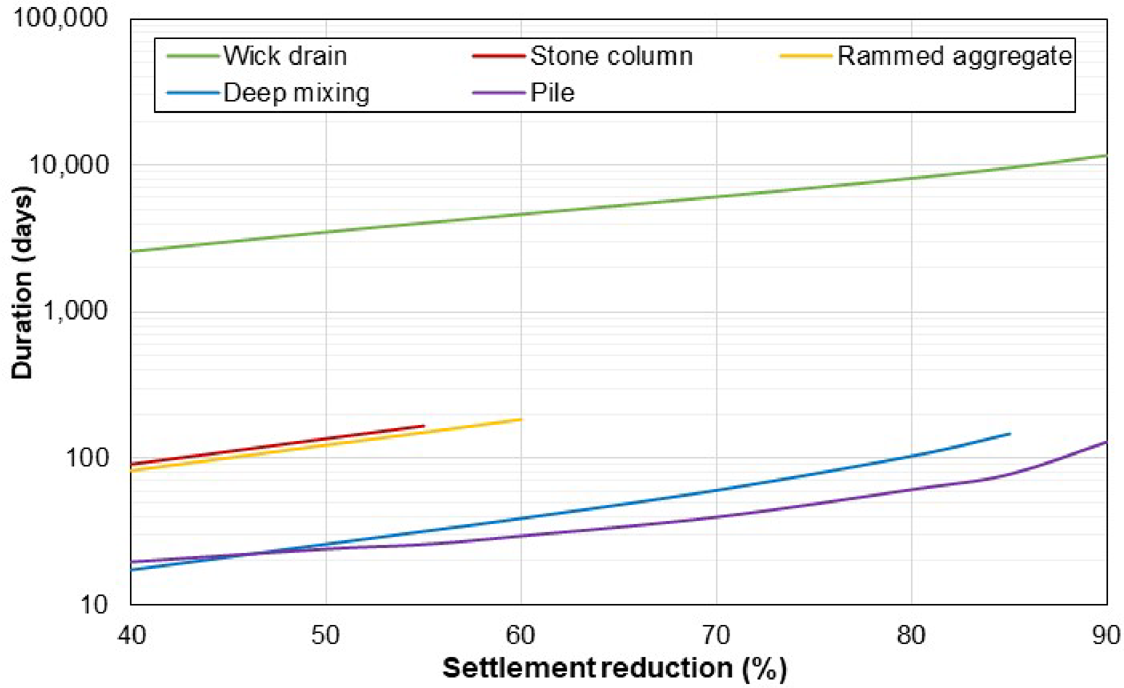


Figure A-4: Cost and duration of different methods (PI = 50)



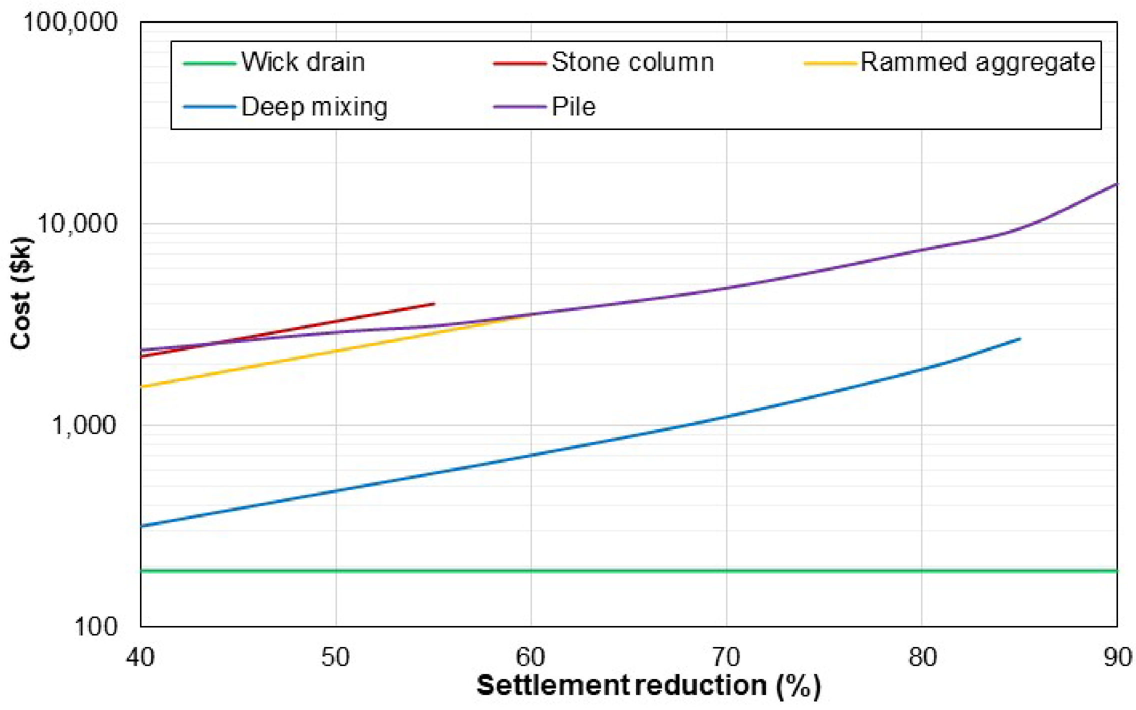


Figure A-5: Cost and duration of different methods (PI = 60)

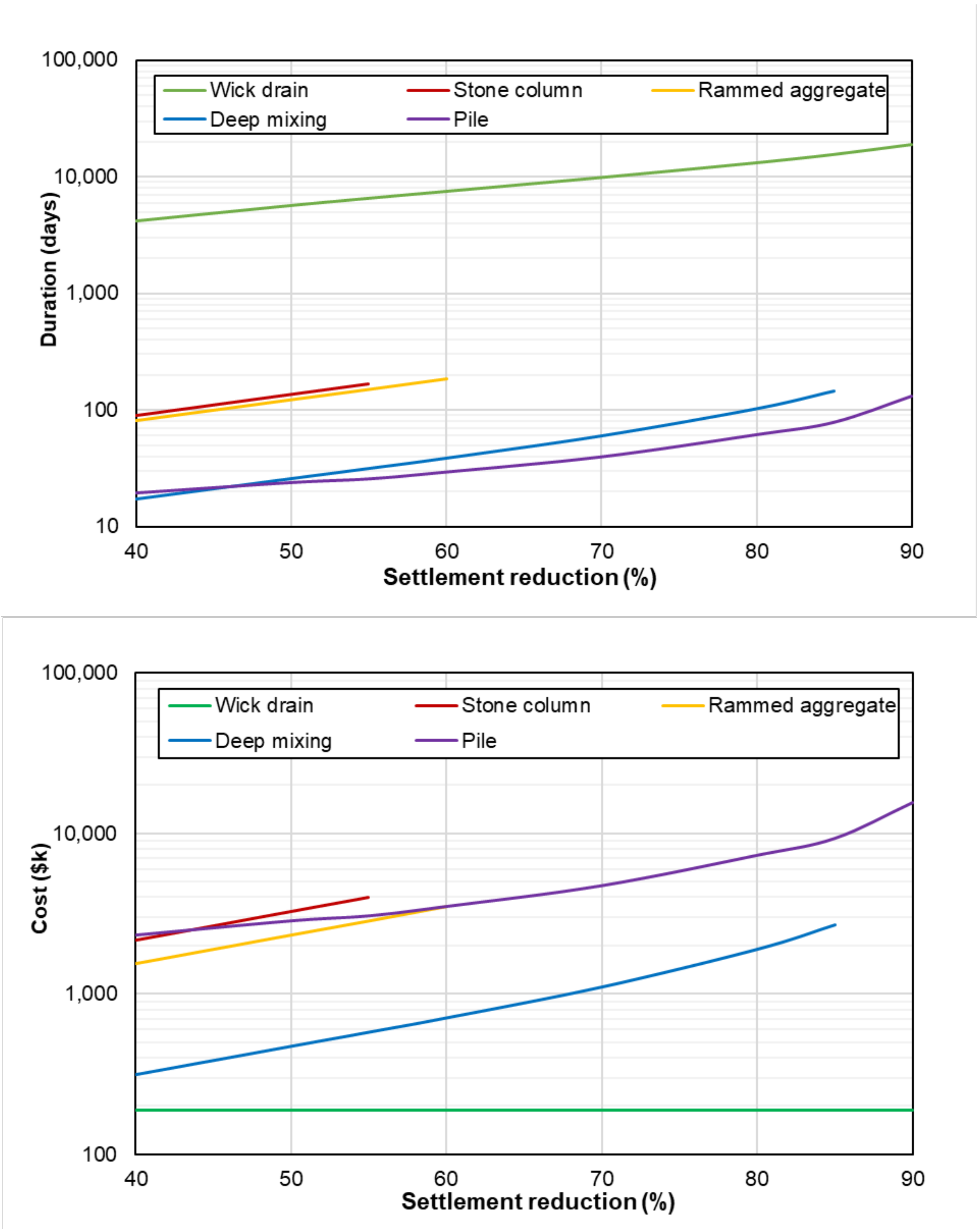


Figure A-6: Cost and duration of different methods (PI = 70)