Technical Report Documentation Page

1. Report No. FHWA/TX-09/0-6022-1	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle RECOMMENDATIONS FOR DESIGN, CONSTRUCTION, AND MAINTENANCE OF BRIDGE APPROACH SLABS: SYNTHESIS REPORT		5. Report Date August 2008 Published: April 2009
		6. Performing Organization Code
7. Author(s) Anand J. Puppala, Sireesh Saride, Ekarut Archeewa, Laureano R. Hoyos, and Soheil Nazarian		8. Performing Organization Report No. Report 0-6022-1
9. Performing Organization Name and Address Department of Civil Engineering The University of Texas at Arlington, Arlington, Texas 76019 Department of Civil and Environmental Engineering The University of Texas at El Paso, El Paso, Texas 79968		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. Project 0-6022
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P. O. Box 5080 Austin, Texas 78763-5080		13. Type of Report and Period Covered Technical Report: September 2007 – August 2008
		14. Sponsoring Agency Code

15. Supplementary Notes
Project performed in cooperation with the Texas Department of Transportation and the Federal Highway Administration.
Project Title: Recommendations for Design, Construction, and Maintenance of Bridge Approach Slabs
URL: http://tti.tamu.edu/documents/0-6022-1.pdf

16. Abstract

Bridge approaches provide smooth and safe transition of vehicles from highway pavements to bridge structures. However, settlement of the bridge approach slab relative to bridge decks usually creates a bump in the roadway. The bump causes inconvenience to passengers and increases the cost of maintenance and repairing of the distressed approach slabs. Typically, the Texas Department of Transportation (TxDOT) spends millions of dollars annually to mitigate the bump problem across the state. The present research aims to better understand the mechanisms that cause the bump problem, to review currently used methods to mitigate this problem around the world, and to develop the methods that are appropriate for researching them in real field conditions. As a part of this research, a synthesis was prepared by conducting a comprehensive literature review of the past research on the subject and also by conducting a survey with all 25 districts to understand the local conditions that contribute to the bump problem in the bridges. The literature review also identified several technologies that were used to mitigate the problem. All these details along with district wide surveys are covered in this synthesis report.

17. Key Word Bump, Settlement, Embankment, Foundation, Erosion, Approach Slab, Mitigation Techniques, Maintenance Measures	18. Distribution Statement No restrictions. This document is available to the public through NTIS: National Technical Information Service Springfield, Virginia 22161 http://www.ntis.gov

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	186	

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RECOMMENDATIONS FOR DESIGN, CONSTRUCTION, AND MAINTENANCE OF BRIDGE APPROACH SLABS: SYNTHESIS REPORT

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Report 0-6022-1
Project 0-6022

Sponsored by the
Texas Department of Transportation
In Cooperation with the
U.S. Department of Transportation
Federal Highway Administration

August 2008
Published: April 2009

The University of Texas at Arlington
Arlington, Texas 76019-0308

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The contents of this report reflect the views of the authors/principal investigators who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the views or policies of the Federal Highway Administration (FHWA) or the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation. The researcher in charge was Dr. Anand J. Puppala, P.E., Department of Civil Engineering, The University of Texas at Arlington, Texas.

ACKNOWLEDGMENTS

This study was sponsored by the Texas Department of Transportation (TxDOT) under Research Project 0-6022. The authors would like to express their thanks to the Project Committee and those in the TxDOT Districts who provided valuable information during the development of the research program. Special thanks go to Project Director, Richard Williammee, P.E., for his comments and careful review of the draft report. The authors also would like to thank the Project Monitoring Committee members David Head, P.E.; Bahman Afsheen, P.E.; Darrell Anglin, P.E.; Bernie Holder, P.E.; Jon Holt, P.E.; Mark McClelland, P.E.; Taya Retterer; Stanley Yin, P.E.; and German Claros, Ph.D, P.E. for their valuable input during the preparation of this report.

The findings, opinions, recommendations, and conclusions expressed in this report are those of the authors and do not necessarily reflect the views of the sponsor and administrators.

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1. INTRODUCTION

Bridge approaches provide smooth and safe transition of vehicles from highway pavements to bridge structures and vice versa. However, settlement and/or heave related movements of bridge approach slabs relative to bridge decks usually create a bump in the roadway. This is a typical occurrence at the end of the bridge decks and requires a solution, because this uneven transition may cause severe damage to bridge decks, inconvenience to passengers, reduced steering control for travelers, distraction to drivers, lower public perception of transportation agency's image, increased maintenance work/costs, and constant delays to rehabilitate the distressed lanes. In addition, when heavy trucks travel over these pavement transitions, impact loads created can severely damage the structure, and at the very least, cause an uncomfortable or unsafe ride.

In the United States, state departments of transportation (DOTs) have been spending considerable amounts of their maintenance budgets to alleviate problems caused by the bumps every year. Bridge approach settlement is reported to induce damage to 25percent of the bridges nationwide (approximately 150,000 bridges) with an estimated annual maintenance or repair costs well over $100 million (Briaud et al., 1997). Additionally, as per FHWA's report on 'Priority, Market-Ready Technologies and Innovations' (FHWA-HRT-04-053), traffic congestion due to repair works results in 5.7 billion of person-hours of delay. For each person, this delay averages to 36 hours per year. Maintenance of bridge bumps are an important part of the bridge repair works, and hence bump repairs often result in traffic delays and congestion problems.

Briaud et al. (1997) reported that 30 percent of bridges in Texas, i.e., 13,800 out of 46,000 bridges were subjected to the same distress. A recent TxDOT survey on bridges shows that the total number of bridges in Texas is approximately 49,829, which is 40 percent more than the next state with the next highest number of bridges in the U.S.A. (Report on Texas Bridges, 2006). This number includes both on-system (32,674) and off-system (17,155) bridges (Report on Texas Bridges, 2006). Another study cited annual costs for "bump" repairs in Texas around $7 million (Seo, 2003). Constant maintenance work, closure of lanes, traffic control, and traffic delays has made this a major if not a premier maintenance problem in Texas and in the majority of states.

Several mitigation methods using driven piles, drilled shafts, stone columns, controlled low strength materials or flowable fills, geosynthetics, and low density materials have been utilized with mixed results (Briaud et al., 1997; Seo, 2003). One of the important objectives of this research is to synthesize the causes of approach settlement and bump problems and then enlist various approach settlement mitigation technologies along with any new or unproven technologies that show potential to reduce bump and differential settlement problems for subgrade conditions of Texas. An attempt was made to review the recent literature on this subject topic, and a summary of the findings from the literature review is included in this report.

The following sections discuss the definition, each issue, and probable causes of the bump problem at the end of the bridge.

1.1 Definition of the "Bump"

Generally, roadway and embankments are built on subgrade foundation and compacted fill materials, respectively, which undergo load induced compression with time. These compressions lead to settlements. The total settlement of the bridge is typically much smaller than the total settlement of the roadway or adjacent embankment and results in considerable differential settlement at the intersection. As a result, a noticeable bump develops at the bridge ends.

In general, the "Bump" can be defined as the differential settlement at the area between the bridge and roadway interfaces. Differential settlement is an occurrence normally found where two foundations of two collaborating structures have been built by different concepts. In highway engineering, this phenomenon can be found at the intersection between the bridge and the roadway. Normally, differential settlements (change in grade) of 0.3 percent in any direction for a 40-year design period are allowed (Hsi, 2007). Typically the bridges need to rest on deep foundations such as pile, pier, or other types of deep foundation systems resting on a firm foundation material such as bedrock. In contrast, the acceptable settlement of roadway and embankment is higher than that of the bridge system (Hsi, 2007).

The "Bump" can affect drivers varying from feeling uncomfortable to being hazardous for their lives (Hopkins, 1969; Ardani, 1987). In order to eliminate the effects of the bump, the approach slab must be built to provide a smooth grade transition between these two structures (bridge and roadway). Another function of the approach slab is to keep the magnitude of differential settlement within a control limit (Mahmood, 1990; Hoppe, 1999). However, in practice it is found that the approach slabs also exceed differential settlements (Mahmood, 1990;

Hoppe, 1999). In such cases, the approach slab moves the differential settlement problem at the end of the bridge to the end of the slab connecting with the roadway.

Hence, the "Bump" or "Approach Settlement" can be defined as the differential settlement or heave of the approach slab with reference to the bridge abutment structure.

1.2 Bump Tolerances

The differential settlement near the bridge approach is a common problem that plagues several bridges in the state of Texas (Jayawickrama et al., 2005). One of the major maintenance problems is to establish severity levels of the bump that require remedial measures. The differential settlement tolerances need to be established for consideration of when to initiate the repair works.

Walkinshaw (1978) suggested that bridges with a differential settlement of 2.5 inches (63 mm) or greater needs to be repaired. Bozozuk (1978) stated that settlement bumps could be allowed up to 3.9 inches (100 mm) in the vertical direction and 2.0 inches (50 mm) in the horizontal direction. Several researchers define the allowable bumps in terms of gradients as a function of the length of the approach slab. Wahls (1990) and Stark et al. (1995) suggested an allowable settlement gradient as 1/200 of the approach slab length. This critical gradient was also referred by Long et al. (1998), which was used by the Illinois DOT for initiating maintenance operations.

Das et al. (1990) used the International Roughness Index (IRI) to describe the riding quality. The IRI is defined as the accumulations of undulations of a given segment length and is usually reported in m/km or mm/m. The IRI values at the bridge approaches of 3.9 (mm/m) or less indicates a very good riding quality. On the other hand, if the IRI value is equal to 10 or greater, then the approach leading to the bridge is considered as a very poor riding quality. Albajar et al. (2005) established a vertical settlement on the transition zone of 1.6 inches (4 cm) as a threshold value to initiate maintenance procedures on bridge approach areas. In Australia, a differential settlement or change in grade of 0.3 percent both in the transverse and the longitudinal direction and a residual settlement of 100 mm (for a 40 year design period) are considered as limiting values for bridge approach settlement problems (Hsi and Martin, 2005; Hsi, 2007).

In Texas, the state of practice for repair strategies is different from District to District, and these repairs are typically based on visual surveys (Jayawickrama et al., 2005) and International Roughness Index (IRI) values (James et al., 1991). In the study by James et al. (1991), it was indicated that several Districts in Texas have reported bump problems and a few Districts have explored methods such as Urethane injection to moisture control to mitigate settlements. However, these methods have only provided temporary relief as the settlement continues to increase with the service life. As a part of Jayawickrama et al. (2005) study, researchers visited three bridge sites in the Waco, Houston, and San Antonio Districts where Urethane injection was adopted to mitigate approach settlement problems. Their findings are discussed in detail in the subsequent sections of this report.

1.3 Synthesis

The previous investigations have helped in the identification of the mechanisms causing and use of technologies to mitigate this problem. However, it is still necessary to revisit this bump problem by researching the most recent ground improvement technologies that have been used or have potential to mitigate this bump problem. Such techniques should be easily implementable in both new bridge construction projects and existing bridge maintenance repairs, be less expensive, and depend on sound and rational design methods.

The main focus of this synthesis is to compile the existing and on-going national and worldwide research on the process of bridge approach settlement, methods explored to reduce them, and new research efforts to explore new ground and foundation improvement methods to reduce the differential soil settlement problems. This synthesis is presented in several sections, first explaining the mechanisms causing the bump followed by several design alternatives to mitigate the approach settlement problem of new bridges and maintenance measures for distressed approach slabs.

There have been many studies employed across the states in the U.S.A. to study the causes of the problem and the methodologies to solve it (Hopkins, 1969, 1985; Stewart, 1985; Greimann et al., 1987; Laguros et al., 1990; Kramer and Sajer, 1991; Ha et al., 2002; Jayawickrama et al., 2005; White et al., 2005, 2007). The causes can be very variable and are still too complex to identify them easily. However, the primary sources of the problem can be broadly divided into four categories: material properties of foundation and embankment; design criteria for bridge foundation, abutment, and deck; construction supervision of the structures; and maintenance criteria. It should be noted that not all the factors contribute to the formation of the bump concurrently. The following section discusses the mechanisms that trigger the bump problem in detail.

2. MECHANISMS CAUSING THE FORMATION OF 'BUMP'

Bridge approach settlement and the formation of the bump is a common problem that draws significant resources for maintenance and creates a negative perception of the state agencies in the minds of transportation users.

White et al. (2005) define the term "bridge approach," not just in terms of the approach slab alone, but in terms of a larger area, covering from the bridge structure (abutment) to a distance of about 100 ft away from the abutment. This definition includes the backfill and embankment areas under and beyond the approach slab as significant contributors to the settlements in the bridge approach region.

Many factors are reported in the literature that explains the mechanisms causing the formation of bumps on the bridge transition (Hopkins, 1969; Stewart, 1985; Kramer and Sajer, 1991). According to Hopkins (1969), the factors causing differential settlement of the bridge approaches are listed as:

a. Type and compressibility of the soil or fill material used in the embankment and foundation;
b. Thickness of the compressible foundation soil layer;
c. Height of the embankment; and
d. Type of abutment.

Kramer and Sajer (1991) and Briaud et al. (1997) concurred with these observations later based on extensive surveys of various state DOTs in the U.S.A. Stewart (1985) performed a research study for Caltrans and this study concurred with the finding reported by Hopkins (1969), in particular the observations noting that the original ground and fill materials contribute the maximum settlement to the approach slab. Based on the results obtained from a field study performed at Nebraska, Tadros and Benak (1989) confirmed that the primary cause of this problem is due to the consolidation of foundation soil but not the consolidation of the compacted embankment fill. The proper compaction of the embankment in accordance with the construction specifications has an important influence on the settlement of embankment fill material. Also, the swell and shrink behaviors of the foundation/backfill soil and vibration or movements of the backfill soil (in case of granular fill) due to moving traffic loads may significantly impact the development of the approach faults (Hopkins, 1969, 1985).

Ardani (1987), Wahls (1990), and Jayawickrama et al. (2005) also reported that both the time-dependent settlement (primary/secondary consolidation) of foundation soil beneath the embankment and the approach slab embankment, as well as the poor compaction of embankment adjacent to the abutment and erosion of soil at the abutment face and poor drainage system around the abutment, are the major contributors to approach settlement problems.

Wahls (1990) claimed that the approach-slab design and the type of abutment and foundation can affect the relative settlement of the slab and bridge abutment. Abutments supported by the shallow foundations and when these foundations lay within the approach embankment fill will settle along with the embankment. In addition, Wahls (1990) concluded that the lateral creep of foundation soils and lateral movement of abutments can potentially cause this problem.

Laguros et al. (1990) reported that factors including the age of the approach slab, height of embankment, skewness of the bridge and traffic volume influence the bridge approach settlement. The flexibility of the approach pavements has a considerable influence as well. Laguros et al. (1990) observed greater differential settlement in flexible pavements than rigid pavements during initial stages following construction (short term performance), while both pavement types performed similarly over the long term. More details are provided in later sections.

Other factors that influence the creation of the bump include the type of bridge abutment and approach slab design (Mahmood, 1990; Wahls, 1990). Design of abutment structures is not unique and varies as per the connection of the slab with the abutment. The abutments are characterized as mainly integral (movable) or non-integral (conventional or stub) type of abutments (Greimann et al., 1987). For an integral abutment, the bridge deck slab is monolithically connected to the abutment, and the abutment is allowed to move laterally along with the bridge deck slab; while for a non-integral one, the bridge deck is independent of the abutment, and the longitudinal movements of the bridge deck are taken care of by roller/pin-bearing plates.

Weather changes also contribute to the differential settlement between the bridge and the approach slab as in the case of integral abutments when seasonal temperature changes from summer to winter (Schaefer and Koch, 1992). Weather changes often lead to soil displacement behind the abutment eventually leading to void development under the approach slab (Schaefer and Koch, 1992; White et al., 2005). This creates water infiltration under the slab, which leads to erosion and loss of backfill material (Jayawickrama et al., 2005).

White et al. (2007) carried a comprehensive field study of 74 bridges in Iowa to characterize problems leading to poor performance of bridge approach pavement systems. White et al. (2007) claimed that subsurface void development caused by water infiltration through unsealed expansion joints, collapse and erosion of the granular backfill, and poor construction practices were found to be the main contributing factors of the approach slab settlements in Iowa.

Other research studies from outside the U.S.A., including Australia and China show that the bump at the end of the bridge is a major concern in highway and freeway constructions. Hsi and Martin (2005) and Hsi (2007) reported that the approach settlement problems were observed due to very soft estuarine and marine clays in subsoils at the construction of the Yelgun-Chinderah Freeway in New South Wales, Australia. Hsi (2007) reported that rapid construction of deep approach embankments over very soft clay subgrades often experienced the long term settlement of the soft subgrade, which has attributed to causing settlements at the approach slabs.

In the following, three studies by Briaud et al. (1997), Seo (2003), and White et al. (2007) listed factors that contribute to bumps. Briaud et al. (1997) summarized various factors that contributed to the formation of bumps/settlements at the approach slabs in Figure 1. These factors were grouped and ranked in the following order in which they contribute to the soil movements: fill on compressible foundation; approach slab too short; poor fill material; compressible fill; high deep embankment; poor drainage; soil erosion; and poor joint design and maintenance.

Seo (2003) performed a circular track test involving the approach slab, which was repeatedly loaded by a vehicle model**.** Seo (2003) listed the following observations:

1. Number of cycles of loading over the approach slab is proportional to the increase in the bump;
2. Shorter approach slabs result in higher displacements of the slab;
3. More highly compacted stiffer soils result in less deflection of the slab;

4. The velocity of vehicles has an influence on the increase in magnitude of the bump; and
5. The weight of vehicles relates to the degree of the settlement.

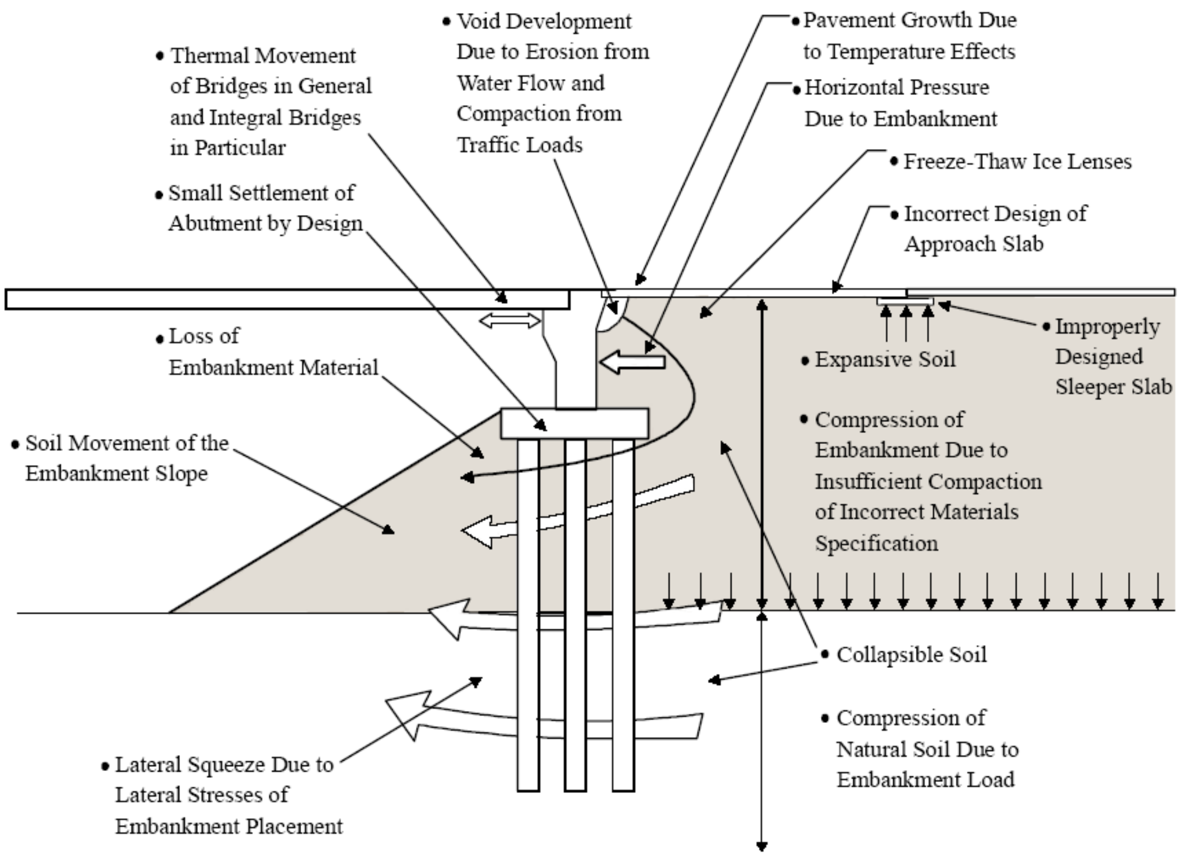


Figure 1 – Schematic of Different Origins Lead to Formation of Bump at the End of the Bridge (Briaud et al., 1997)

A recent study conducted by White et al. (2007) summarized the following factors as contributors to differential settlements of the approach slab:

1. Backfill materials under poorly performing approach slabs are often loose and under compacted.
2. The foundation soil or embankment fill settles.
3. Many bridge approach elevation profiles have slopes higher than 1/200, which is considered a maximum acceptable gradient for bridge approaches.

4. Voids develop under bridge approaches within one year of construction, indicating insufficiently compacted and erodible backfill material.
5. Inadequate drainage is a major bridge approach problem. Many abutment subdrains are dry with no evidence of water, are blocked with soil and debris, or have collapsed.
6. Many expansion joints are not sufficiently filled, allowing water to flow into the underlying fill materials.

This report presents the following major factors that caused approach bumps by summarizing the above studies as well as a review of other investigations that addressed this bump problem:

1. Consolidation settlement of foundation soil;
2. Poor compaction and consolidation of backfill material;
3. Poor drainage and soil erosion;
4. Types of bridge abutments;
5. Traffic volume;
6. Age of the approach slab;
7. Approach slab design;
8. Skewness of the bridge; and
9. Seasonal temperature variations.

Salient details of these factors are presented in the following subsections.

2.1 Consolidation Settlement of Foundation Soil

Consolidation of foundation soil under an approach embankment is regarded as one of the most important contributing factors to bridge approach settlements (Hopkins, 1969; Wahls, 1990; Dupont and Allen, 2002). It usually occurs because of dynamic traffic loads applied at the embankment surface and static load due to the embankment weight itself (Dupont and Allen, 2002). However, this foundation settlement problem is difficult to address and repair them in-situ, because of the variability in the engineering properties of soils and the complexity of accessing the foundation after construction as it is buried deep below the roadway surface (Wahls, 1990).

Foundation problems usually are more severe in cohesive soils than in non-cohesive soils. Since consolidation occurs rapidly in non-cohesive soils, they do not normally represent a serious problem. On the other hand, cohesive soils, such as soft or high plasticity clays, represent a more critical situation, because of their time dependent consolidation behavior. In addition, cohesive soils are more susceptible to lateral or permanent plastic deformation, which can exacerbate the approach settlement problem.

Typically, settlement of soils can be divided in three different phases (Hopkins, 1969); initial, primary, and secondary consolidation, which are explained in the following.

Initial Consolidation

The initial settlement is the short-term deformation of the foundation when a load is applied to a soil mass. This type of settlement does not contribute to the formation of bumps, because it usually occurs before the construction of the approach structure (Hopkins, 1969). The soil saturation level affects the total contribution of this settlement, and for partially saturated soils, this initial settlement will be generally larger than that of saturated soils.

Primary Consolidation

Primary settlement is the main factor that contributes to the total settlement of soils. The gradual escape of water due to the compression of the loaded soil is believed to be the reason for this type of settlement. This primary settlement lasts from a few months for granular soils, to a period of up to ten years for some types of clay (Hopkins, 1973). This significant difference is attributed to the small void ratio and high permeability of granular soils.

Secondary Consolidation

This phase occurs as a result of changes in void ratio of the loaded soil after dissipation of excess pore pressure (Hopkins, 1969). In this case particles and water in the soil mass readjust in a plastic way under a constant applied stress. For the case of very soft, highly plastic or organic clays, secondary consolidation can be as large as the primary consolidation, while in granular soils, it is negligible (Hopkins, 1969).

To mitigate or minimize the settlement, a primary objective of any bridge construction project should include a complete or comprehensive investigation of the foundation soil before the construction of the approach embankment starts (Wahls, 1990). Previous studies have shown that the stresses applied to the foundation subgrades come primarily from the embankment loading rather than the bridge or traffic loads, except for shallow depths (less than 10 ft) (Hopkins, 1969; Wahls, 1990; Dupont and Allen, 2002). Therefore, geotechnical studies have to be carried out with extensive foundation investigations, including laboratory tests to evaluate compression and consolidation potential to better estimate the anticipated post-construction settlements (Dupont and Allen, 2002). It is also important to study the possible shear failures in the foundation that cause lateral deformations and surface settlement problems. This type of failure is more likely to appear in peat and other organic materials.

2.2 Poor Compaction and Consolidation of Backfill Material

To minimize construction costs, approach embankments are usually constructed with the most readily available material at or near the site. But when low quality materials (such as locally available soft, cohesive expansive soils and soils sensitive to freeze-thaw) are used, the approach settlements can be induced in terms of bigger "bumps." In general, cohesive soils are more difficult to compact to their optimum moisture content and density when compared to coarser or granular fill materials (Hopkins, 1973).

Poor compaction control of the embankment material is found to be a factor, resulting in low density and highly deformable embankment mass (Lenke, 2006). Poor compaction can also be attributed to limited access or difficulty in access within the confined working space behind the bridge abutment (Wahls, 1990). Many highway agencies require only granular fills that can be better compacted and are able to reach their maximum consolidation in less time than more cohesive soils (Wahls, 1990; Lenke, 2006). The TxDOT Bridge Design Manual (2001) notes that either improper backfill materials used for mechanically stabilized earth (MSE) or the inadequate compaction of the backfill materials in the embankment are the contributing factors to the backfill failure.

Compaction type and project schedule are also of great importance (Dupont and Allen, 2002). Field inspectors should ensure that proper compactive effort and compaction levels of the fill material are reached during construction. It is common practice that bridge abutments are constructed before the embankment fill placement and compaction. This practice makes the compaction of the area closest to the bridge more difficult because the equipment access to this critical area becomes limited (Burke, 1987).

In addition to compression of the backfill material, lateral stability and shear strength are of great importance to the overall stability against the approach settlement. For the case of the foundation soil, lateral confining forces are significant, while on embankment fills, the confinement effects are much less pronounced (Wahls, 1990). Hence, slope design, material selection, and loads applied to the backfill need to be carefully evaluated to anticipate or minimize the final settlement (Wahls, 1990).

2.3 Poor Drainage and Soil Erosion

Several researchers from different state DOTs including Texas DOT, Virginia DOT, Iowa DOT, and Colorado DOT reported the importance of the surface and subsurface drainage and soil erosion near the bridge abutment and embankment interface. Wahls (1990), Jayawickrama et al. (2005), Mekkawy et al. (2005), White et al. (2005), and Abu-Hejleh et al. (2006) identified the drainage system of the abutment and embankment as one of the most important factors that affect approach settlement. The dysfunctional, damaged, or blocked drainage systems cause erosion in the abutment and slope, increasing soil erosion and void development. The dysfunctional drainage systems may be caused by either incorrect construction or improper design. Williammee (2008) observed that incorrect placement of the drainage pipes such as outlet flow line higher than inlet flow line in a newly constructed bridge can impair the drainage system. Briaud et al. (1997) explains how the poor joints between the pavement and the abutment structure as seen in Figure 2 can lead to soil erosion.

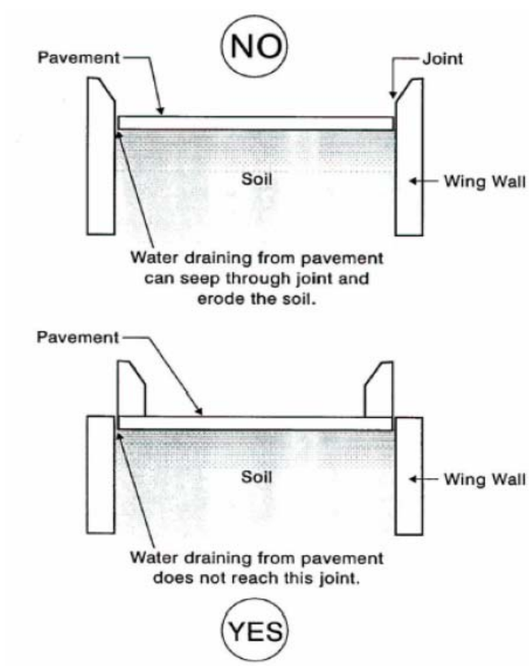


Figure 2 – Cross Section of a Wingwall and Drainage System (Briaud et al., 1997)

Jayawickrama et al. (2005) noted that the erosion of soil at the abutment face and poor drainage of embankment and abutment backfill material can induce serious approach settlement problems. This observation was based on the survey responses obtained from various TxDOT District officials. The intrusion of surface water (rain) through weak expansion joints (openings) between the approach slab and bridge abutments can erode backfill material and further amplify the problem of approach slab settlements (Jayawickrama et al., 2005). Based on the detailed study of a few TxDOT bridges, they noted that these joint openings resulted from the poor construction practices such as poor compaction of backfill material near the abutments, poor construction of joint sealants, and poor surface and subsurface drainage systems.

In addition, the expansion joints should transfer traffic loads, prevent surface water from entering into the abutment, and allow pavement expansion without damaging the abutment structure (Wolde-Tinsae et al., 1987). Based on a comprehensive research study performed by White et al. (2005) on many bridges in Iowa, most of the expansion joints of the bridges inspected were not sufficiently filled, allowing water to flow into the underlying fill materials. On the other hand, cracks were often encountered next to closed joints in bridge approaches because of the crushing and cracking of neighboring concrete, allowing for leakage of water as well.

Similar observations were made by Mekkawy et al. (2005), which are discussed here. Based on field investigations in different states, Mekkawy et al. (2005) reported that inadequate drainage and subsequent severe soil erosion contributed to settlement problems of 40 percent of the bridge approach slabs that were surveyed by them. Moisture flow into the backfill coupled with poor drainage conditions can cause failure of embankment, backfill, and bridge abutments either by excessive settlement or by soil strength failure. Typically, water can seep into the embankment fill material via faulty joints and cracked concrete pavement sections. The leaked water can soften the embankment fill and can cause internal erosion as the fines typically wash out from the fill material. Without approach slabs, water leakage will immediately induce settlement; with approach slabs, voids beneath the slab will form, amplifying the erosion by compression of the soil.

The erodability of soils is based on their grain size distribution. Some soil gradation guidelines can be found for soils that are erosion resistant and those that are prone to erosion (Briaud et al., 1997; Hoppe, 1999). As indicated in Figure 3, a gradation band of material in the sand to silt size materials is a bad choice for embankments and backfill unless additional preventive actions, such as providing appropriate drainage design or erosion control systems, are taken (Briaud et al., 1997).

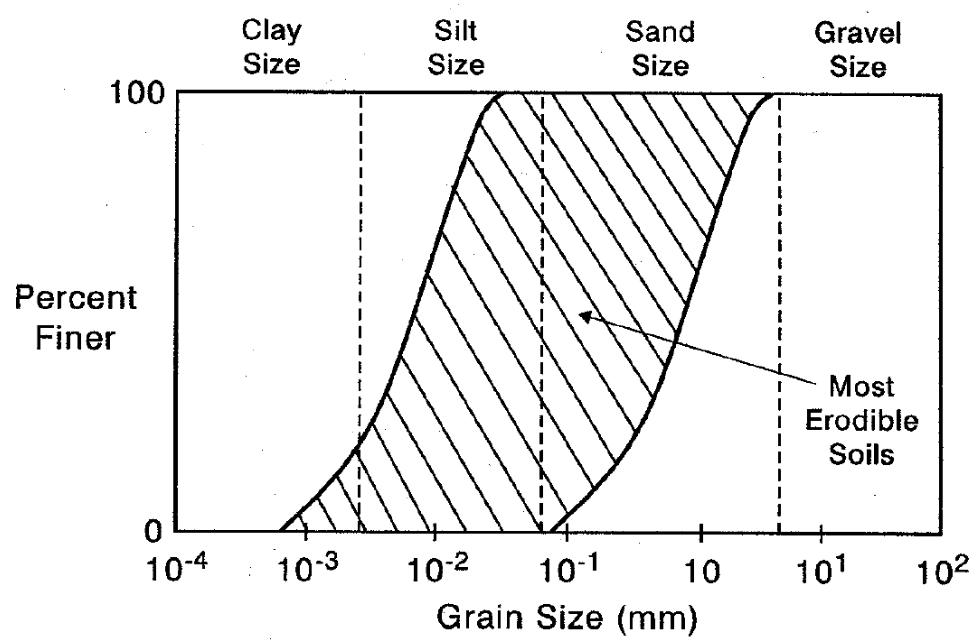


Figure 3 – Range of Most Erodible Soils (Briaud et al., 1997)

2.4 Types of Bridge Abutments

Abutments must be compatible with the bridge approach roadway and they must have backwalls to keep the embankment from covering up the beam ends and to support possible approach slabs (Figure 4). They also usually have wingwalls to keep the side slopes away from the structure and to transition between the guard rail and the bridge rail as shown in Figures 4 and 5.

Abutments are characterized as integral (movable) or non-integral (conventional or stub) types (Greimann et al., 1987). In the integral type, the bridge deck slab is monolithically connected to the abutment, and the abutment is allowed to move laterally along with the bridge deck slab; while in the non-integral type, the bridge deck is independent of the abutment, and the longitudinal movements of the bridge deck are taken care of by roller/pin-bearing plates (Greimann et al., 1987). The advantages of integral bridge abutments are reduced construction and maintenance costs, minimum number of piles required to support the foundation, and enhanced seismic stability (Greimman et al., 1987; Hoppe and Gomez, 1996). To avoid the use of the bearing plates and to reduce potential maintenance problems (such as frequent repair of bearing plates, expansion joint sealants) associated with non-integral bridge abutments, the use of integral bridge abutments has been increased since the 1960s (Horvath, 2000; Kunin and Alampalli, 2000). The following sections describe the advantages and disadvantages of both types of abutments.

Integral Abutments

Figure 4 shows a simplified cross section of an integral abutment bridge. The approach slab system of an integral bridge consists of the backfill, the approach fill, and the soil foundation. If an approach slab and a sleeper slab are used, they are also considered in the system. Integral abutment bridges are designed to carry the primary loads (dead and live loads) and also the secondary loads coming from creep, shrinkage, thermal gradients, and differential settlements. Integral abutments are rigidly connected to the bridge beams and deck with no expansion joint.

Even though integral abutments present structural advantages over non-integral abutments, they also introduce thermal movements in the approach system that can aggravate the bump problem on the approach system (Schaefer and Koch, 1992; White et al., 2005). Hence, special attention has to be paid in this type of abutment to the lateral loads imposed on the foundation piles due to horizontal movements induced by temperature cycles (Wahls, 1990).

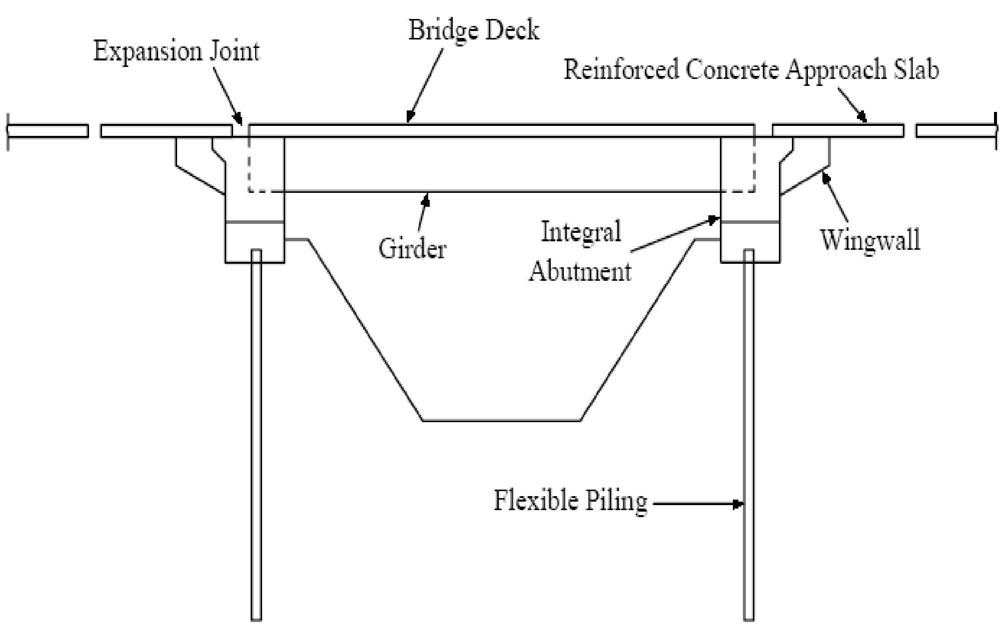


Figure 4 – A Simplified Cross Section of an Integral Abutment Bridge

(Greimann et al., 1987)

Non-Integral Abutments

A simplified cross-section of a non-integral abutment is shown in Figure 5. In this case, abutments are supported on bearing connections that allow longitudinal movements of the superstructure without transferring lateral loads to the abutment. The non-integral bridge abutment is separated from the bridge beams and deck by a mechanical joint that allows for the thermal expansion and contraction of the bridge (Nassif, 2002).

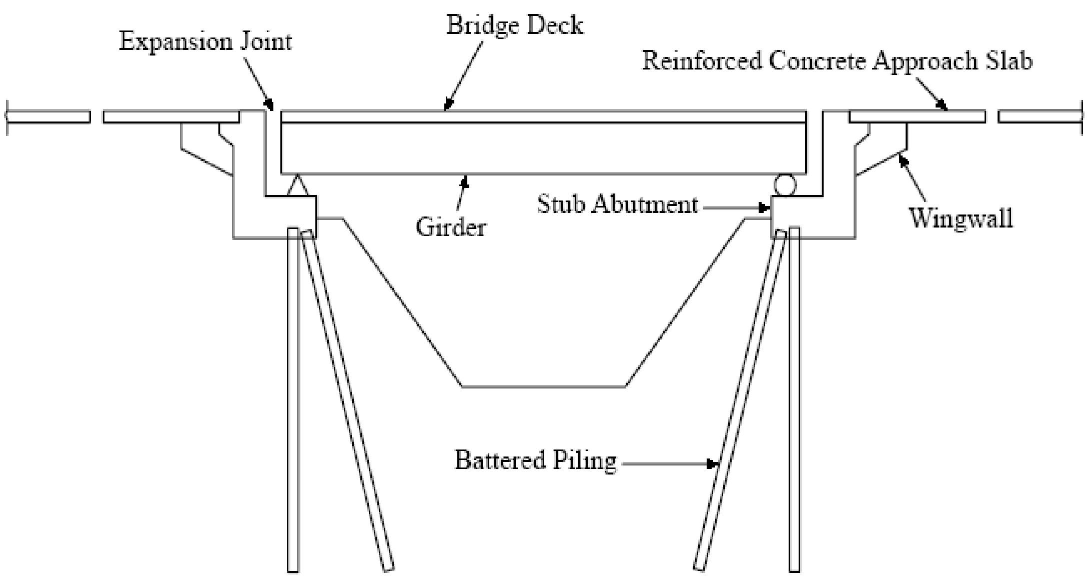


Figure 5 – A Simplified Cross Section of a Non-Integral Abutment Bridge (Greimann et al., 1987)

Three major types of non-integral abutment bridges can be found in the literature. These are: Closed or U-type, Spill-through or Cantilever and Stub or Shelf abutments (Hopkins and Deen, 1970; Timmerman 1976; Wahls 1990; TxDOT Bridge Design Manual, 2001).

Closed Abutment or U-Type

A simplified cross-section of a closed abutment is shown in Figure 6a. The U-type abutments have two side walls and a front wall resting on spread footings below natural ground (TxDOT Bridge Design Manual, 2001). For this type of abutment, the side walls are long enough to keep the embankment from encroaching on the bridge opening. In addition, the taller the abutment is, the longer the sidewalls will be. The compaction of the embankment fill is rather difficult in these abutments, because of confined space near the abutment and due to the wall, which is extended over the whole height of the abutment (TxDOT Bridge Design Manual, 2001). These abutments are also subjected to higher lateral earth pressures than other types.

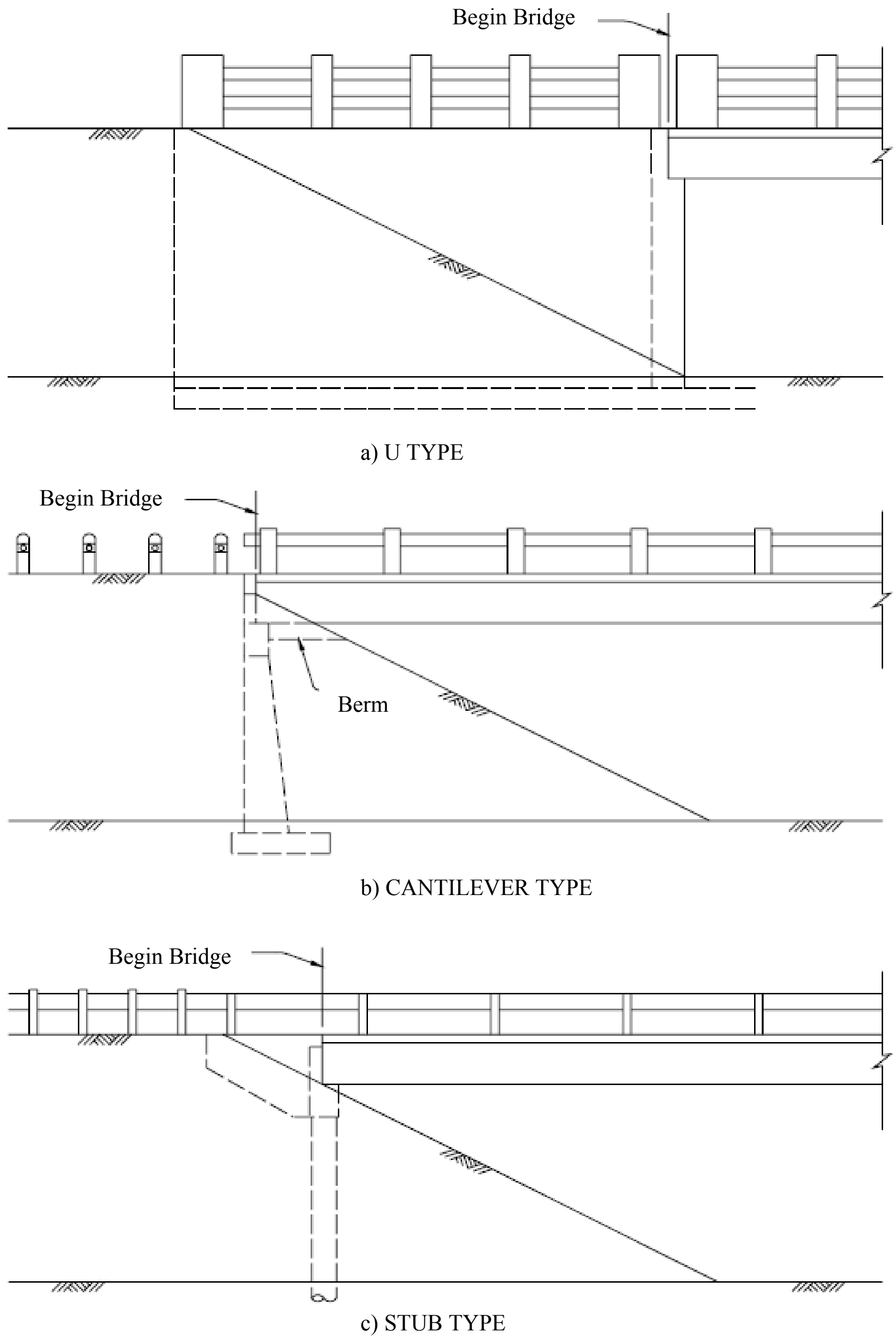


Figure 6 – Non-Integral Abutment Types (TxDOT Bridge Design Manual, 2001)

Spill-Through or Cantilever Abutment

A simplified cross-section of a spill-through abutment is shown in Figure 6b. A spill-through abutment is supported on the columns and hence, the compaction of the backfill material between the columns and near the abutment is very difficult. Cantilever type abutments have variable width rectangular columns supported on spread footings below natural ground (TxDOT Bridge Design Manual, 2001). The fill is built around the columns and allowed to spill through, on a reasonable slope, into the bridge openings. A great number of these types of abutments have been constructed in Texas and they have performed well in the past (TxDOT Bridge Design Manual, 2001). However, this type of abutment presents detailing and construction problems, as well as high construction costs.

Stub or Shelf Abutment

A simplified cross-section of a stub type abutment is shown in Figure 6c. A stub abutment is constructed after the embankment, so its height is directly affected by the embankment height. The compaction of the backfill material is relatively easier, compared with the closed type except for the soil behind the abutment (TxDOT Bridge Design Manual, 2001).

Most of the abutments in Texas were of the "stub" or "shelf" type, constructed by driving piling or drilling shafts through the compacted fill and placing a cap backwall and wingwalls on top. The header bank is sloped from the top of the wingwall through the intersection of the cap and backwall into the bridge opening. The bridge must be considerably longer than with U-type abutments but slightly shorter than the cantilever types. The extra length of this abutment is justified on the basis of cost and aesthetics (TxDOT Bridge Design Manual, 2001).

Although more economical, stub abutments have maintenance problems. The "bump at the beginning of the bridge," caused by fill settlement is particularly noticed on stub type abutments (TxDOT Bridge Design Manual, 2001).

From the past experiences with these non-integral abutment bridges, TxDOT officials attribute the approach settlements to the poor construction practices due to inaccessibility to compact the backfill/embankment fill near the vicinity of the abutment leading to the aggressive approach settlements (Jayawickrama et al., 2005).

2.5 Traffic Volume

Heavy truck traffic has been found in some studies to be a major factor contributing to the severity of this bump along with the age of the bridge and approach, especially for the late 70s or early 80s (Wong and Small, 1994; Lenke, 2006). High-volume traffic has been found as a compelling reason for including approach slabs in the construction of both conventional and integral bridges. Lenke (2006) noted that "the bump" was found to increase with vehicle velocity, vehicle weight, especially heavy truck traffic, and number of cycles of repetitive loading, in terms of Average Daily Traffic (ADT). On the other hand, Bakeer et al. (2005) have concluded that factors such as speed limit and traffic count have no distinguishable impact on the performance of the approach slabs.

2.6 Age of the Approach Slab

The age of the approach slab is an important factor in the performance of different elements of bridge structures, especially at the expansion joints next to the approach slab, which could negatively affect the backfill performance in terms of controlling settlements underneath the slab (Laguros et al., 1990; Bakeer et al., 2005). Another factor known as alkali-silica reactivity (ASR) formed under the concrete approach slabs and is known to induce expansion stresses. These stresses can potentially lead to slab expansion and distress in the approach slabs, approach joints, and vertical uplift of the slabs and pavement preceding the slabs (Lenke, 2006).

Bakeer et al. (2005) studied the influence of approach age by investigating a number of approach slabs built in the 1960s, 1970s, 1980s, and 1990s. Based on the condition ratings, the newer pile- and soil-supported approach slabs were generally in better condition than the older ones. The IRI ratings showed that pile-supported approach slabs built in the 1980s performed better than those built in the 1990s and that the approach slabs built in the 1990s performed better than those built in the 1970s.

Laguros et al. (1990) reported that the flexibility of the approach pavements has a considerable influence as well. They observed greater differential settlement in flexible pavements than rigid pavements during initial stages following construction (short term performance), while both pavement types performed similarly over the long term.

2.7 Approach Slab Design

The purpose of the approach slab is to minimize effects of differential settlement between the bridge abutment and the embankment fill, to provide a smooth transition between the pavement and the bridge, to prevent voids that might occur under the slab and to provide a better seal against water percolation and erosion of the backfill material (Burke, 1987). However, a rough transition can occasionally develop with time in bridge approaches due to differential settlements between the abutment and roadway. This can be attributed to the different support systems of the two structures connected by the approach slab. The approach slab and the roadway are typically constructed over an earth embankment or natural soil subgrade, whereas the bridge abutment is usually supported on piles.

Insufficient length of approach slabs can create differential settlements at the bridge end due to high traffic induced excessive destruction in the approach slab (Briaud et al., 1997). Based on an extensive survey performed by Hoppe (1999) in 39 states, approach slabs lengths varied from 10 to 40 feet and thicknesses ranged from 8 to 17 inches. Some studies based on the IRI ratings, report that 80 foot-long slabs performed the best, and no significant difference was found when compared to 100 foot-long slabs (Bakeer et al., 2005).

The rigidity of the approach slab is also a major contributing factor. Dunn et al. (1983) compared the performance of various approach slab pavements in Wisconsin and reported that 76 percent of the flexible approaches rated poor, 56 percent of the non-reinforced approaches rated fair, and 93 percent of the reinforced concrete approaches rated good. All these ratings are based on the performance of the approach slab in controlling the differential settlements.

2.8 Skewness of the Bridge

Skew angle also has a significant effect on the formation of approach settlements and the overall bridge performance. Skewed integral bridges tend to rotate under the influence of cyclic changes in earth pressures on the abutment (Hoppe and Gomez, 1996). According to Abendroth et al. (2007) design of skewed integral abutment bridges must account for the transverse horizontal earth pressure applied along the skew. Also, the change in position of the ends of an abutment can be attributed to a combination of two effects: the temperature-dependant volumetric expansion or contraction of concrete in the pile cap and abutment, and the rigid-body translation and rotation of the abutment due to the longitudinal expansion or contraction of the superstructure for a skewed integral abutment bridge. This study also recommended that when skewed integral abutments are used, they should be placed parallel to each other and ideally be of equal height (Abendroth et al., 2007).

Nassif (2002) conducted a finite element study to understand the influence of skewness of bridge approaches and transition slabs on their behavior. It was found that the skew angle of the approach slab resulted in an uneven distribution of the axial load, so that only one side of the axles actually had contact with the approach slab. Figure 7 shows that for the same loading conditions, the tensile axial stresses on skewed approach slabs are found to be 20 to 40 percent higher than the same on straight approach slabs. In addition, the pinned connection at the edge of the approach slabs that connects them with the bridge abutment prevented any displacement taking place along this edge, thus providing more strength to the elements of this region (Nassif, 2002).

Additionally, higher rates of settlements at the bridge exit were considered to be accountable to the effect of the skew angle of the approach slab as well as improper compaction conditions in hard-to-reach soil areas close to the abutments (Nassif, 2002).

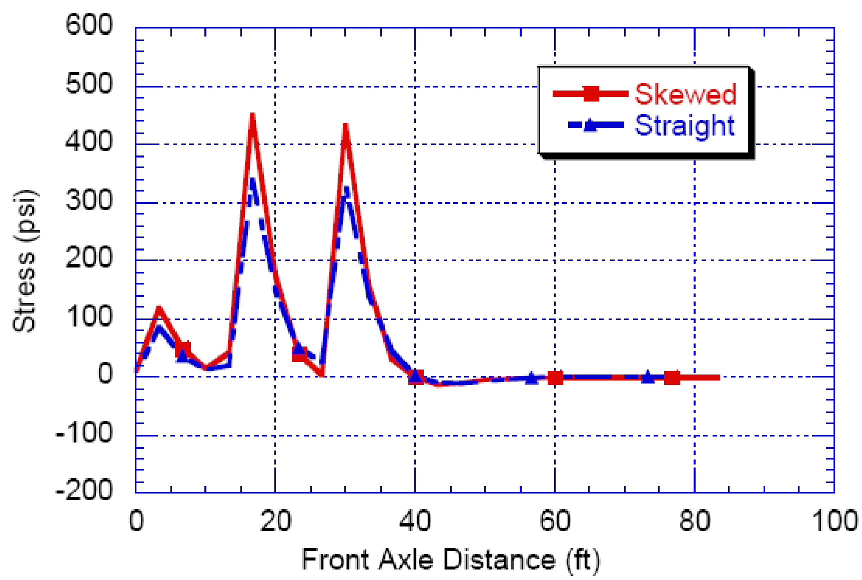


Figure 7 – Variation of Tensile Axial Stress with Front Axle Distance for Skewed and Straight Approach Slabs (Nassif, 2002)

2.9 Seasonal Temperature Variations

Some of the factors that contribute to differential settlement between bridge and approach slab, especially for integral abutments, are seasonal temperature changes between summer and winter in the bridge deck (Schaefer and Koch, 1992; Arsoy et al., 1999; Horvath, 2005; White et al., 2005).

This temperature change causes cyclical horizontal displacements on the abutment backfill soil, which can create soil displacement behind the abutment, leading to void development under the approach slab (White et al., 2005). As a result, the infiltration of water under the slab and therefore erosion and loss of backfill material may accelerate.

Due to seasonal temperature changes, abutments move inward or outward with respect to the soil that they retain. During winter, the abutments move away (outward) from the retained earth due to contraction of the bridge structure while in summer they move towards (inward) the retained soil due to thermal expansion of the bridge structure (Arsoy et al., 1999; Horvath, 2005) At the end of each thermal cycle, abutments have a net displacement inward and outward from the soil that is usually retained (see Figure 8). This is attributed to the displacement of an 'active soil wedge,' which moves downward and towards the abutment during winter but cannot fully recover due to inelastic behavior of the soil during the summer abutment movement. This phenomenon was noted in all types of embankment materials (Horvath, 2005). Besides, these horizontal displacements are observed to be greater at the top of the abutment, and hence the problem is aggravated when the superstructure is mainly constructed with concrete (Horvath, 2005). Figure 9 shows how the expansion-contraction movements of the bridge with the seasonal temperature change will lead to the creation of voids below the approach slab.

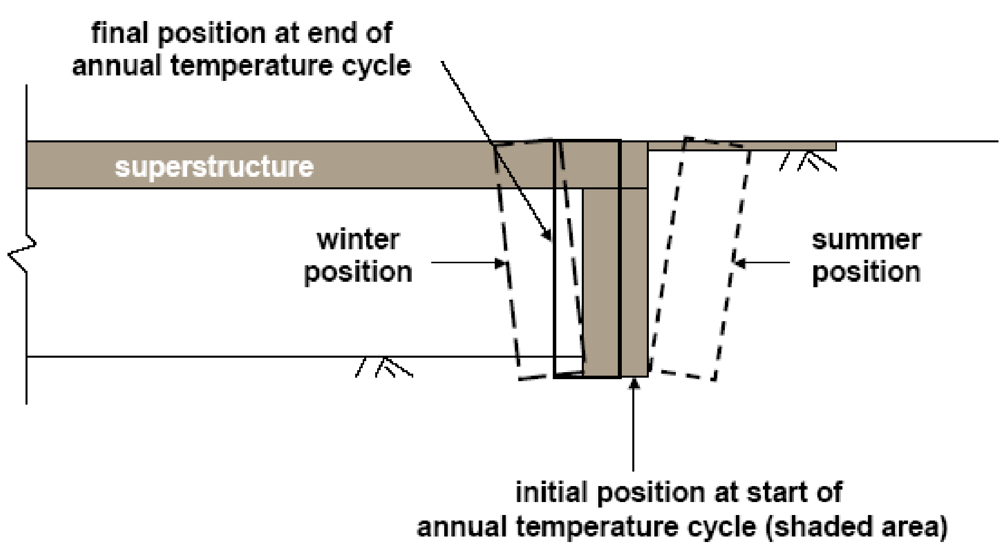


Figure 8 – Thermally Induced IAB Abutment Displacement (Horvath, 2005)

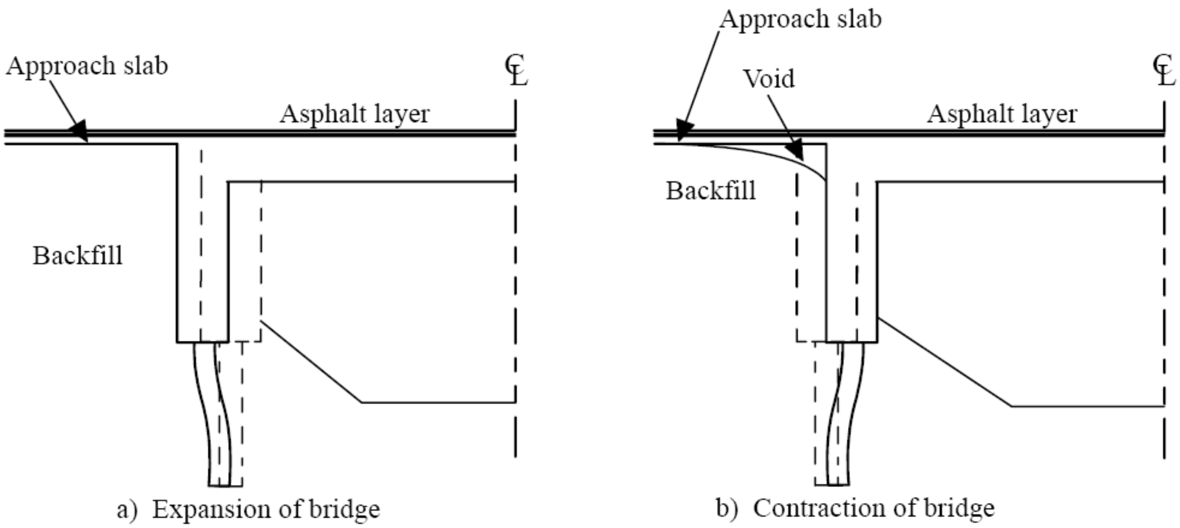


Figure 9 – Movement of Bridge Structure (Arsoy et al., 1999)

The temperature effect on the bridge-abutment interaction also creates pavement growth due to friction between the pavement and its subbase (Burke, 1993). After the pavement expands, it does not contract to its original length because of this friction. This residual expansion accumulates after repeated temperature cycles, resulting in pavement growth that can be rapid and incremental at pressure relief joints (Burke, 1993). The pressure generated will transmit to the bridge in terms of longitudinal compressive force and therefore, should be considered by engineers when designing the pressure relief joints.

James et al. (1991) documented a case of severe abutment damage for a bridge without pressure relief joints through a numerical study. This numerical stress analysis indicated that the damage was caused by the longitudinal growth of continuous reinforced concrete pavement, causing excessive longitudinal pressures on the abutments.

The cycle of climatic change, especially the temperature change, also can cause certain irreversible damage to the pavement or bridge approach slabs in terms of ice lenses due to frost action. Here ice lenses are derived from freezing and thawing of moisture in a material (in this case soil) and the structure that are in contact with each other (UFC, 2004). The existence of freezing temperature and presence of water on the pavement either from precipitation or from other sources such as ground water movement in liquid or vapor forms under the slabs can cause frost heave in pavements. This phenomenon causes the pavement rising because of ice crystal formation in frost-susceptible subgrade or subbase that can affect the durability of concrete. The frost induced heave is not a serious problem in pavements in dry weather areas like Texas.

As noted by the above sections, bump or differential settlements are induced by several factors either by individual mechanisms or by combination mechanisms. In the following sections, different treatment or repair techniques adapted for new and existing bridges are detailed.

3. MITIGATION TECHNIQUES FOR APPROACH SETTLEMENTS OF NEW BRIDGES

This section is a summary of various methods adopted for mitigating potential settlements expected in new bridges. These techniques are listed based on various groups of treatments such as improvement of foundation soil, improvement of backfill material, design of bridge foundation, design of approach slab, and effective drainage and erosion control methods.

3.1 Improvement of Embankment Foundation Soil

The behavior of foundation soil beneath the embankment and embankment fill is one of the important factors in the better performance of bridges (Wahls, 1990). Generally, if the foundation soil is a granular material type, such as sand, gravel and rock, which do not undergo long term settlements, then the differential settlement of the bridge structure can be negligible. On the other hand, if the approach embankments are constructed on cohesive soils such as clays, then those soils can undergo large settlements either from primary and/or secondary consolidation settlements. These settlements will subsequently lead to the settlements of embankment structures and thereby formation of the bumps or approach settlement problems leading to poor performance of bridge approaches. Several attempts have been made by many researchers both from the U.S.A. and abroad to mitigate these unequal settlements arising from highly compressible embankment fills (Wahls, 1990; Dupont and Allen, 2002; White et al., 2005; Abu-Hejleh et al., 2006; Hsi, 2008).

When the soil/fill underneath the structure is not suitable for construction, the recommended approach is to enhance the properties of the foundation soil such that they undergo less compression due to loading (White et al., 2005). Successful ground improvement methods include preloading the foundation soil (Dupont and Allen, 2002) excavation and replacement of existing soft soil, reinforcement of soil to reduce time-dependent post construction settlements and also lateral squeeze (White et al., 2005). Lightweight embankment materials are also effectively used as embankment fills in order to reduce the embankment loads applied on the foundation soils (Saride et al., 2008).

The selection of ground improvement technique for a particular project is mostly based on the type of soil and partly on the depth of the loose layer, degree of saturation, ground water table location, and permeability. If the soil is granular material, then the ground improvement techniques such as surcharge (or) preloading, dynamic compaction, compaction piles, grouting, and gravel columns are preferred (Wahls, 1990; Abu-Hejleh et al., 2006). If the soil is cohesive in nature, excavation and re-compaction, preloading, installation of wick drains, dynamic compaction, stone columns, lime treatment columns, and grouting are proposed (Wahls, 1990; Abu-Hejleh et al., 2006).

Based on the review of literature, the stabilization techniques to improve the embankment foundation soil are grouped as per the soil type. Table 1 summarizes these ground improvement techniques adopted, not limited to one, for each foundation soil in a chronological order of their importance and the level of settlement problem.

Table 1 – Summary of Ground Improvement Methods Based on Soil Type

Technique	Cohesionless soils	Cohesive soils
Excavation and Replacement	✕	✓
Preloading w or w/o Surcharge	✓	✓
Dynamic Compaction	✓	✓
Grouting	✓	✓
Wick Drains	✕	✓
Compaction Piles	✓	✕
Gravel Columns	✓	✕
Lime Treatment	✕	✓
Stone Columns	✕	✓
Soil Reinforcement	✓	✓
Geopier	✓	✓

Most of the techniques in combination are chosen for a particular field situation. For example, preloading with the installation of wick drains will lead to faster consolidation settlement of weak soft foundation soil. These techniques are again divided into three sub categories such as mechanical, hydraulic, and reinforcement techniques based on the function of each stabilization technique (Table 2).

Table 2 – Summary of Ground Improvement Techniques Based on the Function

Embankment Soft Foundation Soil Improvement Techniques		
Mechanical	**Hydraulic**	**Reinforcement**
Excavation and replacement Preloading and surcharge Dynamic compaction	Sand drains Prefabricated drains Surcharge loading	***Columns*** Stone and Lime Columns Geopiers Concrete Injected Columns Deep Soil Mixing Columns
		Deep foundations In-situ: Compacted piles CFA piles Driven piles: Timber and Concrete piles
		Geosynthetics Geotextiles/Geogrids Geocells

The following sections describe each ground improvement technique and available literature information with respect to approach settlement problems.

3.2 Mechanical Modification Techniques

Excavation and Replacement

In this method, the undesirable top soil is excavated and replaced with a select fill from borrow sites. The removal and replacement concept is one of the options considered when the proposed foundation soils are prone to excessive consolidation (Luna et al., 2004; White et al., 2005; Wahls, 1990; Hoppe, 1999; Chini et al., 1992). Dupont and Allen (2002) reported that around 32 states in the U.S. replace the foundation soil near the bridge approach when they have low bearing stresses. The excavation can be done in the range of 10 ft (3 m) to 30 ft (10 m) from the top soil surface. The selected fill material from the borrow pit must be controlled carefully to avoid pocket entrapments during the compaction process.

Presently the difficulties involved in this excavation and replacement method are due to the difficulty in maintaining uniform replacement and expenses involved in the complete removal and land-filling of undesirable soil. Because of these reasons, this method becomes less favorable. Tadros and Benak (1989) discussed this technique in detail and reported that the excavation and replacement technique may be the most economical solution, only if the compaction areas are underlain by a shallow bedrock or firm ground.

Preloading/Precompression

One of the effective methods reported in the literature to control foundation settlement is to pre-compress the foundation soil (Dupont and Allen, 2002). According to Bowles (1988), pre-compression is a relatively inexpensive and effective method to improve poor foundation soils. Bowles (1988) noted that this technique is used to accomplish two major goals; one is to eliminate settlements that would otherwise occur after the structure is built and the second is to improve the shear strength of the subsoil by increasing the density, reducing the void ratio, and decreasing the water content.

The pre-compression technique in embankment construction is a process in which the weight of embankment will be considered as a load inducing the consolidation settlement and completing the process before the beginning of actual pavement or roadway construction. In this method, the construction is delayed, even up to one year in most of the cases, so as to allow embankment settlement prior to roadway construction before the placement of approach pavement (Cotton et al., 1987). Even though this method could be effective in reducing foundation settlement and maintenance costs, many highway agencies do not implement this technique due to lengthy construction periods that could cause significant problems in construction schedules and increase in total project costs (Hsi, 2007). Hence, this technique is often combined with other ground improvement methods such as vertical drains and surcharge loading which will enhance the properties of subsoils from mechanical and hydraulic modifications, resulting in faster enhancements. Design of vertical drains deal with the hydraulic properties of the soil, and hence these details are covered in modifications by hydraulic methods.

Surcharge Loads

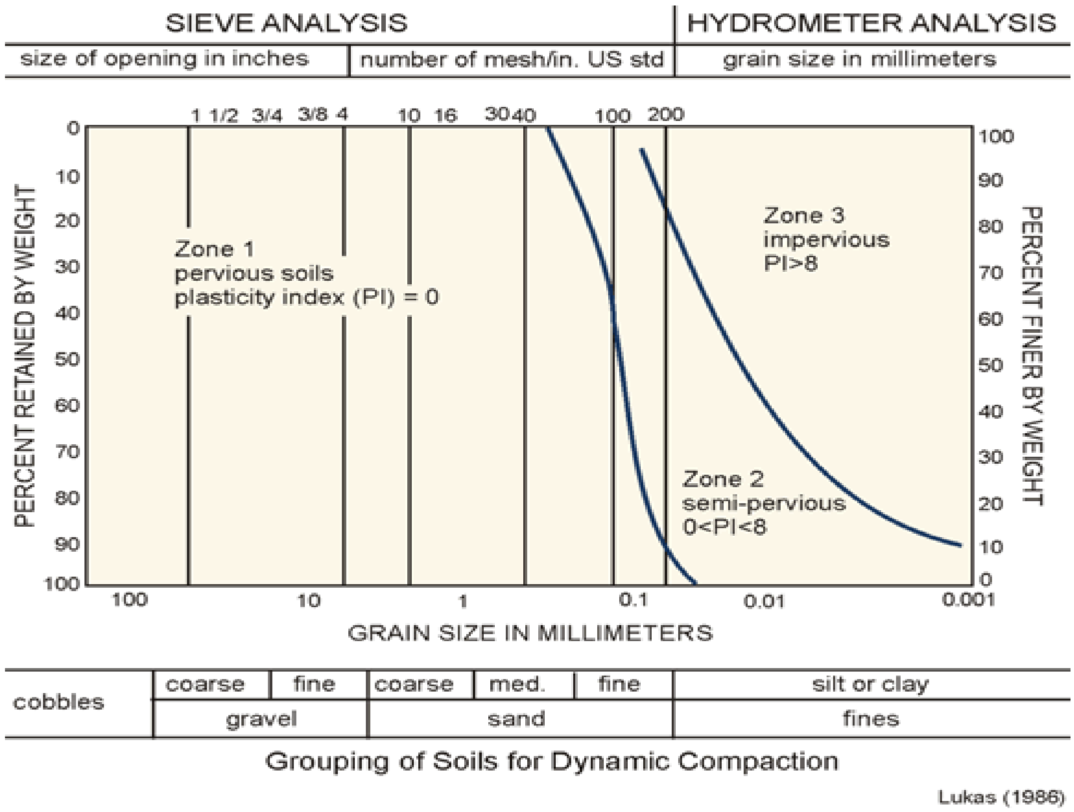
A temporary surcharge load might also be applied on top of the embankment to accelerate the consolidation process (Bowles, 1988; Hsi, 2007). In order to achieve this, the applied surcharge load must be greater than the normal load, i.e., the weight of the embankment in this particular case. However, the desired extra load, in terms of extra height of embankment, has to be limited by its slope stability. In order to eliminate this limitation, sometimes a berm is constructed for this purpose. The costs of berm construction, excessive fill placement, and its removal will result in an increased overall project cost and duration. These costs have to be weighed against the costs involved in avoiding construction delays (Bowels, 1988).

Dynamic Compaction

The dynamic compaction is another alternative to improve the foundation soil. This technique is best suitable for loose granular deposits than medium to soft clays. Heavy tamping and dynamic consolidation are also called dynamic compaction (Hausmann, 1990). In this technique, a heavy weight is repetitively dropped onto the ground surface from a great height (Lukas, 1986). During this process, densification of a saturated or nearly saturated soil are achieved due to sudden loading, involved shear deformation, temporary high pore pressure generation (possibly liquefaction), and subsequent consolidation (Lukas, 1986; 1995).

Generally the weight of the tamper mass ranges from 6 to 170 tons, and the drop height is between 30 and 75 ft (Lukas, 1986). The use of a small mass falling from a lower height, usually 12 tons dropping from 36 ft is typically employed during small scale tamping operations (Hausmann, 1990). The parameters such as degree of saturation, soil classification, permeability, and thickness of the clay layer influence the suitability of a particular soil deposit for the dynamic compaction technique. Based on the grain size and the plasticity index (PI) properties of soils, Lukas (1986) characterized and grouped them into three different zones as shown in Figure 10.

This figure shows that the Zone I (pervious soils) soils are best suited for dynamic compaction. Zone II soils (semi-pervious) require longer duration to dissipate dynamic compaction induced excess pore water pressure to obtain the required level of improvement. Hence, soils in Zone II require multiple phases of dynamic compaction. It can be observed that the soils grouped under Zone III are not suitable for dynamic compaction. The effective depth of dynamic compaction can be as deep as 40 ft (12 m) but usually ineffective for saturated impervious soils, such as peats and clayey soils (Wahls, 1990). Besides, this technique is not feasible when the area of improvement required is smaller such as for highway embankments of confined widths (Hausmann, 1990). The application of this technique in highway related projects is less when compared to the other applications which include compacting sanitary landfills, rocky areas, dams, and air fields (Lukas, 1995). No documented cases where this method was used for mitigating settlements of fills underneath the slabs were found in the literature.



Zone 1: Best
Zone 3: Worst (consider alternate methods)
Zone 2: Must apply multiple phases to allow for pore pressure dissipation

Figure 10 – Grouping of Soils for Dynamic Compaction (Lukas, 1986)

3.3 Hydraulic Modification Techniques

Vertical Drains

Vertical drains in the form of sand drains were successfully used to enhance the consolidation process by shortening the drainage path from the vertical to the radial direction (Nicholson and Jardine, 1982). Recently, the usage of sand drains has been replaced by prefabricated vertical drains, also called as wick drains, accounting for their ease in installation and economy. Wick drains basically consist of a plastic core with a longitudinal channel wick functioning as a drain and a sleeve of paper or fabric material acting as a filter protecting the core. Configurations of different types of prefabricated vertical drains (PVDs) available in the market are shown by Bergado et al. (1996) as shown in the Figure 11. Typically PVDs are 100 mm wide and 6-8 mm thick and available in rolls (Rixner et al., 1986). The main purpose of prefabricated vertical drains is to shorten the drainage path and release the excess pore water pressure in the soil and discharge water from deeper depths thereby assisting in a speedy consolidation process of soft soils. Generally vertical drains are installed together with preloading to accelerate the consolidation process (Rixner et al., 1986; Bergado et al., 1996).

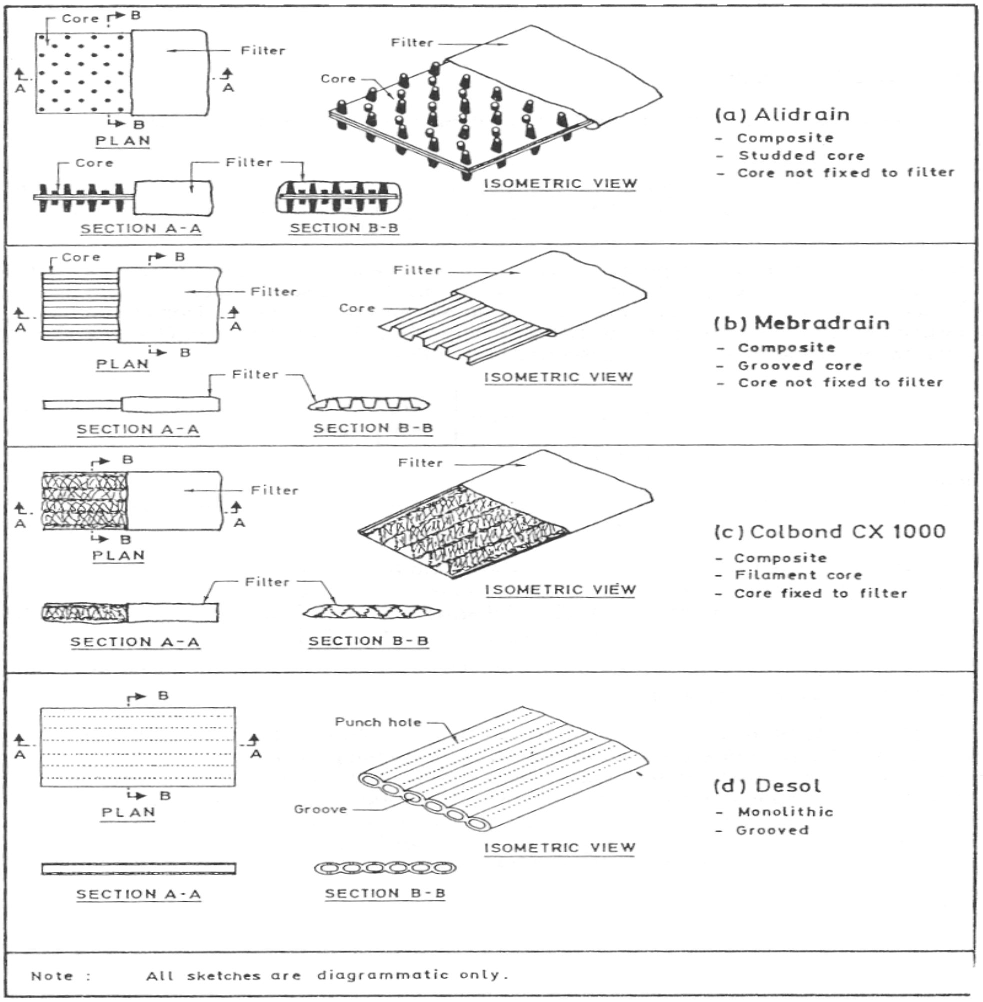


Figure 11 – Configurations of Different Types of Prefabricated Vertical Drains (Bergado et al., 1996)

Based on classic one-dimensional consolidation theory by Terzaghi (1943), Barron (1948) developed a solution to the problem of consolidation of the soil specimen with a central sand drain using two-dimensional consolidation by accounting for radial drainage. Later, Hansbo (1979) modified Barron's equation for prefabricated vertical drain application. The discharge capacity, spacing, depth of installation, and width and thickness of the wick drains are prime factors controlling the consolidation process. These design factors again depend on the in-situ conditions of the project location (Hansbo, 1997). These design procedures are described in detail by Hansbo (1979; 1997; 2001).

The first application of vertical sand drains for settlement control was experimented in California in the early 1930s and the first prototype prefabricated vertical drains were pioneered by Kjellman in Sweden in 1937 (Jamiolkowaski et al., 1983). Several researchers have reported the successful application and functioning of vertical sand and wick drains in highway embankment constructions from all over the world (Atkinson and Eldred, 1981; Bergado et al., 1988; Indraratna et al., 1994; Bergado and Patawaran, 2000). A typical arrangement of vertical drains in a soft soil under embankment with surcharge load is shown in Figure 12.

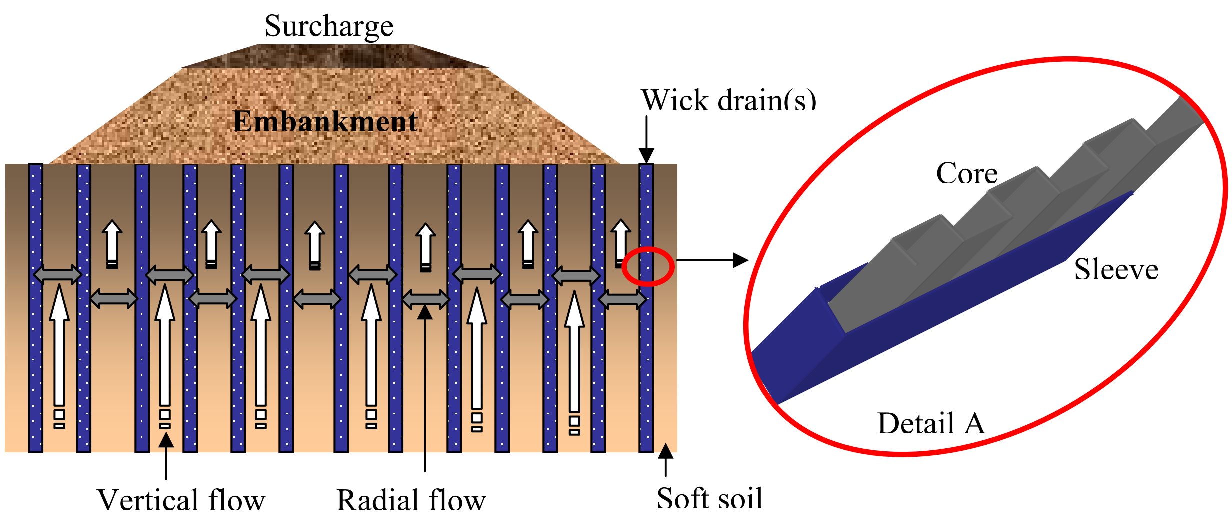


Figure 12 – Preloading with Prefabricated Vertical Drains to Reduce Consolidation Settlements

Hsi and Martin (2005) and Hsi (2007) described the successful use of wick drains along with reinforcing geotextile layers to mitigate unequal and differential settlements anticipated in highway approach embankments constructed over soft estuarine and marine clays in New South Wales, Australia. The proposed freeway connecting Yelgun and Chindera cities has nine flyovers and 39 freeway bridges over creeks and waterways having most of them located on soft estuarine and marine clays. The involved risks due to the very soft nature of these soils including long-term time dependent consolidation settlements, short-term instability of the embankment, and increase in fill quantity due to excessive settlement of embankment fill lead to the adopting of ground improvement techniques. They reported that installation of wick drains at a spacing of 1-3 m c/c on a grid pattern (Figure 13) allowed speedy construction of embankment over these soft soils.

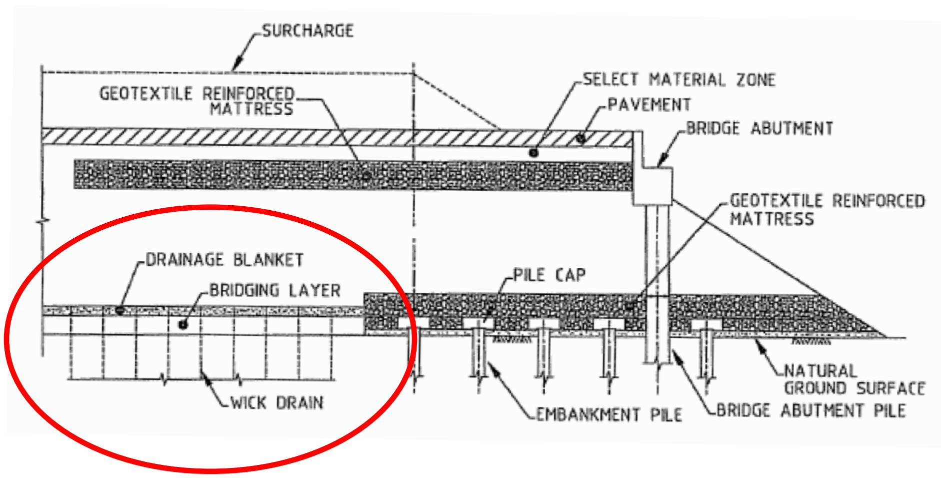


Figure 13 – Schematic Arrangement of Approach Embankment Treatment with Wick Drains and Driven Piles (Hsi and Martin, 2005)

To increase the embankment stability against potential slip failure, which was anticipated due to the speedy construction operations on soft soil, high strength geotextile reinforced mattresses were placed on the surface of the soft ground before placing the embankment (Hsi and Martin, 2005). The embankment near the bridge abutment was supported on timber driven piles to reduce the differential settlements between the approach embankment and the pile supported bridge abutments. These details about timber driven piles are discussed in the following appropriate section. The embankment section and the soft soil were instrumented with settlement plates to assess the risks during and after the construction.

Figure 14 (a, b) presents the measured and predicted settlements in soft foundation soil during and after construction stages. In this figure, the long-term settlements were predicted based on the ratio ($c_\alpha/1+e_0$) where, c_α is the secondary compression index and e_0 is the initial void ratio. The long-term differential and total settlements are predicted from back-calculated analysis of measured data from settlement plates also presented in the same Figure. From this graph, it can be noted that the reduced rate of long-term creep settlements after the removal of the surcharge and after the completion of construction (Hsi and Martin, 2005).

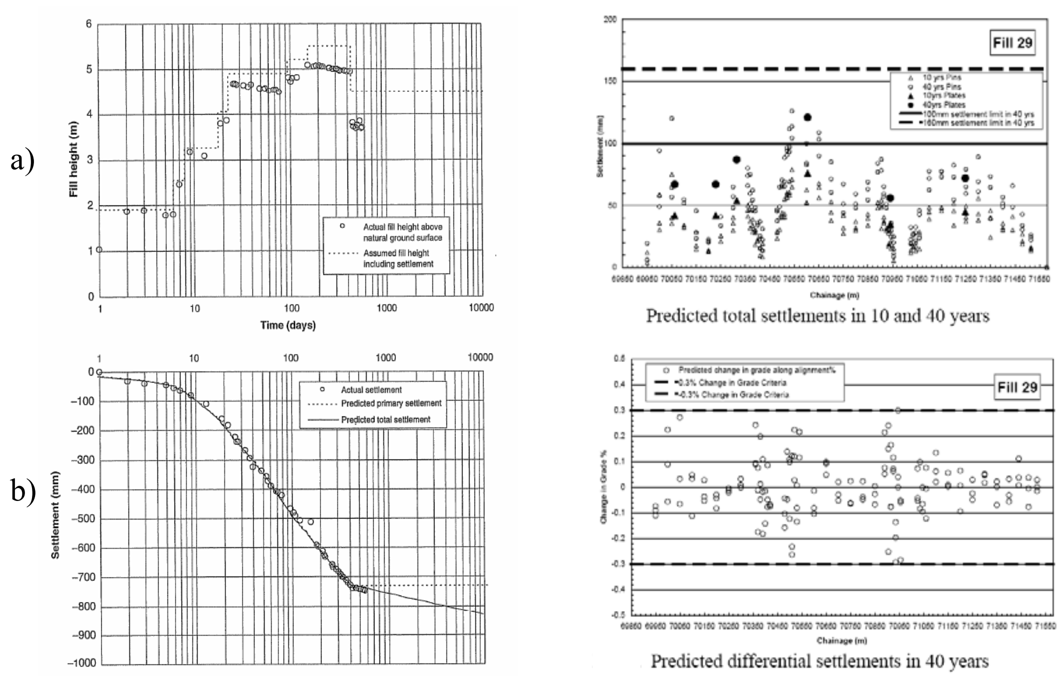


Figure 14 – Measured and Predicted Settlements with Time (Hsi and Martin, 2005)

3.4 Reinforcement Techniques

A wide variety of soil reinforcement techniques are available from which to choose. In all these techniques, good reinforcement elements are inserted to improve the selected property of the native weak soil. These inclusions include stone, concrete, or geosynthetics. Based on the type of construction of these methods, they are grouped as column reinforcement, pile reinforcement, and geosynthetic reinforcements. The following sections describe each technique in detail with the focus on controlling bridge approach settlements.

Column Reinforcement

Stone Columns

The stone columns technique is one of the classic solutions for soft ground improvement. This concept was first used in France in 1830 to improve a native soft soil (Barksdale and Bachus, 1983). The stone columns are a more common method to improve the load carrying capacities of weak foundation soils (Barksdale and Bachus, 1983; Michell and Huber, 1985; Cooper and Rose, 1999; Serridge and Synac, 2007), provide long term stability to the embankments and control settlements beneath the highway embankments (Munoz and Mattox, 1977; Goughnour and Bayuk, 1979; Barksdale and Bachus, 1983; Serridge and Synac, 2007). The secondary function of the stone columns is to provide the shortest drainage path to the excess pore water to escape from highly impermeable soils (Hausmann, 1990). This technique is best suitable for soft to moderately firm cohesive soils and very loose silty sands. In the United States, a majority of the stone column projects are adopted for improving silty sands (Barksdale and Bachus, 1983).

Stone column construction involves the partial replacement of native weak unsuitable soil (usually 15-35 percent) with a compacted column of stone that usually penetrates the entire depth of the weak strata (Barksdale and Bachus, 1983). Two methods are generally adopted to construct the stone columns including vibro-replacement, a process in which a high pressure water jet is used by the probe to advance the hole (wet process) and vibro-displacement, a process in which air is used to advance the hole (dry process).

In both the processes, stone is densified using a vibrating probe, also called vibroflot or poker, which is 12 to 18 in. (300 to 460 mm) in diameter. Once the desired depth is reached, stone is fed from the annular space between the probe and the hole to backfill the hole. The column is created in several lifts with each lift ranging from 1 – 4 ft thick. In each lift, the vibrating probe is repenetrated several times to densify the stone and push the stone into the surrounding soil. This procedure is repeated till the column reaches the surface of the native soil. Figure 15 shows the construction stages of stone columns.

Successful application of stone columns to improve the stability of highway embankments constructed over soft soils in Clark Fork, Idaho, (Munoz and Mattox, 1977) and in Hampton, Virginia (Goughnour and Bayuk, 1979). Stone columns can also be used to support bridge approach fills to provide stability and also to reduce the costly maintenance problem at the joint between the fill and the bridge. Based on an experience report circulated by a vibroflotation foundation company, Barksdale and Bachus (1983) have reported that stone columns were successfully used at Lake Okaoboji, Iowa, and Mobridge, South Dakota, for a bridge approach and an embankment structure built on soft materials.

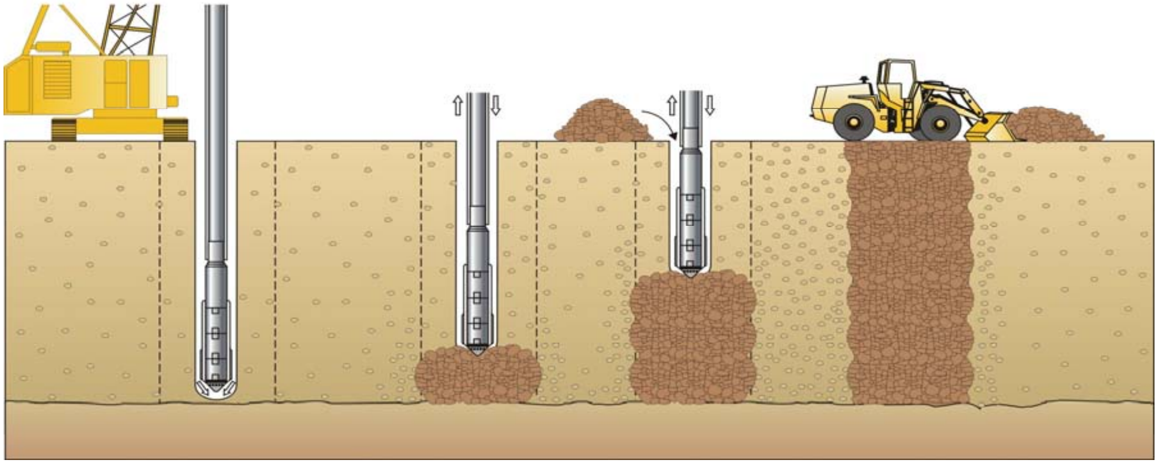


Figure 15 – Construction Stages of Stone Column (Hayward Baker; http://www.haywardbaker.com/services/vibro_replacement.htm)

Serridge and Synac (2007) reported the successful use of stone columns along with vibro-concrete columns in supporting highway embankment constructed over soft soil in South Manchester, UK. Figure 16 shows the schematic of the combination of ground improvement techniques used beneath the highway approach embankment. Prior to the actual construction, trial stone columns were constructed at a relatively low cost to verify the performance of the stone columns. Figure 17 depicts the performance of the stone columns in controlling settlements. Results from settlement plates show that the settlements occurring due to actual work were much smaller than the measured settlements in the trial sections.

The application or use of the stone columns technique is widely accepted and adopted in European countries (Barksdale and Bachus, 1983). In addition, McKenna et al., (1975) have reported a neutral performance of stone columns in soft alluvium supporting high embankment. They reported that the columns had no apparent effect on the performance of the embankment based on the comparison of instrumentation results obtained from both the piled and un-piled ground.

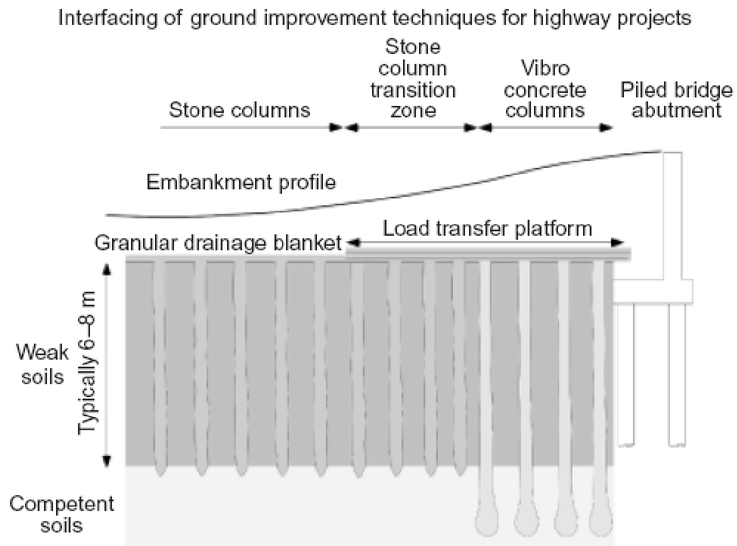


Figure 16 – Interfacing of Ground Improvement Techniques beneath Embankment Approach to Piled Bridge Abutment (Serridge and Synac, 2007)

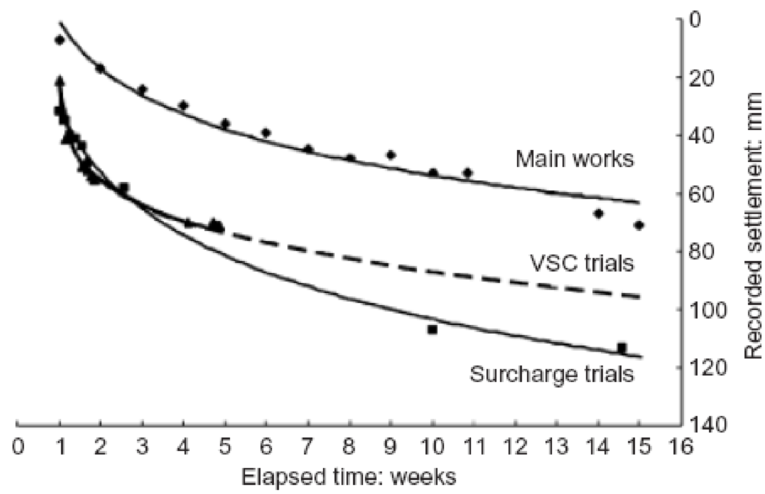


Figure 17 – Settlement Monitoring Results for Both Surcharge Trials on Untreated and Soil Reinforced with Stone Columns (Serridge and Synac, 2007)

Compaction piles

A series of compaction piles are used to improve the foundation soil, only when the deep deposits of loose granular soils such as sand or gravel are present and they can be densified by vibro-compaction or vibro-replacement methods (Hausmann, 1990). In these techniques, a probe is inserted into the soil until it reaches the required treatment depth (Hausmann, 1990). Then, the loosely deposited sands are vibrated in combination with air- or water-jet at a design frequency. Some amount of granular backfill materials are added to compensate for the void spaces resulting from the compaction. Finally, the probe is removed and the compacted granular backfill column is left in-situ. Figure 18 (a, b) depicts the sequential operations involved in the construction of compaction piles. Normally, the spacing of compaction piles is between 3 and 10 ft (1 and 3 m) and the depth of improvement can be achieved up to 50 ft (15 m) (Wahls, 1990). However, the vibro-compaction has its own limitation upon the grain size distribution of the granular fill material, which must contain fine material less than 20 percent (Baumann and Bauer, 1974) as shown in Figure 19.

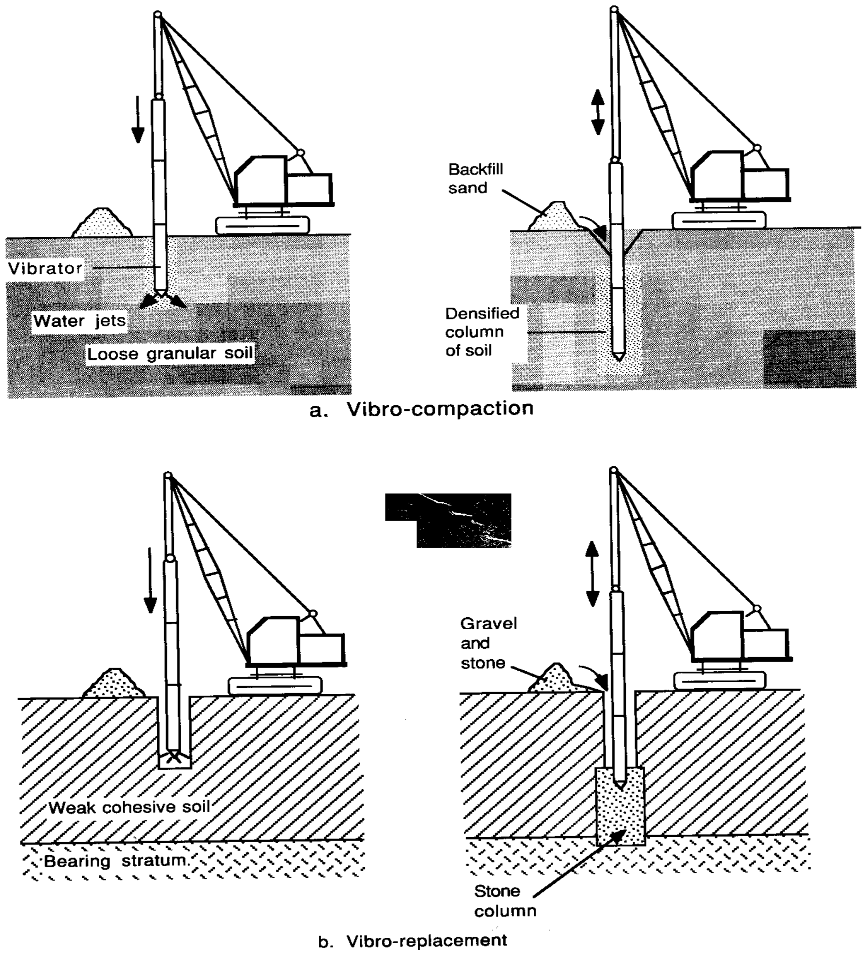


Figure 18 – Sequential Operations Involved in the Construction of Compaction Piles (Hausmann, 1990)

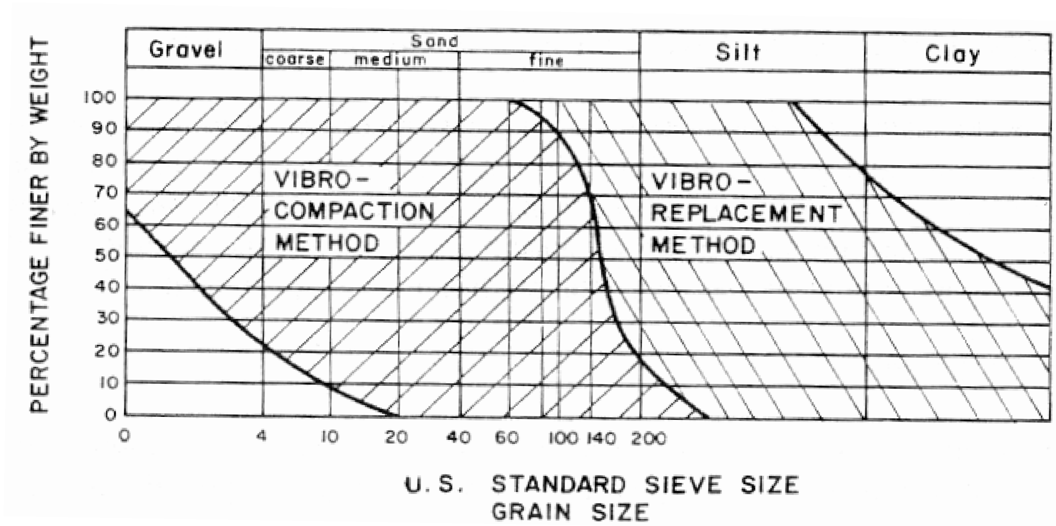


Figure 19 – Range of Soils Suitable for Vibro-Compaction Methods

(Baumann and Bauer, 1974)

Application of compaction piles to reduce the bridge approach settlements are not widely reported in the literature except in a few reported in Japan and Thailand. Sand compaction piles were used to support a road test embankment constructed at Ebetsu in Hokkaido, Japan (Aboshi and Suematsu, 1985). A combination of ground improvement techniques chosen in this project includes sand compaction piles and lime/cement columns. The embankment was constructed using mechanically stabilized earth with grid reinforcement. A schematic of the ground improvement techniques adopted in this study is shown in Figure 20. A control embankment was also constructed on native soft soil without any treatment. They reported that the combination of sand and lime/cement columns could support the embankment as high as 8 m, while the control embankment of height 3.5 m was collapsed exhibiting high deformations on the subsoil and heavy cracks in the embankment section.

Similar studies were carried out by Bergado et al. (1988; 1990) on soft Bangkok clay and confirmed that the granular compaction piles along with mechanically stabilized earth would be an economical alternative to support bridge approach embankments and viaducts.

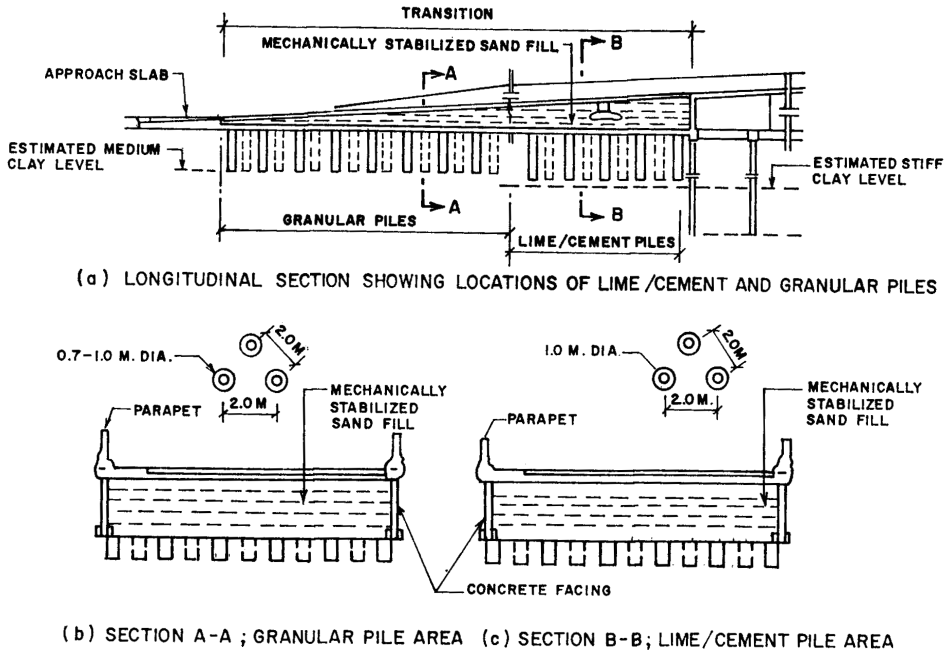


Figure 20 – Schematic of Granular Compaction Piles with Mechanically Stabilized Earth to Support Bridge Approach Embankments (Bergado et al., 1996)

Driven piles

To eliminate the impact of embankment settlement on the abutment piles a nest of driven piles consisting of timber piles or precast concrete piles can be installed adjacent to the abutment under the embankment (Hsi, 2007). These driven piles are expected to transfer the embankment loads on to the stiffer layers beneath; as a result, negligible settlements can be expected on the embankment surface.

Hsi (2007) reported the use of timber and concrete piles installed on a 2 m c/c square grid near the pile supported bridge abutment to arrest the differential settlements between the abutment and the embankment constructed along the Yelgun-Chinderah freeway in New South Wales, Australia. A series of pile caps (1 m square each) overlain by a layer of geotextile reinforced rock mattress (0.75 m thick) was also placed over the piles to form an effective bridging layer to transfer the embankment loads on to the piles as shown in Figure 21. This method allowed for earlier construction of the abutment piles and hence earlier completion of the bridges to allow haulage and construction traffic through the alignment. The data obtained from the settlement plates and pins installed in the embankment section revealed that the total creep settlements are reduced considerably.

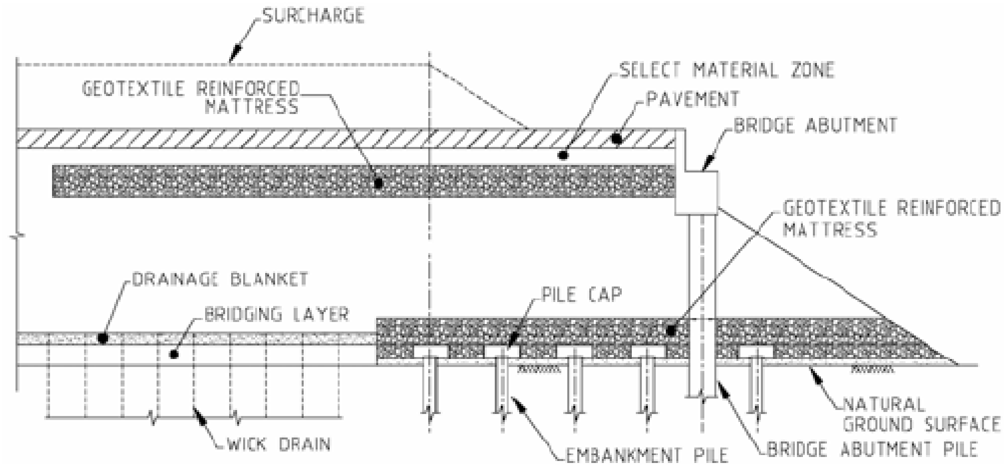


Figure 21 – Schematic of Bridge Approach Embankment Supported on Driven Piles (Hsi, 2007)

Geosynthetic Reinforcement

Whenever highway embankments are constructed over soft soils, the embankment load is distributed over a large area. These soft soils often exhibit failure due to excessive settlements or due to insufficient bearing capacity (Liu et al., 2007). A variety of techniques are available to increase the stability of these structures as discussed above. The application of geosynthetics in supporting highway embankments is gaining popularity (Magnan, 1994). In conventional piled embankment construction, the spacing is very close between piles, which leads to higher construction costs. However, introducing a layer of geosynthetic reinforcement in the form of geotextile or geogrid at the base of the embankment would not only bring down the cost but also increase the stability of the embankment structure (Liu et al., 2007).

Maddison et al. (1996) reported that the combination of a geosynthetic layer at the base of the embankment constructed over highly compressible peats and clays along with a series of vibro-concrete columns has proven to be the most effective method to increase the stability of the embankment structure and reduce long term settlements (Figure 22).

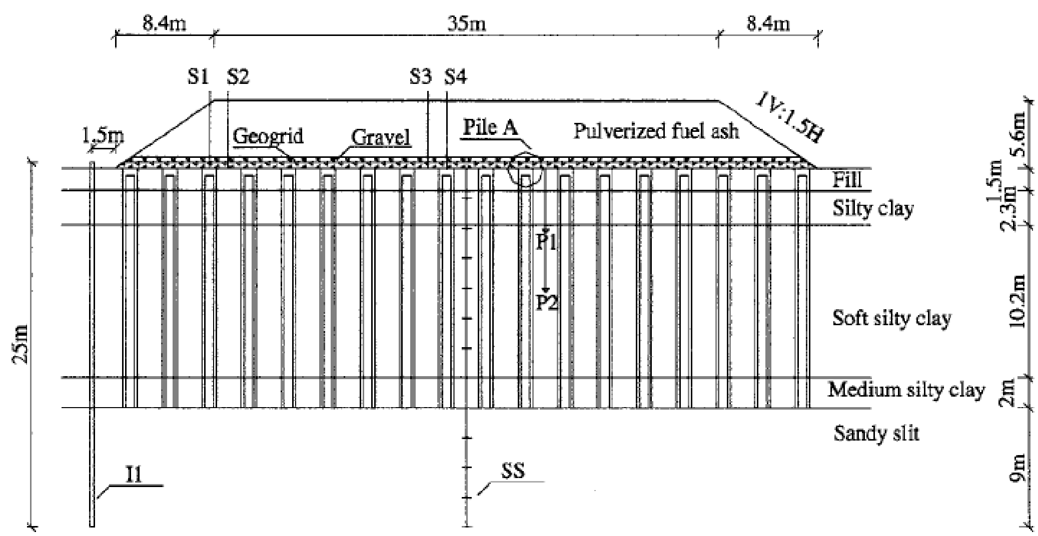


Figure 22 – Cross Section of Embankment with Basal Geogrid and Columns (Liu et al., 2007)

A more recent development in geosynthetic reinforcement is to provide a confinement to the foundation soil using geocells (Bush et al., 1990; Rowe et al., 1995). A geocell is a three dimensional, honey comb-like structure of cells interconnected at joints. These geocells provide lateral confinement to the soil against lateral spreading due to high structural loads and thereby increase the load carrying capacity of the foundation soil (Bush et al., 1990; Rowe et al., 1995; Krishnaswamy et al., 2000). The application of geocells as a foundation mattress for embankments constructed on soft soils has been studied by many researchers (Bush et al., 1990; Cowland and Wong, 1993; Rowe et al., 1995; Lin and Wong, 1999; Krishnaswamy et al., 2000).

Cowland and Wong (1993) reported a case study of the performance of a geocell mattress supported embankment on soft clay. A 10 m high embankment was constructed over soft ground comprised of a lagoon deposit overlain by alluvium supported by geocell foundation in Hong Kong (Figure 23). The embankment was extensively instrumented with inclinometers, pneumatic piezometers, hydrostatic profile gauges, settlement plates, surface settlement markers, and lateral movement blocks to verify the design assumptions and also to control the speed of the staged construction. Typical instrumentation data is presented in Figure 24. Results revealed that the geocell mattress performed very well in most of the instrumented sections. The measured settlement of the embankment was less than 50 percent of the predicted settlement with geocell foundation mattress. They reported that they measured excessive settlements due to construction on soft lagoon deposits. Overall, they concluded that the geocell foundation mattress behaved as a much stiffer raft foundation supporting the embankment.

Jenner et al. (1988), making use of slip line theory, have proposed a methodology to calculate the increase in bearing capacity due to the provision of a geocell mattress at the base of the embankment resting on soft soil. Krishnaswamy et al. (2000) carried out a series of laboratory model tests on geocell mattress supported earth embankments constructed over a soft clay bed. Lin and Wong (1999) illustrated the use of mixed soil and cement columns along with geotextile mattress at the base of the embankment in reducing the differential approach settlements. These details are discussed in previous sections. In all these cases, geocell mattress, either backfilled with good granular construction material or locally available mixed soils enhanced the load carrying capacity of the foundation soil and reduction in short term and long term settlements. Hence geocell mattress can be an economical alternative for shallow to moderate soft soil deposits. However, field studies are lacking on this method and its potential in real field conditions.

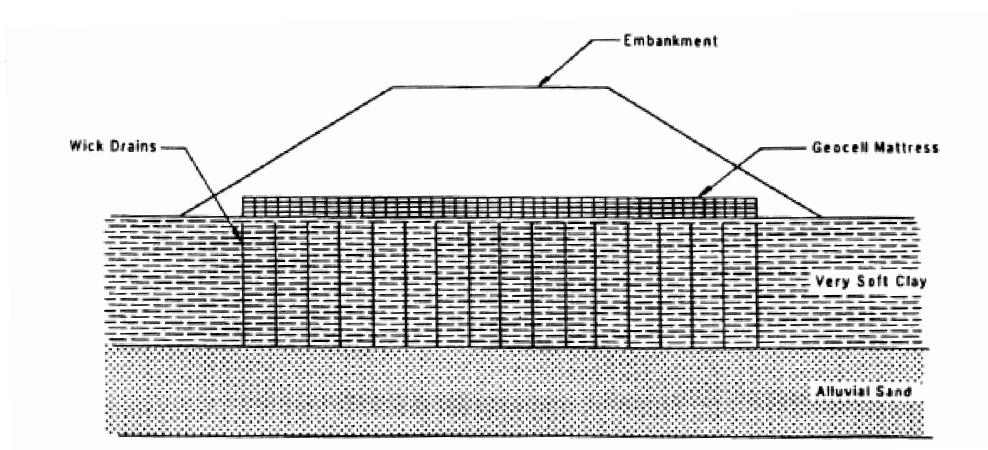


Figure 23 – Geocell Foundation Mattress Supported Embankment (Cowland and Wong, 1993)

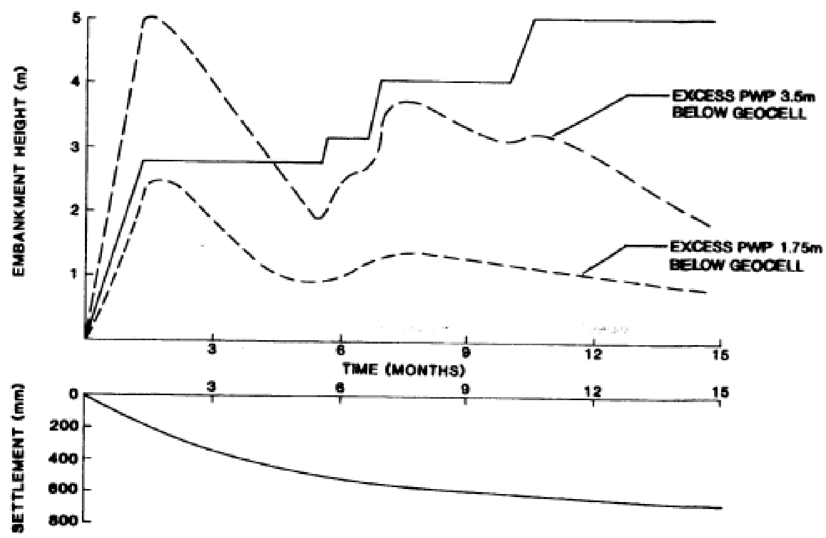


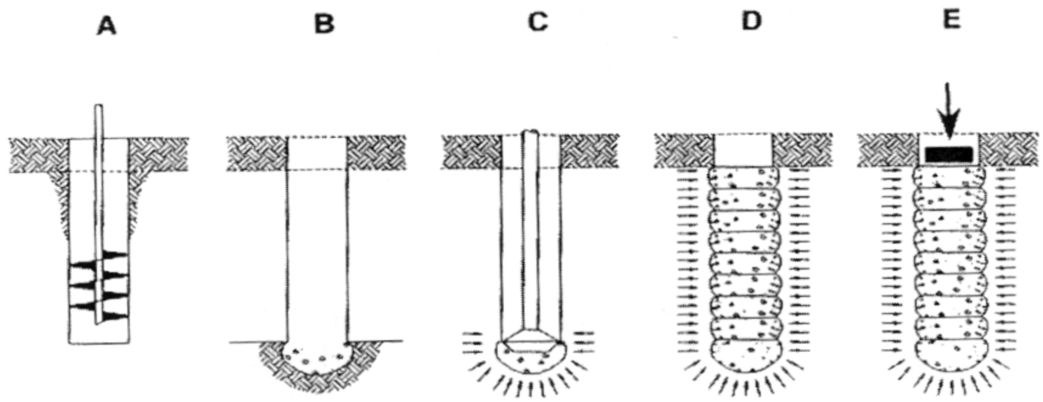
Figure 24 – Typical Load/Settlement-Pore Pressure/Time Profiles for Embankment Section (Cowland and Wong, 1993)

3.5 New Foundation Technologies

Geopiers

Geopiers, also called as short aggregate piers, are constructed by drilling the soft ground and ramming selected aggregate into the cavity, formed due to drilling, in lifts using a beveled tamper (Lien and Fox, 2001). The basic concept in this technique is to push/tamp the aggregate vertically as well as laterally against the soft soil to improve the stiffness against compressibility between the piers. These short piers can also allow radial drainage due to their open graded stone aggregate structure to accelerate the time dependent consolidation process and also to relieve excess pore water pressures generated in the soft soil (Lien and Fox, 2001).

The geopier soil reinforcement system has been adopted in transportation related applications such as roadway embankments and retaining walls to mitigate settlement of these structures (Lien and Fox, 2001; White and Suleiman, 2004). The design and construction details of these short piers are well documented in the literature (Lawton and Fox, 1994; Minks et al., 2001; White and Suleiman, 2004). Figure 25 demonstrates the schematic of the geopier construction sequence. Figure 26 presents the typical geopier system supporting the highway embankment.



A. Drill cavity.
B. Place stone at bottom of cavity.
C. Ram stone to form bottom bulb.
D. Densify stone in lifts to form undulated-shaft.
E. Preload top of *Geopier* element.

Figure 25 – Geopier Construction Sequence (Lien and Fox, 2001)

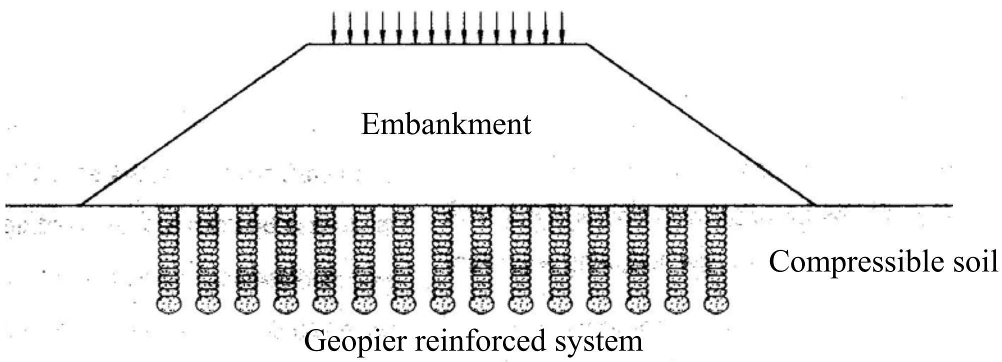


Figure 26 – Typical Geopier System Supporting the Embankment (Lien and Fox, 2001)

White et al. (2002) demonstrated the performance of the geopier system over stone columns in supporting highway embankments in Des Moines, Iowa. The purpose of the reinforcement technique was to reduce the magnitude and increase the time rate of consolidation settlements and to facilitate rapid abutment construction. These two sections were instrumented with settlement plates to measure during and post construction settlements. Prior to the embankment construction, geotechnical measurements were made to characterize both the sections by performing standard penetration tests (SPTs), borehole shear tests (BSTs), and full scale load tests. The SPT tests performed through production columns revealed that the average N-Values of 11 and 17 were obtained for stone columns and geopiers, respectively. Figure 27 compares the settlement readings with the increase in fill height obtained from settlement plates from both the stone columns section and the geopier system. It can be seen that the settlement of the matrix soil near the stone column is three times higher than the settlements observed in the matrix soil next to the geopier system.

White and Suleiman (2004) demonstrated the design procedures for short aggregate pier systems for a highway embankment construction. They observed two types of failure mechanisms, namely, bulging and plunging of the piers in their study on short aggregate piers. They recommend that the design of piers should be carried out based on the tip resistance to prevent bearing capacity problems.

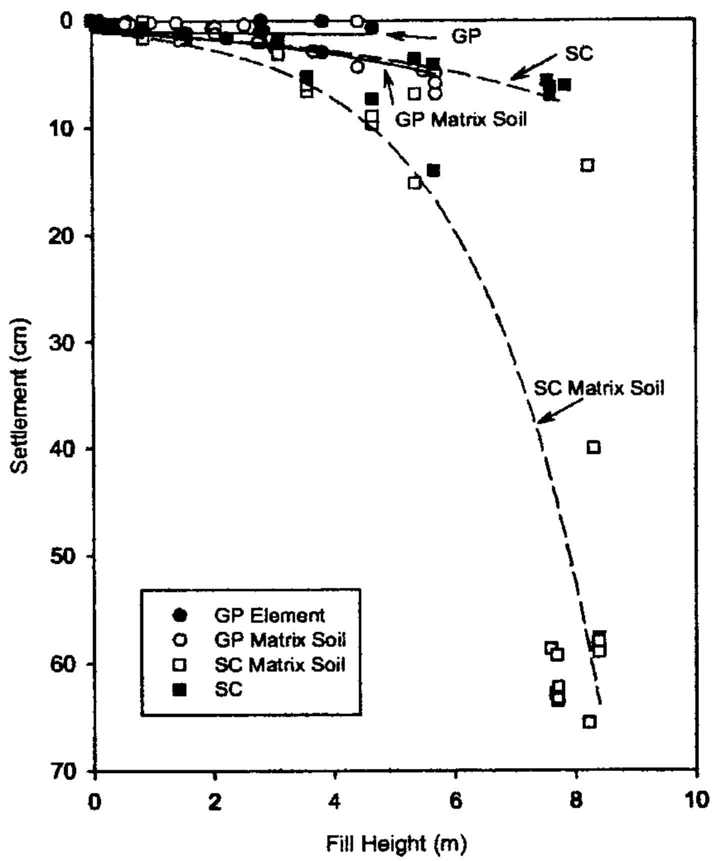


Figure 27 – Comparison of Settlement with Fill Height for Both Stone Column and Geopier Systems (White and Suleiman, 2004)

Deep Soil Mixing (DSM)

Deep Soil Mixing (DSM) technology, was pioneered in Japan in the late 1970s, and has gained popularity in the United States over many years in the field of ground improvement (Barron et al, 2006). DSM is a process to improve soil by injecting grout through augers that mix in with the soil, forming in-place soil-cement columns (Barron et al., 2006). Recently, the cement binder has been replaced with many other cementatious compounds such as lime, flyash or a combination of any two compounds. Hence, in a broader sense, the DSM technique is an in-situ mixing of stabilizers such as quicklime, cement, lime-cement, or ashes with soft and/or expansive soils to form deep columns to modify weak subgrade soils (Porbaha, 1998).

Figure 28 presents a typical DSM operation and resulting columns in the field. The DSM treated columns provide substantial improvements to soil properties such as strength and compressibility. The DSM columns have been used on several state highways to improve the stability of earth structures, to improve the bearing capacity of soils, to reduce the heave and settlement of embankments and roadways, to provide lateral support during excavations, to improve seismic stability of earthen embankments constructed over soft soils, and to reduce bridge approach settlements. This stabilization technique has been proven effective on soft clays, peats, mixed soils, and loose sandy soils (Rathmayer, 1996; Porbaha, 1998; Lin and Wong, 1999; Porbaha, 2000; Bruce, 2001; Burke, 2001).

The success of DSM-based ground treatment methods has lead to improved processing and novel installation technologies with the use of different additives incorporated as either dry or wet forms to stabilize subsoils. Currently, there are more than 18 different terminologies used to identify different types of deep soil mixing methods (Porbaha, 1998; 2000). Irrespective of these terminologies, the stabilization mechanisms are similar and their enhancements to soil strength and compressibility properties are considerable. The development of new applications should take advantage of the unique characteristic of Deep Soil Mixing in which rapid stabilization is possible in a short period of time, which will lead to accelerated construction in the field. Although the initial demand for DSM was to gain higher strength at lower cost, the recent complex construction dilemmas in expansive soils and other problematic soils have led to a greater need of evaluating this technology for expansive soil modification in field settings (Porbaha and Roblee, 2001).

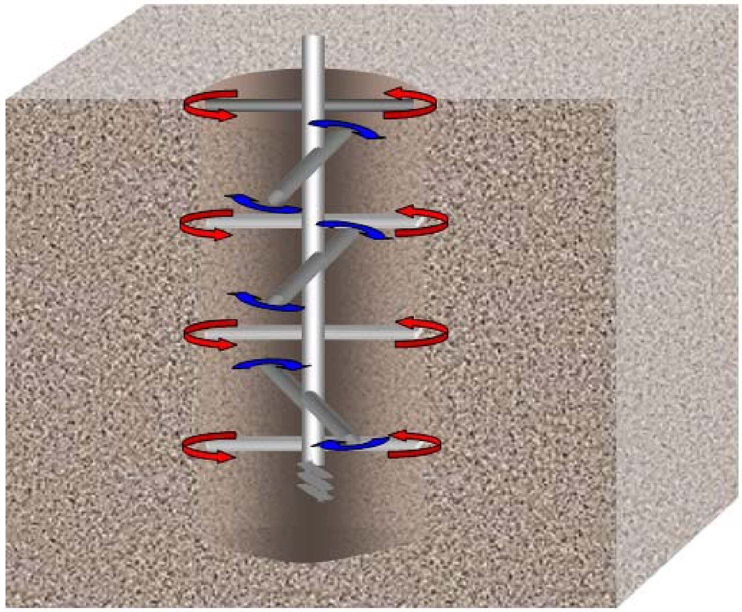


Figure 28 – Deep Soil Mixing (DSM) Operation and Extruded DSM Columns (Porbaha and Roblee, 2001)

This technology has been used by various state highway agencies such as Caltrans, Utah DOT, and Minnesota DOT in cooperation with the National Deep Mixing (NDM) Program, a research collaboration of the FHWA with 10 state DOTs. Several other case studies are reported both in and outside the U.S. for the use of DSM columns to reduce embankment settlements. Recently, TxDOT initiated Research Project (0-5179) to evaluate the DSM columns in mitigating the pavement roughness in expansive soils. The results from two instrumented sites demonstrate that the DSM is a promising technique to mitigate the pavement roughness.

Lin and Wong (1999) studied the deep cement mixing (DCM) technique to improve the strength of a 20 m thick soft marine clay with high moisture content to reduce the total and differential settlements at bridge embankments constructed along Fu-Xia expressway in the southeast region of China. The bridge abutments were planned to place on deep pile foundations with little to no allowable settlements. The maximum settlement of the embankment fill on the soft marine clays was predicted as 300 mm. To alleviate these differential settlements between pile-supported abutments and embankment fills, soil-cement deep soil mixing columns were selected to reinforce the embankment foundation soil.

Prior to the construction of the actual embankment(s) along the proposed Fu-Xia Expressway, trial embankment sections 2.7 km long were constructed to verify the efficiency of the selected ground improvement techniques such as prefabricated sand drains, plastic band drains, and deep cement mixing columns. They employed varying lengths of DCM columns with the longest columns placed near the bridge abutment and shorter columns away from the abutments as shown in Figure 29. This profile of DCM columns was adopted to increase the stiffness of the embankment towards the bridge abutments to result in gradual decrease in the settlements towards the bridge. A combination of band drains with a sand mat adjacent to the DCM treatment was to facilitate faster drainage of the pore water and to reduce the differential settlements between the DCM treated section and the rest of the untreated embankment sections.

The test embankment was heavily instrumented with inclinometers, settlement plates, multipoint settlement gauges, soil pressure cells, and piezometers to verify the performance of the DCM columns. Most of this instrumentation was done to the DCM columns and to the soft soil in between the columns except inclinometer casings. Inclinometer casing was installed at the embankment toe. Figure 30 shows the complete instrumentation used in their study. The monitoring results indicated that the settlement and lateral movement of soft clay treated by the DCM columns was reduced significantly. Use of the DCM columns of varying lengths having longer columns towards the pile supported abutments allowed the construction of the embankments to their full design height in a short period of time, with acceptable post-construction total differential settlement at the bridge approaches.

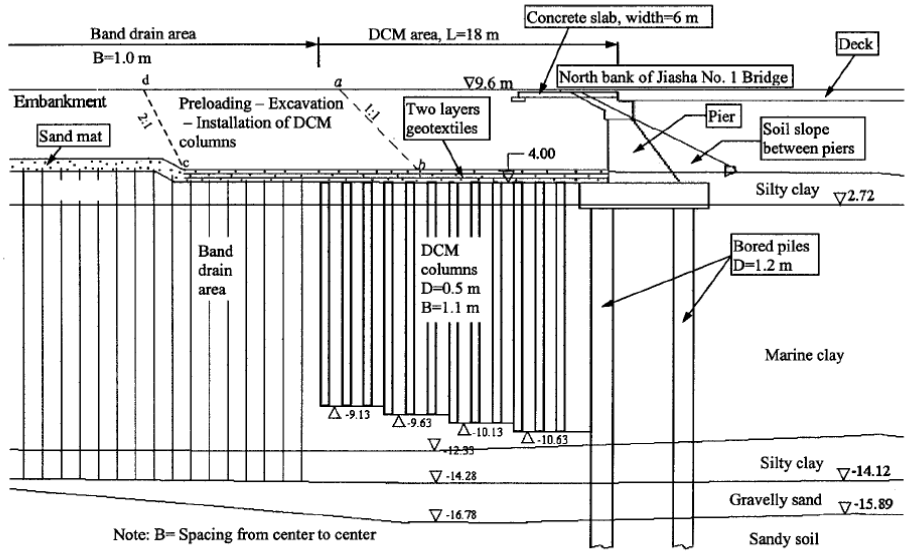


Figure 29 – Schematic of DCM Columns with Varying Length to Support Highway Embankment over Soft Marine Clay (Lin and Wong, 1999)

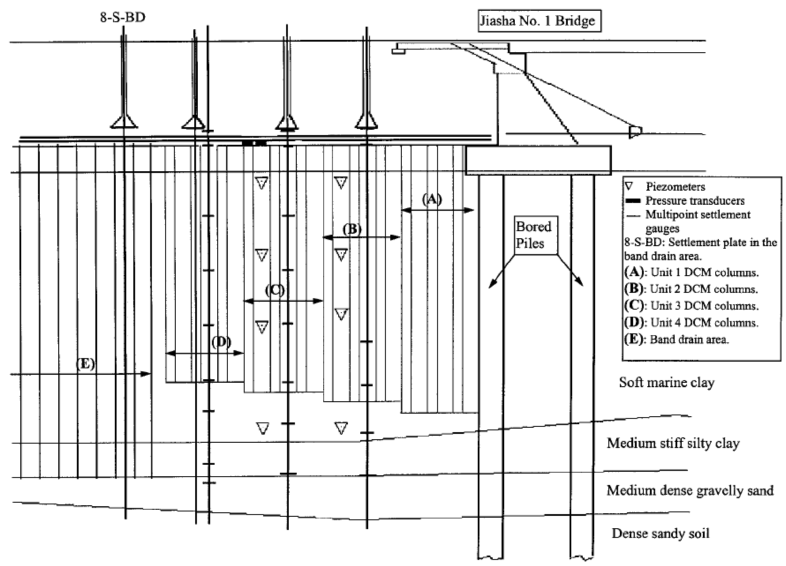


Figure 30 – Instrumentation Details of DCM Treated Embankment (Lin and Wong, 1999)

A similar technique (soil-cement columns) was used as a remediation method by Shen et al. (2007) to mitigate differential settlement of approach embankments along the Saga airport approach road constructed on Ariake clay in Japan. The actual road project was to connect the Saga city with the Saga airport in Japan. After the construction and open for traffic for two and half years, the low embankment adjacent to the bridge abutment settled 0.92 m though the predicted residual settlement due to traffic-load was about 0.2 – 0.4 m over the following 20 years period. Then a detailed geotechnical investigation was carried out which revealed that these road sections were underlain by thick layers of highly sensitive, soft Ariake clay.

Therefore, three remediation techniques were considered such as an asphalt concrete overlay, approach cushion slab method, and column approach (CA) method to mitigate these differential settlements between the approach embankment and the piled abutments. The first two conventional methods were selected based on Japanese pavement design guidelines. Conventional methods were also adopted at two different sections of the road project to compare the cost and performance of the column approach method. In the CA technique, the road approach (transitional zone) was supported by a row of soil-cement columns with lengths reduced with the increased distance away from the rigid piled abutment structure to smoothen the settlement profile within the transition zone as shown in Figure 31.

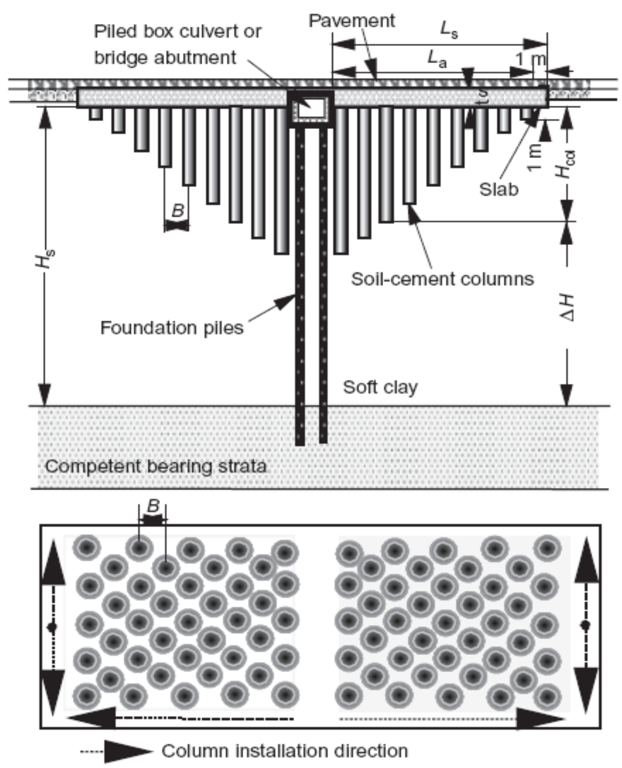


Figure 31 – Section and Plan View of Soil-Cement Pile Supported Approach Embankment (Shen et al., 2007)

A connecting slab was used to transfer the embankment loads to the CA system. The details of the design parameters of the CA method such as length of the soil-cement columns, spacing between columns, and details of the connecting slab are clearly described by Shen et al. (2007). They reported that the column approach method is proven to be economical and efficient in mitigating the differential settlements though the initial construction costs are higher than the conventional treatment methods discussed. Figure 32 shows that the CA method is economical when the differential settlements are more than around 300 mm.

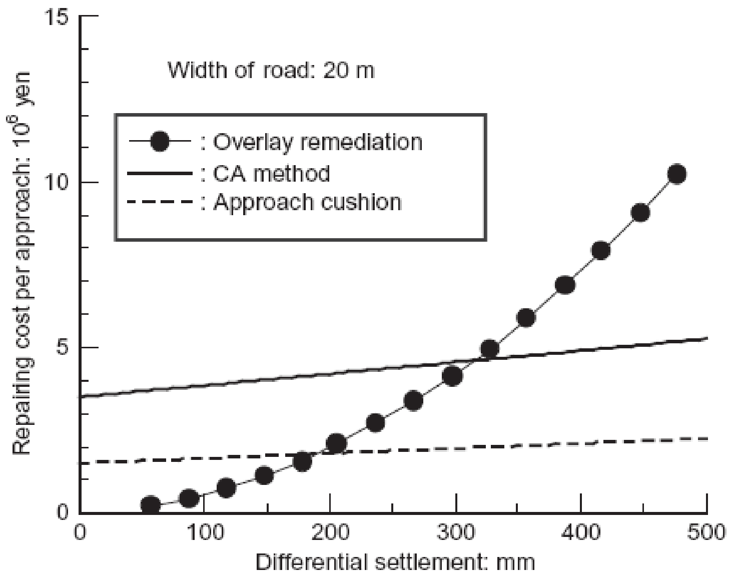


Figure 32 – Maintenance Cost with Differential Settlements (Shen et al., 2007)

Concrete Injected Columns

Concrete injected columns (CICs) are an innovative technique where a soil displacement pile mechanism is used to create in-situ concrete columns without reinforcement (Hsi, 2007; 2008). CICs are installed by inserting a displacement tool (auger) into the soft soil by rotating and pushing the tool. Upon reaching the final level, concrete is pumped through the hollow stem of the tool during extraction of the tool as shown in Figure 33. Inserting reinforced casing into the CICs is optional and the depth to which the reinforcement casing can be installed is also limited (Hsi, 2008). Typically these columns are prepared at 500 mm diameter and the length of these columns can be extended to reach a stiff strata or shallow bed rock. This technique is widely used to reinforce the very soft to soft foundation soils (Hsi, 2007). CICs were recently adopted to control the excessive long term settlements of approach embankments constructed on estuarine and marine soft clays along Brunswick Heads – Yelgun upgrade Pacific highway, Australia.

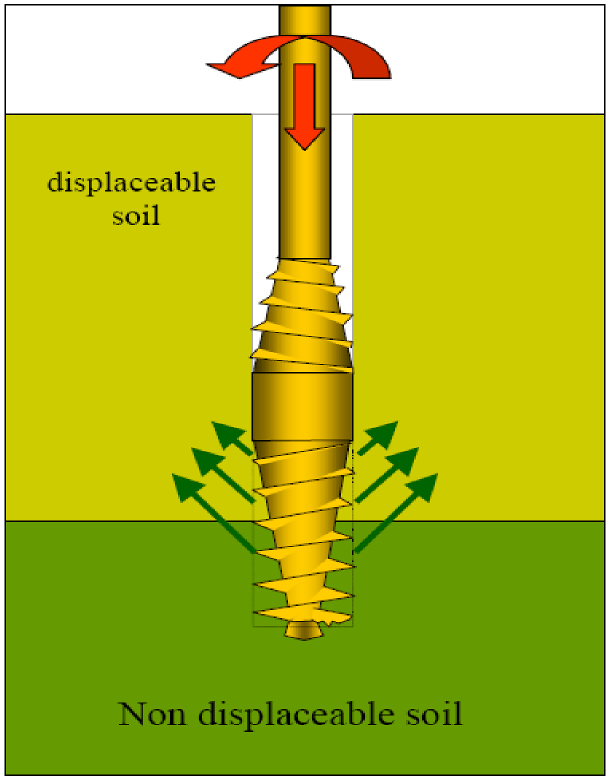


Figure 33 – Installation of Concrete Injected Columns (Hsi, 2008)

Two geometric patterns (Zone 1 & 2) of CICs are installed in the soft foundation soil as shown in Figure 34. Zone 1 was for the support of the approach embankment, and Zone 2 was to eliminate abrupt differential settlement between the closely spaced approach embankment section (Zone 1) and rest of the embankment. The spacing adopted for CICs in Zone 2 is around 2 m c/c and hence provided with a pile cap. These CICs were covered with a pair of geotextile blankets to uniformly distribute the embankment loads to the CICs. This combination of CICs with geotextiles provided a competent base for the embankment.

To assess the performance of CICs, the embankment section was instrumented with inclinometers to measure the lateral movements of the embankment due to construction activity and further, settlement plates to measure the settlement of the embankment. Figure 35 presents the data obtained from the settlement plates. The data obtained from this instrumentation imparted that the settlements are well within the allowable limits stipulated for this project. These limits are that the pavement was required to achieve a maximum of 100 mm residual settlement and a change in grade of 0.3 percent in any direction over the 40 year design life of the pavement. In addition, this technique allowed constructing the pile foundation for the abutment prior to preloading the embankment, which led to a reduction in total project costs.

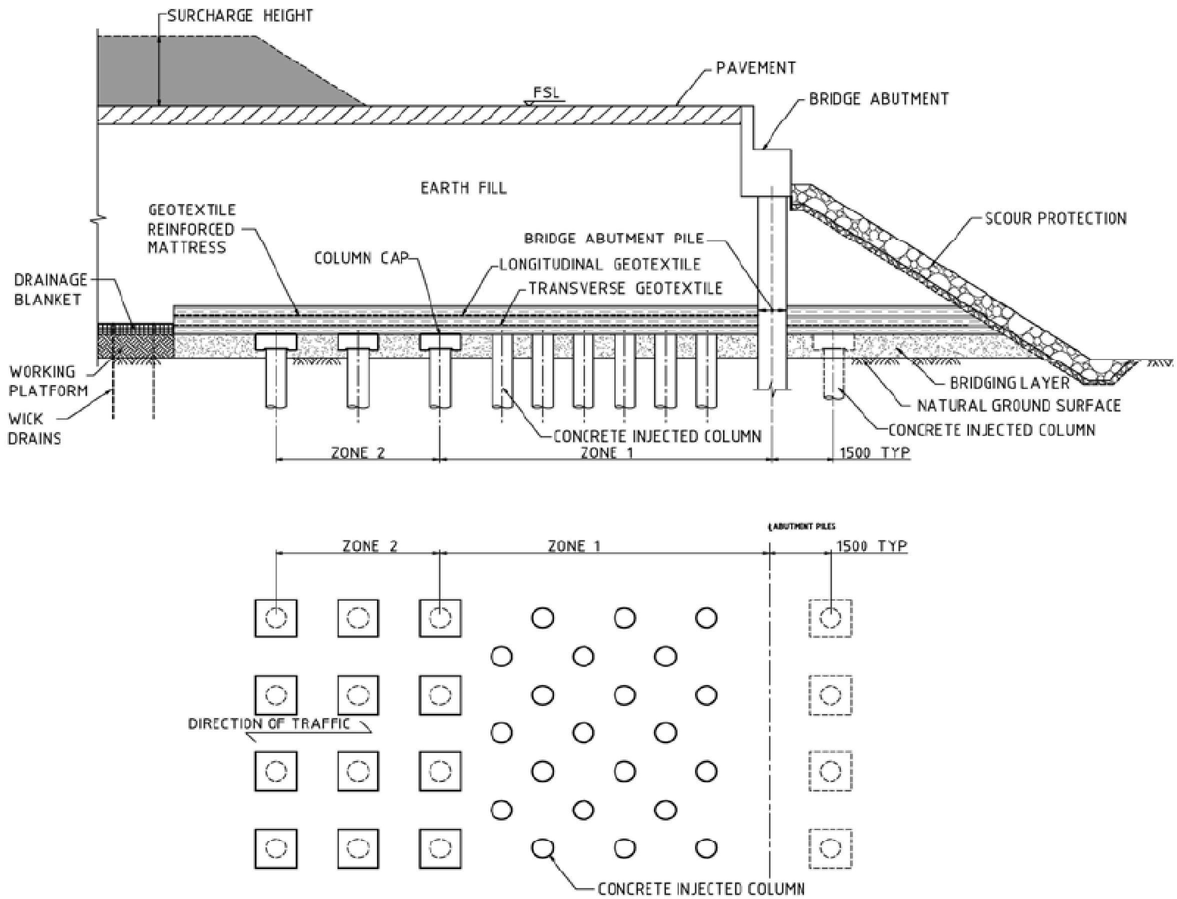


Figure 34 – Bridge Approach Treatment with Concrete Injected Columns (Sectional and Plan view) (Hsi, 2008)

Continuous Flight Auger Cast Piles (CFA)

Continuous Flight Auger Cast Piles (CFA) are installed by rotating a continuous-flight hollow shaft auger into the soil to reach a specified depth. High strength cement grout or sand or concrete is pumped under pressure through the hollow shaft as the auger is slowly withdrawn. If this process uses pressure grouting, these CFA piles are sometimes termed as Auger Pressure Grouted (APG) piles. The resulting grout column hardens and forms an auger cast pile (Neely, 1991; Brown et al., 2007). Reinforcing, when required, can be installed while the cement grout is still fluid, or in the case of full length single reinforcing bars, through the hollow shaft of the auger prior to the withdrawal and grouting process (Neely, 1991; Brown et al., 2007).

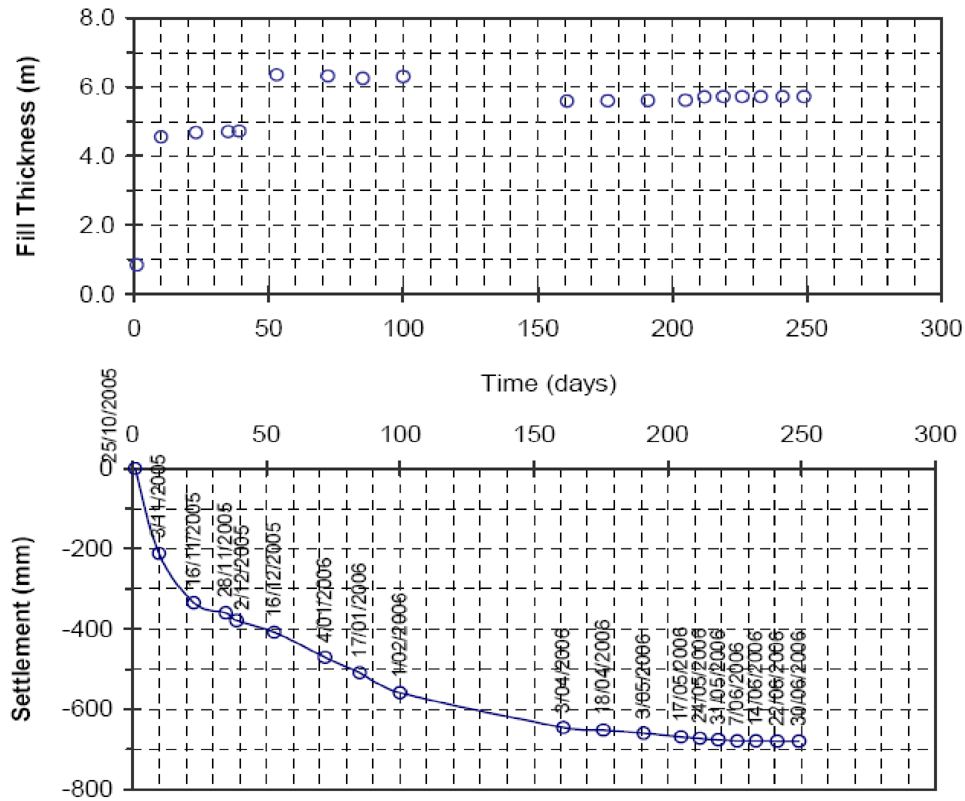


Figure 35 – Settlement Profiles Obtained from Settlement Plates (Hsi, 2008)

Auger cast piles can be used as friction piles, end-bearing piles, anchor piles; auger cast vertical curtain wall, lagging wall and sheet pile wall (Brown et al., 2007). The advantages of CFA piles over other pile types (driven piles) include less noise, no objectionable vibrations, no casing required, can be installed in limited headroom conditions, and soil samples can be obtained from each borehole (Brown et al., 2007). The typical dimensions reported are from 12 inches to 18 inches. However, auger cast piles with diameters of 24, 30, and 36 inches have been successfully utilized with tests being conducted as high as 350 tons.

O'Neill (1994) and recently Brown et al. (2007) summarized the construction systems of augered piles and documented different methods available to estimate the axial capacity of CFA piles. Figure 36 shows the construction procedures for continuous auger cast piles and screw piles. Brown et al. (2007) summarized the advantages and disadvantages of CFA piles and driven piles. Although several advantages of CFA piles have been stated, the major two disadvantage aspects of these piles must be noted. First, the available QA methods to assess the structural integrity and the pile bearing capacity of these piles are not reliable. Second, the disposal of associated soil spoils when the soils are contaminated. In addition, CFA piles were not considered by public transportation departments in the U.S. prior to the 1990s because of the lack of design methods. The use of CFA piles has been increased in the U.S. after recent developments in automated monitoring and recording devices to address quality control and quality assurance issues (EBA Engineering Inc., 1992; Brown et al., 2007).

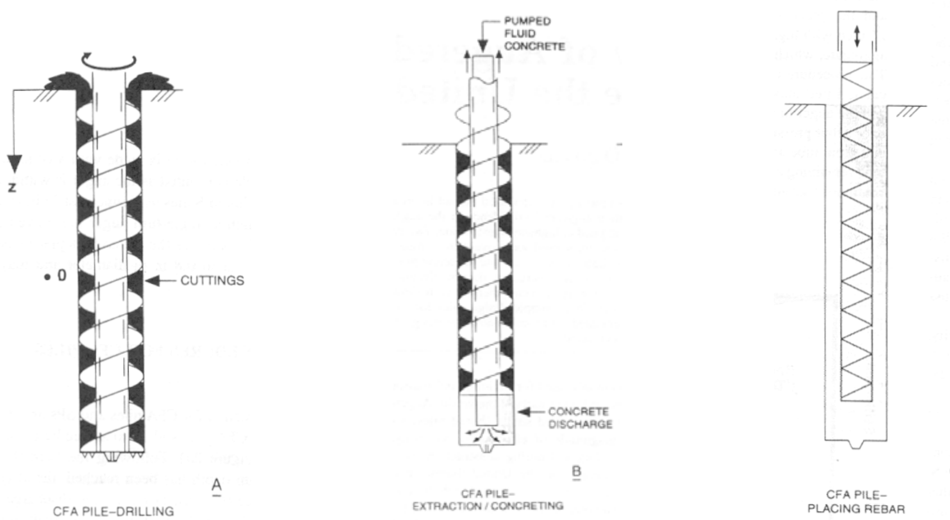


Figure 36 – Construction Procedures for Continuous Auger Piles (O'Neill, 1994)

Since CFA piles behave somewhere between drilled shafts and driven piles, CFA piles have been designed using both approaches (Zelada and Stephenson, 2000; Brown et al., 2007). McVay et al. (1994) reported the successful use of auger cast piles in coastal shell-filled sands in Florida. They concluded that the equipment selection, drilling rate, grout's aggregate size, grout pumping, augur removal, and grout fluidity significantly affect the quality and the load carrying capacity of the augered piles. They summarized different empirical methods to estimate the capacities of auger cast piles which include, Wright and Reese method, Neely's method, and LPC (Laboratorie Des Ponts et Chausses) method. McVay et al. (1994) compared the measured load-settlement data with predicted capacities from these methods. The Wright and Reese method gave reasonable predictions of capacities at 5 percent settlement of the pile diameter.

They also concluded that the use of 5 percent of the pile's diameter for the failure criteria to be acceptable for typical augured cast piles in the 12 in. diameter range.

Vipulanandan et al. (2004) studied the feasibility of CFA piles as a bridge abutment foundation alternative to the driven pile system on a new bridge constructed by the Texas Department of Transportation (TxDOT) near Crosby, Texas. They noticed few construction issues for the installation of the CFA piles including the difficulties involved in reinforcing the entire depth of piles due to excessive grout velocity and/or lack of timely workmanship by the contractor. They also reported that the load carrying mechanism of the CFA piles was entirely due to the mobilization of the side friction resistance of the pile based on the pile load test on the instrumented test piles. They also concluded that the cost involved in installing the CFA pile system was 8 percent less than that of the driven pile system for the same length of the foundations. In addition, the CFA piles are having a higher factor of safety against axial loading than the other foundations.

CFA piles to support approach embankment are considered only when the foundation soil is highly compressible and the time required for the consolidation settlement is very high, and when minimization of post-construction settlements and construction delays are required (Brown et al., 2007). Only a few studies are available in the literature where the CFA piles were used to support the embankment in order to mitigate settlements. Figure 37 shows the CFA pile supported railway embankment in Italy. Pile support was used to increase the stability of the embankment against excessive settlement anticipated due to extra fill on the existing embankment and load due to increased rail traffic. The CFA piles were capped using concrete filled cylinders and the fill overlain by the pile caps are reinforced with geotextiles. Performance details of these systems for settlement control are not yet documented.

Other details on the CFA piles including construction sequences, materials required, equipment specifications, and performance based design factors of these CFA piles can be found in Brown et al. (2007).

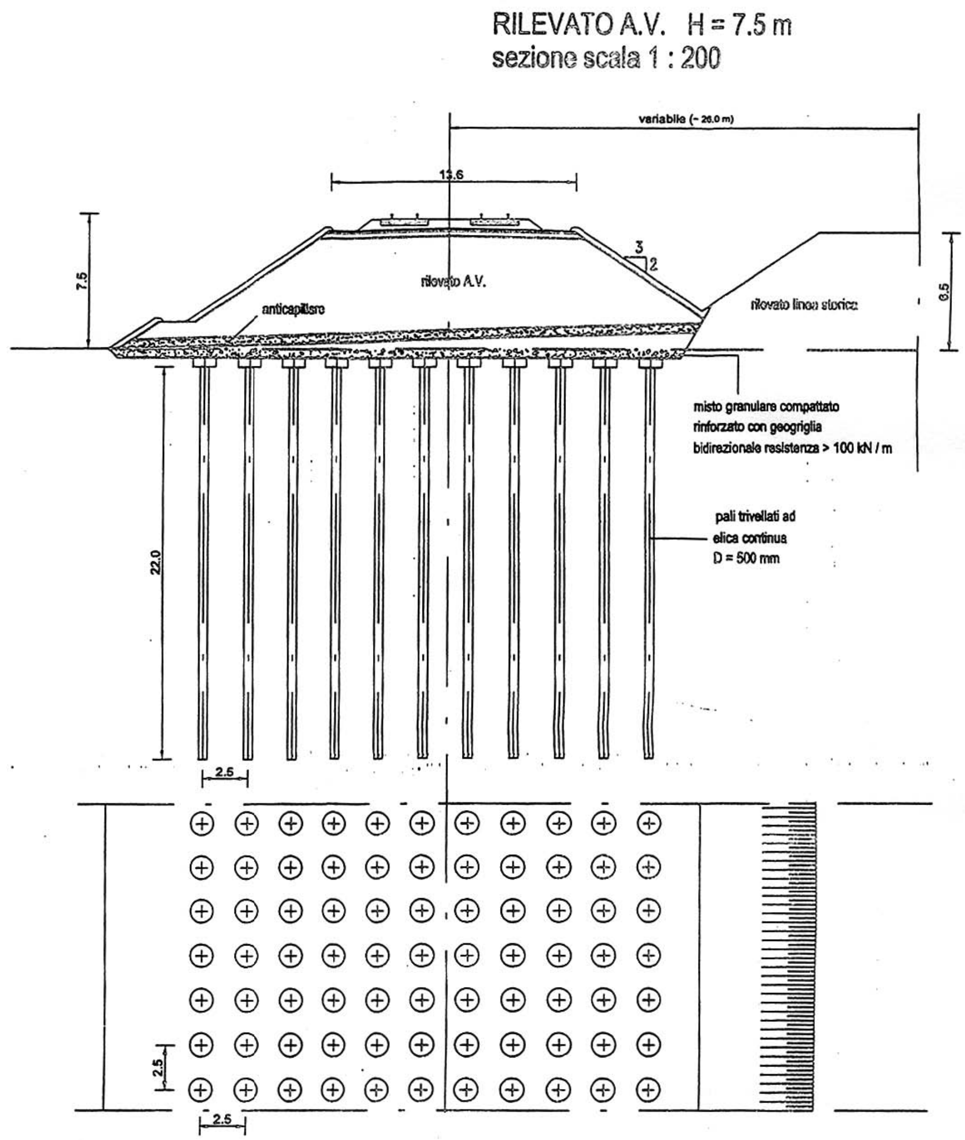


Figure 37 – CFA Pile Supported Railway Embankment for Italian Railway Project (Brown et al., 2007)

3.6 Improvement of Approach Embankment/Backfill Material

The bridge approach embankment has two functions: first to support the highway pavement system and second to connect the main road with the bridge deck. Most of the approach embankments are normally constructed by conventional compaction procedures using materials from nearby roadway excavation or a convenient borrow pit close to the bridge site. This implies that the serviceability of the embankment, in the aspects of slope stability, settlement, consolidation, or bearing capacity issues, depends on the geotechnical properties of the fill materials (Wahls, 1990). In addition, since the embankment must provide a good transition between the roadway and the bridge, the standards for design and construction considerations both in materials quality requirements and compaction specifications must be specified in order to limit the settlement magnitude within a small acceptable degree (Wahls, 1990).

Generally, the materials for embankment construction should have these following properties (White et al., 2005):

- being easily compacted,
- not time-dependent,
- not sensitive to moisture,
- providing good drainage,
- erosion resistance, and
- shear resistance.

Dupont and Allen (2002) cited that the most successful method to construct the approach embankments is to select high quality fill material, with the majority of them being a coarse granular material with high internal frictional characteristics. Several research methods have been attempted to define methods to minimize potential of settlement and lateral movement development in the approach embankments, and these studies are discussed in the following.

Hoppe (1999) studied the embankment material specifications from various DOTs. The results from his survey are presented in Tables 3 and 4. It can be seen from Table 3 that 49 percent of the state agencies use more rigorous material specifications for an approach fill than for a regular highway embankment fill. Furthermore, the study also shows that typical requirements for the backfill materials among the different states varied with one another. One common requirement followed by several states is to limit the percentage of fine particles in the fill material in order to reduce the material plasticity. As an example, the allowable percentage of material passing the No. 200 (75-micron) sieve varies from less than 4 percent to less than 20 percent. Another requirement commonly found is to enhance the fill drainage properties by a requisite of pervious granular material.

From the same study by Hoppe (1999), two other conclusions can be further drawn from Table 4. First, in many states, a 95 percent of the standard proctor test compaction condition is generally specified for the compaction of approach fill.

Table 3 – Embankment Material Specifications (Hoppe, 1999)

State	Same/Different from Regular Embankment	% Passing 75 mm (No.200 sieve)	Miscellaneous
AL	Same		A-1 to A-7
AZ	Different		
CA		<4	Compacted pervious material
CT	Different	<5	Pervious material
DE	Different		Borrow type C
FL	Same		A-1, A-2-4 through A-2-7, A-4, A-5, A-6, A-7 (LL<50)
GA	Same		GA Class I, II or III
ID			A yielding material
IL	Different		Porous, granular
IN	Different	<8	
IO	Different		Granular; can use Geogrid
KS			Can use granular, flowable or light weight
KY		<10	Granular
LA			Granular
ME	Different	<20	Granular borrow
MA	Different	<10	Gravel borrow type B, M1.03.0
MI	Different	<7	Only top 0.9 m (3 ft) are different (granular material Class II)
MN		<10	Fairly clean granular
MO			Approved material
MS	Different		Sandy or loamy, non-plastic
MT	Different	<4	Pervious
NE			Granular
NV	Different		Granular
NH	Same	<12	
NJ	Different	<8	Porous fill (Soil Aggregate I-9)
NM	Same		
NY		<15	<30% Magnesium Sulfate loss
ND	Different		Graded mix of gravel and sand
OH	Same		Can use granular material
OK	Different		Granular just next to backwall
OR	Different		Better material
SC	Same		
SD	Varies		Different for integral; same for conventional
TX	Same		
VT	Same		Granular
VA	Same		Pervious backfill
WA			Gravel borrow
WI	Different	<15	Granular
WY	Different		Fabric reinforced

Table 4 – Lift Thickness and Percent Compaction Requirements (Hoppe, 1999)

State	Lift Thickness, mm(inch)	% Compaction	Miscellaneous
AL	203(8)	95	
AZ	203(8)	100	
CA	203(8)	95	For top 0.76 m (2.5 ft)
CT	152(6)	100	Compacted lift indicated
DE	203(8)	95	
FL	203(8)	100	
GA		100	
ID	203(8)	95	
IL	203(8)	95	For top, remainder varies with embankment height
IN	203(8)	95	
IO	203(8)	None	One roller pass per inch thickness
KS	203(8)	90	
KY	152(6)	95	Compacted lift indicated; Moisture = +2% or -4% of optimum
LA	305(12)	95	
ME	203(8)		At or near optimum moisture
MD	152(6)	97	For top 0.30 m (1ft), remainder is 92%
MA	152(6)	95	
MI	230(9)	95	
MN	203(8)	95	
MO	203(8)	95	
MS	203(8)		
MT	152(6)	95	At or near optimum moisture
NE		95	
NV		95	
NH	305(12)	98	
NJ	305(12)	95	
NY	152(6)	95	Compacted lift indicated
ND	152(6)		
OH	152(6)		
OK	152(6)	95	
OR	203(8)	95	For top 0.91 m (3ft), remainder is 90%
SC	203(8)	95	
SD	203-305(8-12)	97	0.20 m (8 inch) for embankment, 0.30 m (12 inch) for bridge end backfill
TX	305(12)	None	
VT	203(8)	90	
VA	203(8)	95	+ or – 20% of optimum moisture
WA	102(4)	95	Top 0.61 m (2 ft), remainder is 0.20 m (8 inch)
WI	203(8)	95	Top 1.82 m (6 ft and within 60 m (200 ft), remainder is 90%
WY	305(12)		Use reinforced geotextiles layers

Second, the approach fill material is normally constructed at a lift thickness of 8 inches. In Texas, a loose thickness of 12 inches compacted to 8 inches of fill is commonly used and the percent compaction is not always specified. Dupont and Allen (2002) also conducted another survey of 50 state highway agencies in the USA in order to identify the most common type of backfill material used in the embankments near bridge approaches. Their study shows that most of the state agencies, i.e., 38 states use granular material as the backfill; 3 states use sands; 6 states use flowable fill; while 17 states use compacted soil in the abutment area.

A few other research studies were conducted to study the limitations of the percent fine material used in the embankment fill. Wahls (1990) recommended that the fill materials should have a plasticity index (PI) less than 15 with percent fines not more than 5 percent. The FHWA (2000) recommended backfill materials with less than 15 percent passing the No. 200 sieve. Another recommendation of the backfill material by Seo (2003) specifies the use of a backfill material with a plasticity index (PI) less than 15, with less than 20 percent passing the No. 200 sieve and with a coefficient of uniformity greater than 3. This fill material is recommended to be used within 100 feet of the abutment.

For the density requirements, Wahls (1990) suggested two required density values; one for roadway embankments and the other for bridge approaches. For embankment material, the recommended compaction density is 90 to 95 percent of maximum dry density from the AASHTO T-99 test method, while the density for the bridge approach fill material is recommended from 95 to 100 percent of maximum dry density from the AASHTO T-99 test method. Wahls (1990) also stated that well-graded materials with less than 5 percent passing the No. 200 sieve are easy to be compacted and such material can minimize post construction compression of the backfill and can eliminate frost heave problems.

Seo (2003) suggested that the embankment and the backfill materials within the 100 foot-length from the abutment should be compacted to 95 percent density of the modified proctor test. White et al. (2005) also recommended the same compaction of 95 percent of the modified proctor density for the backfill. White et al. (2005) also used a Collapse Index (CI) as a parameter to identify an adequacy of the backfill material in their studies. The CI is an index, which measures the change in soil volume as a function of placement water content. It was found that materials placed at moisture contents in the bulking range from 3 percent to 7 percent with a CI value up to 6 percent meet the Iowa DOT specifications for granular backfills.

In the current TxDOT Bridge Design Manual (2001), the approach slab should be supported by the abutment backwall and the approach backfill. Therefore, the backfill materials become a very important aspect in an approach embankment construction. As a result, the placement of a Cement Stabilized Sand (CSS) "wedge" in the zone behind the abutment is currently practiced by TxDOT. The placement of the CSS "wedge" in the zone behind the abutment is to solve the problems experienced while compacting the fill material right behind the abutment. This placement also provides a resistance to the moisture gain and loss of material, which are commonly experienced under approach slabs. The use of CSS has become standard practice in several Districts and has shown good results according to the TxDOT manual.

Apart from the embankment backfill material and construction specifications, the other alternatives, such as using flowable fills (low strength and flowable concrete mixes) as backfill around the abutment, wrapping layers of backfill material with geosynthetic or grouting have also been employed to solve the problem of the excessive settlements induced by the embankment. The use of these construction materials and new techniques increases construction costs inevitably. However, the increased costs can be balanced by the benefits obtained by less settlement problems. For example, the use of geosynthetic can prevent infiltration of backfill into the natural soil, resistance against lateral movements and improves the quality of the embankment (Burke, 1987). Other benefits are explained while describing these new methods in the following sections.

Mechanically Stabilized Earth (MSE) Wall

Mechanically Stabilized Earth (MSE) wall has been rapidly developed and widely used since the 1970s (Wahls, 1990). The MSE method is a mitigation technique that involves the mechanical stabilization of soil with the assistance of tied-back walls. As shown in Figure 38, a footing of the bridge is directly supported by backfill; therefore, a reinforcement system in the upper layer of the embankment where the backfill is most affected by the transferred load from the superstructure must be carefully designed (Wahls, 1990). On the contrary, the facing element of the wall does not have to be designed for the loading, since the transferred load from the bridge in the MSE scheme does not act on the MSE wall (Wahls, 1990).

Based on a study conducted by Lenke (2006), the results of research shows that the MSE walls tend to have lesser approach slab settlements than other types of bridge abutment systems due to these following reasons: first, the MSE walls will have excellent lateral constraints provided by the vertical wall system; second, the tie back straps in the MSE system can provide additional stability to the embankment. These two reasons can minimize lateral loads in the embankment beneath the abutment. Consequently, the potentials of lateral settlements are reduced (Dupont and Allen, 2002).

Other advantages of the use of MSE walls are that it reduces the time-dependent post construction foundation settlements of very soft clay as noted by White et al. (2005). Also, the MSE wall with the use of geosynthetic reinforced backfill and a compressible material between the abutment and the backfill can tolerate a larger recoverable cyclic movement as noted by Wahls (1990) and Horvath (1991).

Regarding construction aspects, the MSE walls have recently become a preferred practice in many state agencies (Wahls, 1990). First, the MSE is considerately an economical alternative to deep foundation or treatment of soft soil foundation. Second, the MSE can be constructed economically and quickly when compared to conventional slopes and reinforced concrete retaining walls. Third, a compacted density in the MSE construction can be achieved easily by increasing lateral constraint. Finally, the MSE is also practical to build in urban areas, where the right of way and work area are restricted (Wahls, 1990). Abu-Hejleh et al. (2006) cited that the use of an MSE wall for an abutment system should be considered as a viable alternative for all future bridges and it is reported as one of the practical embankment treatment systems to alleviate the bridge bump problem. An example of an MSE wall abutment is shown in Figure 38.

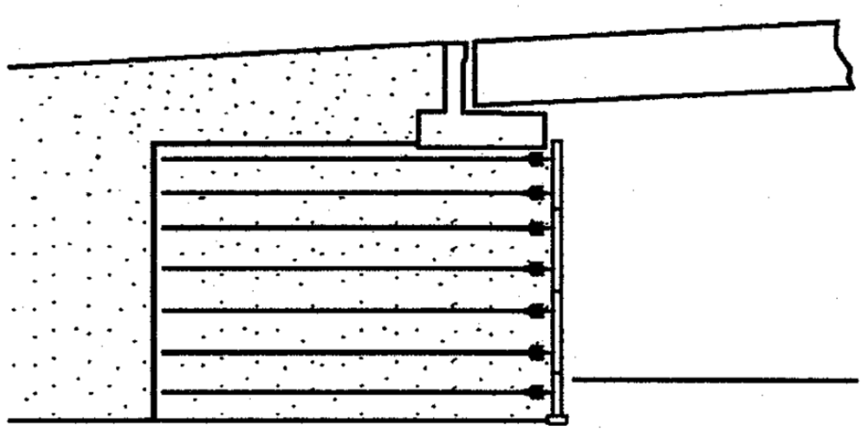


Figure 38 – Typical Mechanically Stabilized Abutment (Wahls, 1990)

Geosynthetic Reinforced Soils (GRS)

Geosynthetic Reinforced Soil (GRS) is recommended as a method to achieve a backfill compaction at the optimal moisture content, especially for a coarse-grained backfill material (Abu-Hejleh et al., 2006). The GRS is a geosynthetic-reinforced soil structure constructed either vertically or horizontally in order to minimize the uneven settlements between the bridge and its approach. Figure 39 shows a schematic diagram of a GRS wall structure and a complete typical GRS system after construction. Based on the studies performed by Abu-Hejleh et al. 2006, it was discovered that with the use of GRS, the monitored movements of the bridge structure were smaller than those anticipated in the design or allowed by performance requirements. In addition, they also stated that with the use of GRS systems, post construction movements can be reduced substantially, thus the bump problem at the bridge transition is minimized.

Another advantage of geosynthetic-reinforced soil is that it increases backfill load carrying capacity and reduces erosion of the backfill material; both can help in the mitigation of approach bumps. Some states have also used layers of geosynthetic-reinforcement soil in combination with shallow foundations to support the bridge abutment (Abu-Hejleh et al., 2000).

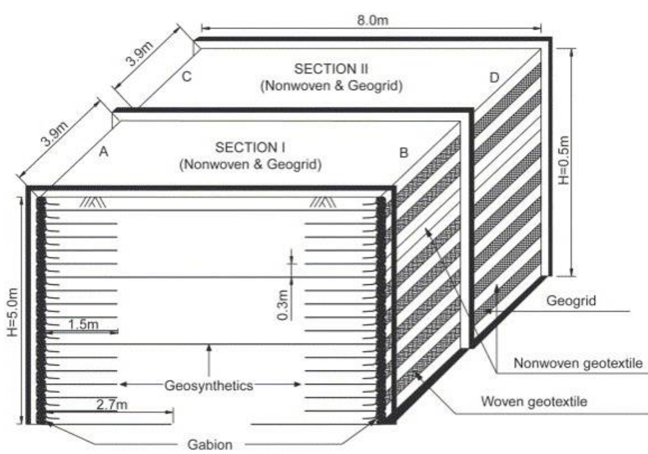


Figure 39 – Schematic Diagram of a GRS Wall and GRS System after Construction (Won and Kim, 2007)

According to Wu et al. (2003), the GRS system becomes a more viable alternative than other conventional bridge abutments. It provides many advantages, such as being more ductile, more flexible (hence more tolerant to differential settlement), more adaptable to the use of low quality backfill, easier to construct, more economical, and less over-excavation required.

Wu et al. (2003) also presented a case study where the GRS was used in a condition in which each footing bears several preloading cycles greater than their design load and sustained for several minutes. It was found that after the first few cycles of preloads, the observed settlement reduced to negligible amounts and subsequent service settlements were less than 0.5 inch. The Wyoming Highway Department has used multiple layers of geosynthetic reinforcement within compacted granular material since the 1980s (Monley and Wu, 1993).

Edgar et al. (1989) stated that none of the 90 approach slabs placed on geosynthetic reinforced embankments required maintenance or repair only after 5 years of service. Excellent performance of these systems was also reported by Abu-Hejleh et al. (2006) for both short- and long-term performance of the GRS approaches.

Wu et al. (2006) summarized the advantages of the GRS bridge abutments with flexible or rigid facing over conventional reinforced concrete abutments as follows:

- GRS abutment increases tolerance of foundation settlement to seismic loading;
- GRS abutments are remarkably more stable and have higher ductility;
- With a proper design and construction, "bumps" can be alleviated;
- GRS abutments are constructed more rapidly and less expensive;
- GRS abutments do not require embedment into the foundation soil for stability;
- The lateral earth pressure behind a GRS abutment wall is much smaller;
- GRS performs satisfactorily longer under in-service conditions; and
- The load-carrying capacity by GRS is significantly greater.

The GRS bridge-supporting structures can be grouped into two types: "rigid" facing and "flexible" facing structures (Wu et al., 2006). Flexibility or rigidity of GRS walls is explained in relation to its deformation capability and its responses to temperature changes during different seasons (Wu et al., 2006). If the construction is done in cold dry seasons (fall/winter), the GRS walls present a rigid response whereas constructions of GRS walls during warm, wetting, and thawing seasons result in GRS walls with a flexible response, capable of undergoing relatively large deformations.

Rigid facing is typically a continuous reinforced concrete panel, either precast or cast in-place. Rigid facing offers a significant degree of "global" bending resistance along the entire height of the facing panel, thus offering greater resistance to global flexural deformation caused by lateral earth pressure exerted on the facing. A typical cross section of a GRS system with rigid facing is shown in Figure 40.

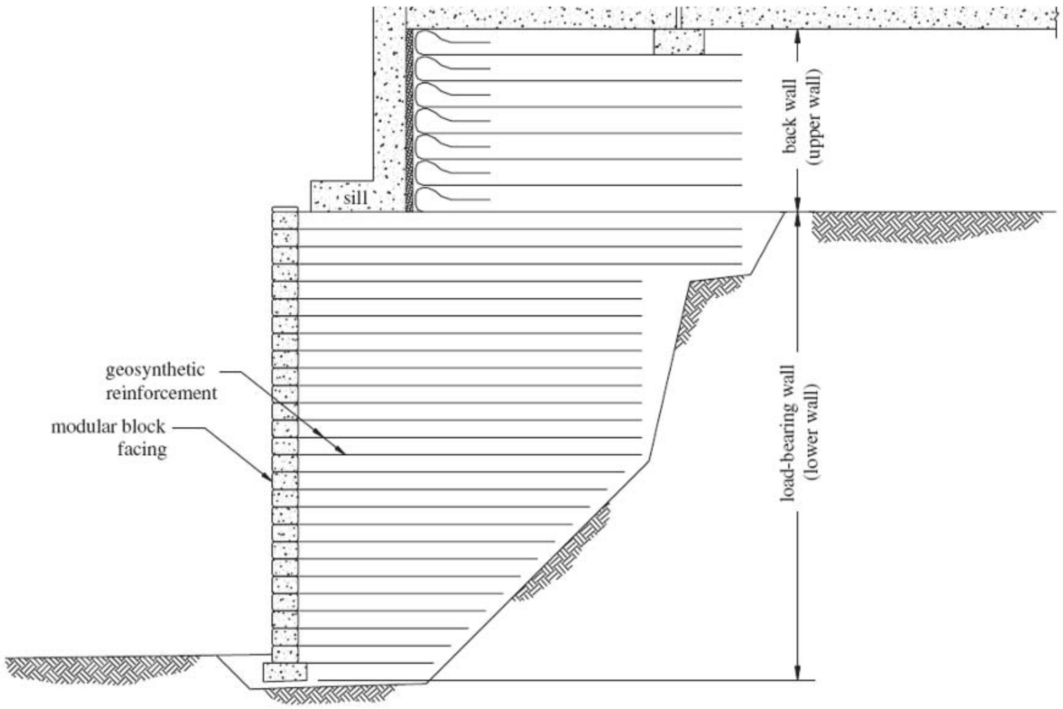


Figure 40 – Typical GRS Bridge Abutment with a Segmental Concrete Block Facing (Wu et al., 2006)

Flexible facing is typically a form of wrapped geosynthetic sheets, dry-stacked concrete modular blocks, timbers, natural rocks, or gabions. These wall structures have shown great promise in terms of ductility, flexibility, constructability, and costs. The main advantages of this system over the rigid facing are summarized in the following (Abu-Hejleh et al., 2003, Wu et al., 2006):

- Larger mobilization of the shear resistance of the backfill, thus taking more of the lateral earth pressure off the facing and connections;
- More flexible structure, hence more tolerant to differential settlement; and
- More adaptable to low-quality backfill.

Guidelines of GRS walls are provided by the Colorado DOT for designing and constructing GRS bridge abutments (Abu-Hejleh et al., 2000) and a few of the assumptions used in this guideline are presented here:

- The foundation soil should be firm enough to limit post construction settlement.
- The desired settlement of the bridge abutment should be less than 1 inch (25 mm).

- The maximum tension line needed in the internal stability analysis should be assumed nonlinear.
- Ideally construction should be done in the warm and dry season.
- The backfill behind the abutment wall should be placed before the girders.

Overall, the GRS system walls have been used with success to alleviate approach settlement problems. However, very few state DOTs have implemented this in practice, probably due to the limited amount of familiarity of this method.

Lightweight Fill

Another concept to reduce the vertical loading or stress from the embankment as it exerts itself on the foundation subsoil is the use of lightweight material as an embankment fill material. The reduction of embankment weight or load increases the stabilities of the embankment and also reduces the compression on the underlying foundation soil. As a result, the settlement potential of the embankment will be decreased.

The lightweight fills such as lightweight aggregate, expanded polystyrene, lightweight concrete, or others can be used to achieve this benefit (Luna et al., 2004; Dupont and Allen, 2002; Mahmood, 1990). Based on the surveys conducted by Hoppe (1999) approximately 27 percent of responding DOTs have already experimented with the use of non-soil materials behind bridge abutments.

Horvath (2000) recommended the use of geofoam as a light weight compressible fill material (Figure 41). Other materials could be used as alternative lightweight backfill material; some of these alternative construction materials included shredded tires and expanded polystyrene. However, it must be kept in mind that the suitable fill material must not have only the lightweight property, but it must have other required properties, such as, high strength, high stiffness, and low compressibility properties.

Hartlen (1985) listed some satisfactory requirements for the lightweight fill material as follows:

- Bulk density less than 63 pcf. (1000 kg/m^3);
- High modulus of elasticity and high angle of internal friction;
- Good stability and resistance against crushing and chemical deterioration;
- Non-frost active;

- Non-corrosive to concrete and steel; and
- Non-hazardous to the environment.

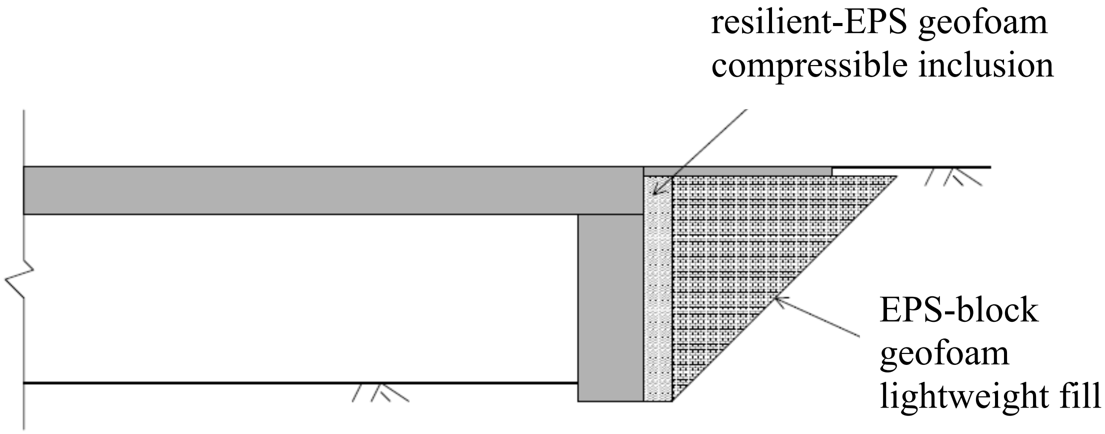


Figure 41 – A Design Alternative by Using Geofoam as a Backfill (Horvath, 2000)

Flowable Fill (Flowfill)

Flowable fill is a low-strength mixing concrete used as a backfill behind the abutment wall to reduce the possibility of approach settlements near the surface, resulting from the compression of the backfill itself (Abu-Hejleh et al., 2006). According to Folliard et al.(2008), the fluidity of flowable fill makes it a rapid and efficient backfilling material. The low-strength mixing concrete works well to prevent erosion of the backfill and to improve constructability/compactability of the fill behind the walls and around corners. The self-leveling ability property allows the flowable fill material to fill voids without the need of any compaction (Folliard et al.,2008). Although, this method is an expensive construction practice, it is still a practical alternative in certain field and construction scenarios where the use of such practice justifies the higher costs (Abu-Hejleh et al., 2006).

Snethen and Benson (1998) summarized that the use of flowable fill as an embankment material to reduce the potential for developing the bump at the end of the bridge seems to be a simple, reasonably cost effective, and less time-consuming method. This study also concluded that use of the flowable fill as an embankment material has resulted in the reduction of the lateral earth pressure and settlement of the approach embankment.

According to the Colorado DOT specifications, the maximum lift thickness for flowable fill material is 3 ft and a placement of additional layers is not permitted until the flowable fill has lost sufficient moisture to be walked on without indenting more than 2 inches. CDOT specifications do not specify any need for vibration because the vibration may stiffen the flowfill by allowing the setting to occur faster in the field. CDOT specifications for the flowfill backfill are listed in Table 5 (Abu-Hejleh et al., 2006).

Table 5 – CDOT Material Requirements for Flowable Fill Backfill

Ingredient	Lb/C.Y.
Cement	50
Water	325 (or as needed)
Coarse Aggregate (AASHTO No.57 OR 67)	1700
Fine Aggregate (AASHTO M6)	1845

In a separate section, the use of flowable fills for remediation of approach slab settlements will be discussed.

Grouting

Edgar et al. (1989) reported that in a high-speed passageway, ground stabilization methods could be utilized to reduce maintenance requirements. In this study, the use of cement-treated backfill instead of conventional granular backfill material was chosen to reduce the hydro-collapse and increase soil strength. The grouting technique has been also recommended for mitigation of settlement of the embankment in the case of embankments underlain by organic peat layers, which can be easily compressed and consolidated (Byle 1997; 2000). It was found that the pressure grouting method was also successful in preventing the loss of materials. However, the main objective of the grouting technique is to restrict the limited mobility displacement (LMD) of the material, as described by Byle (1997; 2000).

Figure 42 shows that the sleeve pipes can be installed in different angles of 50°, 30°, and 20° from the horizontal surface (Sluz et al., 2003). The angle at which the sleeve port pipes installed in the soil is important and must be modified by monitoring the amount and the rate of settlement. Details including the settlement after mitigation and the type of grout used were not listed in the report.

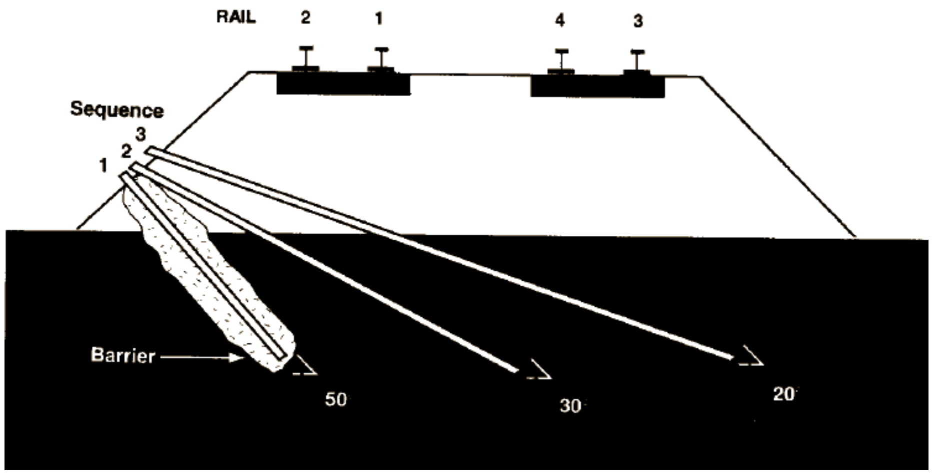


Figure 42 – Sleeve Port Pipe Installation Plan (Sluz et al., 2003)

Other Recommendations

If possible, the slopes of soil embankments should be flattened, which tends to increase the stability and reduce the deformations of the embankment (Luna et al., 2004). Such practice will not be applicable due to high use of ROWs along the embankment sections. If the proposed embankment material is plastic clay with PI greater than 15-20, treatment of the soil or alternate borrow sources should be considered. The select fill also needs to be extended to a certain distance from the abutment, and the distance ranges between 50 and 100 ft and is dependent on the type of embankment and material used as a backfill (Luna et al., 2004).

3.7 Design of Bridge Foundation Systems

The bridge foundation is considered as a major factor in bridge structure design. Bridges can be supported either by shallow or deep foundation systems (Wahls, 1990). In both cases, the foundations should be able to carry the loads from the above superstructures and the traffic volumes, but also to limit the horizontal and vertical movement of the abutment to the acceptable levels (Wahls, 1990). The selection of a safe and economical foundation system requires consideration of structural loads, environmental factors, subsurface conditions, bed rock types and depths, performance criteria, construction methods, and economics (ODOT, 2005).

Spread footings, driven piles, and drilled shafts are generally used as a bridge foundation. According to Wahls (1990), the spread footing has its advantage over the deep foundation in aspect of inexpensive cost. However, the uncertainties in the performance prediction and the potential for scouring make the shallow foundation an unattractive choice for a bridge foundation system. Moreover, since the compaction of backfill near the abutment is difficult to achieve, the possibilities of loads from superstructure and traffic volume stressing the poorly compacted backfill and contributing to the settlement of bridge approaches can be high (Wahls, 1990).

For those reasons the deep foundations including driven piles or drilled shafts are preferred to support the bridges. The deep pile foundations have been demonstrated to be the most efficient means of transferring heavy loads from superstructures to substructures and bearing materials without significant distress from excessive settlement (Abu-Hejleh et al., 2006). Hopkins (1985) cited that the settlement of the bridge abutment resting on pile foundations is usually negligible. However, due to the fact that the bridges supported by pile foundations do not usually settle as much as the approach embankments, the differential settlement between these two adjacent structures can lead to the bump problems at the bridge approach. Hopkins and Deen (1970) stated that the differential settlement between the abutment and the approach slab is usually high for pile support abutments.

The abutment with embedded pile caps can develop resistance to the movement of the bridge structures as the bridge superstructure expands and contracts with temperature variations, which is also claimed as a cause of high applied stresses on the pile foundations and a reduction of pile axial load capacity (Greimann et al., 1983). Greimann et al. (1986) performed a three dimensional non-linear finite element analysis to study pile stresses and pile soil structure interaction of integral abutment bridges from thermal fluctuations. They found that the thermal expansion of the bridge reduces the vertical load carrying capacity of the piles. They reported that the vertical load carrying capacity for H piles in very stiff clays is reduced by approximately 50 percent for 2 inches of lateral displacement and approximately 20 percent for 1 inch lateral displacement.

Girton et al. (1991) measured the maximum of pile stress at the Boone Bridge and the Maple River Bridge. They found that the maximum pile stresses were only 60 percent and 70 percent of the nominal yield stress at both sites, respectively. Lawver at al. (2000) reported that the maximum measured pile stresses were slightly above the nominal yield stress of the pile. Arsoy et al. (2002) investigated the performance of H-piles, pipe piles, and pre-stressed reinforced concrete piles subjected to cyclic lateral displacements. Based on that study, it was concluded that H-piles loaded on the weak axis were the best alternative to support the integral abutments. An example of bridge foundation construction using H-piles is illustrated in Figure 43.

Figure 43 – Example of Bridge Foundation Using Steel H-Piles

The use of precast, pre-stressed concrete (PC) piles in the foundation of bridge piers has been used as a valuable alternative for bridge construction for a long time (Abendroth et al., 2007). However, due to some concerns over pile flexibility at the abutment ends, potential for concrete cracking induced by thermal expansions and seismic movements, and deterioration of the pre-stressing strands due to long-term exposure to moisture, these PC piles in the integral abutment bridges have not been extensively used (Abendroth et al., 2007).

According to a survey conducted in several states by Abendroth et al. (2007), the main reasons to avoid PC piles for bridge abutments are attributed to inadequate ductility (48 percent), insufficient research on the subject matter (52 percent), limit availability (33 percent), and high cost of the foundations (24 percent). In the last 10 years, the potential use of PC piles for integral abutments was reported in a few studies (Kamel et al., 1996; PCI, 2001; Burdette et al., 2004). However, the available literature presents different conclusions regarding the suitability of PC piles for this application (Abendroth et al., 2007).

The precast, pre-stressed concrete piles typically utilize both skin friction and end bearing conditions to carry the vertical loads. The results from the study by Abendroth et al. (2007) showed that with respect to construction costs the usage of PC piles is more economical than the H-steel pipes in sandy and gravelly soils. Moreover, the study also showed that these precast, pre-stressed concrete piles usually experience less lateral displacement than the H-piles and lower longitudinal movements than the expected range. However, the study also evidenced pile cracking problems after excavation on the abutments. The cracking problems are attributed to moisture penetration, uncoated pre-stressing strands, and long-term corrosion problems (Abendroth et al., 2007). For these reasons, periodic inspection of the abutment piles is recommended to detect any additional concrete cracking or deterioration.

When piles are selected for a bridge foundation system, the ability of the foundation piles to carry the vertical loads even when the piles are subjected to temperature-induced displacements must be considered (Arsoy, 1999). The lateral displacements may reduce vertical-load carrying capacities of piles, resulting in pile failure if lateral loads are higher than the elastic buckling load (Greimann and Wolde-Tinsae, 1988). Another important factor is the length of the pile, because it controls the allowable settlement of the structure. Bakeer et al. (2005) indicated that due to loading requirements and to minimize settlement, bridge piers and abutments needed to be supported on relatively long piles or piles with tips driven into stiff soil.

One negative effect that needs to be taken into account for the design of pile foundations is the consideration of negative skin friction or down-drag from compressible soils around the pile lengths. Figure 44 presents a schematic of the process that produces down-drag forces on piles. Down-drag is the sum of the negative shaft resistance along the length of the pile where the soil is moving downward relative to the pile, and this drag is always treated as a downward acting load (AASHTO, 2004). Some of the successful methods to mitigate down-drag are listed below (Narsavage, 2007):

- Use larger H-pile sections to increase factored structural resistance for piles on rock.
- Use more piles and reduce the applied load for piles not driven to refusal on rock.
- Reduce soil settlement that occurs after pile driving by preloading and/or using wick drains.
- Reduce soil settlement by using lightweight embankment fill material.
- Use bituminous pile coating.

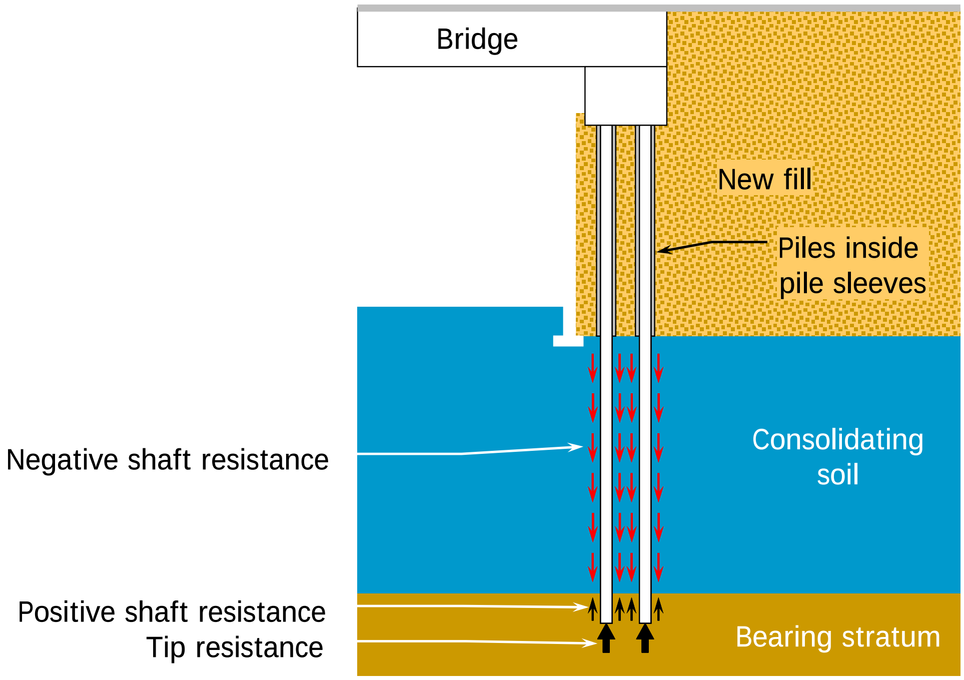


Figure 44 – The Down-Drag in Piles (Narsavage, 2007)

In order to avoid the downward drag problems, the use of shallow foundations has been suggested (DiMillio, 1982). Generally, the shallow foundations are typically 50 percent to 60 percent less expensive and require less construction time than deep foundations (DiMillio, 1982). Some recent studies have demonstrated again the feasibility of implementing shallow foundations for major bridges in the United States (Abu-Hejleh et al., 2003). For example, the Founders/Meadows bridge foundation was built on footings supported directly by a geosynthetic-reinforced soil system, eliminating the use of traditional deep foundations (piles and caissons) altogether. A typical section of the GRS system of this bridge foundation is detailed in Figure 45.

However, the shallow foundations have their own disadvantages. In a study by Grover (1978), he compared the behavior of bridges supported by shallow and pile foundations in Ohio. The result of this study indicated that for the bridge constructed in 1960s, 80 percent of the abutments supported by shallow spread footings experienced more than 2.5 inches of settlement and 10 percent of them experienced more than 4 inches of settlement. As a result, Ohio DOT specifications asked for deep pile supported bridge abutments in the place of shallow foundation supported abutments (Grover, 1978).

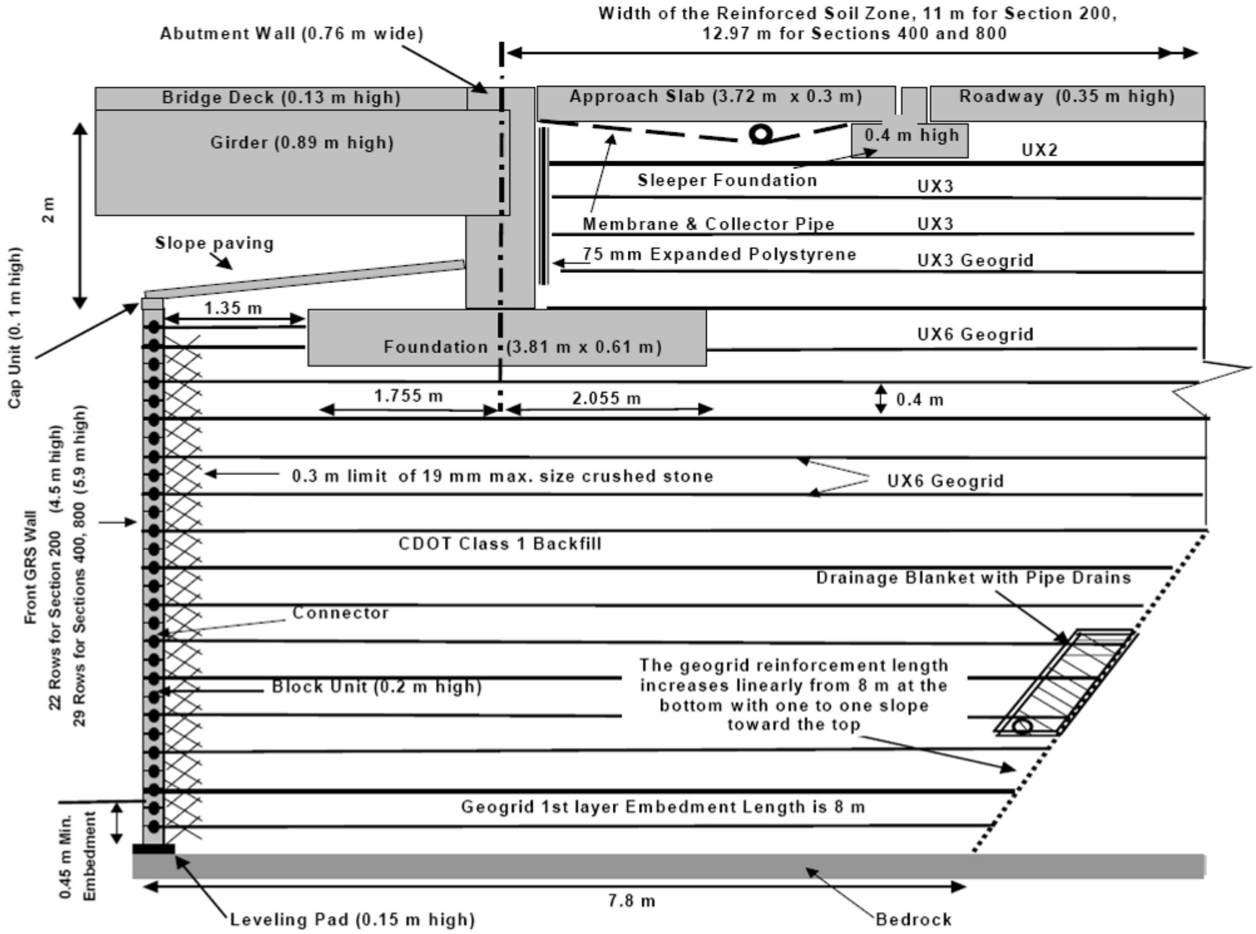


Figure 45 – Typical Section through Front and Abutment GRS Walls

(Abu-Hejleh et al., 2006)

According to the TXDOT Bridge Design Manual (2001), the spread footing was only an alternative used as bridge foundations in Texas in early bridge design, although other options, such as timber and concrete piles were already available. Since the late 1930s, the steel H-pile was introduced and then became widely used and a few caissons, pneumatic and open, were used for larger stream crossings. A drilled shaft technology, which was developed in the late 1940s, and pre-stressed concrete pile foundations have now become a dominate foundation in bridge construction in Texas.

Design of Bridge Abutments

The type of the bridge abutment plays an important role (Mahmood, 1990). Generally, two types of abutments are used widely in the United States, a non-integral (or conventional) and an integral type (Greimann et al., 1987).

The non-integral or conventional type of bridge abutments (Figure 46) have bearing connections and expansion joints to provide the superstructures with a certain amount of lateral movement between the abutment and the bridge deck (Wahls, 1990) The lateral load caused by the lateral movement or the thermal strains in the deck will be lessened by both types of connections (White et al., 2005). However, increased traffic loads and frequent application of de-icing salts during winter could deteriorate the expansion joints and bearing connections, which can lead to costly maintenance problems (Horvath, 2000). These non-integral abutments are commonly used in many states including Texas.

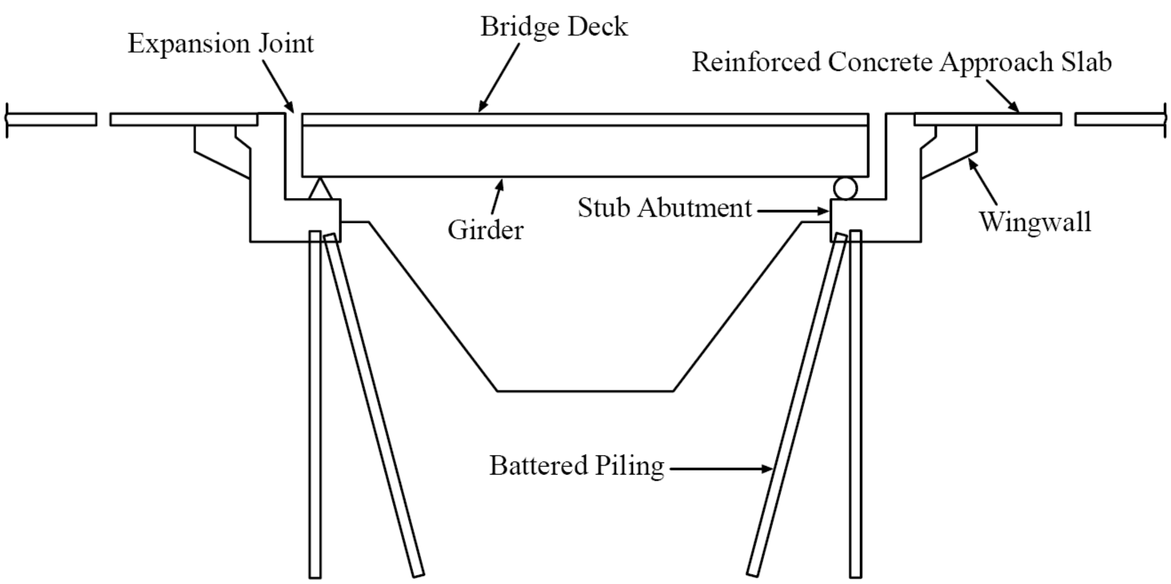


Figure 46 – Simplified Cross Section of Non-Integral Abutment Bridge (Greimann et al., 1987; White et al., 2005)

The integral bridge abutment type (Figure 47) was developed in order to eliminate the use of bearing plates and to reduce potential maintenance problems (Horvath, 2000). The integral abutment is a stub abutment connected to the bridge superstructure tightly without any expansion joints (Wahls, 1990). The rigid connections are conventionally included thermal stresses from the bridge deck to the abutment in their design criteria (Wahls, 1990). The advantages of this rigid connection are (Greimman et al., 1987; Hoppe and Gomez, 1996);

- simple and reduced construction and maintenance costs,
- minimum number of piles required to support the foundation, and
- improved seismic stability.

The use of integral bridge abutments has been increased since the 1960s, because it avoids the use of the bearing plates and the potential maintenance problems associated with non-integral bridge abutments (Wahls, 1990; Horvath, 2000; Kunin and Alampalli, 2000).

Pierce et al. (2001) stated that the bridge approaches with integral abutments tend to reduce the surface roughness. However, Wahls (1990) reported a problem related to cracking and bulking at the approach pavement due to a lateral cyclic movement of the abutment from thermal movement induced stresses at the bridge decks. Schaefer and Koch (1992) and Arsoy et al. (1999) also specified that the same lateral cyclic movements exerted on the backfill soils from daily temperature changes may form voids at the face of the abutment, which contribute to the total approach settlement. The voids are observed within one year of bridge construction, indicating insufficient backfill moisture control/compaction followed by soil collapse upon saturation (White et al., 2005).

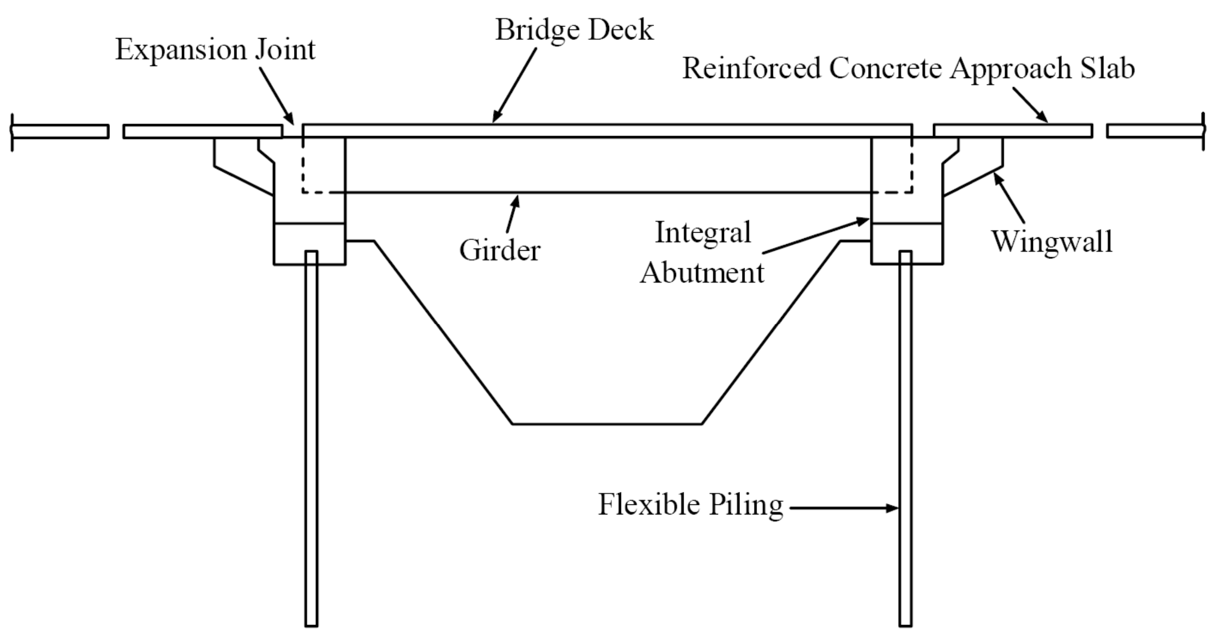


Figure 47 – Simplified Cross Section of Integral Abutment Bridge
(Greimann et al., 1987; White et al., 2005)

Lateral movement is a common occurrence of the integral bridges (Kunin and Alampalli, 2000; Arsoy et al., 2002; Arockiasamy et al., 2004). The bridge superstructures will be expanded and contracted by seasonal air temperature fluctuations according to concrete thermal strain characteristics. Because the bridge deck and abutment are integrally connected, both structures will laterally move together. The movement of the structures resulting from the temperature of the bridge deck seasonal changes can cause a cyclic loading subjected toward the approach backfill and the foundation. When the temperature rises, the bridge deck expands and then the superstructure including the bridge abutment moves against the retained embankment soil. The lateral movement induces the stress in the soil and sometimes can reach the passive pressure limit (Schaefer and Koch, 1992). On the other hand, when the temperature lowers, the superstructure and the abutment move away from the soil and leave voids at the interface between the abutment and the backfill. The size of the voids can become bigger if the weather gets colder. The development of the voids can be a cause of soil erosion that increases the size of the void behind the abutment and below the approach slab as shown in Figure 48.

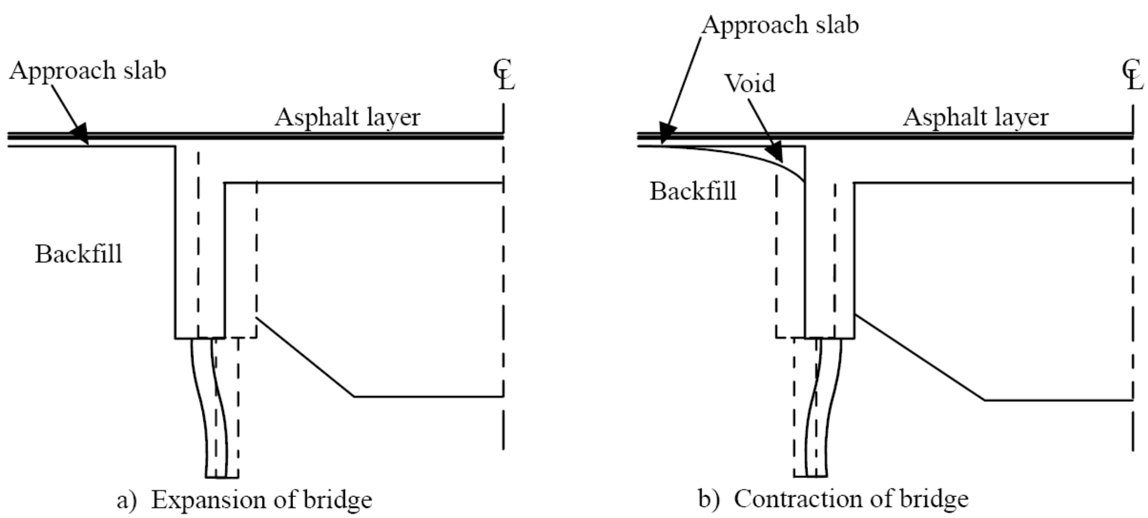


Figure 48 – Movement of Bridge Structure with Temperature (Arsoy et al., 1999)

Wahls (1990) suggested that the performance of an integral abutment can be improved by installing compressible elastic materials between the abutment and the backfill. The material should have elastic properties that permit large recoverable cyclic movements and hydraulic properties that provide adequate drainage without erosion of fines from the backfill. Horvath (2000) advocated the use of geofoam as a compressible material. The successful use of compressible and collapsible materials behind the abutment was reported by the North Dakota DOT and Illinois DOT (Wahls, 1990; Kunin and Alampalli, 2000).

According to Mekkawy et al. (2005) and White et al. (2005), insufficient drainage is also another problem often found at the bridge abutments. Water that is collected on the bridge pavement can cause severe damage to the bridge approach. If collected water can flow into the underlying fill materials due to inefficient seals at the joints between the bridge approach slab and the abutments, the water can erode the backfill material, resulting in voids development under the bridge abutments. Therefore, an efficient drainage system should be incorporated in the design of bridge approaches, such as drainage inlets at the end of a bridge deck to collect surface water before getting to the approach slab (Abu-Hejleh et al., 2006).

Furthermore, providing additional surface or internal drainage to keep water off the slopes is recommended for correcting the superficial erosion of embankments (Wahls, 1990). Keeping the water away from the soil is a simple significant factor in reducing the settlement of the soil. Construction costs added to incorporate a good drainage system are not high when compared to the expensive maintenance costs that might be experienced in the service life of a bridge (Dupont and Allen, 2002).

3.8 Design of Approach Slab

The bridge approach slab is a part of a bridge that rests on the abutment at one end and on the embankment or a sleeper slab on the other end (Wahls, 1990). The slabs are designed to provide a smooth transition between the bridge deck and the roadway pavement and to minimize the effect of differential settlements between the bridge abutment and the embankment fill (White et al., 2005). There are two types of approach types used by highway agencies. Some agencies use a bituminous approach pavement because it can be maintained easily by overlay type rehabilitation. However, the use of bituminous approaches with Portland concrete roadways is still not highly preferred by the DOTs (Wahls, 1990).

Other agencies use a reinforced concrete slab, because they believe the rigid approach slab is successful in preventing the bridge approach settlement (Wahls, 1990). In this case, one end of the slab is connected to the main structure by two ways. In the first alternative (Figure 49), the slab is connected directly to the bridge deck by extending the main reinforcement from the bridge deck to the approach slab; while in the second alternative (Figure 50), the approach slab is connected to the abutment by using a dowel/tie bar (White et al., 2005).

Based on a survey on over 131 bridges in Texas by James et al. (1991), they found that the bridges with flexible pavement had a smoother transition than those with rigid pavement. However, Pierce at al. (2001) reported that the approach slab with asphalt overlays tend to increase surface roughness. According to the TxDOT Bridge Manual (2001), the use of approach slabs is only an option, and Districts have had success with and without their use. However, if the approach slab is constructed with the non-integral bridge system, the use of a dowel/tie bar must be implemented between the slab and the abutment (Hoppe, 1999).

James et al. (1991) stated that the roughness or IRI values of the approach slab are influenced by the longitudinal pavement movements resulting from temperature cycles. They also mentioned that the approach pavement settlement/roughness can be attributed to impact loads due to poor design and constructed expansion joints.

White et al. (2005) stated that the performance of the approach slabs depends on these following factors: approach slab dimensions, steel reinforcement, use of a sleeper slab, and type of connection between the approach slab and the bridge.

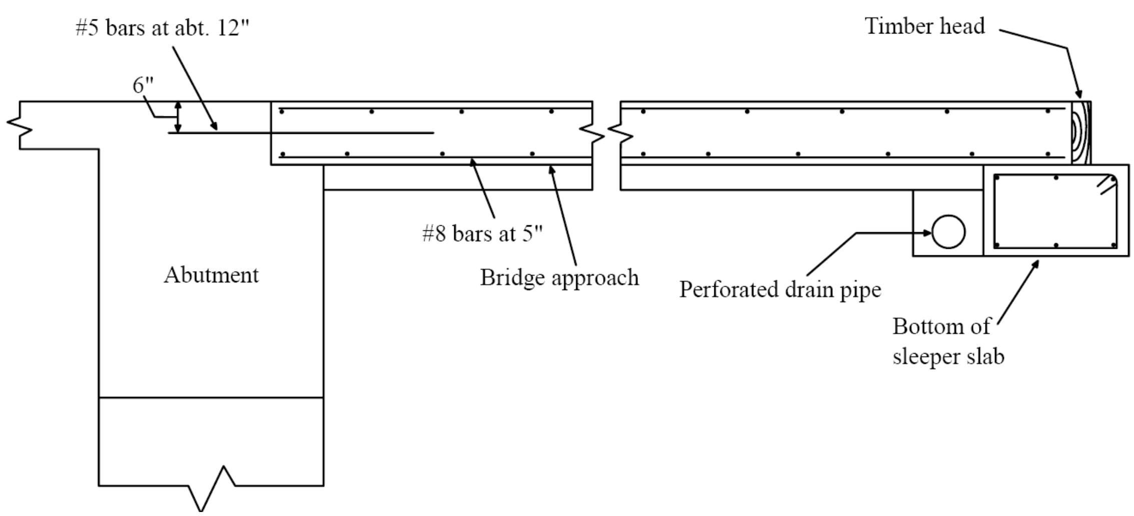


Figure 49 – Bridge Approach Connected to Bridge Deck (Missouri DOT, 2003)

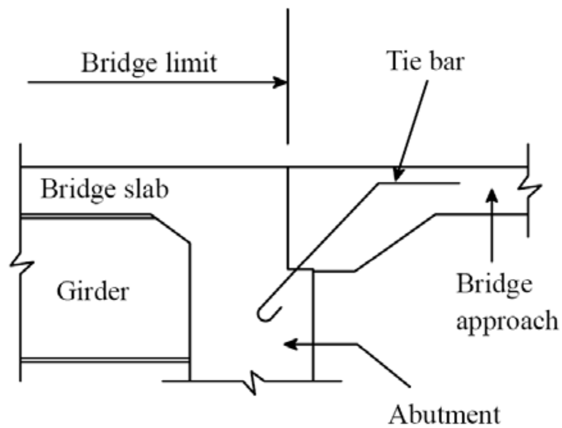


Figure 50 – Bridge Approach Connected to Abutment (Ohio DOT, 2003)

Slab Dimensions

Most of the reinforced concrete approach slabs used in the U.S.A. have lengths varying from 20 to 40 ft (6 to 16 m) (Wahls, 1990). According to an extensive survey conducted by Hoppe (1999) of different state agencies, typical approach slab dimensions for the various states surveyed are collected and summarized in Table 6. From the table, it can be seen that most approach slab dimensions vary between 15 - 30 ft (5 - 10 m) in length and 9 - 17 inch (23 - 43 cm) in thickness.

Some states consider the use of a short span slab and this is attributed to causing the bump problem (Lenke, 2006). As a result, some of these states move towards the use of a slab longer than 40 ft (16 m) (LaDOTD, 2002). For example, the Illinois DOT prefers the design of a slab length of 100 ft (30 m), and the Louisiana DOT uses continuous slab lengths from 80 to 120 ft (24 to 36 m) (Wahls, 1990). In both cases, the bridge abutments are pile supported.

Other research summary studies by Briaud et al. (1997), Abu-Hejleh et al. (2006), and Lenke (2006) suggest a criterion to calculate the slab length based on the maximum slope of the approach slab, which is defined as the change in elevation between the beginning of the approach slab (at the sleeper slab) and the bridge abutment divided by the length of the approach slab. The slope of the approach in their studies is defined as:

$$S = \frac{s_f - s_a}{L} \leq \frac{1}{200}$$

where S is the longitudinal slope of the approach slab, L is the length of the approach slab, and s_f and s_a are the settlements of the foundation (embankment and natural soil foundation) and the abutment, respectively. For example, if a settlement analysis indicates a differential settlement

Table 6 – Typical Approach Slab Dimensions Used by Various DOTs (Hoppe, 1999)

State	Length (ft)	Thickness (in)	Width limited to
AL	20	9	Pavement
AZ	15	N/A	N/A
CA	10-30	12	Curb-to-Curb
DE	18-30	N/A	N/A
FL	20	12	Curb-to-Curb
GA	20-30	10	Curb-to-Curb
IA	20	10-12	Pavement
ID	20	12	Length
IL	30	15	Curb-to-Curb
IN	20.5	N/A	N/A
KS	13	10	Curb-to-Curb
KY	25	N/A	Curb-to-Curb
LA	40	16	Curb-to-Curb
ME	15	8	Curb-to-Curb
MA	N/A	10	N/A
MN	20	12	Pavement
MS	20	N/A	Curb-to-Curb
MO	25	12	N/A
NV	24	12	Curb-to-Curb
NH	20	15	N/A
NJ	25	18	N/A
NM	15	N/A	Curb-to-Curb
NY	10-25	12	Curb-to-Curb
ND	20	14	Curb-to-Curb
OH	15-30	12-17	N/A
OK	30	13	Curb-to-Curb
OR	20-30	12-14	Curb-to-Curb
and	Skew	angle	N/A
SD	20	9	N/A
TX	20	10	N/A
VT	20	N/A	N/A
VA	20-28	15	Pavement
WA	25	13	Pavement
WI	21	12	N/A
WY	25	13	Curb-to-Curb

***N/A: Information is not available or not applicable between the abutment and the beginning of the approach slab ($s_f - s_a$) equal to 1.5 in., then the length of the approach slab must be greater than 300 in., or 25 ft. From the equation it can be easily understood that when the same settlement happens at both ends of the slab, then a shorter approach slab will be needed.

One way of minimizing "the bump" is to lengthen the approach slab (Lenke, 2006). Seo (2003) suggested that the approach slabs should have a minimum length of 20 ft and should be designed to support full traffic loading in a free span to account for any unexpected erosion beneath the slab.

Other aspects of approach slabs that are of interest to designers are the acceptable degree of the longitudinal slope. Several research reports recommended a maximum allowable change related slope of 1/200 (Wahls, 1990; Stark et al. 1995; Briaud et al. 1997; Seo, 2003). Long et al. (1998) also proposed a relative gradient of less than 1/200 to ensure rider comfort and a gradient of between 1/100 and 1/125 as a criterion for initiating remedial measures.

Wong and Small (1994) suggested that the slab with an angle can lessen the bump problem. They studied the effect of orientation of approach slabs on pavement deformation by varying the slopes of the approach slab at 0, 5, and 10 degrees with the horizontal and compared those results with no slab tests in a one-fourth scaled model as shown in the test set-up in Figure 51. They concluded that the horizontal slab contributes little to remedy the bump problem. On the contrary, the slab with an angle sloped down beneath the pavement can alleviate the bump problem better than a horizontal one due to the fact that the deformations at the surface at the pavement above the slab are more gradual and the rate of change of the surface gradient is small.

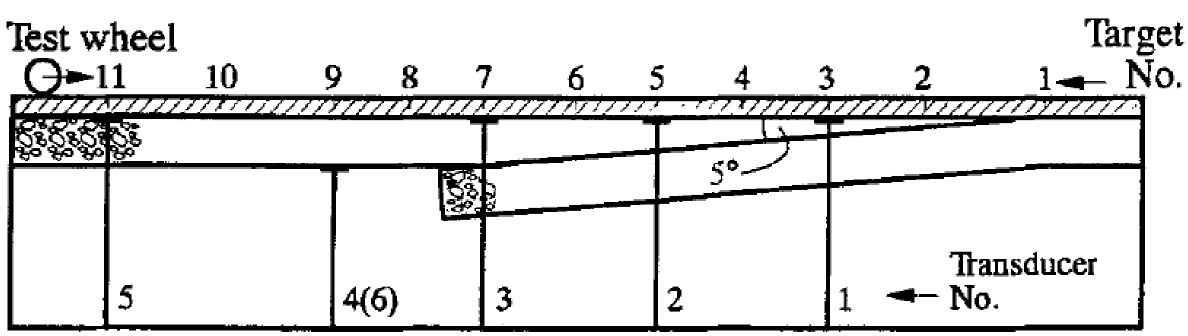


Figure 51 – Test Setup for Subsoil Deformation (Wong and Small, 1994)

The slab thickness is another factor that needs to be considered for a slab design. Normally, the thickness of the rigid approach slab is uniform. Nassif (2002) conducted a numerical analysis on New Jersey's approach slabs. They concluded that the slab thickness is the most effective parameter in reducing the tensile stresses in the critical elements. From the same study, Nassif (2002) also suggested a constant thickness of the approach slab and embedded beam design to the New Jersey DOT for their use. Overall, the slab thickness can vary depending on the considerations of the length of the slab, other structures, and the foundation (Lenke, 2006). The thickness of the slab can be designed as a taper shape in different sections in order to provide more flexibility in areas near the abutment (Wahls, 1990).

Regarding the type of slabs, Cai et al. (2005) studied different types of approach slabs by performing 3D finite element analyses. They recommended the use of a ribbed slab type, as seen in Figure 52, over the flat slab type, especially for long approach spans. Since, the internal forces and deformation of the ribbed slab can be lessened due to its slab-on-beam behavior, the thickness of bridge decks or slabs can be reduced when compared with the flat slabs.

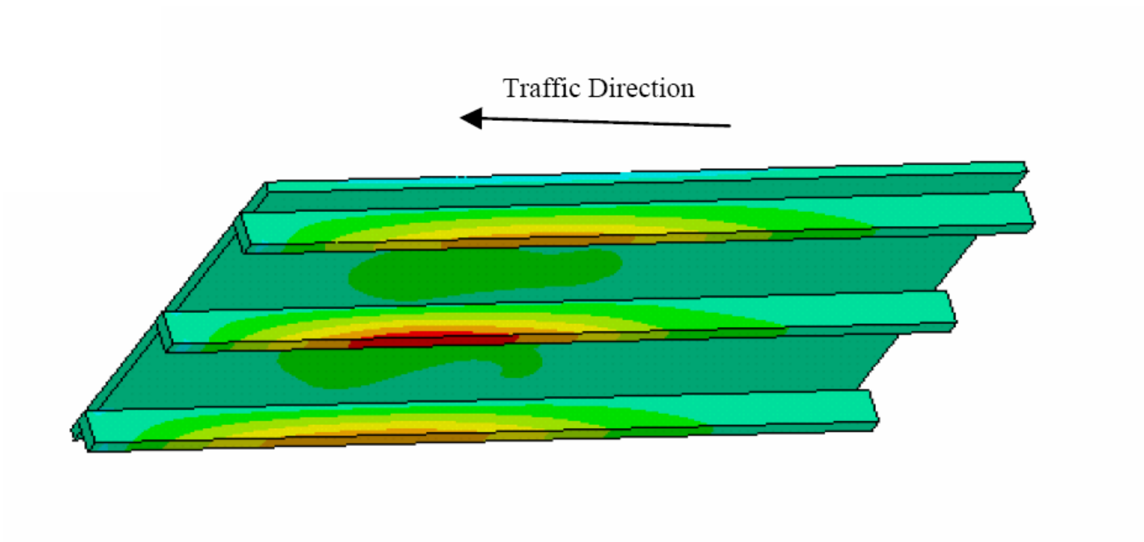


Figure 52 – The Ribbed Slab as an Approach Slab (Cai et al., 2005)

In terms of the width of the approach slab, the curb-to-curb method is the preferred (Briaud et al., 1997). By matching the width of the slab with the width of the bridge decking (between bridge guardrails or barriers), a few advantages can be realized. These are better erosion control of the underlying embankment soils and effective drainage pouring water away from the bridge structure and approach slab system (White et al., 2005). Since these two factors contribute to the bump problem, use of such widths of approach slabs are often recommended (Briaud et al., 1997; White et al., 2005).

According to the TxDOT Bridge Manual (2001), the use of an approach slab is optional. However, when the use of an approach slab is utilized, the approach slab should have a thickness of 13 in. The slab must also be a lightly reinforced concrete slab, which precedes the abutment at the beginning of the bridge, and follows the abutment at the other end of the bridge.

This manual also cites that TxDOT discourages the use of approach slabs on wingwalls based on previous experience in Texas. Due to the difficulty in compaction of the backfill, and the potential loss of backfill material, the approach slab becomes a slab supported on three sides (i.e., at the two wingwalls and the abutment backwall). Without the bearing on the backfill, it leads to the development of a void underneath the slab and consequently leads to bumps. For that reason, the standard approach slab and the wingwalls designed to carry out the load are not reinforced. Hence, TxDOT suggests that the approach slab should be supported by the abutment backwall and the approach backfill only.

The appropriate backfill material is considered an essential component under the slab. TxDOT is currently supporting the placement of a cement stabilized sand (CSS) "wedge" in the zone behind the abutment (TxDOT Bridge Manual, 2001). The use of CSS can solve the problem of difficult compaction behind the abutment. Furthermore, CSS wedges are resistant to the moisture gain and loss of material, which are common occurring under approach slabs. The use of CSS has become standard practice in several Districts and has shown good results (TxDOT Bridge Manual, 2001). The Fort Worth District in TxDOT uses a cement treated flexible base beneath the approach slab for the same purpose. The 3 ft. deep flex base is prepared by compacting four equal layers (9 in. thick) of Type 1 cement treated (2.4 percent by weight) base material as shown in Figure 53. However, approach slabs with the cement treated flexible base have also experienced the same settlement problems since the heavier flexible base has further consolidated the embankment fill and thus creating a larger "bump" (Williammee, 2008).

An additional component to the approach slab, which is not widely applied, is the use of a sleeper slab. A sleeper slab is a concrete foundation slab placed transversally at the approach slab and opposite to the bridge end (Ha et al., 2002; Seo et al., 2002). Generally, one end of the rigid approach slab rests on the abutment or connects directly with the bridge deck, while the other end sits directly on the embankment or otherwise on a sleeper slab (Wahls, 1990). An example of a sleeper slab for an integral abutment system is illustrated in Figure 54. The sleeper slab is a hidden slab placed under both the approach slab and the roadway pavement.

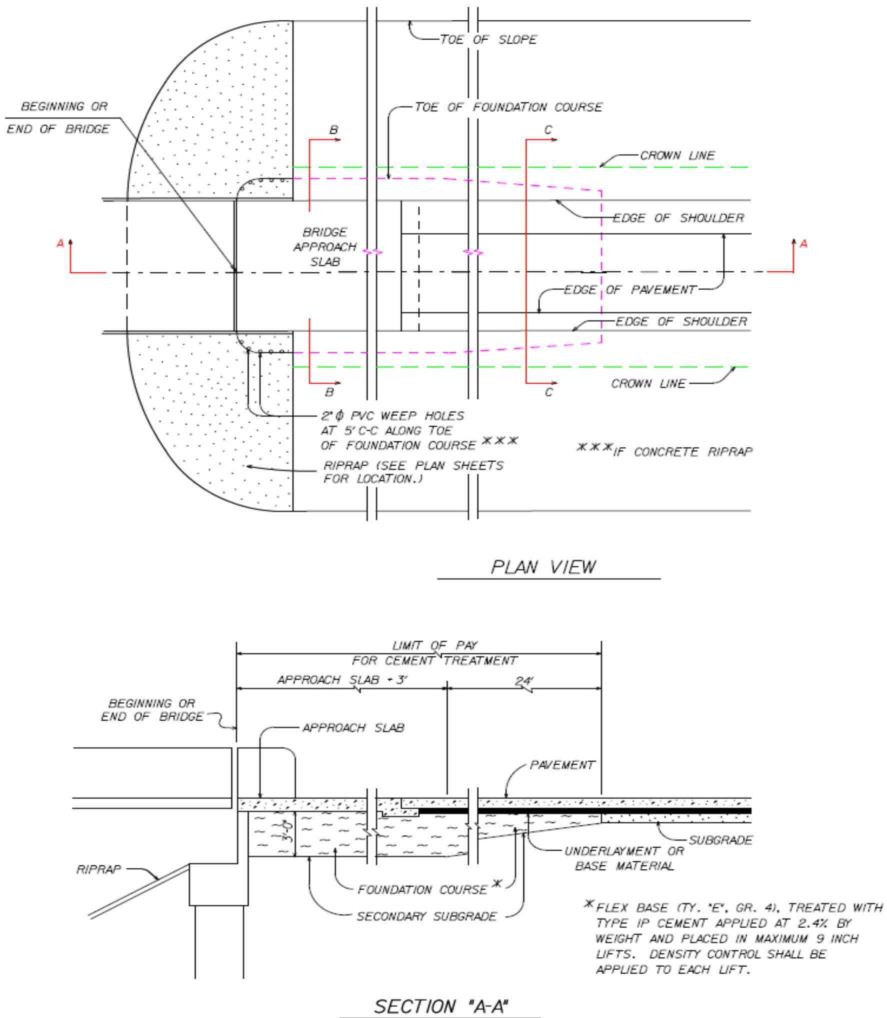


Figure 53 – Schematic of Bridge Approach Slab Arrangement Adopted by the Fort Worth District of TxDOT, Texas

Dupont and Allen (2002) conducted a survey on the 50 state highways agencies in the U.S.A. Their study shows from 48 states agencies, which use approach slabs that only 31 states use sleeper slabs. Of the 31 states, 14 states said the sleeper was effective, 2 states said it was not, while 15 states were not sure.

The design purpose of the sleeper slab is to minimize the possibility of the differential settlement by allowing the approach slab to settle with the embankment, thus preventing the bump at the bridge (Dupont and Allen, 2002). However, the improper design of the sleeper slab geometry may lead to settlement problems as well (Lenke, 2006). In addition, when expansion joints are placed on top of the sleeper slabs, cracking and crushing of the approach slab concrete may occur due to the closure of the expansion joints and dragging of the approach slab (Abu-Hejleh et al., 2006).

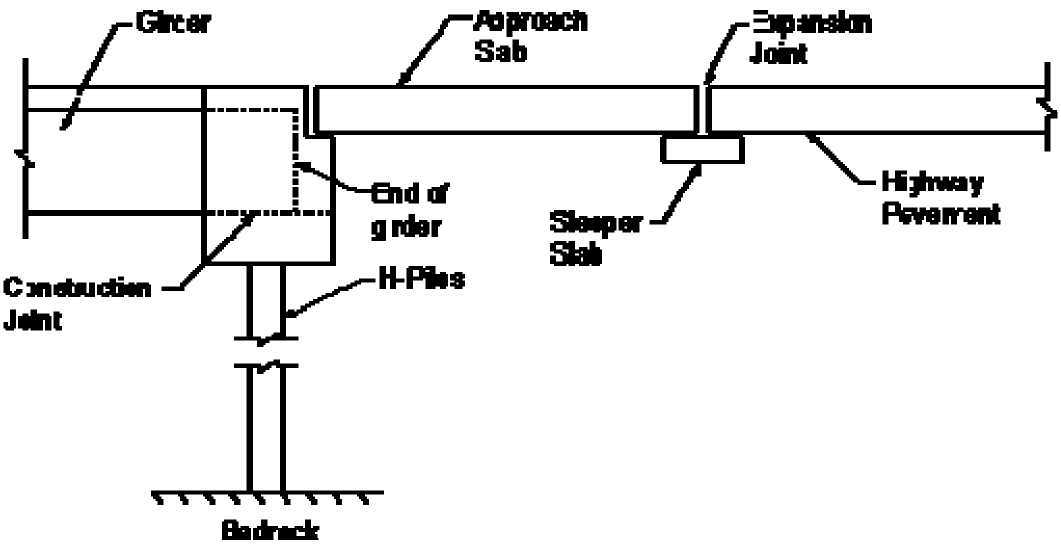


Figure 54 – Schematic of an Integral Abutment System with a Sleeper Slab (Seo et al., 2002)

The minimum recommended length of the sleeper slab is 1.5 m. (Seo, 2003). The width of the sleeper slab supporting the approach end of the approach slab should be 5 ft to prevent the bearing failure within the backfill material under the slab (Seo et al., 2002; Lenke, 2006), while other researchers suggest the use of widths of 3 to 4 ft (Cai et al., 2005; Abu-Hejleh et al., 2006).

Some studies have reported 16 in. thickness sleeper slabs to prevent settlement or the creation of voids beneath the slab (Abu-Hejleh et al., 2000), while other studies recommend the use of thickness of 20 in. (Luna, 2004). Other design considerations of sleeper slabs include placement of drainage material beneath the entire slab and perforated pipes along the sleeper beam to evacuate water infiltrated from expansion joints placed on top of the slab (Luna, 2004; Abu-Hejleh et al., 2006).

Since the sleeper slab is typically supported on the backfill material used in the abutment, similar compaction efforts of the backfill material should be required underneath this slab. Compaction specifications require the maximum density of the fill to be at least 95 percent of the maximum dry density per AASHTO T-99 (Luna, 2004). Two new supporting systems for the sleeper slab are suggested by Abu-Hejleh et al. (2006). The first system consists of placing higher quality MSE backfill or flowfill under the sleeper slab rather than under the approach slab. The second supporting system consists of using driven piles to support the sleeper slab and using cheaper backfill material behind the abutments and expansion joint device, typically placed on top of the sleeper slab. In some cases where the settlement problem would be significant and continuous for extended periods, elimination of the approach and sleeper slabs altogether should be considered. As an alternative, full-depth asphalt approach slabs could be used with maintenance overlays as needed (Abu-Hejleh et al., 2006).

3.9 Effective Drainage and Erosion Control Methods

According to Mekkawy et al. (2005) and White et al. (2005), insufficient drainage is another problem often attributed to the settlements near the bridge abutments. Water collected on the bridge pavement can flow into the underlying fill materials due to ineffective seals at the joints between the bridge approach slab and the abutments and this infiltrated water can erode the backfill material. The material erosion can cause void development under the bridge abutments, resulting in the eventual settlements of the bridge approach slabs. Hence, the design of bridge approaches has to be incorporated with an efficient drainage system, such as providing drainage inlets at the end of a bridge deck to collect surface water before getting to the approach slab (Abu-Hejleh et al., 2006).

Also, additional surface or internal drainage to keep water off the slopes is also recommended for correcting the superficial erosion of embankments (Wahls, 1990). Keeping the water away from the soil is a simple and a significant factor in reducing the settlement of the soil. Construction costs added to incorporate a good drainage system are not high when compared to the expensive maintenance costs that they might experience during the service life of the bridge (Dupont and Allen, 2002). Hence, all efforts should be made to design the bridges with effective seals and good drainage conditions in and around the bridge structures.

Some of the recommendations reported in the literature to improve drainage conditions include the use of a large diameter surface drain and gutter system in the shoulder of the approach slab and use of a geo-composite vertical drainage system around the embankments, with both drainage systems having the potential to increase the drainage capacity (White et al., 2005). This study also recommended the use of porous backfill material or limiting the percentage of fine particles in the fill material to reduce material plasticity and enhance drainage properties.

Based on a survey conducted by Hoppe (1999), the allowable percentage of fine material passing the 75-micron (No. 200) sieve in the backfills varied from less than 4 percent to 20 percent by different state agencies. From the same study, it was noted that typical provisions in state agencies include plastic drainpipes, weep holes in the abutments, and the use of granular, free-draining fill. The use of geosynthetic materials, fabrics, and geo-composite drainage panels in the bridge systems was also reported. Other alternatives including the use of a thick layer of tire chips as an elastic zone behind the abutment with a high capacity of drainage was also successfully implemented (White et al., 2005).

Other recommendations including the grading off of the crest to direct runoff away from the back slope and the use of interceptor drains on the back slope are also cited (Wu et al., 2006). It is also recommended to perform periodic maintenance to minimize runoff infiltration and install a combination of granular drain materials, geotextiles or a geo-composite drain along the back and the base of the fill (Wu et al., 2006).

When the MSE structures are used, the drainage systems are recommended to construct in many locations; for example, in the retained soil to intercept any seepage or trapped groundwater, or behind and beneath the wall to interrupt water levels before intersect of the structure (Wu et al., 2006). To reduce surface water infiltration into the retained fill and reinforced fill, an impermeable cap and adequate slopes to nearby surface drain pipes or paved ditches with outlets to storm sewers or to natural drains should be provided.

Internal drainage of the reinforced fill can be attained by the use of a free-draining granular material that is free of fines (less than 5 percent passing the No. 200 sieve). Arrangement should be provided for drainage to the base of the fill to prevent water exiting the wall face and causing erosion and/or face stains. The drains should have suitable outlets for discharge of seepage away from the reinforced soil structure (Elias et al., 2001). A suggested drainage system for MSE walls is depicted in Figure 55 (Abu-Hejleh et al., 2006).

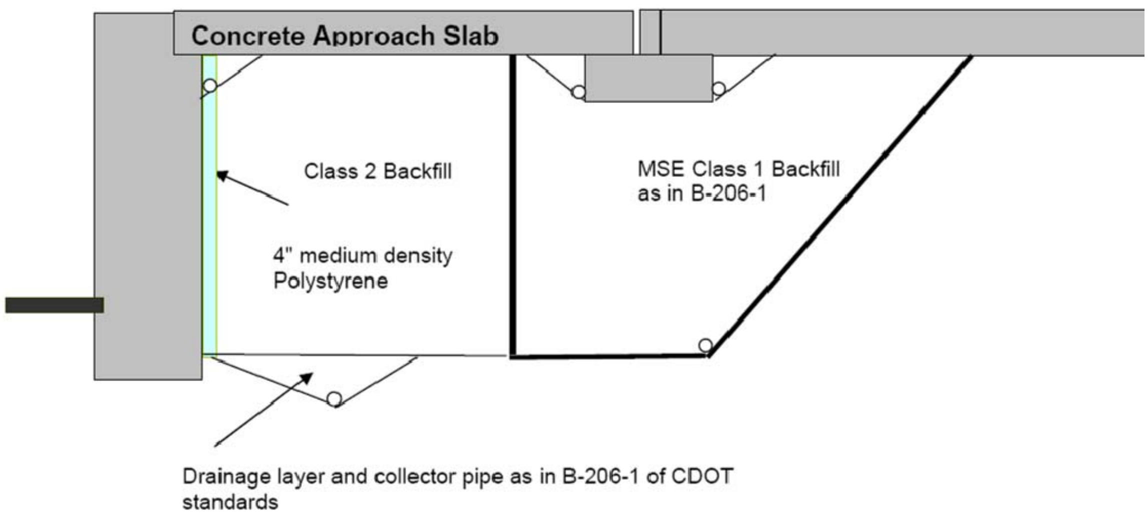


Figure 55 – MSE Walls System under Sleeper Slab (Abu-Hejleh et al., 2006)

Gabions, which are sometimes used to ensure stability of the wall face during construction, have function also as a drainage layer after completion of construction and also as a buffer at the interface between the highly rigid concrete facing and the deformable backfill (Japan Railway Technical Research Institute, 1998).

Another approach to provide an adequate internal drainage system behind the abutment and wingwall is to construct a layer of filter material before placement of the backfill and then install a 6 in. diameter perforated pipe at the bottom to collect excess water (Abu-Hejleh et al., 2006). This water is then carried out by a non-perforated pipe directly through the wingwall (see Figure 56). This study also recommended placement of a drainage inlet in the approach slab, or end of deck, to collect the bridge surface water before reaching the expansion joints. In addition, it is also recommended that horizontal drainage measures should be installed from the side of the structure to remove the water from the interface zone between the embankment (often a granular soil layer) and the foundation soil (usually a cohesive soil layer) (Abu-Hejleh et al., 2006).

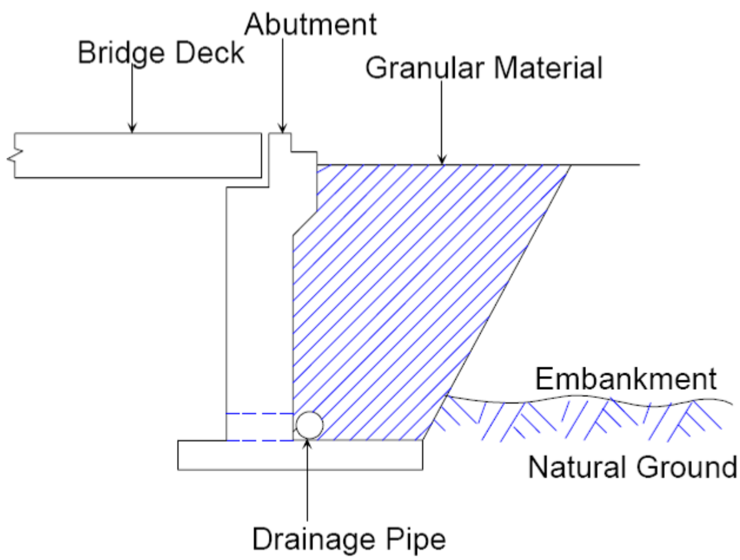


Figure 56 – Drainage Layer of Granular Material and Collector Pipe (Nassif, 2002)

Briaud et al. (1997) encouraged the use of a curb-to-curb design for erosion control and effective drainage of water away from the bridge structure and approach slab system. Figure 57a shows a poorly designed approach slab that will allow water into the backfill and embankment materials promoting erosion and weakening of these granular materials. On the contrary, Figure 57b shows a system that will prevent infiltration into the soils below the approach slab. Stewart (1985) suggested that the pavement should even be placed as a cantilever system over the wingwall to further mitigate infiltration below the approach slab.

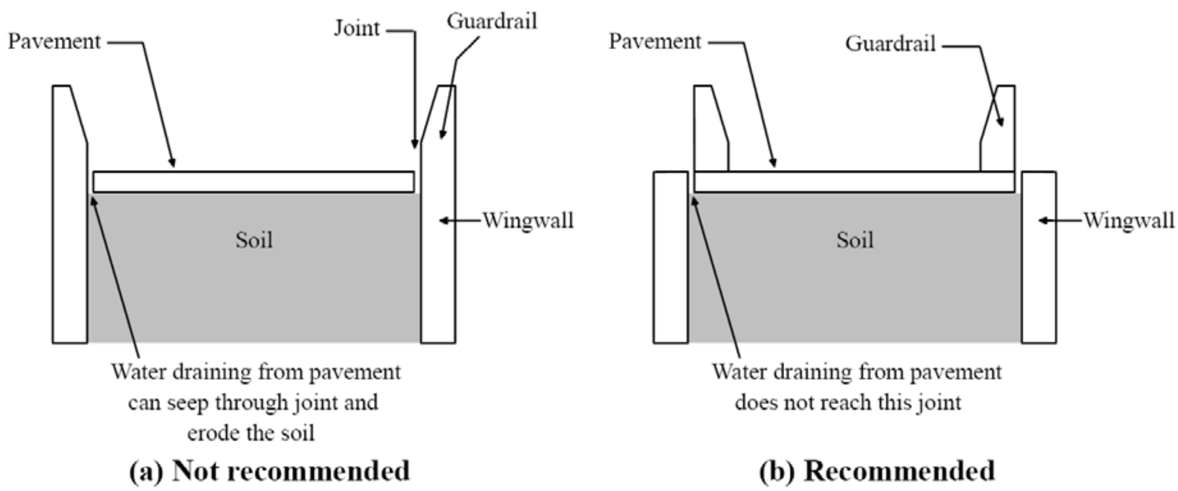


Figure 57 – Approach Slab Joint Details at Pavement Edge (Briaud et al., 1997)

Figure 58 provides an excellent example of good drainage and erosion control on the embankment face underneath the bridge where riprap was effectively used to prevent scour on the face which could cause erosion under the approach slab and bridge abutments (Lenke, 2006). In this research, the use of concrete slope protection on the embankment faces and sides and drainage channels were claimed to be successful in mitigating erosion problems and facilitating adequate drainage conditions (see Figure 59).

Figure 58 – Riprap used for Erosion Control (Lenke, 2006)

Figure 59 – Concrete Slope Protection with Drainage Gutter and Drainage Channel (Lenke, 2006)

White et al. (2005) performed a review of several drainage designs implemented by various state agencies to compare different state-of-practice approaches in the United States. The review showed that three main variations of drainage systems were practiced in the U.S. These are: (1) porous backfill around a perforated drain pipe; (2) geotextiles wrapped around the porous fill; and (3) vertical geo-composite drainage system (Figures 60 to 62).

From this study, it was reported that wrapping the porous fill with geotextiles has helped in reducing erosion and fines infiltration. Another interesting observation was from Table 7 that reported approximately 14 out of 16 states have used a combination of two or more of these alternatives to increase the drainage efficiency. It was also reported that the Texas practice is predominantly using porous fills and geotextiles as drainage systems (White et al., 2005).

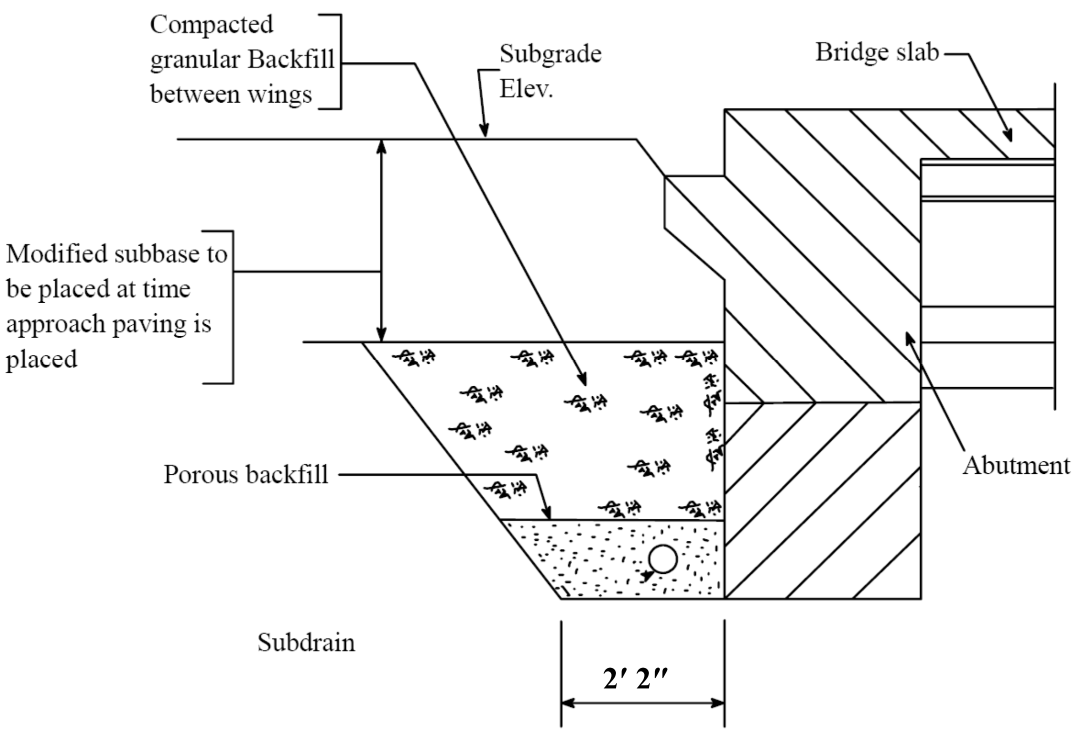


Figure 60 – Schematic of Porous Fill Surrounding Subdrain (White et al., 2005)

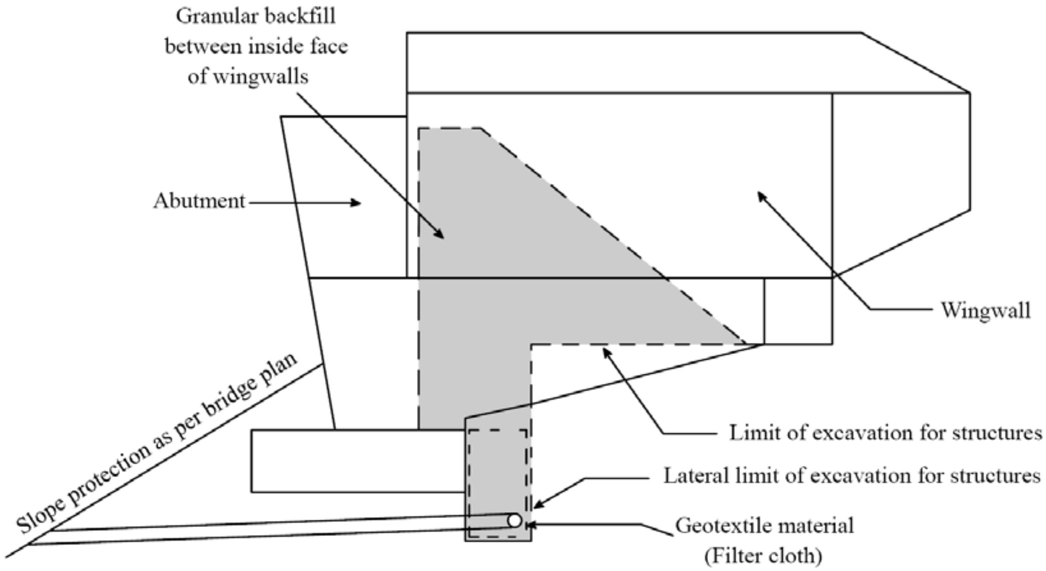


Figure 61 – Schematic of Granular Backfill Wrapped with Geotextile Filter Material

(White et al., 2005)

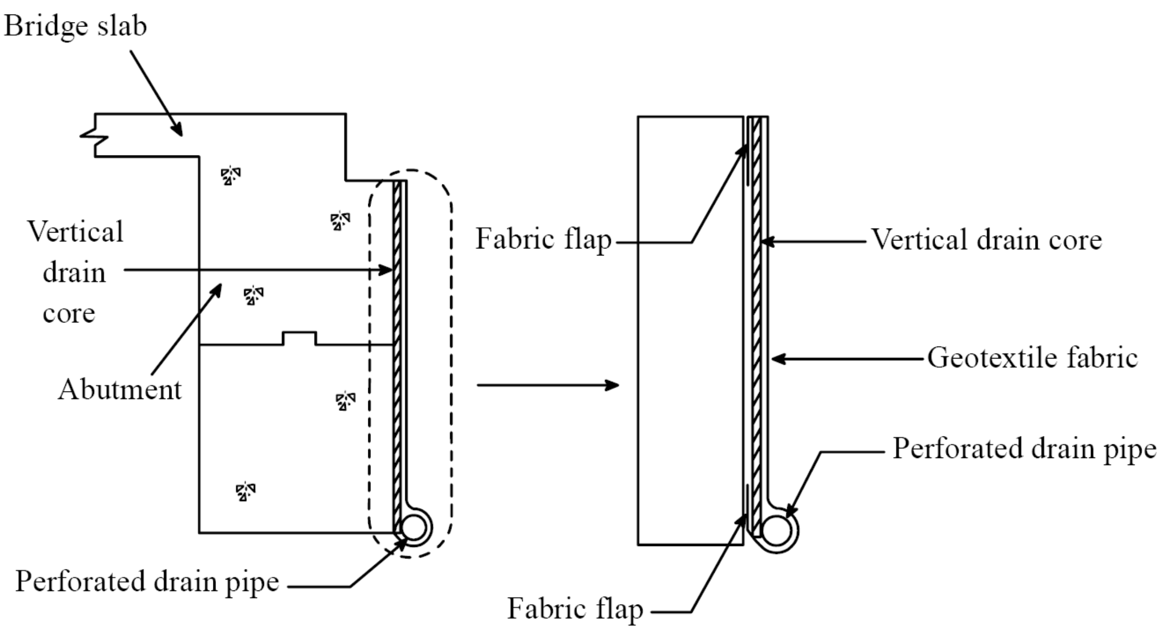


Figure 62 – Schematic of Geocomposite Vertical Drain Wrapped with Filter Fabric

(White et al., 2005)

Table 7 – Drainage Method Used by Various States (White et al., 2005)

State	Porous Fill	Geotextile	Geocomposite Drainage System
Iowa	X	-	-
California	X	X	X
Colorado	-	X	X
Indiana	X	X	-
Louisiana	X	X	X
Missouri	-	X	X
Nebraska	-	X	X
New Jersey	X	X	X
New York	-	-	X
North Carolina	X	X	-
Oklahoma	X	X	-
Oregon	X	X	-
Tennessee	X	X	-
Texas	X	X	-
Washington	X	-	-
Wisconsin	X	X	-

Mekkawy et al. (2005) and White et al. (2005) concluded from a series of large scale laboratory experiments that the porous backfill behind the abutment and/or geocomposite drainage systems would improve the drainage capacity and would reduce the erosion around the abutment, which will mitigate the differential settlements caused by the erosion and void formation of the backfill material.

4. MAINTENANCE MEASURES FOR DISTRESSED APPROACH SLABS

Maintenance and rehabilitation techniques have also been used to treat distressed approach slabs (Wahls, 1990; Hoppe, 1999). It is estimated that bridge approach maintenance costs are at least $100 million per year in the United States (Briaud et al., 1997; Nassif, 2002). Many states indicate that the best practice to minimize the presence of bridge bumps is to establish up-to-date maintenance activities, by scheduling periodic repair activities in addition to occasional required maintenance (Dupont and Allen, 2002). Depending on the circumstances, maintenance of distressed approach slabs is comprised of asphalt overlays, slab jacking, and approach slab adjustment or replacement techniques (Dupont and Allen, 2002).

It is also reported that in the case of conventional bridges, much of the cost of maintenance is related to repair of damage at joints because such joints require periodic cleaning and replacement (Briaud, 1997; Arsoy, 1999). Other times, pavement patching at the ends of the bridge represents most of the maintenance costs. For longer bridges, the pavement patching lengths are longer due to problems experienced by the temperature induced cyclic movements (Hoppe, 1999). However, Arsoy (1999) noted that integral abutment bridges perform well with fewer maintenance problems than conventional bridges.

Also, a periodic cleanout and maintenance schedule is required for all drainage structures on the bridge and bridge approach system to ensure proper removal of water away from the structure and to minimize runoff infiltration into underlying fill layers (Lenke, 2006). Most frequently, maintenance of drainage structures and joints is lacking and must be improved in order to take full advantage of these design features (Lenke, 2006; Wu et al., 2006).

Lenke (2006) presented his study showing many cases of poor maintenance at the expansion joints between the bridge deck, approach slab, approach pavement, and drainage systems, resulting in many bridge replacement and rehabilitation costs. He suggested that to prevent stress buildup at the expansion joints between the bridge structure, the approach slab and the pavement system, a good maintenance by cleaning and replacement (when necessary) is required. Such stresses can not only cause damage to the deck and the abutment, but can also cause distortions of the approach slab.

Lenke (2006) also identified another maintenance issue resulting from Alkali-Silica Reactivity (ASR) problems. The stresses caused by ASR expansion can lead to severe damage at the joints connecting the bridge deck to the approach slab and the approach slab to the preceding concrete pavement. These ASR expansion stresses can cause spalling and resultant crack widening, which regularly requires joint filling with bituminous materials work (Lenke, 2006).

White et al. (2005) also conducted a comprehensive study in a case of lack of maintenance of drainage structures, such as clogged or blocked drains, animal interaction, and deterioration of joint fillers, gutters and channels. The study showed that due to the lack of maintenance many problems about maintenance occurred, resulting in numerous and costly repair operations. White et al. (2005) also pointed out some potential causes of bridge approach settlement discovered during the maintenance activities. For example, they mentioned that the loose and not properly compacted backfill materials can cause poorly performing approach slabs. Coring operations revealed that voids are highest near the bridge abutment and decreased with distance with void sizes ranging from 0.5 inch to 12 inches. Snake cameras used at sub-drain outlets demonstrated that most of the investigated subdrains were not functioning properly. The subdrains were either dry with no evidence of water or blocked with soil fines and debris or had collapsed. Some of these problems are attributed to erosion induced movements in the fill material from moisture infiltration. This signifies the need for constant maintenance of joints and drains so that infiltration into the soil layers will be low. Along with the maintenance, reconstruction or rehabilitation of distressed approach slabs are very necessary.

Several soil stabilization techniques were found in the literature to stabilize the fill under the approach slab. These techniques are intended to smooth the approaches by raising the sleeper slab and approaches, especially if application of an asphalt overlay is not feasible (Abu-Hejleh et al., 2006). The most important techniques are pressure grouting under the slab, slab-jacking or mud-jacking technique, the Urethane method, and compaction or high pressure grouting. Most of these techniques are often used as remedial measures after problems are detected. However, the same could be applied even in new bridge constructions. A brief overview of these methods is presented below.

4.1 Replacement Method

Highly deteriorated approach slabs due to the formation of a bump are mostly replaced with the new approach slabs. This process is the most expensive and time taking process as the construction process results in frequent closure of lanes, traffic congestion, etc. A new internal research project has been initiated by the California Department of Transportation to examine different replacement alternatives for deteriorated approach slabs. In this project, prefabricated

Fiber Reinforced Polymer (FRP) decks as well as FRP gridforms and rebars were investigated as replacement options. Full scale approach slabs were tested under simulated wheel loads. Performance of the approach slabs were also examined under simulated washout conditions. Figure 63 shows the test schematic.

Figure 63 – Simulated Approach Slab Deflection Due to Washout by UC Davis Research Team (http://cee.engr.ucdavis.edu/faculty/chai/Research/ApproachSlab/ApproachSlab.html)

4.2 Mud/Slab Jacking

Mud/slab jacking is a quick and economical technique of raising a settled slab section to a desired elevation by pressure injecting of cement grout or mud-cement mixtures under the slabs (EM 1110-2-3506, 20 Jan 84). According to EM 1110-2-3506, slab jacking is used to improve the riding qualities of the surface of the pavement, prevent impact loading over the irregularities by fast-moving traffic, correct faulty drainage, prevent pumping at transverse joints, lift or level other structures, and prevent additional settlement.

In this method, the mud grout is prepared using the topsoil that is free from roots, rocks, and debris mixed with cement and enough water to produce a thick grout. This grout is injected to fill the void spaces underneath the approach slab through grout holes made through the approach slabs (Bowders et al., 2002). The injection is performed in a systematic manner to avoid cracks on the approach slab as shown in Figure 64. Precautionary measures need to be taken near to side retaining walls and abutment walls (Luna et al., 2004).

Even though this technique has been successfully adopted by several states including Kentucky, Missouri, Minnesota, North Dakota, Oklahoma, Oregon, and Texas for lifting the settled approach slabs, the mud/slab jacking can be quite expensive. Mud jacking may also cause drainage systems next to the abutment to become clogged and is difficult sometimes to control the placement of the material (Dupont and Allen, 2002). Other difficulties including limited spread of grout into voids, large access holes which must be filled and lack of sufficient procedural process made this technique as uneconomical (Soltesz, 2002). Abu al-Eis and LaBarca (2007) reported that the cost of this technique was between $40 and $60 per one square yard of pavement used based on two test sections constructed in Columbia and Dane counties in Wisconsin.

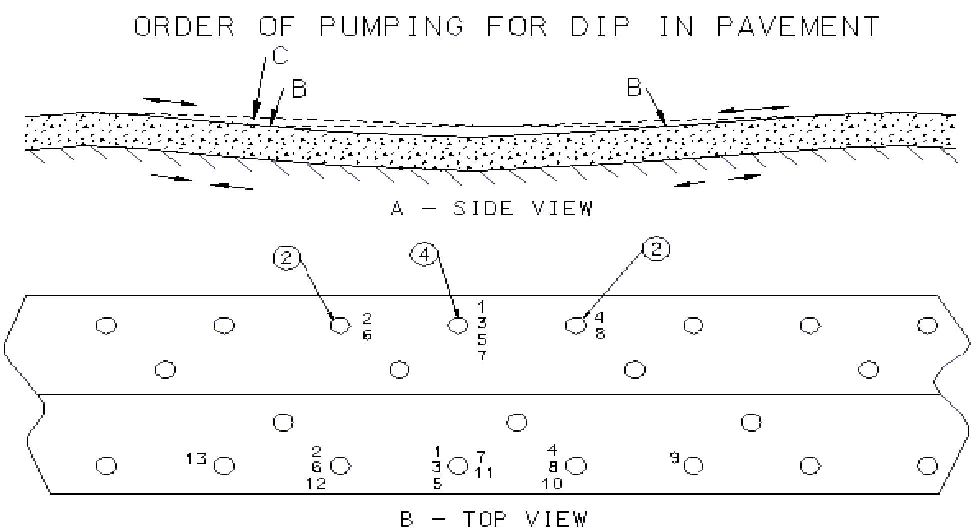


Figure 64 – Mud-Jacking Injection Sequences (MoDOT)

4.3 Grouting

Pressure Grouting under the Slab

The presence of voids beneath the approach slab can lead to instability, cracking, sinking, and pounding problems (Abu-Hejleh et al., 2006). In order to mitigate the problem, pressure grouting is commonly used for bridge approach maintenance practice as a preventive measure (White et al., 2005; 2007). Pressure grouting under the slab is used to fill the voids beneath the approach slab through injection of flowable grout, without raising the slab (Abu-Hejleh et al., 2006).

According to White et al. (2007), undersealing the approach slab by pressure grouting normally has two operations within the first year after completion of approach pavement construction. The first operation is done within the first 2–6 months, while the second one is employed within 6 months after the first undersealing. The grout mix design consists of Type 1 Portland cement and Class C fly ash at a ratio of 1:3. Water is also added in the grouting material to achieve the specified fluidity (Buss, 1989). Moreover, in order to avoid the lifting of the approach slab, grout injection pressures are kept to less than 35 kPa (White et al., 2007).

Abu-Hejleh et al. (2006) stated that the construction techniques for this method are to drill 1-7/8 in. holes through the concrete or asphalt approach slabs using a rectangular spacing as shown in Figure 65. The depth is determined by the ease of driving the stinger or outlet tube, which is pounded into the hole (Abu-Hejleh et al., 2006). A fence post pounder is used to hammer the stinger and extension pieces into the soil (Abu-Hejleh et al., 2006). As the stinger is pounded down, the operator can determine if the soil is loose or soft and if there are voids under the slab.

Although grouting under the approach slab is commonly used for bridge approach settlement as a mitigation method, White et al. (2007) stated that the grouting is not a long term solution for this problem. The grouting does not prevent further settlement or loss of backfill material due to erosion (White et al., 2005; 2007).

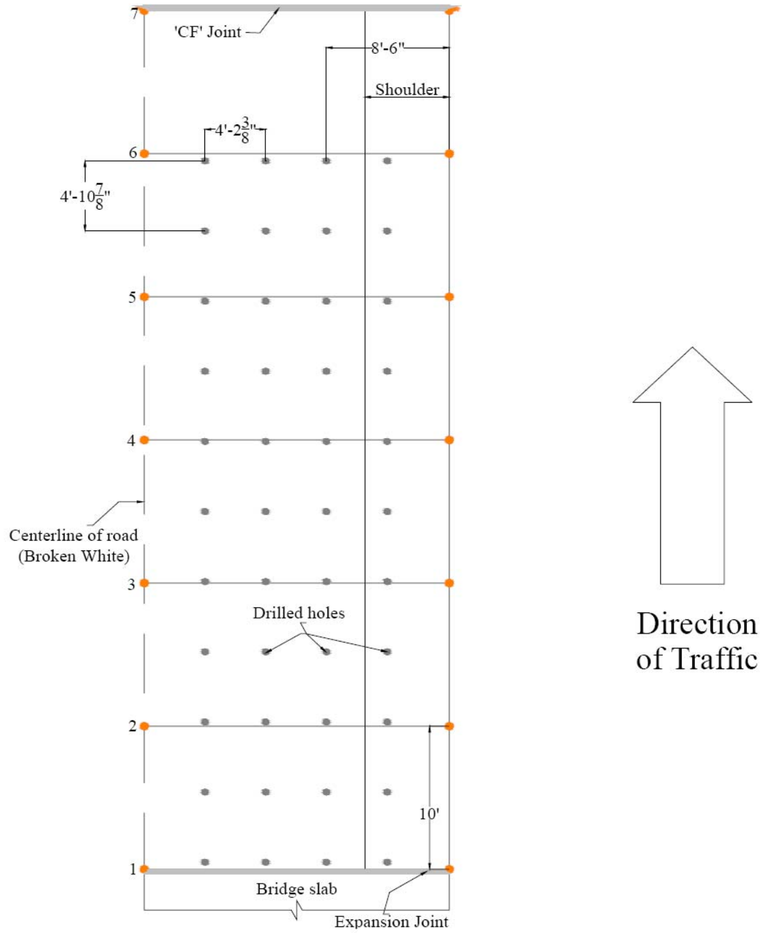


Figure 65 – Location of Holes Drilled on an Approach Slab (White et al., 2005)

Compaction or High Pressure Grouting

Compaction grouting is a method for improving soil by densifying loose and liquefaction soils and resulting in increasing the soil strength (Miller and Roykroft, 2004). The compaction grouting is a physical process, involving pressure-displacement of soils with stiff, low-mobility sand-cement grout (Strauss et al., 2004).

According to the ASCE Grouting Committee (1980), the grout generally does not enter the soil pores but remains as a homogenous mass that gives controlled displacement to compact loose soils, gives controlled displacement for lifting of structures, or both. Abu-Hejleh et al. (2006) also stated that apart from soil densification, the compaction grouting is also employed to lift and level the approach slab and adjacent roadways.

The compaction grouting can be used to stabilize both shallow and deep seated soft layers (Abu-Hejleh et al., 2006). Section 211 of the CDOT Standard Specifications prescribes the grouting must be low slump and a low mobility grout with a high internal friction angle. When the technique is used in weak or loose soils, the grout typically forms a coherent “bulb” at the tip of the injection pipe; thus, the surrounding soil is compacted and/or densified (Miller and Roykroft, 2004). For relatively free draining soils including gravel, sands, and coarse silts, the method has proven to be effective (Abu-Hejleh et al., 2006).

4.4 Urethane Injection Technique

The Urethane injection technique was first developed in 1975 in Finland to lift and under seal concrete pavements and subsequently adopted in several States in lifting concrete pavements (Abu al-Eis and LaBarca, 2007). In this process, a resin manufactured from high density polyurethane is injected through grout holes (5/8-inch diameter) made through the approach slab to lift, fill the voids and to under seal the slab (Abu al-Eis and LaBarca, 2007). The injected resin will gain 90 percent of its maximum compressive strength (minimum compressive strength is 40 psi) within 15 minutes. Once the voids are filled, the grout holes are filled with inexpansive grout material. Elevation levels are taken before and after the process to ensure the required lifting is achieved (Abu al-Eis and LaBarca, 2007).

As reported by Abu al-Eis and LaBarca, (2007), the Louisiana Department of Transportation successfully adopted this technique for two different bridge approaches and observed that the international roughness index (IRI) values were reduced by 33 percent to 57 percent after monitoring for four years. This method involves the precise liquid injection of high-density polyurethane plastic through small (5/8 in.) holes drilled in the sagging concrete slab (Abu al-Eis and LaBarca, 2007). Once it is applied, the material expands to lift and stabilize the slab, while filling voids in the underlying soil and under sealing the existing concrete (**http://www.stableconcrete.com/uretek.html**). Based on the manufacturer provided information, this technology is simple and rapid. It can lead to a permanent solution and also can resist erosion and compression over a time period.

Brewer et al. (1994) first evaluated the Urethane injection technique to raise bridge approach slabs in Oklahoma. They reported that three test slabs out of six were cracked during or after the injection and in one case, the PCC slab broke in half during the injection. The Michigan Department of Transportation reported that this technique provided temporary increase in base stability and improvement in ride quality for one year (Opland and Barnhart, 1995). Soltesz (2002) noticed that the Urethane treatment was successful even after two years where the injection holes are properly sealed. The Oregon Department of Transportation researchers reported that the Urethane material was able to penetrate holes with diameters as small as 1/8 inch and which was the added advantage of this technique to fill the minor pores of the subbase and lift the pavement slabs (Soltesz, 2002).

Abu al-Eis and LaBarca, (2007) reported that the cost of this technique was between $6 to $7 per pound of foam used, which was calculated based on two test sections constructed in Columbia and Dane Counties in Wisconsin. They summarized the cost comparison of this technique with other slab lifting methods (as shown in Table 8) and concluded that this technique is expensive when compared to other methods if calculated based on direct costs. They also reported that this technique is very fast and can open the lanes for traffic immediately after the treatment. The amount of Urethane resin used in each project is also questionable as this quantity is directly used in the cost analysis. Considering this fact, TXDOT amended its Special Specification 3043-001, which requires a Special Provision for determining the quantity of polymer resin used for "Raising and Undersealing Concrete Slabs." Regarding the Special Specification 3043-001, the quantity of the resin utilized will be calculated by one of the following methods:

1. Payment will be made according to the actual quantity of polymer resin used in the work by weighing each holding tank with components by certified scales before and after each day's work.
2. Payment will be made according to the actual quantity of polymer resin used in the work by determining the weight of material placed by measuring the depth of polymer resin in the holding tanks before and after each day's work. A professional engineer and a site engineer must approve the calculation method, which is based on the certified measured volume of each tank and the unit weight of each component to determine the weight of resins used in the work.

Table 8 – Cost Comparison for Four Slab Faulting Repair Methods

Location	Method	Total Cost	Cost per yd^2	Days to Complete
I-30 (80 yd^2)	URETEK	$19,440	$243	0.75
	Slab Replacement	$34,000	$425	3
	HMA Overlay	$3,630	$45	1
	Mud-jacking	$3,000	$38	1
USH 14 (53.4 yd^2)	URETEK	$6,260	$117	0.5
	Slab Replacement	$22,670	$425	3
	HMA Overlay	$3,375	$63	1
	Mud-jacking	$3,000	$56	1

Several Districts in Texas use this method as a remediation method and based on the present research contacts, these methods are deemed effective. Researchers visited two bridge approach slab repair works recently initiated in Hill County, Texas, and another completed several years back on several highways in and around Houston, Texas. Both visits were made in late February 2008.

Figure 66 shows the schematic and photographic view of the bridge site with the void developed under the approach slab. The cause of the problem was identified as the erosion of the granular backfill material under the approach slab. Figure 67 depicts the position of the approach slab during and after the injection process. During and immediately after the injection process, researchers observed a few minor hairline cracks on the approach slab as shown in Figure 68. The minor cracks on the surface of the approach slab during this injection operation are relatively common and they will not lead to further distress of the approach slab. The post performance of this method is very crucial to address the expansion of these hairline cracks and movements of repaired approach slabs. A simple field monitoring study including elevation surveys and visual inspection of these minor cracks would reveal the effectiveness of this technique.

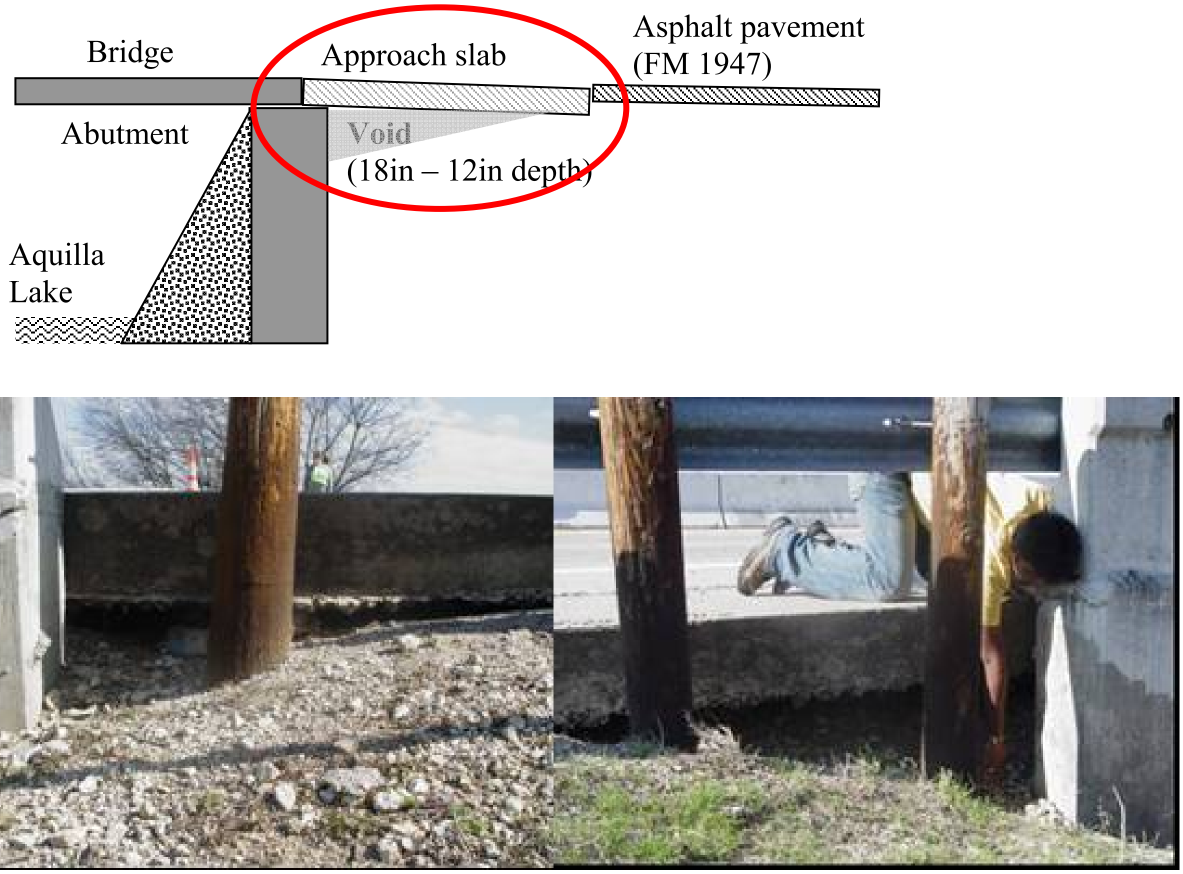


Figure 66 – Schematic of the Approach Slab with Developed Void under the Bridge at FM 1947 Hill County, Texas

Figure 67 – Position of Approach Slab during and after the Urethane Injection Process

Figure 68 – Hairline Crack Observed on the Approach Slab during the Urethane Injection

As per the discussions with TxDOT engineers in Houston, the process was quite effective. Several Houston sites that were visited were repaired utilizing this injection method 10 years ago and they are still functioning adequately. The work reported in the Houston District was instrumental in the development of the TxDOT Special Specification for the use of the Urethane injection method for lifting the distressed approach slabs.

4.4 Flowable Fill

Flowable fill or controlled low-strength material is defined by ACI Committee 229 as a self-compacting, cementitious material used primarily as a backfill in lieu of compacted fill. The flowable fill has other common names, such as unshrinkable fill, controlled density fill, flowable mortar, flowable fill, plastic soil-cement, and soil-cement slurry (Du et al., 2006). This controlled low-strength filling material is made of cement, fly ash, water, sand, and typically an air-entraining admixture (NCHRP, 597). A significant requisite property of flowable fill is the self-leveling ability, which allows it to flow; no compaction is needed to fill voids and hard-to-reach zones (Abu-Hejleh et al., 2006). Therefore, the flowable fill is commonly used in the backfill applications, utility bedding, void fill, and bridge approaches (Du et al., 2006).

A primary purpose of using flowable fill is as a backfill behind the abutment. CDOT has used the flowable fill backfill behind the abutment wall in an effort to reduce the approach settlements since 1992 (Abu-Hejleh et al., 2006). The other new applications for the flowable fill are for use as a subbase under bridge approaches and a repair work of the approaches (Du et al., 2006). Historically, the application of using flowable fill as a subbase was first employed in Ohio by ODOT (Brewer, 1992).

In Iowa, the flowable fill is a favorable backfill used as a placement under the existing bridges, around or within box culverts or culvert pipes, and in open trenches (Smadi, 2001). Smadi (2001) also cited that the advantages of flowable mortar are not only due to its fluidity, but also due to its durability, requiring less frequent maintenance. Moreover, the flowable mortar is also easily excavated. Therefore, the maintenance works, if required, can be done effortlessly (Smadi, 2001). Figure 69 shows details of flowable mortar used under a roadway pavement.

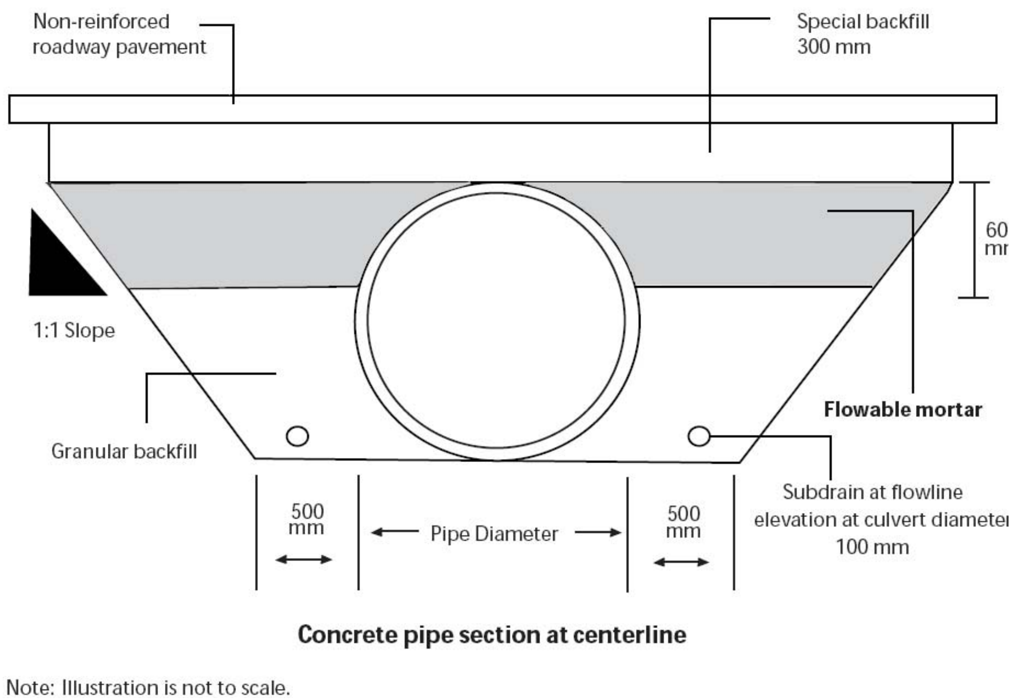


Figure 69 – The Flowable Mortar Used under a Roadway Pavement (Smadi, 2001)

In Texas, the flowable fill was used for the first time for repairing severe settlements of bridge approaches at the intersection of I-35 and O'Conner Drive in San Antonio in 2002 by TxDOT (Folliard et al, 2008; Du et al., 2006). For this practice, TxDOT used a specialized mixture using flowable fill, which consisted of sand, flyash, and water; no cement (Williammee, 2008). The compressive strength of cored samples indicated that the long-term strength and rigidity of the flowable fill were strong enough to serve this purpose (Folliard et al, 2008). After the mixture proportions were adjusted to have adequate flowability for the application, the flowable fill has shown a great success for repairing the approaches (Du et al., 2006; Williammee, 2008). Recently, the flowable fill was used in the Fort Worth District in place of a flexible base beneath the approach slab. The 3 ft deep flex base is prepared with Type 1 cement (2.4 percent by weight) as a base material as shown in Figure 70.

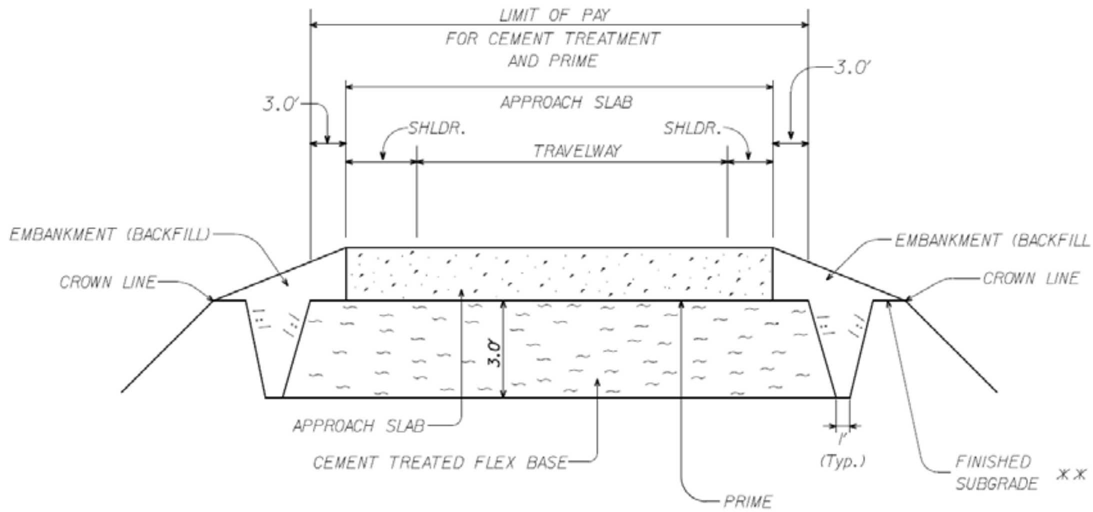


Figure 70 – The Flowable Fill Used as a Base Material (Du, 2008)

4.5 Other Methods

Several other techniques are also available to mitigate the settlement problem caused in the approach slab area, and these techniques are discussed in the following.

Precambering

If the approach pavement settlement cannot be controlled economically, a pre-cambered roadway approach may be applied (Tadros and Benak, 1989). Hoppe (1999) recommended implementing pre-cambering of bridge approaches for up to a 1/125 longitudinal gradient. The pre-cambering is used to accommodate the differential settlement that will inevitably occur between a structure constructed on deep foundations and adjoining earthworks.

Briaud et al. (1997) recommended pre-cambering with gradient values of less than 1/200 of the approach slab length to compensate for the anticipated post-construction settlements. The pre-cambered design utilizes a paving notch that supports a concrete slab. The notch must be effectively hinged, which allows the concrete slab moving radially (see Figure 71). The flexible pavement over the slab will absorb some movement below it but not to a great extent (Briaud et al., 1997). The pre-cambered approach system also requires an accurate assessment of settlement potential (if possible). The pre-cambered approach design could be specified in situations where time is not available for more conventional settlement remediation, such as preloading, wick drains, and others (Luna, 2004).

Wong and Small (1994) conducted laboratory tests to investigate the effects of constructing approach slabs with an angle from the horizontal on reducing the bump at the end of the bridge. It was found that horizontal slabs suffered a rapid change in surface deformation with the formation of obvious bumps, while pre-cambering the slabs with angles of 5° to 10° provided a smoother transition.

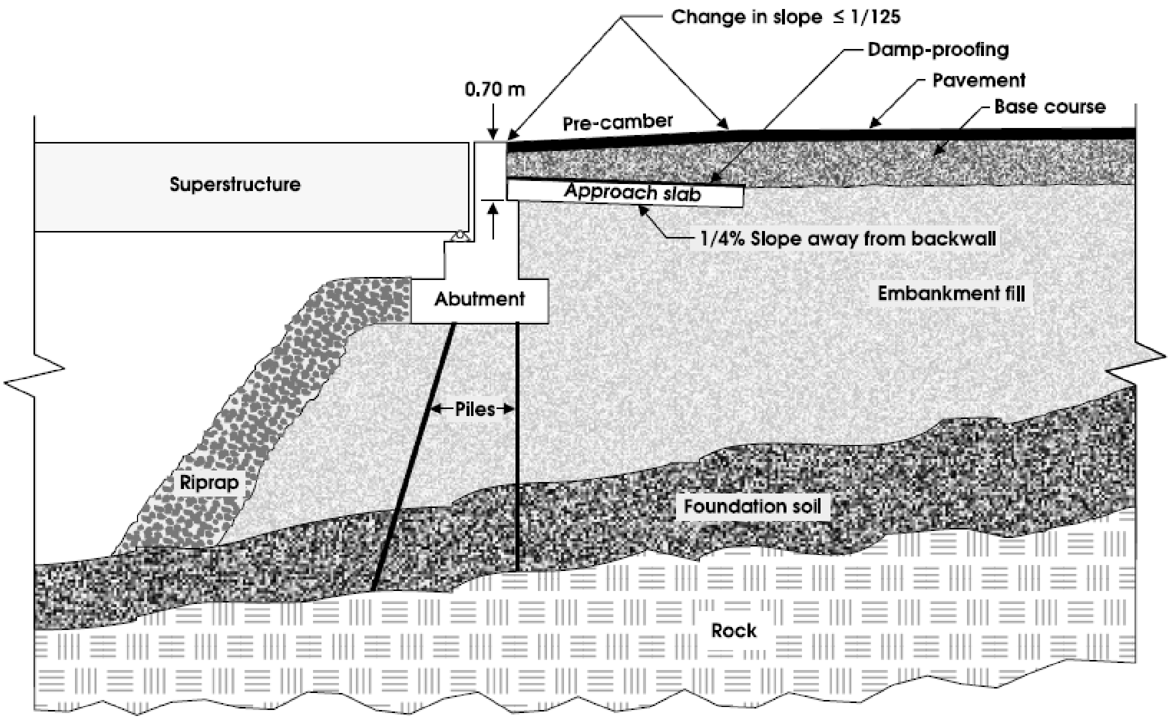


Figure 71 – Pre-cambered Approach Design (Hoppe, 1999)

Lightweight Fill Materials

The lightweight materials such as Expanded Polystyrene (EPS) Geofoam and Expanded Clay Shale (ECS) can be used either as a construction embankment fill material for new bridge approach embankments or can be used as a fill material during the repair of distressed approach slabs. Description of this method was presented earlier in Section 3.3.

4.6 Expanded Polystyrene (EPS) Geofoam

Expanded Polystyrene (EPS) Geofoam is a lightweight material made of rigid foam plastic that has been used as fill material around the world for more than 30 years. This material is approximately 100 times lighter than conventional soils and at least 20 to 30 times lighter than any other lightweight fill alternatives. The added advantages of EPS Geofoam including reduced loads on underlying subgrade, increased construction speed, and reduced lateral stresses on retaining structures has increased the adoptability of this material to many highway construction projects. More than 20 state DOTs including Minnesota, New York, Massachusetts, and Utah adopted the EPS Geofoam to mitigate the differential settlement at the bridge abutments, slope stability, alternate construction on fill for approach embankments and reported high success in terms of ease and speed in construction, and reduced total project costs.

Lightweight EPS Geofoam was used as an alternate fill material at Kaneohe Interchange in Oahu, Hawaii, while encountering a 6 m thick layer of very soft organic soil during construction. A total of 17,000 m^3 of EPS Geofoam was used to support a 21 m high embankment construction (Mimura and Kimura, 1995). They reported the efficiency of the material in reducing the pre- and post-construction settlements. Figure 72 shows the construction of the embankment with the EPS Geofoam.

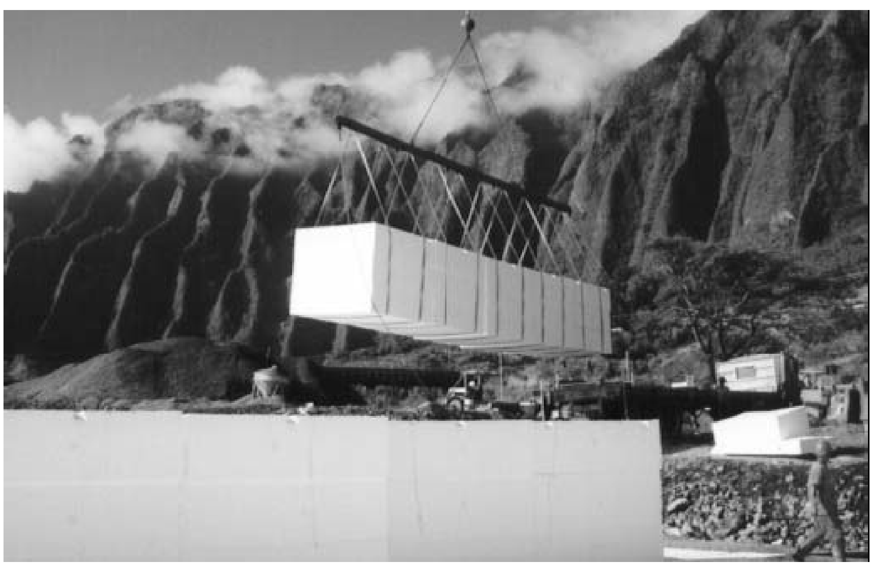


Figure 72 – Emergency Ramp and High Embankment Constructed Using the EPS Geofoam at Kaneohe Interchange in Oahu, Hawaii (Mimura and Kimura, 1995)

4.7 Expanded Clay Shale (ECS)

For nearly a century, expanded clay shale aggregates (ECS) have been used successfully around the world for various geotechnical applications as fill materials and to reduce overburden pressures (Expanded Shale, Clay & Slate Institute, 2004). ECS is a light weight aggregate prepared by expanding select minerals in a rotary kiln at temperatures of over 1000ºC (Holm and Ooi, 2003). The ECS is available throughout the world's industrially developed countries. Consideration of ECS as a remedy to geotechnical problems stems primarily from the improved physical properties of reduced dead weight, high internal stability, and high thermal resistance (Stoll and Holm, 1985). These advantages arise from the reduction in particle specific gravity, stability that results from the inherent high angle of internal friction, the controlled open-textured gradation available from a manufactured aggregate which assures high permeability, and high thermal resistance due to high particle porosity (Holm and Valsangkar, 1993).

ECS lightweight aggregates are approximately half the weight of conventional fills using common materials. Because of the high internal friction angle of these materials they can reduce vertical and lateral forces by more than one-half (Holm and Valsangkar, 1993). The lightweight aggregates have been commonly used in case-in-situ structural lightweight concretes for high rise buildings and bridges for several years (Holm and Valsangkar, 1993). Table 9 shows the general engineering properties of ECS (after ESCS, 2004).

Table 9 – General Properties for ECS (ESCS, 2004)

Aggregate Property	Test Method	Commonly Specifications for ECS	Typical for ECS Aggregates	Typical Design Values for Ordinary Fills
Soundness Loss	AASHTO T 104	<30 %	<6 %	<6 %
Abrasion Resistance	ASTM C 131	<40 %	20 – 40%	10 – 45%
Compacted Bulk Density	ASTM D 698	<70 lb/ft^3	40– 65 lb/ft^3	100-130lb/ft^3
Stability	ASTM D 3080	According to project	35º - 45º	30º - 38º (fine sand- sand & gravel)
Loose Bulk Density	ASTM C 29	Dry<50 lb/ft^3 Saturated<65 lb/ft^3	Dry 30-50 lb/ft^3	89-105 lb/ft^3
pH	AASHTO T 289	5 – 10	7 – 10	5 – 10

ECS aggregates provide a practical, reliable, and economical geotechnical solution (DeMerchant and Valsangkar, 2002). Their applications to geotechnical solutions are gaining popularity in recent years due to their promising engineering behavior. One such application of this material is to alleviate the overburden pressure on soft clay subgrades when used as an embankment fill material (Saride et al., 2008). The ECS material was recently used as an embankment backfill on a highway overpass along SH 360 in Arlington, Texas. The main intent of the research was to reduce the pressures exerted on the cohesive subgrades supporting the embankment and to reduce the differential settlements of the material at the approach embankment.

Figure 73 shows the typical cross section of the ECS embankment fill used at the project site. To evaluate the performance of the ECS as an embankment fill material and to understand the fill movements and their patterns, vertical inclinometers were installed; one in the median and another at the exterior slope of the high rise embankment (Figure 73). Elevation surveys were also conducted at regular intervals to check the surface settlements. Results from instrumentation for the past two years show a satisfactory performance of the ECS as fill in reducing the embankment settlements (Saride et al., 2008).

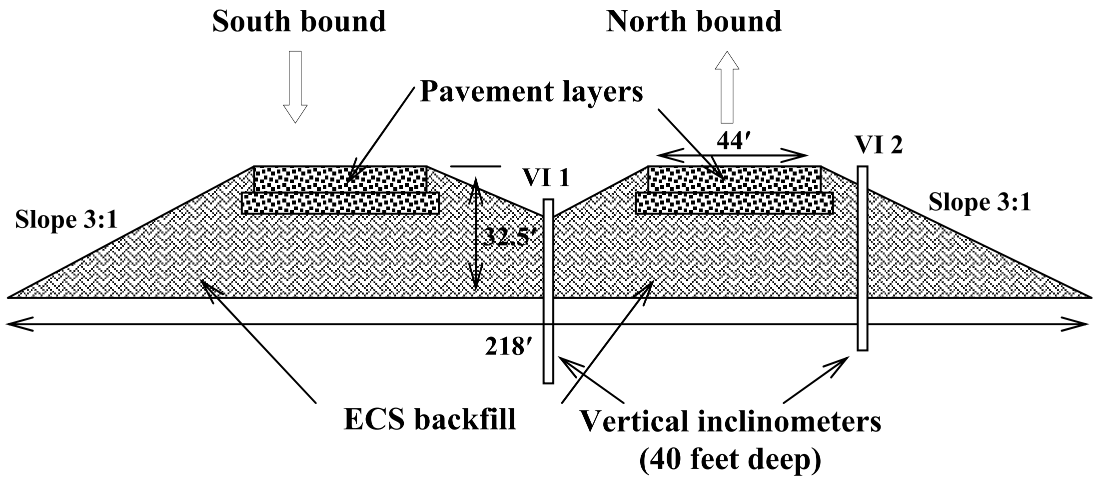


Figure 73 – Typical Cross Section of ECS Backfilled Approach Embankment, SH 360, Arlington, Texas

It should be noted that the north end of the bridge at this site was constructed using a local clay fill material and this approach slab settled even before the bridge was opened to the traffic. The slab was repaired using an asphalt overlay and researchers are currently monitoring this site. Additional settlement has occurred necessitating correction of the northbound departure slab in the near future.

5. USDOT REPORTS SUMMARY

In this section, a list of various state DOT studies as shown in Figure 74 that have addressed bridge approach settlement problems is compiled. Approximately 38 reports were collected from various DOTs and major findings of these reports are listed in Table 10.

Figure 74 – Summary of State DOTs Performed Research on Bridge Approach Settlements

Table 10 – Summary of State DOTs Work on Bridge Approach Settlements

No.	Agency	Title/Work	Topics Covered and Salient Information	Remarks
1.	KyDOT (Hopkins, 1969)	Preliminary survey done on the existing bridges to calculate settlement of highway bridge approaches and embankment foundations by using special experimental design and construction features at selected bridge sites	• Concrete bridge approaches are better than bituminous bridge approaches • Progressive failure or creep of the approach is a cause for the development of an approach fault • Erosion of soil from abutments contributes to development of defective bridges. • Traffic is not a cause for the settlement • Backfilling around abutments with a granular material did not arrest the development of faulted approaches • Settlement of the approach foundation and embankment contributes significantly to settlement of bridge approaches and approach pavements • Replacing the soft compressible material with rock or compacted material • Pre-consolidate using surcharge fill • Allow sufficient time for consolidation of the foundation under the load of the embankment • Use of vertical sand drains and drainage system • Longitudinal camber is provided at the approaches	Research Report
2.	WSDOT (DiMillio, 1982)	Performance of Highway Bridge Abutments Supported by Spread Footing on Compacted Fill	• Spread footing on compacted fill supporting the bridge abutment is very reliable and inexpensive • The superstructure with a spread footing can withstand temperate settlement (1-3 in.) without distress	Research and Implementation Report
3.	IDOT (Greimann et al., 1984)	Deign of Piles for Integral Abutment Bridge	• The ultimate load capacity for frictional piles was not affected by lateral displacements of up to 4 in. for H-piles and up to 2 in. for timber and concrete piles • The ultimate load capacity was considerably decreased if lateral displacements greater than 2 in. for end-bearing H- piles	Research Report
4.	KyDOT (Hopkins, 1985)	Long term movements of highway bridge approach embankments and pavements by surveying and observation of six bridge sites from 1966 to 1985	• Settlement of bridge approach foundations contributes significantly to settlements of approach pavements • Improper compaction, lateral movements, erosion of materials, and secondary compressions are the causes for long-term movement of bridge approaches	Synthesis Report

No.	Agency	Title/Work	Topics Covered and Salient Information	Remarks
5.	Caltrans (Stewart, 1985)	Survey of Highway structure approaches	• Structure approach slab policy • Design policies and procedures • Structure approach slab design concepts • Construction sequence and details for rehabilitation projects	Synthesis Report
6.	IDOT (Greimann et al., 1987)	Pile design and tests for integral abutment bridges due to the effect of temperature changes	• Horizontal displacement had no effect on the vertical load capacity • Use of a pre-drilled hole is recommended as a pile construction detail to reduce the pile stresses significantly when horizontal displacements of the pile occur	Research and Implementation Report
7.	NCDOT (Wahls, 1990)	Design and construction of bridge approaches and to revise and update the report of KyDOT (1969)	• Bridge approach settlements are caused due to time-dependent consolidation of embankment, poor compaction, drainage, and erosion of abutment backfill • Lateral creep of foundation soils and movements of the abutment • Type of abutment and foundation also affect the performance • Differential settlement can be minimized by using shallow foundations	Synthesis Report
8.	OKDOT (Laguros et al., 1990)	Evaluation of causes of excessive settlements of pavements behind bridge abutments and their remedies for the future	• Settlement problem is due to the absence of drainage • Major portion of the settlement occurs within first twenty years • Skewed approaches have higher approach settlement than non-skewed approaches	Research Report
9.	SDDOT (Schaefer and Koch, 1992)	Survey done to isolate and determine the mechanisms controlling backfill to reduce void development under bridge approaches	• Thermal induced movements of integral abutments are responsible for void development • No problem with the material used as a backfill • Voids are not developed due to erosion • Cracking is due to loss of support • Mud jacking does not affect the formation of voids • Non-integral abutment reduces the problem of voids • Maintenance cost increases by using integral abutments	Synthesis Report

No.	Agency	Title/Work	Topics Covered and Salient Information	Remarks
10.	TxDOT (Briaud et al., 1997)	Survey of Settlement of Bridge Approaches	• Accuracy of design rules, Geo technical aspects, Team work, Compaction Control, Repairable slab • No movement due to temperature changes, Drainage, Backfill Materials, Thorough inspection	Synthesis Report
11.	ODOT (Snethen et al., 1998)	Construction of CLSM approach embankment to minimize the bump at the end of the bridge	• The use of Control Low-Strength Material (CLSM) as an approach embankment fill material as a simple and cost effective method to reduce the potential for developing the bump at the end of the bridge	Research Report
12.	SDDOT (Reid et al., 1999)	Use of fabric reinforced soil wall for integral abutment bridge end treatment and investigate the effectiveness of present design	• Voids reduced by using the rubber tire chips behind the integral abutment • Cyclic movements do not affect the voids	Research Report
13.	VTRC (Hoppe et al., 1999)	Survey done to create guidelines for the use, design, and construction of bridge approach slabs to minimize differential settlements	• Full width approach slab reduces erosion of approach fill, consequently reduces differential settlement • Pre-cambering phenomenon is done to reduce differential settlements beneath the bridges due to differing foundations	Synthesis Report
14.	VTRC/VDOT (Hoppe, 1999)	Guidelines for the use, design, and construction of bridge approach slabs	• Full-width approach slabs are used. It reduces erosion of the approach fill • Placing approach slabs below the road surface facilitates resurfacing operations • Drainage system between the top of the approach slab and the surface of the road should be provided • Pre-cambering may be employed to compensate differential settlement at bridge approaches resulting from differing foundations beneath the bridge and the roadway	Research Report
15.	CDOT (Abu-Hejleh et al., 2001)	Design & Performance studies on GRS wall to support bridge embankment and approach road	• GRS walls were designed to support shallow foundations of the bridge structure • Monitored movements were significantly smaller than expected movements in design	Research and Implementation Report

No.	Agency	Title/Work	Topics Covered and Salient Information	Remarks
16.	CDOT (Abu-Hejleh et al., 2001)	Results and Recommendations of Forensic Investigation of Three Full-Scale GRS Abutment and Piers in Denver, Colorado	• GRS abutment and piers are practical alternatives used in bridge support • GRS should not be used in a scour situation • GRS piers are suitable for remote locations, since it can be constructed or repaired by using small construction equipment within a few days	Research Report
17.	Kentucky Transportation Center, (Dupont, and Allen, 2002)	Survey on movements and settlements of highway bridge approaches	• Lowered approach slabs with asphalt overlays • Require settlement periods and/or surcharges prior to final construction • Design maintenance plans concurrent to construction plans • Implement specifications for select fill adjacent to abutments • Improve drainage designs on and around approaches • Reduce the side slope of embankments and improve approach slab design	Synthesis Report
18.	NJDOT (Nassif, 2002)	Finite element modeling of bridge approach, transition slabs using ABAQUS, and identifying the probable cause of cracking	• Increasing the concrete compressive strength and the steel reinforcement yielding stress, approach, and transition slab thickness • Settlement and void development coupled with heavy truckloads are the most probable factors causing crack development	Research Report
19.	TxDOT (Ha et al., 2002)	Investigation of settlement at bridge approach slab Expansion joint: Survey and site investigations	• The number one reason for the bump is the settlement of the embankment fill followed by the loss of fill by erosion • The settlement at the bridge approach is worse when the embankment is high and the fill is clay • The settlement at the bridge approach is lessened when an approach slab is used and the abutment fill is cement stabilized	Synthesis Report
20.	VTRC/VDOT (Arsoy et al., 2002)	Performance of Piles Supporting Integral Bridges	• Steel H-piles oriented in the weak-axis bending area is a good choice for support integral abutment bridges • Pipe Piles will cause higher stress in the abutments than steel H-piles • Concrete piles are not a suitable choice. Tension cracks due to cyclic lateral load can reduce their vertical load capacity	Research Implementation Report

No.	Agency	Title/Work	Topics Covered and Salient Information	Remarks
21.	TxDOT (Seo, 2003)	The bump at the end of the bridge: an Investigation	• The compressibility of the soil is contributing to the development of the bump • The transition zone of the approach embankment is about 12 m with 80 percent of the maximum settlement occurring in the first 6 m for a uniform load case • The size of the sleeper slab and support slab influences the settlement of the slab. The optimum width of both slabs is 1.5 m • A single-slab at least 6 m long and 0.3 m thick is recommended for an approach slab	Synthesis and Research Report
22.	MoDOT (Luna, 2004)	Evaluation of Bridge Approach Slabs, Performance and Design	• Slopes of embankment should be flattened to 2.5H:1V • A material low in fines content should be used for abutment embankments • If the embankment fill material has a plastic limit greater than 15-20, the soil should be treated • A geosynthetic reinforced backfill behind the abutments reduces the lateral loads on the bridge structure, adds confinement of the fill soils and increases the stiffness of the embankment • The sleeper slab drain should be placed at an elevation below the bottom of the sleeper beam and specify at least 2 ft of crushed or shot rock beneath the sleeper beam and approach slab • Shallow foundations will make the bridge foundation less expensive and more deformation compatible with the embankment earth structure	Research Report
23.	Iowa DOT (Mekkawy et al., 2005)	Simple Design Alternatives to Improve Drainage and Reduce Erosion at Bridge Abutments	• Three alternatives are recommended to improve drainage and alleviate erosion: 1) use geocomposite drain with granular backfill reinforcement, 2) use tire chips behind the bridge abutment, and 3) use porous backfill material	Research Report
24.	Iowa DOT (White et al., 2005)	Identification of the best practices for design, construction, and repair of bridge approaches	• Use porous backfill behind the abutment and/or geo composite drainage systems • Use a more effective joint sealing system at the joint between road and bridge approach • Reduce time-dependent post construction settlements	Research Report

No.	Agency	Title/Work	Topics Covered and Salient Information	Remarks
25.	LTRC/LADOT (Cai et al., 2005)	Determination of interaction between the bridge concrete approach slab and embankment settlement. The Finite element analysis was carried out in the present study	• After settlement is increased to a larger value, it no longer affects the performance of slab since approach slab completely loses its contact with soil and becomes a simple beam • The developed procedure can be used in designing the approach slab to meet the established deformation requirements • Due to over stress of bolts and dowel bars, cracking is seen	Research Report
26.	TxDOT (Jayawickrama et al., 2005)	Water intrusion in base/subgrade material at bridge ends	• Saturated base/subgrade material at the end of bridge could be a major problem • Use of geotextiles fabric beneath the joints to avoid loss of material by erosion • Approach slab stabilization to control void development and cross/slot stitching of approach slabs and concrete pavements for controlling further development of cracks	Research Report
27.	VTRC/VDOT (Hoppe, 2005)	Field Study of Integral Backwall with Elastic Inclusion	• An elastic inclusion consisting of a layer of elasticized Expanded Polystylene (EPS) 0.25 m significantly reduced earth pressures and approach settlements at the semi-integral bridge • The well-compacted select backfill material at bridge approaches is necessary • Short approach slabs could be sufficient to provide a grade transition • Shorter approach slabs would be easier for the superstructure to push and pull during cyclic movements, and would exert less stress on the backwall if they settle • Thermally induced lateral movements of the superstructure may not be equal at both abutments	Research Report
28.	NMDOT (Lenke, 2006)	Settlement Issues – Bridge Approach Slabs	• MSE walls have fewer problems with approach slab settlement issues than other types of bridge abutment systems	Research Report

No.	Agency	Title/Work	Topics Covered and Salient Information	Remarks
29.	CDOT (Abu-Hejleh et al., 2006)	Flowfill and MSE bridge approaches: Performance, Cost and Recommendations for Improvements	• Flowfill is recommended in certain difficult field conditions (e.g., to fill and close up voids, in areas where compaction is difficult, easier to place around an embankment slope) • The use of the MSE or GRS abutment system is the best system to alleviate the approach bridge bump problem • The high quality backfill materials should be placed under the sleeper slab • The length of the approach slab should be related to the depth of the abutment wall and the magnitude of the projected post-construction settlements • The drainage system is very important to collect and drain any surface water before it reaches and softens the soil layers located beneath or around the sleeper slab	Research Report
30.	VDOT (Hoppe, 2006)	Field Measurements on Skewed Semi-Integral Bridge with Elastic Inclusion: Instrumentation Report	• Data obtained by monitoring earth pressure cells, load cells, and strain gages would be useful for future endeavors	Research Report
31.	Iowa DOT (White et al., 2007)	"Underlying" Causes for Settlement of Bridge Approach Pavement Systems	• Void development from backfill collapse following saturation, severe backfill erosion, poor surface and subsurface water management, and poor construction practices mainly contribute to settlement problems of the approach pavements of bridges • Erosion can lead to problems including: exposure of the H-piles, failure of the slope protection cover, severe faulting in the approach pavement, and loss of backfill around subdrain elements • Problems in void development, water management, and pavement roughness were generally more pronounced with integral abutment bridges than non-integral • Backfill materials should be placed outside the range of bulking moisture contents and should be less susceptible to erosion • The surface water management system should be designed to shed water to the base of the embankment and the subsurface drainage system to provide an easy pathway for infiltrating water to escape	Research Report

No.	Agency	Title/Work	Topics Covered and Salient Information	Remarks
32.	California DOT (On going)	Replacement Alternatives for Deteriorated Approach Slabs	• Using test sections	Under Research (Structures Group)
33.	WVDOT	Study of Bridge Approach Behavior and Recommendations on Improving Current Practice	• NA	Synthesis Report

6. MITIGATION TECHNIQUES RANKING ANALYSIS

A non-parametric ranking analysis was performed to rank a few of the techniques presented in this synthesis. The presented methods are collected into two groups. The first group focuses on novel methods used for foundation and fill improvement and these methods include Deep Soil Mixing (DSM), Continuous Flight Auger (CFA) piles, MSE wall, and other methods, and the second group deals with techniques normally used for approach slab maintenance such as Hot Mix Asphalt (HMA) overlays, slab replacement, Urethane injection and others. Four criteria including 'Technique Feasibility,' 'Construction Requirements,' 'Cost Considerations,' and 'Overall Performance' are considered, and for each criterion a ranking was assigned to each method.

For technique feasibility, three levels of ranking (shown in parentheses) were considered, and these were: (1) they have been already implemented and proven as well design methods; (2) technique is effective but still under research; (3) and they are ineffective. Table 11 presents the ranks given for the methods listed in each group. All methods of the first group are novel and yet to be evaluated and hence they are assigned a rank of two (2). Ranks given in Group Two are also presented in the same table.

Three criteria used in 'Construction Requirements' are: (1) requires mobilization of heavy equipment; (2) and requires quality control during construction. Cost ranking was based on the costs of the construction for performing the field work. The last factor for the ranking analysis is based on the Overall Performance of each method. This rank was based on the available literature. Table 11 presents all these ranks for each method.

In conclusion, after each mitigation method has been considered and analyzed according to the four criteria, the mitigation techniques were ranked. The results show that for the novel foundation and fill improvement, six methods show early promise and can be recommended to be evaluated in this research, while for the maintenance measures the mud/slab jacking, grouting and Urethane injection showed promise and hence considered for further research evaluation.

Table 11 – Ranking Analysis of Mitigation Techniques for Bridge Approach Settlement

New or Maintenance Measure	Mitigation Method	Technique Feasibility (a)			Construction Requirements (b)			Cost Considerations (c)			Overall Performance (d)			Is this method recommended for present research?
		Ineffective	Effective but under research	Proven, well design method	Low	Medium	High	Low	Medium	High	Not proven	Ineffective	Effective	
Novel Methods for Foundation and Fill Improvement	MSE Walls/GRS		✔				✔			✔	✔			✖
	Geofoam		✔			✔			✔		✔			✔
	Lightweight Fill		✔			✔			✔		✔			✔
	Flowable Fill		✔		✔				✔		✔			✔
	DSM		✔		✔				✔				✔	✔
	CFA		✔			✔			✔		✔			✔
	Concrete Injection Columns		✔			✔				✔	✔			✖
	Geopiers		✔			✔			✔				✔	✔
Maintenance Measures	HMA Overlay			✔		✔		✔				✔		✖
	Mud/ Slab Jacking		✔		✔			✔					✔	✔
	Slab Replacement			✔		✔			✔			✔		✖
	Grouting			✔	✔				✔		✔			✖
	Urethane Injection		✔		✔					✔	✔			✔

a – Whether the method is in the research stage or the implementation stage; b – Difficulties in construction; i.e., the need of using heavy equipment; c – Costs vary from low to high based on material, equipment and mobilization costs

7. TXDOT DISTRICTS' SURVEYS

The last section of this synthesis report focuses on each TxDOT Districts' practices with respect to this approach settlement problem. As part of this research (Task 2), a survey of all the Districts in TxDOT was performed to collect and understand the problems encountered and the solutions used to minimize the bumps at the end of the bridges.

The researchers distributed a survey questionnaire to all 25 Districts and a total of 16 District responses were received. In a few cases, responses from different engineers from the same District were received. All these results were tabulated and analyzed in the following sections:

Q1. Have you encountered bridge approach settlement/heaving problems in your District?

Figure 75 presents that 17 out of 18 Districts (94 percent) have encountered the bridge approach settlement. Among the 17 Districts, 6 Districts (33 percent) have experienced both settlement and heaving problems, while 11 Districts (61 percent) have only encountered the bridge approach settlement. The Odessa District reported that they have no problems either with bridge approach settlement or heaving.

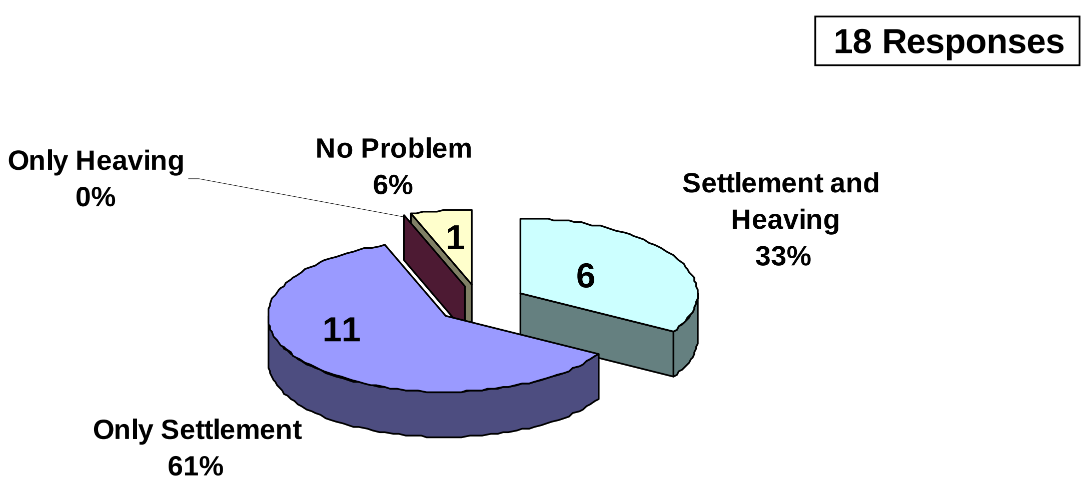


Figure 75 – Number of Districts that Encountered Bridge Approach Settlement/Heaving

Q2. Please select the procedure followed to identify this problem in the field.

Figure 76 presents further responses from 17 Districts, who noted the bridge approach settlement/heaving. These responses related to the procedures followed for identifying the heave/settlement problem at the bridge approaches. The majority of them noted this problem from visual observations. Some other forms of identification of this problem were through evaluation of rideability and from the received public complaints as mentioned by 15 and 10 Districts, respectively. Only two Districts have reported that they have used Rideability (International Roughness Index) measurements whereas three other Districts have noted that they used other methods including notification from Maintenance Offices to identify the problem.

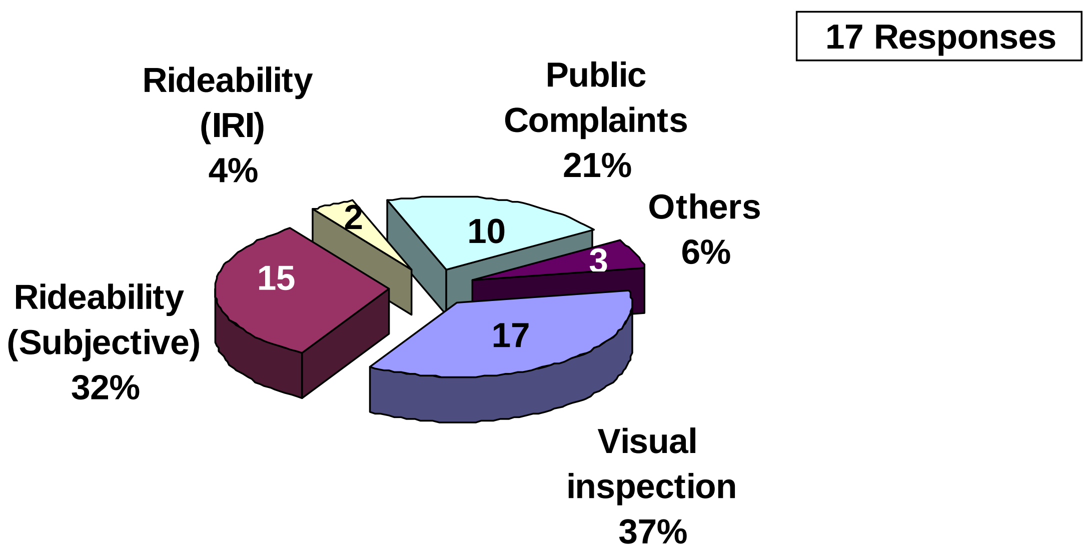


Figure 76 – Procedure to Identify the Problem in the Field

Q3. Have you used TxDOT Item 65 for bridge rating assessments?

Based on Figure 77 responses, 47 percent of 17 Districts noted that they have not used TxDOT Item 65 for bridge rating assessments, while 41 percent replied that they have used the method. Two Districts did not answer.

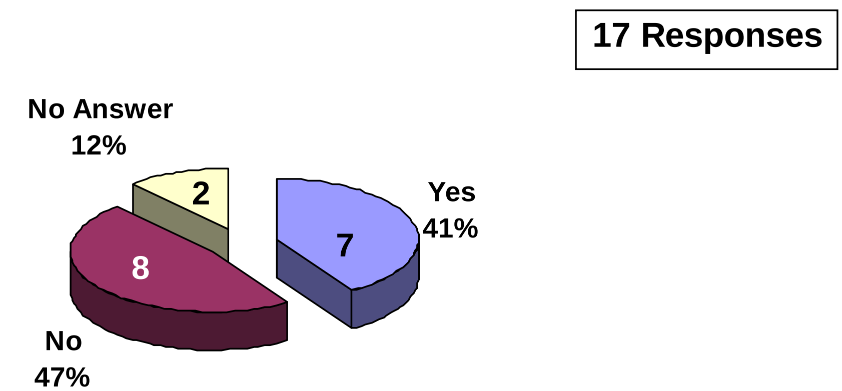


Figure 77 – Number of Districts that Use TxDOT Item 65 for Bridge Rating Assessments

Q4. Have you conducted any forensic examinations on the distressed approaches to identify potential cause(s) of the problem?

For the question related to whether a District has conducted any forensic examinations on the distressed approaches to identify potential cause(s) of the problem, most of Districts (53 percent) reported in the negative (Figure 78).

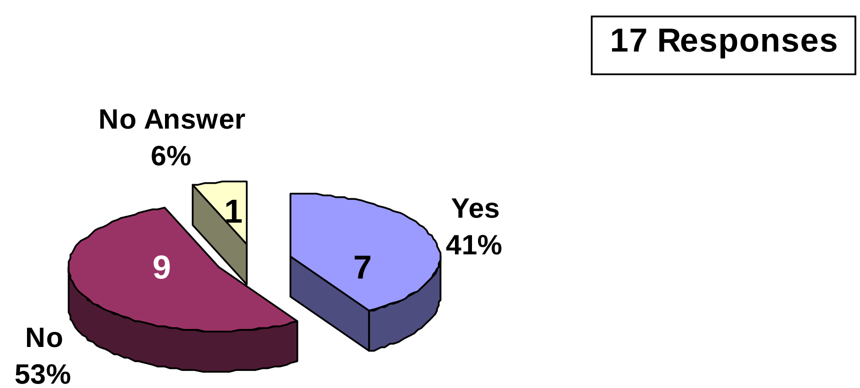


Figure 78 – Number of Districts that Conducted Any Forensic Examinations on the Distressed Approaches to Identify Potential Cause(s) of the Problem

Q5. In your opinion, what would be the major factor contributing to the approach settlements in your District? (If necessary, Please check more than 1 choice)

Figure 79 shows various factors that the Districts attributed to the settlement or heaving problem. It should be noted that the Districts were asked to select more than one response. As a result, the total responses do not total 17. The following summarizes each of the factors and the number of responses received:

- Natural subgrade: 6 responses
- Construction practices: 13 responses
- Drainage and Soil erosion: 12 responses
- Void formation: 10 responses
- Compaction of Fill: 15 responses
- Others: 3 responses

Other responses received included poor design in old practices and sulfate problems.

Figure 79 – Factors Attributed to the Approach Settlement Problems

Q6. Do you perform any geotechnical investigations on embankment fill and foundation subgrade material?

Fifty nine percent (59 percent) of the respondent Districts noted that they typically perform geotechnical investigations on fill and foundation subgrades (Figure 80).

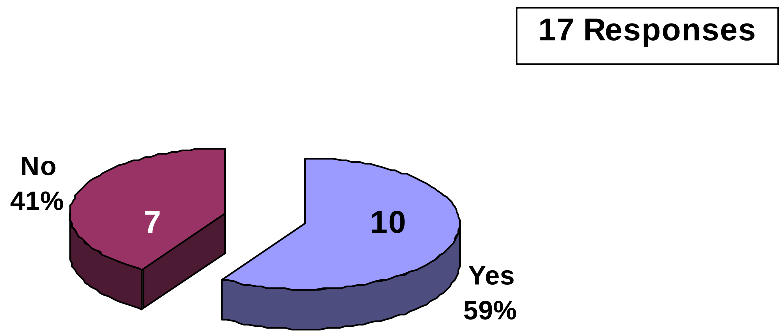


Figure 80 – Number of Districts that Perform a Geotechnical Investigation on Embankment Fill and Foundation Subgrade Material

Q7. Please list the PI requirement of the embankment material to be used as a fill material?

Figure 81 shows various PI specifications listed by the Districts that they followed in the selection of embankment fill material. As per Figure 81, the maximum PI of the fill material used by select Districts was around 40 while most of them required it to be less than 25.

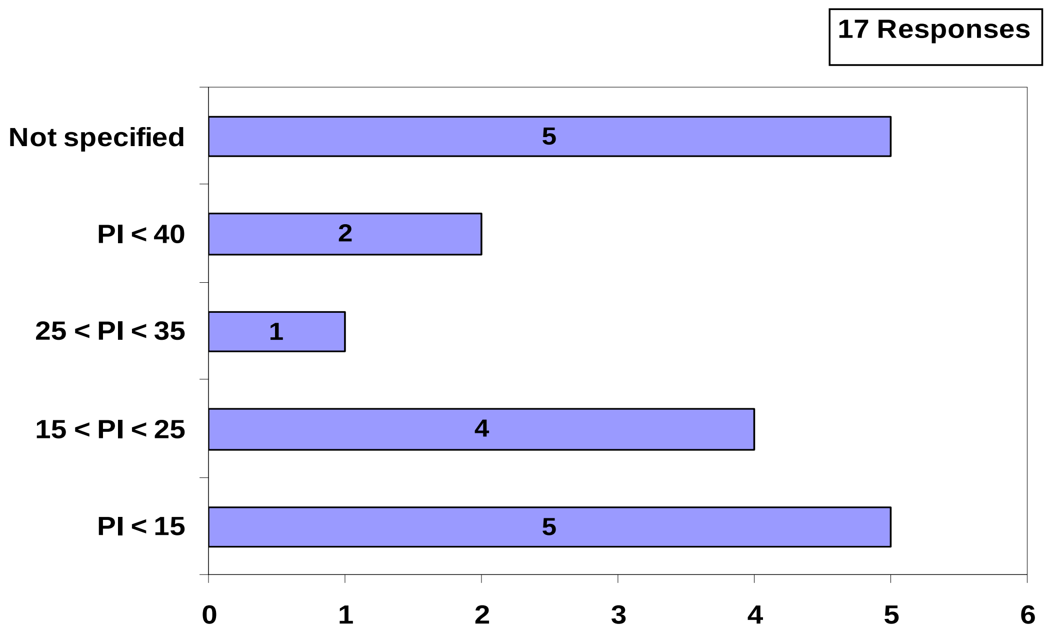


Figure 81 – PI Value Required for Embankment Material

Q8. Are there any Quality Assessment (QA) studies performed on compacted fill material?

Figure 82 presents Districts' responses related to Quality Assessment (QA) studies performed on compacted fill material; Figure 81 results show that 15 out of the 17 Districts (88 percent) noted that they have used the Nuclear Gauge for compaction Quality Assessment (QA) studies. Seven Districts used sampling and laboratory testing, while only one District used the dynamic cone penetrometer (DCP) for the same purpose.

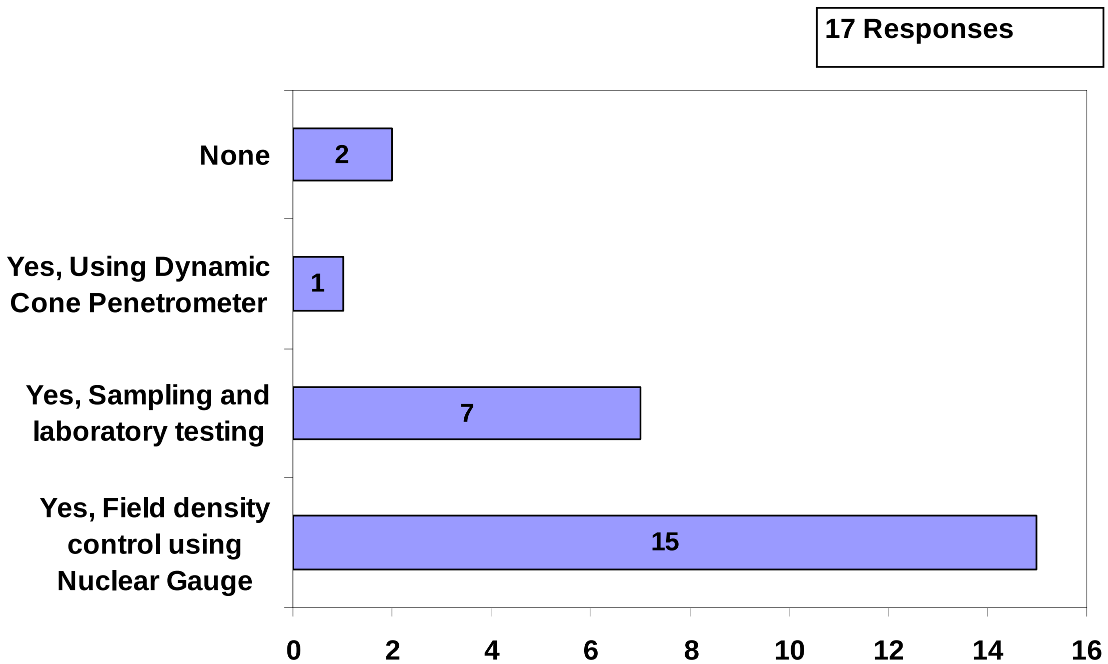


Figure 82 – Number of Districts Conducting Quality Assessment (QA) Studies on Compacted Fill Material

Q9. List the number of bridge approach slab related repair/maintenance works that have occurred in the District.

Figure 83 lists the number of maintenance jobs that were taken up by the Districts. The results show that the number of repair jobs varied across a wide range with a few of them listing less than 5 to some mentioning above 20.

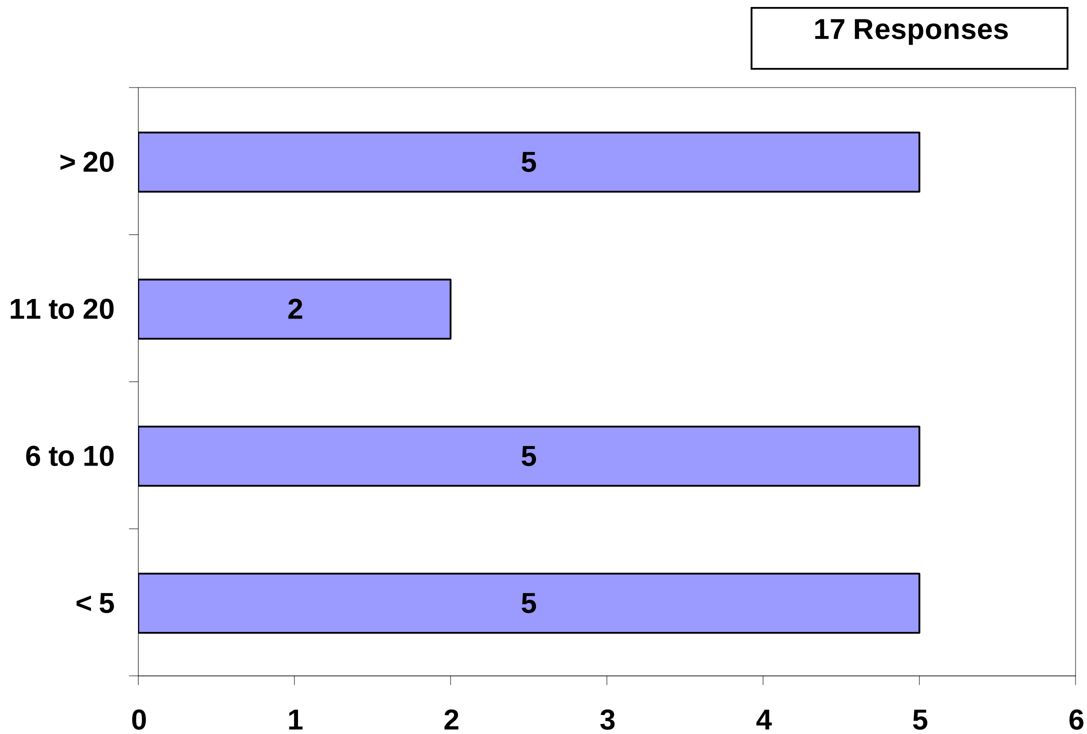


Figure 83 – Number of Bridge Approach Slab Related to Repair/Maintenance Work in the District

Q 10-11. Please check the remedial/maintenance measures taken in your District. (Please check more than one choice)

Figure 84 lists various remediation methods used by the Districts to repair the heave/bumps. Survey results revealed that the level-up or milling of the approach slab is a frequently used maintenance measure by the majority of the TxDOT Districts (17 out of 18 respondents, 94 percent). With respect to its performance, only 3 Districts noted that this method is working well, 8 Districts as good and 6 Districts as fair. Use of Urethane injection was the second choice by the Districts as 10 Districts (55 percent) have selected this as their remedial measure. With respect to its performance, Districts rated this technique as a very well (2 Districts), good (2 Districts) and well (2 Districts), while 4 Districts rated this method as fair. Other remedial measures include reconstruction of the approach slab, treatment of the subgrade, chemical treatment of the backfill, and the installation of effective drainage and reinforced backfill material. Performance rating of these methods is listed in the same figure. Two other Districts responded that they have employed other methods such as pressure grouting and cement stabilized sand.

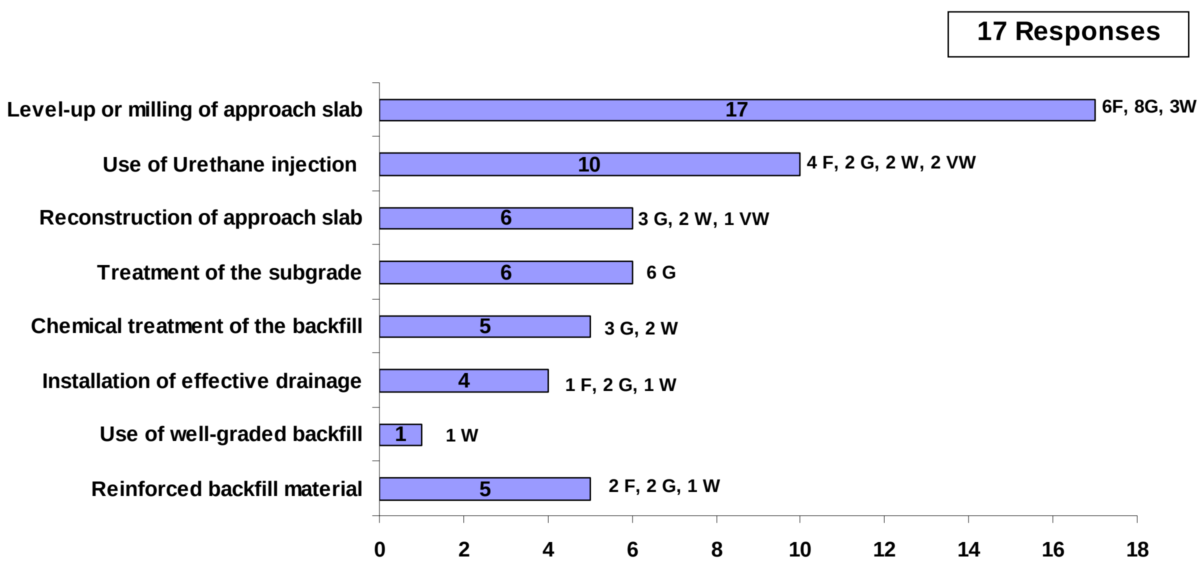


Figure 84 – Remedial/Maintenance Measures Taken in Responded Districts and its Performance

(Note: VW – Very Well; W – Well; G – Good; F – Fair)

Q12. Do you have any specific recommendations for fill material used for embankments?

Table 12 gives the information that controlling the PI value is the most recommended method given by the Districts in this survey, either by using chemical treatment in the subgrade and backfill or by using density control compaction. The other recommendations are using rock embankment under the approach slab, select fill material, using two sacks of concrete at approach and backwall and even quality control during embankment construction.

Table 12 – Recommendations for Fill Material Used for Embankments

District	Recommendations for fill material used for embankments
Abilene	PI $\leq$ 15, or lime treat to reduce PI$\leq$ 15
Austin	1. Use rock embankment under the approach slabs to prevent settlement issues with success. 2. PI requirements to insure non-plastic materials.
Brownwood	1. Select fill for drainage behind abutment walls. 2. Cement or lime treat subgrade.
Dallas	Graded backfill material with PI 10 to 25 with density controlled compaction
El Paso	2 sacks of concrete at approach slabs and backwall
Fort Worth	1. Test embankment for compliance with requirements at beginning of the bridge, end of the bridge, and at 25' intervals for a distance of 150' from each bridge end. 2. Embankments are supposed to be constructed to the final subgrade elevation prior to the excavation for abutment caps and approach slabs. 3. Additional density testing of roadway embankments near bridges.
Houston	1. Lower the LL/PI 2. Good compaction 3. Cement stabilized backfill
Laredo	Item 132
Pharr	Cement stabilized backfill
Waco	Cement stabilized backfill

Q13. Has your District implemented any remedial methods to control the erosion/slope failure problems?

Figure 85 shows that 5 out of the 17 Districts (30 percent) responding have employed Turf growth and Geosynthetics methods to control the erosion/slope failure problems, while 5 other Districts have implemented only Geosynthetics to manage the problem. Six Districts have done nothing and some Districts have chosen other methods, such as, rock riprap, flatten the slope, flexible reinforcement, improve drainage, water intrusion, and erosion control. Nevertheless, none of 17 Districts has chosen the baling method to control the problem.

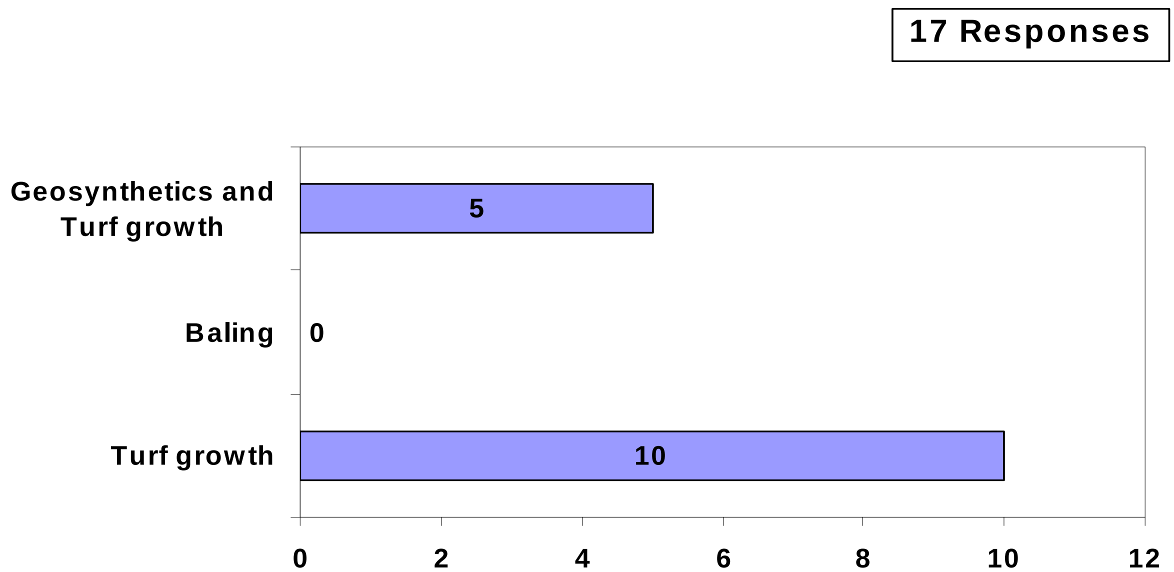


Figure 85 – Methods to Control the Erosion/Slope Failure

Q14. Do you have any maintenance related approach slab repair activity coming up?

Table 13 – Maintenance Work to Approach Slab in the Next Year

District	Any maintenance related approach slab repair activity coming up Yes, where
Brownwood	1. Brady, Texas, Brady creek bridge. We are adding approach slabs to the structure on US 377 to help anchor a rotating abutment. Job will start about June 1, 2008. 2. Adding approach slabs to US 283 bridge over Jim Ned creek to push joint issues away from bridge deck.
El Paso	Reconstruct approach slab
Laredo	IH 35 (RMN 74 - 82) La Salle Co
Waco	FM 1947 at Aquilla Lake

Q15. Do you anticipate any new bridge construction in your District in the next year?

Table 14 – New Bridge Construction in Each District in the Next Year

District	Any new bridge construction in your District in the next year Yes, where
Abilene	US 84/BI-20 @ FM 3438
Austin	SH 45
Brownwood	US 67 Comanche
Bryan	FM 1915 at Lipan and S. Elm Creek, Milam County; SH 6 at BS 6, Brazos County; Varios Off-sytem bridges
Childress	12 bridges, Districtwide
Dallas	SH 121, SH 161, SH 274, IH 35E, US 75, US 380, FM 2499
El Paso	Spur 601
Fort Worth	Spur 303 at Village Creek (2208-01-051) FM 1885 at Dry Creek (0649-02-028) FM 1191 at Board Tree Creek (1333-03-016)
Laredo	DC in Laredo under construct., OP in Eagle Pass
Lufkin	SH 94 at Neches River and Reliefs in Angelina & Trinity Counties (0319-04-066) Long King Creek and Reliefs in Polk County (1193-02-019) Barnett Creek in Polk County (1193-02-020)
Odessa	BI 20 E at the intersection of JBS Parkway
Pharr	FM511
Wichita	Many
Yoakum	FM 1823 @ West Carancahua Creek in Jackson Co

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APPENDIX A:
TxDOT Research Project 0-6022
"Survey on Bridge Approach Settlements"

Bridge approaches are designed to provide smooth and safe transition of vehicles from highways to bridge pavements and vice versa. Settlement and heave related movements of embankment materials under bridge approach slabs relative to bridge pavements create a dip/bump in the roadway. This uneven transition causes pavement damage, unacceptable ride quality, potential loss of vehicle control, reduced speed from driver uncertainty of bump severity, increased maintenance costs, user delay from roadway repairs, and loss of public image. TxDOT recently initiated Research Project 0-6022, titled "Recommendations for Design, Construction, and Maintenance of Bridge Approach Slabs," with objectives of summarizing current state-of-the-art methods and then studying the effectiveness of promising methods/techniques to control settlement/bump problems on select bridges in the State.

As part of this research, the UTA and UTEP research teams have prepared the following short survey on bridge approach dip/bump problems. The main intent of this survey is to identify the severity of problems experienced by TxDOT, remedial steps taken so far to mitigate them, and probable sites for possible implementation with the proposed mitigation methods.

We request a small portion of your valuable time to assist us in our research work for TxDOT by filling in the following 16 questions.

"Districts Survey on Bridge Approach Settlements for TxDOT Project 0-6022"

Name: **District:**

Title: **Office:**

Please click or check (with X) to the following questions. We thank you in advance for your input.

1. **Have you encountered bridge approach settlement/heaving problems in your District? (Please check more than one choice)**

   ☐ Yes, settlements ☐ Yes, heaving ☐ No

   If the answer to the above question is **NO**, then please move forward to Question No. 15.

2. **Please select the procedure followed to identify this problem in the field?**

   ☐ Visual inspection

   ☐ Rideability (subjective)

   ☐ Rideability (Pavement Roughness Index or IRI Measurements/Profilograph)

   ☐ Public Complaints

   ☐ Others, specify_________________

3. **Have you used TxDOT Item 65 for bridge rating assessments?**

   ☐ Yes ☐ No ☐ Others, specify___________

4. **Have you conducted any forensic examinations on the distressed approaches to identify potential cause(s) of the problem?**

   ☐ Yes ☐ No

   If your answer is **YES**, please specify the method (reevaluation of geotechnical properties of fill, reevaluating the design methods, field instrumentation, surveys) followed for performing forensic examination: _______________________________

5. **In your opinion, what would be the major factor contributing to the approach settlements in your District? (If necessary, please check more than 1 choice)**

   ☐ Natural subgrade ☐ Compaction of fill ☐ Void formation

   ☐ Drainage/soil erosion ☐ Construction practices ☐ Others, specify___________

6. **Do you perform any geotechnical investigations on embankment fill and foundation subgrade material?**

☐ Yes, on both ☐ Yes, on subgrade ☐ Yes, on embankment fill
☐ No

7. **Please list the PI requirement of the embankment material to be used as a fill material?**

Recommended PI value(s): ___________

8. **Are there any Quality Assessment (QA) studies performed on compacted fill material?**

☐ Yes ☐ No

If YES, Please list them:

☐ Field density control of each fill using Nuclear Gauge

☐ Sampling and laboratory testing (please give an example)

☐ Indirect methods using Dynamic Cone Penetrometer (DCP)

☐ Others, Specify ______________________________

9. **List the number of bridge approach slab related repair/maintenance work that has occurred in the District?**

☐ < 5 ☐ 6 to 10 ☐ 11 to 20 ☐ > 20

10. **Please check the remedial/maintenance measures taken in your District? (Please check more than one choice)**

☐ Level-up or milling of approach slab ☐ Reconstruction of approach slab

☐ Use of Urethane injection ☐ Use of well-graded backfill

☐ Reinforced backfill material ☐ Installation of effective drainage

☐ Chemical treatment of backfill ☐ Treatment of subgrade

☐ Other, specify___________

11. What is the post-performance of the mitigation method implemented in your District from those checked in No. 10 above?

Method: ___________

☐ Very well ☐ Well ☐ Good ☐ Fair

Method: ___________

☐ Very well ☐ Well ☐ Good ☐ Fair

Method: ___________

☐ Very well ☐ Well ☐ Good ☐ Fair

Method: ___________

☐ Very well ☐ Well ☐ Good ☐ Fair

12. Do you have any specific recommendations for fill material used for embankments?
Specify: ____

13. Have you implemented any remedial methods to control the erosion/slope failure problems in your District?

☐ Turf growth ☐ Baling ☐ Geosynthetics

☐ Others, Specify ___________________________________

14. Do you have any maintenance related approach slab repair activity coming up?

☐ Yes ☐ No

If the answer is YES, please specify the location: ______________________________

15. Do you anticipate any new bridge construction in your District in the next year?

☐ Yes ☐ No

If your answer is YES, Please specify the location: ______________________________

16. We would like to contact you if we have any follow-up questions. Please list your email or phone number where we can reach you.
Email: Tel:

We thank you very much for your input. We request that survey responses be emailed to anand@uta.edu (as a scanned copy) or mailed to: Anand J. Puppala, PhD, PE, Professor, Box 19308, Department of Civil and Environmental Engineering, The University of Texas at Arlington, Arlington, TX 76019, USA.

APPENDIX B:
NEW RAPID TEST PROCEDURE FOR MATERIAL QUALITY AND FIELD COMPACTION

Rapid Test Procedure to Verify Field Compaction

During the last two decades, several rational procedures to design pavements including the current Mechanistic-Empirical Pavement Design Guide (M-EPDG) were introduced by the National Highway Research Program. Most of these methods consider the foundation layer modulus, strength, and permeability properties of the compacted materials as the material response parameters (Rahman et al., 2007). As per Holtz and Kovacs (1981), dry density and moisture content correlate well with the engineering properties such as strength, stiffness, compressibility, and permeability, and thus they are conveniently used as construction control parameters in the field.

It is well known that compaction of the underlying plays an important role in the performance of pavements. Achieving a good compaction of the subgrade would provide sufficient strength and stiffness for the upper layers of the pavement. Reaching this target is not a simple task due to material heterogeneity, difficulty in maintaining prescribed moisture content and lift thickness, and variability in equipment and operators of the compactors. These are the reasons why quality control (QC) and quality assurance (QA) play a major role in the construction process (Labuz et al., 2008).

There are many different methods available to control the embankment compaction in the field. They can be either destructive or nondestructive type field test methods. Both of these QC methods are fully based on laboratory compaction tests. Destructive methods involve in excavation and replacement of some of the compacted fill material whereas nondestructive methods indirectly measure either density or stiffness and moisture content of the compacted fill. The destructive methods such as the coring method, sand cone method, and rubber balloon method are proven to be time consuming and at times they may not provide accurate results when the borrow materials are highly variable (Holtz and Kovacs, 1981). On the contrary, nondestructive methods including nuclear or density gauges can provide timely onsite results. Other devices such as the dynamic cone penetrometer and the light weight deflectometer are also used for QC and QA. Following Table B1 summarizes all the field compaction control methods (QC/QA) available to verify the achieved degree of compaction. Figure B1 shows the different types of compaction control methods.

Table B1 – Test Methods for QC/QA

Test method	Soil property	Test type
Sand cone	Density and moisture content	Destructive
Drive core	Density and moisture content	Destructive
Nuclear Gauge	Density and moisture content	Non-destructive
Dynamic cone penetrometer	Strength	Semi-destructive
Soil Stiffness Gauge	Stiffness modulus	Non-destructive
Light weight deflectometer	Stiffness modulus	Non-destructive


(a) Coring **(b) Nuclear gauge**


(c) Light weight deflectometer **(d) Stiffness Gauge**

Figure B1 – Different QC/QA Test Methods for Compaction Control

(White et al., 2007)

The sand cone and drive core methods require laboratory determined dry density of the soil to evaluate the in-situ compacted dry density. Otherwise the results from these tests will be erroneous to check the degree of field compaction. Other problems with these destructive methods are associated with the determination of the volume of the excavated material when the compacting materials are of the gravel type. The nuclear gauge apparatus uses Gamma radiation where the amount of Photon and Neutron scatter determines the density and water content of the compacted soil. Calibration of the nuclear gauge with known compacted materials is always an issue. In addition, the choice of the nuclear gauge system demands skilled and authorized personal to operate the system. Not doing so can easily produce erroneous values.

The dynamic cone penetrometer is a semi-destructive type device that provides the strength characteristics of pavement layers. The test involves dropping a 17.6 lb (8 kg) hammer from a height of 2 ft. (575 mm) and measuring the penetration rate of a 0.8 inch (20 mm) diameter cone. The penetration index, usually denoted as the dynamic cone penetration index (DPI) which typically has units of inch per blow, is inversely related to the penetration resistance of the material. Several researchers have discussed the dynamic cone penetration testing (Burnham and Johnson, 1993; Gabr et al., 2000; Siekmeier et al., 2000; Gabr et al., 2001; Amini, 2004; Ampadu and Arthur, 2006). ASTM D 6951-03 specifies the following relationships between the DPI and CBR values:

$$CBR = \frac{292}{DPI^{1.12}}$$, for all soils except for CH and CL soils with CBR < 10

$$CBR = \frac{1}{(0.017019DPI)^2}$$, for CL soils with CBR < 10

$$CBR = \frac{1}{(0.002871DPI)}$$, for CH soils

The soil stiffness gauge, sometimes called GeoGauge, is a non-destructive type device used to measure the in-situ deformation characteristics of the compacted soil. This device rests on the soil surface and vibrates at 25 frequencies ranging from 100 to 196 Hz (Meher et al., 2002; White et al., 2007).

The vibrating device produces small dynamic forces and soil deflections, from which soil modulus can be calculated as:

$$E_{sgg} = \frac{F}{\delta}\frac{(1-\nu^2)}{(1.77R)}$$

where E_{sgg} = Modulus of soil obtained from soil stiffness gauge
F = Dynamic force caused by the vibrating device
δ = Deflection measured with a geophone
ν = Poisson's ratio
R = Radius of the annular ring of the device

The soil modulus is averaged over the 6-12 in. depth beneath the stiffness gauge. Once the modulus is calculated, the soil properties are obtained by a regression model developed by the manufacturer (Meher et al., 2002). Often, prior knowledge of the soil's dry density and moisture content are necessary to develop the model. Meher et al. (2002) reported that the use of the GeoGauge in compaction control has mixed results as the calibration equations are soil specific. The calibration equations developed by many researchers to induce the FHWA were compared with the soil that they tested. Several state DOTs including NMDOT, NJDOT, MODOT, and NYSDOT evaluated the performance of the stiffness gauge to control the compaction in the field. As reported by Meher et al. (2002), all these state DOTs experienced mixed results using the GeoGauge as compaction control method.

Another type of non-destructive method is a light weight deflectometer (LWD) which is used to determine the elastic modulus of the compacted soil. In this test method a 22-lb (10 kg) weight is dropped to produce a dynamic load on a plate. A load sensor measures the load pulse, and a geophone at the center of the plate measures the corresponding soil deflection. The soil modulus is then calculated using the relation:

$$E_{LWD} = \frac{f(1-\nu^2)\sigma_0 r}{h_0}$$

where, E_{LWD} = elastic modulus
ν = Poisson's ratio (ν = 0.40)
σ_0 = peak applied stress at surface
r = plate radius
h_0 = peak plate deflection
f = factor that depends on the stress distribution.

All these methods and devices have very low productivity means that only a small portion of the compacted area is tested (Labuz et al., 2008). As reported by Arasteh (2007), these methods have the following disadvantages:

1. Provides little or no on-the-fly feedback;
2. Density properties are not measured until after the compaction is complete; and
3. Density measurements may not be representative of the entire compacted area.

The above discussed devices and methods for QC/QA are typically used to assess less than one percent of the actual compacted area (TRB, 2008). All these factors contributed to the development of new compaction control methods that make use of the advent of computers such as the intelligent compaction method, and another method known as rapid impact compaction (RIC) method and both will be described in the following sections.

Intelligent Compaction (IC)

The term Intelligent Compaction (IC) refers to a compaction method that uses a vibrator roller that continuously measures and reports the stiffness of the material being compacted, while at the same time, it automatically adjusts its compaction effort by modifying the instantaneous settings such as force, amplitude, and frequency of the roller based on the measurements taken to avoid undercompaction or overcompaction (Moore, 2006; Camargo et al., 2006). The rollers are equipped with either accelerometers and/or machine energy meters to calculate an index parameter that is related to modulus, stiffness, or bearing capacity of the soil. The roller must also be equipped with a documentation system that allows for continuous recordation of the roller location and the corresponding stiffness related output. By integrating measurement, documentation, and control systems, the use of IC rollers allows for real-time corrections in the compaction process (Gallivan, 2008). Besides, IC technology provides an opportunity to collect and evaluate information for 100 percent of the project area (White et al., 2007).

Specifications for IC technology are not yet fully developed for all states but MnDOT has performed considerable amount of research in this area. To support the advancement of IC technology in the United States, the FHWA and 12 state DOTs including TxDOT have launched a new pooled-fund study, ‘Accelerated Implementation of Intelligent Compaction Technology for Embankment Subgrade Soils, Aggregate Base, and Asphalt Pavement Materials.’ TxDOT is currently attempting to collect data on lime treated soils as this has not been explored within the

United States to any extent. This attempt is being made due to the utilization of high amounts of treated material in its highway construction operations due to highly expansive and cohesive soils.

The following section introduces a new and novel compaction technique that could be considered for compacting inaccessible critical zones such as backfill very next to the bridge abutment or inside U-type abutments, which are otherwise very difficult to achieve the required degree of compaction with conventional rollers.

Rapid Impact Compaction (RIC) Technique

Rapid Impact Compaction (RIC) is an innovative and recently developed ground improvement method, which uses controlled dynamic compaction at a fast blow rate (Dumas et al., 2003). The RIC method was originally developed in the early 1990s in the United Kingdom for rapid repair of explosion damage to military airfield runways (Dumas et al., 2003; Kristiansen and Davies, 2004). This technique is comprised of a modified hydraulic piling hammer acting on a circular steel plate, which remains in contact with the ground during the treatment operation. As a result, the energy is applied more efficiently to the ground than in a conventional drop weight Dynamic Compaction (DC) process where the weight may fall on an irregular surface in such a way that much of the energy is dissipated in deforming the irregularities of the ground. The RIC method could be adopted to compact the fills where the accessibility is impossible for conventional type compactors (rollers). For example, compaction of backfill next to a bridge abutment or retaining structure is very difficult as these zones are inaccessible for conventional rollers. A portable rapid impact compactor may be adopted for such critical jobs.

Figure B2 – Rapid Impact Compactor Used for Compacting 13 ft Thick Sand Layer for a Building Foundation (www.terrasystemsonline.com)

The RIC typically employs a 7-ton hammer that is hydraulically raised to a maximum height of 4 feet and then allowed to free-fall. The tamper generally strikes the plate at a rate of 30 to 40 blows per minute. Table B2 summarizes the main characteristics of the RIC method.

Table B2 – Summary of RIC Specifications

RIC Specification	Quantity
Height of rig	25 ft
Length of rig	30 ft
Width of rig	12 ft
Approximate working weight	57 t
Ram weight	6 t or 7 t
Maximum drop	4 ft
Maximum energy	56,000 ft-lb
Blows per minute	30/40
Foot diameter	5 ft

Although RIC and deep dynamic compaction (DDC) methods are similar in that both utilize a falling weight to compact the ground, they have important differences. Table B3 shows the main differences between these methods.

Table B3 – Comparison of RIC against DCC

Specification	RIC	DCC
Tamper	7.5 t	20 t
Maximum drop height	4 ft	80 ft
Maximum energy per impact	60,000 ft-lb	3.2 million ft-lb
Maximum impact rate	30-40 blows per minute	2 blows per minute
Maximum energy per minute	2.4 million ft-lb	6.4 million ft-lb

In addition, other important difference is related to the manner in which the ground responds to treatment. The RIC method is a top-down process while DC is bottom-up process. The first few blows in RIC create a dense plug of soil immediately beneath the compaction foot. Further blows advance this plug deeper, which compacts soil to a deeper layer. This process progresses until small increments of penetration of the compaction foot can be achieved with increasing blows. Additional passes are typically offset from the primary pass to ensure effective treatment coverage. The effective depth of treatment in the case of RIC can also computed using the same equation that is used for the DC method.

$$d_{\max} = 0.5\sqrt{WH}$$

where, W = mass of the tamper
H = height of fall

The typical effective depth of improvement is around 10 ft (Dumas et al., 2003; Kristiansen and Davies, 2004). However, more depths of improvement (9 m to 10 m) were achieved in Asia (Kristiansen and Davies, 2004). Figure B3 compares the effective depth of improvement of both the DC and the RIC methods.

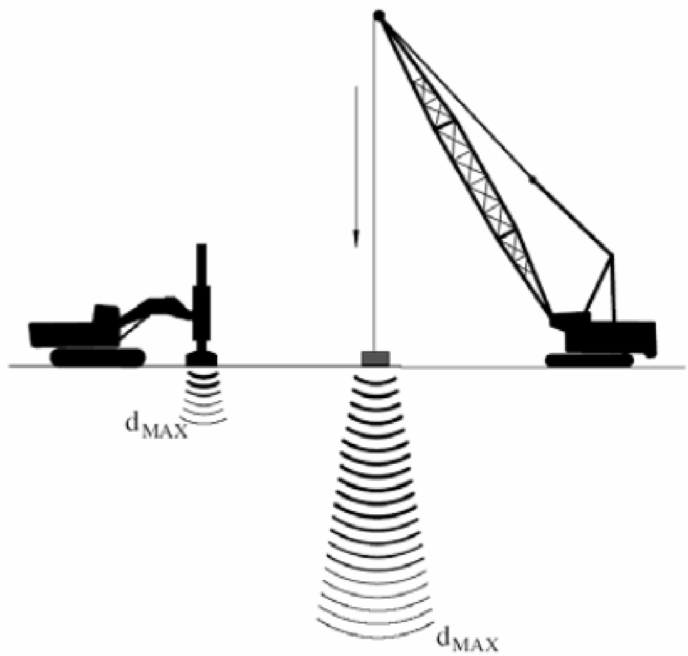

Figure B3 – Comparison of the Qualitative Improvement Achieved from Dynamic Compaction and Rapid Impact Compaction (TerraNotes)

The distribution of peak particle velocities, which represents the disturbance caused to the soil structure was not reported; however, the impact force applied is much smaller than DC, RIC can be employed to proximity of the existing structures. Visualizing the advantages of RIC, this method can be employed to compact the inaccessible backfill soils close to the backwall/bridge abutments and retaining structures. Further research is also necessary to evaluate various aspects of this method when adopted for such applications. The peak particle velocities and the direction of distribution of impact energy are critical issues.