Wisconsin Highway Research Program

Evaluation of Bridge Approach Settlement Mitigation Methods

Final Report

by

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Executive Summary

Extensive research has been conducted, in the past two decades, to study the causes and mitigation methods of bridge approach settlement. Bridge approach settlements cause unsafe driving conditions, rider discomfort, poor public perception of the state infrastructure, structural failure of bridges, and long-term maintenance costs. The literature has indicated that poor performance of pavement, bridge abutment and type, consolidation of the backfill materials, consolidation of the foundation's soils, and poor drainage are contributors to bridge approach settlement. Many mitigation techniques have been used to control the settlement, but the methods selected depend on the specific site. Specifying more stringent backfill materials and compaction requirements as well as providing proper drainage are effective ways in helping to alleviate the problem. Techniques to repair the bump include asphalt patching or overlays, slab jacking, and replacement of an approach slab.

The purpose of this study is to document the performance and effectiveness of two mitigation techniques, geosynthetic reinforced fill and flowable fill, installed behind four Wisconsin bridges. Two of the bridges (Hemlock and Cranberry bridges) are founded on granular soil foundations that are relatively incompressible. The other two bridges (Western Avenue and Beloit bridges) are founded on compressible foundations. This was done to investigate the effectiveness of the chosen mitigation techniques (geosynthetic-reinforced fill and flowable fill) in reducing approach settlements for two different foundation conditions: incompressible and compressible. There was no attempt to reduce the consolidation of the compressible foundation soils.

Based on the literature research, site visits and field test measurements of the four bridges, the following comparisons and conclusions can be made:

- The movements of the approach fills that have granular foundation soils (Hemlock and Cranberry) and less than 5 to 7 feet of fill were insignificant over five years compared with the movements of the approach fills (Western and Beloit) with cohesive foundation soils over two years.
- Embankment side slopes that settle and slough (Western and Beloit) resulted in erosion and/or movement of backfill material.
- The flowable fill and geosynthetic reinforced fill on granular soil foundations did not outperform the structure backfill (Hemlock and Cranberry).
- The flowable fill and geosynthetic reinforced fill on cohesive soil foundations did outperform the structure backfill (Beloit and Western).

More observations and recommendations for future research are presented at the end of this final report.

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1.0 INTRODUCTION

Over the past 20 years, extensive research has been conducted to study the causes and mitigation methods of bridge approach settlement or “the bump at the end of the bridge.” The bridge approach settlement is defined as “the difference in elevation of approach pavements and bridge decks caused by unequal settlement of embankments and abutments.” Many Departments of Transportation (DOTs) are significantly impacted by bridge approach settlement, as it causes unsafe driving conditions, rider discomfort, poor public perception of the state infrastructure, structural failure of bridges, and long-term maintenance costs.

The bump is noticeable with about ½-inch of differential settlement between the bridge and approach (Wahls 1990), becomes problematic at 1 inch (Zaman et al. 1994), and causes serious riding discomfort at about 2 to 2.5 inches (Stark et al. 1995). In lieu of specifying tolerable movement as total settlement, Wahls (1990) indicated that tolerable movement should be measured as differential settlement over span length. A slope of less than or equal to 1 inch per 250 feet (1/250) for continuous spans and 1/200 for simply supported spans was considered acceptable. Once the bridge approach settlement becomes unacceptable, DOTs need to repair, provide maintenance, or reconstruct the bridge approach.

Briaud et al. (1997) indicated that at least 25 percent of the 600,000 bridges in the US, or about 150,000 bridges, are affected by bridge approach settlement. Similar statistics were shown by other studies. The Stark et al. (1995) study reported that 27 percent of the 1181 bridges in Illinois had significant differential bridge approach movement and that adjacent states such as Iowa, Wisconsin, Michigan, Ohio, Indiana, Missouri, and Kentucky exhibited similar percentages. Ha et al. (2002) reported that 24.5 percent of the Texas DOT bridges indicated a bump. Another study conducted by Luna et al. (2003) for Missouri DOT (MoDOT) reported that 17 percent of the bridges exhibited bridge approach settlement and an additional 15 percent required remediation.

The cost of repairing the bump ranges from $60 to $187 million with an average of $100 million per year (Briaud et al. 1997 and Schafer and Koch 1992). Other statistics were gathered from Kentucky DOT, which spends about $1000 per bridge per year (Dupont and Allen 2002), and Texas DOT, which reported spending a total of about $6.3 million per year (Ha et al 2002). If the bridge needs to be replaced, which Briaud et al. (1997) estimated to be another 35 percent of the 600,000 US bridges, $78 billion would be spent.

Because of the considerable amount of money spent on repairing bridge approach settlement, DOTs and the FHWA have funded numerous studies to determine the causes, mitigation methods, and maintenance techniques of bridge approach settlement. The present research “Evaluation of Bridge Approach Settlement Mitigation,” sponsored by the Wisconsin Department of Transportation (WisDOT), Project I.D. 0092-00-13, is aimed at selecting the most cost-effective methods that can be competently executed during construction and that can reduce overall maintenance costs in Wisconsin. The purpose of this report is to document the performance and effectiveness of two mitigation techniques, geosynthetic reinforced fill and flowable fill, installed behind four Wisconsin bridge abutments. This report includes an extensive literature review, discussion of the field investigation, and performance evaluation of field results of these four bridges.

2.0 LITERATURE REVIEW

An extensive literature review was conducted to determine the causes, mitigation methods, and maintenance techniques of bridge approach settlement from previously conducted research.

2.1 CAUSES OF BRIDGE APPROACH SETTLEMENT

At first, bridge approach settlement appears to be a simple problem solved by improved compaction of backfill material. However, it is a complex interaction between soil and structure with many variables. One of the first research studies addressing the concern of bridge approach settlement was the 1969 NCHRP Synthesis 2 (TRB 1969). Over 20 years later, a Kentucky DOT survey by Allen (1985) indicated that it still was a problem. To update and summarize Synthesis 2, FHWA funded the NCHRP Synthesis 159 by Wahls (1990). Wahls (1990) as well as many other earlier studies completed by Laguros et al. (1990), James et al. (1990), Schaefer and Koch (1992), Stark et al. (1995), Briaud et al. (1997), and Hearn (1997) identified the causes of bridge approach settlement, which have been grouped into five major categories:

- Poor Performance of Approach Pavements
- Types of Bridge Abutments and Foundation Support
- Deformation of Embankment Fill
- Deformation of Foundation Soil
- Poor Drainage

A number of factors within each category lead to one of these five major causes. A summary of these factors is illustrated in Figure 1, and a corresponding brief description is listed in Table 1. A complete discussion is presented in Sections 2.1.1 to 2.1.5.

TABLE 1: Summary of Causes of Bridge Approach Settlement

Category		Causes	
1	**Poor Performance of Approach Pavement**	**A**	Deformation in Flexible Pavement: Rutting, shoving or cracking
		B	Failures in Concrete Pavements: transverse cracking, joint faulting, corner breaks, or blowup
		C	Improper placement of roadway grades
2	**Type of Bridge Abutments and Foundation Support**	**A**	Lack of maintenance of expansion joints of Non-Integral Abutments causing temperature induced stresses on bridge abutment
		B	Ratcheting or cyclic movement of integral abutments resulting in lateral movement of abutment and increased lateral earth pressures
		C	Vertical movement of foundations (shallow vs. deep) in relationship to embankment stiffness
		D	Improper Abutment or Wingwall Design
3	**Vertical and Lateral Deformation of Backfill**	**A**	Inadequate compaction of backfill due to limited space, improper construction equipment, contractor care, soil type, and/or lift thickness
		B	Volumetric changes of backfill due to temperature differences and drainage (i.e., frost heaving, thaw, collapsible soils, and swelling)
		C	Post-construction consolidation of cohesive soils due to the embankment self-weight, traffic loads, and weight of asphalt overlays
		D	Bearing capacity failure of sleeper slab footing under approach slabs
4	**Vertical and Lateral Deformation of Foundation Soil**	**A**	Lateral squeeze of weak foundation soils due to increase vertical stresses (i.e., embankment weight)
		B	Consolidation settlement (primary & secondary) of silt, clay and organic soils due to increased effective stress
		C	Slope stability failures due to soils with low shear strengths
5	**Poor Drainage**	**A**	Erosion of side slopes at abutment causing localized movements of backfill behind and in front of abutment. Also, loss of fines through the granular construction layer/pad below the abutment (usually constructed to facilitate construction operations) and the subsequent movement due to fines migration
		B	Instability of slopes at the abutment from rise in water level
		C	Increase in hydrostatic pressure behind abutment
		D	Poor pavement drainage causing ice lensing, soft subgrades, and pumping that causes faulting in concrete pavements and cracking in flexible pavements

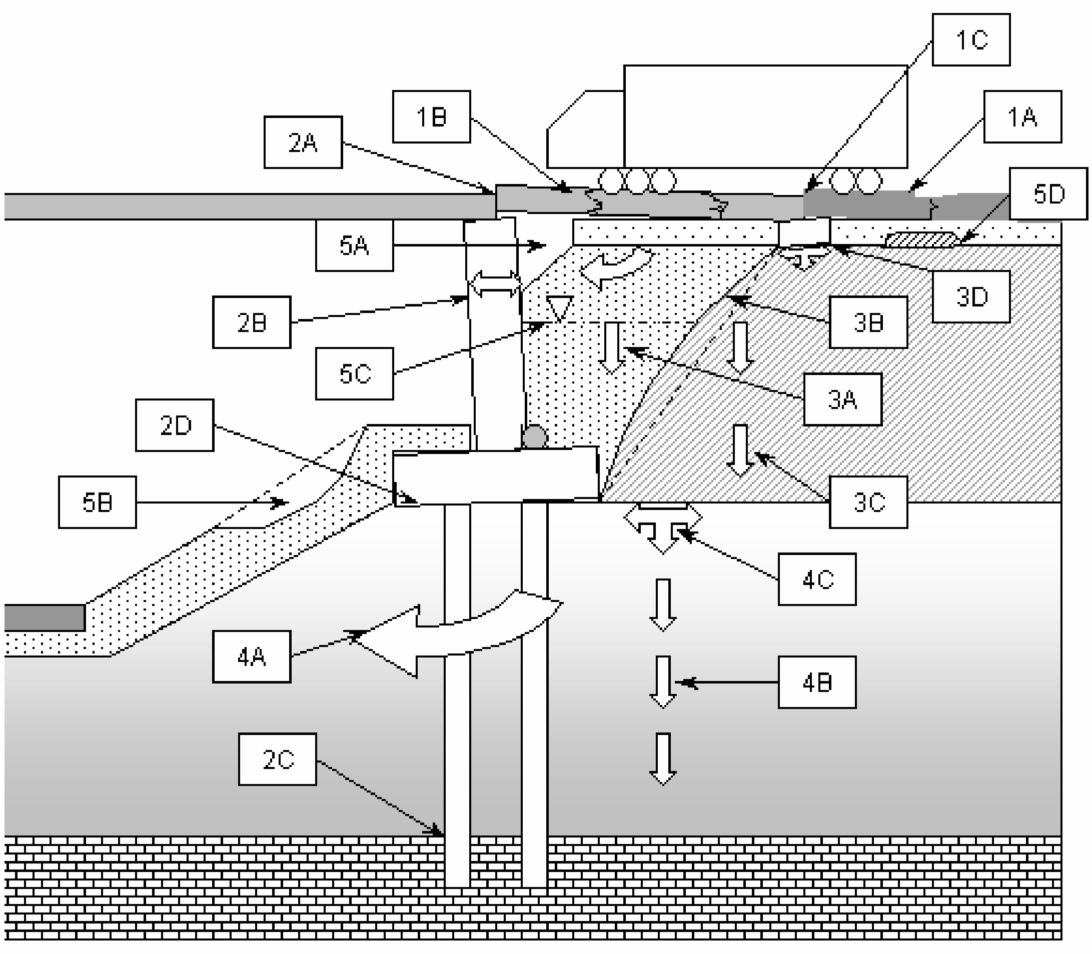


Figure 1: Schematic Illustrating Causes of Bridge Approach Settlement

2.1.1 Poor Performance of Approach Pavements

Poor performance of the approach pavements is affected by mix design, environmental factors, quality of materials, and construction. Pavement performance is not the most significant cause of bridge approach settlement; however, it can contribute to the overall settlement (Lagurous et al. 1990).

Deformations of Flexible Pavements

Deformations of flexible pavements are plastic and transpire over time because asphalt is a thermoplastic material that changes with temperature, age, drainage, and wear. Rutting, shoving, or cracking are some of these deformations.

Rutting occurs in the wheel paths of vehicles as a result of temperature and improper asphalt mixes. When temperatures rise, asphalt becomes more viscous and may deform under loading. Mixes containing too much asphalt and rounded aggregates are more likely to rut. An example of rutting is illustrated in Figure 2.

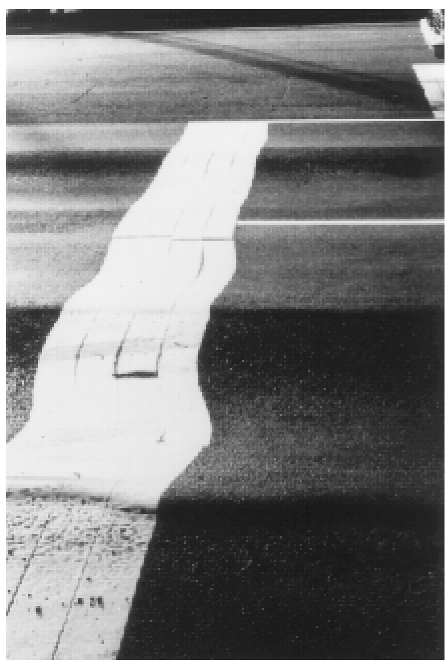


Figure 2: Rutting of Flexible Pavement (Cebon 2005)

Shoving, the lateral deformation of asphalt, typically occurs at intersections from braking or accelerating of vehicles. Temperature and improper asphalt mixes cause shoving for the same reasons as explained for rutting. Shoving may also result when asphalt is placed adjacent to a stiffer material in the direction of traffic. A stiffer material such as concrete does not allow the asphalt to move anywhere except upward, thus causing a bump. An example of shoving is shown in Figure 3.

Figure 3: Shoving of Flexible Pavement (FHWA 2005)

Another type of flexible pavement deformation is cracking, which can be described as reflective, thermal and fatigue (otherwise known as alligator) cracking. In the study conducted by Pierce et al. (2001), some form of cracking was observed in flexible pavements at 60 percent of the 25 bridges that were visited. Reflective cracking results from asphalt overlays of concrete pavement. Cracks reflect up through the asphalt from the concrete joints as shown in Figure 4.

Figure 4: Reflective Cracking of Asphalt Pavement (FHWA 2005)

Thermal cracking is caused by cyclic changes in temperature and improper grades of asphalt. Low temperatures cause the asphalt to contract and induce tensile stresses in the pavement. When temperatures rise, the asphalt expands. After numerous cycles of contracting and expanding, thermal cracking results as shown in Figure 5.

Figure 5: Thermal Cracking of Flexible Pavement (WSDOT 2005)

Fatigue or alligator cracking is failure of the pavement in tension due to repeated traffic loads in the wheel path over time. If traffic, and more so, truck traffic, is greater than what was designed for, the pavement becomes overloaded. Tensile cracks typically form at the bottom of the asphalt layer and then project up to the surface. An example of fatigue cracking is illustrated in Figure 6.

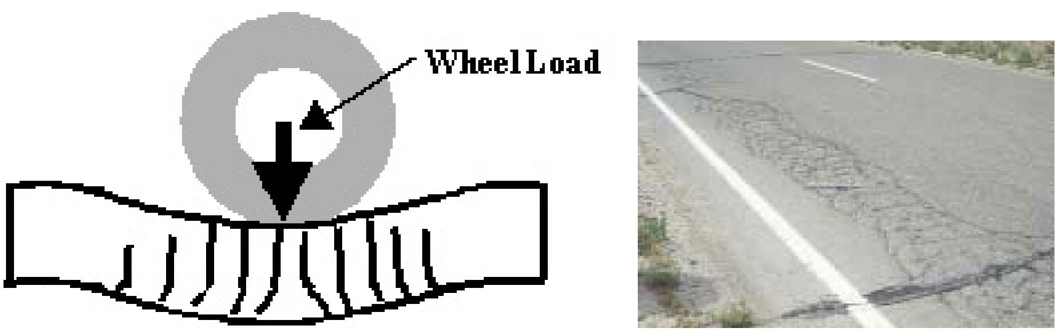


Figure 6: Fatigue or Alligator Cracking of Flexible Pavement (FHWA 2005)

Failures in Concrete Pavements

Similar to flexible pavements, poor performance of rigid or concrete pavements can cause differential settlement at the bridge approach. Unlike asphalt, concrete increases in strength with time and breaks suddenly.   As James et al. (1991) explain, rigid pavements have more severe roughness than flexible pavements. The movement is more brittle and defined as a failure instead of a deformation. Concrete failures can be caused by temperature, improper reinforcement of slab, joint deterioration, materials, and traffic loading. Some examples include transverse cracking, corner breaks, joint faulting, and blowup. Of the 25 bridges visited for the South Carolina DOT study, Pierce et al. (2001) indicated that faulting was observed in 60 percent of the bridges, and joint spalling was observed at greater than 70 percent of the bridges. There are other types of concrete failures; however, only the most relevant to bridge approach settlement are discussed.

Steel reinforcement is typically required in concrete because concrete is weak in tension. Tension results from bending of the slab due to variation in moisture between the top and bottom (slab warping or curling), from traffic loads, and if the slab is unsupported. A concrete approach slab may be used in front of an abutment to span any voids resulting from vertical or horizontal movement of the embankment fill. The concrete slab may fail if it is not sufficiently reinforced, if the reinforcement is not placed properly, or if the traffic loading is greater than the design loading. This type of failure can be categorized as either transverse cracking or corner breaks, as shown in Figures 7 and 8, respectively.

Figure 7: Transverse Cracking (Washington 2005)

Figure 8: Corner Break (FHWA 2005)

Improper design or deterioration of the joints may also affect differential movement of concrete pavements. Concrete pavements naturally crack, and joints should be included in the concrete to control the cracking. Once the concrete slabs have cracked at the joints, dowels are used to transfer the traffic load between adjacent slabs. If these dowels and expansion joints are not present or have been improperly installed, individual concrete slabs will tend to rotate in the direction opposite of the traffic, producing a bump or thump between slabs. This condition, called joint faulting, becomes worse as the difference in elevation between slabs and the impact loading increases. Joint faulting is shown in Figure 9.

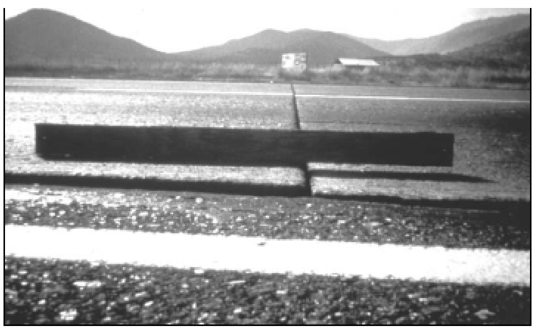


Figure 9: Joint Faulting in Concrete Pavements (Washington 2005 and FHWA 2005)

Another failure that relates to joints is blowup. Blowup is the upward movement of abutting concrete slabs. If there is not enough room between slabs when the concrete expands due to the rising temperatures, blowup could result as shown in Figure 10.

Figure 10: Blowup of Concrete Pavement (Washington 2005 and FHWA 2005)

When the concrete is placed, the contractor must be careful to avoid activities that decrease concrete strength. This can result from segregation of aggregates due to excessive vibration, increasing the water-cement ratio by adding water, and improper curing.

Improper Placement of Roadway Grades

Improper placement of the final roadway grade is the third prevailing cause of bridge approach settlement due to pavement performance. This is typically caused by a survey error, poor earthwork operations, shifted formwork during placement for concrete, or poor compaction of asphalt.

2.1.2 Types of Bridge Abutments and Foundation Support

The types of bridge abutments, foundation support, and their designs directly affect the lateral and vertical movement between the bridge abutments and the approach pavements. The performance of the bridge could be structurally affected if there is greater than 2 inches laterally and more than 4 inches vertically (Wahls 1990).

Types of Bridge Abutments

Bridge abutments can be subdivided into closed, stub / sill or spill-through abutments. Closed, or otherwise referred to as full-height, abutments retain the entire embankment height between the bridge and underpass. Closed abutments are constructed before the embankment and cost more than the other abutment types. An illustration of a closed abutment is in Figure 11.

Stub or sill abutments are partial height abutments that retain only a portion of the embankment and have a front slope. Stub abutments are constructed shortly after the embankment is constructed. These are typically less expensive than the closed abutments because the lateral loading from the soil behind is reduced. An example of a stub abutment is shown in Figure 12.

Spill-through abutments consist of pier columns that extend from the bridge to a footing at the bottom of the grade separation. A slope is placed from the top of the embankment through the columns to the bottom of the embankment. Spill-through abutments are constructed prior to the embankment and have lower lateral earth pressures than closed abutments. Spill-through abutments are sometimes specified if an additional span is anticipated (to be constructed) in the future, in which case the abutment becomes a pier. A spill-though abutment is illustrated in Figure 13. It is to be noted that WisDOT does not use spill-through abutments at this time, even though they have been used in the past.

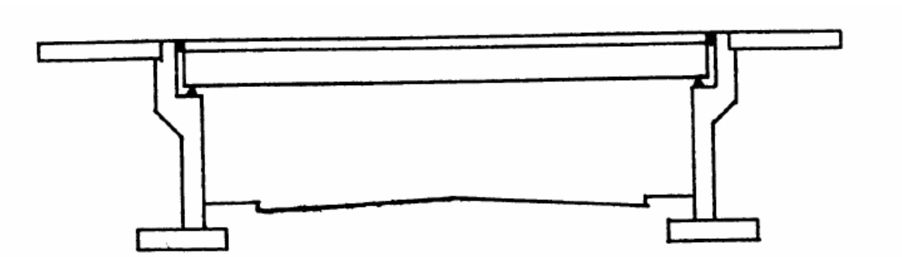


Figure 11: Closed Abutments (WisDOT Bridge Manual 2005)

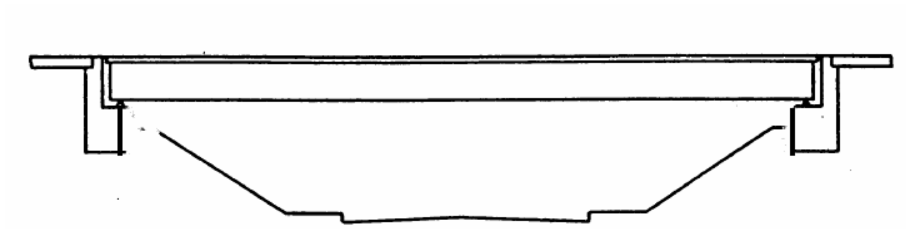


Figure 12: Stub Abutments (WisDOT Bridge Manual 2005)

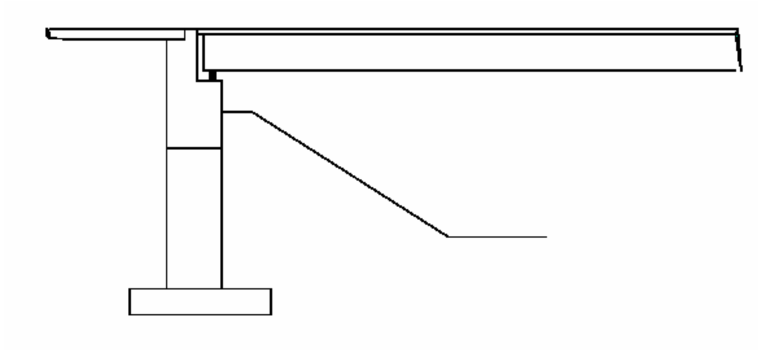


Figure 13: Spill-through Type Abutments (WisDOT Bridge Manual 2005)

The closed, stub, and spill-through abutments can be categorized as non-integral, semi-integral, or integral as shown in Figures 14 to 16. Non-integral abutments contain an expansion joint between the bridge deck, abutment, and/or approach slab (if any). The expansion joint allows for lateral deformations of the bridge relative to the abutment. If maintained, the expansion joint works properly as designed; however, if debris accumulates in the joint, the bridge is not allowed to expand.

Integral abutments are the opposite of non-integral, because they do not contain an expansion joint. The bridge deck, abutment, and/or approach slab (if any) are directly tied to each other. Allen (2002) indicated that 33 out of 50 State DOTs use integral abutments. These abutments are commonly used because they are cost-effective to construct and maintain. However, because the abutment and bridge are connected, “ratcheting” may result. Ratcheting, as defined by Horvath (2004), is lateral movement of abutments due to cyclic temperature changes. When temperatures fall in the winter, the bridge contracts and the abutments move towards the bridge and away from the abutment backfill. Horvath (2004) indicated that the lateral deflection of the abutments is the greatest at the top and typically about 1 inch. When the abutments deflect outwards, the backfill sloughs, and a void is created under the approach pavement. When summer arrives, the bridge expands, and the abutment tries to move back towards its original position but is resisted by the passive pressure of the sloughed backfill. This passive lateral earth pressure is greater than the active, for which the bridge has typically been designed. Horvath (2004) stated that the effect of ratcheting may be more significant than originally thought because it may take years or even decades to develop a structural failure.

Semi-integral abutments are abutments between integral and non-integral. For example, the girders may rest on a beam seat with an expansion joint; however, the concrete deck rests directly on the abutment. Figure 15 shows an example of a semi-integral abutment.

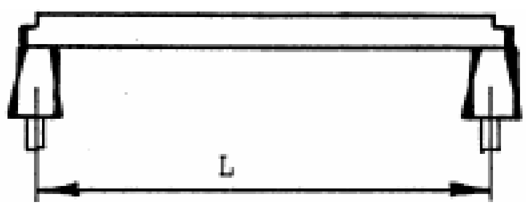


Figure 14: Integral Abutment

(WisDOT Bridge Manual 2005)

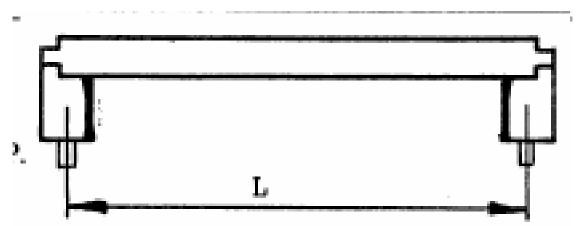


Figure 15: Semi-Integral Abutment

(WisDOT Bridge Manual 2005)

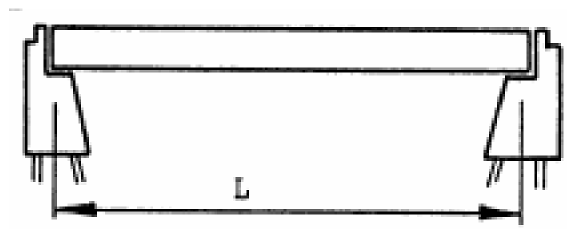


Figure 16: Non-integral Abutments (WisDOT Bridge Manual 2005)

Types of Foundation Support

Foundation support of abutments can be categorized as shallow or deep. Shallow foundations typically consist of concrete footings that bear directly on the soil or rock. Depending on the type of bridge abutment, the elevation of the footing could be located in the embankment fill (stub) or on the foundation soil or rock (closed or spill through).

The loads and moments from the abutment and the bridge are transferred and distributed across the shallow footing. The applied footing pressure must be checked against the allowable bearing capacity of the soil beneath it. If the shear strength of the soil or rock is exceeded, a bearing capacity failure occurs, resulting in sudden, excessive movement of the overlying bridge and approach.

If the applied footing pressure is less than the allowable bearing capacity, a sudden bearing failure may not occur, but some settlement may. Settlement of the underlying soil should be estimated and then compared with how much relative movement the abutment is allowed before the bridge is damaged. Settlement consists of three types: immediate, primary consolidation, and secondary consolidation.

Immediate or elastic settlement is the movement of soil that takes place directly after construction of the structure. Immediate settlement is based on the theory of elasticity, which states that at low stress levels, strain or settlement linearly increases with stress at a rate that is dependent on the elastic properties of soil: modulus of elasticity, E, and Poisson's ratio, υ. In granular soils, immediate settlement makes up most of the total settlement. Because immediate settlement occurs prior to placing final grades of the bridge deck and approach pavement, this movement is neglected.

In cohesive soils such as clays and silts, primary and secondary consolidation settlements constitute the majority of the total settlement. Primary consolidation is the settlement of soil particles that results from pore water pressure dissipation through previous boundaries, thus resulting in effective stress increase (over time). The dissipation of excess pore water pressure causes volumetric changes in the soil (because of water loss) thus causing consolidation settlements.

The magnitude of primary consolidation settlement is calculated by determining the change in void ratio, which is dependent on the vertical effective stress of the soil at each depth, the compression index, the recompression index, and stress history of the soil. The stress history determines if the cohesive soil has been normally consolidated or overconsolidated. An overconsolidated soil is a soil that has been previously loaded or has experienced an increase in effective stress beyond its present (in situ) vertical effective stress. This maximum stress is called preconsolidation pressure. Examples of previous loads include embankments or glaciers. As shown in Figure 17 (a), if an applied stress is less than the preconsolidation pressure, settlement is small because the recompression slope of the curve (recompression index) is small. If the soil is normally consolidated, the soil has never experienced an additional effective stress beyond its present vertical effective stress. Greater magnitudes of primary settlement will result in this case, as shown in Figure 17 (b), because the virgin compression slope (compression index) is much steeper (than the recompression index).

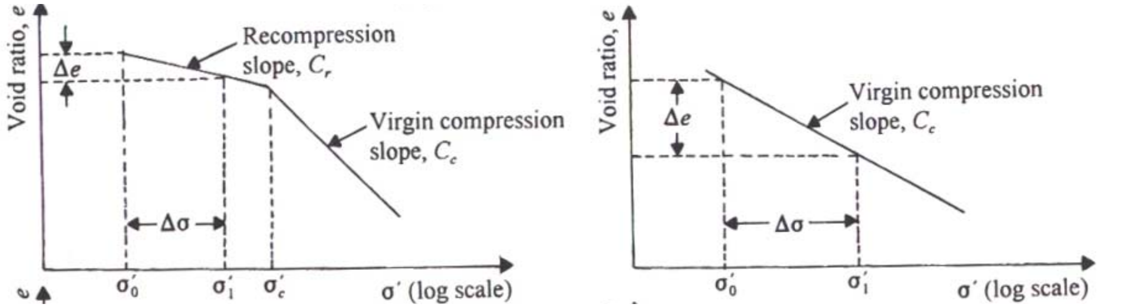


Figure 17: Effective Stress, σ', versus Void Ratio, e, Curves for
(a) Overconsolidated Soil and (b) Normally Consolidated Soil (Das 1997)

The time rate at which primary settlement occurs is dependent on the thickness of the cohesive layer, its permeability, and on its drainage condition (one- or two-way drainage). For example, a cohesive layer with two-way drainage settles at a greater rate (four times faster) than a layer with the same thickness but with one-way drainage.

Secondary consolidation begins at the end of primary consolidation and is caused by the slippage or reorientation of soil particles under a constant effective stresses. In inorganic soils, secondary consolidation is less significant than primary. In organic soils, secondary consolidation can be more significant than primary, especially when the structure has a long service life.

Once immediate, primary consolidation and secondary consolidation are estimated, these should be checked against the serviceability requirements of the bridge. If accurately estimated, the designer may be able to use shallow foundations and take into account the predicted settlement. This is sometimes critical in determining construction schedules and staging. If the settlement magnitude and/or rate are not estimated accurately, a difference in elevation between the bridge abutment and the approach may likely result. Allen (2002) indicated in his survey that 32 of 50 State DOTs have used shallow footings, and 29 DOTs stated that the shallow footings have been successful. This statistic, however, may be dependent on the bearing soil of the shallow footing. Ha et al. (2002) stated that 92.3 percent of bridges in Texas are supported on deep foundations. Similar surveys also indicated that deep foundations are selected in the majority of designs because of inadequate data to predict accurate settlements or because bearing capacity or settlement of abutments do not satisfy serviceability requirements.

Deep foundations consist of driven or drilled piles, drilled shafts, or other structural elements, which transfer bridge loads to harder soils. If the foundation is primarily end bearing or supported on very dense soil or rock, settlement of the foundation will be negligible. If the foundation element supports the load in skin friction (adhesion of the pile or shaft and the soil), some settlement will occur but is typically very minor (less than 1 inch). Because deep foundations have little settlement, the relative settlement between the approach slab and the abutment may be greater than the case of an abutment supported on shallow foundations. Nonetheless, experts tend to disagree whether or not pile or shaft supported abutments contribute to bridge approach settlement. Most do agree, however, that if the bridge approach settlement does occur directly behind the abutment, greater impact loads may result. Once a bump is formed, impacts loads may be 4 to 5 times that of a static traffic load used in design (Briaud et al. 1997). Traffic speeds may also influence the impact load (Das et al. 1999 and Pierce et al. 2001).

Design of Abutments and Wingwalls

Improper design of the abutments and wingwalls can result in movement of the structure itself. Three failure mechanisms: bearing capacity, sliding, and overturning, must be checked in the design of wingwalls. Because the bridge girders and deck do not restrain the wingwalls, the wall can deflect laterally because of lateral earth pressures as shown in Figure 18.

Figure 18: Lateral Movement or Bulging of Wingwall due to Earth Pressure

Sliding may occur if the frictional resistance between the footing and the bearing soil is less than the lateral force caused by lateral earth pressure against the wingwall. The frictional resistance is dependent not only on the bearing soil but also on the width of the footing and the weight of the wall.

Overturning or tipping may occur if there is not enough weight (wall and backfill above the heel) to counteract the lateral force pushing the wingwall over its toe. The lateral earth pressure, or the tipping force, is caused by the soil present within the active earth pressure zone behind the wall. This zone represents the soil that slips forward toward the wall and is defined by a failure plane. The failure plane can be determined as shown in Figure 19. Engineers should design for the soil parameters within this zone. In some cases, contractors infringe on this zone with the embankment material, which may exert greater earth pressures on the wall than designed.

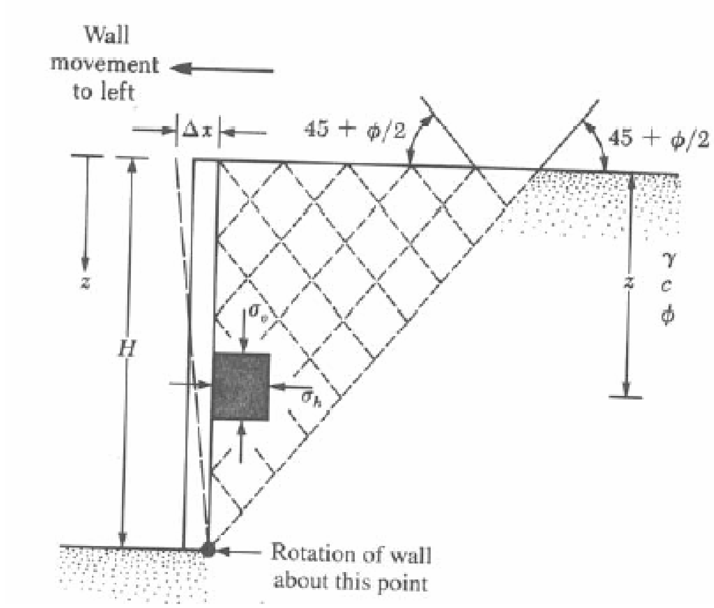


Figure 19: Active Earth Pressure Zone defined by Approximate Failure Plane
(from Das 1995)

The resistance to overturning is dependent on the weight of the structure as well as the length of the toe and heel. Therefore, if the footing is not sized or designed properly, the wall may fail in sliding or overturning, which will result in lateral movement at the top of the wingwall. The backfill soil will then move along with the wall, creating a void under the approach. Figures 20 to 22 show schematics of external stability failures such as sliding, overturning, and bearing from Sabatini et al. (1997).

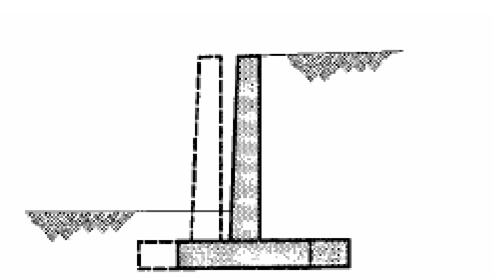


Figure 20: Sliding Failure
(from Sabatini et al. 1997)

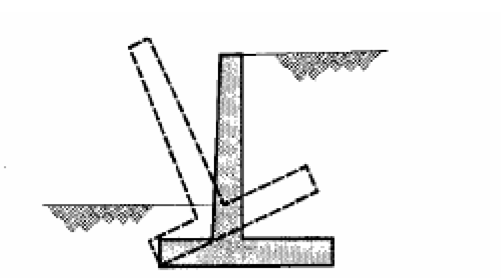


Figure 21: Overturning
(from Sabatini et al. 1997)

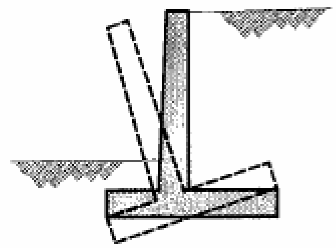


Figure 22: Bearing Capacity Failure (from Sabatini et al. 1997)

2.1.3 Vertical and Lateral Deformation of Backfill

The deformation of the backfill directly behind the bridge abutments and in the approach embankments has been perceived and proven to be one of the major contributors to the bridge approach settlement problem. The causes of vertical and horizontal deformation of the backfill result from lack of compaction, volumetric changes in the soil, post-construction consolidation settlement, and bearing capacity failure of the embankment soil under the sleeper slab.

Compaction of Backfill

<u>Inadequate fill compaction is typically one of the most perceived causes of bridge approach settlement. Hoppe (1999) indicated that 50 percent of the states in his survey indicated difficulty in compacting around abutments.</u> Parsons et al. (2001) explained that compaction is only accomplished if soil particles shear. Causes that result in a lack of soil shearing or compaction include the following:

- Too thick lifts
- Improper compaction equipment for the type of backfill soil being placed
- Not enough compactive effort due to poor workmanship of the contractor not covering the entire area

- Not enough compactive effort near sill abutments and corbels or between abutment and embankment where access is limited (Kramer and Sajer 1991)
- Compacting backfill outside the specified tolerance of optimum moisture (i.e., backfill that is too wet or too dry): If water is added to the soil, it will act as a lubricant and allow particles to shear (Parsons et al. 2001). However, if too much water is added, the water will replace the air voids, which may cause consolidation. It is best to compact either –1 or +2 percent of optimum moisture
- Lack of inspection or testing of relative density of the soil
- Use of cohesive soils as backfill: Allen (2002) indicated that 17 out of 50 states may use compacted clay as backfill behind abutments. Clay backfill is stiffer (Carrier 2000) and performs well at or below optimum moisture, but if placed above optimum, the clay will creep under load (Barrett et al. 2002). In addition, clay backfill requires more compactive effort than granular soils, which is sometimes difficult to attain in restrictive areas

Parsons et al. (2001) indicated that embankments with lower relative compaction than 90 percent of standard proctor did not perform as well as those with greater than 90 percent, but some embankments even compacted greater than 90 percent did not perform well. As Carrier (2000) noted, specifications need to consider water content in addition to relative compaction.

Volumetric Changes of the Backfill

Volumetric changes of the embankment fill can result from the freeze/thaw cycle, swelling soils, or collapsible soils. All of these conditions cause vertical and horizontal deformations in the soil. If the volume of the soil increases, heave occurs, inducing upward stress on the pavement or approach slab as well as increased lateral earth pressure on the abutments or wingwalls. If the volume decreases, settlement will result.

Cohesive soils are more susceptible than granular soils to frost heave caused by the freeze/thaw cycle because they have lower permeability and do not drain easily. Water becomes trapped within the air voids between particles and may freeze when temperatures decease below the freezing point within the soil. Wisconsin Building Code (2004) indicates that the frost depth is as much as 5 feet. The embankment fill typically within 5 feet of the surface may heave if water is not drained in the winter. When temperatures increase, the ice thaws and typically the soil becomes weaker and more compressible, resulting in vertical deformation. The freeze/thaw cycle not only causes problems in structure footings and embankment fills; it is also one of the major factors in performance failures of pavements in areas of seasonal climates.

Volumetric changes can also occur in swelling soils. Shale and fat / plastic clays are called swelling if they expand when exposed to water and shrink with the loss of water. Also, a large and sudden settlement could result from the volumetric changes of collapsible or metastable soils, which are defined as unsaturated soils that collapse upon saturation (Das 1995). Swelling, expansive, collapsible and metastable soils are rarely found in Wisconsin.

Post-construction Settlement of Backfill

As discussed earlier, total settlement is comprised of immediate, primary consolidation, and secondary consolidation. Because backfills behind the bridge abutments are granular in most states (including Wisconsin), very little post-construction consolidation occurs. However, approach embankments behind the bridge abutment backfills are typically constructed with cohesive soils and thus are more likely to experience post-construction consolidation settlement from increased stress due to the self-weight of the embankment, traffic loads, continuation of asphalt overlays (Chew et al. 2004), and applied bearing pressure from a sleeper slab if present.

Structure backfill is usually specified for approach fills for Wisconsin bridges. Section 3.3 describes the specifications of Wisconsin structure backfill, its compaction requirements, and methods of inspection.

Bearing Capacity Failure of Backfill under the Sleeper Slab

The fourth type of deformation due to backfill under the approach is from a bearing capacity failure of the sleeper slab. Bearing capacity failure was described in Section 2.1.2, and that discussion is also applicable here. If an approach slab is present, the end of the slab may rest upon a sleeper slab or footing. If the sleeper slab footing is not designed properly for the soils underneath it, the footing could fail or settle excessively. It is to be noted that WisDOT does not currently use sleeper slabs or footings at the end of approach slabs.

2.1.4 Vertical and Lateral Deformation of Foundation Soil

The deformation of foundation soil can be a major contributor of bridge approach settlement, especially if weak. The causes of vertical and horizontal deformation in the foundation soil result from lateral squeeze, post-construction consolidation settlement, and global stability failure.

Lateral Squeeze

Lateral squeeze or sliding of the foundation soil is the horizontal movement of weak soil when subjected to a vertical load that is greater than its shear strength. Hannigan et al. (1998) state that lateral squeeze may occur if the weight of the fill is greater than 3 times the undrained shear strength of the foundation soils. This typically occurs if weak soils are underlain by an incompressible layer that does not allow the weaker soil to move vertically. Soft clays, loose silts, organic soils, and peat may be susceptible to lateral squeeze. When the foundation soil slides, this not only creates a vertical settlement at the top of the approach embankment and abutment backfill, but it also applies a lateral load on any deep foundation. If not designed properly, piles could buckle and shafts could crack due to this additional lateral force. Figure 23 shows a schematic of the result of lateral squeeze.

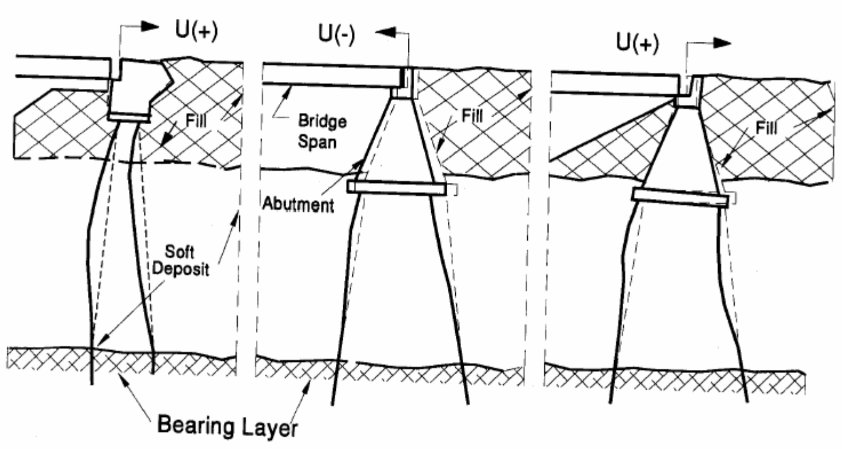


Figure 23: Lateral Squeeze of Weak Foundation Soil (Hannigan et al. 1998)

Post-construction Foundation Settlement

As discussed earlier, total settlement is comprised of immediate, primary consolidation, and secondary consolidation. If the foundation soils are cohesive, post-construction consolidation settlement may result from the weight of the approach embankment (Figure 24).

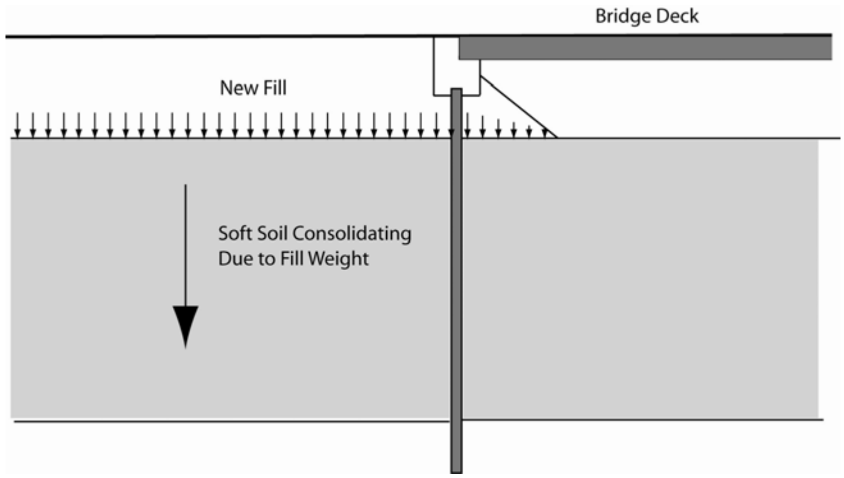


Figure 24: Consolidation of Foundation Soil Due to Fill Weight

Slope Stability Failure

Foundation soil failure can be attributed to slope instability. Slope stability or rotation failure occurs when the shear strength of the foundation soil cannot resist applied loads. Slope stability failures will result in scarps or cracking at the top of embankments and/or slopes. A weak foundation soil, high or differential water table, and/or heavy embankment loads can cause failures. A schematic showing slope stability failure is illustrated in Figure 25, and a photograph of the scarp at the top of a failed embankment is shown in Figure 26.

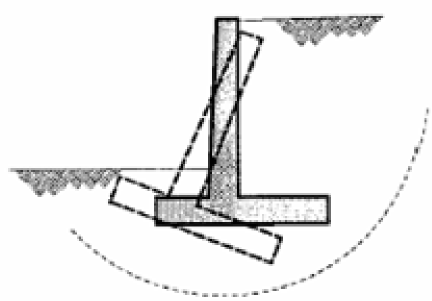


Figure 25: Schematic of Slope Stability Failure (from Sabatini et al. 1997)

Figure 26: Scarp of Slope Stability Failure

2.1.5 Poor Drainage

A major cause of bridge approach settlement is poor drainage behind, and in front of, the bridge abutments and under the approach pavements. Poor drainage can result in surface erosion, slope stability failures, increases in hydrostatic pressures, and pumping of fines. The study conducted by White et al. (2005) for the Iowa DOT determined that poor water management was the major problem of most bridges that they inspected.

Erosion of Slopes

Local erosion of the front and side slopes occurs when the slopes are not properly protected and when water is allowed to drain along the slopes. Water from the top of the bridge, from the backfill, and from the embankment should be diverted to a drainage ditch or storm sewer system that is located at the bottom of the slopes. Weep holes, storm sewers, and vertical drain pipes should not be allowed to stop short within or at the top of the slope. The water will likely cause erosion of the surface materials or piping of material under protected slopes. A broken slab as a result of piping, and undermining as result of erosion are shown in Figure 27. Figure 28 illustrates erosion and piping of material along the front slope.

Figure 27: Broken Slabs and Undermining of Soil due to Erosion of Slopes

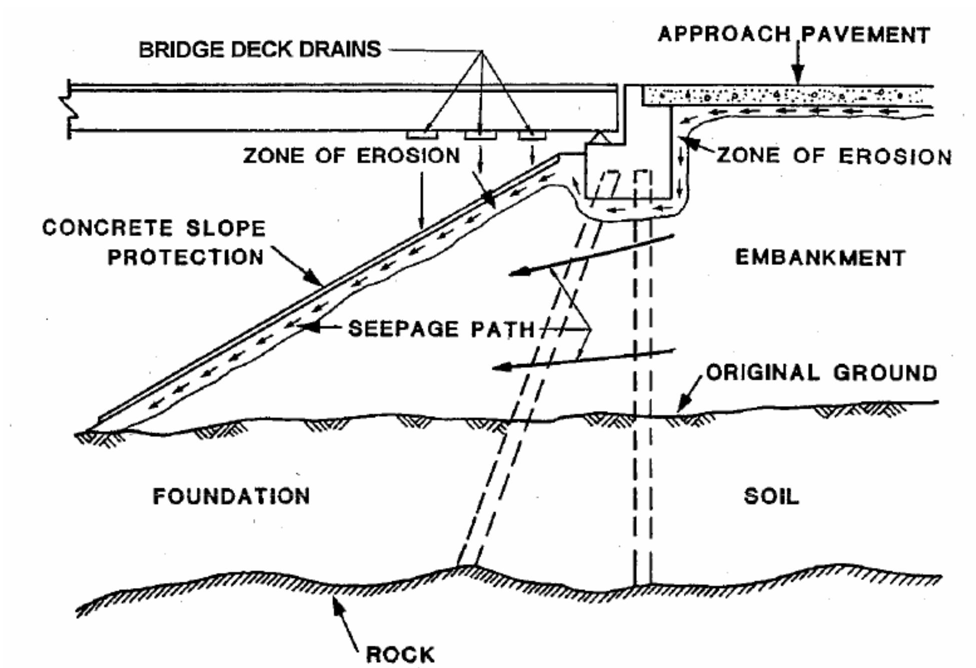


Figure 28: Erosion of Front Slope (from Stark et al 1995)

Vegetation, riprap, or other means can prevent washing away of soil. Loss of material in slopes in front of the abutments causes soil from around the eroded area to collapse. For shallow foundations, the design may be compromised because of three factors: lack of confinement required for bearing capacity, a reduction in passive earth pressure to resist sliding and overturning, and possibly excessive settlement due to the movement of soil from beneath the footing.

If erosion of material occurs near the top of the side slopes, soil from the backfill or embankment will tend to collapse or fill in the voids along the side slopes. This typically creates a void under the approach pavements and can cause faulting of concrete or cracking of asphalt. A void, and thus subsequent settlement, can also be created from the piping and undermining of the sleeper slab (Luna et al. 2003). Figure 29 presents the start of a void due to erosion of the side slopes.

Figure 29: Void beneath Concrete Curb along Side Slope

Slope Stability Failure of Abutment Slopes

Another result of erosion is slope stability failure. Because soil in front of the abutment wall is being taken away, the counterweight to resist a rotational movement of the abutment decreases. In addition, poor drainage even without erosion may cause slope stability failure. A rise in the water table reduces the shear strength of soil. As described in Section 2.1.4, slope stability failures will cause cracking or sloughing at the top of the backfill or embankment, as shown in Figure 30.

Figure 30: Slope Stability Failure of an Embankment

Hydrostatic Pressure

Another consequence of poor drainage is the increase in hydrostatic pressure behind the abutment wall. White et al. (2005) found that many underdrains inspected as part of an Iowa DOT study did not function properly because the drains were dry, blocked with fines, or had collapsed. If the water behind the abutment does not drain freely and accumulates against the abutment wall, hydrostatic pressures exert a lateral force on walls. This hydrostatic force, if not designed for, can be at least 1.5 times the active earth pressure. When added together, this is more than two times what is typically designed for with a free-draining structure. These significant forces and moments on the abutment wall as well as the wingwalls could cause the bridge to deflect or even to fail. Therefore, drainage behind the abutment is crucial to the design of the bridge. Maintaining and designing drains to be free of debris and silt is critical.

Lack of Drainage under Approach Pavements

Without proper drainage under the approach pavement, ice lensing, softened subgrades, and pumping could result. If water remains trapped within the base material, ice lenses may form inside air voids under the pavement during freezing temperatures. Ice has a 10 percent greater volume than the same mass of water. This volume change or heave causes significant uplift forces and results in pavement distress and deformation.

Absorption of water in cohesive soils over a prolonged time period will cause softened subgrades. Softened subgrades will likely settle from consolidation and may cause migration of fines from a cohesive embankment into the base material. Contamination of fines decreases the rate at which water can flow in the base and increases its susceptibility to frost.

Another concern with poor drainage is pumping or bleeding, which is defined as the seepage of water up through the pavement joints. If cohesive particles have been introduced into a saturated base course, pumping allows the fines to move upward through the joints and thus creates voids under the pavement due to erosion. Pavement distress and deformation could result from the loss of material under the pavement. Examples of pumping are presented in Figure 31.

FIGURE 31: Pumping

2.1.6 SUMMARY

Bridge approach settlement is a complex interaction between the bridge, pavement, embankment backfill, and foundation soil. Typically, the settlement is attributed to a multiple number of causes; however, the causes that create the greatest magnitudes of movement are typically due to improper compaction of backfill behind the abutment, deformation of cohesive soils within the embankment, deformation of weak foundation soils, and poor drainage of newly placed fills. Sites that have older existing structures that are being replaced or repaired are not as susceptible to settlement because the embankments and foundations have already been subjected to increased vertical stresses (Ha et al. 2002). Cohesive soils are more problematic and are greater contributors to bridge approach settlement than granular soils because cohesive soils are frost-susceptible, absorb water and may swell, settle over time, and may become weaker when exposed to water. Studies completed by Laguros et al. (1990) and Ha et al. (2002) confirmed that higher cohesive embankments resulted in greater settlements. In order to control or prevent some of these problems, numerous mitigation methods have been considered.

2.2 MITIGATION METHODS

It is apparent from the literature review carried out in this research that the three major causes of bridge approach settlement are: deformation of backfill, deformation of foundations soils, and poor drainage. This section will address mitigation methods, summarized in Table 2, that have been used in an attempt to alleviate the aforementioned causes of bridge approach settlement. Depending on site conditions, one or more of these may be required. For example, a site with a strong bedrock foundation will likely not require foundation improvements, but may need to address the embankment backfill and the drainage issues.

As Briaud et al. (1997) stated, proper mitigation takes into consideration all issues in design, construction and inspection. Design requires proper specifications, appropriate calculations and investigations, teamwork, life cycle cost analysis, and change of the structure over its service life. During construction, contractors should follow design plans and specifications by using the proper equipment and qualified labor, and inspectors are required to verify that the contractors are following the specifications and plans. Considering all three phases of a project and evaluating all mitigation methods, the bump at the end of the bridge will likely be minimized or alleviated. The mitigation methods are briefly discussed in the Section 2.2.1 through 2.2.3 and were developed in research reports by Wahls (1990), Laguros et al. (1990), James et al. (1990), Schaefer and Koch (1992), Stark et al. (1995), Briaud et al. (1997), and Hearn (1997). Specific studies pertaining to particular methods are referenced in the appropriate sections.

Table 2: Mitigation Methods of Bridge Approach Settlement

Cause	Mitigation Method
Deformation of Backfill	More Stringent Backfill and Compaction Specification
	Scheduling a Delay in Construction Work
	Geosynthetic Reinforced Earth
	Controlled Low Strength Materials (CLSM)
	Lightweight Fills
	Reinforced Concrete Approach Slab
	Hydraulic Fills
Deformation of Foundation Soil	Removal and Replacement of Weak Foundation Soils
	Ground Improvement (mechanical or chemical)
	Surcharging
	Supporting Embankment on Deep Foundations
Drainage	Flatter Side Slopes
	Backfill and Surface Drains
	Diverting Water away from the Abutment
	Geotextile Separators
	Increasing Surface Drainage
	Maintaining Watertight Joints
	Extending Wingwalls
	Extending Limits of Backfill Prism
	Limiting P200 material

2.2.1 Methods to Reduce Deformation of Backfill

This section describes methods used to reduce deformation or improve the performance of the backfill behind the abutment as well as the embankment itself. Davis (2003) indicated that stiffness of bridges is approximately twice that of an approach. So, the method of stiffening the embankment prior to construction will provide a smoother transition to an unyielding bridge (Luna et al. 2003). Mitigation techniques to reduce backfill deformation include more stringent backfill and compaction specifications, scheduling construction delays, geosynthetic reinforced earth, lightweight fills, controlled low strength materials (CLSM), reinforced concrete approach slabs, and hydraulic fills.

More Stringent Backfill and Compaction Specifications

Ha et al. (2002) indicated that 80 percent of settlement occurs within the first 20 feet of the bridge, where the backfill is placed. One effective way to improve the performance of the bridges is by controlling the backfill materials and compaction specifications. Specifications should include a relative compaction standard with a target moisture content, a maximum number of fines or P200 material, backfill limits, and inspection criteria.

Parsons et al. (2001) noted that embankments with lower than 90 percent of Standard Compaction did not perform as well as those greater than 90 percent; however, even some backfills with greater than 90 percent perform poorly. The "target" moisture range near optimum must also accompany the compaction specification. The study by Parsons et al. (2001) included a survey of 32 states, which indicated that 27 states (or 84 percent) used relative compaction, only 19 states (59 percent) required 95 percent or greater of Standard Proctor, and 25 states (78 percent) specified a moisture target range with the majority within ± 2 percent of optimum. As a conclusion to the study, Parsons et al. (2001) recommended that Kansas DOT use 95 percent of maximum density based on Standard Proctor (AASHTTO T-99) and a target moisture within ± 2 percent of optimum. Stark et al. (1995) as well as Hoppe (1999) also recommended 95 percent of Standard Proctor with 6 to 8 inch lifts.

Specifications should also limit the fines content as well as extend the backfill beyond the end of the footing heel. Reducing the fine content reduces or even eliminates consolidation settlement in the backfill and increases water flow. Allen (2002) conducted a survey as part of the Kentucky DOT research project, which indicated that of the 50 states surveyed, 38 states use compacted granular backfill (76 percent) but as many as 17 states (34 percent) may use compacted clayey soils. Ha et al. (2002) found that the embankments made of clay resulted in higher settlements than those made of granular materials Therefore, Ha et al. (2002) recommended using less than 15 percent fines passing #200 Sieve (P200) and specified compaction requirements within 100 feet of the abutment. Hoppe (1999) also concluded limiting the fines to within 4 to 20 percent of P200.

One of the last steps to control backfill and compaction specifications is its verification. Parsons et al. (2001) indicated that 25 out of 32 states verified compaction with a nuclear density gauge while others used a visual method, a sand cone method, or none at all. Testing is one of the most critical steps in this process. Parsons et al. (2001) recommended using field-testing such as nuclear density gauge, sand cone or drive cylinder over visual methods.

Scheduling a Delay in Backfill Operations

Scheduling a delay in the backfill and compaction operations is another way to control settlement. If the contractor can allow for the embankment itself to settle before finishing final roadway grades, post-construction settlement can be reduced or significantly minimized. Surcharging above final roadway grades will also help this process. The time of surcharging or waiting is dependent on the height of the embankment and the type of backfill material. This method, if feasible within the time constraints of the project, is one of the most cost-effective methods.

Geosynthetic Reinforced Backfill

This method uses geosynthetics placed in layers to reinforce granular backfill material, which results in a much stiffer backfill mass. A cross-section illustrating geosynthetic reinforced backfill is shown in Figure 32. CE (1999) reported that closer vertical spacing with reduced tensile strength is more effective than wider vertical spacing with stiffer geosynthetics.

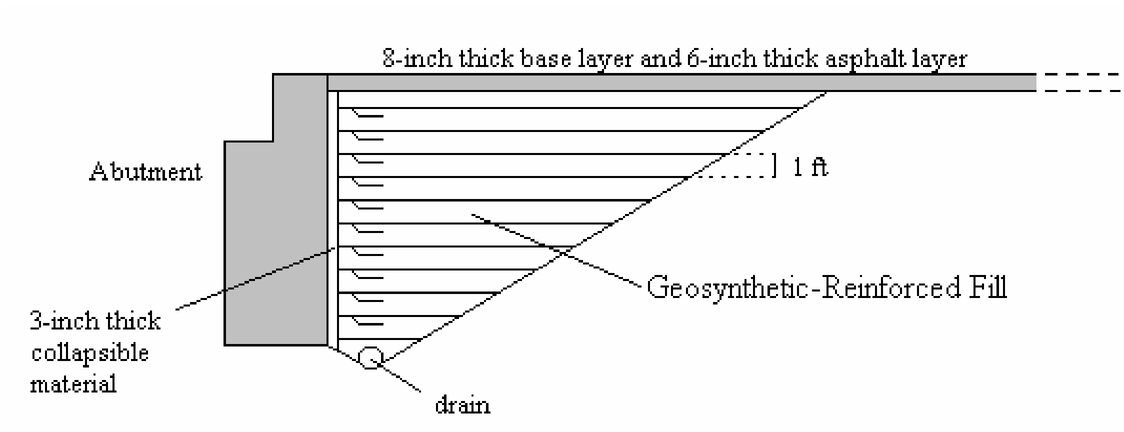


Figure 32: Geosynthetic Reinforced Backfill

Monley and Wu (1993) have found that a collapsible inclusion may be placed between the reinforced earth and the abutment to allow the reinforced fill to deform laterally, thus putting the reinforcement layers in tension. Once tension has been established, the reinforced earth acts as one solid mass. As a result, Monley and Wu (1993) concluded that the settlement and lateral earth pressure are smaller and more uniform than those resulting from granular backfill. This was also concluded in a research study completed by South Dakota DOT (Reid et al. 1999), Colorado DOT (Abu-Hejleh et al. 2000, 2001), and North Dakota DOT (Marquart 2002).

In the South Dakota study, reinforced granular backfill was used behind the three integral bridge abutments and separated with rubber tire chips. The findings indicated that the development of the void underneath the approach slab decreased, the lateral earth pressure was reduced, and it was important to maintain tension; otherwise, lateral deformation resulted. Reid et al. (1999) also recommended using foam concrete or expanded polystyrene foam as a collapsible inclusion.

In the Colorado DOT study (Abu-Hejleh et al. 2000, 2001), the abutments of a two-span structure were supported by shallow footings on geosynthetic reinforced soil. The results of the study indicated that the approach settlements were virtually eliminated. The study also indicated that the lateral earth pressure was 4.2 to 36.4 percent of the design value. The lower earth pressure was attributed to the conservative design parameters (34-degree friction angle and zero cohesion).

The North Dakota DOT study included a 3-year evaluation of a bridge with approach slabs underlain by either granular backfill or geosynthetic reinforced fill. The study found that the granular fill settled 57 percent more than the geosynthetic reinforced fill and that the granular fill was still consolidating after 3 years.

The advantages of the reinforced earth method are that it is cost-effective, simple and fast to construct, has good seismic performance, and is able to tolerate greater deformation without structural failure. Disadvantages are that placing reinforced earth is somewhat labor intensive. It should also be noted that the reinforced earth method only alleviates settlement within the backfill mass itself and not within the foundation. Fahel et al. (2000) conducted a study of two embankments, one without and one with geosynthetics at the base of the embankment, which was underlain by soft organic clay. The study showed that the reinforced embankment reduced lateral displacement and rotational failure of the slopes; however, the vertical settlement was the same as the embankment without reinforcement. This conclusion was also confirmed with a study completed by Lau and Cowland (2000) of an earth embankment with basal reinforcement adjacent to the Shenzhen River in Hong Kong, which was underlain by soft muds.

Lightweight Fills

Another method for reducing settlement within the backfill and the foundation soil is the use of lightweight fills such as rubber tire chips and sand, bark, sawdust, peat, ash, slag, cinders, lightweight aggregate, expanded clay-shale, expanded polystyrene (EPS) blocks, and lightweight concrete foam or cellular concrete (LD-CLSM). By reducing the weight of the backfill, the driving force of the settlement is reduced. Some of these lightweight materials such as bark, sawdust, and peat are biodegradable and typically not recommended. Ash, slag and cinders may be used; however, environmental impacts may limit their use. Lightweight aggregate and expanded clay-shale may be economical to use if there is a nearby supply. For lightweight fills, EPS blocks, and LD-CLSM have been relatively popular due to their availability, consistent quality, and ease of construction. Nonetheless, their cost does not warrant their use in many cases.

EPS blocks are low-density cellular plastic foam solids as shown in Figure 33, which can range between 1 and 3 pounds per cubic foot (pcf). The advantages of EPS blocks are that they are lightweight, are easy to install and modify, and provide thermal insulation for frost-heave problems. Design concerns using these blocks include puncturing of the blocks from concentrated loads, chemical attack of gasoline or other organic fluids, flammability, degradation due to ultraviolet waves from sunlight exposure, insect infestation, buoyancy, and differential icing if the roadway is not drained properly (Negussey, 1997).

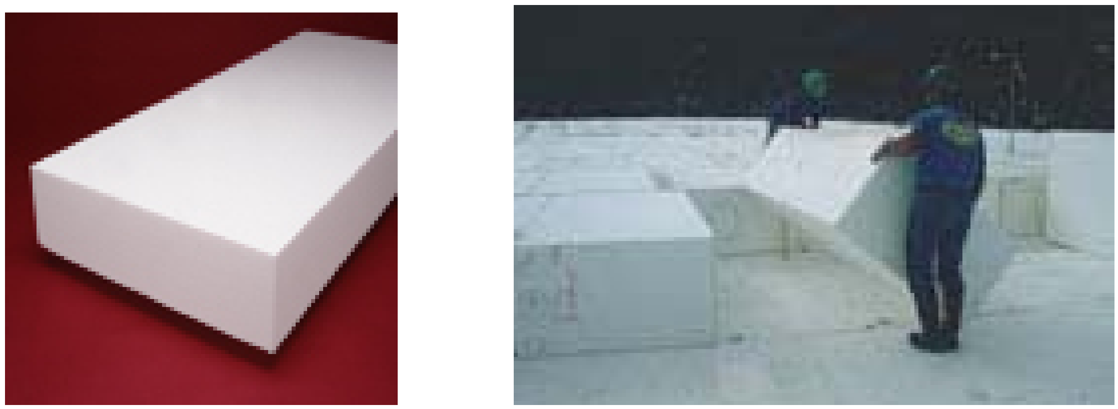


Figure 33: EPS Blocks (R-Control 2005)

Controlled Low Strength Materials (CLSM)

CLSM is defined as self-compacting cementitious material that is in a flowable state at the time of placement and has a specified compressive strength of 1200 psi or less at 28 days, but is defined as excavatable if the compressive strength is 300 psi or less at 28 days (ACI 1999). CLSM contains water, cement, flyash, admixtures, and aggregates. ACI (1999) reports that the wet density of CLSM ranges between 85 and 145 pcf, but the dry density is substantially less than the wet density due to water loss. CLSM can be made lighter with the inclusion of preformed foam to form lightweight foam concrete (LD-CLSM).   LD-CLSM only contains water, cement and preformed foam and has a wet density ranging between 18 and 120 pcf. Figure 34 shows placement of CLSM.

Figure 34: CLSM Placement (from AJ Volton 2005)

Advantages of CLSM and LD-CLSM are that they are durable, excavatable, self-leveling, cure rapidly, are incompressible after curing, and are flowable around confined spaces. They can also reduce possible cave-ins and use environmentally impacted materials such as fly ash within their mix (Trejo et al. 2004, Hajafi and Tia 2004, and Newman et al. 1993). Hajafi and Tia (2004) noted that the use of CLSM eliminates the need for compaction and reduces equipment needs, labor costs and the amount of inspection. CLSM can be placed all in one pour; however, LD-CLSM must be placed in successive lifts so that the air voids formed by the foam do not collapse. In regards to the lateral earth pressures against the abutment, Schmitz et al. (2004) concluded that the lateral earth pressure after curing is negligible, but during placement, the structure must be designed to temporarily support fluid pressures. Snethen et al. (1997) found that the lateral earth pressure was higher in the center layer of the flowable fill at curing due to the speed of hydration and the length of the drainage path. At the center, water could not dissipate or evaporate as fast as points near the surface.

Some disadvantages of CLSM include buoyancy of the lighter weight CLSMs, anchoring of lightweight pipes, the requirement of forms, shrinkage, frost susceptibility, drainage, bleeding, and earth pressure during its fluid state (Hajafi and Tia 2004, Newman et al. 1993, Schmitz et al. 2004). The compressive strength can be very sensitive to the mix. Therefore, a trial mix is recommended if this material is expected to be excavated in the future. ACI (1999) noted that blockage of pumping equipment can result if there is segregation of particles, high fines content, or improper mixing. Also, the final grade level after placement will likely be lower than during placement because of the reduction in volume of the material as water is released. ACI (1999) has reported that settlement equal to 1/8 to 1/4 inches per foot of depth is typical and that designers need to consider subsidence in their quantities and in plan preparation. Finally, another disadvantage is the cost of CLSM fills which is considerably higher than the cost of structure fills even though CLSM fills are less labor intensive.

Overall, the performance of CLSM has been good, and a survey by Trejo et al. (2004) indicated that 42 out of 44 State DOTs have specifications for CLSM. A study of US 177 bridges in Oklahoma compared different backfills behind bridge abutments (Snethen and Benson 1998, and Snethen et al. 1997), and the results of the CLSM approach showed very little movement prior to placement of the pavement.

Reinforced Concrete Approach Slabs

The intended function of a reinforced concrete approach slab is to bridge the voids and settlement of the underlying backfill to produce a smoother transition by stiffening the approach and to seal surface water from entering the backfill (Ha et al. 2002). Hoppe (1999) surveyed 39 State DOTs and found that 55 percent use approach slabs on all integral bridges and the remaining use them in excess of 80 percent. The survey by Allen (2002) indicated that 32 out of 48 State DOTs have viewed approach slabs as successful. It is to be noted that WisDOT uses approach slabs, without sleeper slabs (footings), where the approach slab is tied to the abutment on one end and rests directly on the approach embankment fill on the other end.

Typically, the approach slabs are tied to the abutment on one end and rest on a sleeper slab in the embankment fill at the other end (Wahls 1990); however, two studies have tried different techniques to support the approach slabs. In Louisiana, the DOT has supported the approach slabs on piles (Das et al. 1999). The lengths of the piles decrease the farther they are from the abutment. However, because the piles are stopped within a consolidating fill, the piles have been subjected to down drag and have settled themselves. In research conducted by Wong and Small (1994), the approach slabs were placed at an angle from the horizontal with the idea that the roadway stiffness gradually increases as the depth of the slab decreases. The study researches slabs placed at 5 and 10 degrees from horizontal and showed that the approach settlement was most gradual with the slab at 10 degrees. Results also indicated that the angled approach slabs only needed to be as long as the critical depth, or the depth, which is influenced by traffic loading.

Advantages of approach slabs on integral abutments include reduced cost by elimination of expansion joints, improved seismic performance, and a smoother ride over the bridge (Arsoy et al. 1999). Hoppe (1999) found that 81 percent of the 39 State DOTs perceived that approach slabs did provide a smoother ride, and 41 percent indicated that approach slabs reduced the impact to the bridge. However, 75 percent of the DOTs thought that approach slabs were costly, and 52 percent stated they had maintenance issues with recurring settlement.

Hydraulic Fills

Hydraulic fills consist of granular soils that are mixed with water and placed wet. Typically, the granular soils and water are mixed in a concrete mixing truck and are dumped into the confined area specified as the backfill prism. After the water drains, the backfill compacts by gravity. In Wisconsin, the hydraulic fill is constructed by placing granular materials and then flooding the site, rather than mixing in a truck. In general, the hydraulic fill method is fast and less expensive than other methods due to the reduced labor required. However, the compaction may be uneven due to segregation, and post-construction settlement may occur if the water is not drained properly.

2.2.2 Techniques to Improve Foundation Soils

This section describes methods used to reduce deformation or to improve the performance of the foundation soils. Depending on the strength of the foundation soils, these mitigation methods may not be necessary. Overconsolidated clays and silts and bedrock deposits typically do not require any foundation improvement; however, loose silts, weak clay deposits, or organic soils are settlement prone and typically require improvement. Mitigation techniques include, but are not limited to, removal and replacement of weak soils, ground improvement by mechanical or chemical means, surcharging with or without wick drains, and supporting the embankment on deep foundations.

Removal and Replacement of Weak Foundation Soils

This method is simply excavating the weak foundation soils and replacing them with better materials. Depending on the depth of the weak soils, this can be a cost-effective solution. Typically, if weak soils are within 10 to 15 feet below grade and above the water table, excavation and replacement is less expensive than any of the other mitigation methods for foundation soils.

Some DOTs have replaced weak soils as much as 30 feet by means of the displacement method. Instead of removing the material by excavation, the weak material is pushed out by continually adding a backfill heavier than the foundation soils (Wahls 1990). This technique reduces the cost of excavation and can be performed underwater; however, it is typically uncontrolled and often results in soft pockets due to improper placement of backfill.

Ground Improvement (mechanical or chemical)

Ground improvement of foundation soils can be achieved by mechanical or chemical means. Improvement by mechanical means includes stone columns, rammed aggregate, dynamic compaction, or vibrocompaction. Improvement by chemical means includes deep soil mixing or grout or lime stabilization. Ground improvement techniques are limited to the depth of weak soils. Most are effective up to 30 feet below grade but this depends also on the soil type and density. For instance, dynamic compaction or vibrocompaction methods are effective only in granular deposits. In addition, these methods are typically more expensive than other alternatives and can produce varying results.

Surcharging

If time is available during construction, surcharging can be a cost-effective solution, especially if weak soils are deeper than what is feasible to excavate and replace. Surcharging is the controlled and staged placement of the embankment at or above the final elevation of the roadway. The weak soils under the embankment are then allowed to settle under the newly placed fill. Depending on the thickness, depth, and drainage patterns, the time to surcharge could be in the range of 6 to 24 months for as much as 90 percent of the total consolidation settlement. With the use of wick drains, the time of surcharging decreases. The amount of settlement in the time allowed can also be increased by increasing the height of the embankment to greater than its final height.

Supporting Embankment on Deep Foundations

The fourth mitigation method for deformation of foundation soils is to support the embankment on deep foundations. Deep foundations include piles (H-piles, pre-cast concrete, concrete-filled steel pipe, timber, micro-piles, auger-cast) or drilled shafts. The loads from the embankment can be transferred to the foundations by means of a concrete slab or by geogrids. Many studies have been conducted on geo-reinforced embankments on deep foundations. Multiple layers of geosynthetic reinforcement act as a load transfer platform and allow greater spacing of foundations than a concrete reinforced slab. Tension in the reinforcement facilitates soil arching between the piles (Collin et al. 2005, Han and Collin 2005, and Stewart and Filz 2005). As much as 15 to 20 percent reduction in the number of piles has been reported (Vega-Meyer and Shao 2005). Because the embankment is supported by a rigid foundation, the settlement of the embankment is minimal (Eith et al. 2005).

The use of deep foundations reduces, if not eliminates, the majority of concerns that cause deformation of foundation soils; however, this is typically one of the most costly foundation mitigation methods.

2.2.3 Methods to Enhance Drainage

The third major cause of bridge approach settlement is poor drainage. Ways to improve and enhance drainage include:

- Increase side slopes to greater than 2 horizontal to 1 vertical (i.e. make slopes flatter).
- Place drains at the back and/or low points of the backfill prism in order to intercept groundwater. White et al. (2005) determined that geocomposite drainage systems increased drainage 7 to 12 times greater than porous backfill drains. In Wisconsin, it is required to install a perforated pipe underdrain at the back of the abutment at the low point of the backfill prism (on top of the abutment foundation). The pipe underdrain must be day-lighted and must have a minimum diameter of 6 inches and a minimum slope of 0.5%. The pipe is enclosed in a 1.5 ft × 1.5 ft size I coarse aggregate. The aggregate is wrapped with a geotextile fabric type DF (Drainage Fabric) with 1.5 ft overlap.
- Place drains within the pavement base to intercept surface water from entering the backfill prism.
- Wrap drains in a geotextile fabric so that the drains do not clog. White et al. (2005) determined that geotextile wrapped fabrics had 4 times more drainage that porous backfill drains.
- Place geotextile fabric at the interface between the embankment material and backfill and between the pavement base and backfill to mitigate migration of fines.
- Route surface water and groundwater to storm sewers or drainage swales that effectively divert water away from the abutment without eroding surrounding soil.
- Eliminate weep holes in the abutment or drop drains from the bridge deck.
- Maintain watertight joints.
- Add geotextile fabric under the slope protection.
- Extend the wingwalls back in order to enclose backfill near the abutment.
- Limit P200 material to less than 15 percent in backfill and in pavement bases within 5 feet below grade.

- Extend the limits of the backfill prism to at least 2 feet beyond the back of the footing at the bottom of the abutment with a backslope of at least 1 horizontal to 1 vertical.

2.3 MAINTENANCE TECHNIQUES

Wahls (1990) noted that the three most used maintenance techniques of bridge approach settlement included asphalt overlays, slabjacking, and replacement of the approach slab. Stark et al. (1995) indicated that the type of corrective measures is dependent on what is most cost-effective for the remaining service life of the structure. Asphalt overlays and slab jacking are speedy as opposed to replacement of the approach slab. This means the first two methods require less traffic control and are viewed by the public as less intrusive than replacement of the approach slab.

Asphalt Overlays

Asphalt overlays or patching are typically the least expensive of the three methods described in Section 2.3 but do not address the cause of the problem. In most cases, adding weight to the backfill induces greater settlement. The cost is about $200 for an asphalt wedge (one approach), which is a temporary fix, and about $4000 for an overlay of one approach (Allen 2002).

Slab jacking

Slab jacking is a technique that lifts the concrete approach slab by injection of material, typically grout or foam, underneath the slab. A number of spaced holes are typically drilled through the slab so that a uniform lift can occur. If the slab is raised unevenly or jacked with too much force, the slab may break, thus requiring a replacement. The cost of slab jacking is in the low thousands for one approach (Allen 2002 and Schafer and Koch 1992).

Replacing the Approach Slab

If the approach slab has faulted, broken, or settled excessively and cannot be jacked, the slab will likely have to be replaced. This is the most costly of the three alternatives and is in the range of about $10,000 to $15,000 (Allen 2002 and Schafer and Koch 1992). Based on unit prices from WisDOT, a concrete pavement approach slab is about $87 per square yard. For example, if the bridge had 2 approach slabs that were 30 feet wide by 25 feet long, the cost would be about $14,500.

3.0 FIELD TEST SITES

To investigate mitigation methods in Wisconsin, WisDOT has funded the present research study titled: "Evaluation of Bridge Approach Settlement Mitigation." As part of the study, four bridges were to be selected and evaluated over seven years as case studies for two mitigation techniques: geosynthetic reinforced fill, and flowable fill (CLSM). The goal of the research study is to determine the most cost-effective backfill methods that can be competently executed during construction and that can reduce maintenance costs in Wisconsin. The purpose of this report is to document the performance and effectiveness of two mitigation techniques, geosynthetic reinforced fill and flowable fill, conducted on four Wisconsin bridges. Section 3 will address the selection of the field test sites, description of the four bridges selected, specifications of the backfill materials, and instrumentation for field monitoring.

3.1 SELECTION OF FIELD TEST SITES

In 2002, two bridges located along State Highway 173 in Nekoosa, Wisconsin (District 4) were selected. These bridges are: the Cranberry bridge and the Hemlock bridge. Two mitigation techniques: CLSM fill and geosynthetic reinforced fill were applied to these two bridges that were constructed and instrumented in 2002. These two bridges were selected from a list of 15 bridges. Each bridge was evaluated based on Annual Daily Traffic (ADT), traffic speed, embankment fill height and width, abutment height, soil conditions, pavement type, and existing conditions based on a site visit. The optimum bridge sites for this research were those that would be constructed as a new overpass; have fill / abutment heights greater than 10 feet; have good foundation soils; be paved with asphalt; and have a high ADT. The reason of choosing a “good foundation soil” criterion is to determine if the approach settlement in the two selected bridges is due to backfill settlement rather than foundation settlement.

In continuation of the study, two additional bridges were selected. These two bridges are: the Western Avenue over Cedar Creek bridge (B-66-135) in Washington County, and the Wisconsin and West Beloit Avenue over the Root River bridge (B-40-700) in Milwaukee County, Wisconsin. A total of 18 bridges in District 2 were reviewed prior to the selection of these two bridges. Each bridge was evaluated based on Annual Daily Traffic (ADT), traffic speed, embankment fill height and width, abutment height, soil conditions, pavement type, and existing conditions based on a site visit. The optimum bridge sites for this part of the research were those that would be constructed as a new overpass; have fill / abutment heights greater than 10 feet; be wider and higher than the existing bridge; have fair to poor foundation soils; be paved with asphalt; have a high ADT; and have poor existing performance.

These optimum conditions were grouped into five major categories: traffic, soil conditions, abutment, embankment loading due to widening and/or filling, and existing performance. Each bridge was then given a rating for each category with 1 being low, 2 being moderate, and 3 being high. A high rating was equal to the most optimum condition. For instance, an abutment with a fill height between 7 and 9 feet would receive an abutment rating of 2 whereas an abutment greater than 13 feet high would receive a 3. Bridges with poor cohesive foundation soils were viewed as optimum because the previous Nekoosa bridges (District 4) had been founded on granular soils.

After rating each category for each bridge, an overall rating was calculated based on a weighted average, as follows:

Overall Rating (%) = $100 * (3*R_T + 5*R_S + 2*R_A + 3*R_E + 2*R_P) / 45$ .... Equation 1

where R_T = Traffic Rating; R_S = Soil Rating; R_A = Abutment Rating; R_E = Embankment Rating; R_P = Existing Performance Rating

Greater influence factors were placed on the soil, traffic and embankment conditions, as these were perceived as the major contributors to bridge approach settlement of the five individual ratings. The bridges were then ranked in order of highest overall rating. Those that had higher ranks but were anticipated to be constructed with an approach slab were eliminated. In order to effectively determine the settlement and the movement of the approach with the instrumentation selected for this study, an asphalt approach was necessary.

Based on the overall ratings, rank, and pavement type, the top two bridges, Western Avenue over Cedar Creek (B-66-135) and West Beloit Avenue over the Root River (B-40-700), were selected and approved for field study. A complete listing of bridges reviewed is included in Appendix A.

3.2 DESCRIPTION OF SELECTED FIELD TEST SITES

The four bridge sites that were selected for monitoring included:

- B-71-116: STH 173 over Hemlock Creek in Nekoosa, Wisconsin (Hemlock Bridge)
- B-71-127: STH 173 over Cranberry Ditch in Nekoosa, Wisconsin (Cranberry Bridge)
- B-66-135: Western Avenue over Cedar Creek in Washington County, Wisconsin (Western Bridge)
- B-40-700: West Beloit Road (CTH T) over Root River in Milwaukee County, Wisconsin (Beloit Bridge)

The following sections 3.2.1 through 3.2.4 describe the site conditions, subsurface and groundwater conditions, and existing and proposed design details.

3.2.1 B-71-116: STH 173 over Hemlock Creek

The Hemlock Bridge is located in a rural area in Central Wisconsin along STH 173. The area is relatively flat and surrounded by woods. Hemlock Creek flows to the south and is about 120 feet wide and 5 feet deep with a sandy bottom.

The existing structure B-71-01 at the site was a single span steel truss with a concrete slab supported on concrete abutments. The existing structure was built in 1949 and was approximately 125 feet long and 24 feet wide. The elevation of the existing grade was approximately 980 feet, MSL. From structure survey reports, the existing bridge was in fair condition (Hardinger 2000). The approach pavement and the 8-foot embankment fill were reported to be in good condition (Althaus 2000).

For the design of the new pavement and bridge, an ADT was estimated to be 2000 vehicles per day (vpd) in 2002 and 2600 vpd in 2022 at a design speed of 60 miles per hour (mph).   The design traffic load was estimated to be 671,600 ESALs. The existing pavement was 6 inches asphalt over

4.5 inches crushed aggregate base course in contrast to the new 6 inches asphalt over 12 inches of crushed aggregate base course. The new roadway grade behind the bridge is only about 1 to 2 feet higher than existing, but about 2 to 5 feet of new fill was placed on embankment side slopes. Originally the plan was to incorporate approach slabs that were 24 feet long, 30 feet wide, and 12 inches thick; however, due to the change order as a result of the research project, the approaches were modified to all asphalt.

The Hemlock Bridge is a 2-span 36-inch prestressed girder bridge that extends about 23.5 feet from either side of the existing structure and into the existing embankment. The total span is about 148.5 feet long across the creek. The creek had a normal water elevation of 967 feet, MSL in 1999 and has a 100-year high water elevation of 974.8 feet, MSL, which is within a few feet from the underside of the girders. The width of the new bridge is 36 feet, or 12 feet wider than the existing. The new bridge and approach is skewed at 30 degrees, which is parallel to the creek.

The abutments for the Hemlock Bridge are classified as semi-integral and stub abutments, with a paving notch. The heights of the abutments are approximately 8.9 feet from the bottom of the footings to the top of the proposed grade. Wingwalls extend about 10 feet directly behind the back of the abutment and are 3.25 feet wide resting on the embankment fill. The backfills behind the abutments and wingwalls were selected to be WisDOT Structure Backfill on the East side and Flowable Fill (CLSM) on the west side. The slopes in front of the abutment are graded at about 1.5H:1V and covered with heavy riprap underlain by a geotextile fabric. The side slopes are inclined at about 2.5H:1V and covered with topsoil.

The subsurface conditions consist of very loose to loose fine-grained sand / silty sand to an elevation ranging between 954 and 959 feet, MSL (see Appendix B and page 125). The sand is underlain by a 5 to 10-foot thick dense to very dense silt layer over dense fine-grained sand. Below the sand and silt, granite bedrock is encountered from 68 to 73 feet below grade or between Elevations 908 and 913 feet, MSL. The bases of the abutments as well as the embankment are located within the very loose to loose sand / silty sand. The abutments are supported on single rows of seven 10x42 H-piles. The 55-ton piles were estimated to be 55 and 85 feet long, extending to bedrock, for the East and West abutments, respectively.

The Hemlock Bridge was constructed in the summer of 2002 and completed in one stage. The test pile records from WisDOT (2002) indicated that bearing capacities were 56 tons at 49 feet at the East Abutment and 73 tons at 75 feet at the West Abutment. Photos of the construction are presented in Figures 35 and 36. Plans sheets dated 2001 from WisDOT (including a design plan and elevation view, contour map, and subsurface exploration sheet), the site investigation report (Althaus 2000), and the structure survey report (Hardinger 2000) are included in Appendix B.

Figure 35: Looking at West Abutment of Hemlock Bridge

Figure 36: Close-up View of Flowable Fill at West Abutment of Hemlock Bridge

3.2.2 B-71-127: STH 173 over Cranberry Ditch

The Cranberry Bridge is located east of Hemlock Bridge along STH 173. The area is relatively flat and surrounded by woods, cranberry bogs, and marsh. The southerly flowing ditch is about 50 feet wide and 5 feet deep with mucky silt bottom (Althaus 2001).

The Cranberry Bridge replaced the existing structure B-71-762, which was a single span concrete slab supported on concrete abutments. The existing structure built in 1968 was approximately 51 feet long and 24 feet wide. The elevation of the existing grade was approximately 985.5 feet, MSL. From the structure survey reports (Hardinger 2001), the existing bridge was in fair condition. The 4-foot approach fills were reported to be in good condition (Althaus 2001).

For design of the new pavement and bridge, an ADT was estimated to be 2200 vpd in 2002 and 3300 vpd in 2022 at a design speed of 60 mph. The design traffic load was estimated to be 737,300 ESALs. The existing pavement was 6 inches of asphalt over 4.5 inches of crushed aggregate base course in contrast to the new 6 inches of asphalt over 12 inches of crushed aggregate base course. The new roadway grade behind the bridge is only about ½ to 1 foot higher than existing. Fill along the embankment side slopes is up to 7 feet higher than existing. No approach slabs were incorporated for this bridge design.

The new Cranberry Bridge is a two-span concrete flat slab supported on concrete abutments with a paving notch. The span is about 48.6 feet long across the ditch and is shorter than the existing bridge. The creek had a normal water elevation of 978.7 feet, MSL in 2000 and a 100-year high water elevation of 983.9 feet, MSL, which is within a few feet from the top of the road. The width of the bridge is 36 feet, or about 12 feet wider than the existing. The new bridge and approach are skewed at 15 degrees, which is parallel to the ditch.

The abutments for the Cranberry Bridge are classified as pile-encased abutments per WisDOT, which are similar to stub and integral abutments. The abutment heights are approximately 9.8 feet from the bottom of the footings to the top of the proposed grade. The 2-feet wide wingwalls flare about 11 feet from the back of the abutments at 30 degrees. The backfills behind the abutments and wingwalls were selected to be WisDOT Structure Backfill on the east side and geosynthetic reinforced fill on the west side. The backfill is drained by a granular material wrapped in geotextile connecting to a 2-inch weep hole that exits in front of the abutment at the top of the front slope. The slopes in front of the abutment are inclined at 1.5H:1V and covered with heavy riprap underlain by a geotextile fabric. The side slopes are graded at 2.5H:1V to 4H:1V and covered with topsoil.

The subsurface conditions consist of very loose to loose fine to medium and medium to coarse sand to elevations ranging between 961 and 963 feet, MSL (see Appendix C and page 146). The sand is underlain by a 10-foot thick loose silt layer over medium dense to dense fine- to medium-grained sand. Below the sand and silt, sandstone bedrock is noted at about 45 feet below grade or Elevation 942.5 feet, MSL. The bases of the abutments as well as the embankment are located within the very loose to loose sand. The abutments are supported on single rows of eight 10x42 piles H-piles. The 55-ton piles were estimated to be 40 feet long for both abutments, which extend to bedrock.

The Cranberry Bridge was constructed in the summer of 2002 and completed in one stage. The test pile records indicated that bearing capacities were 93 tons at 39 feet at the East Abutment.

Photos of the construction are presented in Figures 37 and 38. Plans sheets dated 2001 from WisDOT (including a design plan and elevation view, contour map, and subsurface exploration sheet), a site investigation report (Althaus 2001), boring logs, and the structure survey report (Hardinger 2001) are included in Appendix C.

3.2.3 B-66-135: Western Avenue over Cedar Creek

The Western Bridge is located in the Town of Jackson, Washington County, Wisconsin. The surrounding area is rolling to hilly farmland and is wooded adjacent to the creek. Surface water of the area drains into Cedar Creek, which flows to the northwest. The creek is about 20 feet wide and 2 feet deep with a sandy silt bottom.

Figure 37: Cranberry Bridge Construction looking west

Figure 38: Compaction of Structure Backfill behind West Abutment of Cranberry Bridge

The Western Bridge replaced the existing structure P-66-70, which was a single span steel girder bridge supported on concrete abutments. The existing structure was built prior to 1941 and was approximately 43 feet long and 24 feet wide. The elevation of the existing grade was approximately 851 feet, MSL.

Based on site visits and the structure survey report by Olsen (2003), the bridge, substructure and approaches were in poor to very poor condition. Photos of the existing bridge taken in March 2004 are presented in Figures 39 to 43. As shown in the photos, the bridge had spalling concrete, and voids were noted between the approaches and the bridge. The approaches had numerous asphalt patches and had settled significantly. Two borings adjacent to the existing abutments indicated the asphalt pavement was 16.5 to 17 inches thick over 3 feet of base course. A third boring located 12 feet west of the west abutment boring indicated only 5 inches of asphalt over 1.2 inches of base course. In addition, the wingwalls appeared to have rotated because voids were noted between the bridge and the walls. Steep embankments surrounded the wingwalls, and greater lateral deflections were noted on the north side. Erosion was observed in a ditch located parallel to the embankment on the northeast side of the bridge.

Figure 39: Looking East at Western Bridge

Figure 40: Approach at West Abutment of Western Bridge

Figure 41: Approach at East Abutment of Western Bridge

Figure 42: Looking at South side of Western Bridge

Figure 43: Drainage Ditch on Northeast Side of Western Bridge

For the replacement pavement and bridge, an ADT was estimated to be 1300 vpd in 2004 and 2100 vpd in 2024 at a design speed of 50 mph. The design traffic load was estimated to be 292,000 ESALs. The new pavement consists of about 4 inches of asphalt over 18 to 24 inches of base and subbase. The new roadway grade behind the bridge is about 1 foot lower than the existing, but up to 5 to 10 feet of fill is noted along the embankment side slopes. No approach slabs were incorporated for this bridge design.

The new Western Bridge is a single span 27-inch prestressed concrete box girder bridge. The span is about 65.5 feet long, which is 8.5 to 12.5 feet longer on either side than the existing bridge. The creek has a measured water elevation of 838.4 feet, MSL in 2003 and a 100-year high water elevation of 848.3 feet, MSL, which is within a few feet from the top of the road. The width of the new bridge is about 36 feet, which is about 12 feet wider than the existing. The new bridge and approach is skewed parallel to the creek at 40 degrees.

The abutments for the Western Bridge are classified as sill (stub) and integral abutments per WisDOT, without a paving notch. The abutment heights are approximately 8 feet from the bottom of the footings to the top of the proposed grade. Wingwalls are about 2.5 feet wide and flare about 7.5 feet from the back of the abutment at 40 degrees. The backfills behind the abutments and wingwalls were selected to be WisDOT Structure Backfill on the west side and geosynthetic reinforced fill on the east side. The backfill is drained by a 6-inch pipe underdrain wrapped in geotextile that exits through the wingwall and drains to the creek. The slopes in front of the abutment are inclined at 1.5H:1V and are covered with heavy riprap underlain by a geotextile fabric. The side slopes are graded at 2.5H:1V and are covered with topsoil and/or riprap.

The subsurface conditions beneath the pavement consist of 10 feet of stiff to very stiff clayey silt to silty clay fill with gravel and cobbles over 1 to 4.5 feet of loose peat (see Appendix D and page 163 and 165). The peat is underlain by silty clays and clayey silts that are medium stiff to stiff and extend to an elevation between 820 and 825 feet, MSL. A dense to very dense clayey sand / sandy silt / sandy clay layer is noted below the clay and silt. The borings were terminated by auger refusal in a limestone gravel layer between 43 and 45 feet below grade. The bases of the abutments as well as the embankment are located at the top of the loose peat. The abutments are supported on single rows of seven 10-3/4 inch cast-in-place concrete pipe piles. The 55-ton piles were estimated to be 23 feet long for both abutments, extending into a dense to very dense clayey sand.

The Western Bridge was constructed in September through October of 2004 and completed in one stage. Photos of the construction are presented in Figures 44 to 47. Plan sheets dated 2003 by

Omni (including a design plan and elevation view and subsurface exploration sheet), geotechnical exploration report by Arnold (2003), and the structure survey report (Olsen 2003) are included in Appendix D.

3.2.4 B-40-700: West Beloit Road over Root River

The Beloit Bridge is located along County Trunk Highway T (CTH T) or Beloit Road in the City of Greenfield, Milwaukee County, Wisconsin. Marshy wetlands and woods surround the area adjacent to the Root River, which flows to the south and is part of the Root River Parkway, a Milwaukee County Park. The topography is gently rolling to the east and west of the parkway, where residences, businesses, and heavy traffic are located. The river is about 15 feet wide and 1 foot deep with a sandy bottom (McMahon 2003).

Figure 44: West Abutment at Western Bridge (Structure Fill)

Figure 45: Placing backfill on Reinforcement at East Abutment of Western Bridge (Geosynthetic Fill)

Figure 46: Compacting Backfill above Reinforcement at East Abutment of Western Bridge

Figure 47: Underdrain exiting the East Abutment of Western Bridge

The Beloit Bridge replaced the existing structure P-40-727, which was a single span concrete slab supported on concrete abutments. The existing structure was built in 1935 and was approximately 27 feet long and 51 feet wide. The elevation of the existing grade was approximately 729 feet, MSL.

Based on a site visit and structure survey report by McMahon (2003), the bridge, substructure and approaches were observed to be in poor condition. Photos of the existing bridge taken in March 2004 are presented in Figures 48 to 51. As shown in the photos, the bridge had spalling concrete, exposed rebar, and distressed concrete. The eroding banks of the river appear to consist of silts and clays. The asphalt approaches indicated some transverse cracking. The shoulders were gravel, and steep embankments surrounded the bridge.

Figure 48: Looking East at Beloit Bridge

Figure 49: Approach at East Abutment of Beloit Bridge

Figure 50: Approach at West Abutment of Beloit Bridge

Figure 51: Erosion of banks at south side of Beloit Bridge

For the replacement pavement and bridge, an ADT was estimated to be 14,400 vpd in 2003 and 26,000 vpd in 2023 at a design speed of 40 mph. The design traffic load was estimated to be 2,766,700 ESALs. The new pavement consists of about 8 inches of asphalt over 12 to 18 inches of base and subbase. The new roadway grade behind the bridge is about 7 feet higher than existing. No approach slabs were incorporated for this bridge design.

The new Beloit Bridge is a single span concrete flat slab supported on concrete abutments. The span is about 50.5 feet long across the river and a new bike path, which is 23.5 feet longer than the existing bridge. The creek has a measured water elevation of 719.8 feet, MSL in 2002 and a 100-year high water elevation of 729.4 feet, MSL, which is 7 feet from the roadway or about the level of existing grade. The width of the new bridge is about 71 feet, or 20 feet wider than the existing. The new bridge and approaches are skewed parallel to the river at 5 degrees.

The abutments for the Beloit Bridge are classified as pile-encased abutments, which are similar to stub and integral abutments, without a paving notch. The abutment heights are approximately 13.5 feet from the bottom of the footings to the top of the proposed grade. Wingwalls are about 3 feet wide and 21 feet long. The backfills behind the abutments and wingwalls were selected to be WisDOT Structure Backfill on the east side and Flowable Fill on the west side. The backfill is drained by a 6-inch pipe underdrain, wrapped in geotextile, which extended around the wingwall and drained to the bottom of the slope. The slope in front of the abutment is inclined at 1.5H:1V and covered with heavy riprap underlain by a geotextile fabric. The side slopes are graded at 3H:1V and covered with topsoil and/or riprap.

The subsurface conditions consist of 3 to 4 feet of sand and gravel fill over 2.5 feet of stiff to very stiff silty clay (see Appendix E and page 189). The fill is underlain by 2 to 3.5-foot thick layer of very soft black to greenish-gray organic silt. The elevation of the top of the organic silt layer is about 721 feet, MSL. Below the organic silt is 5 feet of loose silty sand over alternating layers of medium stiff to hard silty clay and dense silty sand. The bases of the abutments are located about ½ to 1 foot above the organic silt layer. The abutments and attached wingwalls are supported on single rows of 27 H-piles that are 10x42 H-pile sections. The 35-ton piles were estimated to be 35 feet long for both abutments, extending into a very stiff to hard silty clay.

The Beloit Bridge was constructed between May and November of 2004 and completed in two stages. Due to the high volume of traffic, the bridge had to remain open. The north side of the bridge was constructed first between May and July of 2004, as the east and westbound traffic remained on the south side of the existing bridge. The traffic was then switched, and the south side of the bridge was constructed between July and November 2004. The north side of the bridge was supported by sheet piles and wire-faced baskets lined with geotextile at the center line of the roadway. Photos of the construction are presented in Figures 52 to 54. Plan sheets dated 2003 by Ayres (including a design plan and elevation view, contour and subsurface exploration sheet), an abbreviated version of the geotechnical exploration report by Singh (2003), and the structure survey report by McMahon (2003) are included in Appendix E.

Figure 52: Looking East at West Abutment of Beloit Bridge during Staged Construction

Figure 53: Excavation of West Abutment of Beloit Bridge

Figure 54: Behind East Abutment of Beloit Bridge

3.3 BACKFILL SPECIFICATIONS

As explained in Section 3.2, one approach of each bridge would be backfilled with Structure Backfill per WisDOT Standard Specifications and the other approach would be backfilled with either flowable or geosynthetic reinforced fills, as indicated in Table 3.

Table 3: Backfill Types of Field Test Sites

Bridge	Description	Abutment	Backfill
B-71-116	Hemlock Bridge	East	Structure Backfill
		West	Flowable Fill
B-71-127	Cranberry Ditch Bridge	East	Structure Backfill
		West	Geosynthetic Reinforced Fill
B-66-135	Western Bridge	East	Geosynthetic Reinforced Fill
		West	Structure Backfill
B-40-700	Beloit Bridge	East	Structure Backfill
		West	Flowable Fill

This Section describes the cost and backfill specifications of the three backfill types that were implemented during construction of the bridge approaches.

WisDOT Structure Backfill

Structure Backfill is typically used as backfill behind abutments and retaining walls on the majority of WisDOT bridges. The gradation specifications for Structure Backfill are noted in Section 210 (WisDOT 2004) and as follows: “Furnish and use mixture of sand and gravel, crushed gravel, crushed stone, crushed concrete, or other fragmented mineral. The maximum material size used shall have 100 percent passing a 3-inch (75 mm) sieve, not less than 25 percent by weight passes a No. 4 (4.75 mm) sieve and, of the material passing the No. 4 (4.75mm) sieve, not more than 15.0 percent passes a No. 200 (75 μm) sieve.“

Standard Compaction is typically specified for compacting Structure Backfill on the majority of WisDOT bridges. The compaction requirements are noted in Section 206.3.13 of the 2004 WisDOT Standard Specifications, which read: “Unless specified otherwise, place backfill in continuous horizontal layers no more than 8 inches (200 mm) thick. If practical, uniformly raise layers on all sides of each substructure unit or culvert. Surround the stone used in backfilling by finer material. Compact each layer, before placing the next layer, by using engineer-approved rollers or portable mechanical or pneumatic tampers or vibrators.”

<u>Inspection of compaction is limited to visual means for Standard Compaction unless otherwise specified.</u> Testing can be required if Special Compaction, Section 207.3.6.3, is used. Special compaction specifications state that at least 95 percent of the maximum dry density must be

achieved according to AASTHO T-99, Method C, except replacing the material retaining on the ¾ inch sieve with No. 4 to ¾ inch material.

For field tests, standard compaction of Structure Backfill was used because this is most commonly specified on WisDOT plans. The Structure Backfill was tested to verify that the gradation requirements were met prior to placing. The sieve laboratory sheets for each of the bridges are included in Appendix F. All of the sources met the gradation specifications with exception of the north half of the east abutment of the Beloit Bridge.

Flowable Fill

For approaches backfilled with Flowable Fill, a low strength slurry backfill material was used. The slurry had to be self-leveling and achieve a minimum 28-day strength of 100 pounds per square inch (psi). The required mix design consisted of 3200 pounds of sand, 200 pounds of fly ash, 50 pounds of Type 1 Portland Cement, 7.5 oz of Water Reducer, and 45 Gallons of Water for 1 cubic yard. No compaction specifications were necessary, as the Flowable Fill was placed in one lift. The mix design submittal for the Flowable Fill used on the West Abutment of Beloit Bridge is included in Appendix F.

Geosynthetic Reinforced Fill

For the approaches with geosynthetic reinforced fill, a 3-inch foam insulation board was installed behind the abutment. The geosynthetic reinforced fill consisted of 12-inch layers of WisDOT structure backfill compacted at 95 percent relative compaction between layers of Geolon HP570 fabric. The fabric had a grab tensile strength of 475 lb (MD) and grab tensile elongation of 12% (MD). The certifications for the fabrics used for the Cranberry and Western Bridges are included in Appendix F. The sequencing of the geosynthetic reinforced fill as specified is shown in Figure 55.

Estimated Cost Comparison of Backfill Types

Present value costs were estimated for the three backfill types. These costs are not a life-cycle analysis and do not include any maintenance costs that may occur in the future. Unit prices, presented in Table 4, were obtained from the WisDOT website (WisDOT Unit Prices 2005), recently bid projects in the Milwaukee area, correspondence with concrete manufacturers (Mix On-site 2004), correspondence with geotextile manufacturers (GSI 2005), and US Cost Guides (Get-A-Quote 2005). For a cost comparison between the backfills, a 10-foot high by 36-foot wide exemplar abutment with 140 cubic yards of excavation and backfilling was used. The total cost rounded to the nearest hundred dollars for each backfill type is presented in Table 5.

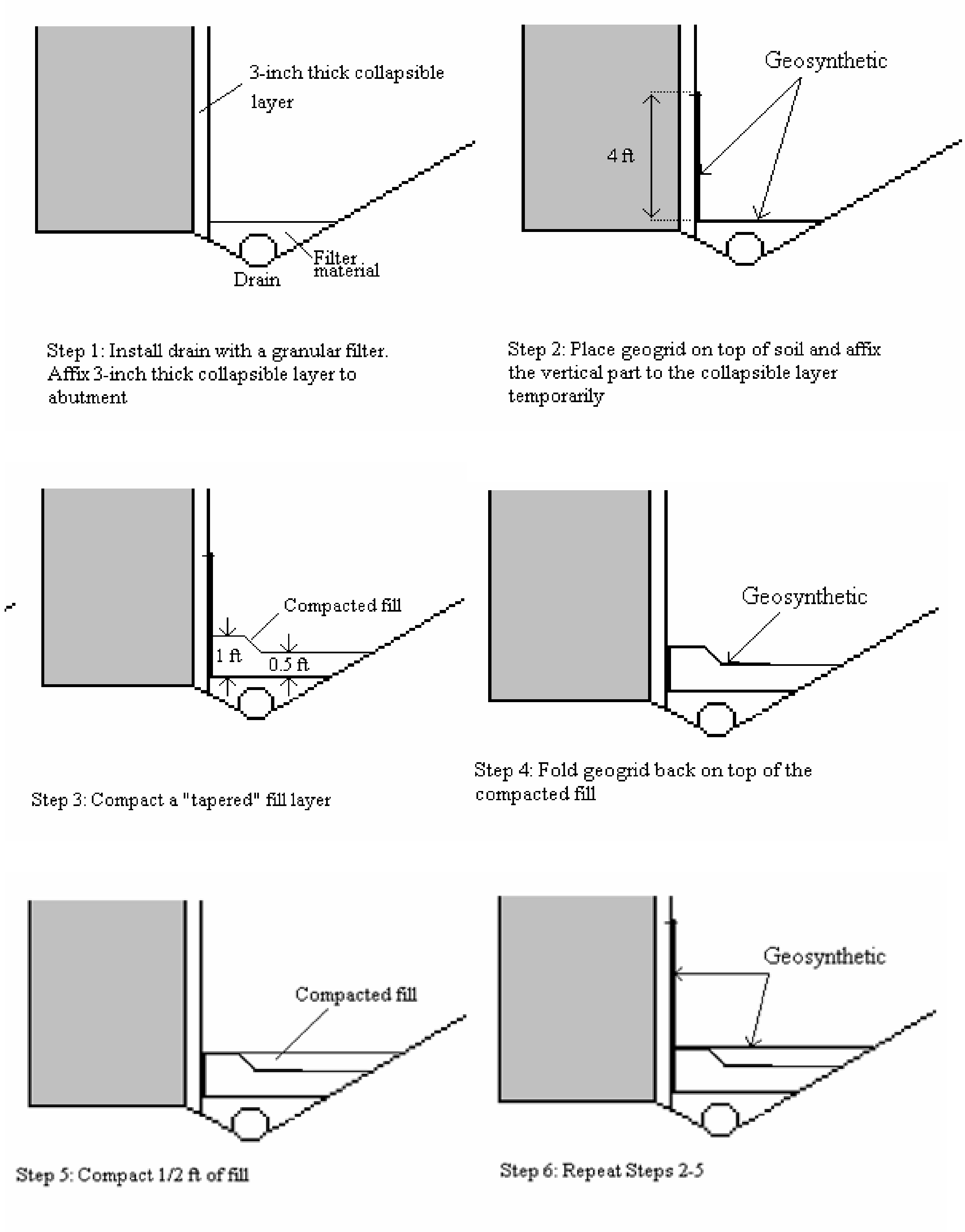


Figure 55: Placement of Geosynthetic Reinforced Fill

Table 4: Unit Prices for Cost Estimate of Backfill Types

Item	Unit	Unit Price		Quantity	Cost	
		Low	High		Low	High
Structure Backfill	yd^3	$11	$18	140	$1,600	$2,600
Excavation of Stiff Clay	yd^3	$25	$35	140	$3,500	$4,900
Flowable Fill	yd^3	$60	$85	140	$8,400	$11,900
Geotextile	yd^2	$1.50	$3.00	375	$600	$1,200
Insulation Board	ft^2	$3.00	$4.00	450	$1,400	$1,800

Table 5: Estimation of Total Cost for Backfill Types

Backfill Type	Excavation		Backfill		Geotextile & Insulation		Total	
	Low	High	Low	High	Low	High	Low	High
Structure Backfill	$3,500	$4,900	$1,600	$2,600	$0	$0	$5,100	$7,500
Flowable Fill	$3,500	$4,900	$8,400	$11,900	$0	$0	$11,900	$16,800
Geosynthetic Reinforced Fill	$3,500	$4,900	$1,600	$2,600	$2,000	$3,000	$7,100	$10,500

3.4 FIELD INSTRUMENTATION PLAN

After construction of the bridges and placement of the asphalt approaches, the field instrumentation was installed. The instrumentation included survey markers at the surface of the asphalt approaches and inclinometers with telescopic casings placed in the backfill and terminated within five feet of dense or very stiff soil layer.

Survey Markers

The purpose of the survey markers was to estimate the surface settlement of the approach pavement over time by surveying the elevations along the approach. For the Hemlock and Cranberry Bridges, markers were placed at 10-foot intervals within 100 feet from the bridge abutment. This was modified for the Western and Beloit Bridges where markers were placed at 5-

foot intervals within 50 feet from the abutment. The monitoring plan consisted of taking elevation shots at each of these points as well as the bridge benchmark to convert to the elevations to those used in design and construction. At least four dates were proposed after construction for all Bridges. The dates at which elevations were taken for each bridge are presented in Table 6.

Table 6: Survey Dates

Bridge	Construction Completion	Installation of Markers	1st Reading	2nd Reading	3rd Reading	4th Reading
Hemlock	8/02	8/02	8/13/02	8/11/03	8/12/04	10/25/06
Cranberry	8/02	8/02	8/13/02	8/11/03	8/12/04	10/25/06
Western	10/04	10/15/04	10/20/04	2/24/05	4/13/05	9/29/06
Beloit	7/04 (North) 11/04 (South) [a]	11/16/04 1/14/05 [b]	1/14/05	2/24/05	4/13/05	9/29/06

(a) Due to staged construction and high traffic volume, markers and inclinometers could not be installed. Closure of temporary lanes was not permitted, so instrumentation was installed when both sides of the bridge were complete.

(b) Local municipality would not allow the second instrumentation to be installed until after the traffic conflicts from the first installation were resolved.

Inclinometers

The purpose of the inclinometers is to measure the lateral and vertical deflection of the backfill and foundation soils over time. The inclinometer casings that were used in all field sites consisted of both rigid and telescopic sections that snap together and are made of 2.75-inch diameter polyvinyl chloride (PVC) casing. The rigid sections came in 5- or 10-foot lengths. The telescopic sections came in 2.5-foot lengths, which can collapse a maximum of 12 inches. The purpose of the telescopic sections is to measure the vertical settlement of specified soil layers below the surface. Because the sections collapse as the soil around it settles, it can be determined whether the approach settlement is a result of the backfill behind the abutment or of the foundation soils. A photo of the casing sections is shown in Figure 56.

Figure 56: Rigid and Telescopic Sections

The boring logs from the previous geotechnical explorations at each of the bridges were used to determine the depths where inclinometer casings were terminated and where the telescopic sections were placed. Typically, telescopic sections were placed in the backfill zone in the upper 5 to 10 feet below grade, just below the backfill zone at 10 to 15 feet below grade, and in the foundation soils at greater than 15 feet below grade. A summary of the approximate depths of the inclinometer and depths of the telescopic sections with corresponding soil type is shown in Table 7.

WisDOT drilling crews installed inclinometers shortly after construction at a distance of 5 feet behind the abutments and in the center line of the approaches. The inclinometers were placed in boreholes that were drilled using a 4-1/4" hollow stem auger and/or mud rotary techniques. At 5-foot intervals, the boreholes were sampled using a 2-inch split spoon barrel driven into the soil with a 140-pound hammer falling 30 inches. Blowcounts were recorded for every 6 inches of penetration.

Table 7: Summary of Inclinometer and Telescopic Section Locations

Bridge	Abutment (Inclinometer)	Total Length / Depth of Inclinometer	Telescopic Sections (a)		
			1st Depth (Soil Type at this Depth)	2nd Depth (Soil Type at this Depth)	3rd Depth (Soil Type at this Depth)
Hemlock	East IN1	44.25’ / 46.0’	10.23’ (Base of Structure Backfill)	42.66’ (Dense Sand)	--
	West IN2	44.25’ / 46.0’	10.17’ (Base of Flowable Fill)	42.58’ (Dense Sand)	--
Cranberry	East IN1	34.17’ / 36.0’	10.17’ (Below Base of Structure Backfill)	32.63’ (Dense Sand)	--
	West IN2	34.17’ / 36.0’	10.17’ (Below Base of Geo-synthetic Reinforced Fill)	32.60’ (Dense Sand)	--
Western	East IN1	29.25’ / 30.0’	1.45’ (Geosynthetic Reinforced Fill)	13.94’ (Loose Silt Beneath Peat)	26.44’ (Hard Clay)
	West IN2	29.94’ / 30.0’	2.54’ (Structure Backfill)	14.44’ (Peat)	26.90’ (Hard Clay)
Beloit	East IN1	34.5 / 35.0’	2.47’ (Structure Backfill)	14.93’ (Organic Silt/ Loose Sand)	27.43’ (Hard Silty Clay)
	West IN2	34.5 / 35.0’	4.52’ (Flowable Fill)	16.98’ (Organic Silt / Loose Sand)	29.44’ (Hard Silty Clay)

The inclinometer casings were aligned in the borehole by a series of grooves in the casing sections. The grooves are manufactured at ¼ points, which allow measurements to be taken along the same line each time. The grooves were positioned so that one set was located perpendicular to the bridge (A axis) and the other was parallel to the bridge (B-axis). The grooves are shown in Figure 57. Once the inclinometer casing was aligned, the boreholes were backfilled with cement slurry, and flush-mounted caps were placed in the pavement to protect the casing from traffic.

Figure 57: Grooves inside Casing

Inclinometer readings and vertical settlement readings were taken near the same time as the survey elevations as shown in Table 8.

Table 8: Inclinometer and Vertical Settlement Reading Dates

Bridge	Construction Completion	Installation of Inclinometers	1st Reading	2nd Reading	3rd Reading	4th Reading	5th Reading
Hemlock	8/02	11/02	11/8/02	9/5/03	8/31/04	10/25/06	-
Cranberry	8/02	11/02	11/8/02	9/5/03	8/31/04	10/25/06	-
Western	10/04	10/15/04	10/16/04	2/24/05	4/13/05	9/29/06	8/30/07
Beloit	7/04 (North) 11/04 (South) [(a)]	11/16/04 1/14/05 [(b)]	1/14/05	2/24/05	4/13/05	9/29/06	8/30/07

(a) Same (a) note as on Table 4; (b) Same note as on Table 4

Readings for the inclinometer were taken at 2-foot intervals for the total length of the casing using the Digitilt Inclinometer Probe shown in Figure 58.

Figure 58: Digitilt Inclinometer Probe

The inclinometer probe has an upper and lower wheel sets that can be inserted in the casing using one of the two groove sets. The inclinometer probe measures the tilt of the casing at two feet intervals. When inserting the inclinometer probe in the casing, the upper wheel is placed in the direction of the expected movement (deformation). The direction of movement was anticipated to be towards the abutment on the A-axis and towards the center line on the B-axis. If the movement is opposite of what was anticipated, the tilt readings are negative. To transfer the tilt readings to the Digitilt Data Mate for the data storage, the probe is connected to a control cable as in Figure 59. A pulley assembly is also shown in Figure 59. This assembly helps to hold the inclinometer probe in place while lowered.

Figure 59: Digitilt Data Mate, Control Cable and Pulley Assembly

Vertical settlement readings are taken using a tape measure with an end hook shown in Figure 60. The hook is first lowered to the bottom of the casing and then pulled upward slowly. A slot is located in the center of the telescopic sections between the grooves. The end hook will grab this slot when the tape is pulled upward. The tape can be read with reference to the top edge of the inclinometer casing. The depth to the slot will change if the telescopic casing and the surrounding soils settle.

Figure 60: Tape Measure with End Hook

A few complications were noted during the reading of measurements at the Western Bridge. During the initial survey at IN2, the probe was blocked at 14 feet below grade. Grout appeared lodged within the grooves, so readings were recorded only from 2 to 14 feet. In addition, the readings taken at IN1 were not valid and therefore were not reported. On the date of the 2nd reading at the Western Bridge, water present in the inclinometer casing froze in both IN1 and IN2 between 2 and 6 feet below grade. To obtain the readings, the ice was removed by adding salt and by breaking it with a hand-held auger.

4.0 PERFORMANCE EVALUATION OF FIELD TEST SITES

The performance of each field test site was addressed by conducting a site visit and evaluating the collected data discussed in Section 3.4. A summary of the evaluated data for each bridge is included in Appendices G through J.

4.1 B-71-116: STH 173 over Hemlock Creek

Numerous site visits have been carried out to all four bridges since they were constructed. Up to the date of this report preparation (June 2007), the Hemlock Creek bridge and its approaches appear to be in very good condition. On August 31, 2004, a site visit was conducted at the same time of the 3rd inclinometer readings. The bridge appeared to be in good condition with no observable cracking or movement. Photos of the bridge taken at the site visit are presented in Figures 61 and 62.

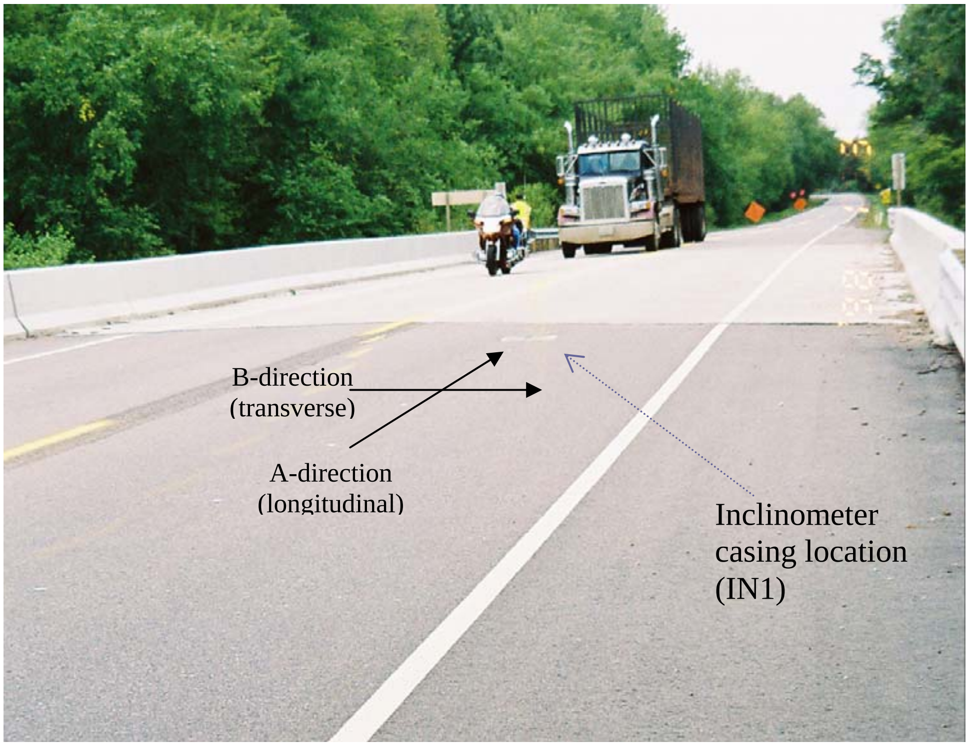


Figure 61: Looking East at Hemlock Bridge

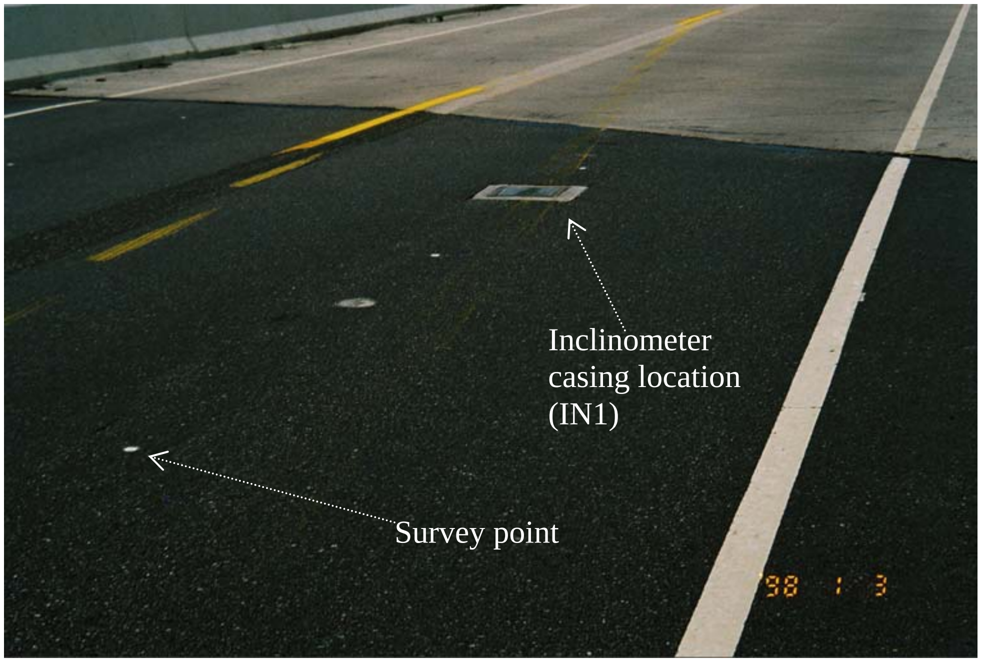


Figure 62: Approach at East Abutment of Hemlock Bridge

Survey Elevations

Visual inspection carried out in October 2006 indicated very small differential settlements (less than 0.1 inches) at the interface between the approach fills and bridge. Survey elevations were taken over a 4-year period from the end of bridge construction on both the eastbound and westbound approaches. Survey elevations of the eastbound lanes approaching the west abutment and the westbound lanes approaching the east abutment are presented in Figures 63 and 64, respectively. The survey indicated differential settlement (see Appendix G) of about 0.3 to 0.5 inches near both abutments (within 7 feet). The differential settlement decreased further away from the abutment until the differential movement was positive. The point where the differential movement changed from negative to positive was about 27 feet from the east abutment (Structure Fill) and 47 feet from the west abutment (Flowable Fill). The transition slope (differential settlement divided by approach length) was calculated to be about 0.015 inches per foot for the westbound lanes and about 0.015 inches per foot for the eastbound lanes.

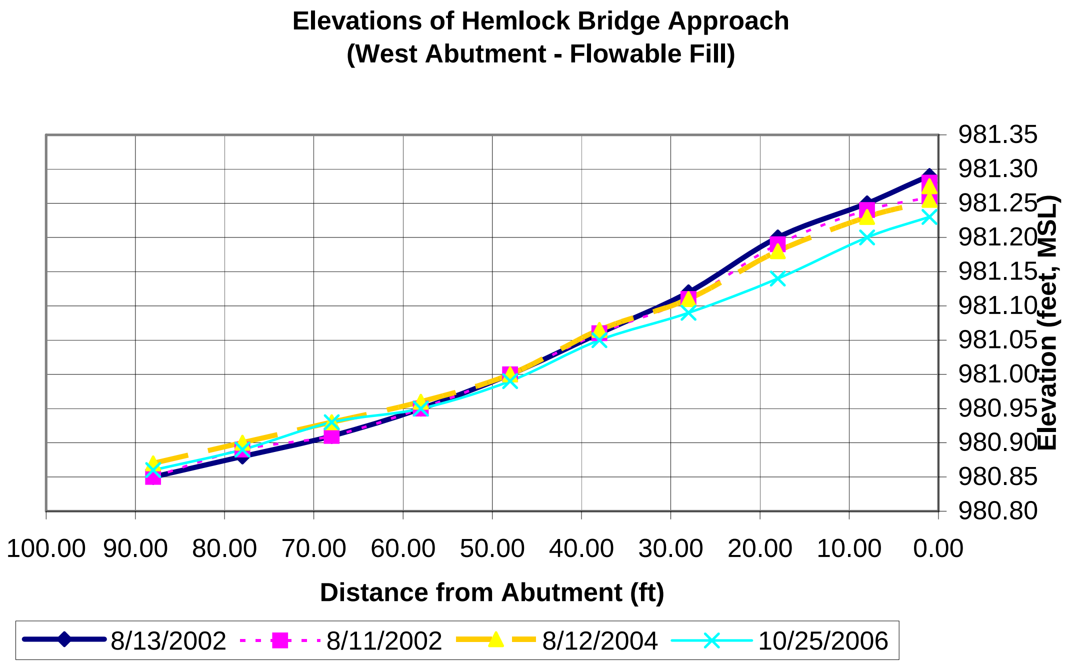


Figure 63: Elevations of Hemlock Bridge Approach at West Abutment (CLSM Fill)

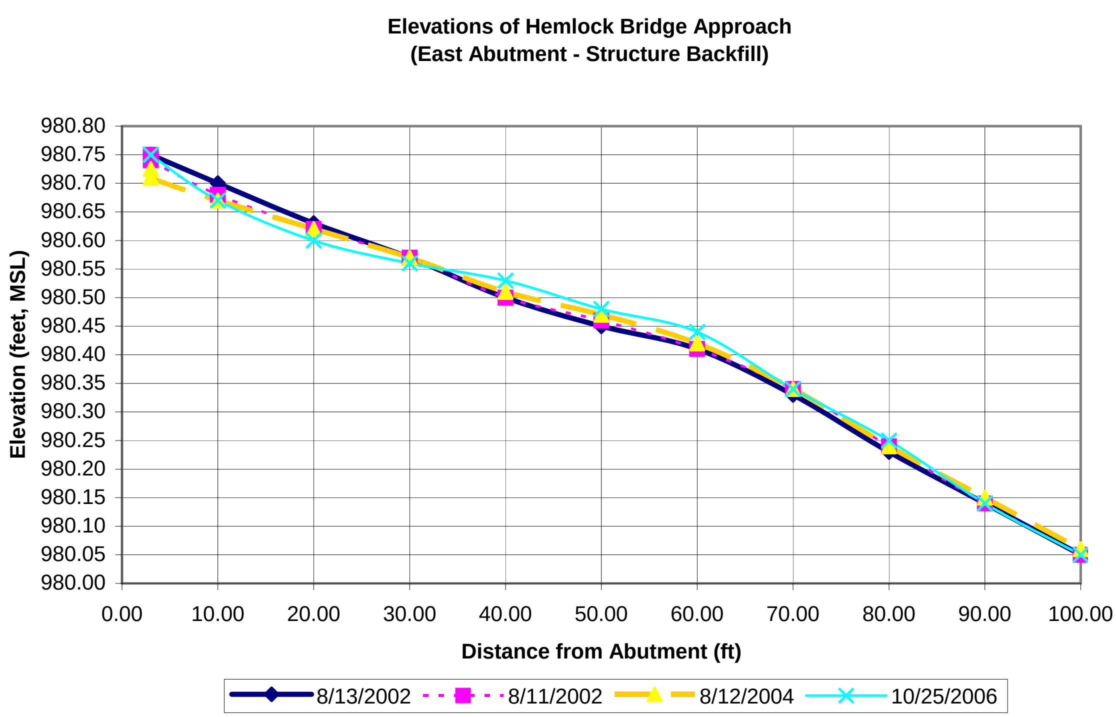


Figure 64: Elevations of Hemlock Bridge at East Abutment (Structure Fill)

Lateral Movement in Inclinometer Readings

Inclinometer casing IN1 was installed 5 feet behind the east abutment in structure backfill as indicated in Figure 61. Note that in all inclinometer installations the A-direction (longitudinal direction) is in the traffic direction. Also, the B-direction (transverse direction) is perpendicular to the A-direction and pointing away from the center line of the roadway. Based on readings over four years, as shown in Figure 65a, the maximum movements were about 0.25 inches (away from abutment) at 5 feet below grade (bg), and in the transverse direction approximately 0.16 inches (away from the center line of the roadway) at 7 feet bg. Overall, these movements are within a quarter of an inch over four years and are perceived as insignificant.

Inclinometer casing IN2 was installed 5 feet from the west abutment in flowable fill. The results, shown in Figure 65b, indicated a maximum of 0.22 inches (away from abutment) at 2 feet bg. Along the B-axis, the casing moved a maximum of 0.18 inches (away from the center line of the roadway) at 5 feet bg, and 0.25 inches (also away from the center line of the roadway) at 6 feet bg. Similar to the measurements in IN1, these movements are perceived as insignificant.

Vertical Settlement Measurements

The telescopic casing measurements indicated a vertical settlement less than 0.1 inches in the structure backfill and the flowable fill. The vertical settlement of the foundation soil under the two fills is less than a quarter of an inch.

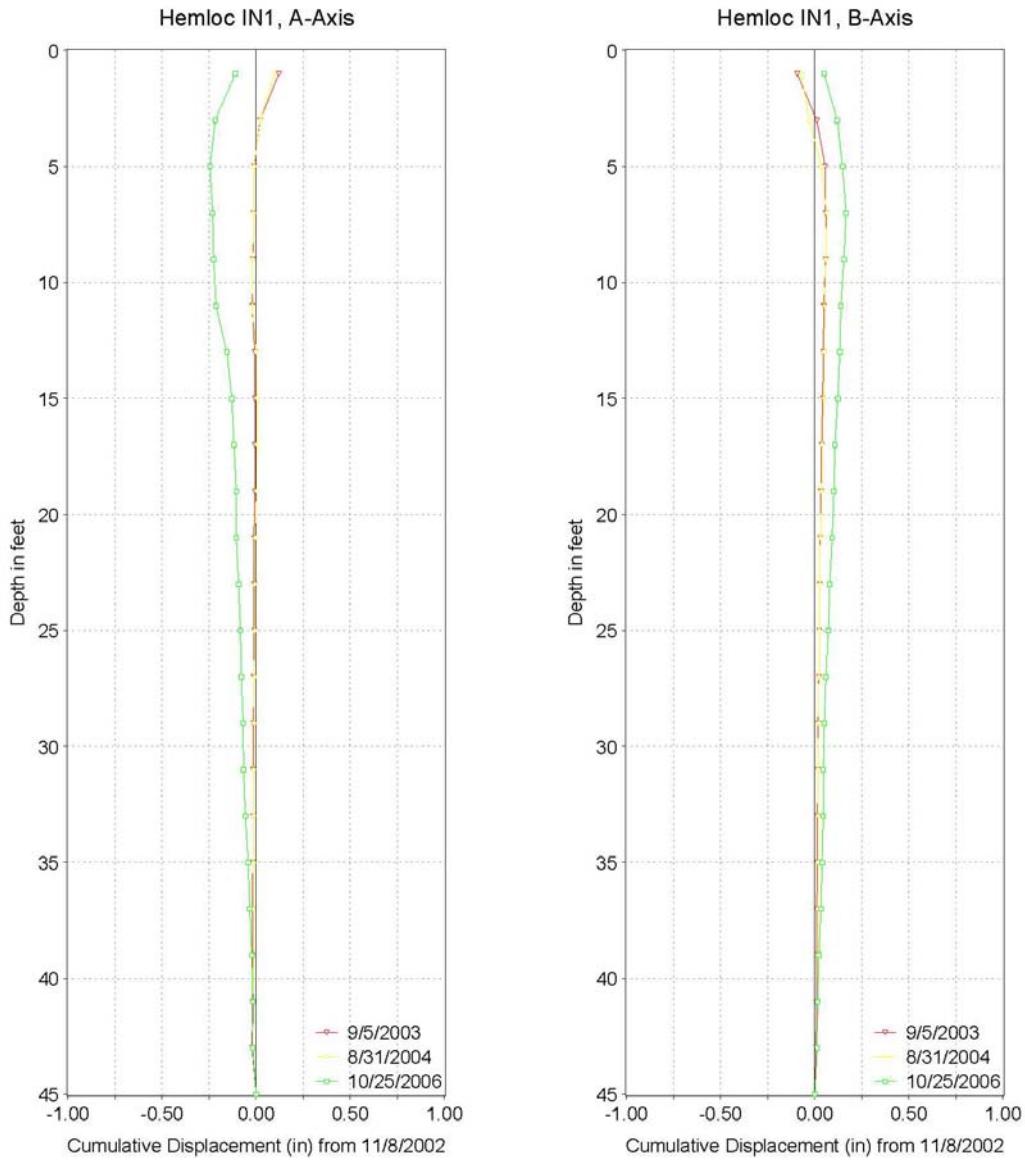
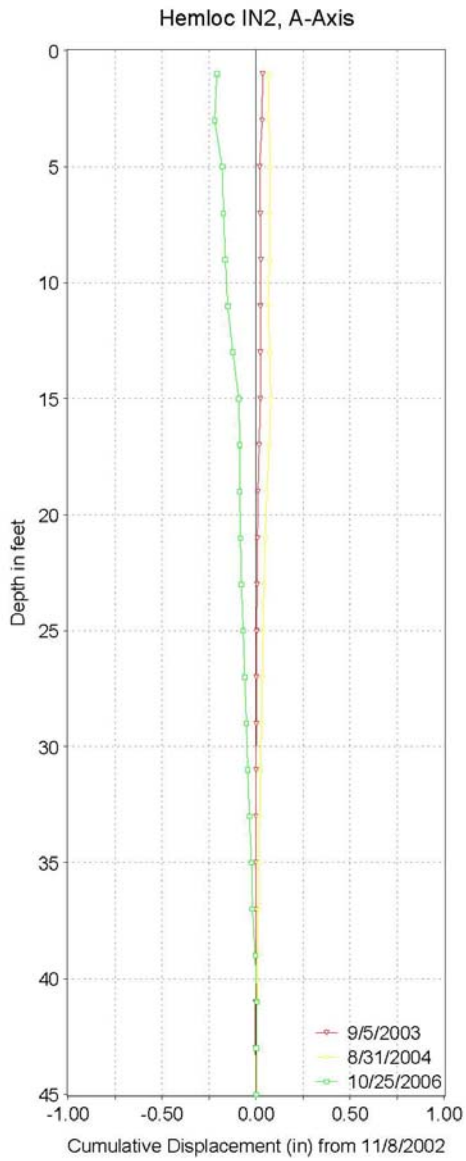


Figure 65a: Lateral Movement of IN1 at Hemlock Bridge



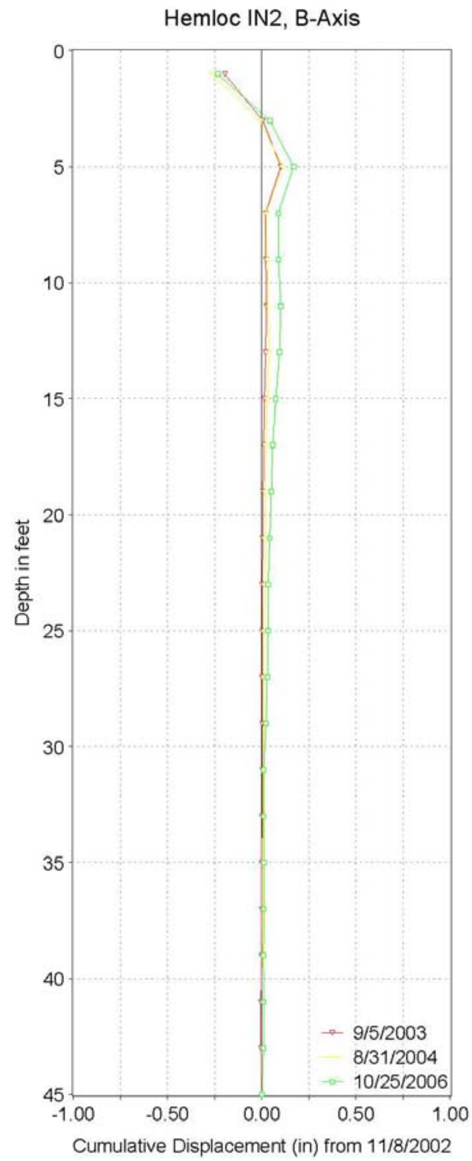


Figure 65b: Lateral Movement of IN2 at Hemlock Bridge

Summary

Up to the time of preparing this report there has been no maintenance work done on this bridge and its approaches since construction. Based on all the observations and data, it appears that very little differential movement has resulted at either abutment over the past four years. Settlement likely has not resulted because of the following reasons:

- Subsurface conditions indicated fill would be placed on loose sand over dense silt. Vertical movement of the loose sand likely occurred during construction.
- The amount of new fill placed was only 1 to 2 feet higher than the previous existing grade at the abutments and only 2 to 5 feet higher along embankment slopes.
- Previous reports of the existing bridge and approaches indicated they were in good condition.
- Inclinometers were installed 3 months after construction.

4.2 B-71-127: STH 173 over Cranberry Ditch

On August 31, 2004, a site visit was conducted at the same time as the 3rd inclinometer readings. The Cranberry Ditch bridge appeared to be in good condition with no observable cracking or movement. Photos of the bridge taken at the site visit were not available. Up to the time of preparing this report (June 2007) the bridge and its approach fills remain in very good condition.

Survey Elevations

Survey elevations were taken over a 4-year period from the end of bridge construction on both eastbound and westbound approaches. Visual inspection carried out in October 2006 indicated the presence of small differential settlement at the interface between the bridge and the approach fills. Survey elevations of the eastbound lanes approaching the west abutment and the westbound lanes approaching the east abutment are presented in Figures 66 and 67, respectively. The survey indicated differential settlement of about 0.5 inches near the east abutments (within 7 feet) and 0.4 inches near the west abutment. The differential settlement (see Appendix H) decreased the further away from the abutment until the differential movement was positive. The point where the differential movement changed from negative to positive is about 38 feet from the east abutment (structure fill) and 58 feet from the west abutment (geosynthetic reinforced fill). A transition slope was calculated to be of about 0.013 inches per foot for the east abutment and 0.006 inches per foot for the west abutment.

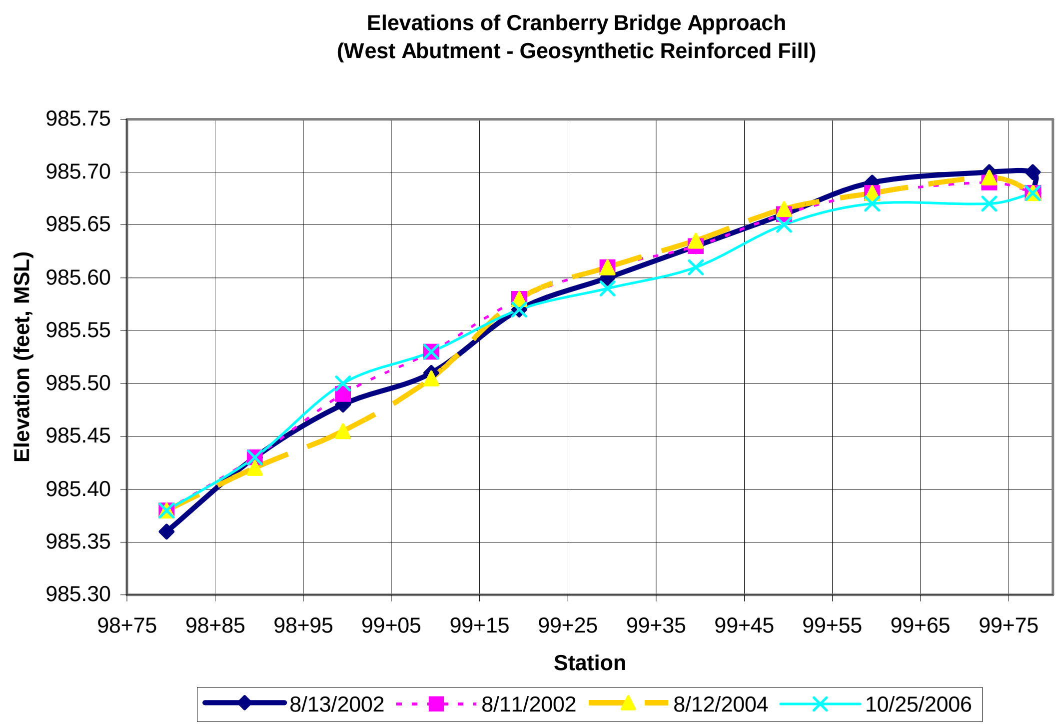


Figure 66: Survey Elevations of Cranberry Bridge at West Abutment (Geosynthetic Fill)

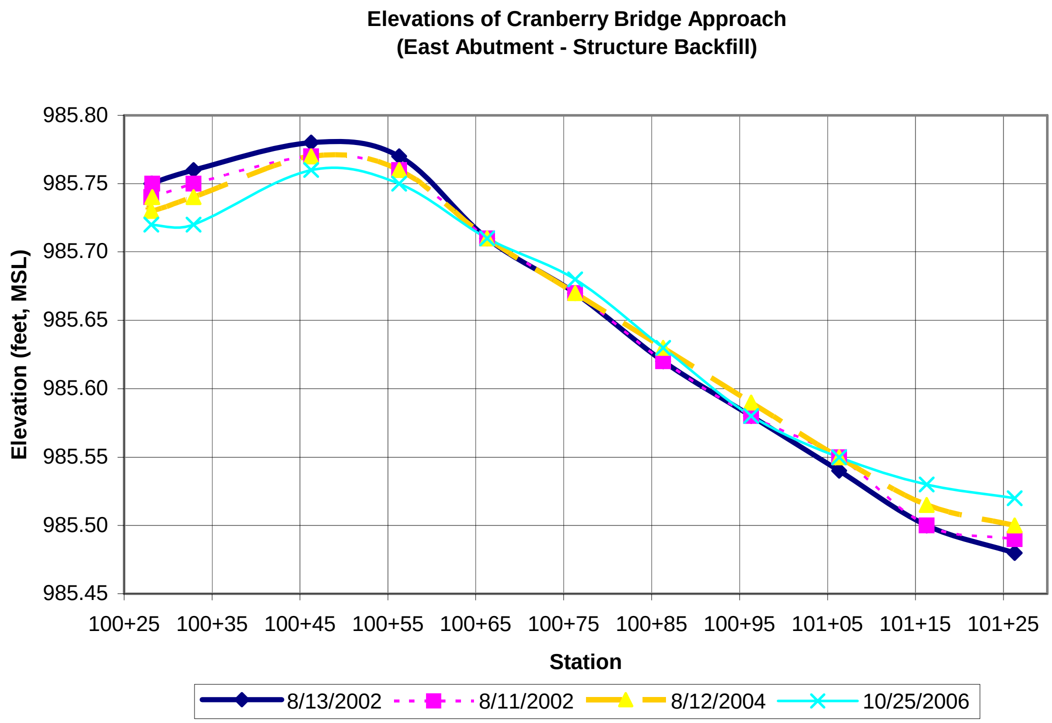


Figure 67: Survey Elevations of Cranberry Bridge at East Abutment (Structure Fill)

Lateral Movement in Inclinometer Readings

Inclinometer casing IN1 was installed 5 feet of the east abutment in structure backfill. Based on reading over four years, as shown in Figure 68a, the inclinometer casing moved a maximum of about 0.3 inches (away from abutment) at 2 feet bg and 0.15 inches (also away from the abutment) at 12 feet bg. In the transverse direction, the maximum lateral displacement is 0.3 inches (away from the center line of the roadway) at 2 feet bg. Overall, these movements are within a few tenths of an inch over four years and are perceived as insignificant.

Inclinometer casing IN2 was installed 5 feet from the west abutment in the geosynthetic reinforced fill. The results, shown in Figure 68b, indicated a maximum of 0.16 inches (away from abutment) at 5 feet bg along the A-axis and only 0.15 inches (away from the center line of the roadway) at 2 feet bg along the B-axis. Similar to the measurements in IN1, these movements are perceived as insignificant.

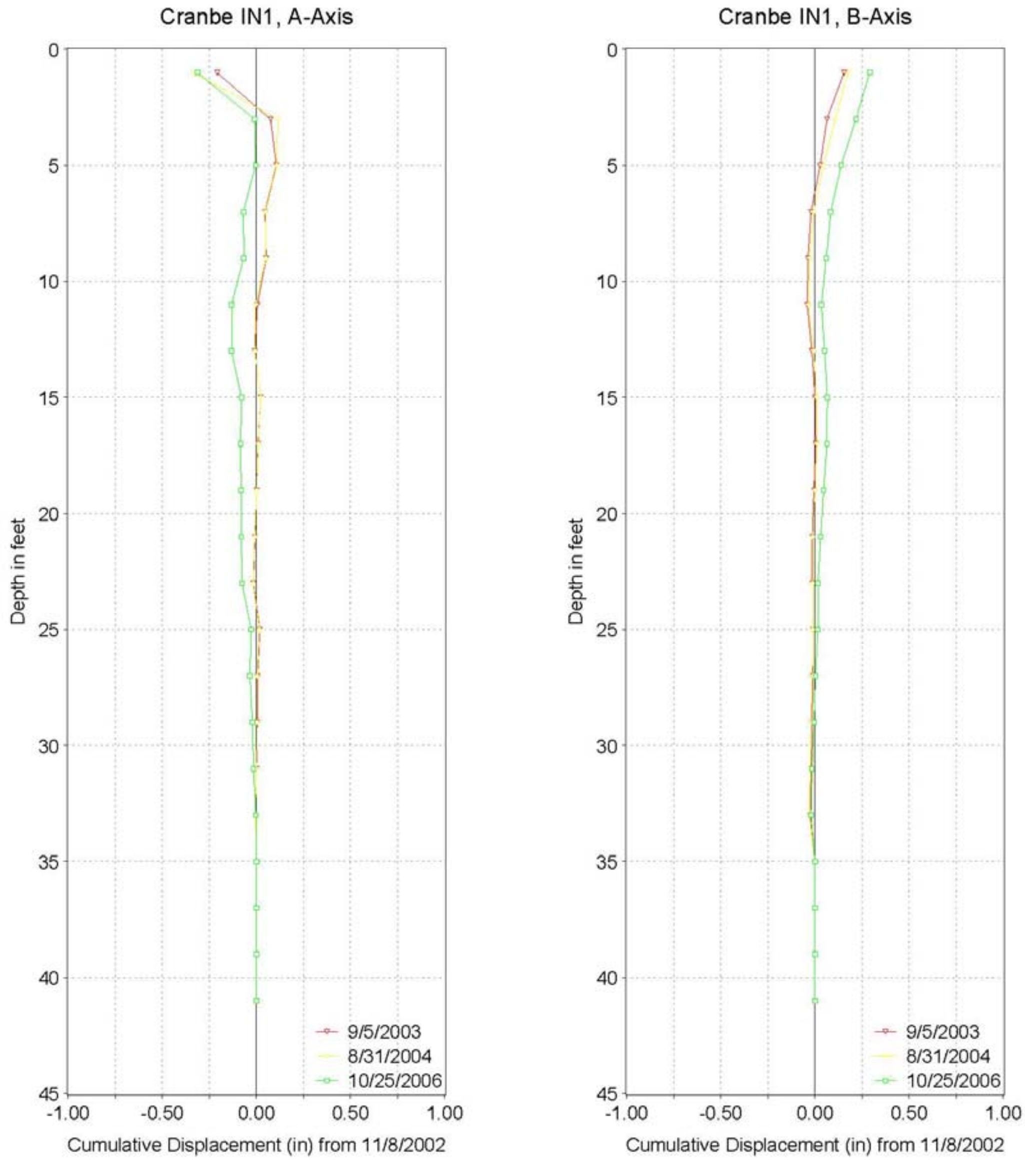


Figure 68a: Lateral Movement of IN1 at Cranberry Bridge

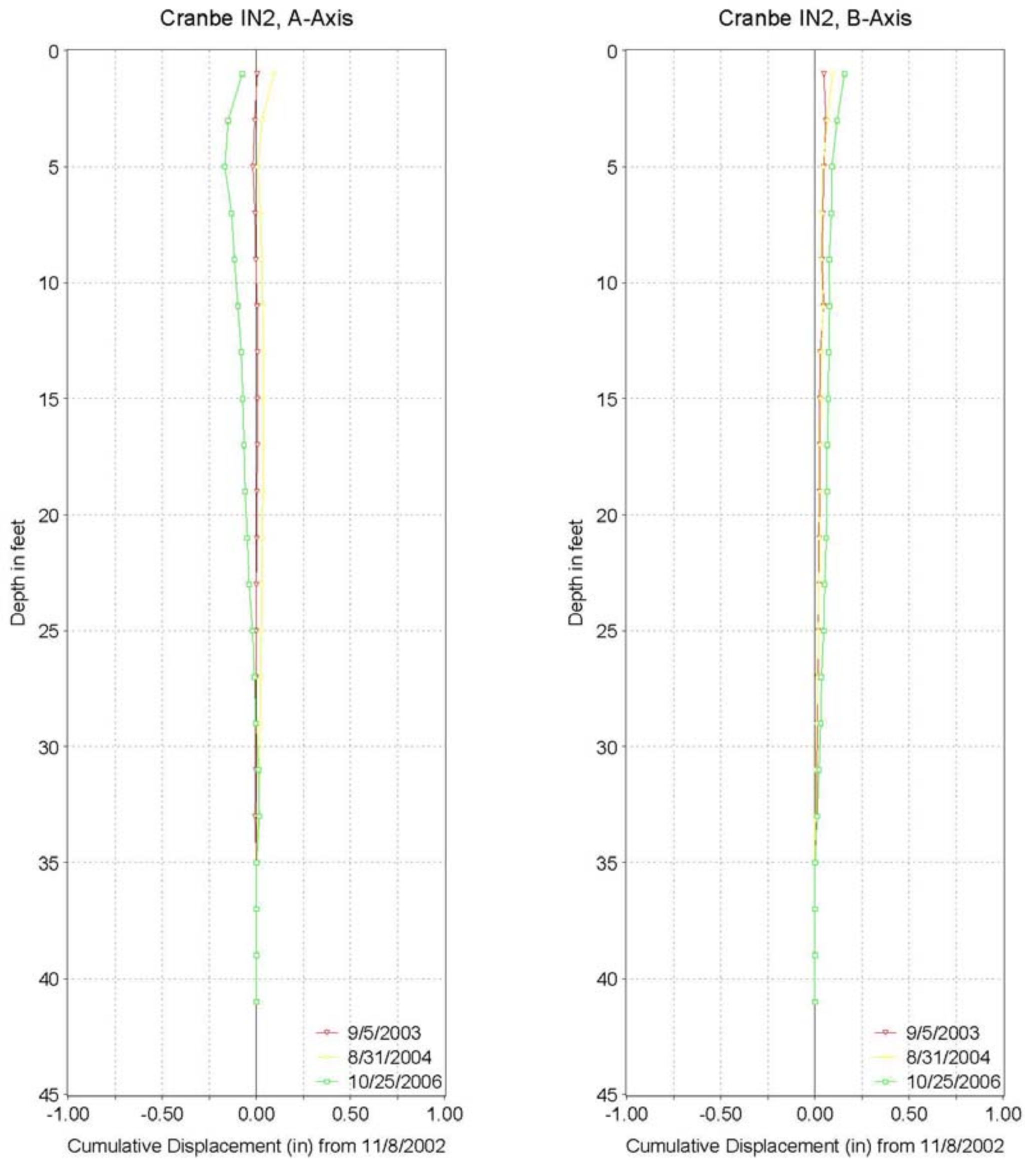


Figure 68b: Lateral Movement of IN2 at Cranberry Bridge

Vertical Settlement Measurements

The telescopic casing measurements did not indicate any significant vertical movement in the fills or foundation soils (±1/8 inch).

Summary

Up to the time of preparing this report there has been no maintenance work done on this bridge and its approaches since construction. Based on all the observations and data, it appears that very little movement has resulted at either abutment since construction (four years). No significant difference in performance was noted between the structure backfill and the geosynthetic reinforced fill. Settlement likely has not resulted because of the following reasons:

- Subsurface conditions indicated fill would be placed on very loose to loose sand. Vertical movement of the loose sand likely occurred during construction.
- The amount of new fill placed was only ½ to 1 foot higher than the previous grade at the abutments and up to 7 feet higher along a portion of the embankment slopes.
- Previous reports of the previous bridge and approaches indicated they were in good condition.
- Inclinometers were installed 3 months after construction.

4.3 B-66-135: Western Avenue over Cedar Creek

The last site visit to the Western Avenue bridge was carried out in September 2006. The bridge and its two approach fills appeared to be in good condition. Prior to that, on April 13, 2005, a site visit was conducted about 6 months after the construction of the bridge (Figure 69). It was observed that some of the asphalt at the interface between the roadway and back of the west bridge deck was cracking (structure fill). The approach pavement on the westbound lanes appeared to be performing better than the eastbound lanes as shown in Figure 70 and 71. Embankment side slopes were sparsely vegetated, as grass is not grown yet. The north

Figure 69: Looking West at Western Bridge

Figure 70: Approach at East Abutment of Western Bridge (Geosynthetic Fill)

Figure 71: Approach at West Abutment of Western Bridge (Structure Fill)

Figure 72: Slope instability along south side of Western Bridge

Figure 73: Erosion of fill at Northwest corner of Western Bridge

embankment appeared to be performing better than the south, as some sloughing at the top of the slope was observed as shown in Figure 72. Erosion of granular material behind the concrete deck and abutment were also noted as shown in Figure 73. Overall the bridge appeared to be in good condition.

Survey Elevations

Survey elevations were taken over a 2-year period from the end of bridge construction on both eastbound and westbound approaches. Figures 74 and 75 present the survey elevations at the west and east abutments, respectively. The survey indicated differential settlement (see Appendix I) of about 0.23 inches within 5 feet of the west abutment (structure fill), and only indicated positive movement at the east abutment (geosynthetic reinforced fill). The positive movement is likely due to survey, as the 2nd reading showed 0.01 inches of vertical settlement, which is considered negligible. Along the eastbound lanes, the differential settlement decreased the further away from the abutment until the elevations showed an upward or positive movement in the pavement. The point where the differential movement changed from negative to positive is about 15 feet from the West abutment. A transition slope was calculated to be about 0.015 inches per foot.

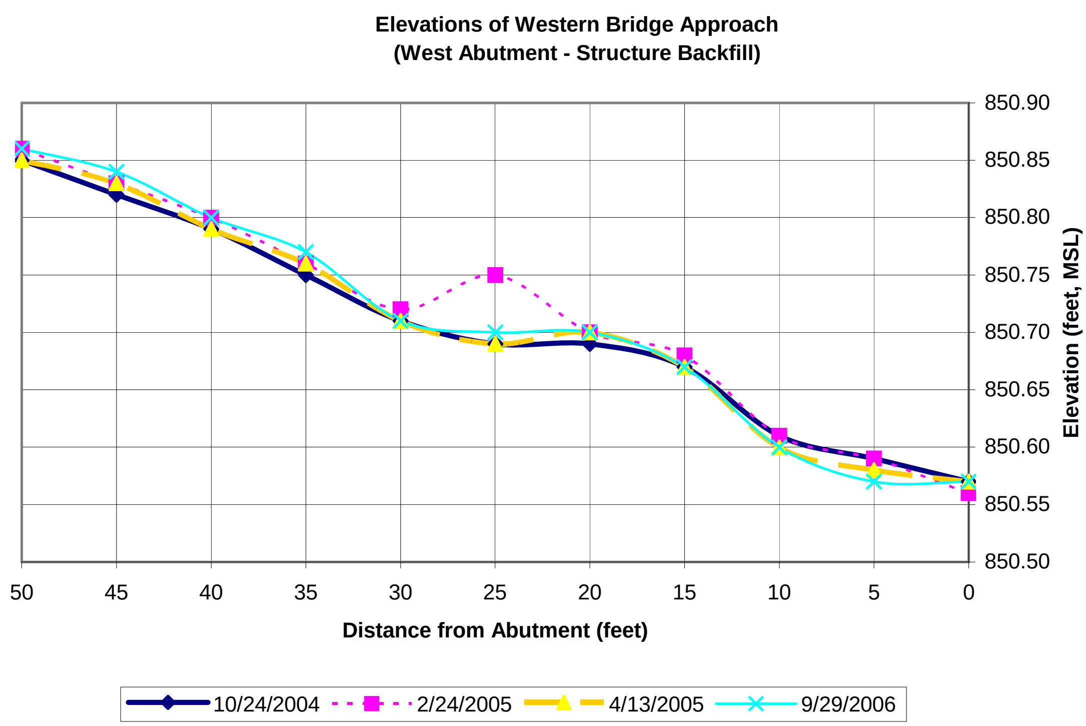


Figure 74: Survey Elevations of Western Bridge at West Abutment (Structure Fill)

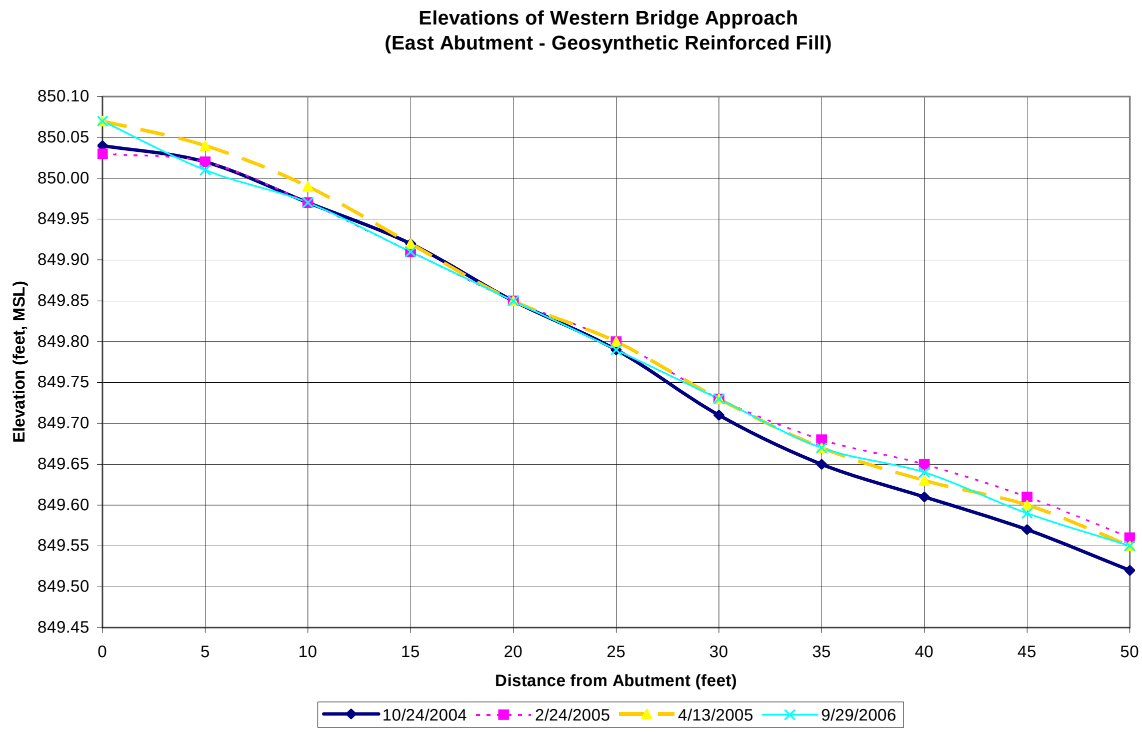


Figure 75: Survey Elevations of Western Bridge at East Abutment (Geosynthetic Fill)

Lateral Movement in Inclinometer Readings

Inclinometer casing IN1 was installed 5 feet behind the east abutment in geosynthetic reinforced fill. Based on readings over the past two years, as shown in Figure 76a, the inclinometer casing moved a maximum of about 0.25 inches (away from abutment) from 0 to 10 feet bg. In the transverse direction, the maximum lateral displacement is approximately 0.15 inches (away from the center line of the roadway) at 2 feet bg.

Inclinometer IN2 was installed 5 feet from the west abutment in structure fill. Readings were taken over 2 years. The results, shown in Figure 76b, indicated a maximum movement of 0.12 inches (away from abutment) at 4 feet bg, and in the transverse direction about 0.1 inches (away from the center line of the roadway) at 5 feet bg. This movement is considered negligible. The net movement appears to be towards the southwest or outside corner of the abutment where erosion is occurring.

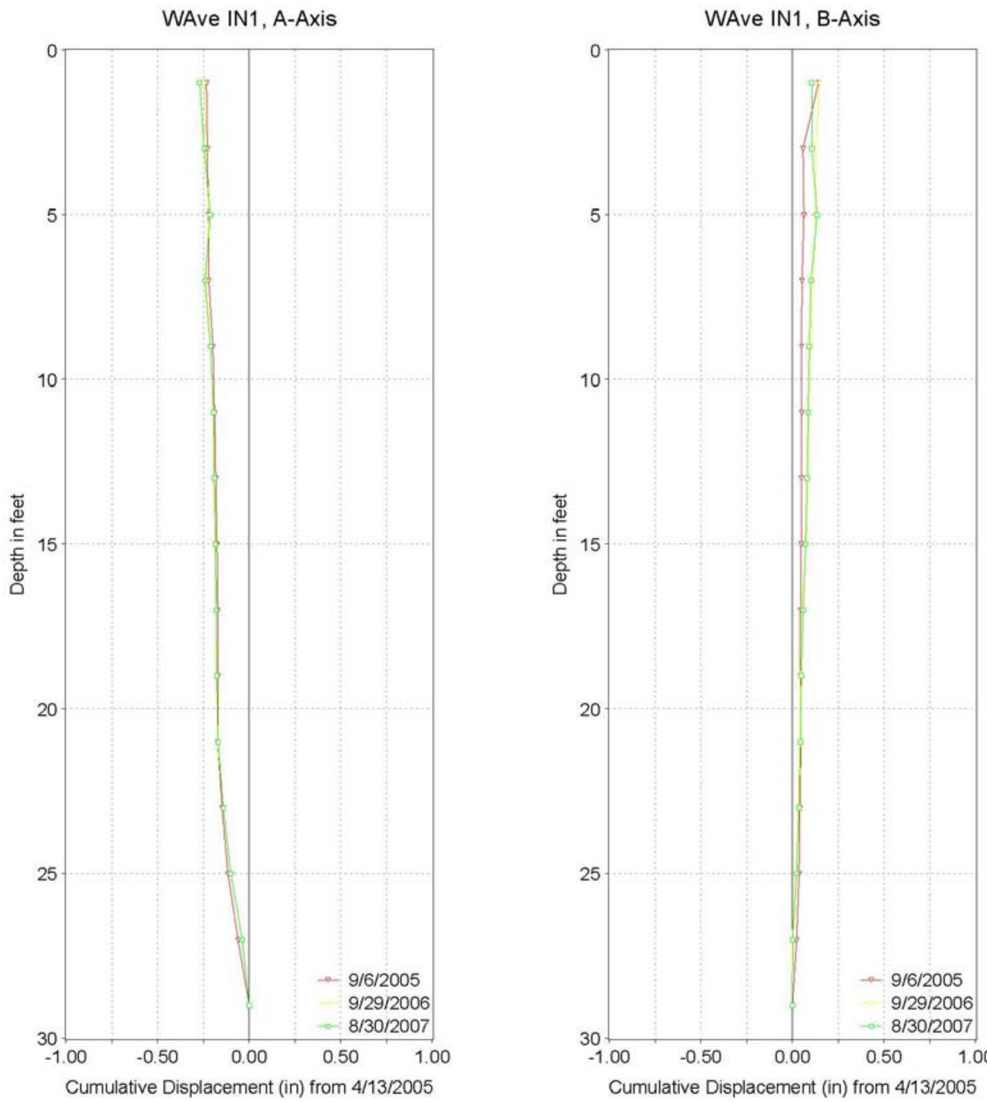


Figure 76a: Lateral Movement of IN1 at Western Bridge

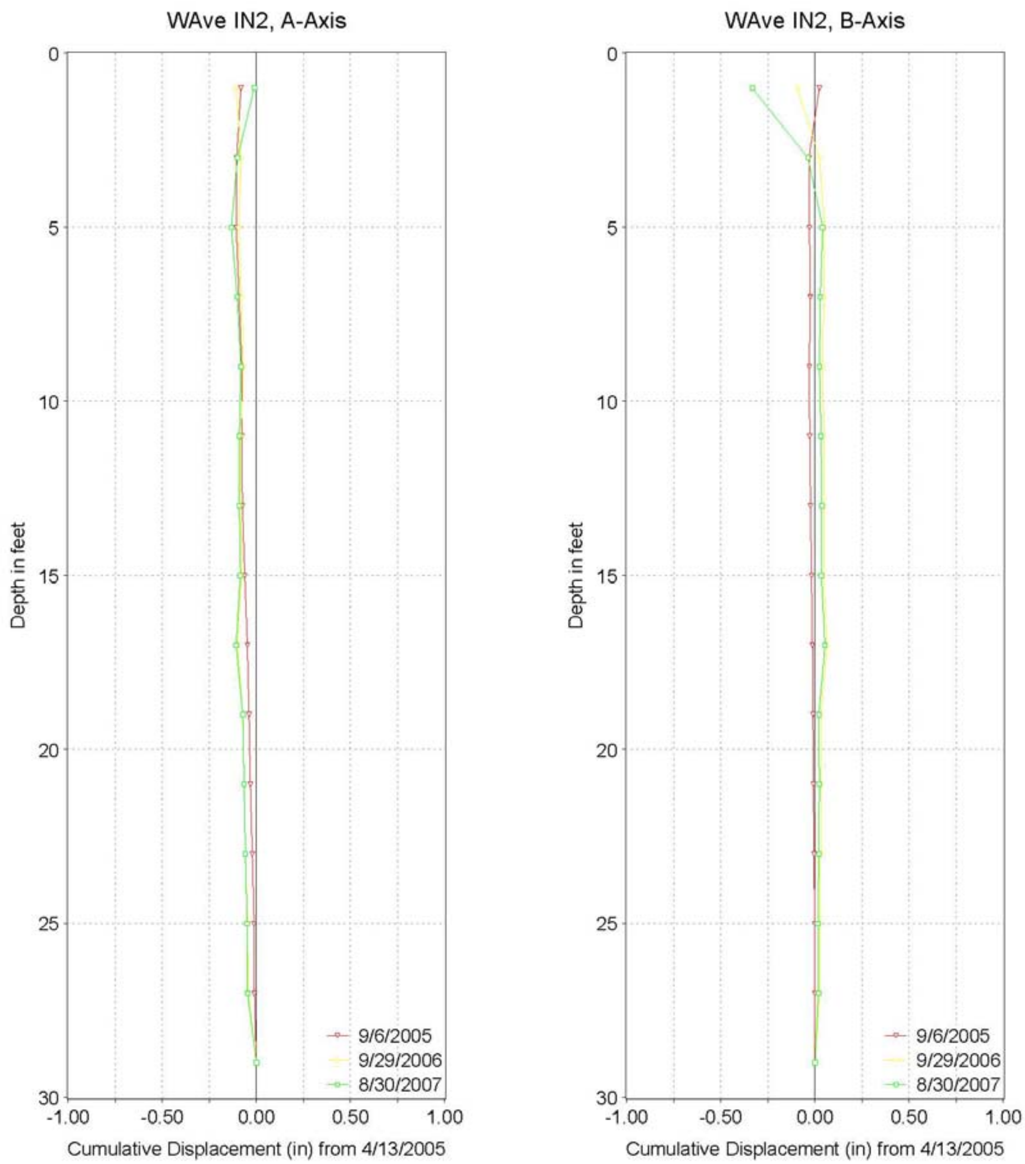


Figure 76b: Lateral Movement of IN2 at Western Bridge

Vertical Settlement Measurements

Readings indicated negligible movement in all telescopic sections for both the structure backfill and the geosynthetic reinforced backfill abutments.

Summary

Based on all the observations and data, it appears that very little movement has resulted at the east abutment backfilled with geosynthetic reinforced fill. Some lateral movement has occurred in the structure backfill but recent readings do not indicate any more significant movement. Also, it can be observed that the general direction of the displacement of the inclinometer casings is toward the sides of the bridge where erosion was noted.

4.4 B-40-700: West Beloit Road over Root River

The last site visit to the west Beloit road bridge was carried out in September 2006. The bridge and its two approach fills appeared to be in good condition. On April 13, 2005, a site visit was conducted after the 3rd field measurements were taken for the bridge. An overview photograph is shown in Figure 77. The eastbound approach appeared to be in good condition with a few asphalt pieces missing near the interface with the west abutment. This is likely due to loosely placed asphalt and difficulty of compaction at the roadway-concrete deck interface. The eastbound approach (flowable fill) shown in Figure 78 appeared to be in better condition than the westbound approach (structure fill) shown in Figure 79. A noticeable dip in the asphalt was observed (westbound). It was also noted that the embankment side slopes beyond the wingwalls were settling, as noted in Figure 80, which was more prevalent on the south side than on the north. This could be attributed to the staged construction. The north side was allowed to settle before final grading and seeding. Vegetation had not yet taken, as construction was just completed in November 2004. Erosion was also observed from underneath the concrete sidewalks on the south side of the bridge. A void up to 3 inches was noted and more prevalent on the southwest corner.

A close up photograph of the void on the southeast corner is shown in Figure 81.

Figure 77: Looking East at Beloit Bridge

Figures 78: Approach at West Abutment of Beloit Bridge (CLSM Fill)

Figure 79: Approach at East Abutment of Beloit Bridge (Structure Fill)

Figure 80: Slope Instability Southwest of Beloit Bridge

Figure 81: Void on Southeast corner of Beloit Bridge

Survey Elevations

Survey elevations were taken over a 2-year period from the end of bridge construction on both eastbound and westbound approaches. Elevations for both approach lanes at the West and East Abutments are presented in Figures 82 and 83, respectively. Along both approaches, the survey measurements at 3 months, 6 months, and 2 years were higher (0.1 to 0.6 inch) than the measurements of the initial survey and could be attributed to survey error. The differential movement between the 3-month and 2-year readings showed significant movement (0.3 to 0.6 Inches) along the eastbound approach (flowable fill). Along the westbound approach (structure backfill), the survey between the 3-month and 2-year readings indicated a 0.5 inches of settlement at about 2 feet behind the abutment, which was also observed during the site visit.

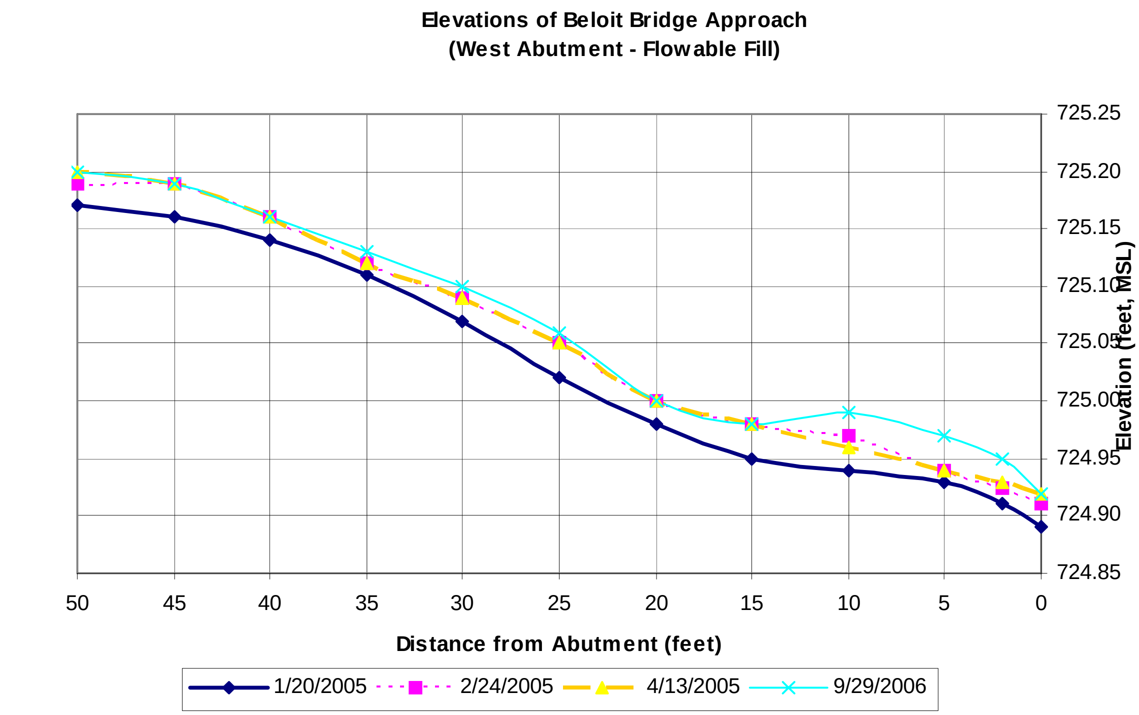


Figure 82: Survey Elevations of Beloit Bridge at West Abutment (CLSM Fill)

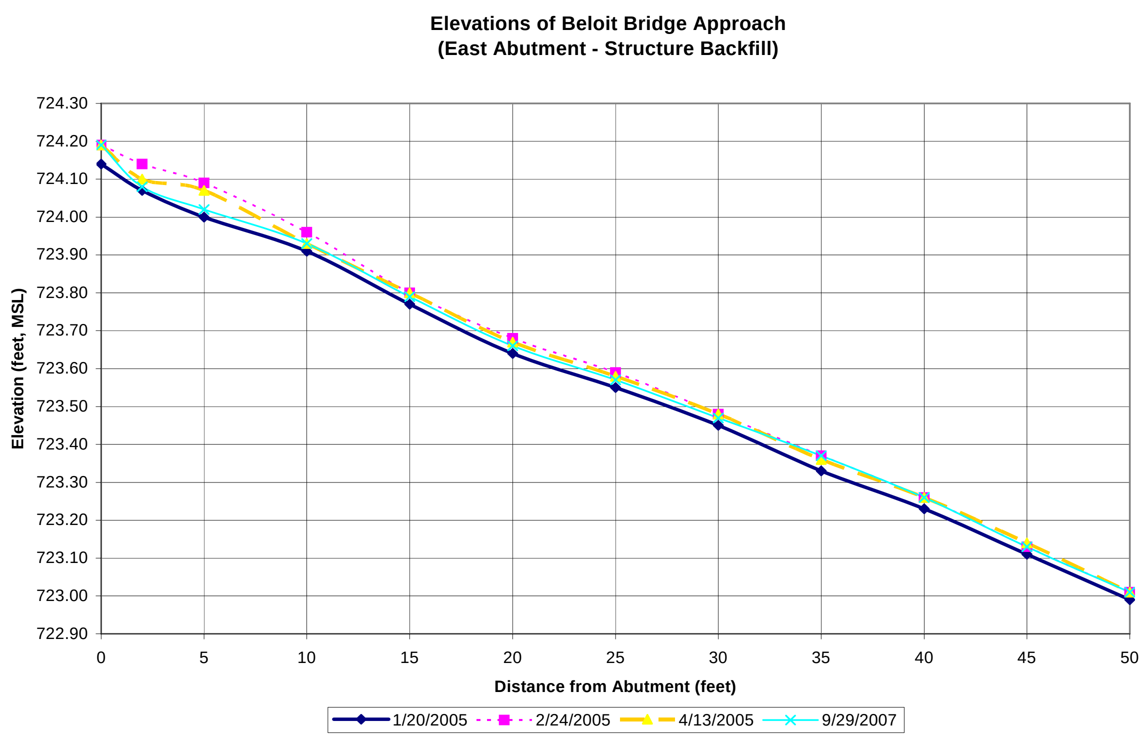


Figure 83: Survey Elevations of Beloit Bridge at East Abutment (Structure Fill)

Lateral Movement in Inclinometer Readings

Inclinometer casing IN1 was installed 5 feet of the east abutment in structure backfill. Based on readings over the past two years, the inclinometer casing moved a maximum of about 0.1 inches (away from abutment) at 4 feet bg, as shown in Figure 84a. In the transverse direction, shown in the same figure, the maximum lateral displacement is approximately 0.1 inches (away fro center line of the roadway) at 5 feet bg. The net movement appears to be towards the northeast or outside corner of the wingwall where erosion is occurring. Inclinometer casing IN2 was installed 5 feet from the west abutment in flowable fill. Readings were taken over 2 years, as shown in Figure 84b, and the results indicated negligible movement in both directions.

Vertical Settlement Measurements

Insignificant settlement (± 1/8 inch) was recorded at both abutments.

Summary

Based on all the observations and data, it appears that very little movement has resulted at the west abutment backfilled with flowable fill. Vertical movement of approximately 0.5 inches has been observed at the approach at the east abutment backfill with the structure backfill. This could be attributed to the greater amount of fines in the backfill. Also, the general direction of lateral movement of the casing has been toward areas where erosion has been noted.

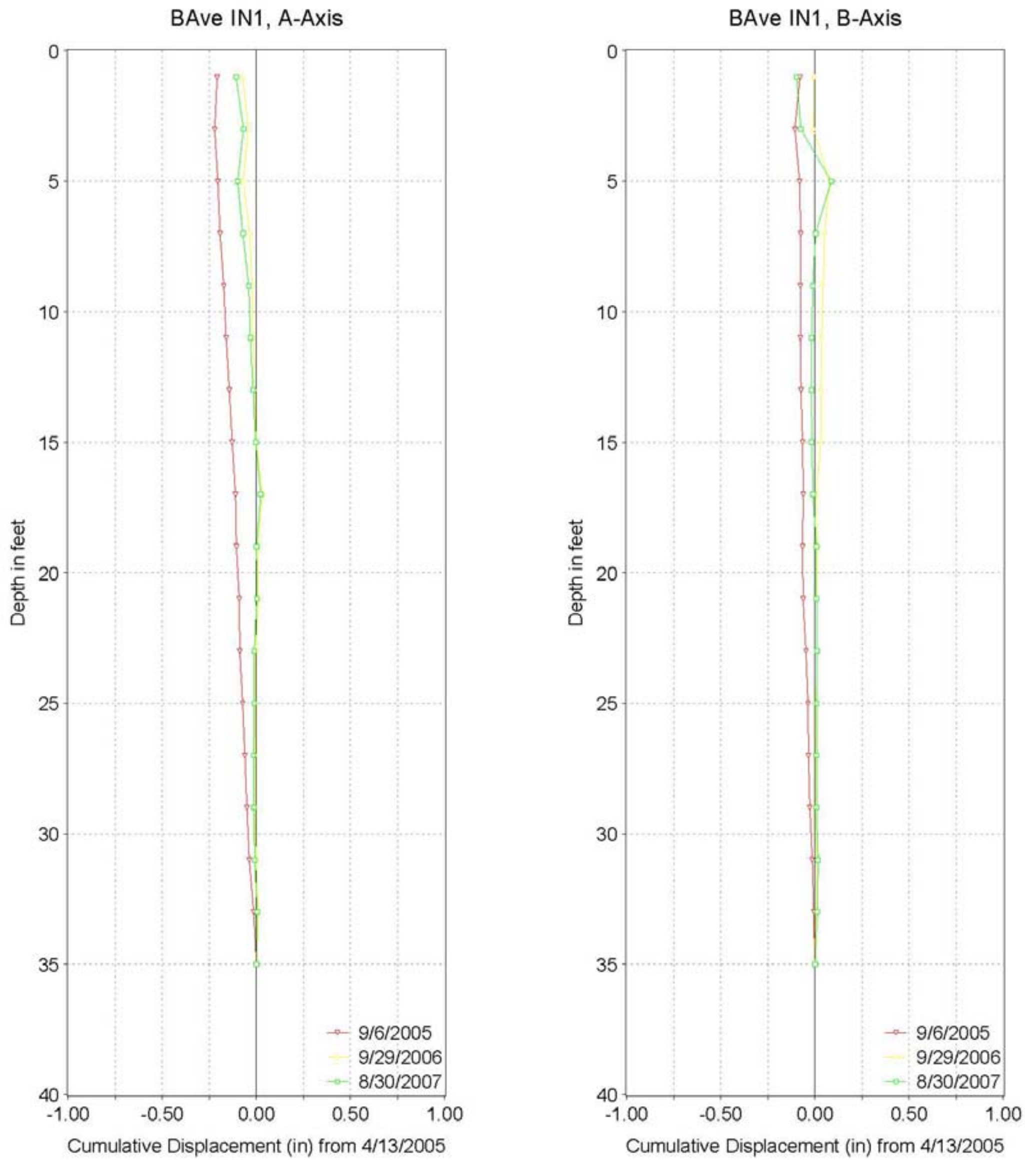


Figure 84a: Lateral Movement of IN1 at Beloit Bridge

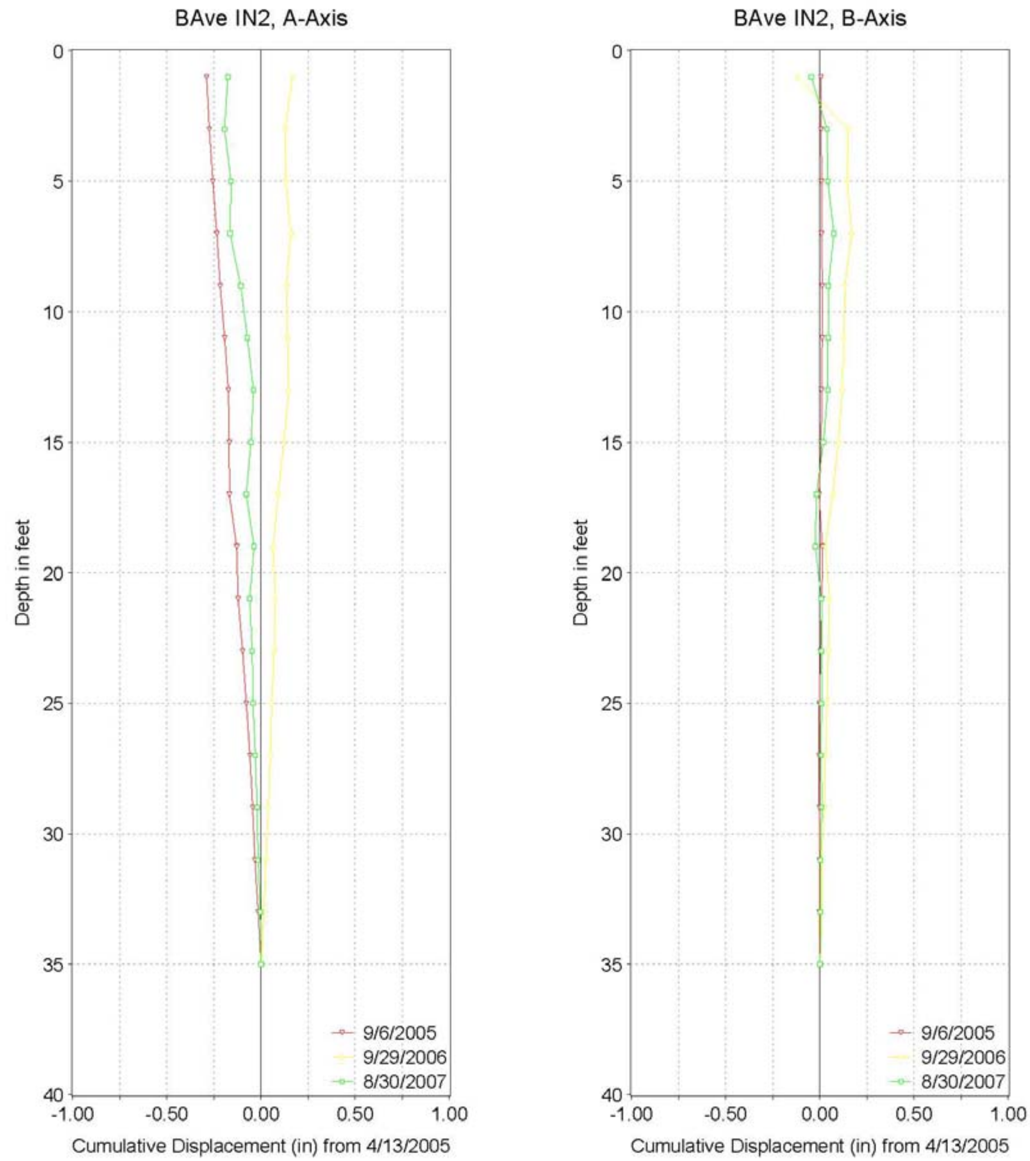


Figure 84b: Lateral Movement of IN2 at Beloit Bridge

5.0 CONCLUSIONS

Bridge approach settlement is a complex interaction between the bridge, backfill soils, foundation soils, and drainage. The literature has indicated that deformation of the backfill materials, deformation of the foundation soils, and poor drainage are the greatest contributors to bridge approach settlement. Many mitigation techniques have been used to control the settlement, but the methods selected depend on the specific site. Specifying more stringent backfill materials and compaction requirements as well as providing proper drainage are effective ways in helping to alleviate the problem. Techniques to repair the bump include asphalt patching or overlays, slab jacking, and replacement of an approach slab.

Of the many mitigation methods, flowable fill and geosynthetic reinforced fill behind bridge abutments were selected to be studied as part of the WisDOT Research Study, "Evaluation of Bridge Approach Settlement Mitigation." This report addresses four bridges (Hemlock, Cranberry, Western, and Beloit).

Based on the literature research, site visits and field test measurements of the four bridges, the following comparisons and conclusions can be made:

- The movements of the approach fills that have granular foundation soils (Hemlock and Cranberry) and less than 5 to 7 feet of fill were insignificant over five years compared with the movements of the approach fills (Western and Beloit) with cohesive foundation soils over two years.
- Embankment side slopes that settle and slough (Western and Beloit) resulted in erosion and/or movement of backfill material.

- The flowable fill and geosynthetic reinforced fill on granular soil foundations did not outperform the structure backfill (Hemlock and Cranberry).
- The flowable fill and geosynthetic reinforced fill on cohesive soil foundations did outperform the structure backfill (Beloit and Western).
- The cost of flowable fill is greater than geosynthetic reinforced fill for small quantity jobs.
- On the Western Bridge, the west abutment backfilled with structure backfill showed the greatest lateral movement, which could be a combination of foundation soils, erosion, and bridge type.
- On the Beloit Bridge, the east abutment backfilled with structure backfill showed the greatest vertical movement, which could be attributed to the greater percentage of fines.

6.0 RECOMMENDATIONS FOR FUTURE STUDY

For future test sites to be monitored as part of this WisDOT Research Project, the following recommendations should be considered as part of the field test selection and instrumentation plan:

- The contractor is more careful in terms of approach fill compaction control when alerted to the fact that the bridge will be the subject of a special study such as the one carried out in this research. The comparison with the resulting well-compacted structure backfill becomes unrepresentative since such care in quality control in usually not the case. Nonetheless, the proposed mitigation methods showed promising behavior when compared to well-compacted structure approach fills.
- Optimum field test sites are sites where softer cohesive foundation soils are present and where the new bridge is not a replacement or where significant fill (greater than 7 feet of fill) will be placed.
- At least four field readings are recommended to be taken within 6 months of bridge completion.
- Inclinometers need to be installed as soon after construction as possible to obtain initial surveys.
- Multiple interviews with the construction engineer are recommended as well as frequent visits to observe the abutment and backfill construction.
- Additional inclinometers should be considered to be installed within the embankment slopes or at the toes of embankment slopes adjacent to the bridge.
- Settlement plates within the fill and in the foundation soil should be considered in addition to the telescopic sections for redundancy.
- Laboratory and field tests need to be carried out to investigate the effectiveness of using hydraulic fills as a method for alleviating bridge approach settlements.

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