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SD Department of Transportation
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Use of Fabric Reinforced Soil Wall for Integral Abutment Bridge End Treatment

Study SD96-02
Final Report

Prepared by
South Dakota State University
Department of Civil and Environmental Engineering
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DISCLAIMER

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16. Abstract

The bump at the end of the bridge has long plagued motorists, designers, and maintenance personnel. The bump develops from the differential movements of the approach road and the bridge abutment. This movement produces discomfort to motorists, increased damage to vehicles, traffic hazards, and increased maintenance costs. This damage done by the movement of these systems can literally impair the structural integrity of the bridge in extreme cases. The problem is experienced in all states and is a recurring problem for transportation officials in the state of South Dakota. Previous research has shown that the bump was caused by the development of voids under the bridge approach slabs in integral abutment bridges. Void development was shown to be a direct consequence of the thermal-induced movement of the bridge beam/abutment system. To accommodate this mechanism, the State of South Dakota implemented a new design using a geotextile reinforced soil wall in the abutment backfill to create a gap between the backfill soil and bridge abutment. The objectives of this research was to determine if constructing an approach fill to an integral abutment structure with a void between the abutment wall and the fill would alleviate development of voids under the approach slabs to the end of the bridge. Additionally, the research was to investigate alternative design methodologies.

The work included the monitoring of three bridges constructed with a gap installed between the abutment and the backfill to document the performance of the newly implemented design. A review of designs used by other states was conducted and alternative backfill materials were studied for use in the model test. Additionally, the feasibility of using an alternative backfill was investigated using the SDDOT model integral abutment system at the Brookings DOT Maintenance Yard.

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CHAPTER 1

EXECUTIVE SUMMARY

1.1 Introduction

The bump at the end of the bridge has long plagued motorists, designers, and maintenance personnel. The bump develops from the differential movements of the approach road and the bridge abutment. This movement produces discomfort to motorists, increased damage to vehicles, traffic hazards, and increased maintenance costs. The damage done by the movement of these systems can literally impair the structural integrity of the bridge in extreme cases.

In integral abutment systems, abutment wall movement due to temperature cycles has been identified as a leading cause of movement in backfill materials. Based on field and model studies of the occurrence of voids under approach slabs, the South Dakota Department of Transportation has implemented the use of a geotextile reinforced soil wall for integral abutment bridge end treatment to provide a gap into which the abutment wall can cycle without disturbing the backfill. This investigation was prompted in order to monitor the effectiveness of this design through field studies and to develop other alternative backfill designs through the use of model studies.

1.2 Purpose

The overall purpose of this research was to determine if constructing an approach fill to an integral abutment structure with a void between the abutment wall and the fill will alleviate problems with the development of a void under approach slabs to the end of the bridge. Additionally, an alternative system for alleviating the problem was to be developed and tested. To meet this goals, the following sub-objectives were identified:

1. To design, construct, monitor, and evaluate an actual bridge end backfill reinforced wall similar to that used by the Wyoming DOT (WYDOT). This

will be accomplished by constructing full-scale fabric reinforced soil walls as the approach backfill on actual bridge approaches of new and/or retrofitted bridges.

2. To determine the feasibility of using other materials as a bridge end backfill material. This shall be investigated primarily through the conduct of model scale tests of alternative backfill materials using the SDDOT model integral abutment system at the Brookings DOT Maintenance Yard. Alternative means of providing a void behind the abutment wall, through the use of other materials were investigated using this model.

1.3 Approach

In the spring of 1996 a study of the problem was undertaken by the Civil and Environmental Engineering Department at South Dakota State University (SDSU). To meet the research objectives and tasks, a field study and a model study were planned.

The general purpose of the field study was to monitor three bridges constructed using a design based on WYDOT's design. Data was collected on these bridges to determine the effectiveness of this new design for reducing void development under bridge approaches. Data was then taken to be analyzed to determine if the design or construction of the approach embankments should be altered.

A model study was to be performed to investigate alternative backfill designs to reduce void development under approach slabs. A review of the literature was to be performed to identify candidate materials in an effort to determine the engineering materials available for such an application. The model study was then to be designed and constructed using the selected material. Testing of the model was to be completed after developing an instrumentation plan. Results were then to be compared with a similar model study performed by Schaefer and Koch in 1992 using a past SDDOT backfill design.

1.4 Significant Findings

The following are the most significant findings from the model study of this investigation:

1. Development of voids under the approach slab is reduced by using a vertical layer of rubber tire chips behind the integral abutment.
2. Earth pressure data shows that a composite design incorporating the use of a vertical layer of rubber tire chips behind the integral abutment greatly relieves the passive earth pressures induced on the retained fill, which reduces the mechanisms causing void development.
3. Inclinometer data from the model tests show embankment soils are not significantly affected by abutment movement.
4. Rearrangement of the rubber tire chips as the abutment cycles causes permanent consolidation of the rubber tire chip layer. This consolidation causes movement of the adjacent soil, resulting in void development under the approach slab that is similar to the case when no rubber tire chip layer is used, but to a smaller amount.

The following are the most significant findings from the field study of the geotextile reinforced wall:

1. The use of a geotextile-reinforced wall behind the integral abutment reduces the development of voids under the approach slab sections.
2. The six-inch gap width between the abutment and geotextile wall is adequate to prevent the abutment from inducing passive pressures on the reinforced soil.
3. Removal of the gap separator after construction results in an initial movement of the geotextile wall from 0.5 to 2 inches. A high degree of construction quality control with regards to the tension maintained in the geotextile during construction helps minimize that initial movement.

4. Measurement of the gap width shows a definite trend in movement with respect to temperature. The gap closes during the warmest temperatures due to expansion of the bridge system. Gap width measurements show contraction during the cold temperature periods.
5. Compilation of data shows the cycle of gap width measurements to be consistent with changes in temperature showing that the geotextile wall has ceased moving in any measurable amount.
6. Although the construction of the gap between the abutment and backfill has prevented passive pressures from developing under the driving lanes, no constructed gap exists behind the wing wall sections. These soils continue to be affected by the cyclic movement of the wing walls leading to the formation of a gap in these areas. This has caused soil to erode from this section into the gap behind the abutment. Significant amounts of erosion into the gap will eventually create conditions that will lead to development of voids under the approach slab.

1.5 Conclusions

The results of data collected for the model study and the field studies provide a wealth of information. Model study results were compared with past data in order to provide a comprehensive analysis of the results. This information could then be used to determine the effectiveness of a model study backfill design in relation to past designs and the current backfill design incorporated by the SDDOT. Model study data could then be compared to field study results. The field study results provide information on the effectiveness of a new backfill design incorporated by the SDDOT.

The model study showed that the development of voids under the approach slab is reduced by using a vertical layer of rubber tire chips behind the integral abutment. A design incorporating the use of a layer of rubber tire chips behind the integral abutment provides a cushion into which the abutment can cycle into and away from without significantly affecting the backfill soils.

Earth pressure data shows that a composite design incorporating the use of vertical layer of rubber tire chips behind the integral abutment greatly relieves passive earth pressures induced on the retained fill, which reduces the mechanisms causing void development. Pressure cell readings indicated a final cell pressure of approximately 7.5 psi in the case where a rubber tire chip layer was incorporated in the backfill system. A final cell pressure of approximately 60 psi was recorded in the case no layer of rubber tire chips was behind the abutment. This indicates that the movements of the integral abutment into a backfill system incorporating a layer of rubber tire chips significantly lowers the earth pressures imposed on the integral bridge abutment. Earth pressures are greatly reduced and void development due to high earth pressures is significantly reduced for a backfill system incorporating a layer of rubber tire chips.

Inclinometer data, approach slab elevations, and embankment stake elevations reveal similar results in that the cyclic movements of the integral abutment do not significantly affect the embankment soils. Deflections in the backfill evident through inclinometer tube movements were nearly eliminated in the case where a layer of rubber tire chips was used in the backfill system. Movements of the backfill soils were reduced by as much as 3 inches in some instances over the case where no layer of rubber tire chips was used. Embankment movements are greatly reduced and void development due to high lateral and longitudinal movements of the embankment soils are reduced with a backfill incorporating a layer of rubber tire chips.

Rearrangement of the individual rubber tire chips in the tire chip layer causes consolidation in the tire chip layer as the abutment cycles. This allows movement to occur in the retained soils directly behind the rubber tire chip layer. This soil movement allows a void to develop under the approach slab that is similar to the case when no rubber tire chip layer is used. The significant difference is that in the case of the backfill incorporating a layer of rubber tire chips behind the integral abutment, the development of a void under the approach slab is reduced approximately one-half the amount that was recorded during the same time period for the case where no layer of rubber tire chips was used in the backfill.

Based on the results of the model study, it has been determined that the occurrence of a void under the approach slab is not eliminated by using a layer of rubber tire chips between the integral abutment and the retained embankment fill. Results have shown that the layer of rubber tire chips greatly decreases passive earth pressures that had previously been shown by Schaefer and Koch (1992) to develop behind the integral abutment. Other mechanisms that cause void development such as embankment bulging, approach slab uplift, backfill densification, and backfill deformation as passive failure occurs in the backfill are reduced or eliminated. Passive failure of the backfill system occurred in the past model test where no layer of rubber tire chips was used. In the case where a layer of rubber tire chips was used, passive failure did not occur in the backfill system. However, the development of a void directly behind the layer of rubber tire chip layer, under the approach slab, still develops as a result of the consolidation of the rubber tire chip layer as the abutment moves.

Cracking in the embankments occurs at the transition point between the rubber tire chip layer and the backfill material behind the wing wall sections of the embankment. This was due to the movement of the 6-inch depth of embankment soil material placed over the rubber tire chip layer as a cover material for fire protection. The cracks develop as a result of the movement of the integral abutment cycling into and away from the backfill soils. This transition area poses a problem for design in that dissimilar materials are present with a large variation in movement between the two types of materials.

One of the most important aspects of the model test is indeed the development of the void under the approach slab. Void development is reduced, but not eliminated.

The use of a fabric reinforced soil wall behind the integral abutment reduces the development of a void under the approach slab sections. Gap width measurement data has shown that an adequate gap width is being maintained in the areas of measurement through the warm cycles of abutment movement when movement of the bridge structure is at its highest expansion cycle.

Gap width measurements show that an initial movement of the geotextile reinforced soil wall toward the integral abutment occurs after the cardboard separator is removed. This movement is in the order of 0.5 to 2.0 inches in some instances. This reveals some problems associated with the construction of the soil wall. It is important to maintain significant tension in the soil wrapped layers in order to prevent sliding of the soil wall after the cardboard is removed. This sliding was most evident in the south abutment embankment of the White River Bridge. Some of these problems were alleviated before the construction of the north abutment embankment. Although, some movement is evident in all of the soil walls, especially within the first month after the cardboard is removed. Again, this was anticipated and a design incorporating a 6-inch gap was used in order to maintain an adequate gap width after this initial movement occurs. The 6-inch gap appears to be optimum for allowing the movements of the bridge system to occur while providing adequate space for sliding of the geotextile reinforced soil wall to occur without closing the 6-inch gap.

The Big Sioux River Bridges gap width measurement data showed significantly less initial movement of the soil wall after the cardboard is removed than was the case for the White River Bridge. Since the design details of the geotextile reinforced soil wall for the three bridges are very similar, it can be concluded that the Big Sioux River Bridge reinforced soil walls were constructed with better results. The 6-inch gap has shown to be sufficient as evident from all abutment gap width measurements.

Gap width measurement data shows a definite trend in movement of the integral abutment in relation to the time of year in which the measurements are taken. In general, the warmest temperature periods show that the gap has closed due to the expansion of the bridge system. This expansion closes the gap during high temperatures that occur in the summer. Gap width measurements show the widening of the gap during the cold temperature periods as well.

Incorporating a gap between the integral abutment and the backfill soils has eliminated pressures induced to the backfill. This is the case for the full width of the driving lanes, but not behind the wing wall sections of the abutment. There is no gap and soil wall constructed behind the wing wall sections of the integral abutment. This introduces a severe problem in that embankment soils behind the wing wall sections continue to be affected by the cyclic movements of the integral abutments. Temperature induced movements of the integral abutment are affecting the soils behind the wing wall sections of the abutment and are causing a gap to form in these areas. This gap creates a pathway for erosion behind the wing walls exposing the 6-inch gap under the approach slab. This erosion of embankment material allows the movement of soil material into the 6-inch gap, disturbing the backfill system at the sides of the approach slab.

Construction of the transition area between the geotextile reinforced soil wall and the embankment soils behind the wing walls must be performed as design specifications detail. Evidence supports that inadequate construction of the transition areas between the 6-inch gap and the embankment soils behind the wing walls introduces significant displacement and erosion of the embankment soils. In some instances, these soils are penetrating into the 6-inch gap, rendering ineffective the 6-inch gap at the sides of the approach slabs.

1.6 Recommendations

Recommendation One – Use of alternative backfill:

The use of select backfill in the reinforced soil wall behind the integral abutment may not always be necessary. The use of this backfill was initiated to increase the passive resistance of the soil when it was placed directly against the integral abutment and to assure adequate compaction by the contractor in a restricted and confined area. By eliminating the passive pressures and through reinforcement of the soil with the geotextile, alternative backfill material may prove to be suitable. Implementation of this recommendation requires that the alternative material can be adequately compacted in restricted areas and the use of such materials would provide a cost advantage to the state.

Recommendation Two – Reduction in gap width:

A reduction in the installed gap width may be warranted when the initial gap is constructed in the summer (at times of maximum bridge expansion). When a contractor experienced in placement of wrapped face geotextile walls and/or field inspection can ensure the wall is properly constructed (to minimize post construction movement), a six inch gap may be overly conservative. Measurements on the Big Sioux River Bridge show no gap measurements below five inches and in some cases the gap expanded to 12 inches in the winter. If a bridge is constructed during times of maximum expansion, then the installed gap width may be reduced to 3 to 4 inches.

Recommendation Three – geotextile embedment:

Field monitoring and inspection during the construction of the wrapped face geotextile wall is critical to the performance of the system. A poorly placed wall will deflect laterally during construction causing reduction in gap width and surface settlement. The geotextile should be well embedded into the soil and each layer should be tightly wrapped and tensioned prior to soil placement. Some of these problems should be reduced with the design changes implemented in the 1999 MSE BEB design.

Recommendation Four – Erosion behind the wing walls:

A new design of the backfill and wing wall interface needs to be developed to prevent erosion of the backfill material. During bridge expansion, the wing wall pushes into the backfill and then pulls away during contraction. This creates a gap between the wing wall and soil providing a conduit for drainage. This water causes erosion and some of the soil is being washed into the gap behind the bride abutment. Because of this, a better physical barrier needs to be installed to protect the gap from infilling. Additionally, a gap or some compressible material may be needed behind the wing wall to prevent gaps from forming behind the wing wall. A rubber tire chip layer surrounded by a separation geotextile may be a suitable candidate design.

Recommendation Five – Use of rubber tire chips:

The design of a full-scale integral abutment bridge using a layer of vertical rubber tire chips and no geotextile reinforced fill should be developed and tested in the field. This design, if effective, may be less costly and easier to construct then the geotextile wall system tested in this research. Recent advances by other State DOT's has shown increased use and confidence in the use of rubber tire chips for reducing fill weight and reducing lateral soil pressures on structures. The use of tire chips in this application would require careful monitoring of tire chip compaction to prevent the settlement problems observed in the model study portion of this research.

CHAPTER 2

PROBLEM DESCRIPTION

2.1 Problem Statement

The "bump" at the end of the bridge has long plagued motorists, designers, and maintenance personnel. The bump results from differential movements of the approach road and the bridge abutment and causes significant discomfort to motorists, increased damage to vehicles, traffic hazards and safety concerns, increased maintenance costs, and in extreme cases can impair the structural integrity of the bridge. This problem is experienced in all states and is a recurring problem for transportation officials. South Dakota has experienced a particular problem of void development under bridge approaches.

In a previous research project on this problem, SD90-03 – Void Development Under Bridge Approaches, Schaefer and Koch (1992) conducted field and model studies of the occurrence of voids under approach slabs. A field investigation of over 130 bridges showed a primary correlation between void development and integral abutment bridges. The observation of the occurrence of voids under approach slabs in cases where no traffic had yet occurred led to a hypothesis of thermally-induced movements of the bridge beams/abutment walls as the mechanism causing void development. Subsequently, a model study was undertaken to test this hypothesis. One of the most important aspects of the model study was the degree of correspondence between the effects of the abutment movement on the backfill and embankments and the observation of those same features in field studies. Such correspondence provided a large degree of confidence that the model study indeed replicated the field behavior. The model study showed that the development of void space under the approach slab is a direct consequence of the thermal-induced movement of the bridge beam/abutment wall system. The study concluded that the development of voids under the approach slabs to integral abutment structures was an inherent problem in the use of integral abutment systems.

As a result of identification of the mechanism of void development, changes to the approach system need to be made to accommodate this mechanism. A most likely change is to construct the approach system with a void behind the abutment wall so that the thermally induced movements can be accommodated without affecting the backfill. There are a variety of ways to incorporate a void behind the abutment. A method used by the Wyoming Department of Transportation (WYDOT) uses a geotextile reinforced soil wall behind the abutment to build a vertical, self-contained wall capable of holding a vertical shape and forming a void behind the abutment. Another change, which could alleviate the problem, is the use of a more elastic backfill material than presently in use.

2.2 Background Summary

Although numerous factors contribute to the differential movements between bridge abutments and the approach areas to the bridge, previous research has shown that void development under approach slabs to integral abutment bridges occurs due to the elongation and contraction of the bridge beams due to seasonal temperature variations. As the bridge beams expand and contract they alternately push into and pull away from the backfill behind the abutment wall, leading to the development of a void near the abutment wall under the approach slab. The observation of such voids in the field has been reported by Jorgenson (1983), Kreamer and Sajer (1991), and Schaefer and Koch (1992). The size of the void varies markedly, with Schaefer and Koch (1992) reporting measured voids from 13 to 360 mm (0.5 inch to 14 inches) in height and extending as much as 3 m (10 ft) away from the abutment wall.

Schaefer and Koch (1992) confirmed the mechanism of void development as being the result of thermally induced movements of the integral abutments through the use of a model. The model study showed that the development of void space under the approach slab is a direct consequence of the thermal-induced movement of the bridge beam/abutment wall system. The study concluded that the development of voids under the approach slabs to integral abutment structures was an inherent problem in the use of integral abutment systems. The void development was not considered to be isolated to one mechanism resulting from the abutment movement, but rather to be the result of the cumulative effects of embankment bulging as the backfill deforms, approach slab uplift, backfill densification as particle breakage occurs, and backfill deformation as passive failure occurs in the backfill. The relative contribution of each of these mechanisms to void development would be difficult to discern. It was suggested that the largest increases in void size occurred when passive failure likely occurred and this mechanism was probably the most important one.

The void development under approach slabs identified during the Schaefer and Koch (1992) study would be most easily eliminated in future structures by returning to the use of non-integral abutment structures. However, this is unlikely because of the proven engineering and cost saving benefits provided by integral abutment design. A potential solution is then to construct the integral abutment system with a void behind the wall such that the thermally induced movements do not affect the backfill. Another potential solution would be to place an elastic material between the bridge abutment and the backfill to reduce the passive pressures on the retained fill.

CHAPTER 3

OBJECTIVES

3.1 Defined Objectives

The overall objective of this research was to determine if constructing an approach fill to an integral abutment structure with a void between the abutment wall and the fill will alleviate problems with the development of a void under approach slabs to the end of the bridge. To meet this overall objective the following sub-objectives were identified:

1. To design, construct, monitor and evaluate an actual bridge end backfill reinforced wall similar to that used by WYDOT. This will be accomplished by constructing full-scale fabric reinforced soil walls as the approach backfill on actual bridge approaches of new and/or retrofitted bridges.
2. To determine the feasibility of using other materials as a bridge end backfill material. This shall be investigated primarily through the conduct of model scale tests of alternative backfill materials using the SDDOT model integral abutment system at the Brookings DOT Maintenance Yard.

3.2 Accomplishments

The objectives of this research were met through the completion of the research tasks listed in Chapter Four and the findings are detailed in Chapter Five. Based on the results of the two year field study, the installation of a six inch gap by use of a geotextile wall behind the integral bridge abutment is sufficient to keep the abutment from contacting the fill. This prevents passive pressures from being developed in the soil thus eliminating the primary mechanism of void development. This has resulted in a decrease, but not elimination, of void development.

In the model study, a vertical layer of rubber tire chips reduced the pressure induced on the backfill from the bridge abutment by an order of magnitude when compared to that of a backfill of just select granular material. For abutments backfilled with only select granular material, pressures as high as 56 psi were measured on the backfill. For the abutment backfilled with a vertical layer of rubber tire chips, the highest pressure on the backfill was 7.5 psi. Deflections as measured by the inclinometers in the backfill showed almost no measurable movement in the backfill protected by the tire chip layer, while the fill of only granular backfill had deflections of up to 2 inches. Changes in elevation of the approach slab also dropped an order of magnitude from 0.25 to 0.5 inches to 0.01 to 0.02 inches for the system using the rubber tire chips.

CHAPTER 4

TASK DESCRIPTION

4.1 Research Plan

The work plan for carrying out the research objectives is detailed below. The plan takes the form following the tasks in the original research proposal and details the methods for performing each of the tasks. The information obtained in the accomplishment of these tasks is discussed in greater detail in Chapter 5.

Task 1

Review and summarize literature regarding bridge end backfill designs that attempt to prevent void development beneath the approach slab.

A literature review was conducted through the use of The Engineering Index ®, Science Citation Index, on-line searches of the Transportation Research Board and an in-library search of recent Transportation Research Records. Additionally, research reports of State Departments of Transportation in Maine, Indiana, Wyoming, Wisconsin, Minnesota, North Dakota, Washington, and South Dakota were obtained.

Task 2

Interview WYDOT geotechnical, design, field, and research personnel to better understand all aspects of their fabric reinforced soil bridge and backfill design.

At the time this project was awarded, the bid had already been let for the White River Bridge and a project modification was being developed to incorporate the WYDOT design into that bridge. The information needed from WYDOT was obtained by SDDOT and passed on to the research team.

Task 3

Work with SDDOT to adapt WYDOT's backfill design to SDDOT's bridge geometry. Bridges to be constructed in 1996 will be selected by Department personnel for use in this study.

Adaptation of the WYDOT design was made through meetings of the research team as well as telephone meetings with the SDDOT personnel.

Task 4

Submit for approval an instrumentation plan to monitor movements in the abutments, embankments, fabric-reinforced walls, approach slabs, and sleeper slabs. The plan should include ways to monitor voids behind the abutment walls and beneath the approach slabs.

Based on the research budget and available equipment, an instrumentation plan was developed to monitor the voids behind the abutment walls and beneath the approach slabs. This plan was developed and approved in cooperation with the SDDOT.

Task 5

Observe the construction of the bridge ends, documenting sequences and construction problems.

Due to the accelerated construction schedule of the White River Bridge only one visit was made during construction. Numerous visits were made during the construction of the Big Sioux River Bridges. Photographs were taken documenting the construction sequence, however no problems were encountered during construction.

Task 6

Monitor the performance of the bridge ends for a period of two years after construction.

A total of twenty sets of measurements over summer and winter temperature cycles were taken on the White River Bridge over a two-year period. Twenty four sets of measurements were taken on the South Bound Big Sioux River Bridge and twelve sets of measurements were taken on the North Bound Big Sioux River Bridge.

Task 7

Review other possible materials that can be used in bridge end backfill designs to alleviate the effects of thermally-induced stresses (i.e., expanded polystyrene blocks, rubber chips from discarded tires).

Through an extensive literature review, numerous alternative backfill materials were identified and examined. These materials included: bark and sawdust, foamed concrete, rubber tire chips, rubber tire chips and soil mix, polystyrene foam, and lightweight fill. This review led to the selection of the material used in the model study.

Task 8

In conjunction with SDDOT, develop a backfill design which utilizes the material with the greatest potential determined in Task 7.

Based on the information obtained in Task 7, rubber tire chips were selected as the material to be used in the model study and a design was developed. This design was finalized in the summer of 1996.

Task 9

For the Brookings DOT model abutment, submit for approval an instrumentation plan to monitor movements in the abutment, approach slab, and sleeper slab for the design developed in Task 8. Also, the plan should include ways to monitor any deformation of the backfill or the embankments.

Through discussion with DOT personnel, the design of the model test using the rubber tire chips and the instrumentation plan was approved for testing. The instrumentation plan included measurements of pressures on the backfill, internal backfill movements, embankment movements, slab movements, and void development.

Task 10

Using the SDDOT model integral abutment at the Brookings DOT Maintenance Yard, construct a bridge end using the backfill design and instrumentation plan developed in Tasks 8 and 9, documenting construction sequences and problems. This should be constructed during the 1996 construction season.

Construction of the model began in late summer of 1996. However, severe weather conditions and an early winter resulted in a premature end to construction. Construction of the model and installation of instrumentation was completed in 1997.

Task 11

Using accelerated testing, monitor and evaluate the bridge end model.

In the summer of 1997, the model was tested using the accelerated testing methods. All necessary data was successfully collected for analysis.

Task 12

Compare the costs of SDDOT's present bridge end backfill design and performance to that constructed in Task 5. Also, for the model abutment constructed in Task 10, estimate the costs associated with the backfill design and performance as if it were used in a full size bridge.

Bid prices were obtained from 1996 and 1997 on 13 bridges using bridge end backfill (BEB) without the mechanically stabilized earth (MSE) wall and on 18 bridge bids on MSE BEB projects constructed from 1996 through 1998. A comparison of costs between present bridge construction costs and the costs associated with the system monitored in this research was performed. Also, for the model abutment constructed in Task 10, costs were estimated as if it were used in a full size bridge. All costs were adjusted for inflation to 1998 costs.

Task 13

Submit an interim report by December 13, 1996, summarizing literature, research methodology, data, findings, conclusions and preliminary recommendations.

Written and verbal progress reports were submitted throughout the duration of the project.

Task 14

Submit an executive summary and a final report that incorporates the interim report and results of the full evaluation period.

The executive summary and final report were submitted in August, 1999.

Task 15

Present findings and conclusions to SDDOT's Research Review Board upon completion of the project.

An executive presentation was made to the SDDOT Research Review Board in August, 1999.

CHAPTER 5

FINDINGS AND CONCLUSIONS

5.1 Introduction

The research was conducted as outlined in the task descriptions with some deviations as discussed in Chapter Four. The tasks outlined in the original research proposal focus on three general areas: a) Literature Review, b) Model Study, and c) Monitoring of the White River and Big Sioux River Bridges. The results of the research tasks will be presented with respect to their relevance to these general areas.

5.2 Literature Review

The review of the literature focused on three areas: a) previous SDDOT studies, b) designs used by other states, and 3) alternative backfill materials. The purpose of this portion of the study was to examine what other organizations are doing to reduce void development under approach slabs and to determine if any new approaches could be developed using alternative materials.

5.2.1 Temperature Effects on Integral Abutment Bridges

Three previous studies have been conducted by the SDDOT that are related to this research. Two of these studies focused on the temperature effects on integral abutment bridges during construction (Holman, 1972; Lee and Sarsam, 1973). These studies were initiated to examine the thermally induced stresses on the bridge during and after construction. The study was conducted during the time the State of South Dakota was beginning the transition to the construction of integral abutment bridges. As part of these programs, a model bridge, with a pile-supported integral abutment was constructed to monitor the thermally induced stresses on the bridge. This study did not consider any effects these thermal movements would have on the abutment backfill.

5.2.2 Void Development Under Bridge Approaches

Several years after implementation of integral abutment design methods and bridge structures in South Dakota, problems began to develop in bridge approach systems. The development of voids under the approach slab was observed to be causing cracks to develop in the approach concrete slab. A study was then conducted (Schaefer and Koch, 1992), to determine the extent and cause of the problem within the state. The field study of over 130 bridges in South Dakota showed that most integral abutment bridge systems and some non-integral abutment bridge systems suffered void development under the approach slab. Of the 79 integral abutment bridges constructed since 1980, voids under the approach slab were observed in every bridge but one.

Some of the problems that had been observed from the development of voids under the approach slab were approach slab cracking and embankment cracking. Cracking of the approach slab occurs due to the unsupported loading of the slab from the development of voids and was evident in one-half the bridges inspected. In most instances, the cracking occurred at the end of the dowel bars that protrude from the bridge deck into the approach slab. Cracking of the approach embankment typically developed from the contact of the wing wall and the abutment while radiating out 45 degrees.

In order to confirm that the primary mechanism causing the development of voids under the approach slab is the thermally induced movements of the integral abutment, a model study was performed in order to replicate abutment movement. This study used the same model that was constructed for the initial research from the early 1970's.

When the model study was completed and the data analyzed, it was determined that the primary mechanism causing void development under the approach slabs was the thermally induced movements of the integral abutment. This mechanism was also determined to be responsible for the development of cracks in berms and approach embankments as the abutment expands and contracts. It was determined that void development effects are compounded by the combination of several mechanisms. These mechanisms are typically embankment bulging as the backfill deforms, approach slab uplift, backfill densification as particle breakage occurs, and backfill deformation as passive failure occurs in the backfill. Cracking of the approach slab is a direct consequence of the loss of support under the approach slab, which affects the flexural stresses induced in the slab.

5.2.3 Designs Used by Other States

Geotextiles have been in use for several years for reinforcement purposes. Recently, research has incorporated the use of geotextiles in order to reduce or eliminate the mechanisms causing the bump at the end of the bridge behind integral abutment bridges. This was accomplished by constructing a backfill that was well drained and generally self-supportive. Both the Oklahoma Department of Transportation (OKDOT) and the Wyoming Department of Transportation (WYDOT) have produced studies evaluating the performance of such a design to eliminate the bump at the end of the bridge. The following will briefly describe their research performed thus far.

5.2.4 Oklahoma Department of Transportation Backfill Designs

In an effort to reduce or eliminate the bump at the end of the bridge problem, the OKDOT developed a research program to evaluate five differing approach embankments. The purpose of this research was to evaluate the performance of four experimental approach embankments and compare them with an identically instrumented control section constructed using typical materials and methods of construction (Snethen *et al.*, 1997).

The control section for this research was constructed on the north end of bridge A (A2) and was composed of an unclassified borrow material. A geotextile reinforced granular backfill was constructed on the south end of bridge B (B1). The north end of bridge B (B2) was constructed with controlled low strength backfill. The south end of bridge C (C1) was constructed using dynamically compacted granular backfill. The north end of bridge C (C2) was constructed with flooded and vibrated granular backfill. The performance of the geotextile-reinforced backfill was of interest due to its similarity to the type of design studied in this research.

5.2.4.1 Embankment Instrumentation

In all five cases, instrumentation was used in order to allow for a comparison of data after completion of the test period. Lateral stresses exerted on the abutments were measured with three total pressure cells positioned along the centerline of the abutment wall. Deformation was measured using amplified liquid settlement gages. One gage was placed on the centerline and one 10 feet west of the centerline, both 2 feet vertically below the base of the abutment wall and 6 feet horizontally behind the abutment wall. When movement occurred, the buried gage moved and the head relative to a reference reservoir located on the west wing wall was measured. Monitoring of the water pressure, lateral movement, and additional vertical deformation was performed with one open-tube piezometer installed on the centerline of each experimental backfill. Three inclinometer casings with telescoping couplings were used to measure lateral and vertical movements. Surface deflections were also measured at various points. These points were installed on the surface of the asphalt pavement once paving was completed.

5.2.4.2 Geotextile Reinforced Wall

The approach backfill design used behind the south abutment of bridge B was a geotextile reinforced soil wall. It was chosen due to its ability to support its own weight. The wall was constructed using a common overlapping technique in which the tension on the folded portion of the geotextile is resisted by the pressure of the overlying lift. This ultimately creates a freestanding wall supported by its own weight.

The excavation was first leveled to a 5:1 incline away from the abutment wall. A base lift of granular material was then placed. Several 4 ft X 12 ft X 2 inch panels of collapsible honeycomb cardboard were longitudinally attached to the abutment and wing walls. The panels were left in place and collapsed prior to paving by wetting the cardboard. Once the first row of cardboard spacers had been set, construction of the reinforced soil wall began. The estimated cost was approximately $25,000 for installation over a 5-day period.

5.2.4.3 Results

The geotextile reinforced soil wall embankment showed an increase in the lateral earth pressure with depth. This backfill system was designed to be self-supporting and should never become in contact with the abutment wall. However, the geotextile wall has moved and closed the two-inch gap. Since the backfill had become in contact with the wall, it became evident that possible reasons for this problem should be determined. It was determined that the geotextile could have had some amount of elongation and that the density of the sand within the backfill could be insufficient. The average relative density for the eight layers was only 25.8 percent, which resembles a loose sand material.

Conclusions after evaluating the performance of each of the backfill systems revealed that each of the approach embankments has advantages and limitations with regard to materials, construction procedures, and performance. The geotextile reinforced wall is an excellent option for addressing both settlement and lateral earth pressures. However, some problems arose with the materials used in the construction and construction method (Snethen, et. al., 1997). In order to alleviate some of the problems, the geotextile used should have low elongation under load. A woven geotextile may have been a more appropriate fabric. The geotextile rollback on the top of each layer should be more uniformly tensioned and anchored in some way to hold the tension. Thinner compaction lifts may also be necessary to achieve better compaction in the backfill material. A relative density specification should also be used for compaction control rather than a proctor specification.

5.2.5 Wyoming Department of Transportation Backfill Design

In the late 1980's, WYDOT developed a backfill design using a fabric reinforced soil wall behind the abutment to produce a gap between the reinforced soil wall and the integral abutment. Research involved construction of four embankments using different techniques for reinforcing the embankment and supporting the soil behind the abutment. Embankment construction schemes included: 1) no geotextile reinforcement, 2) reinforced soil backfill directly against the face of the abutment, 3) initial 2 inch void between the reinforced soil backfill and abutment, and 4) initial 6 inch void between the reinforced soil backfill and the abutment (Edgar *et al.*, 1989). The first embankment represented the construction design used by the WYDOT prior to reinforcing the embankments. The second scheme had been the construction technique used by the WYDOT at the time the research was performed, and the third and last construction schemes were those developed by the University of Wyoming for the purpose of testing and comparison with past and existing design practice.

5.2.5.1 Embankment Instrumentation

The embankments were instrumented using several different devices. Twenty-two total pressure cells were installed in the embankments to measure vertical and lateral pressures developed in two of the reinforced embankments. Eleven pressure cells were installed in the other two embankments. Three of the 11 cells were installed horizontally to measure the vertical stress. The other eight cells were mounted flush with the surface of the abutment in pairs 12.5 feet apart and at the same depth.

Sixteen inclinometer casings were installed in the four embankments in combination with vertical Sondex tubes. Six horizontal Sondex tubes were installed horizontally through holes in the abutment in the three geotextile reinforced embankments.

5.2.5.2 Embankment Construction

Each of the embankments was constructed using a different design scheme. In three of the embankments, the existing material had to first be excavated. This excavated material was then replaced with a select granular backfill containing ¾-inch crushed rock. These embankments were reinforced with a MIRAFI ® 600X heavy duty woven polypropylene geotextile fabric. The forth embankment was left in place without reinforcing geotextile fabric (Edgar *et al.*, 1989).

The approach embankment used in bridge design was an earth ramp that supports an approach slab leading up to the bridge structure. The embankment slopes were positioned with high slopes to reduce cost, which ultimately inhibits the possibility of lateral spreading of the embankment and backfill material. This allows vertical displacement to occur under the approach slab in the form of a void.

In an effort to control these deformations, the WYDOT initiated the use of several layers of geotextiles in the embankment for a distance of 24 to 40 feet from the back of the abutment wall. The bottom layer of the high strength fabric is typically placed on the plane of the top elevation of the abutment footing (Edgar *et al.*, 1989). The edges of the geotextile are draped up the sides of the back face of the abutment and the wingwalls. A lift of approximately 9 to 10 inches is then placed and compacted on the fabric. An additional berm is then placed along the walls to provide a form for the next lift. The geotextile is then lapped over that berm and the second lift is completed. The lapped edge extends about 3 feet from the wall into the embankment and is then buried under a second lift of fill. The fabric was placed easily and quickly. The sheets were placed and overlapped at least 2 feet. While backfilling, no direct contact came between the fabric and the equipment. A minimum of 6 inches of backfill was placed within each lift. The lifts were constructed until the desired number of lifts for each embankment were completed.

The construction of the embankment with the 2-inch cardboard between the fill and the abutment was a simple technique. The cardboard was easy to install and was done so without any apparent difficulty. The construction of the embankment with the 6 inch plywood form was more difficult due to the removal of the form necessary to be removed after each lift was placed.

5.2.5.3 Results

It was determined that the reinforced embankment is an effective technique to control short-term deformations (Edgar *et al.*, 1989). Installing a void between the fabric reinforced soil wall and the abutment wall is an effective design technique for reducing lateral loads on abutments.

It was noticed that some elongation of the geotextile occurred toward the abutment after completion of construction and the forms were removed. In the case for the plywood form, the lifts slid completely into the abutment, practically closing the void created by the forms. It was determined that this could be a result of insufficient overburden weight on the embedded length of the geotextile when the forms were removed or that the Rankine failure inside the wrap was sufficient to allow additional fill to collapse.

The fabric reinforced embankment constructed using the cardboard separator showed lower lateral earth pressures than those measured for the fabric reinforced embankment constructed directly against the abutment without a separator. This embankment developed higher lateral pressure cell readings in the top cells more than any of the cells located at further depths. In all three reinforced embankments, smaller lateral movements occurred under the roadway surface compared to larger lateral movements in the side slopes. This was expected to be from the way in which the embankment was constructed. The sides were not wrapped with geotextile reinforcement and therefore allowed lateral movements to occur more readily. It was determined that if long-term movements became significant, then the sides would need to be constructed with transverse fabric reinforcement. The unreinforced embankment experienced larger vertical deformations compared to the reinforced embankments. Approximately 9.72 inches of deformation was recorded versus 1.5 inches of deformation for the reinforced embankment directly behind the abutment. This was attributed to lack of compaction of the fill material in the embankment without reinforcement. The reinforced embankment was stiffer which resulted in less settlement. It was determined that heavily reinforced concrete approach slabs are necessary to span the voids caused by the differential movement in the embankments. It was also observed that deformation occurred in all the embankments with more occurring in the unreinforced embankment. This movement was non-uniform and unpredictable other than that larger vertical deformations occurred close to the abutment (Edgar *et al.*, 1989).

5.2.6 Alternative Backfill Materials

In order to evaluate alternative backfill materials, a comprehensive investigation of possible fill materials is needed. Fill materials that possess the engineering characteristics desirable for applications as backfill for integral abutments are lightweight fill materials that exhibit elastic capabilities. A lightweight fill is any material that is used to replace a heavier in-situ soil for reducing the load burden on subgrade soil. There are essentially four primary types of lightweight fill materials. These are bark and sawdust, foamed concrete, shredded tires, and polystyrene foam.

5.2.6.1 Bark and Sawdust

Wood chips provide a good working area for fill and light machinery. The material does not displace in front of machinery or fill and has sufficient stability to withstand submersion in water. However, wood chips are easily displaced in flowing water conditions.

When using sawmill residue or timber corduroy as a road embankment material, the bio-deterioration of the ligno-cellulosic material has to be controlled. More than 50 percent of wood is made up of cellulose, 10 percent to 35 percent is composed of lignin. Lignin is a more complex carbohydrate than cellulose. However, both will break down under aerobic conditions. Wood material is a viable material for a lightweight fill application. The biodegradation characteristics of this material require design parameters to combat this process. Wood materials are considered a by-product and therefore are generally inexpensive (Kohlnhofer and Marti, 1992).

5.2.6.2 Foamed Concrete

Foamed concrete has been used in building construction for floors, roof decks, insulation, and recently has been used as a lightweight fill material. The primary applications in highway construction thus far has been in bridge abutments, mostly due to its failure to exhibit horizontal pressures on the adjoining structures and the high strength characteristics it possesses. Various classes of the material are produced with different strengths and densities.

The cost of this fill material is 40 to 45 dollars per cubic yard, which may make it an economical alternative for larger projects like bridge abutments (Kohlnhofer and Marti, 1992).

5.2.6.3 Rubber Tire Chips

The use of rubber tire chips as a fill material has previously been investigated by Ahmed and Lovell (1993), Humphrey, *et al*., (1993), Upton and Machan (1993), Edil and Bosscher (1992), Manion and Humphrey (1992), Engstrom and Lamb (1994), and Ahmed (1993). Based on a review of these works, a number of systems using rubber tire chips have been identified. These systems include: 1) Rubber tire chips and a soil cap, 2)

A soil capped layer system of equally distributed horizontal sections of rubber tire chips and soil, and 3) A capped mix of rubber tire chips and soil. In order to evaluate the potential of these systems toward meeting research goals, the engineering properties of backfill materials used in highway applications must be established.

A major concern in using tire chips in highway structures is the large compressibility of chips observed in various field and laboratory studies. The elastic properties of tire chips also cause a problem by producing higher deflections than soil in a fill material. For example, Engstrom and Lamb (1994) do not recommend shredded tires for use under hard surfaced pavements unless a substantial minimum overburden thickness of 5 feet is used to reduce settlement.

It is important to identify the beneficial engineering properties of tire chips as a fill material as well. For instance, compacted shredded tires are more porous than washed gravel. When rubber tire chips are used in a road base or subbase, they provide improved drainage below the pavement and extend the life of the roadway. Since rubber tire chips are elastic, they more efficiently distribute the loads from roadways over unstable soils. The most important benefit of using tire chips is reduced weight of fill, which helps increase stability, reduce settlements, reduce backfill pressure on retaining structures, and correct or prevent slides on slopes (Ahmed, 1993). Stockpiles of waste tires will not be depleted for a considerable amount of time, which will prevent material costs from increasing. Using shredded tires as a fill material will help free up other more expensive and valuable fill materials as well.

The engineering properties of each system will be presented to obtain parameters in analyzing the available systems of consideration for a model study. Some of the properties are compressibility, shear strength, hydraulic conductivity, resilient modulus, and compaction behavior. The following will analyze each of these engineering properties for each candidate system based on previous research.

Compressibility. There are generally three mechanisms that are responsible for the total compression of tire chips. Figure 5.1 illustrates these mechanisms and shows a plot of vertical strain versus log vertical stress for various ratios of rubber-sand mixes. During the first loading, a small compression occurs which is mostly irrecoverable. Then

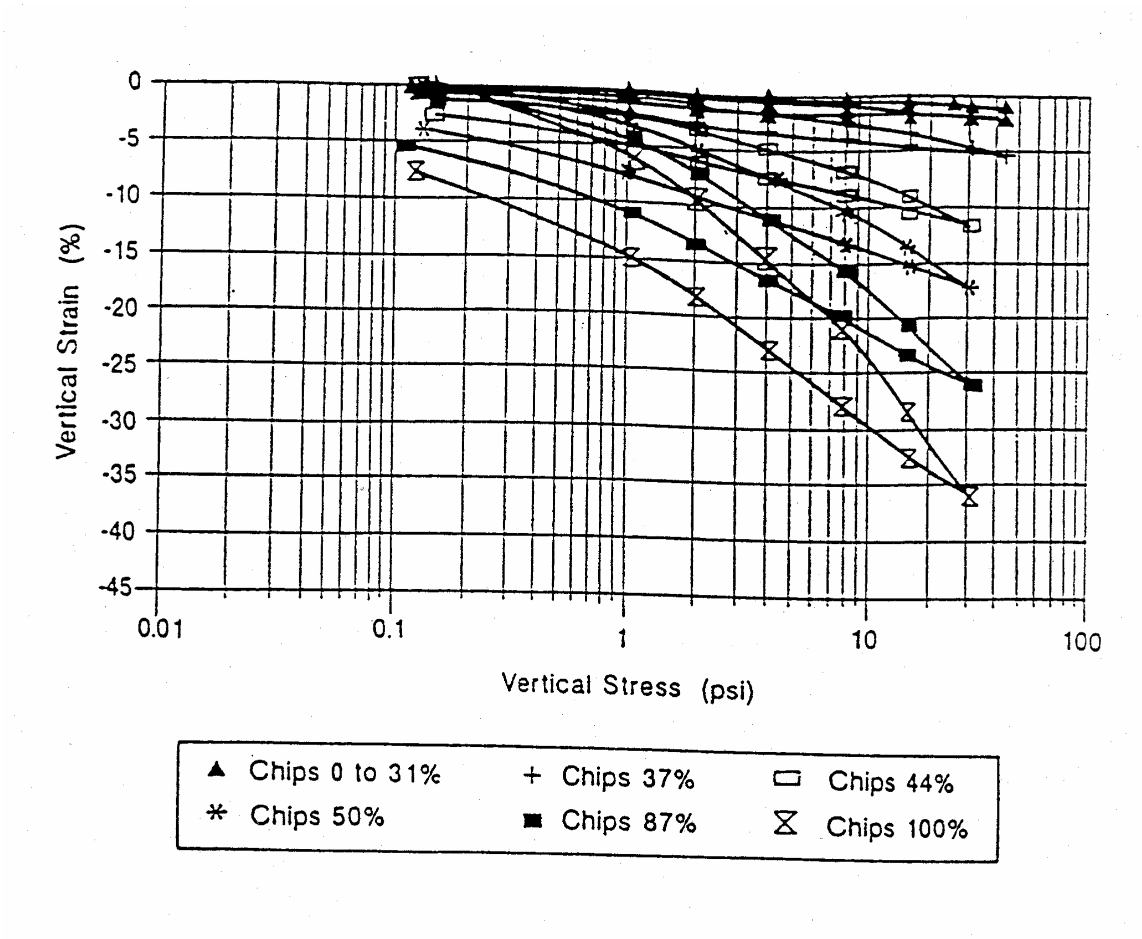


Figure 5.1 Compression Behavior of rubber sand with variation in chip/soil ratios (Upton and Machan, 1993)

compression due to bending or flattening of chips occurs. This is largely responsible for the major portion of total compression and is mostly recoverable on unloading. Compression due to elastic deformation then occurs which is nearly completely recoverable upon unloading of the material (Ahmed and Lovell, 1993).

Tire Chips with a Soil Cap. In a system containing pure rubber tire chips with a soil cap, the horizontal stress increases with increased vertical stress upon loading (Manion and Humphrey, 1992). A gap in the vertical stress-strain curve exists between the initial loading and subsequent loading due to the compression of voids in the sample. At higher stresses, a large slope in the stress strain curve indicates the compression of primarily rubber tire chips. Overall, the results of compression tests done on pure rubber tire chips show the high compressibility of rubber tire chips at low stresses, but decreasing compressibility occurs upon loading to higher stresses (Manion and Humphrey, 1992). The soil cap layer in this candidate system would be used to reduce post-construction settlement, for improving overall aesthetics, and to eliminate the vehicle tire puncture and fire hazards.

Layered System. A layered system of rubber tire chips and sand tested under cyclic loading has been studied by Edil and Bosscher (1992) and is another candidate system that has been considered. This research studied the compressibility of this type of a layered system in order to simulate the loading caused by repetitive traffic loads. The layered system consisted of (from the bottom) a 1 foot layer of tire chips, 1 foot of outwash sand, 1 foot of tire chips, and 2 feet of outwash sand. This configuration was compared to three other model tests. Model test one contained 3 feet of tire chips with a 2 foot outwash sand cap. Model test two contained 4 feet of tire chips with a 1 foot outwash sand cap. Model test three contained 5 feet of tire chips without any soil cover.

The plastic displacement of all model tests, except for model test two, were less than 0.5 inches. The completed results of the model test showed that the layered system exhibited a nearly two-fold increase in stiffness compared to model test one, which administered the most desirable engineering properties of the three other model tests. Model test three, pure tire chips, was only loaded to 50 percent of the load of the other model tests due to substantial cyclic displacements which were beyond the testing machines measurement capabilities. The soil cap layer is placed in this layered system for the same reasons as in the case of pure tire chips.

Soil and Tire Chip Mix. The use of a tire chip and a soil mix is another system that has been considered in the model bridge study. It is important to note that one of the parameters to consider when analyzing the engineering properties of rubber tire chips is the tire chip to soil ratio. It has been found that a change in the tire chip to soil ratio dramatically changes the compression characteristics of the sample material. Edil and Bosscher (1992) have determined that at a vertical load of 2800 pounds, a displacement of about 5 percent occurs in pure sand while a 14 percent displacement occurs under the same load in pure rubber tire chips. In research conducted by Edil and Bosscher (1992), Engstrom and Lamb (1994), Ahmed (1993), and Manion and Humphrey (1992), it was determined that a sand content greater than 40 percent significantly reduces compressibility from 30 to 40 percent to less than 20 percent as seen in figure 5.2. An analysis done on the relationship of mixing tire chips with cohesive soils indicates that the unit weight increases with an increase in clay to chips ratio. In general, the use of rubber tire chips and soil mix reduces post-construction settlement and increases confining pressures in the fill material (Ahmed and Lovell, 1992).

Shear Strength. Research has shown that shear strength governs the stability of the fills in backfill materials. Rubber tire chips have high friction angles based on the angle of repose of the tire chip piles. These angles of repose range from 37 to 43 degrees for loose tire chips and as high as 85 degrees for compacted tire chips. These higher friction angles imply that tire chips exhibit higher friction characteristics when compared to soil. An addition of more than 10 percent of rubber tire chips in a soil results in a shear strength greater than dense sand at low or moderate normal stresses (Edil and Bosscher, 1992). It has been determined that randomly mixed tire chips can reinforce sand to a strength greater than the strength of pure sand at its densest state while resulting in a somewhat lighter fill material. The reinforcing characteristics come from the tire chips that lay across the shear plane. In randomly mixed materials of rubber tire chips and

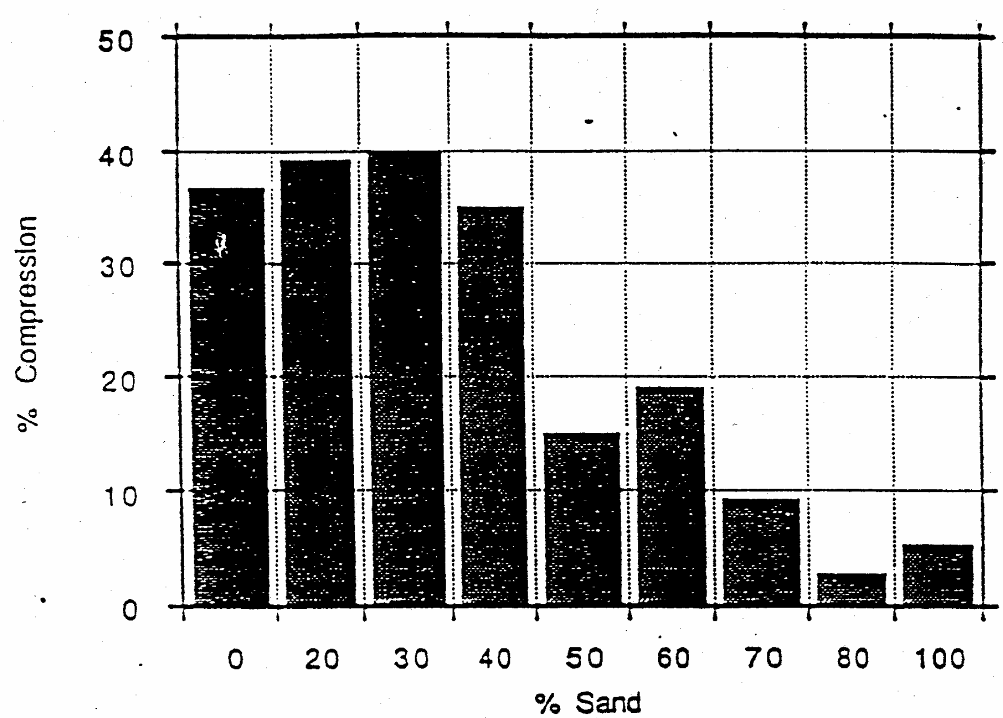


Figure 5.2 Compression vs. sand content of tire chips and sand mixtures (Edil and Bosscher, 1992)

sand, it is more likely that more tire chips will lie across the shear plane, improving strength characteristics of the backfill.

In research conducted by Ahmed (1993), shear strength of rubber tire chips as a fill was analyzed for pure tire chips and a mixed configuration of rubber tire chips and soil. It was determined that the increase in tire chip to soil mix ratios increases the strain at failure and yielded higher maximum deviatoric stresses up to a tire chip to soil mix ratio of about 40 percent. The deviatoric stress drops significantly if the tire chips to soil mix ratio is increased beyond 40 percent. This indicates that a sand fill with tire chips will show a maximum benefit from the reinforcing properties of tire chips at low to medium confining stresses. Samples of tire chips do not fail by yielding and do not have a single shear plane. Instead, samples exhibit strain hardening and become stiffer with increased loading. The tire chips tested at low confining pressures demonstrate symmetrical bulging while samples subjected to high confining pressures compress vertically with little lateral spreading (Ahmed, 1993). It has been determined that for a given tire chip to soil ratio, confining pressure is the most important factor affecting the shear strength of rubber tire chips, not the size of chips and compactive effort. However, a small correlation has been found that relates an increase in stresses with an increase in chip size and compactive effort (Ahmed, 1993). The increase in strength that the rubber tire chips and soil mix provide occurs when there is a maximum chip to soil ratio of about 39 percent, which is considered to be the optimum mix ratio and is similar to the results that Edil and Bosscher (1992) and Manion and Humphrey (1992) have observed in their research.

<u>Compaction Behavior</u>. The most significant factor that controls the compaction behavior of rubber tire chips is the soil to chips ratio. In tests performed with only rubber tire chips, the unit weight of the compacted material remained much lower. The size and moisture content of the rubber tire chips were found not to be significant factors in the density obtainable except in the case where vibratory compaction is used. This is largely due to the smaller chips ability to be compacted into a denser configuration than larger chip sizes. The densities obtainable using vibratory compaction decreased with an

increase in the chip size, but is not effective with chip sizes larger than 0.25 inches (Ahmed, 1993).

Pure Tire Chips. Typically, the density of soil increases with an increase in compaction effort. In compaction tests performed by Ahmed (1993), it was determined that in the case of pure tire chips, density actually decreased with an increase in compactive effort from standard to modified proctor compaction testing. The compactive effort was then decreased by 50 percent and the test results were similar to the standard proctor compaction method. It was determined that some compaction is necessary, even though the optimum density can be achieved with a modest compactive effort.

Tire Chips with a Soil Mix. The density of tire chips with a soil mix decreases with an increase in compactive effort and the chip size does not significantly affect compactive effort. This is similar to the results obtained from tests on pure tire chips. Although not substantial, there was a trend in the test results that shows an increase in density with an increase in the size of chips.

Methods of Compaction. Considerable problems have not occurred during field compaction of tire chips. Typically, a backhoe may be used to mix the soil and rubber tire chips to a uniform distribution. Tracked equipment has been found to be most appropriate for compaction of materials, especially where rubber tire chips with large quantities of wire material are protruding from the chips surface. The wires have been found capable of penetrating rubber tires in compaction equipment. Non-vibratory methods for compaction are more appropriate for compacting chips alone and mixes of chips and fine-grained soils (Ahmed and Lovell, 1992). Laboratory tests show that there may be some advantages to using tire chips with a large quantity of protruding wires as retaining wall backfill due to the lower K_o values obtained (Humphrey *et al.*, 1993).

Resilient Modulus. The resilient modulus of materials defines their recoverable deformation response under repetitive loading. Due to excessive sample displacement and distortion, pure tire chips have not been tested. Data obtained by the Indiana

Department of Transportation (Ahmed, 1993) exhibited the decrease in modulus values with an increase in tire chips to soil ratios. A large reduction in the resilient modulus occurs in sand samples when mixed with tire chips. It was determined that the reduction was due to the level of confining pressures, deviatoric stresses, and percent of rubber tire chips in the mix. The modulus decreases with an increase in chip to soil ratio and can be as high as 80 percent or greater (Ahmed, 1993). Chip size was found not to significantly affect the resilient modulus characteristics of rubber-soils.

Hydraulic Conductivity. The hydraulic conductivity of rubber tire chips has been found to be very high. Edil and Bosscher (1992) conducted a test using 10 inches of tire chips under a driving head of 5 inches. The resulting flow rate was just slightly lower than the rate of flow through the permeameter without a specimen. Hydraulic conductivity under no confining pressure was found to be on the order of 0.00328 ft/s.

In research performed by Ahmed (1993), the values for permeability for pure tire chips are equivalent to values for coarse mineral aggregate. These high permeability values make tire chips a good candidate for drainage layers in pavement systems.

Environmental Impact. In tests conducted by Edil *et al.*, (1990), Extraction Procedure (EP) toxicity test and the Toxicity Characteristic Leaching Procedure (TCLP) test were conducted for barium, cadmium, chromium, lead, and mercury, but not for arsenic, selenium, or silver. Test procedures were followed for evaluating the leaching behavior of metals, anions, and organic and inorganic indicator parameters. Environmental impact studies show that shredded automobile tires are not a hazardous material. The tires showed some, but very little release patterns for all substances. Concentrations of these detectable substances declined with continued leaching for most substances. Therefore, it was concluded that shredded automobile tires show no likelihood of having adverse effects on groundwater quality (Ahmed and Lovell, 1993). The Minnesota Pollution Control Agency (MPCA), has developed guidelines for using tire chips in Minnesota as a lightweight fill material (Ronchak, 1990). The MPCA has recommended using rubber tire chips only in unsaturated zones and where the road surface is designed and built to provide for adequate surface drainage to avoid water seepage through the road surface and the tire chip material. A geotextile fabric is also recommended to be placed above and below the shredded tire material to keep the material together and to prevent surrounding material from migrating into the tire chip fill. A low permeability material must be placed over the tire chips as well to prevent surface water from penetrating into the rubber tire chip fill material (Engstrom and Lamb, 1994).

Summary. Compression tests on rubber tire chips indicate that tire chips are highly compressible upon the first loading cycle but decreases during subsequent loading. Unit weight of soil-tire chips mixtures is controlled the most by the percentage of soil in the mixture and somewhat by soil type. A trend of increasing density with increasing chip size has been found, while other factors such as water content and compactive effort are insignificant. Normal construction machinery can be used with the placement of rubber tire chips except for machinery with rubber tires that can be punctured by exposed wires. Upon initial loading, tire chip-soil mixtures exhibit large plastic deformations as high as 40 percent of the initial placement thickness for pure tire chips. It was determined that clay mixtures have a lower resilient moduli than sand mixtures at the same mixing ratios, and the resilient modulus of soils decreases with an increase in tire chip to soil mix ratios. Tire chips with a 3 foot soil cap used instead of a 1 foot soil cap perform more effectively for a lightweight fill application. Tire chips have high friction values which improves the strength of sand mixtures. A 40 percent chip mix by weight is an optimum value for the quantity of rubber tire chips in a tire chips and soil mixture where large settlements are a concern. This tire chip to soil mixture will produce a dry unit weight of about two-thirds that of soil alone. When confined, tire chips have a hydraulic conductivity of more than 0.00328 ft/s and 0.000328 ft/s, or more, can be encountered in normal highway applications (Edil and Bosscher, 1992). Hydraulic conductivity and compressibility can be reduced by mixing sand with rubber tire chips to 30 to 50 percent by volume. At this amount of a sand mixture, the soil begins to act more like sand. Tire leachate data indicate little or no likelihood of shredded tires having adverse effects on ground water quality (Edil and Bosscher, 1992). The benefits of using rubber tire chips as a fill material include depleting the large number of rubber tire stockpiles, reduced weight of fill, reduced settlements, preventing development of pore pressures during loading of fills, substitution of conventional permeable materials for subdrainage, providing separation of underlying weak or problem soils from mixing with subgrade and base material, conservation of energy and natural resources, and allowing the use of a substitute material for the more expensive conventional fill material. Potential problems with the use of rubber tire chips as a backfill material are the possibility of leachate of metals and hydrocarbons, fire risk, and the large compressibility characteristics that rubber tire chips exhibit.

5.2.6.4 Polystyrene Foam

The use of a plastic polystyrene foam material as a type of geosynthetic product called geofoam has increased significantly in North America during the last few years. However, geofoam is not a new product used in engineering applications. The use of foam materials in construction has been used since the 1960's in other areas of the world. Products such as Expanded Polystyrene (EPS) and Extruded Polystyrene (XPS) have been the primary geofoam products used in engineering applications. There have been many developments in geofoam technology and there are numerous applications. Such applications related to bridge approaches include geofoam as a lightweight fill material, as a compressible inclusion, and for fluid transmission.

Lightweight Fill. Over 24 years of experience with EPS as a lightweight fill material have brought about both a wider use on a global scale and the introduction of many alternative design applications. EPS is the lightest of all available fill materials, for it is non-biodegradable and is easily placed with the use of little or no machinery. Reduced vertical loads have typically been the primary application of EPS in lightweight fill construction. Additional engineering benefits include reduced horizontal loads, simplified designs, and increased speed and ease of construction. EPS has had limited use in the United States as a lightweight fill, but Norway and Finland have extensive experience with using EPS in construction over numerous wetlands and peat bogs. In 1972, the Norwegian Public Roads Administration adopted the use of EPS as a superlight fill material in a road embankment. This project involved the reconstruction of road fills adjacent to a bridge constructed on a piling system embedded on firm ground. Before reconstruction of the embankment, fill materials resting on a 10 foot thick layer of peat above 33 feet of soft marine clay, experienced a settlement rate of more than 8 inches per year. After replacing 3 feet of ordinary fill material with two layers of EPS blocks, the settlements were successfully halted (Frydenlund and Aaboe, 1996).

The reason why lightweight fill material has been a popular geosynthetic function of block-molded EPS since 1972 is that EPS has a density that is only about one percent of normal soils. Therefore, it offers the potential for dramatic vertical stress reduction on subgrades when EPS is incorporated into earthwork. Projects using EPS blocks have generally been used as ultra-lightweight fill and have been used in many states in the United States as well as in the provinces of Canada. There has been an increasing use of EPS blocks on commercial projects as lightweight fill around a variety of structures. With regard to the stress-strain response in compression, a two inch cubic specimen of geofoam is linear elastic up to one percent strain and then is plastic out to 10 to 20 percent strain (Horvath, 1996).

Compressible Inclusion Applications. The most prominent geosynthetic use of EPS geofoam is as a compressible inclusion. In general, a compressible inclusion is any material that is significantly more compressible, at least in one direction, than other materials that it is in contact with. In geotechnical applications, a compressible inclusion is usually placed between a below ground structure and the surrounding ground. The geofoam acts as the most compressible component of the structure-inclusion-ground system, therefore the inclusion will deform more readily than the other system components under an applied stress or displacement (Horvath, 1996). Much like hay bales, geofoam may be used to promote arching in soils over culverts. Similarly, cardboard has been used behind retaining structures to promote mobilization of shear strength in soils. Geotechnical applications for a compressible inclusion include behind earth retaining structures; around foundation elements; and above pipes, culverts, and tunnels. Using a compressible inclusion can result in significant reduction in earth pressures under static and dynamic loading. A compressible inclusion can be used to accommodate ground or structure movement and on a number of different applications to reduce vertical and horizontal earth pressures acting on structures and soil material. This is done by designing a system in which the EPS geofoam material is used as a compressible zone. The effectiveness of this type of application can be measured by the ability of the EPS geofoam to conform to the stress strain response needed for a given application. This geofoam function is one that has seen tremendous research in the past and is continuing in order to develop improved product performance.

The advantage that geofoam inclusions offer is that there is a significantly smaller load on a structure than if there is no inclusion present. A geofoam inclusion becomes a design option when a compressible inclusion is more cost-effective alternative than designing the structure to withstand the greater load. This has been proven to be a most valuable application when rehabilitating an existing structure that has exhibited stress or upgrading an existing structure to resist loads larger than those for which it was designed for originally.

Applications Involving Earth Retaining Structures. In instances for applications behind earth retaining structures, the use of compressible inclusions can offer benefits in a variety of ways. One method is by reduction of lateral earth pressure by shear-strength mobilization. This can occur as a result of controlled yielding in unreinforced soil or by controlled yielding in reinforced soil, as well as other less popular methods. A design of a compressible inclusion called controlled yielding can be created within the retained soil. This generally means that if the retaining structure is rigid and non-yielding, the compression of the inclusion will allow the retained soil to deform to achieve the active state. This use of a compressible inclusion has been termed the “Reduced Earth Pressure (REP) Wall” concept. This concept of controlled yielding can be carried a step further by considering the case where layers of tensile reinforcement are included in the retained soil. In addition, a compressible inclusion can allow sufficient movement so that a largely self-supporting mechanically stabilized earth (MSE) mass develops and the lateral earth pressures on the retaining structure are reduced below the active state (Horvath, 1996). If proper combinations of stiffness, compressibility, and reinforcement are used, the lateral earth pressures can approach zero, even if the retaining structure is rigid and non-yielding. The use of compressible inclusions plus reinforced soil has been termed the “Zero Earth Pressure (ZEP) Wall” concept.

There are several situations where volume changes of earth materials are caused by the physical changes within the soil material and not as a direct result of deformations and shear-strength mobilization. These include swelling and freezing soils; rock that swell due to water absorption, mineral changes, or the release of tectonic stresses.

The movements of the structure must also be accommodated for when considering the distresses placed on soil materials in a compressible inclusion application. There are situations where the lateral displacement of an earth retaining structure is caused by external factors other than lateral earth pressures. This usually occurs in rigid, indeterminate structures that are subjected to high temperature induced strains. These situations include bridges, especially those with integral abutments; navigation locks; and water/wastewater treatment facilities (Horvath, 1996). In these cases, the movements of the structure can lead to lateral earth pressures in excess of at rest and approaching the passive state. In the past, it was appropriate to design these structures for these types of earth pressures. This resulted in repairs of structures due to inadequate design. A more cost-effective alternative is to use a compressible inclusion to allow the structure to move and yet transmit a reduced magnitude of displacement, or stress, to the retained soil. Recent research shows that the use of compressible inclusions plus tensile reinforcement (ZEP-Wall concept) can reduce settlement of the backfill behind an earth retaining structure or bridge abutment while reducing lateral earth pressures (Horvath, 1996). This may offer a possible way to eliminate the bump at the end of the bridge problem that occurs with highway bridges. If a reinforced soil mass and compressible inclusion are used behind a bridge with integral abutments, lateral earth pressures under a variety of loading conditions could be reduced, abutment displacement could be accommodated, and settlement of the approach fill can be reduced (Horvath, 1996). In these situations tensile reinforcement of the backfill soil is necessary. When the retaining structure pushes against the compressible inclusion permanent deformation occurs in the inclusion. When the structures pulls away from the inclusion, a gap forms subjecting the geofoam to the full lateral soil pressure and transient loads. By itself, geofoam can not provide external support to a vertical soil mass, therefore internal support of the soil is necessary through the use of soil reinforcement.

Geocomposite. Another recently developed product is a geocomposite based on geofoam. This is a geoinclusion and is available in Canada and the United States. It consists of a geosynthetic drainage composite, sheet drain or geonet, that is laminated to one face of a panel cut from EPS block. An advantage of the geoinclusion is that in most applications where compressible inclusions are used, fluid drainage is desirable parameter of design. The geoinclusion functions as both a compressible inclusion and a drainage layer. Another unique feature of this type of geoinclusion is the optional use of elasticized EPS for the geofoam component (Horvath, 1996). The elasticized EPS is block-molded and has been subjected to an additional manufacturing step to modify the stress-strain behavior of the EPS. The primary advantage of this is the increase in compressibility of the EPS under relatively low stress conditions.

The use of compressible inclusions in geotechnical engineering applications is not a new concept. The use of hay bales has been used since the early 20th century to induce vertical arching over pipes. Glass-fiber insulation and cardboard have been used in applications involving earth-retaining structures. The problems associated with using these materials are many, for their stress-strain behavior is unpredictable and uncontrollable. This is especially true of cardboard products if they absorb moisture. In addition, these materials are either too compressible or biodegradable. Decomposition of organic material will leave a void that can lead to raveling of overlying soil and the creation of a surface depression. Geofoam is the preferred product because it does not exhibit these problems.

Selection of a geofoam material depends on its engineering properties as well as cost and environmental factors. In applications using geofoam for compressible inclusions, stiffness of the geofoam in the primary direction of movement is the most important property. Stiffness is defined as the stress applied to the surface of the geofoam divided by the surface displacement of the geofoam in a direction parallel to the direction of stress application.

The use of geofoam in the past has revealed that a molded-bead EPS is the best material for uses as a compressible inclusion. Not only is EPS the least expensive geofoam material on a cost per unit volume basis, it can also be manufactured to have a stiffness significantly lower than any other geofoam material. It does not use any fluorocarbon-family gas, which can deplete the earth's upper-atmosphere ozone layer, and does not release any formaldehyde gas after installation.

The minimum density EPS block that can be produced depends on the manufacturing process but is usually 6.24 lbs/ft^3 or slightly less. The basic EPS product that can be used for a compressible inclusion is a full-size block or a panel cut from a block. The dimensions of a block vary depending on the mold size used by the manufacturer. Panels of smaller, arbitrary dimensions can easily be factory cut from blocks.

Fluid Transmission. Geofoam can be used as an active part of a system that provides fluid drainage of ground water and ground borne gases such as methane and radon. In these cases, the geofoam product is designed to function as a highly permeable part of the drainage system. This is done by using a permeable product such as polystyrene porous block, routing a system of channels through the geofoam panel or block, or creating EPS slabs or molded products with preformed channels or grooves. Geofoam base drainage products are not widely used at this time, but there is a potential for greater use when it is more widely recognized that the geofoam can be designed to provide other functions in addition to drainage.

Summary. Geofoam products provide an alternative to many engineering applications, which include applications as a geoinclusion for integral bridge end treatment. Geofoam can be used in a variety of different applications that may inherently provide engineers with a low cost alternative while providing engineering functionality that is both beneficial and cost effective. Geofoam products will be used more in the future as a more diverse product is developed and more applications are discovered.

5.2.7 Summary of Previous Research

The review of literature has revealed that void development under bridge approaches have received a considerable amount of attention from transportation officials for several years. The causes of void development have been identified and several designs have been instigated to alleviate the problem. While the interaction of the cause and effect are highly complex, the following conclusions can be stated based on previous research:

1. Integral abutment bridge performance has been studied and the effects of thermal-induced movements on the backfill have received considerable attention.
2. The mechanisms responsible for void development under the approach slab can be attributed to the thermal-induced cyclic movements of the integral abutment.
3. Alternative backfill materials to relieve lateral loads have been researched extensively.
4. Backfill designs incorporating the use of shredded rubber tire chips has been researched and rubber tire chips have shown to contain engineering characteristics of a backfill material.
5. Backfill designs incorporating the use of a polystyrene foam material as a geoinclusion have been performed, however tensile reinforcement is required in the backfill.
6. Backfill designs incorporating the use of a fabric reinforced soil wall behind the abutment has been performed with good results.

The work reviewed provides a background for the field studies and the model study.

5.3 Model Study

5.3.1 Introduction

The primary goal of the model study was to simulate the thermally induced movements of an integral abutment while analyzing and comparing the effects of the cyclic movement on a composite backfill. These results could then be compared to the results obtained in a previous model study performed by Schaefer and Koch in 1992.

There were several purposes for using a model to simulate thermal effects. The primary purpose was to control the conditions of soil type, construction sequence, construction compaction, and structure behavior. Controlling these parameters allowed for repeatable results that paralleled field thermal conditions. Instrumentation was performed with ease by using a model. The costs involved with instrumenting a structure in the field would have far exceeded the costs of instrumenting a model. A previously constructed model bridge in close proximity to South Dakota State University also contributed to decreased costs. In addition, the work involved proceeded with no interruption to highway traffic flow as would have occurred if a new structure was used.

A jacking system was used to move the bridge against the backfill in order to simulate the thermal movements of the bridge system. Access holes in the approach slab allowed for the voids in the backfill to be measured with precision and ease. Inclinometers in the backfill area provided determination of lateral and longitudinal expansion. Load cells were used on the abutment to determine earth pressures and compaction effects due to long-term cyclic movements.

5.3.2 Description of the Site and Existing Conditions

The structure used for this model test and the previous model test had been built for a 1973 SDDOT research project titled, “Analysis of Integral Abutment Bridges” (Lee and Sarsam, 1973). This research focused on the effects of thermal expansion and contraction of the bridge system. The model bridge structure is located in the Brookings Area SDDOT yard east of Brookings, SD. The availability of this structure made testing an integral abutment bridge system possible while simulating full-scale movements. The bridge structure consists of a partial bridge section, jacking abutment, and integral abutment approximately 19 feet wide and 46 feet in length (Fig. 5.3). The abutment is approximately one-half width and is full depth and pile supported to a depth of 32 feet. The bridge structure is oriented east and west with the west abutment containing the materials needed to support the hydraulic jack equipment. The east abutment was constructed to act as the integral abutment and is pile supported. Additional details concerning the model bridge structure can be found in Lee and Sarsam (1973). The bridge section was modified in the 1992 study by Schaefer and Koch to accommodate larger jacking equipment, but was not altered for this model test.

Soil borings at the site showed a 6-inch layer of black silty sand overlying a 6-inch layer of gravel. A light brown sandy clay is present at 1 to 9 feet. The next 5 feet consists of light brown silty clay. A 6 inch layer of sand was encountered at 10 feet and groundwater is present at a depth of 6 feet. A light brown silty clay is present to depths

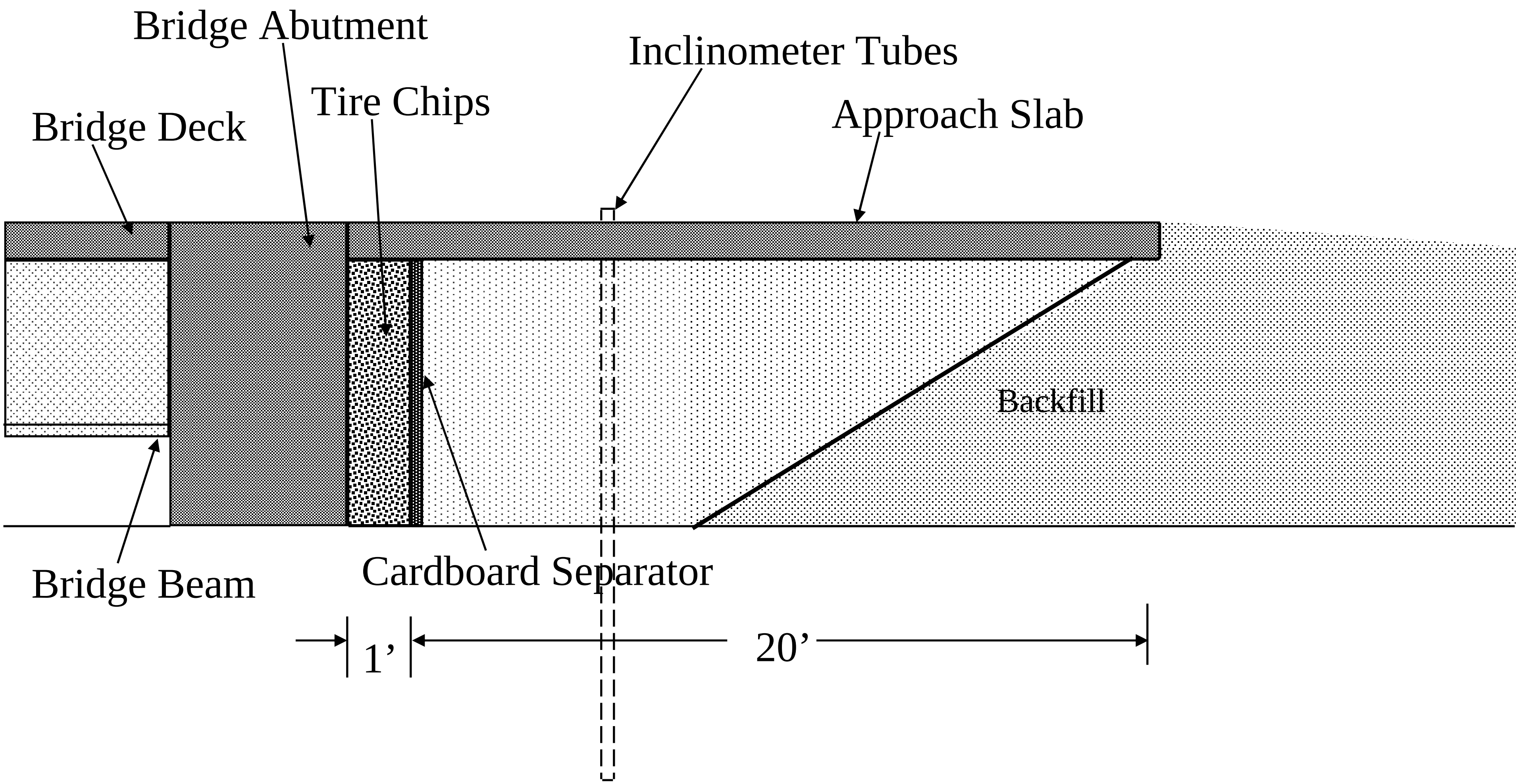


Figure 5.3 Constructed model bridge backfill system using a 12 inch layer of rubber tire chips

of 31 feet overlying a stiff mottled blue-brown clay to depths of 55 feet (Bump, 1970). The site soils are glacial till materials with the upper zone being weathered till.

5.3.3 Backfill Design

Research prior to the implementation of the model test had revealed that the use of lightweight fill materials in approach embankments had not been reported. A review of engineering characteristics such as compressibility, shear strength, compaction behavior, resilient modules, hydraulic conductivity, and environmental impact identified shredded rubber tire chips as a likely candidate for use in an integral abutment backfill design application. An evaluation of previous research has shown that research in using rubber tire chips to date has primarily been in the area of lightweight fills on weak soils with little emphasis on the use of rubber tire chips in a vertical layer to absorb differential movement of the integral abutment. Due to the elastic capabilities of rubber tire chips, a design scheme using a vertical layer of rubber tire chips between the integral abutment and the select granular backfill material was adopted for use in the model study. Compression tests on rubber tire chips had shown that tire chips are highly compressible upon the first loading cycle but decreases during subsequent loading. It was anticipated that the elastic properties of rubber tire chips would allow for the lateral movements of the bridge abutments without affecting the select granular backfill. The rubber tire chips would essentially act as a cushion for the integral abutment to move into and away from the embankment with temperature induced movements, therefore not allowing the select granular backfill to be disturbed.

The SDDOT provided the shredded rubber tire chip material used in the backfill design. Shredded tire chip sizes varied considerably from one to six inches and protruding wires were evident in each chip.

5.3.4 Model Construction Sequence

Due to the uncertainty involved with the compressibility of the rubber tire chips provided by the SDDOT, a vertical layer thickness of approximately 24 inches of rubber tire chips was used. This layer was expected to compress upon compaction of the backfill directly behind the rubber tire chip layer. The layer of rubber tire chips was separated from the backfill material by using cardboard as a separator. This was necessary to assure that backfill material would not enter into and fill the voids present in the rubber tire chip layer.

5.3.4.1 Model Excavation

The bolts holding the individual sections of the approach slab were removed in order to remove the approach slab sections away from the embankment. A large front-end loader was used to remove the individual approach slab sections. After the approach slab sections were removed, embankment soils behind the wing wall sections of the abutment were removed with a large front-end loader. After the embankment material behind the wing wall sections of the abutment were removed, a small front-end loader was used to remove the select granular backfill material in the center of the embankment. This was accomplished by driving the small front-end loader on top of the approach embankment and digging down to the bottom of the abutment between the inclinometer tubes. Most of the select backfill that needed to be excavated was removed in this fashion.

The select backfill material was completely removed from the abutment wall to 2 feet behind the inclinometer tubes. The backfill was then sloped up to the top of the existing embankment at about a 1H: 1V slope. The north inclinometer was excavated around barring no apparent damage. The pressure cells were not damaged during excavation as well. The excavation process took approximately a half-day to complete.

5.3.4.2 Model Construction

Construction of the Model Bridge began on September 18, 1996. A side view illustration of the constructed model bridge backfill system can be found in figure 5.3. Cardboard was placed at a distance of 24 inches from the abutment wall. Wooden stakes were driven into the ground on the backfill side of the cardboard to provide stability while placing the rubber tire chip layer. After the rubber tire chips had been placed, an attempt was made to compact the rubber tire chips with a jumper packer. The rubber tire chips could not be compacted in this manner due to the elastic nature of the rubber tire chips and the dynamic nature of the compactor. A vibratory compactor was considered as an alternative but could not be used in the space available.

Select granular backfill was then placed adjacent to the cardboard and compacted with a jumper compactor until sufficient compaction had occurred. Heavy grade cardboard was then placed between the select granular backfill material and the embankment material. Soil was then placed on both sides of the select granular backfill. The embankment material was then compacted in the same manner as the select granular backfill material and soil density determinations were taken with a rubber balloon density device. Densities were taken at approximately 60 inches from the face of the abutment wall and 36 inches from the inclinometer tubes. This was done at every 24 inches as fill was placed and compacted. The embankments are shown in figure 5.4 and the densities obtained can be seen in table 5.1.

More soil was added on the south embankment and compacted by acquiring two passes over the soil with the jumper compactor. Approximately 12 inches of loose soil had been added. Construction of the backfill system continued in this manner until construction was nearly completed.

Sample Number	Location	Depth	Dry Unit Weight
1	South Embankment	5 ft	117.03 lbs/ft^3
2	North Embankment	4 ft	123.42 lbs/ft^3
3	South Embankment	4 ft	127.25 lbs/ft^3
4	North Embankment	3 ft	148.59 lbs/ft^3
5	South Embankment	1 ft	123.24 lbs/ft^3

Table 5.1. Dry densities for the north and south embankment.

The approach slabs were successfully placed in their approximate positions with considerable difficulty using a large front end loader. A problem occurred placing the slabs into the anchor bolts in the bridge deck while fitting the inclinometer tubes into the

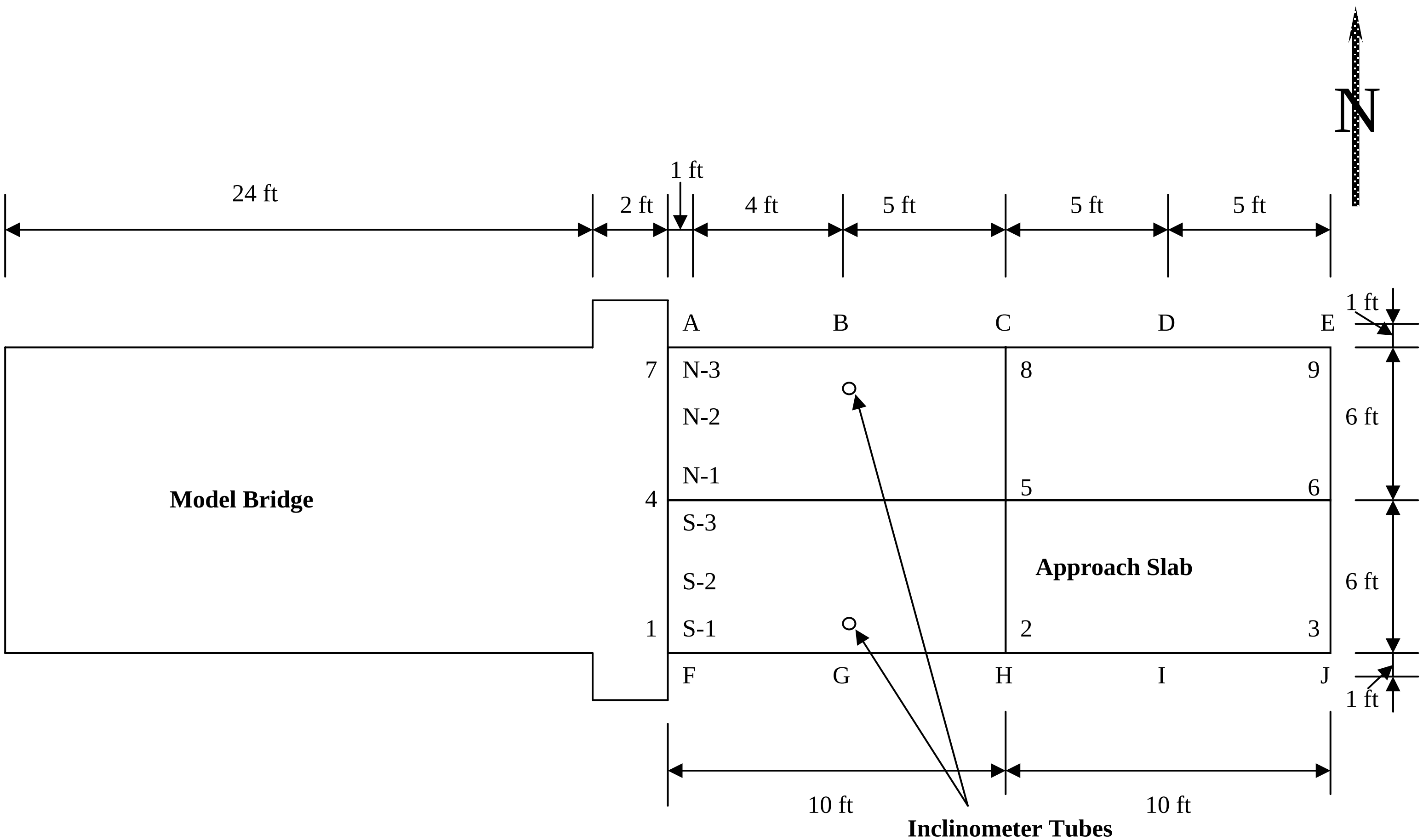


Figure 5.4 Instrumentation for monitoring the backfill, embankments, abutment, and approach slab. (Void development was measured through holes S-1, N-1, etc. Approach slab elevations were recorded from stations 1, 2, 3, etc. Embankment elevations were recorded on elevation stakes A, B, C, etc.)

appropriate holes in the slabs. The loader operator damaged the north inclinometer tube while placing the approach slab sections and the top 12 inches needed to be removed. It was then decided that the slabs would need to be generally placed in their appropriate positions, bolted together, and then pushed with a large front-end loader against the bridge deck.

On May 14, 1997, the remaining embankment fill material was placed and compacted near the wing walls on both sides of the model bridge. Most of the fill material was borrowed from the end of the approach system where additional material was available. Additional anchor bolts were drilled into the deck of the Model Bridge and concrete was used to fill a small void between the bridge deck and the approach slab. Elevation stakes were established around and on the approach slab to monitor vertical movement of the embankment fill material and the approach slab during testing.

5.3.4.3 Instrumentation

Several methods of instrumentation of the model test were implemented as seen in figure 5.4. Void development was measured under the approach slab, pressure cells measured earth pressures being exerted on the backfill, inclinometer tubes measured movement in the backfill, approach slab stations were used to measure the change in elevation of the approach slab, embankment stakes were used to measure elevation changes in the embankment, and abutment movements were monitored with a survey instrument.

<u>Void Development</u>. In order to monitor the development of a void under the approach slab, six holes previously bored through the approach slab along a line 1-foot from the abutment were used. These access holes allowed for the measuring of the development of a void directly behind the rubber tire chip layer. Measuring the void at the interface of the backfill soils and the rubber tire chip layer allowed for the measurement of the largest void that would develop. It was determined before testing that the largest voids would develop between these two dissimilar materials.

Earth Pressure Development. Earth pressures imposed on the backfill were measured with two total pressure cells. Two Slope Indicator Company Model 51482 total pressure cells with Model 514178 transducers had previously been installed in the center of the abutment wall at depths of two and four feet from the top of the abutment. The cells are flat pressure cells 9 inches in diameter and 0.43 inches thick. The pressure cells are constructed of stainless steel and are filled with fluid. This fluid allows for the soil pressure on the flat walls of the cell to be converted to fluid pressure and this pressure can then be measured by a pneumatic pressure piezometer. The standard pressure range of these cells is 1 to 300 psi. The cell pressures were read with a Slope Indicator Model 211 pneumatic indicator with a 0.15 percent digital readout gauge.

Inclinometer Movements. The lateral and longitudinal movements of the backfill were monitored with an inclinometer. The inclinometer instrument used was a Digitilt model 50309-E with a model 50325-E sensor made by the Slope Indicator Company and was supplied by the SDDOT. The Digitilt Inclinometer is a high precision surveying instrument for measuring subsurface displacement or deformation. The instrument was lowered down an existing grooved aluminum inclinometer casing which was approximately 5 feet away from the abutment and in line with the integral abutment piles. The tubes were previously placed in these locations in order to record pile movement effects on the backfill. Inclination readings were taken at frequent intervals of depth and were then converted to displacements. The system accuracy for a near vertical casing is $\pm$ 0.025 feet per hundred feet of casing or better. The borehole sensor is one inch in outside diameter and 33 inches long with cable connector attached.

Abutment Movements. In order to monitor the bridge abutment movements during loading, a survey instrument was used to establish a line of site. Graduated measurement plates were installed on the bridge deck on both sides of the abutment. This allowed for the movements of the abutment to be monitored with the survey instrument to ensure that equal deflections occurred on each side and followed cyclic movements that had previously been determined. This provided a check system to determine if both sides of the abutment were moving relative to one-another as well. The survey instrument was available from the Civil Engineering Department at South Dakota State University (SDSU).

Embankment Stake Movements. Stakes were driven into the embankment fill along the approach slab to allow measurement of the approach slab movement in the vertical direction. Monitoring locations were set up on the approach slab as well to enable approach slab movements to be measured with a surveying instrument.

5.3.4.4 Testing Methodology

Movement of the backfill system, embankment material, and approach slab were monitored in the same manner as with the previous model study performed by Schaefer and Koch (1992). Dial gages were not used in this test due to the ineffectiveness of using them in such close proximity to the integral abutment, as indicated in the past model test.

Loading and testing procedures for abutment movement cycles were also followed as in previous research by Schaefer and Koch (1992). A past review of temperature variations around the state had previously been made to determine the amount of abutment movement that should be induced to the backfill system. Data from around the state showed that the maximum variations in temperature experienced at different locations were very similar and therefore, the maximum temperature change data from Brookings had been used in computing the cyclic movements of the integral abutment. This data was then used to calculate movements corresponding to temperature changes with respect to the normal, high, and low temperatures. Coefficient of thermal expansion values of 6.5×10^{-6} in/in/°F and 6.0×10^{-6} in/in/°F were used for steel and concrete respectively. A 300 foot bridge was assumed an average length and the abutment movements were calculated for one-half of this length. An initial movement cycle of May normal temperature was assigned as zero and a pattern of movement was determined. Since it was desired to perform the model test for a one-year cycle of abutment movement, a one-day cycle of movement was developed for each month. Each cycle represented abutment movement from the normal, to the high, to the low of each month, and then to the normal of the next month.

Loading was performed with two 200-ton hydraulic jacks for expansion cycles and one 60-ton hydraulic jack was used for contraction cycles. Testing of the backfill system involved moving the abutment with the hydraulic jack equipment until the desired range of movement was accomplished for each cycle. Readings of the inclinometers, pressure cells, embankment stakes, void development holes, and the approach slab were taken at each step of cycled movements. Testing took approximately one month to complete.

General Observations. An initial movement cycle of May normal temperature was assigned as zero and a pattern of movement was determined. The 1-year pattern of movement used for testing can be seen in figure 5.5. Starting at 0.00 abutment movement, the first movement was a positive movement, or into the backfill simulating a thermal expansion period. The May high movement simulated was +0.5 inches and was achieved without incident. The backfill was then allowed to sit for a period of one hour and the pressures developed were monitored with the pressure cells behind the abutment. A pressure of approximately 2.7 psi on the top pressure cell and 3.6 psi on the bottom pressure cell developed after the abutment movement was completed. This remained consistent throughout the 1 hour equalization period. An initial equalization period of 1 hour allowed adequate time for movements in the rubber tire chip layer and the embankment soils to stabilize before recording instrumented data. This behavior differed from the past model test in that the pressures slightly decreased over a period of time. This decrease in pressure with time did not occur in this test primarily due to the layer of rubber tire chips between the backfill material and the integral abutment. The May low cycle was then performed and was a negative abutment movement, or away from the backfill simulating a contraction cycle. A May low cycle of abutment movement of –0.65 inches was performed by pulling the integral abutment away from the backfill. Pressures of approximately 1.0 psi on the top pressure cell and 1.1 psi on the bottom pressure cell were recorded. The contraction to the June normal cycle of abutment movement of +0.3 inches was then performed and the backfill was then allowed to equalize for a period of one hour. Again, the pressures remained constant for the entire

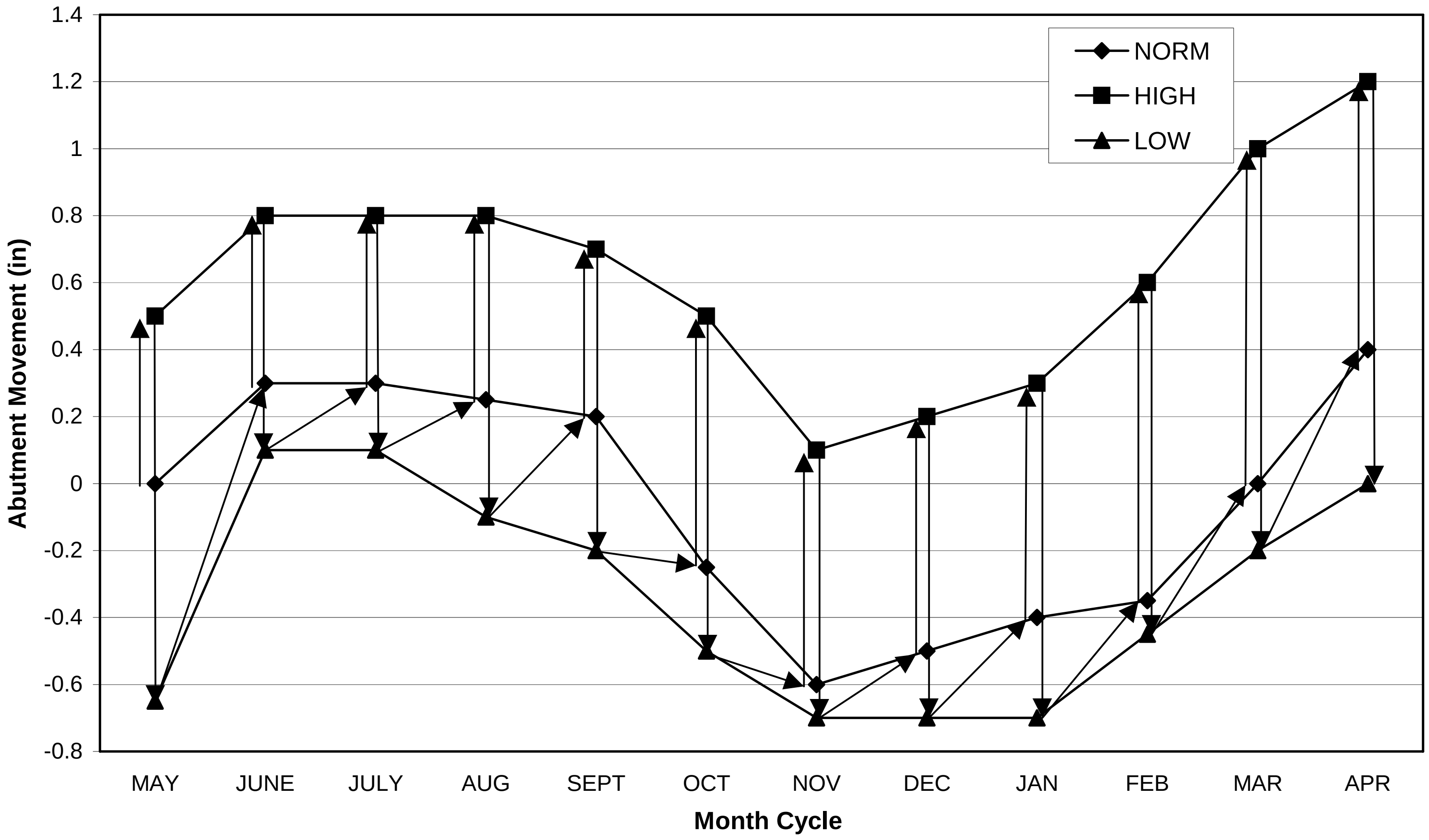


Figure 5.5 Monthly abutment movement cycles developed from temperature data for May 1990 through April 1991 to simulate a one-year cycle of abutment movement (Schaefer and Koch, 1992)

equalization period and it was determined that the equalization time could be reduced to under 20 minutes.

During the movement from the August high to the August low, a crack began to develop parallel to the abutment in the embankment fill. The crack developed on both sides of the approach slab and followed a line 12 to 19 inches from the wing wall down the embankment. It has been determined that this crack developed due to the eight inch thick layer of cover soil that had been placed over the rubber tire chips. As the abutment cycled into and away from the backfill, the embankment material on top of the rubber tire chips was compressed and contracted until this crack developed over the rubber tire chip/backfill soil interface. These cracks continued to become larger as more cycles of abutment movement were completed. The cracks on the August low to August high cycle varied in size from 0.25 inches to 0.5 inches.

5.3.4.5 Test Results

Past testing methods were adopted in order to allow for a comparison of results using the vertical layer of rubber tire chips with a backfill composed entirely of a select granular backfill. Selected test results are shown to illustrate important points. These results will be compared with past research whenever possible to provide a more complete analysis.

Void Development. Data from the model test using a vertical layer of rubber tire chips between the abutment and select granular backfill showed that a void became evident and increased over time. Data collected on the outside of the approach slab for the measurement of void development in the case for the vertical layer of rubber tire chips model test is located in figure 5.6. The data shows the void becoming progressively larger with each cycle of abutment movement. Data from holes in the center and at the edges of the approach slab revealed that void development near the outside of the approach slab was slightly larger than near the center, which developed a final void of approximately 1 inch on the outside. It has been determined that the contributing factors which allow for the development of a void under the approach slab, in the case where a rubber tire chip layer had been used, arise from the consolidation of

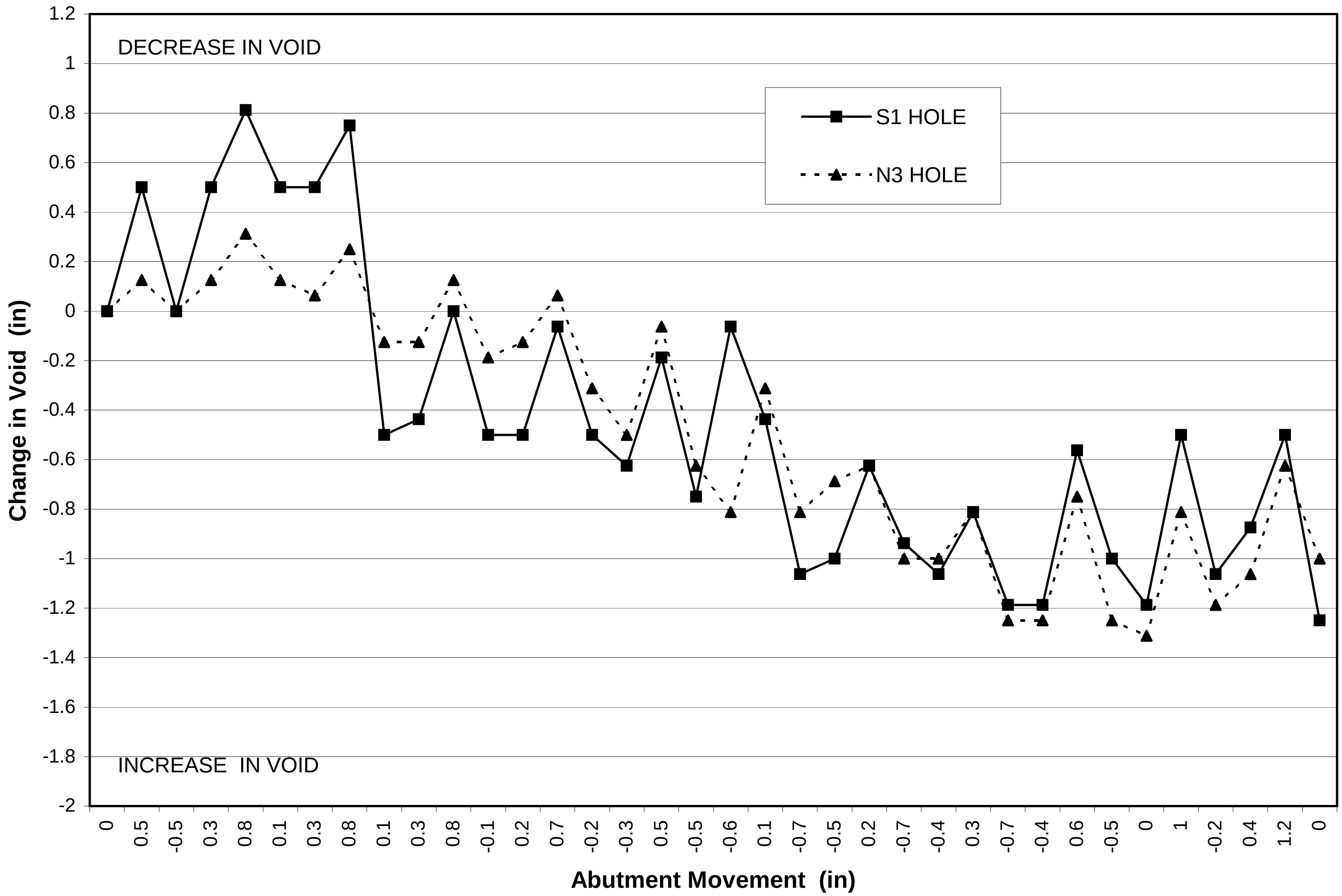


Figure 5.6 Change in void measured directly behind the cardboard separator in the backfill soil for holes S1 and N3

the rubber tire chips as the abutment was moved cyclically. This consolidation allowed movement to occur in the backfill in areas adjacent to the rubber tire chip layer, which produced a void under the approach slab. It is anticipated that while increasing compaction of the rubber tire chips, using a different type of rubber tire chip, or even using a smaller layer of rubber tire chips may reduce the amount of void development, some degree of void development would most likely occur and increase over time with each cycle of abutment movement.

Data from the model test not using a vertical layer of rubber tire chips showed very similar results but with slightly larger values for the total development of voids. This test revealed an increase in final void development of approximately 1.3 inches. Results can be seen in figure 5.7.

<u>Earth Pressure Development</u>. In the model test using a vertical layer of rubber tire chips, cyclic movement of the abutment revealed a relationship between abutment movement and changes in earth pressure, but at a low level. In general, pressures developed in the backfill corresponded to abutment movement as can be seen in figures 5.8 and 5.9. The larger the movement for a cycle, the larger the pressures that developed. The highest pressure of approximately 7.5 psi occurred on the bottom pressure cell in the April high cycle with abutment movement near 1.2 inches from initial. The relatively small pressure values indicate that the abutment movements did not induce significant stresses on the embankment soils. In general, the backfill was not substantially loaded, for the rubber tire chip layer absorbed the abutment wall movement.

In the case where the entire backfill was composed of a select granular backfill, data again clearly showed a relationship between abutment movements and changes in earth pressure, but on a much larger scale. Figures 5.10 and 5.11 show the relative pressures developed in this case. The highest pressure of approximately 56 psi occurred on the bottom pressure cell in the March high cycle with abutment movement near 1 inch from initial. Pressure cell response also showed that earth pressure redistribution occurred during abutment movement steps. A pressure decrease was evident during periods of delay after loading. This is due to a redistribution of stresses occurring in the backfill.

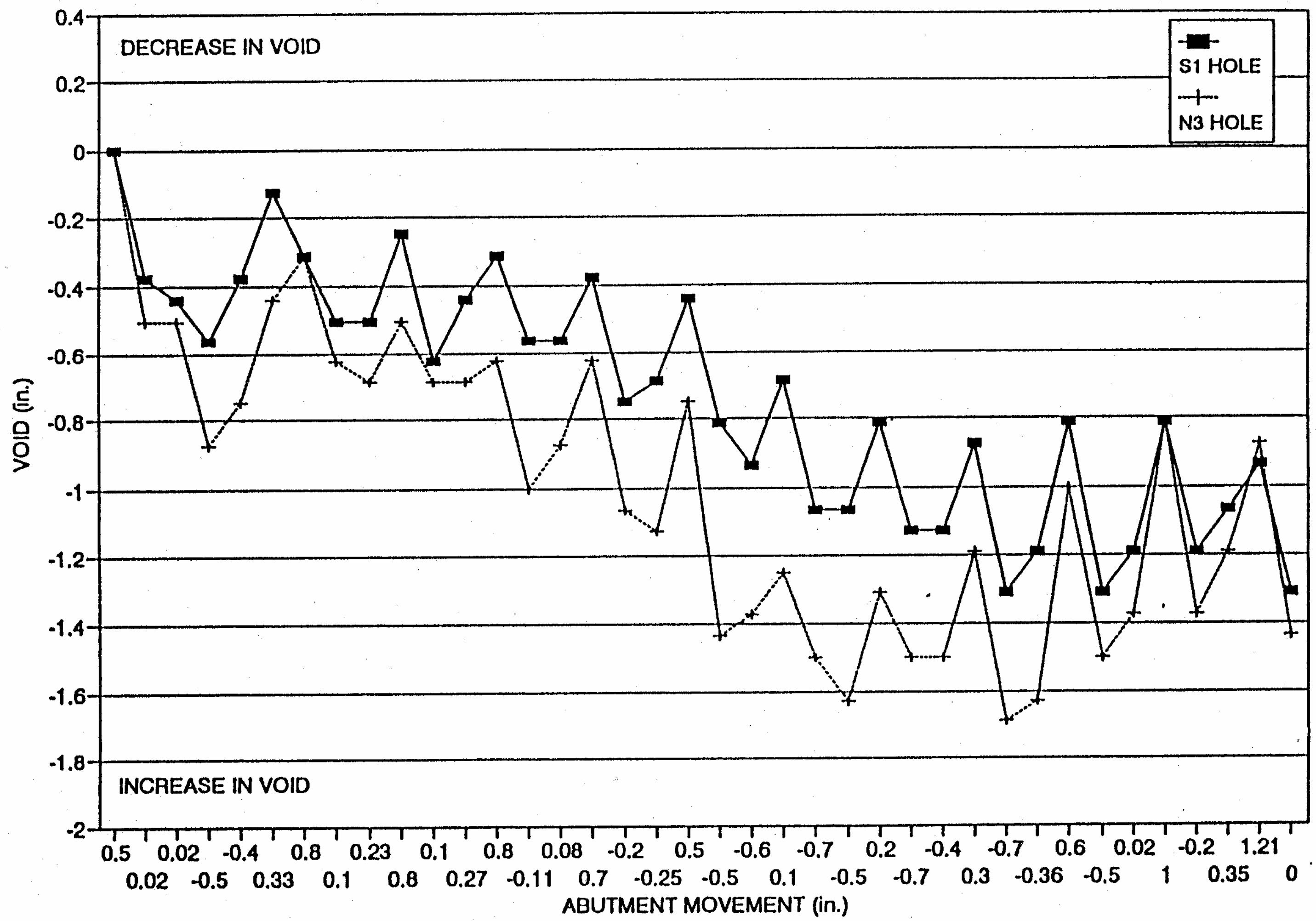


Figure 5.7 Change in void under the approach slab for holes S1 and N3 (Schaefer and Koch, 1992)

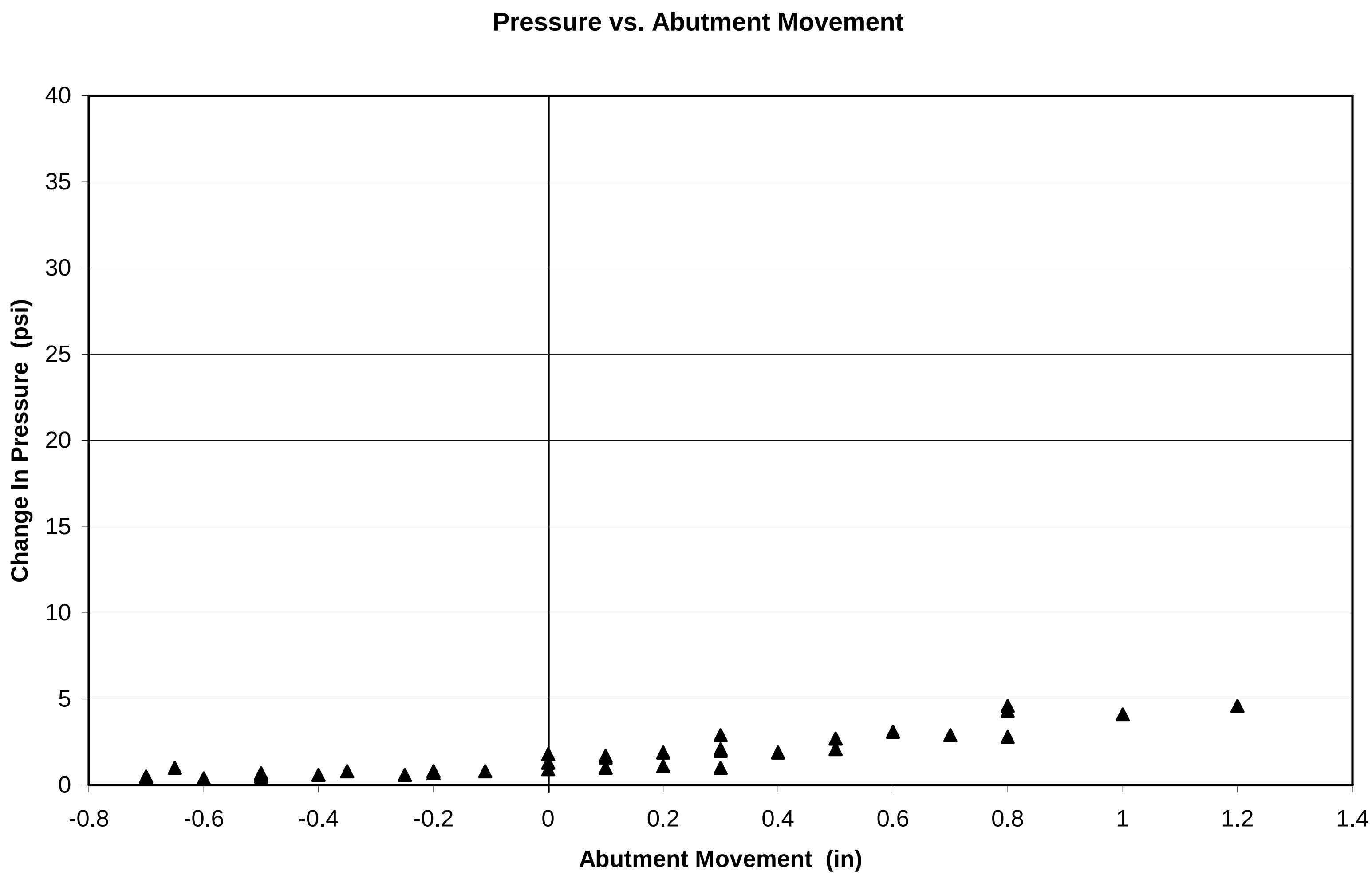


Figure 5.8 Change in pressure relative to abutment movement measured on the face of the abutment for the top pressure cell

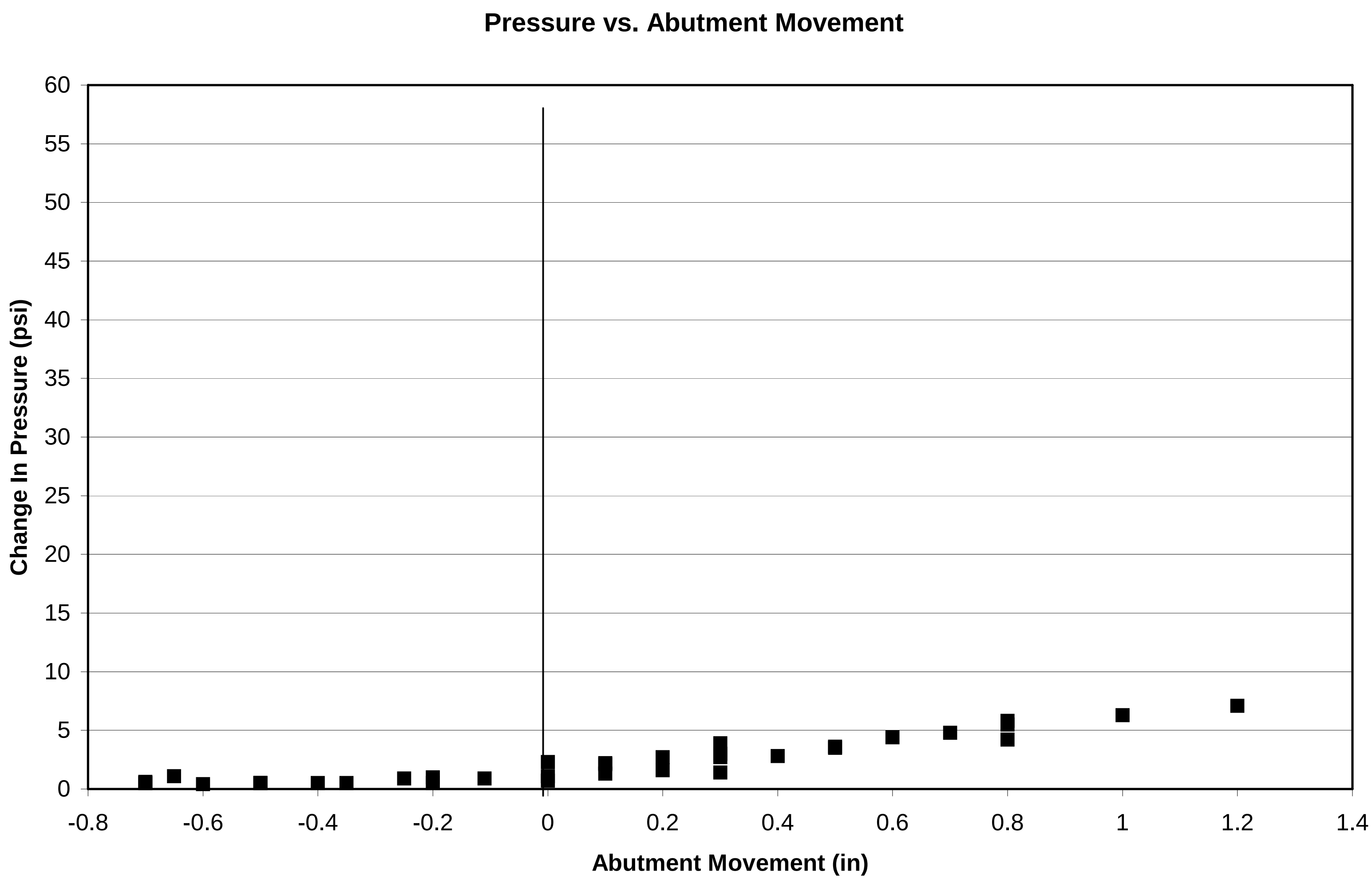


Fig. 5.9 Change in pressure relative to abutment movement measured on the face of the abutment for the bottom pressure cell

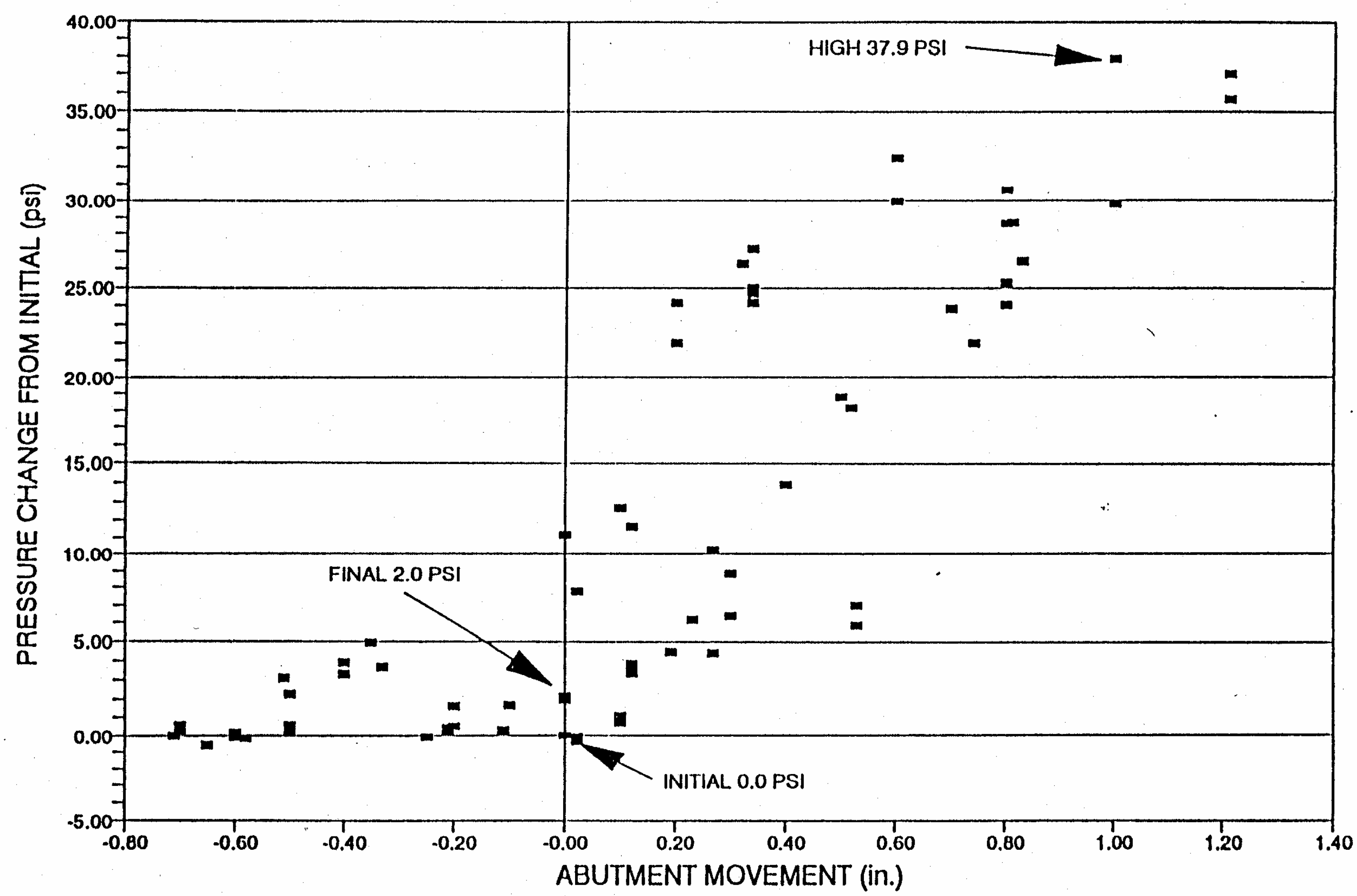


Figure 5.10 Change in pressure of top cell versus abutment movement (Schaefer and Koch, 1992)

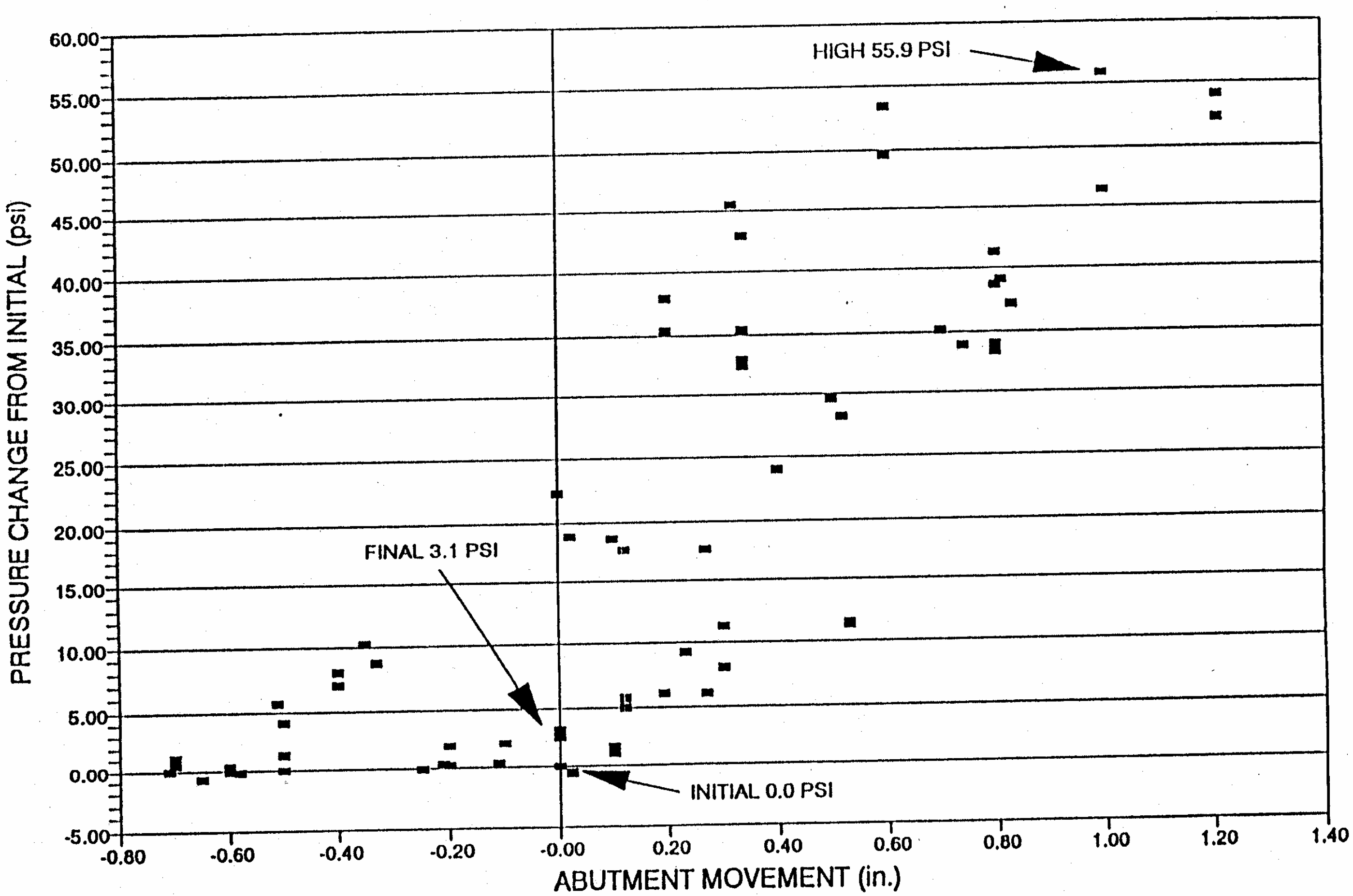


Figure 5.11 Change in pressure of bottom cell versus abutment movement (Schaefer and Koch, 1992)

Inclinometer Movements. Where a vertical layer of rubber tire chips was used in the backfill system, very little movements in the lateral and longitudinal directions occurred. Figure 5.12 shows an example of the longitudinal inclinometer data collected for each model test. The largest movements recorded in the backfill were less than 0.1 inches. In general, the backfill material did deflect in a small amount with subsequent cyclic abutment movement, but very little movement occurred. Deflection occurred more near the bottom, which indicates that the abutment supporting piles affected the subsoil underneath the backfill material more than the abutment wall moving into the backfill and embankment materials.

In the case of the backfill system composed entirely of select granular material, little movement occurred in the lateral direction while movements that are more substantial were evident in the longitudinal directions. The north inclinometer recorded a final lateral expansion of only 0.08 inches while the south inclinometer recorded slightly more with a final lateral expansion of 0.19 inches. Although these values are relatively small, there is evidence to support that the cycling of the abutment wall expanded the embankment outward.

In the cases for the longitudinal deflections, the north and south inclinometer had a final deflection of approximately 2.0 inches away from the abutment while the maximum deflections occurred after the April high abutment movement of 1.2 inches and deflection was measured to be approximately 2.6 inches away from the abutment. Data shows that longitudinal movements of the backfill cycled as the abutment expanded and contracted. It was evident that while mimicking large springtime temperature changes, significant additional cycle and permanent deformations occurred in the backfill and the embankment soils.

Approach Slab Movements. For the model test incorporating the use of a vertical layer of rubber tire chips, the elevation changes in the approach slab were small. Values in the range of 0.01 to 0.02 inches of vertical movement were common from recorded data. This seems to correlate with deflection data in the backfill where little movement in the backfill material was occurring. The degree of error in the surveying method used to record elevation change is approximately plus or minus 0.01 inches.

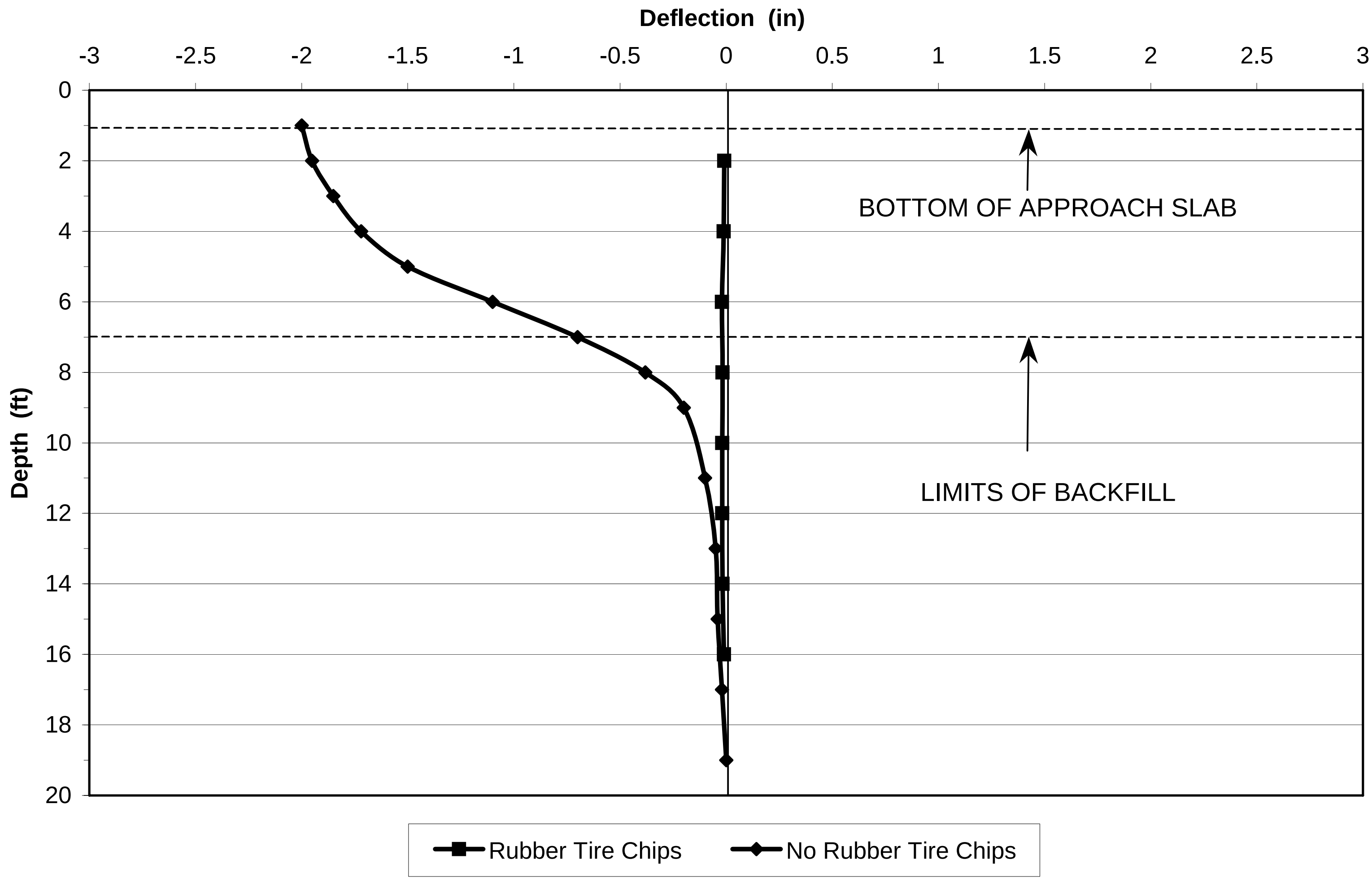


Figure 5.12 South Inclinometer data after the April low movement for the model tests performed with rubber tire chips and without rubber tire chips

In the case of a backfill containing entirely select granular backfill, trends developed in the data where approach slab center stations showed some elevation change with increases of 0.25 to 0.5 inches. This was attributed to the large abutment expansions where upward movement of the backfill occurs during expansion cycles. An apparent final change in elevation of about one-quarter inch occurred for the middle stations on the approach slab. This was calculated to represent approximately seven cubic feet of volume change in the backfill.

Embankment Stake Movements. The current model test using a vertical layer of rubber tire chips showed that very little movement of the embankment stakes occurred. Vertical movements in the order of 0.01 to 0.02 inches are evident. Again, this data shows that the layer of rubber tire chips absorbs a large portion of the induced movements from the integral abutment. Lateral expansion of the embankment material was not evident in this test.

The model test without a rubber tire chip layer showed that the stakes closest to the abutment displayed the most movement in both directions and moved cyclically with abutment movement. All movements were within one-half inch. In general, measured stake movements showed that the embankment soils expanded laterally during the test, but at relatively small amounts.

5.3.4.6 Discussion and Summary

Incorporating the use of a vertical layer of rubber tire chips between the integral abutment and the backfill system will inherently prevent longitudinal and lateral movements in the backfill while relieving relatively high earth pressures exerted on the backfill. Approach slab elevation data and embankment stake elevation data further support the conclusion that the layer of rubber tire chips greatly reduces disturbance of the backfill soil materials. In general, the tire chip layer greatly relieves passive earth pressures on the retained fill, nearly eliminates disturbance of the backfill as evident with the inclinometer tubes and elevation data. These findings support the evidence that a layer of rubber tire chips between the integral abutment and the embankment soils greatly reduces the mechanisms causing void development that were determined to exist in the past model study performed by Schaefer and Koch (1992).

However, it has become evident that consolidation of the rubber tire chip layer due to the cyclic movements of the integral abutment produces movement in the select granular backfill adjacent to the rubber tire chip layer. This movement allows a small void to develop under the approach slab in a similar, but to a smaller degree, as in the case where no rubber tire chip layer had been used. The layer of rubber tire chips behind the integral abutment reduces void development under the approach slab, but does not eliminate it due to effects not evident in the past model test. This effect being the rearrangement of the individual rubber tire chip pieces within the rubber tire chip layer. This rearrangement of the chips allows for a volume change to occur in the rubber tire chip layer, therefore allowing movement to occur in the adjoining embankment soils with each cycle of abutment movement.

5.4 FIELD STUDIES

5.4.1 Introduction

To counteract the mechanism of void development, changes to the approach system were needed. For the field study portion of this study, the approach system design was redesigned and consisted of a geotextile reinforced soil wall behind the integral abutment. This resulted in a vertical, self-contained wall capable of holding a vertical shape and forming an air gap between the abutment and the retained backfill. It was hypothesized that the gap behind the abutment would allow for the thermally-induced movements of the abutment to not affect the backfill. Three bridges were selected by the SDDOT for monitoring and were constructed using this design. In general, the field portion of this study consisted of monitoring the variations in gap width between the abutment and geotextile wall at various times of the year. One of the bridges included monitoring the development of a void under the approach slab as well.

5.4.2 Description of Backfill Design

5.4.2.1 General

The geotextile reinforced soil wall design implemented on these three bridges was adopted by the SDDOT from a WYDOT design and modified to comply with standard SDDOT bridge designs. Figure 5.13 shows the design detail of a typical geotextile reinforced soil wall behind the integral abutment prior to 1999. Figure 5.14 shows the typical design detail used as of 1999. The distinguishing feature of the embankment design is the 6 inch gap incorporated between the abutment wall and the face of the geotextile reinforced soil wall. This air void extends the full width of the driving lanes from the seat of the abutment to the bottom of the approach slab. A gap is installed at this location in order to allow the thermal movements of the integral abutment to occur without affecting the backfill materials. The purpose of the air void is to relieve the high lateral pressures exerted on the backfill soils and to eliminate movement of the embankment soil material, which occur due to the thermally induced movements of the bridge structure. This system is expected to reduce densification of the backfill behind the abutment, thereby reducing void development under the approach slab.

Other features of this design detail are the layers of wrapped geotextile that form the soil wall. These layers are individually filled with a Mechanically Stabilized Earth (MSE) bridge end backfill material and compacted. The construction of the soil wall in relation to the integral abutment, underlying soil material, and the approach system are illustrated in figure 5.13 as well.

5.4.2.2 Material Requirements for MSE Bridge End Backfill and Geotextile

The material requirements for the MSE backfill and the select granular backfill conform to the requirements described in Section 430 and 850 of the South Dakota Standard Specifications for Roads and Bridges (1998).

Section 831 of the SDDOT Standard Specifications for Roads and Bridges (1998) provides the SDDOT standard specifications related to the product requirements of a geotextile material and drainage fabric used in embankment and retaining wall reinforcement, as well as drainage and filtration. The geotextile used in the construction

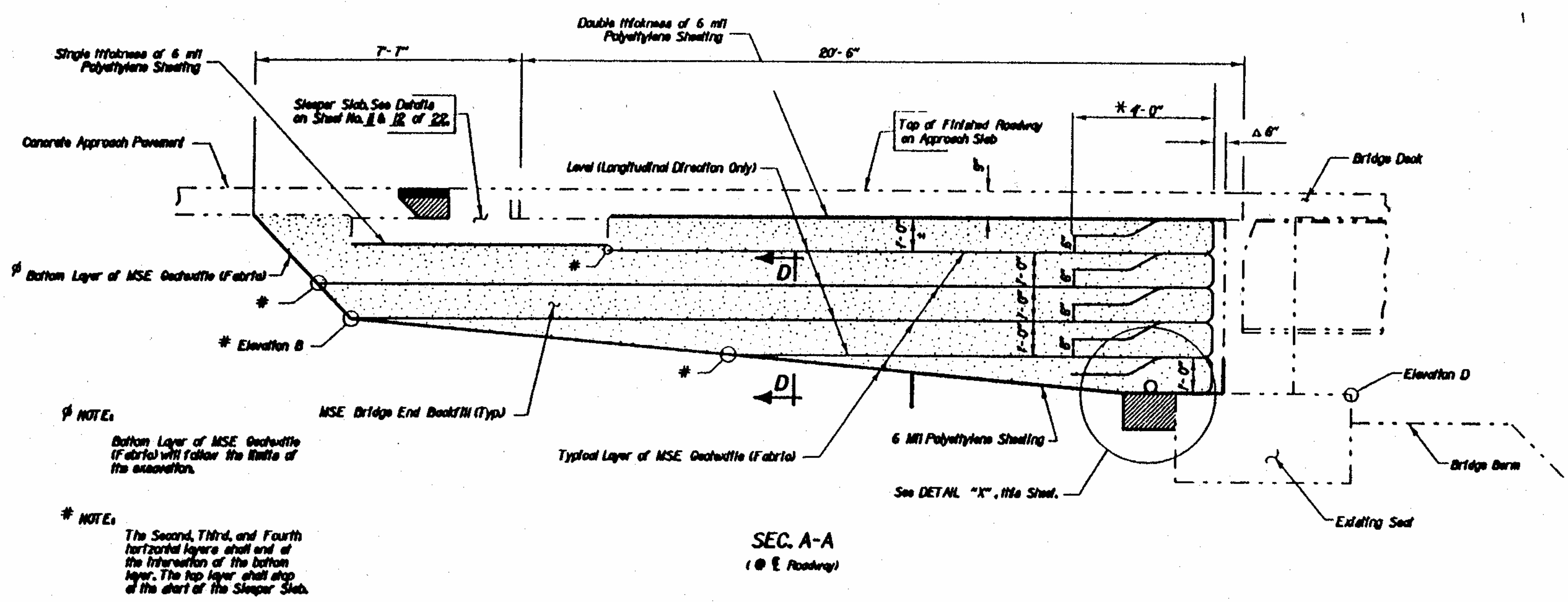


Figure 5.13 Design detail prior to 1999 of a typical geotextile reinforced soil wall behind the integral abutment

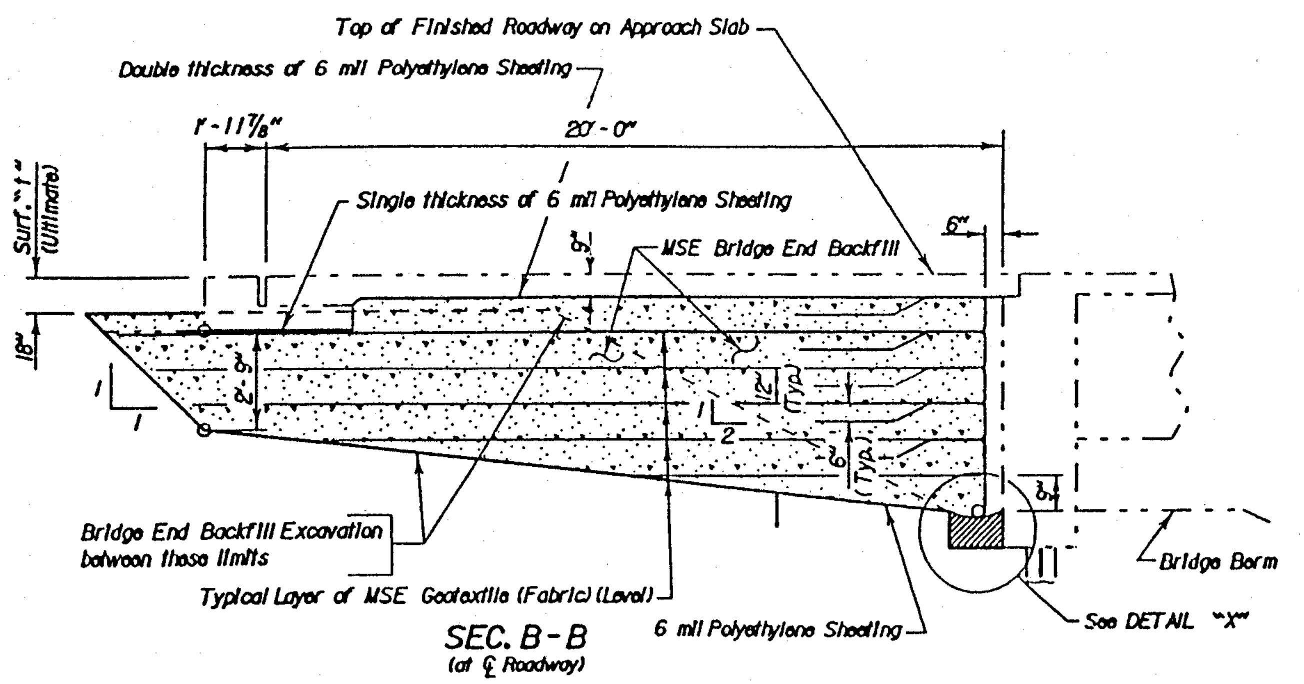


Figure 5.14 Design detail of a typical geotextile reinforced soil wall behind the integral abutment as of 1999

of the geotextile reinforced soil walls is a class I woven geotextile fabric which conformed to the requirements listed in table 5.2.

5.4.3 Description of Sites and Existing Structures

5.4.3.1 Highway 73 White River Bridge

A backfill design used by the WYDOT was adapted by the SDDOT for replacement of a single concrete bridge on Highway 73 across the White River 8 miles south of Kadoka, SD in the spring of 1996. This bridge was initially designed to contain the past SDDOT bridge end backfill design. A design change was made incorporating the use of a geotextile reinforced soil wall behind the abutment.

The bridge structure is composed of five concrete bridge beams and is 441 feet in length with a bridge span containing two integral piers and an integral abutment on each

Fabric Property	Test Method	Embankment & Retaining Wall Reinforcement	Drainage And Filtration
PERFROMANCE CRITERIA DURING SERVICE LIFE			
Equivalent or Apparent Opening Size, US Standard Sieve	ASTM D 4751	40-100	40-100
Permittivity, Sec-1	ASTM D 4491	0.0005	0.2
STRENGTH REQUIRMENTS			
Wide Width Strip Tensile Strength, lbs./in.	ASTM D 4595 (2)	200	40
Elongation at Failure, %	ASTM D 4595 (2)	35 Max.	40 Min
Burst Strength, psi	ASTM D 3786 (Diaphragm Method)	430	130
Trapezoid Tear Strength, lbs.	ASTM D 4533 (Any Direction)	75	25
Puncture Strength, lbs.	ASTM D 4833 (3)	110	25
Seam Strength, lb./in	ASTM D 4884	200	40
ENVIRONMENTAL REQUIREMENTS			
Mildew/Rot Resistance, %	AATCC 30 1988 (5)	100	100
Insect/Rodent Resistance, %	AATCC 24 1985 (5)	100	100

Table 5.2. SDDOT geotextile specification requirements (SDDOT Standard Specifications for Roads and Bridges, 1998).

end. The two driving lanes are 18 feet in width. The general soil conditions in the area are predominately sandy clay soils.

5.4.3.2 Interstate 29 Big Sioux River Bridges

A design incorporating the use of a geotextile reinforced soil wall was used on two bridges on Interstate 29 across the Big Sioux River 11 miles south of Brookings, SD. The south bound bridge was constructed in the fall of 1996 and the north bound bridge in the summer of 1997. The existing bridge structure was a continuous composite bridge with four steel bridge beams with expansion joints on each end of the bridge in front of the abutments. New design details incorporated for construction of these two bridges are the same. The bridges span approximately 398 feet and contain three piers with integral abutments. The two driving lanes for each bridge are 16 feet in width. The abutment backwalls, wingwalls, a portion of the expansion devices, and the top seats of the exiting structure were removed and the abutment and wingwalls were reconstructed to form the new portions of the integral abutments.

5.4.4 Approach Embankment Construction

The embankment design and construction was approximately the same for all three of the bridges monitored. Therefore, the north embankment for the north bound Big Sioux River Bridge will be referred to when discussing the construction and design details of the fabric reinforced soil wall behind the abutment.

After construction of the abutments had been completed, special attention was made in compacting the non-pervious backfill material that would serve as the foundation soil of the drainage system. Compaction around the wing walls was performed as well to produce the non-pervious foundation soil material directly behind the wing wall sections of the abutment. The underdrain system was installed prior to placement of the backfill. The underdrain system consists of a 4 inch diameter perforated plastic drainage tubing. This tubing was installed across the full width of the abutment structure on impervious soil and was screened at the outlet ends with a metal screen or grating to prevent the entrance of rodents.

Construction of the geotextile reinforced soil wall began by removing the remaining existing backfill material with tracked equipment and a front-end loader to 1:10 slope 20 feet behind the abutment. The excavation included all existing soil material the full width of each driving lane. Before placement of the backfill, the ground surface on which the backfill was to be placed was shaped to the established lines and grades, scarified to a minimum depth of 6 inches and recompacted to a minimum of 95 percent of maximum dry density as determined by SDDOT Standard Specifications for Roads and Bridges (1998) Section 104 (AASHTO T99). After compaction of the underlying soil material was performed, a six mil polyethylene sheeting was placed and attached to the abutment to a height of 1 foot with a construction adhesive. The 6-inch cardboard spacer was then placed against the abutment from the abutment seat to the top of the abutment just below the dowel bars protruding from the bridge deck.

Construction of the geotextile reinforced soil wall began by cutting the rolls of geotextile fabric into sections to completely cover the underlying soils from the face of the abutment to the required design length for the first layer. The rolls of woven geotextile were overlapped at the seams a length of 2 feet. Excess fabric (approximately 6 feet) was temporarily attached to the abutment and over the sides for embedment.

The soil material used for the backfill was a select granular backfill as specified in SDDOT Standard Specification for Roads and Bridges, Section 850 (SDDOT, 1998). The specification for the backfill is listed as follows:

> This material shall be free from dirt, vegetable matter, or other foreign substance.
>
> The material shall meet the following gradation requirements by dry weight:
>
> Percent Passing a 1 ½ inch (37.5 mm) sieve………………..100
> Percent Passing a 1 inch (25.0 mm) sieve…………………..95 - 100
> Percent Passing a ½ inch (12.5 mm) sieve………………….25 - 80
> Percent Passing a No. 4 (4.74 mm) sieve…………………...0 – 20
> Percent Passing a No. 8 (2.36 mm) sieve…………………...0 – 5
>
> Abrasion loss shall not exceed 40 percent
>
> Sampling and Testing:
>
> Sampling…………………………..SD 201
> Gradation……………………..……SD 202
> Los Angeles Abrasion…………....AASHTO T 96

The backfill material was a Sioux Quartzite rock fill with less than 5 percent passing the # 200 sieve. This expensive fill material was used in past SDDOT bridge end backfill designs since it had been shown to reduce void development under the approach slab because of its greater passive earth pressure resistance. It was determined that this material would perform well as a backfill material to construct the geotextile reinforced soil wall.

The fill material was placed using a front end loader dumping from the back of the soil wall excavation towards the abutment. The front-end loader dumped the fill while maintaining at least 6 inches of fill over the fabric to avoid direct contact between the geotextile and the front end loader tires. Figure 5.15 shows a design detail used prior to 1999 of the first layer of geotextile reinforced soil wall behind the integral abutment. This design detail shows the first layer at the face of the abutment containing the 4 inch diameter corrugated polyethylene perforated pipe directly behind the abutment seat. The 1 foot length of polyethylene sheet secured to the abutment is also shown. Figure 5.16 shows the design detail used as of the 1999 construction season. Figure 5.17 shows the design detail for the drainage system behind the wing wall sections of the embankment. It can be seen from this detail that drainage fabric was placed and MSE bridge end backfill was then placed over the 4 inch diameter corrugated polyethylene perforated pipe.

Once the material was placed, approximately six men distributed the material to create at 6-inch loose lift. Six inches of additional material were then placed directly behind the abutment in the form of a small mound descending away from the abutment wall. This was done in order to fold the excess fabric hanging on the abutment wall over this grade for embedment of the fabric under the next layer of fill material as seen in figure 5.15. Adequate densities were obtained after two or three passes with the compactor. After the embedment length fabric was placed, another 6 inches of fill material was placed in the same manner as previously in order to cover the 4 foot embedment geotextile length.

Construction of the embankment material behind the wing walls was performed as the lifts of the geotextile reinforced soil wall were completed. Figures 5.18 and 5.19 show the design details for a typical layer of MSE geotextile at the back face of the abutment and the side wrap sections of the geotextile reinforced soil wall used prior to 1999.

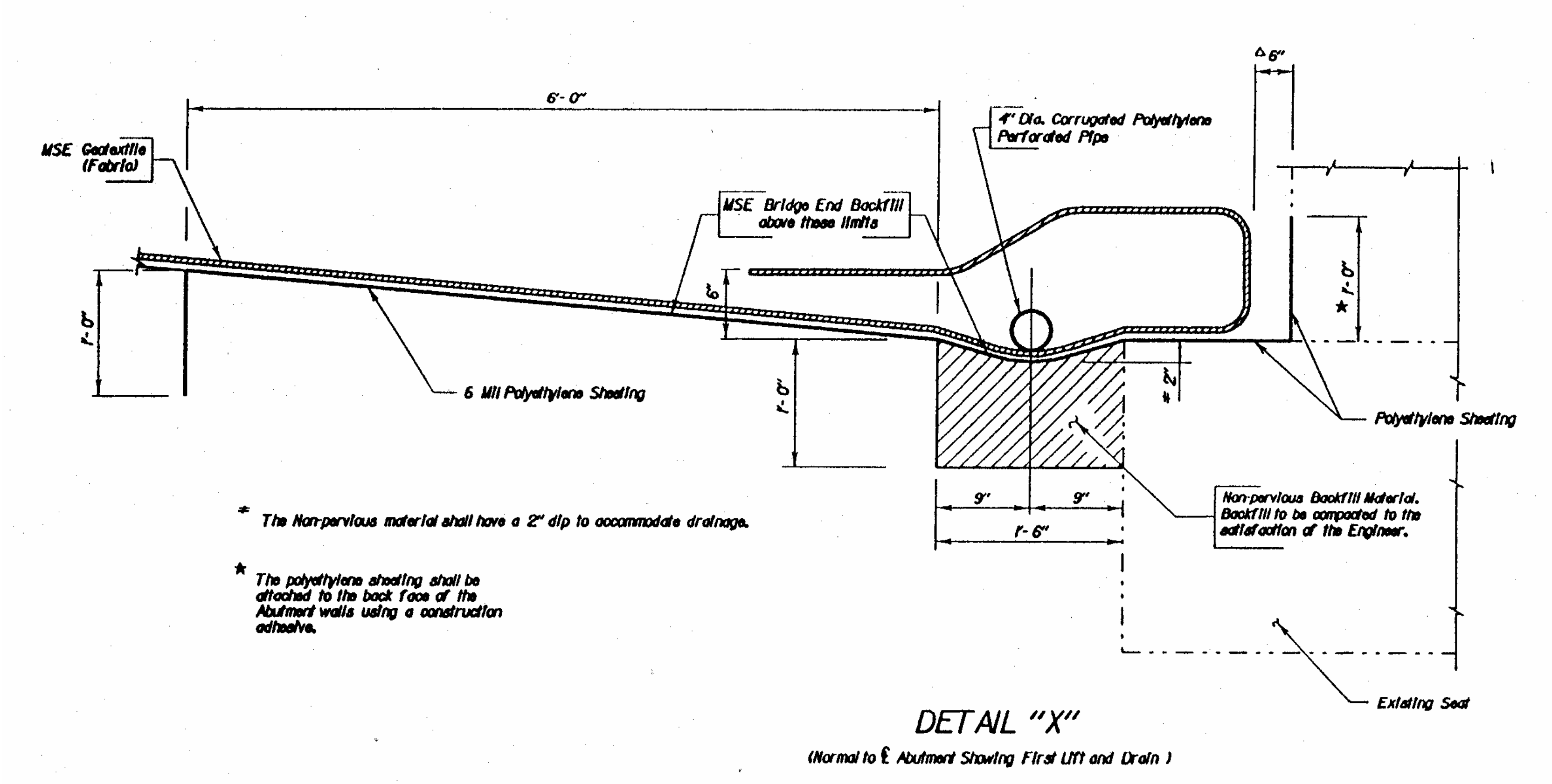


Figure 5.15 Design detail prior to 1999 showing the first layer of the geotextile reinforced soil wall

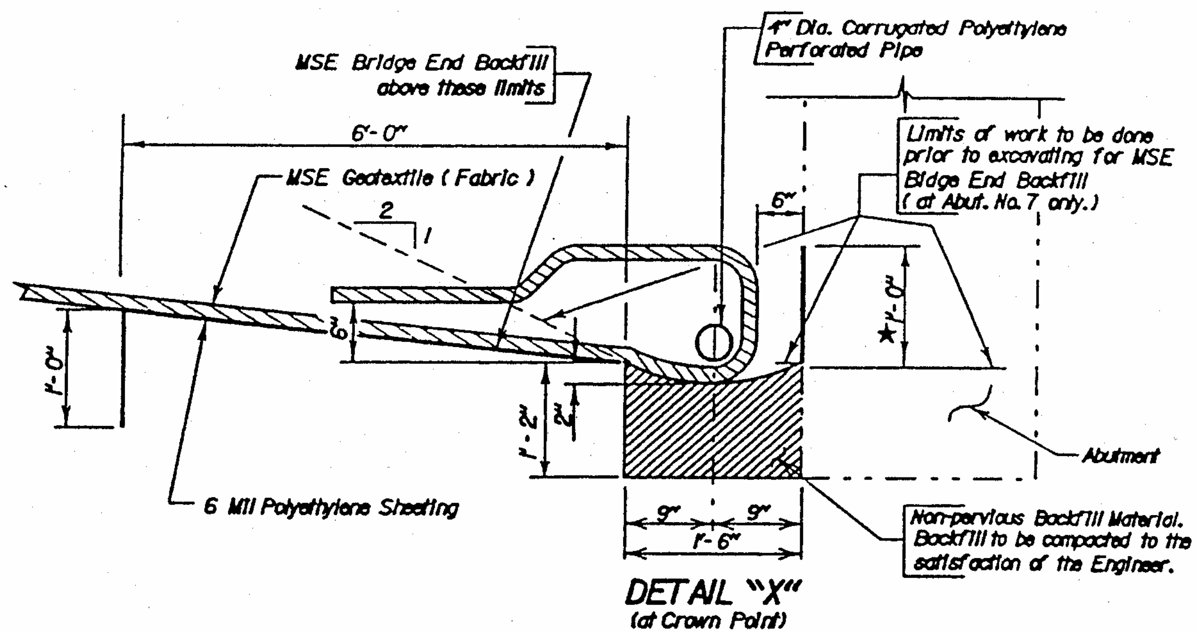


Figure 5.16 Design detail as of 1999 showing the first layer of the geotextile reinforced soil wall

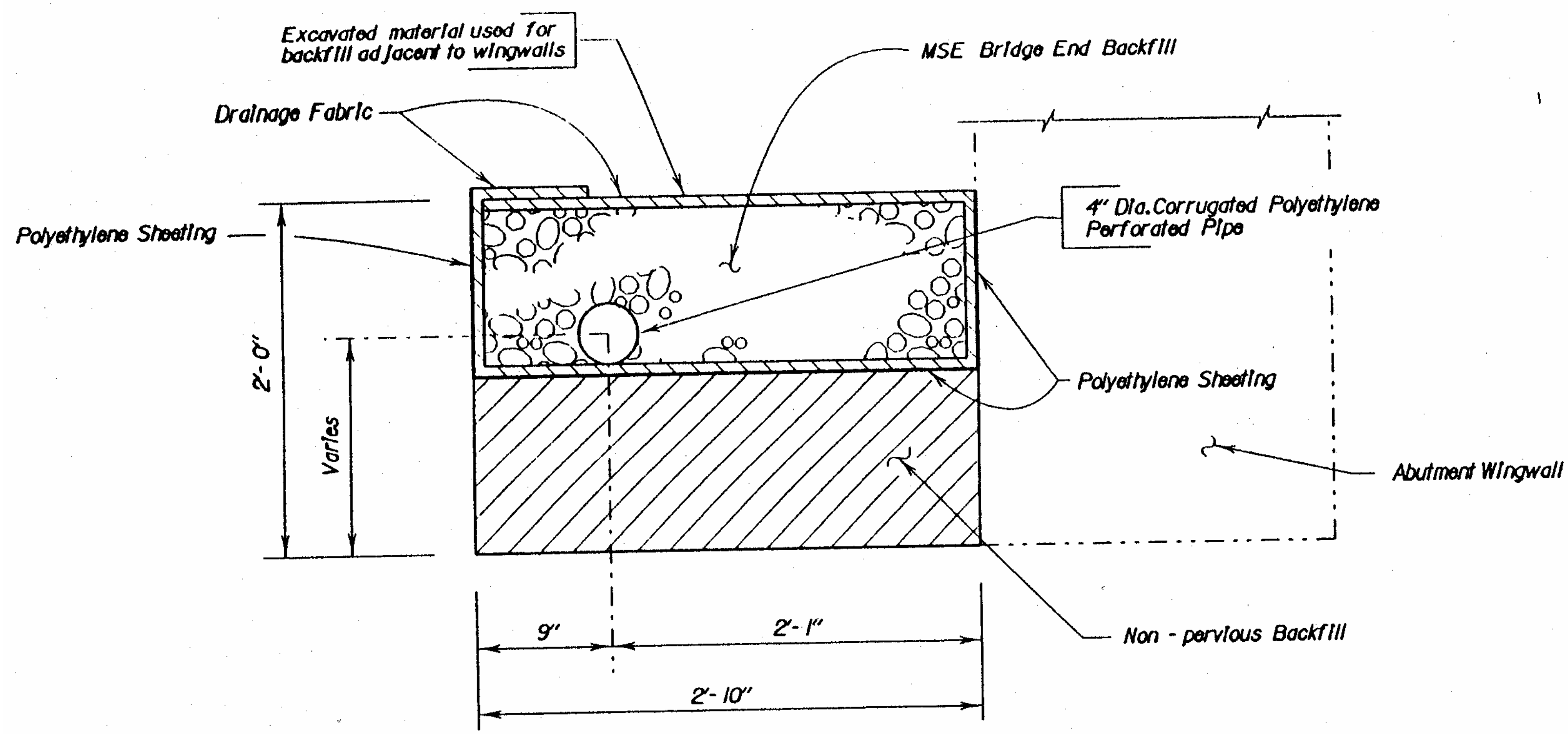


Figure 5.17 Design detail showing the drainage system behind the wing wall sections of the embankment

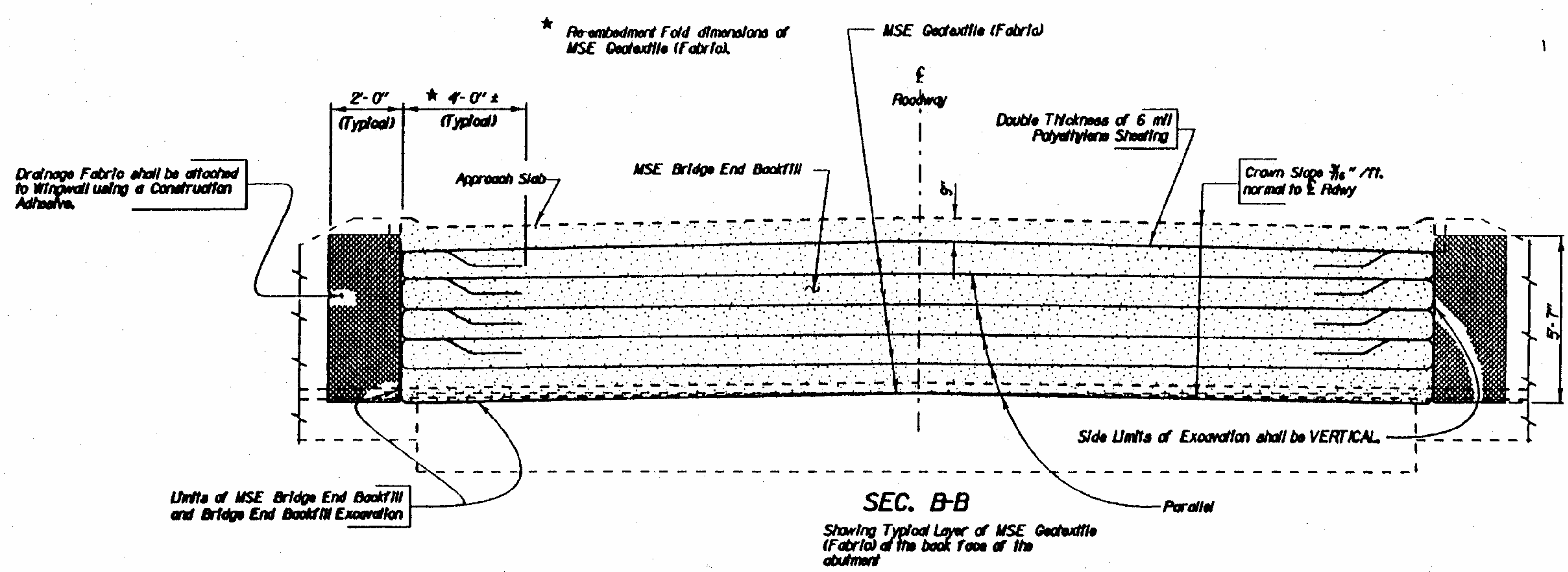


Figure 5.18 Design detail of a typical MSE geotextile reinforced soil wall from the back showing the side wraps and the drainage fabric along the wing walls prior to 1999

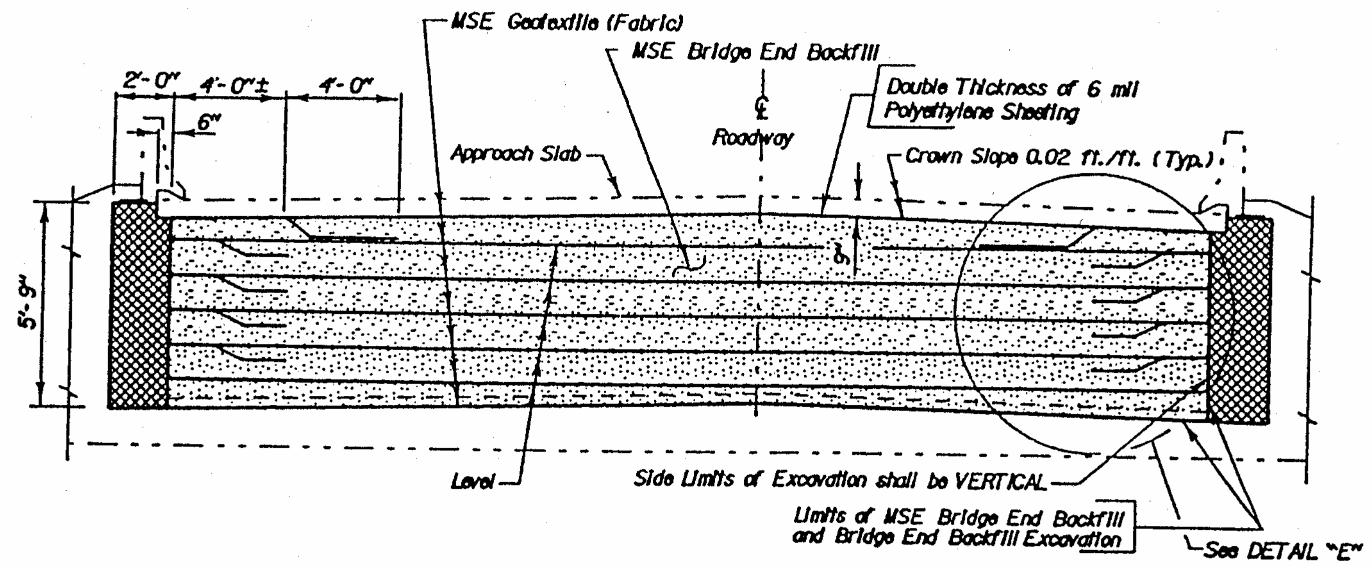


Figure 5.19 Design detail of a typical MSE geotextile reinforced soil wall from the back showing the side wraps and the drainage fabric along the wing walls since 1999

Figures 5.20 and 5.21 show these details as designed starting in the 1999 construction season. The soil wall limits extended the width of the bridge deck and the sides were wrapped in the same manner as for the section facing the abutment. A side wrap section can be seen in figure 5.18. It can also be seen from figure 5.19 that the bottom layer at the sides was constructed differently than the bottom layer at the face of the abutment. The bottom layer excess fabric is wrapped over the second layer of the geotextile reinforced soil wall and embedded to a length of 4 feet.

Three-quarter inch plywood was used as the separation barrier between the soil wall, air void, and the side embankment fill behind the wing walls of the embankment. The plywood was placed along the edge of the soil wall on each side and was placed against the cardboard separator. Drainage fabric was then placed on the face of the wing wall using a construction adhesive and along the sides of the geotextile reinforced soil wall. Embankment soil was then placed behind the wing walls as the fabric reinforced soil wall was constructed and compacted to adequate densities with a jumper compactor. The plywood separator was pulled up as each layer of the fabric reinforced soil wall was constructed so that the plywood would be completely removed upon completion of the last layer.

The remaining layers were constructed in the same manner as the first layer.

5.4.5 Instrumentation

Physical measurements of the gap width, and void development, and bridge length change were obtained to evaluate the performance of the White River Bridge. Only gap width measurements were taken on the Big Sioux River Bridges due to heavy traffic conditions on Interstate 29.

5.4.5.1 Gap Width Measurement

In order to monitor the movement of the geotextile reinforced soil wall, gap width measurement holes were placed in each integral abutment between the bridge beams. Figure 5.22 (a&b) shows the hole locations with respect to the abutment and the bridge beams. The holes were numbered for data collection and analysis purposes. The holes allowed for measurement of the gap width between the soil wall and the abutment

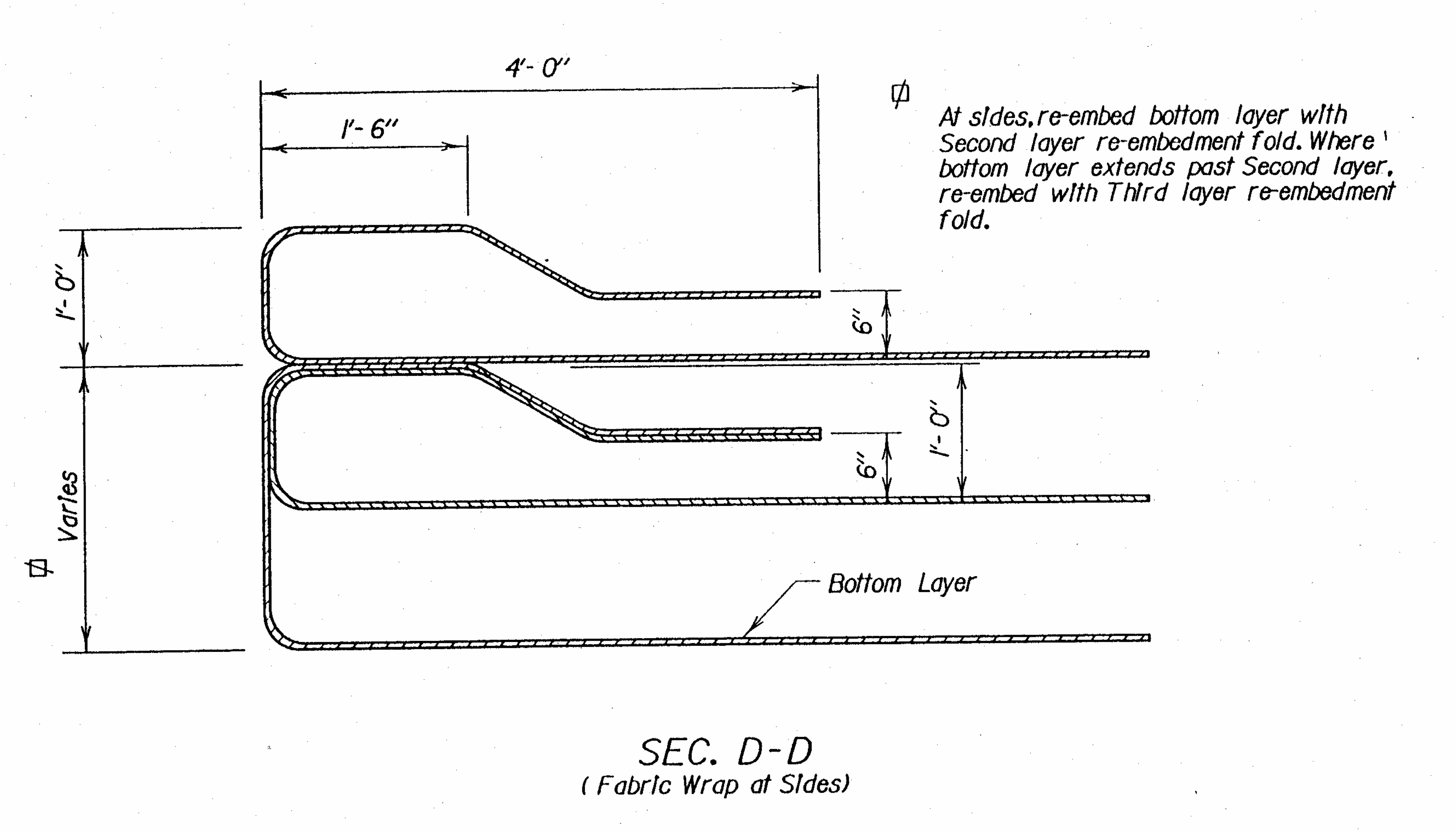


Figure 5.20 Design detail prior to 1999 of a typical side wrap section of the geotextile reinforced soil wall

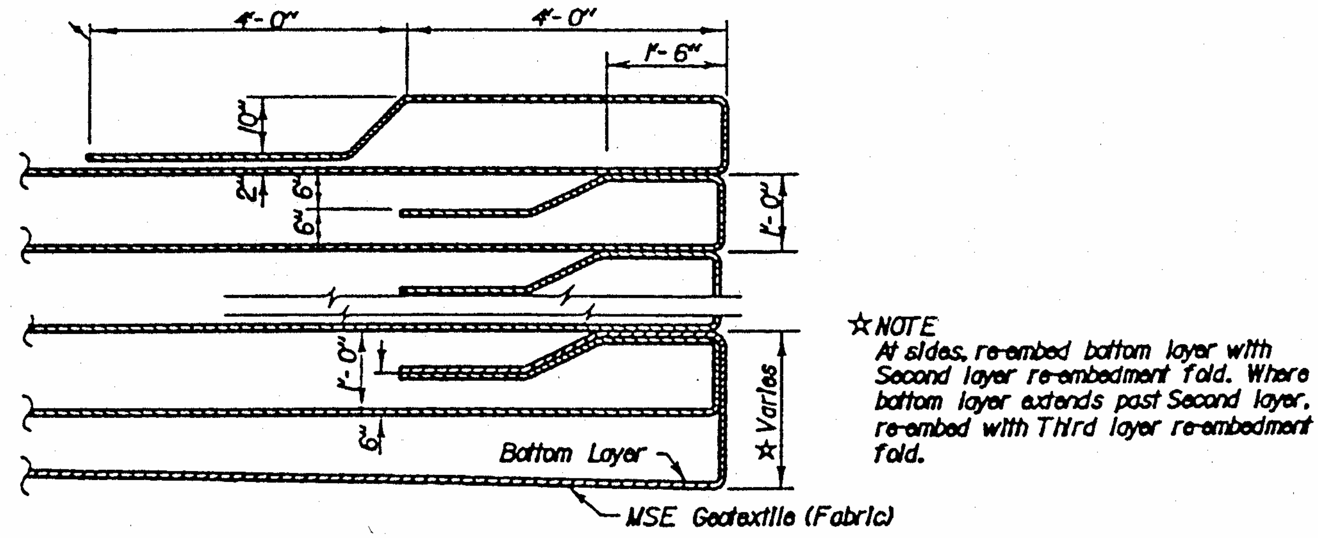


Figure 5.21 Design detail as of 1999 of a typical side wrap section of the geotextile reinforced soil wall

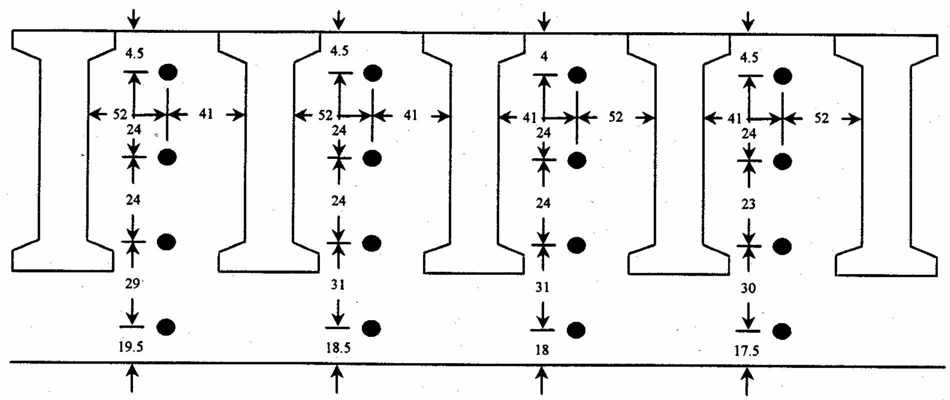


Figure 5.22-a. White River Bridge north abutment gap measurement hole locations in Relation to the abutment and bridge beams

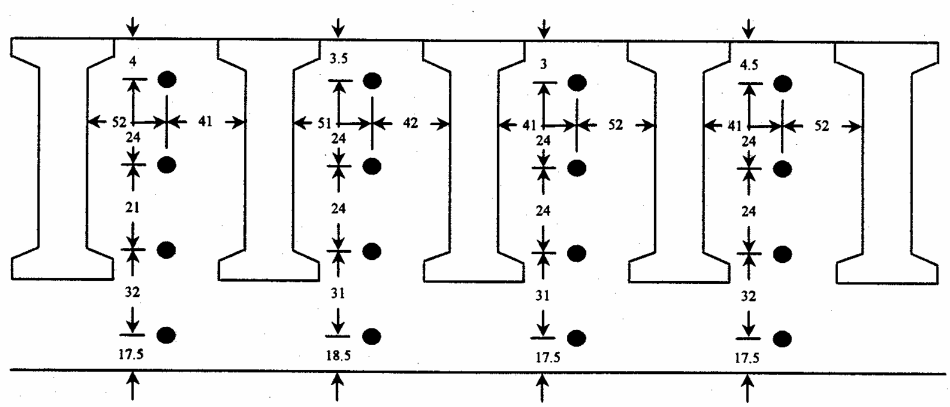


Figure 5.22 b. White River Bridge south abutment gap measurement hole locations in relation to the abutment and bridge beams

through the integral abutment wall. Measurements were performed by inserting a steel rod 1 inch in diameter and 45 inches in length through the integral abutment wall and pushing the rod against the face of the geotextile reinforced soil wall. A length measurement was then recorded with a tape measure of the remaining rod length referenced from a point next to the hole on the abutment wall. Measurements of the gap width for the Big Sioux River bridges were performed in the same manner.

5.4.5.2 Bridge Deck Measurement

Lower traffic volume allowed additional measurements to be performed on the White River Bridge. Bridge length measurements were recorded by measuring distances to each end of the bridge deck from a reference point in the middle. This was done at a distance of 24 inches from the guard rails on each side and at the centerline of the bridge deck. It was proposed that this would allow for determination of the amount of movement that was occurring due to thermally induced movements of the bridge system into and away from the geotextile reinforced soil wall. Results of the data however, were inconclusive due to an inability to produce accurate measurements. Truck traffic disturbed measurement data from the electronic distance measurements and the disruption of survey points on the bridge deck prevented further measurements.

5.4.5.3 Void Development

The development of a void under the approach slab was monitored through void measurement holes placed at various locations in the approach slab. Measurement of void development was monitored in the White River bridge backfill approaches, but not in the Big Sioux River bridges due to heavy traffic conditions.

The void development holes were placed as shown in figure 5.23. Measurements were made by placing a rod through the holes and measuring a vertical distance from a fixed point on the approach slab. Some problems did develop when measuring the development of void under the approach slabs. Upon placement of concrete to form the approach slab, 2 inch diameter pipe was placed in various locations the full depth of the approach slab. When the concrete was poured around the measurement pipes, some of

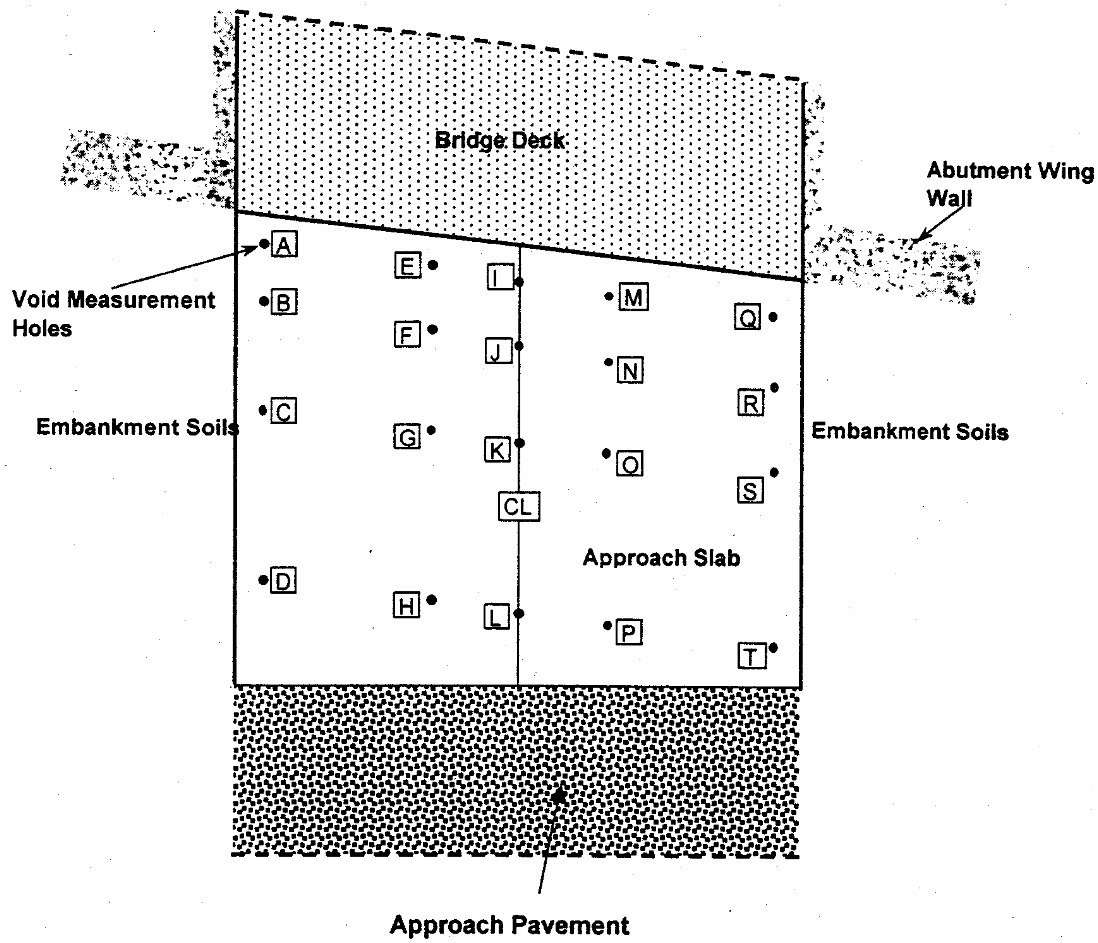


Figure 5.23 Void development hole locations for the north and south approach slab of the White River Bridge

the concrete penetrated into the pipes from the bottom. This prevented the measurement of void to the top of the geotextile reinforced soil wall.

The plastic caps posed a problem in that they could not be removed by unthreading the top cap portion. Therefore, the caps had to be punctured in order to place a 35 inch rod down through a center hole in the cap to measure the void development. This also obstructed the accuracy of the readings by allowing water and sand particles to enter the holes through the top caps. This was the case in many of the holes on both the north and south approach slabs.

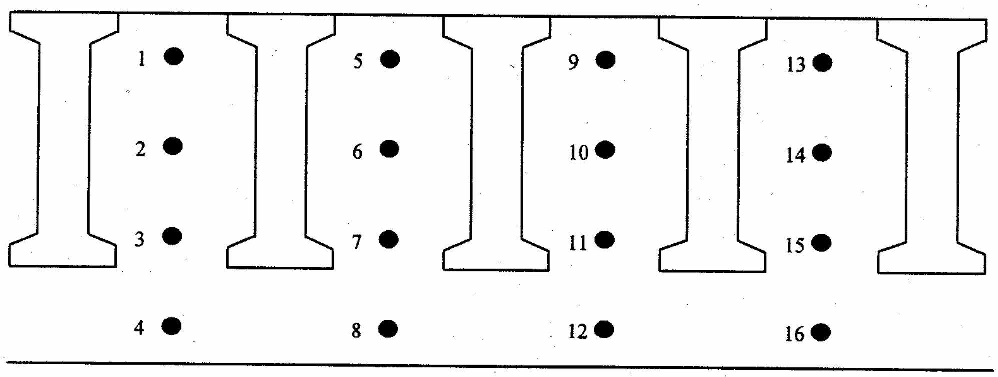
5.4.6 Field Study Results for Highway 73 White River Bridge

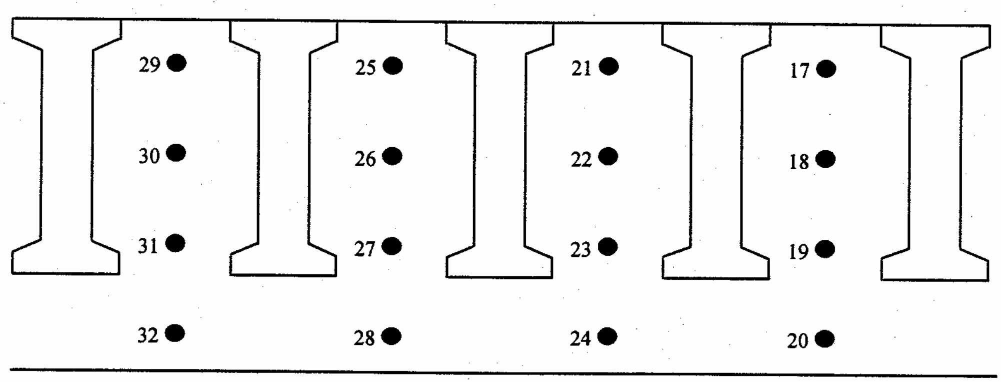
Data was collected for the gap width and for void development under the approach slab for the field study portion of the White River Bridge. This data has been compiled and important points are included. Gap width measurements were made approximately every 1 to 2 months for the White River and the Big Sioux River Bridges. Readings were taken more frequently for the Big Sioux River Bridges due to the relatively close proximity to Brookings, SD. Fewer readings were taken on the White River Bridge due to the large distance needed to travel from Brookings, SD.

5.4.6.1 Gap Width

The White River Bridge has completed over two years of cycled abutment movements. The locations of numbered gap width holes for the north and south abutments, with respect to the bridge abutment and the bridge beams, can be found in figure 5.24 (a&b). Gap width measurement data from the White River Bridge for the coldest and warmest temperatures can be seen in figures 5.25 and 5.26. In order to show movement with respect to variations in the time and year, which reflect temperature variations, only the warmest and coldest temperature periods were included in figures 5.25 and 5.26. Presenting only the warmest and coldest gap measurement periods allows for an analysis of the abutment and soil wall movement without excessive clutter in the graphs.

(a) South Abutment





(b) North Abutment

Figure 5.24 (a&b) White River Bridge abutment gap measurement numbered hole Location

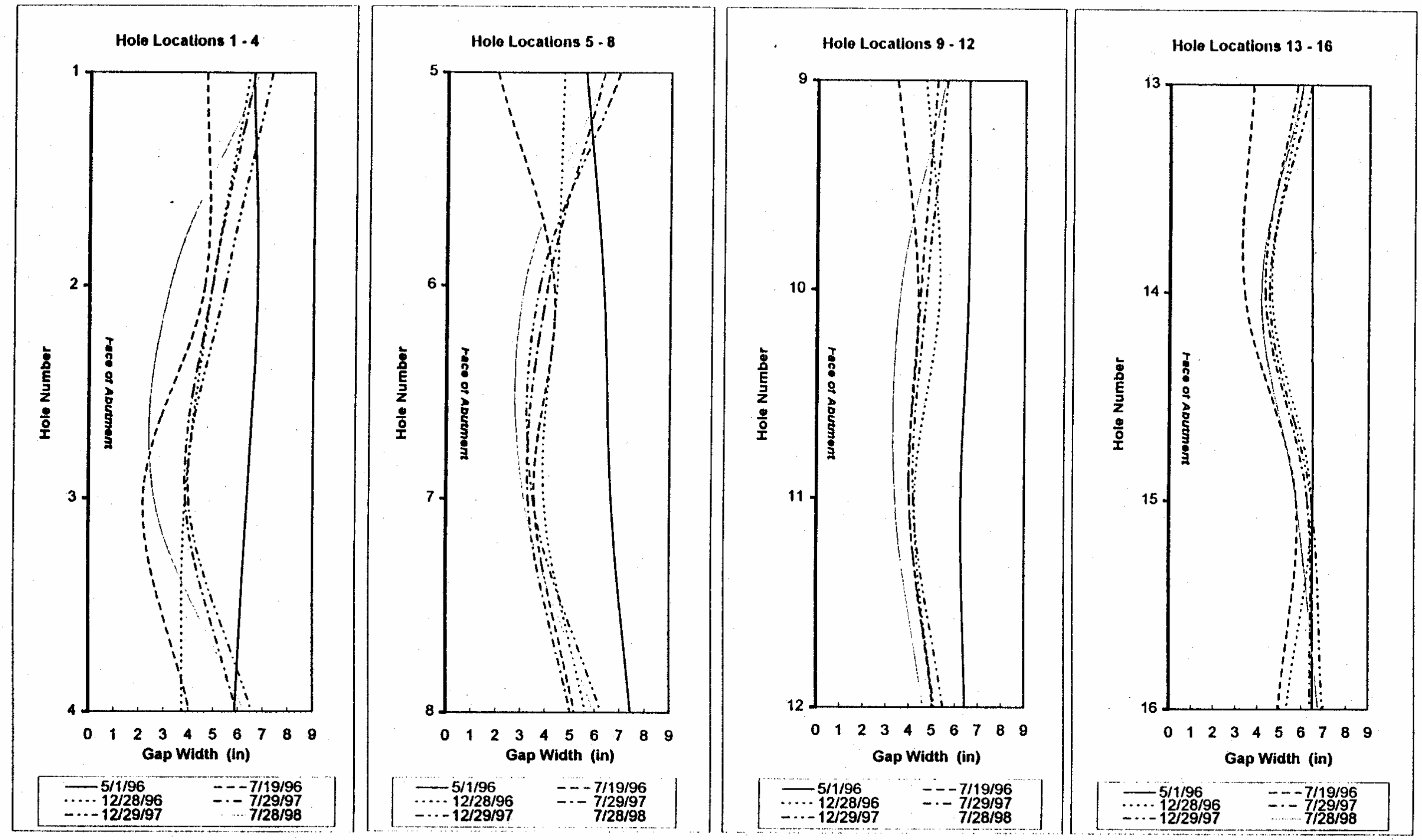


Figure 5.25 White River Bridge south abutment gap width measurements for holes 1 through 16 (Graphs show only the warmest and coldest temperature data from May 1996 through July 1998. Graphs represent gap width measurements from the top to the bottom of the abutment at the given numbered hole locations)

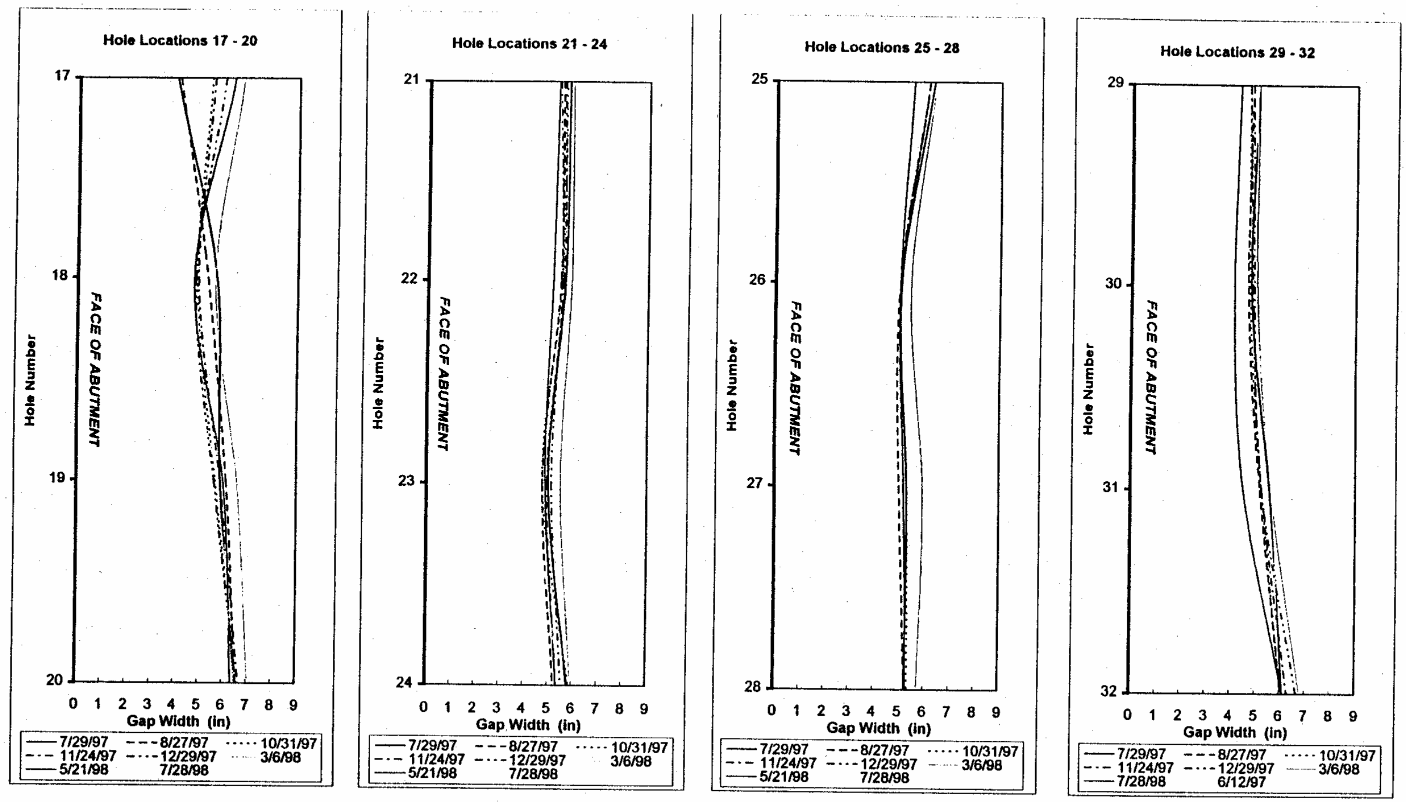


Figure 5.26 White River Bridge north abutment gap width measurements for holes 1 through 16 (Graphs show only the warmest and coldest temperature data from May 1996 through July 1998. Graphs represent gap width measurements from the top to the bottom of the abutment at the given numbered hole locations)

From figure 5.25 and 5.26, it can be seen that gap width measurements show some movement in the bridge system or soil wall. Analysis of data reveals that there is some initial movement of the geotextile reinforced soil wall after the supporting cardboard spacer is removed. This initial movement was in the range of 0.5 to 2 inches in some locations. This has been attributed to the lateral movement of the soil wall after the cardboard spacer is removed. The embedment wrapped geotextile fabric may not have been wrapped tightly enough to prevent sag after the separator was removed, therefore allowing the individual wraps to slide toward the abutment wall until the geotextile fabric had adequate tension to prevent such movement. It was anticipated in the design process that some degree of sag might occur after the supporting cardboard is removed. Hence, a 6 inch gap was used in order to compensate for such movements.

The construction of the White River Bridge geotextile reinforced soil wall embankments behind the integral abutments were the first to be designed and constructed by the SDDOT. This ultimately revealed some problem areas during the construction of the first embankment, the south embankment. The geotextile was not wrapped properly in the corners in the first embankment. This allowed for movement of the geotextile wraps after the supporting cardboard was removed. This was corrected by wrapping the corners in a different fashion in order to maintain adequate tension. This alleviates some of the initial sliding of the geotextile soil wall. Many of the geotextile wraps along the face of the abutment were not wrapped to ensure adequate tension as well. This resulted in the larger movements of the wall face after the supporting cardboard was removed. These problem areas were corrected during construction of the second embankment, the north embankment. A reduction in this initial movement was evident upon evaluation of data collected on the gap width measurements of the second embankment.

Much of the subsequent gap width change is correlated to temperature changes. Preliminary findings show that there are typically 0.5 to 1.0 inch of movement between the coldest (minus 20 degrees Fahrenheit) and the warmest (plus 90 degrees Fahrenheit) temperature periods. This seems to be consistent the full width of the abutment with some variation apparently due to slight discrepancies in measurements. The amount of movement of the abutment that is due to temperature changes and how much is due to soil wall movement has not been conclusively determined, but the cyclic movement of the abutment wall evident from gap width measurements does not show significant movement of the soil wall. Data shows that the designed gap width is performing, as of this time, with at least three inches of gap width available at the highest temperature period throughout the entire face of the north and south integral abutment walls.

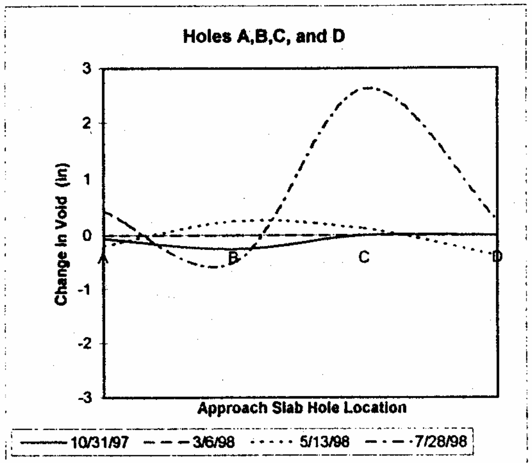
5.4.6.2 Void Development

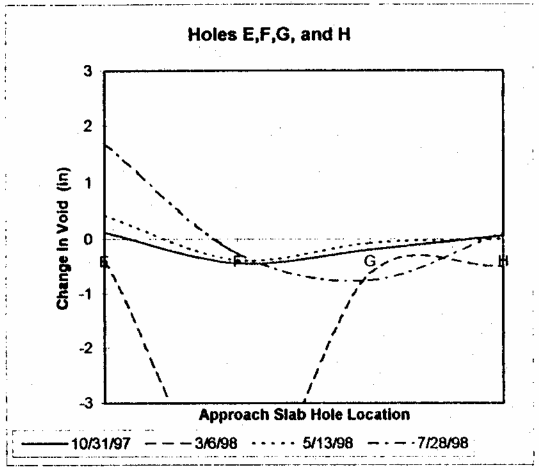
Measurements of void development on the White River Bridge began in the summer of 1997. Readings on the both the south and north approach slab show a general trend of 0.5 to 1.0 inch of void development in areas close to the abutment wall. Data from void development measurements under the approach slabs of the White River Bridge can be seen in figures 5.27 and 5.28. Readings have been difficult to evaluate due to construction problems. The concrete in some of the holes prevents measurement in many of the holes.

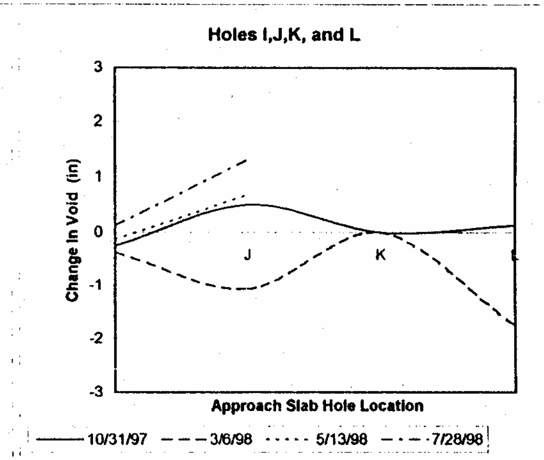
Readings were made in all holes that could be measured. Based on available measurements, data has shown that there has been some movement of the soil wall under the approach slab. The movement appears to be consistent with past research by the WYDOT (Edgar *et al*., 1989) and the OKDOT (Snethen *et al*., 1997) in that the reinforced soil wall does reduce void development, but does not eliminate it. Differential settlement between the bridge structure and the surrounding soil material may decrease over time, therefore reducing the further development of a void.

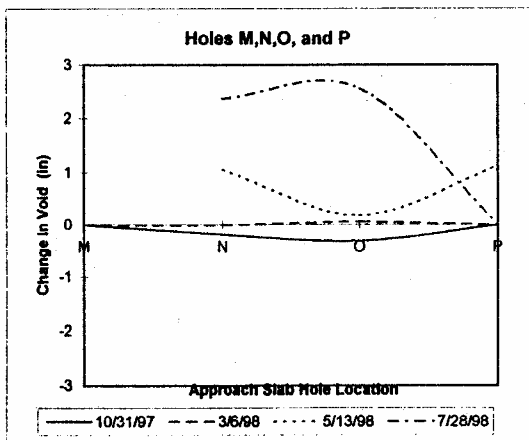
5.4.7 Field Study Results for Interstate 29 Big Sioux River Bridges

Data was collected on two bridges crossing the Big Sioux River in much the same manner as was done for the White River Bridge. Again, gap width measurements were taken for the air void between the geotextile reinforced soil wall and the integral abutment. Fewer holes were established for taking the gap measurements due to the smaller geometry of the bridge structures and integral abutments. Figures 5.29 and 5.30 show the gap width hole locations with respect to the abutment and the bridge beams. Figure 5.31 shows the gap width measurement numbered hole locations for the Big Sioux River Bridges. Nine holes were used to measure gap width in each of the Big Sioux River Bridge integral abutments while 16 holes were used in each abutment on the White


Holes A,B,C, and D
Change in Void (in)
3
2
1
0
-1
-2
-3
A
B
C
D
Approach Slab Hole Location
10/31/97 3/6/98 5/13/98 7/28/98


Holes E,F,G, and H
Change in Void (in)
3
2
1
0
-1
-2
-3
E
F
G
H
Approach Slab Hole Location
10/31/97 3/6/98 5/13/98 7/28/98


Holes I,J,K, and L
Change in Void (in)
3
2
1
0
-1
-2
-3
J
K
L
Approach Slab Hole Location
10/31/97 3/6/98 5/13/98 7/28/98


Holes M,N,O, and P
Change in Void (in)
3
2
1
0
-1
-2
-3
M
N
O
P
Approach Slab Hole Location
10/31/97 3/6/98 5/13/98 7/28/98

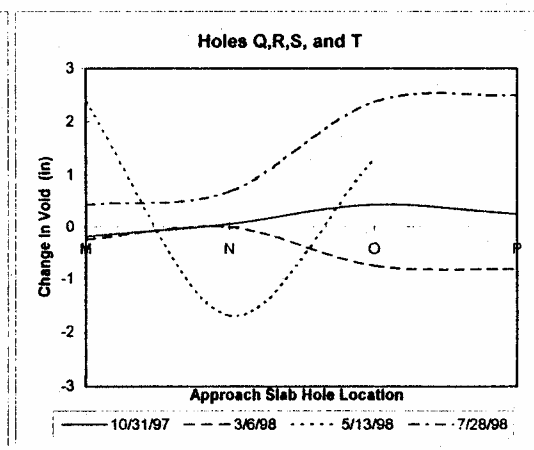

Holes Q,R,S, and T
Change in Void (in)
3
2
1
0
-1
-2
-3
M
N
O
P
Approach Slab Hole Location
10/31/97 3/6/98 5/13/98 7/28/98

Figure 5.27 White River Bridge change in void development under the south approach slab for holes A through T

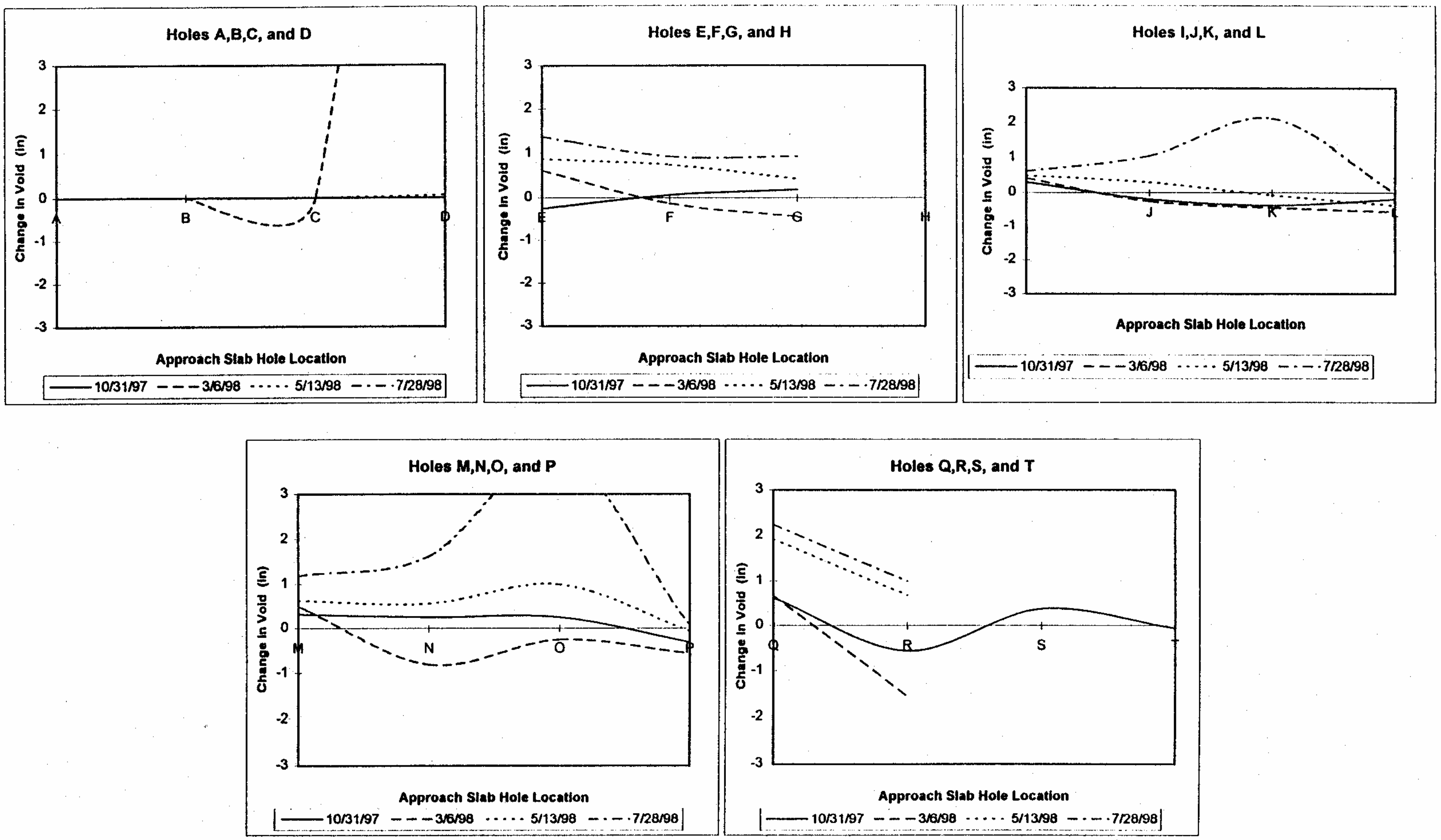
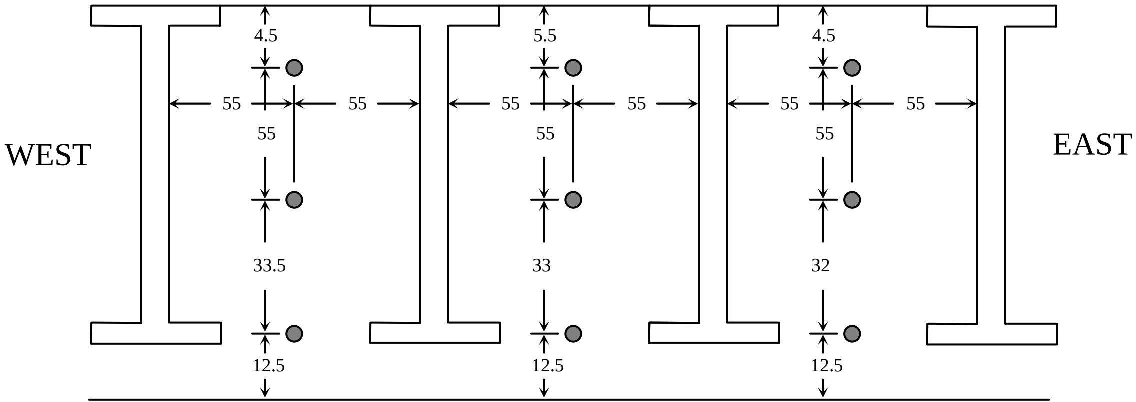
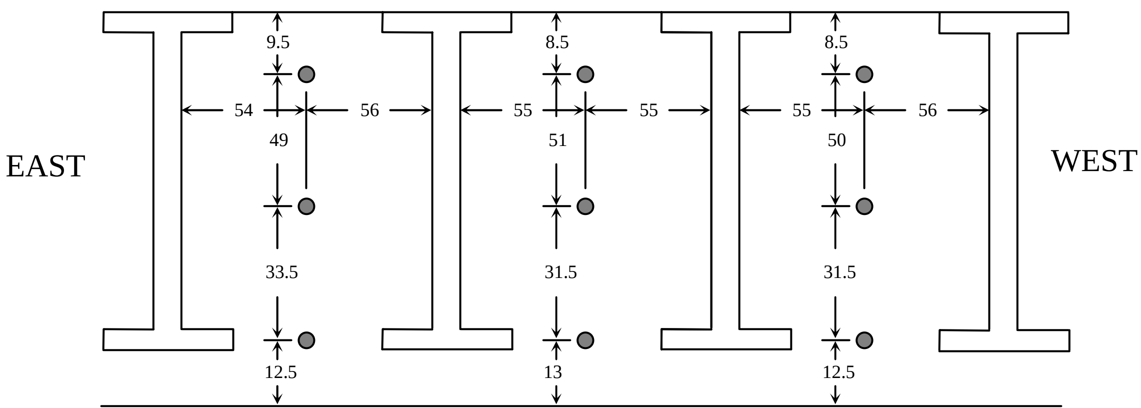


Figure 5.28 White River Bridge change in void development under the north approach slab for holes A through T

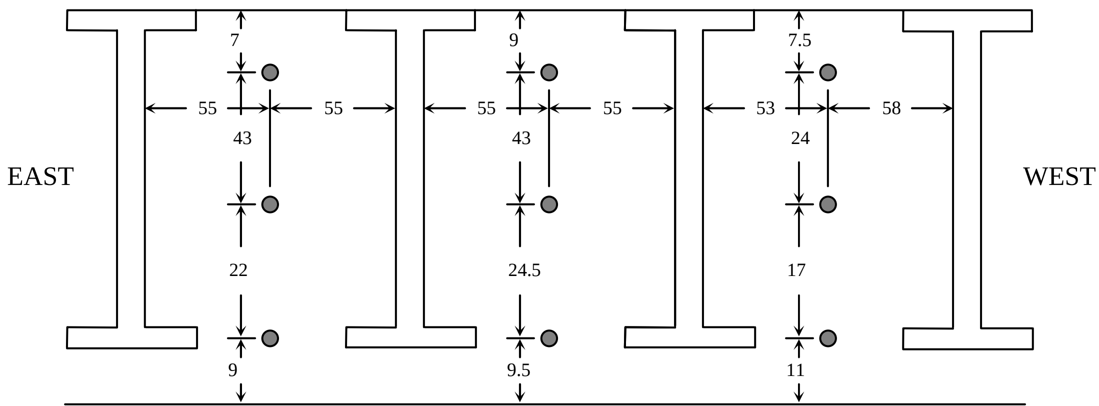


(a). South bound north abutment

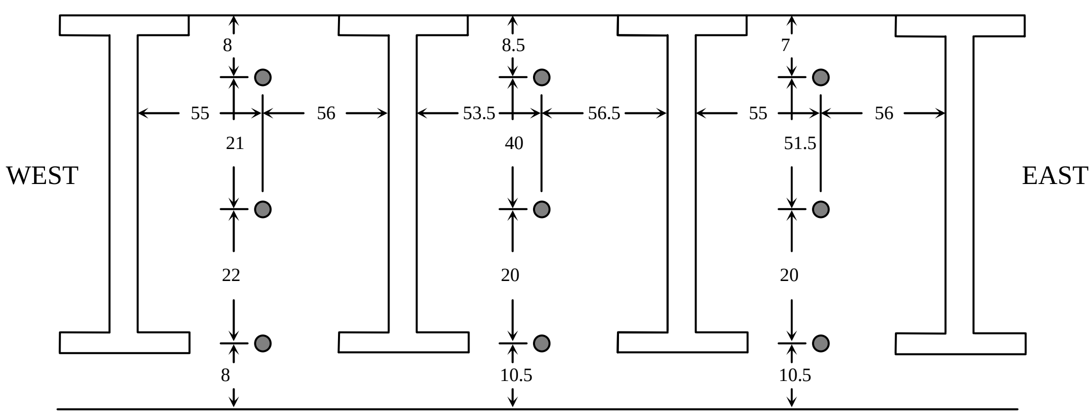


(b) South bound south abutment

Figure 5.29 (a&b) South bound Big Sioux River Bridge abutment hole locations (all dimensions in inches)

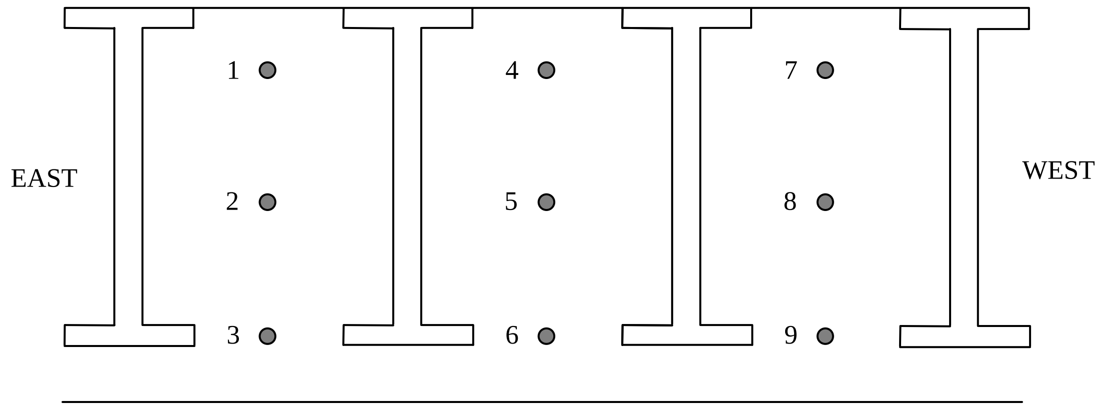


(a) North bound south abutment

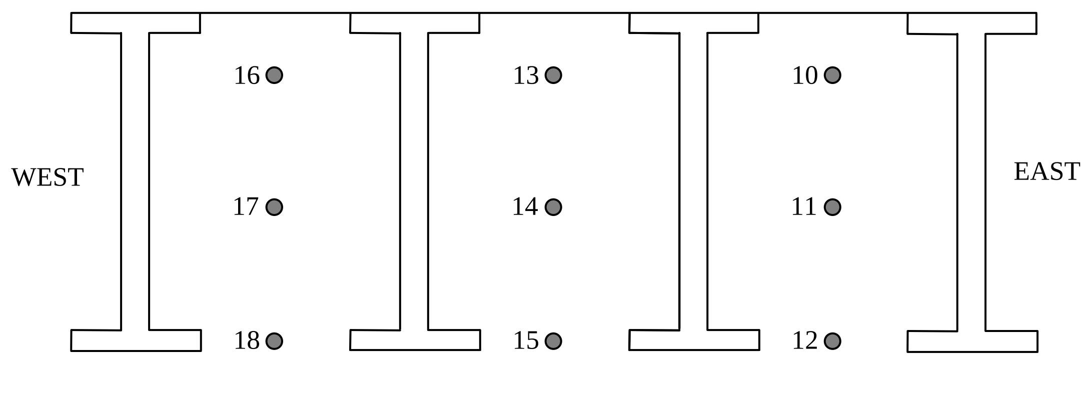


(b) North bound north abutment

Figure 5.30 (a&b) North bound Big Sioux River Bridge abutment hole locations (all dimensions in inches)



a. South Abutment



b. North Abutment

Figure 5.31 (a&b) North and south bound Big Sioux River Bridge numbered abutment hole locations

River Bridge. Void development data was not able to be taken due to heavy traffic conditions.

5.4.7.1 South Bound Gap Width

The south bound Big Sioux River Bridge has completed over a full years cycle of abutment movement. Figures 5.32 and 5.33 show the results of the gap width measurements for the south and north abutments. These graphs represent data collected during the coldest and warmest temperature periods for a one-year cycle of abutment movement.

Upon review of the gap width measurements, preliminary findings show that there is a much smaller initial movement of the soil wall after the supporting cardboard is removed than in the case for the White River Bridge. Subsequent measurements reveal that there is an increase in the gap width larger than what was initially constructed during the coldest temperature cycles. This reduction in initial movement may be a result of improved construction. The increase in gap width may be attributed to the time of year the wall was constructed. The soil walls in the White River Bridge case were constructed in the early spring while the temperature was low, therefore the abutment was in its contraction cycle. The Big Sioux River Bridges were constructed in the mid summer when the temperature is at it highest, typically around 90 degrees during the construction period. Therefore, the abutment was in its largest expansion cycle. This ultimately leads to an increase in gap width during cold temperature periods and a larger gap width is present throughout the entire yearly cycle of abutment movement. Subsequent data show that there has been a change in gap width that correlates with temperature changes. Gap width measurement holes 7 and 16 have an obstruction which has not been able to be removed. This produces the missing lines evident in the figures 5.32 and 5.33.

5.4.7.2 North Bound Gap Width

The northbound Big Sioux River Bridge completed a full years cycle of abutment movement. Figure 5.34 and 5.35 show the variations in gap width during the warmest and coldest temperature periods. From these figures, it can be seen that the wall was initially constructed near the 6 inch designed gap width with some variation, most of

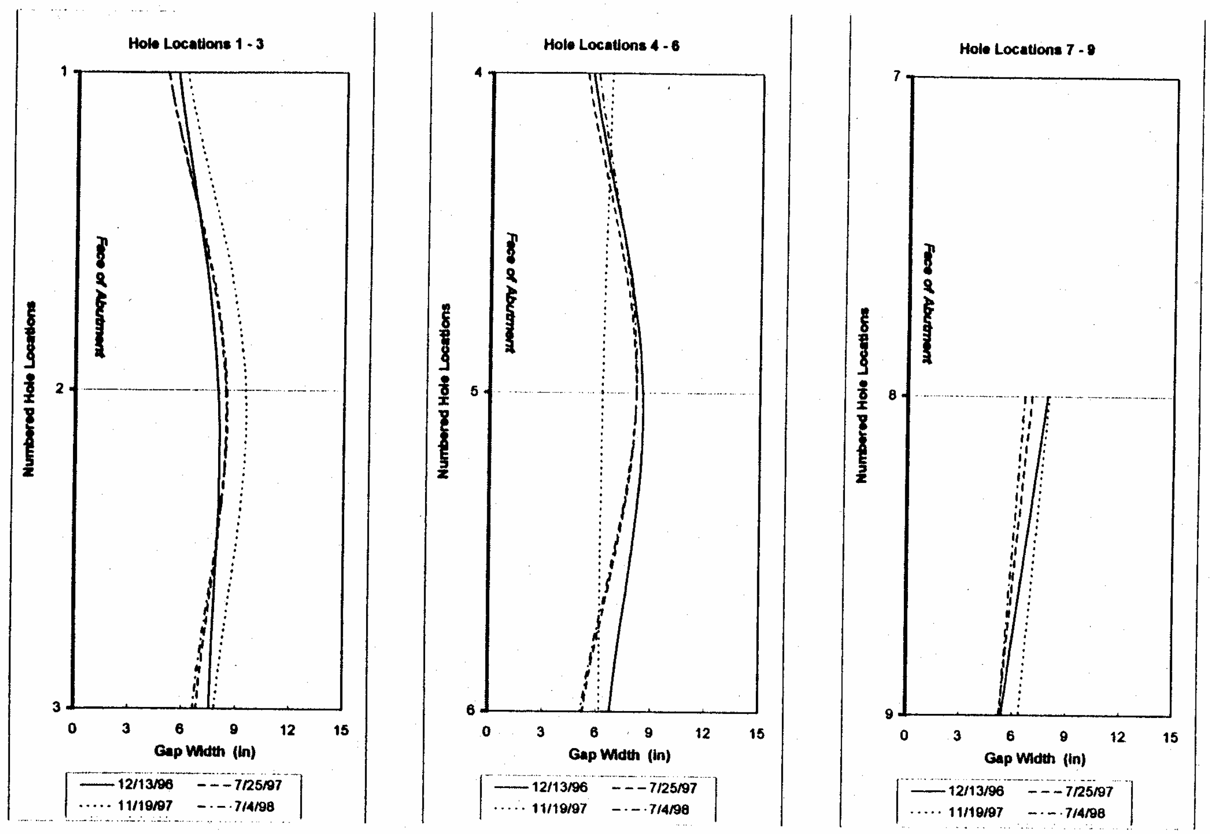


Figure 5.32 Big Sioux River Bridge south bound south abutment gap width measurements for holes 1 through 9 (Graphs show onlythe warmest and coldest temperature data from December 1996 through July 1998. Graphs represent measured gap width measurements from the top to the bottom of the abutment at the given numbered hole locations.

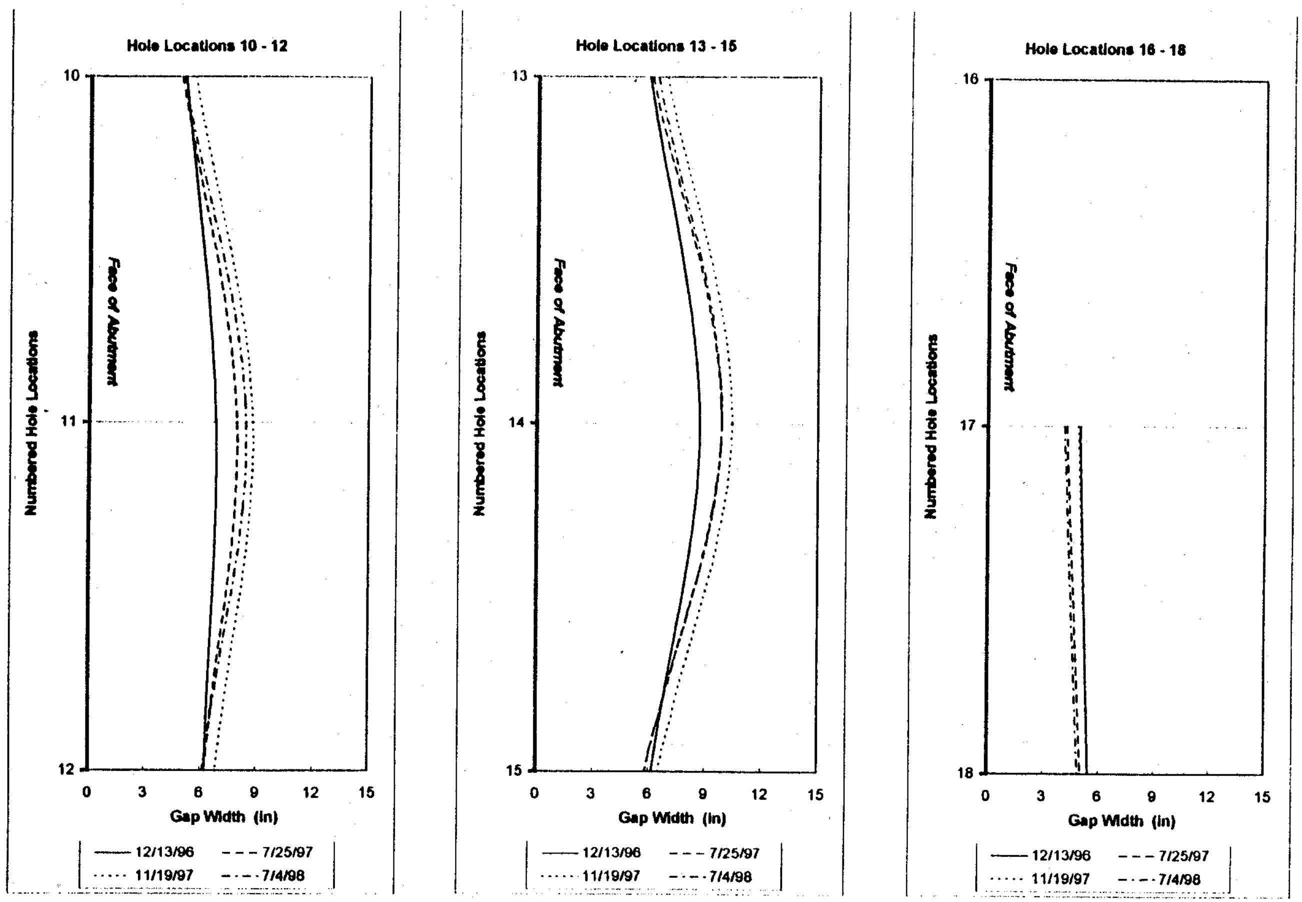


Figure 5.33 Big Sioux River Bridge south bound north abutment gap width measurements for holes 10 through 18 (Graphs show only the warmest and coldest temperature data from December 1996 through July 1998. Graphs represent measured gap width measurements from the top to the bottom of the abutment at the given numbered hole locations.

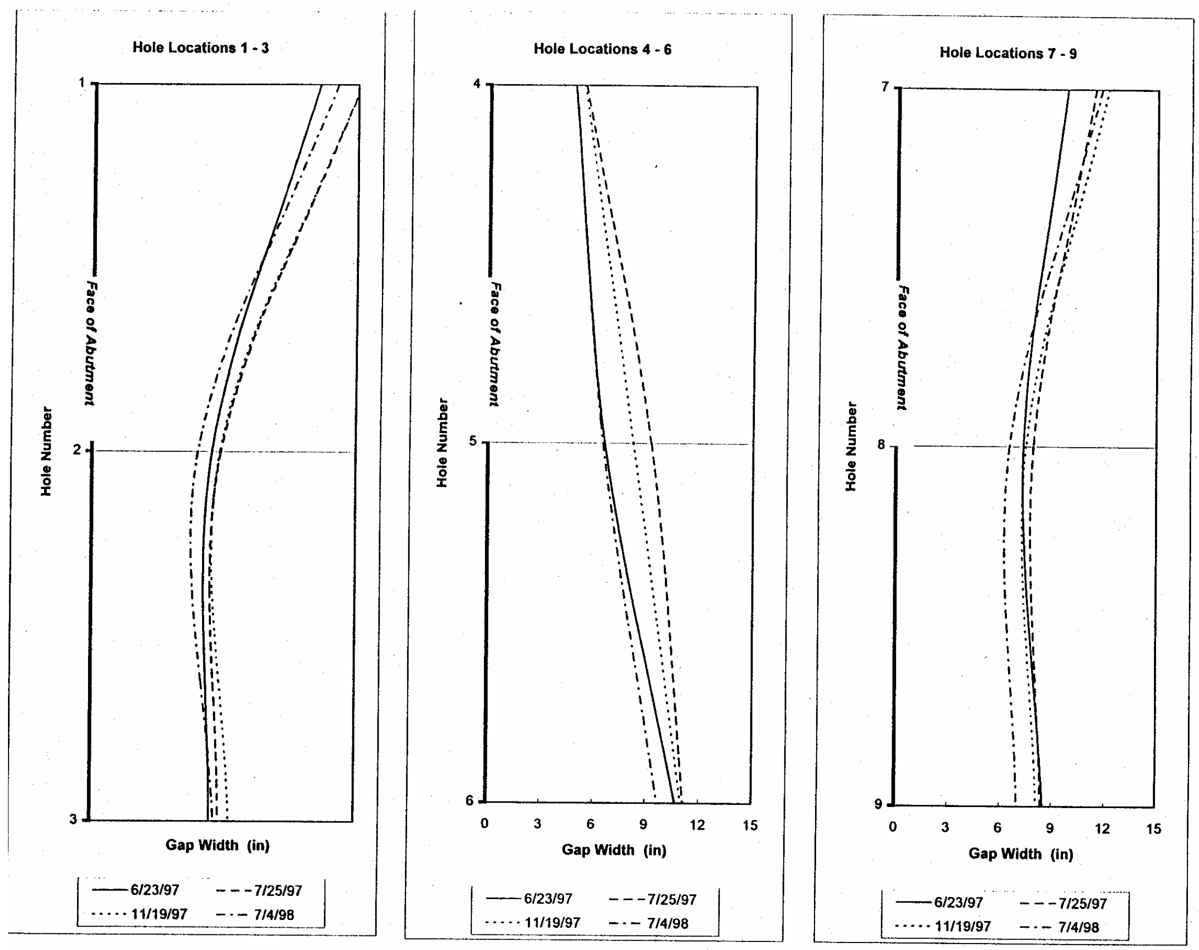


Figure 5.34 Big Sioux River Bridge north bound south abutment gap width measurements for holes 1 through 9 (Graphs show only the warmest and coldest temperature data from December 1996 through July 1998. Graphs represent measured gap widt h measurements from the top to the bottom of the abutment at the given numbered hole locations.

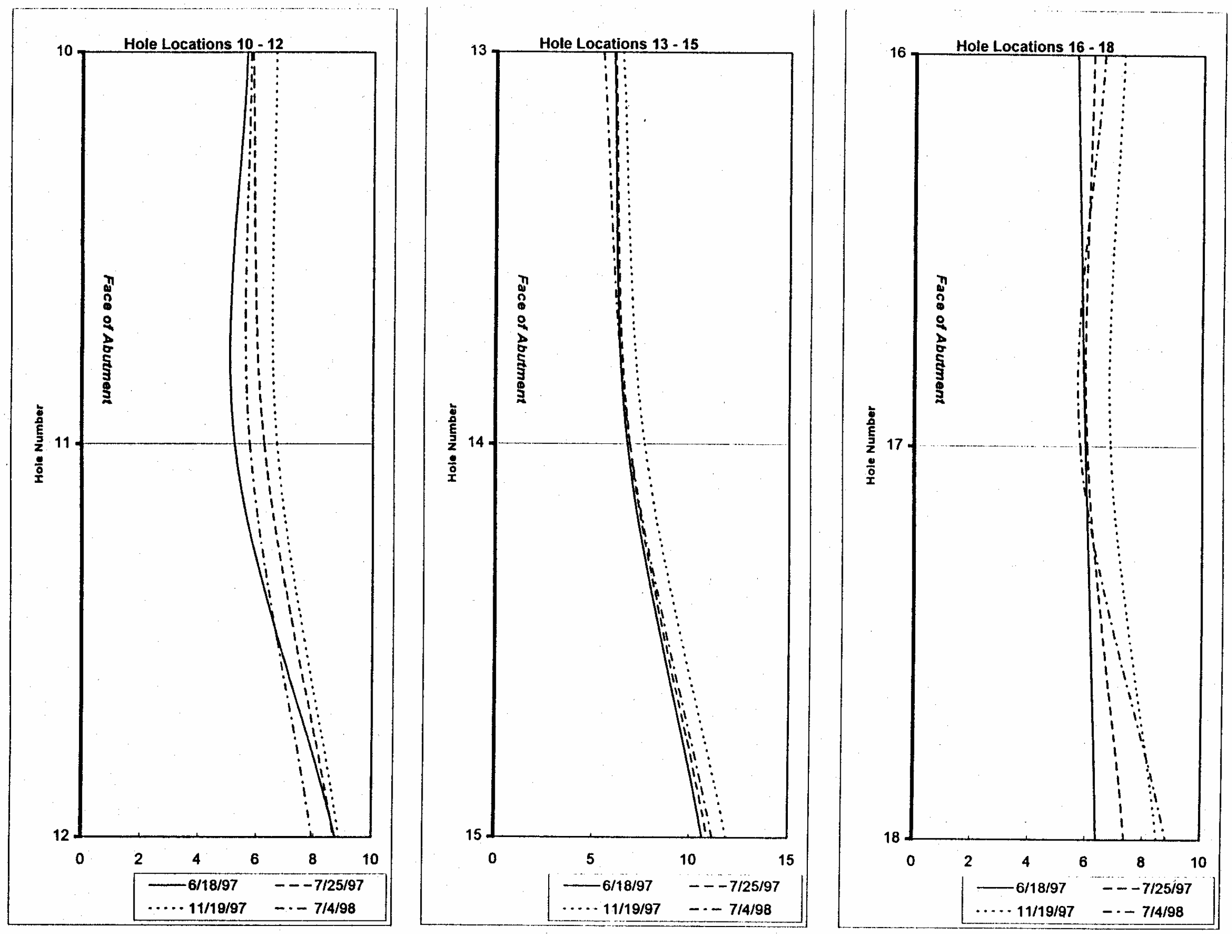


Figure 5.35 Big Sioux River Bridge north bound north abutment gap width measurements for holes 10 through 18 (Graphs show only the warmest and coldest temperature data from December 1996 through July 1998. Graphs represent measured gap width measurements from the top to the bottom of the abutment at the given numbered hole locations.

which is larger. Data has shown that between the warmest and coldest temperature periods, a movement of approximately 1 to 2 inches is common at most of the hole locations. The smallest gap width measured was approximately 5 inches at hole location number four.

5.4.8 General Observations

Shortly after the construction of the White River Bridge and the Big Sioux River bridges, settlement of the embankment material on the embankment side of the wing walls was observed. On the Big Sioux River bridges, the gap has been exposed on the east embankments of the north and south abutments of the north bound bridge and the east embankment of the south abutment of the southbound bridge. This problem was initially contributed to steep embankment construction, lack of supporting vegetation, and water erosion during precipitation events. Design specifications of the approach slabs called for curb and gutter sections on the edges of the approach slab. These curb and gutter sections were not constructed in many of the approach slabs, therefore allowing excess water to exit the approach slab next to the wing walls. This further increased erosion problems of the embankment material behind the wing wall sections. In the spring and summer of 1998, the erosion became severe enough to expose the gap between the integral abutment and the geotextile reinforced soil wall on several of the embankments. After observation of an exposed gap in many of the abutments, it was evident that factors other than just erosion around the wing walls were producing these complications. Asphalt had been placed on top of the embankments behind the wing walls in most cases. Significant soil loss was still evident, which indicates that erosion was not entirely the cause of the soil loss. The gap has been exposed in the west side of the south abutment and the east side of the north abutment on the White River Bridge as well.

Through observations of performance, construction practices, and design details, it has been determined that a construction problem, and possibly a design problem, exists in the transition point between the air void and the embankment material behind the wing walls. Design specifications call for a drainage fabric to be placed along the wingwall and the sides of the geotextile reinforced soil wall. The fabric was then to be glued to the wingwall with a construction adhesive and embankment soil placed behind the wingwall as construction of the geotextile reinforced soil wall was constructed. This allowed for adequate tension in the fabric to contain the side embankment material and prevent the soil from penetrating into the air void in front of the geotextile reinforced soil wall. It has become evident that the soil behind the wingwall is migrating into the air void in some instances and that the drainage fabric is not performing as designed. This could be due in part to improper construction and the cyclic movement of the integral abutment.

Observation of construction has shown that the fabric that is to contain the embankment fill from entering the air void was not placed to specifications. Two feet of fabric was to be placed along the side of the fabric reinforced soil wall and along the wing wall section of the abutment. It has been observed that the fabric, which was glued to the wing walls during construction, has been pulled away from the wing wall on many of the abutments. The adhesive is not holding the fabric against the wing walls. This allows for the movement of soil material in the embankment to migrate behind the fabric. This may be allowing embankment soil to penetrate into the 6 inch air void behind the abutment and the geotextile reinforced soil wall, causing deformation of the embankment soils on the side slopes. If the fabric was not secured firmly behind the abutment, the fabric would tend to slide into the gap. Plywood is used as a separation barrier at the transition area and was pulled up as the layers of the geotextile reinforced soil wall were constructed. Pulling up the plywood may have disturbed the placement of the fabric at the transition area as well. Finally, installation of guardrail posts in this area may have also disrupted the fabric allowing the eroded material to penetrate into the gap.

The temperature-induced movements of the integral abutment further produce problems. Cyclic movements of the integral abutment cause significant densification of the embankment soil material on the side slopes behind the wing wall sections of the abutment. This causes compression in the areas adjacent to the wing walls. Enough deformation has occurred in many of the abutments of all three bridges to expose the 6 inch gap under the approach slab between integral abutment and the geotextile reinforced soil wall. The cyclic movement of the abutment contracts the embankment soils behind the wing wall during expansion cycles of the bridge system. Contraction of the bridge system and subsequent movements away from the embankment soils allows intrusion of water between the embankment soils and the wing wall. The movement of the integral abutment may allow the fabric to slide along the wing wall as well, increasing the soil penetration behind the fabric and into the air void after each cycle occurs. Since many cycles of this nature occur during a one year period, the migration of soil into the void occurs quickly. Since the most recently constructed bridge of all of three bridge structures shows the greatest amount of soil disturbance behind the wingwalls, it has been determined that a large portion of the problem lies in the construction of this transition area. Whether the embankments soils received enough compaction and a proper amount of adhesive was used on the wingwalls to secure the fabric is unknown. Even if these transition areas are constructed to design specifications, the continued cyclic movement of the abutment will cause densification of the embankment material behind the wing wall sections. Over many cycles of movement, enough movement may occur to expose the 6 inch gap between the abutment and the geotextile reinforced soil wall. This has already been observed in just a two year period. This exposure of the gap to the elements introduces further problems such as erosion of soil material into the gap and rodent intrusion. It has become evident in the White River Bridge and the Big Sioux River Bridges that the problem will persist unless construction is improved or a design change is implemented.

5.4.9 Discussion and Summary

A geotextile reinforced soil wall behind the integral abutment is relatively easy to construct. Construction can be completed in a short amount of time with relatively inexperienced labor. Backfill materials such as a select granular backfill perform well as fill material for the geotextile reinforced soil wall in the construction process. A class I woven geotextile fabric can be placed with ease in order to construct the wrapped face wall. A honeycombed cardboard spacer 6 inches thick against the abutment is adequate in order to produce a 6 inch gap between the integral abutment and the geotextile reinforced soil wall. A front-end loader dumping backfill material starting from the area nearest the sleeper slab and working toward the integral abutment allows for easy and effective placement of the backfill material while maintaining the survivability of the woven geotextile fabric. A walk behind plate compactor performs well in order to compact the backfill material for the individual layers of reinforcement and a jumper compactor can be used on the side embankment material behind the wing wall sections of the abutment.

The use of geotextile reinforced soil walls behind the integral abutment reduces short-term void development in the embankment materials under the approach slab. Long-term effects are unknown at this time. Available measurements of the void development under the approach slab for the White River Bridge show that a void is developing under the approach slab. The size of the void in a two year period is relatively small and it is unknown at this point whether or not the void will become increasingly larger over time. Results from model tests performed by Schaefer and Koch in 1992 revealed a void development under the approach slab of approximately 2 inches in one year of cycled abutment movement. The results from measurements of void development under the approach slab indicate a void of approximately 0.5 to 1.0 inch over a two-year period.

Pressures induced to the backfill have been eliminated by the construction of the geotextile reinforced soil wall. It is assumed that earth pressures are still relatively high in the areas directly behind wing wall sections of the abutment. Temperature induced movements of the integral abutment are affecting the soils behind the wing walls and causing significant densification behind the abutment where the geotextile reinforced soil wall is not placed.

The use of a 6 inch gap appears to be the optimum gap width in order to both create a gap which is sufficient in width to allow the movements of the integral abutment to occur and to allow for ease of construction. Movement of the integral abutment in the order of 1 to 3 inches is common. The 6 inch gap allows this movement to occur while providing an excess gap width to compensate for any sliding which may occur in the geotextile reinforced soil wall after the supporting cardboard separator is removed. The 6 inch gap is adequate for maintaining a gap into which the integral abutment can cycle into and away from due to temperature induced changes in the bridge system. Some compensation may be needed for construction of the geotextile reinforced soil wall during high temperature periods, typically around 90 degrees Fahrenheit. Upon contraction of the bridge system during colder temperature periods, the gap may increase in width from what is initially constructed. If this gap increases in the order of 1 to 3 inches, depending on the bridge size, the gap size may become too large and may cause some problems to develop in the transition areas behind the wing wall sections of the integral abutment.

The transition area between the embankment soils behind the wingwalls and the air void between the integral abutment and the geotextile reinforced soil wall have shown to be a problem for construction and design. Inadequate placement of the geotextile material, and possibly the design of the transition between the embankment soils behind the wing walls and the geotextile reinforced soil wall, have been shown to allow densification of the embankment soils behind the wingwalls. Several of the embankments have exposed the 6 inch gap between the integral abutment and the geotextile reinforced soil wall. It has been observed that embankment material is migrating into the air void and filling the gap at the sides in some cases. Construction and design practices may need to be altered in order to correct the problem.

5.5 Costs

In additon to the evaluation of the performance of the various BEB design methodologies, a comparison of their costs is also of interest. The three design methods evaluated include the BEB design developed as a result of research project SD 90-03 (Schaefer and Koch, 1992), which does not use a MSE wall, the MSE BEB evaluated in this research project, and bridges constructed with rubber tire chips.

5.5.1 BEB Costs (No MSE Wall)

Costs of BEB prior to implementation of the MSE BEB were obtained from the SDDOT. Cost data from 13 bridges constructed in 9 different counties from 1994-1996 were used. The widths of these bridges varied from 28 to 89.3 feet, with an average width of 46 feet.

The cost figures from these projects are normalized to 1998 costs using the Consumer Price Index to adjust costs based on the annual rates of inflation. Site conditions, structure size, location, haul distances and other variables may affect the unit cost from one location to another, however the figures do present a reasonable estimate of the costs associated with this construction technique.

Table 5.3 shows the results of this analysis. This table shows the average quantity of materials and average unit cost from the 13 bridges. These quantity and unit cost figures were then used to calculate the average cost per bridge, which was $15,898.54.

Item	Unit	Avg. Quantity	Avg. Unit Cost	Avg. Cost
Granular BEB	Cu. Yd.	442.9	$28.64	$12,684.66
BEB Excavation	Cu. Yd.	161.2	$8.96	$1,444.35
BEB Underdrain Pipe	L.F.	198.6	$8.91	$1,769.53
				$15,898.54

Table 5.3 Cost Data For BEB

5.5.2 Bridges With Wrapped Face Geotextile Wall

Based on the results of bid lettings on 18 bridges bid during 1996-1998 a sampling of unit costs were developed. The bridges were located in various regions across the state as follows: Whitewood, Mitchell I-229 (5 bridges), Sioux Falls (12th St.), Highway 79 (2 bridges), Fall River, SD 50/I90, Union County I-29, Whitewood I-90/RR, Tulare US 281/RR, US 385/Fall River, and SD50/American Creek. Table 5.4 presents the results of this analysis adjusted to 1998 costs. Site conditions, structure size, location, haul distances and other variables may affect the unit cost from one location to another, however the figures do present a reasonable estimate of the costs associated with this construction technique.

Based on these figures, without regard to size and location, the average cost per bridge on installing a wrapped face geotextile wall with a six-inch gap behind the abutment is $26,946.10. For a bridge being repaired, as was the case with the Big Sioux River Bridge, this cost is an added cost of the bridge repair. Therefore this represents a more costly method then simply repairing a bridge and leaving the existing backfill in place. However, if the method prevents or retards void development, then it may prove to be a more cost-effective method when considering life cycle costs.

Item	Unit	Avg. Quantity	Avg. Unit Cost	Average Cost
MSE BEB	Cu. Yd.	459.8	$33.02	$15,182.60
BEB Excavation	Cu. Yd.	340.1	$7.22	$2,455.52
BEB Underdrain Pipe	L.F.	202.9	$9.50	$1,927.55
MSE Geotextile	Sq. Yd.	2891	$2.48	$7,169.68
Drainage Fabric	Sq. Yd.	63.1	$3.34	$210.75
Total Cost				$26,946.10

Table 5.4 Summary of Actual Bridge End Backfill (BEB) Costs with MSE Design

5.5.3 Bridges Constructed With Rubber Tire Chips

An estimate of bridge costs with rubber tire chips cannot be based on those obtained from actual costs since this type of backfill has not been placed in a full-scale structure. To estimate these costs, the line items used in construction of the MSE backfill will be adapted to a rubber tire chip backfill.

In order to install a twelve-inch vertical layer of tire chips behind an existing abutment, some of the backfill must be removed. Because the backfill is a granular material, it is unlikely that a twelve-inch wide trench could be dug without the cohesionless backfill sloughing into the hole. Therefore, the excavation would have to be overcut to a safe slope to allow placement of the drainage system, separation geotextile, and to place and compact the tire chips. In the White River and Big Sioux River Bridges, soil was excavated up to 30 feet from behind the abutment. This amount of excavation was required for proper placement of the MSE Geotextile. For installation of the rubber tire chip layer, this excavation could be conservatively estimated to be half of that required for the MSE wall. This would allow for safe slopes to protect construction workers in the excavation and to allow room for equipment. Therefore, the tire chip wall excavation costs and soil placement unit costs would be similar to the MSE backfill, but the quantity would be reduced by half.

The drainage system required by the MSE backfill would be the same for the rubber tire chip wall, and therefore the costs and volumes are assumed to be the same.

The placement of the fill would be done in a similar manner, meaning the soil would be placed in uniform lifts and compacted. It is also assumed that the backfill material will continue to be the same. However, in the tire chip backfill there is no installed MSE geotextile. Although this labor cost is eliminated, the tire chip backfill will require placement of a separation geotextile and placement and compaction of the tire chips. It will be assumed that, for estimation purposes, the elimination of the installation of the geotextile for reinforcement is offset by the cost of installing the tire chip layer. Therefore the fill placement costs are conservatively assumed to be the same for both methods.

The tire chip wall will need no MSE geotextile, so that cost is eliminated. However, the tire chip wall will require a separation geotextile to separate the tire chips from the backfill. For abutments that are 34 feet wide and 8 feet high, and allowing for installation of overlap seams, approximately 80-sq. yd. of geotextile will be needed per bridge. The separation geotextile would be the same drainage fabric used in the drainage system. Therefore there is an additional cost for drainage geotextile in the tire chip system.

Finally, the SDDOT has been able to obtain waste tire chips for free and no tire chip cost is assumed in this estimate. By comparison, the Maine DOT pays $1.00 per cu. yd. for shredded tire chips.

In summary, the tire chip backfill would require half of the earthwork of the MSE backfill, none of the MSE geotextile, an additional 80 sq. yd. of drainage geotextile, and that the cost of installing the tire chip layer is equal to the cost of placing the MSE wall. With those assumptions applied to the costs of the 18 bridges presented in table 5.4, the costs of a tire chip backfill for those bridges is presented in table 5.5. Discussion of the cost analysis is presented in section 5.6.3.

5.6 Conclusions

5.6.1 Model Study

The results of data collected for the model study and the field studies provide a wealth of information. Model study results were compared with past data in order to provide a comprehensive analysis of the results. This information could then be used to determine

Bridge	MSE Bridge End Backfill	Tire Chip Backfill
Whitewood	$21,737	$9,236
Mitchell	$35,385	$15,601
I-229 (5 structures)	$118,384	$47,446
(per structure)	$23,676	$9,489
Sioux Falls – 12th St.	$49,360	$15,069
Highway 79 (2 structures)	$65,305	$23,332
(per structure)	$32,652	$11,666
Fall River	$22,955	$9,705
SD50/I-90	$13,938	$6,685
I-29 Union County	$31,851	$15,229
I-90/RR Whitewood (2 structures)	$28,592	$11,339
(per structure)	$14,296	$5,670
US 281/RR Tulare	$17,314	$8,316
US 385/Fall River	$33,899	$13,627
SD 50/American Creek	$33,655	$15,405
Average (per structure)	$26,243	$10,611

Table 5.5 Cost comparison of MSE Backfill to Tire Chip Backfill

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the effectiveness of a model study backfill design in relation to past designs and the current backfill design incorporated by the SDDOT. Model study data could then be compared to field study results. The field study results provide information on the effectiveness of a new backfill design incorporated by the SDDOT.

The model study showed that the development of voids under the approach slab is reduced by using a vertical layer of rubber tire chips behind the integral abutment. A design incorporating the use of a layer of rubber tire chips behind the integral abutment provides a cushion into which the abutment can cycle into and away from without significantly affecting the backfill soils.

Earth pressure data shows that a composite design incorporating the use of vertical layer of rubber tire chips behind the integral abutment greatly relieves passive earth pressures induced on the retained fill, which reduces the mechanisms causing void development. Pressure cell readings indicated a final cell pressure of approximately 7.5-psi in the case where a rubber tire chip layer was incorporated in the backfill system. A final cell pressure of approximately 60 psi was recorded in the case no layer of rubber tire chips was behind the abutment. This indicates that the movements of the integral abutment into a backfill system incorporating a layer of rubber tire chips significantly lowers the earth pressures imposed on the integral bridge abutment. Earth pressures are greatly reduced and void development due to high earth pressures is significantly reduced for a backfill system incorporating a layer of rubber tire chips   Inclinometer data, approach slab elevations, and embankment stake elevations reveal similar results in that the cyclic movements of the integral abutment do not significantly affect the embankment soils. Deflections in the backfill evident through inclinometer tube movements were nearly eliminated in the case where a layer of rubber tire chips was used in the backfill system. Movements of the backfill soils were reduced by as much as 3 inches in some instances over the case where no layer of rubber tire chips was used. Embankment movements are greatly reduced and void development due to high lateral and longitudinal movements of the embankment soils are reduced with a backfill incorporating a layer of rubber tire chips.

Rearrangement of the individual rubber tire chips in the tire chip layer causes settlement in the tire chip layer as the abutment cycles. This allows movement to occur in the retained soils directly behind the rubber tire chip layer. This movement allows a void to develop under the approach slab that is similar to the case when no rubber tire chip layer is used. The significant difference is that in the case of the backfill incorporating a layer of rubber tire chips behind the integral abutment, the development of a void under the approach slab is reduced approximately one-half the amount that was recorded during the same time period for the case where no layer of rubber tire chips was used in the backfill.

Based on the results of the model study, it has been determined that the occurrence of a void under the approach slab is not eliminated by using a layer of rubber tire chips between the integral abutment and the retained embankment fill. Results have shown that the layer of rubber tire chips greatly decreases passive earth pressures that had previously been shown by Schaefer and Koch (1992) to develop behind the integral abutment. Other mechanisms that cause void development such as embankment bulging, approach slab uplift, backfill densification, and backfill deformation as passive failure occurs in the backfill are reduced or eliminated. Passive failure of the backfill system occurred in the past model test where no layer of rubber tire chips was used. In the case where a layer of rubber tire chips was used, passive failure did not occur in the backfill system. However, the development of a void directly behind the layer of rubber tire chip layer, under the approach slab, still develops as a result of the settlement of the rubber tire chip layer as the abutment moves.

Cracking in the embankments occurs at the transition point between the rubber tire chip layer and the backfill material behind the wing wall sections of the embankment. This was due to the movement of the 6-inch depth of embankment soil material placed over the rubber tire chip layer as a cover material for fire protection. The cracks develop as a result of the movement of the integral abutment cycling into and away from the backfill soils. This transition area poses a problem for design in that dissimilar materials are present with a large variation in movement between the two types of materials.

One of the most important aspects of the model test is indeed the development of the void under the approach slab. Void development is reduced, but not eliminated.

5.6.2 Field Study

The use of a fabric reinforced soil wall behind the integral abutment reduces the development of a void under the approach slab sections. Gap width measurement data has shown that an adequate gap width is being maintained in the areas of measurement through the warm cycles of abutment movement when movement of the bridge structure is at its highest expansion cycle.

Gap width measurements show that an initial movement of the geotextile reinforced soil wall toward the integral abutment occurs after the cardboard separator is removed. This movement is in the order of 0.5 to 2.0 inches in some instances. This reveals some problems associated with the construction of the soil wall. It is important to maintain significant tension in the soil wrapped layers in order to prevent movement of the soil wall after the cardboard is removed. This movement was most evident in the south abutment embankment of the White River Bridge. Some of these problems were alleviated before the construction of the north abutment embankment. Although, some movement is evident in all of the soil walls, especially within the first month after the cardboard is removed. Again, this was anticipated and a design incorporating a 6-inch gap was used in order to maintain an adequate gap width after this initial sliding occurs. The 6-inch gap appears to be optimum for allowing the movements of the bridge system to occur while providing adequate space for sliding of the geotextile reinforced soil wall to occur without closing the 6-inch gap.

The Big Sioux River Bridges gap width measurement data showed significantly less initial movement of the soil wall after the cardboard is removed than was the case for the White River Bridge. Since the design details of the geotextile reinforced soil wall for the three bridges are very similar, it can be concluded that the Big Sioux River Bridge reinforced soil walls were constructed with better results. The 6-inch gap has shown to be sufficient as evident from all abutment gap width measurements.

Gap width measurement data shows a definite trend in movement of the integral abutment in relation to the time of year in which the measurements are taken. In general, the warmest temperature periods show that the gap has closed due to the expansion of the bridge system. This expansion closes the gap during high temperatures that occur in the summer. Gap width measurements show the widening of the gap during the cold temperature periods as well.

Incorporating a gap between the integral abutment and the backfill soils has eliminated pressures induced to the backfill. This is the case for the full width of the driving lane, but not behind the wing wall sections of the abutment. There is no gap and soil wall constructed behind the wing wall sections of the integral abutment. This introduces a severe problem in that embankment soils behind the wing wall sections continue to be affected by the cyclic movements of the integral abutments. Temperature induced movements of the integral abutment are affecting the soils behind the wing wall sections of the abutment and are causing significant erosion problems in these areas.

Increased densification behind the wing walls allows for the exposure of the 6-inch gap under the approach slab. This allows erosion of embankment material to occur and allows the movement of soil material into the 6-inch gap, disturbing the backfill system at the sides of the approach slab.

Construction of the transition area between the geotextile reinforced soil wall and the embankment soils behind the wing walls must be performed as design specifications detail. Evidence supports that inadequate construction of the transition areas between the 6-inch gap and the embankment soils behind the wing walls introduces significant erosion and displacement of the embankment soils. In some instances, these soils are penetrating into the 6-inch gap, rendering ineffective the 6-inch gap at the sides of the approach slabs.

5.6.3 Cost Analysis

Each design evaluated in this project has different costs associated with their construction. For the BEB without reinforcement and MSE BEB systems, actual cost data were available for analysis. Although bid costs may vary due to structure size, location, availability of contractors and other variables, the figures used provide enough data to provide a basis for comparison. By averaging the material quantities and unit prices listed on each bid, an average bridge cost was developed for the BEB and MSE BEB systems. The results show the average MSE BEB system costs approximately $11,000 more then the BEB. This difference comes almost entirely from the use of the MSE geotextile in the MSE BEB and the higher costs associated with placement of MSE BEB.

In estimating the costs of a bridge end backfill incorporating the use of rubber tire chips, actual bid costs were not available. The analysis presented shows this system could be constructed for $5,000 less than the BEB without reinforcement. This cost difference is based on the assumption that less excavation and thus less soil placement would be required. However, if the earthwork quantities were similar to the quantities used in the unreinforced BEB, then the costs of the two systems would be very similar.

In summary, the MSE BEB has a higher initial cost than the unreinforced BEB, but should have a lower life cycle cost due to reduction in void development and the subsequent maintenance that requires. The rubber tire chip bridge end backfill is estimated to cost less than the unreinforced BEB provided less earthwork is required. As the earthwork quantities increase, the costs will than approach that of the unreinforced BEB. However, with regards to the rubber tire chip system, the estimate presented in this research is qualitative, not quantitative.

CHAPTER 6

IMPLEMENTATION RECOMMENDATIONS

6.1 Introduction

The purpose of this report was to monitor and evaluate the performance of bridge end backfills designed with a wrapped face geotextile wall providing a gap between the bridge abutment and the reinforced fill. Additionally, this research was to examine the use of alternative backfill materials through the conduct of a model scale test.

6.2 Recommendations

Recommendation One – Use of alternative backfill:

The use of select backfill in the reinforced soil wall behind the integral abutment may not always be necessary. The use of this backfill was initiated to increase the passive resistance of the soil when it was placed directly against the integral abutment and to assure adequate compaction by the contractor in a restricted and confined area. By eliminating the passive pressures and through reinforcement of the soil with the geotextile, alternative backfill material may prove to be suitable. Implementation of this recommendation requires that the alternative material can be adequately compacted in restricted areas and the use of such materials would provide a cost advantage to the state.

Recommendation Two – Reduction in gap width:

A reduction in the installed gap width may be warranted when the initial gap is constructed in the summer (at times of maximum bridge expansion). When a contractor experienced in placement of wrapped face geotextile walls and/or field inspection can ensure the wall is properly constructed (to minimize post construction movement), a six inch gap may be overly conservative. Measurements on the Big Sioux River Bridge show no gap measurements below five inches. If a bridge is constructed during times of maximum expansion, then the installed gap width may be reduced to 3 to 4 inches.

Recommendation Three – geotextile embedment:

Field monitoring and inspection during the construction of the wrapped face geotextile wall is critical to the performance of the system. A poorly placed wall will deflect laterally during construction causing reduction in gap width and surface settlement. The geotextile should be well embedded into the soil and each layer should be tightly wrapped and tensioned prior to soil placement. Some of these problems should be reduced with the design changes implemented in the 1999 MSE BEB design.

Recommendation Four – Erosion behind the wing walls:

A new design of the backfill and wing wall interface needs to be developed to prevent erosion of the backfill material. During bridge expansion, the wing wall pushes into the backfill and then pulls away during contraction. This creates a gap between the wing wall and soil providing a conduit for drainage. This water causes erosion and some of the soil is being washed into the gap behind the bride abutment. Because of this, a better physical barrier needs to be installed to protect the gap from infilling. Additionally, a gap or some compressible material may be needed behind the wing wall to prevent gaps from forming behind the wing wall. A rubber tire chip layer surrounded by a separation geotextile may be a suitable candidate design.

Recommendation Five – Use of rubber tire chips:

The design of a full-scale integral abutment bridge using a layer of vertical rubber tire chips and no geotextile reinforced fill should be developed and tested in the field. This design, if effective, may be less costly and easier to construct then the geotextile wall system tested in this research. Recent advances by other State DOT's has shown increased use and confidence in the use of rubber tire chips for reducing fill weight and reducing lateral soil pressures on structures. The use of tire chips in this application would require careful monitoring of tire chip compaction to prevent the settlement problems observed in the model study portion of this research.

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