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USE OF CONTROLLED LOW-STRENGTH MATERIAL AS ABUTMENT BACKFILL

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16 Abstract

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Research was performed in a series of laboratory tests and finite element analyses to determine the lateral pressures generated by CLSM. The effect of CLSM on drainage material was also evaluated in the physical tests. For the physical testing an apparatus was constructed that was six feet tall, two feet deep, and two feet wide. One wall of the apparatus was instrumented with pressure cells at varying heights. Drainage material was also placed on the instrumented wall. The apparatus was then filled with CLSM. Fill was placed in the apparatus and pressures recorded with time.

Lateral pressures in all studies peaked immediately after CLSM placement at pressures approaching full fluid pressure before dropping sharply to near zero. Pressures tended to decline most slowly at the center of the fill, which was consistent with observations of previous researchers.

Finite element analyses were conducted for several CLSM-wall configurations, including the laboratory apparatus and an actual bridge design provided by KDOT with a series of loading configurations. Results from the finite element analyses predicted full fluid pressure when the fill was placed and 25-35 percent of the full fluid pressure after curing. These results were also consistent with previous field observations.

Settlement within the fill should be minimal based on the literature review and finite element analyses. However, the potential for settlement in the foundation soils below the fill is not addressed by the use of CLSM and may still be significant. Comments on how settlement of foundation soils could be addressed are included in Chapter 7.

Based on the results of the literature review, physical testing, and computer modeling, it was recommended that the fill be modeled as a fluid during placement and that a much reduced equivalent fluid pressure be used for estimation of lateral pressures after curing. Recommendations were also made regarding the possibility of wall rotation away from the fill, which could enable fill in a fluid state to migrate into a gap between the abutment and lower lifts of fill that had set up and potentially causing higher than expected lateral pressures. No recommendations for changing the specifications for the drainage material were made as it performed very well in the physical testing.

17 Key Words
Abutments, Bridge, Compaction, Drainage Material, Finite Element, Settlement, and Soils

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THE KANSAS DEPARTMENT OF TRANSPORTATION
TOPEKA, KANSAS

UNIVERSITY OF KANSAS
LAWRENCE, KANSAS

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PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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ABSTRACT

Use of Controlled Low-Strength Material (CLSM) behind bridge abutments has become common practice to avoid the problem of settlement when using compacted soils. CLSM solves the settlement problem within the fill but raises separate concerns. The lateral pressures that CLSM exerts on the bridge abutment are not well understood. Another concern is the potential for clogging of the drainage material on the bridge abutment. This could occur if the fly ash, cement, or water paste flows through the drainage fabric and hardens.

Research was performed in a series of laboratory tests and finite element analyses to determine the lateral pressures generated by CLSM. The effect of CLSM on drainage material was also evaluated in the physical tests. For the physical testing an apparatus was constructed that was six feet tall, two feet deep, and two feet wide. One wall of the apparatus was instrumented with pressure cells at varying heights. Drainage material was also placed on the instrumented wall. The apparatus was then filled with CLSM. Fill was placed in the apparatus and pressures recorded with time.

Lateral pressures in all studies peaked immediately after CLSM placement at pressures approaching full fluid pressure before dropping sharply to near zero. Pressures tended to decline most slowly at the center of the fill, which was consistent with observations of previous researchers.

Finite element analyses were conducted for several CLSM-wall configurations, including the laboratory apparatus and an actual bridge design provided by KDOT with a series of loading configurations. Results from the finite element analyses predicted full fluid pressure when the

fill was placed and 25-35 percent of the full fluid pressure after curing. These results were also consistent with previous field observations.

Settlement within the fill should be minimal based on the literature review and finite element analyses. However, the potential for settlement in the foundation soils below the fill is not addressed by the use of CLSM and may still be significant. Comments on how settlement of foundation soils could be addressed are included in Chapter 7.

Based on the results of the literature review, physical testing, and computer modeling, it was recommended that the fill be modeled as a fluid during placement and that a much reduced equivalent fluid pressure be used for estimation of lateral pressures after curing. Recommendations were also made regarding the possibility of wall rotation away from the fill, which could enable fill in a fluid state to migrate into a gap between the abutment and lower lifts of fill that had set up and potentially causing higher than expected lateral pressures. No recommendations for changing the specifications for the drainage material were made as it performed very well in the physical testing.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Chapter 1

Introduction

Controlled Low Strength Material (CLSM) is a self-compacting cementitious material that is used primarily as backfill in place of compacted soil. The use of Controlled Low Strength Material has increased dramatically over the last two decades. CLSM can be used for backfill in utility trenches and retaining walls, mud jacking, floor/footing support, pavement bases, pipe bedding, subbase stabilization, abandoned tank fills, and sink hole fills.

The research described in this report examined the performance of CLSM behind bridge abutments. Use of CLSM behind bridge abutments has become common practice to avoid settlement, which can occur when using compacted soil as abutment backfill. Settlement is especially noticeable when compacting behind a bridge abutment where it can lead to the development of a bump at the beginning and end of the bridge. A reduction of settlement is one of the biggest advantages of CLSM.

CLSM solves the problem of settlement within the fill but raises separate concerns. The lateral pressures that CLSM exerts on the bridge abutment are not well understood. Another concern when using CLSM is whether it clogs the drainage material on the bridge abutment. This could occur if the fly ash, cement, or water paste flows through the drainage fabric and hardens.

1.1 Characteristics of CLSM

The American Concrete Institute has issued a comprehensive report on CLSM (ACI 229R-99 Controlled Low-Strength Materials) from which most of the information in this introduction was obtained. CLSM is defined as any combination of materials that results in a compressive strength of 1200 psi or less. The combination of materials includes cement, fly ash, aggregate, and water. Admixtures may also be used when conditions require a mix with special properties. Cement provides the cohesion and strength for a CLSM mixture. Type I or Type II cement is normally specified. Fly ash is a by-product of coal burning power plants. Any type of fly ash may be used in CLSM. Fly ash may improve flowability and reduce bleeding, shrinkage, and permeability when added to any type of concrete mix. Fly ash has also been used to increase the strength and reduce volume change in soils. In a CLSM mix the fly ash is used primarily to improve flowability and shrinkage. A mix with a high fly ash content will also be less dense than one with a high aggregate content.

Aggregates usually make up most of a CLSM mixture. Any aggregate that is approved for concrete mixtures can be used in a CLSM mixture, however the flowability requirements of the material usually require the use of a rounded aggregate. Many mixes accomplish this by using local natural sand. Any water may be used that is approved for concrete mixtures. The admixture that is most used is an air entraining agent. Air entrained mixtures improve workability, shrinkage, bleeding, segregation, and increase strength while lowering unit weight.

CLSM can be mixed anywhere that concrete is mixed. Theses sites include batch mix plants, ready-mix concrete trucks, and mobile concrete mixers. The variety of places that are able to mix CLSM makes it readily available. CLSM is transported to the site in a ready-mix truck. These trucks agitate the CLSM to keep the material in suspension and place the mix. The mix can be moved around the site by chutes, conveyors or buckets.

In some situations CLSM can be placed all in one lift. If it cannot be placed in one lift then it may be placed in multiple lifts with each layer allowed to harden before the next is poured.

CLSM mixtures can be designed to be permanent or excavatable. If a structural fill is needed a mixture can be designed to have a compressive strength of 1200 psi. This mix would have to be removed in a process similar to that for concrete removal. Most mixes have a compressive strength of 300 psi or less, while most excavatable mixes have a compressive strength of 50-100 psi. These low strength mixes can be excavated with conventional digging equipment. This is especially useful in utility trenches or any project in which future removal of the CLSM may be required.

1.2 Advantages of CLSM

Soil must meet compaction specifications to be used as fill. Select fill may not be available onsite and must be transported in from another location, which can be expensive. When using soil as backfill it must be compacted. Backfill is compacted in small layers so the layer will be compacted throughout its full thickness. Compaction of soil adjacent to abutments, in trenches, and around similar structures requires special equipment that limits productivity and takes a significant amount of time and effort to complete. The newly compacted material must also be tested for density, which adds to the cost of a project. Compaction may also be delayed during inclement weather.

CLSM is a self-leveling, self-compacting material so no compaction is needed. Since no compaction is needed the special compaction equipment does not have to be mobilized. CLSM consistently achieves density requirements while compacting under its own weight; therefore extensive field-testing is not needed, reducing construction costs and construction time, although slump and cylinder tests may be requested. Another time and cost saving advantage of CLSM is that it is often possible to pour an entire project as a mass instead of in a series of lifts. Time is saved because the mix can be designed to set quickly and support traffic loading within hours. The limited time of construction can make it easier to schedule around weather delays. CLSM may be placed when there is standing water in the excavation and by following the guidelines for cold weather concreting when temperatures are extremely low.

1.3 Disadvantages of CLSM

CLSM also has some disadvantages. Published research on the lateral pressure the CLSM exerts on retaining structures is limited. CLSM is placed in nearly a liquid condition, which could result in its generating large lateral pressures on these structures. Although a benefit of CLSM is that it flows around pipes and oddly shaped structures, this characteristic may cause underground pipes and cables to float when the fill is placed. This requires these utilities to be tied down prior to placement.

1.4 Report Objectives and Summary

Research was performed in a series of laboratory tests to determine the lateral pressures generated by CLSM. The effect of CLSM on drainage material was also evaluated in these tests. An apparatus was constructed that was six feet tall, two feet deep, and two feet wide. One wall of the apparatus was instrumented with pressure cells at varying heights. Drainage material was also placed on the instrumented wall. The apparatus was then filled with CLSM.

A series of finite element analyses were conducted concurrently with the physical testing. The finite element software was used to model the lab conditions first so the computer model results could be compared with the results from the physical testing. The finite element package was then extended to model a bridge configuration provided by KDOT.

Chapter Two covers research that has been conducted on abutments backfilled with CLSM. The testing procedure is briefly covered along with the results and conclusions. Chapter Three covers the equipment and procedures that were used in this research project. The results of these procedures are covered in Chapter Four. Interpretations of the results are discussed in Chapter Five. Chapter Six contains a summary of a finite element analysis of several abutment and loading configurations backfilled with CLSM. Chapter Seven contains the conclusions and recommendations reached based on the results of the project.

Chapter 2

Literature Review

This chapter contains a summary of published literature that describes attempts to determine the lateral pressure that CLSM places on bridge abutments. Previous research has been conducted by the Wisconsin Department of Transportation, the Pennsylvania Department of Transportation, the New Hampshire Department of Transportation, and the Oklahoma Department of Transportation. Most of these studies measured lateral pressure and all measured settlement and performance of CLSM versus granular fill. These studies are summarized below.

2.1 Wisconsin Department of Transportation Study

The Wisconsin Department of Transportation study (Wilson 1999) compared the effectiveness of using CLSM as backfill for bridge abutments. For this study two bridges were instrumented and built in 1996. The west end of both structures was backfilled with CLSM. The east ends of both structures consisted of granular materials compacted utilizing conventional compaction equipment. The performance of the two bridges was monitored for three years by taking levels and comparing settlement profiles. Pavement distresses were also monitored. The two bridges, CTH D and CTH G, were low volume.

The original dimensions for placement of CLSM for bridge CTH D were for five feet of fill next to the abutment tapered back 25 feet to ground level. However, site conditions caused the alteration of the dimensions to three feet of fill tapered back 15 feet. The fill was placed using standard ready-mix trucks. After reaching the required elevation more fill was placed on the center of the roadway to create a crest to help drainage. A total of 45 yards of fill was used. The CLSM consistency varied with each truck. The beginning loads of CLSM appeared to be more fluid than those at the end. Bridge CTH G was also designed for the CLSM to be five feet deep next to the abutment tapered back 25 feet to ground level. No changes to the dimensions were needed during construction. This mix was also delivered to the site by ready-mix trucks. This mix had more consistent properties than bridge CTH D because the amount of water was decreased from the start. The CLSM mix utilized for this project was 68.5% foundry sand, 10.9% class C fly ash, 1.4% cement, and 19.2% water.

Base line profiles were taken immediately after construction on November 19, 1996. Data points were taken at the center and edge lines of both sides of the bridge. Measurements were taken at one foot from the edge of the bridge and every four feet until the entire 25 feet was measured. Readings were repeated on February 3, 1997, August 25, 1997, August 12, 1998, March 2, 1999, and August 23, 1999. At eleven months no cracking had occurred in the pavement. Slight pavement rutting was observed at this stage.

Five adult evaluators were asked to rate the performance of the approaches as they passed over the bridges in a Dodge minivan. If the approach did not have a bump before the start of the bridge it was assumed that little or no settlement had occurred. These evaluators were not told which side of the bridge had been built with CLSM or compacted granular material. All five evaluators rated the CLSM side of bridge CTH G to have less settlement and smoother ride. The evaluators observed no differences between abutments when driven over bridge CTH D.

From the elevation profiles it was determined that the CLSM sides fared slightly better than the conventional granular fill, however the difference in performance was not considered significant.

2.2 Pennsylvania Department of Transportation Study

The Pennsylvania Department of Transportation project (Newman 1993) included instrumenting bridge abutments with pressure cells. The abutments were then backfilled with CLSM. Vertical and horizontal measurements were taken throughout the test. The first site was composed of the north abutment of the Mahoning Creek Bridge and a nearby retaining wall. The second site included the north and south abutment of the Panther Creek Bridge.

The Mahoning Creek Bridge is a two-span structure 245 feet long and 32 feet wide. The north abutment was backfilled with CLSM and the south abutment was backfilled with granular material and broken rock. The north abutment was supported by a foundation of drilled-in steel pin piles bearing in bedrock. The south abutment used a spread footing bearing on the sandstone piers of the original bridge. The depth of the north abutment was 17 feet. An eight-inch granular base was placed above the CLSM and four to six inches of asphalt was placed on the base.

Six pressure cells were placed flush to the finished abutment wall to measure horizontal pressures. Two pressure cells were installed flush with the abutment footing to measure vertical pressures. The cells had a range of 25 psi with corrections for temperature built into the cells. 19 survey points were also established over the north abutment. Three survey points were established over the south abutment. The survey points were used to measure any vertical movement of the pavement. The mix used at this site consisted of 12.7% cement and 87.3% fly ash. Drainage material was placed over 50% of the abutment face. The Drainage material was a type of geosynthetic wall drain mounted in vertical strips. The CLSM was placed in five lifts by pumping from an onsite batch plant. Each of these lifts was allowed to cure overnight. The lifts varied in thickness from one to six and one-half feet. The CLSM had inconsistent compressive strengths varying from 493 psi to 1707 psi. This was caused by the difficulty the contractor encountered in controlling mix proportions using an onsite batch plant.

The Panther Creek Bridge is a single-span structure with a length of 33 feet and a width of 30 feet. The abutments were both 13 feet high and were supported by spread footings bearing on rock. The north abutment was backfilled with a CLSM material consisting of 5% cement and 95% fly ash. The material was placed in five lifts with each lift permitted to harden overnight prior to pouring the next. The lifts were placed in lifts varying in thickness from one to four and one-half feet. The material behind the south abutment consisted of 95% sand and 5% cement by dry weight. For this abutment the CLSM was placed in four lifts, each three feet thick. Five pressure cells measured horizontal pressures and one cell measured vertical pressures of the north abutment. No instrumentation was placed on the south abutment. Geosynthetic material was placed over 50% of the abutment wall. 63 survey points were established to check settlement.

The Mahoning Creek Bridge was surveyed again nine months after construction. The north abutment settled about one-quarter of an inch. The bridge deck settled one-half inch and the granular fill behind the south abutment settled one-half inch. Seven months after construction the Panther Creek Bridge was resurveyed and the north abutment was found to have heaved one-half inch, and the south abutment had both heaved and settled one-half inch in different areas.

The vertical pressures under all CLSM backfilled abutments conformed to the theoretical fluid pressure at the approximate fluid density of the backfill to a depth of 10.5 feet. At greater depths recorded pressures went above the theoretical maximum value of fluid pressure, presumably because of arching during curing. After backfilling was completed, a reduction in vertical pressures to well below the theoretical value was observed.

Horizontal pressures at a point of two and one-half feet above the top of the footing were watched closely as the point of maximum theoretical pressure. Full fluid pressures were measured at that point on the Mahoning Creek Bridge each time a new lift of CLSM was poured until a depth of 13 feet was reached. Since the backfill was placed in lifts and allowed to cure overnight between lifts, shrinkage of the CLSM away from the abutment temporarily reduced the lateral pressure measured by cell C3 as shown in Figure 2.1. However, it appears that the placement of the next lift displaced the abutment wall and allowed full fluid pressures to develop again as indicated by points A and B in Figure 2.1. This phenomena was repeated until the placement of the fifth lift, as the fluid CLSM was not able to penetrate to the depth of cell C3 and the lateral pressure decreased to about zero. Pressures at both abutments were consistent with the theoretical lateral pressures.

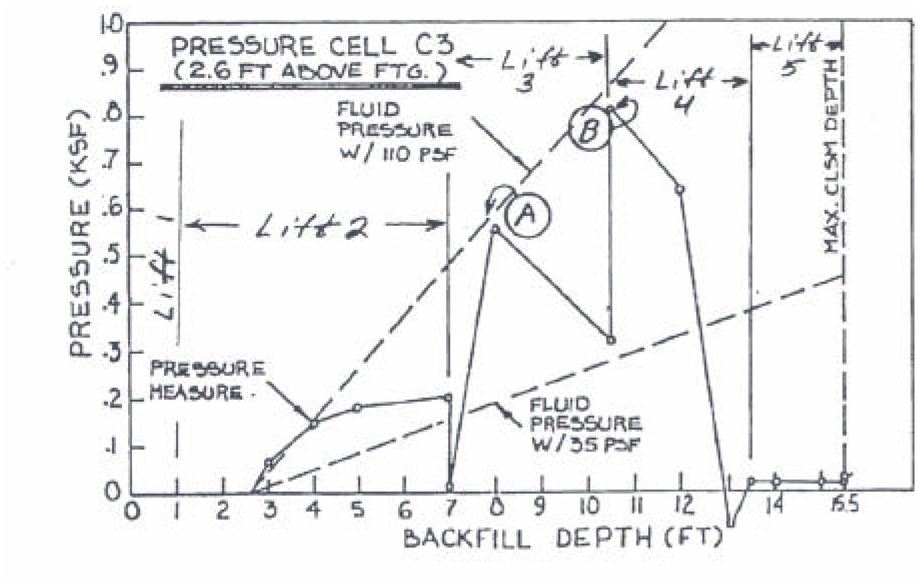

FIGURE 2.1: Horizontal pressures during construction of the Mahoning Creek Bridge (Newman et al., 1993)

2.3 New Hampshire Department of Transportation Study

The University of New Hampshire performed a research project (Gress 1996) for the New Hampshire Department of Transportation in which a test bridge abutment was constructed at Coastal Materials Corporation’s Farmington, New Hampshire plant. This was done by constructing a concrete wall traversing Coastal Materials entrance road. All traffic entering or leaving the plant had to drive over the wall, assuring a continuous flow of heavily loaded truck traffic.

The CLSM was to be approximately five feet tall by 24 feet wide. The east side was backfilled with a granular base material and compacted according to New Hampshire Department of Transportation specifications (98% of modified proctor). The west side was backfilled with CLSM. Both sides of the bridge were covered with a five-centimeter layer of asphalt pavement. The CLSM mix used consisted of 81.75% sand, 1.75% cement, and 16.5% water.

Weekly visual inspections were performed on the bridge abutment. Special attention was given to differential settlement between the concrete wall and the asphalt pavement. After seven months 94 points were surveyed again. This data showed that both the granular fill and CLSM had settled. The settlement of the granular fill ranged from 0.4 – 0.6 inches. The maximum measured settlement was one inch. The CLSM settled an average of approximately 0.2 inches. The maximum measured settlement was 0.6 inches.

The CLSM was determined to have settled less than the granular fill and have more uniform settlements throughout the backfill. CLSM was considered superior to granular fill. The New Hampshire Department of Transportation expected CLSM to perform better on actual bridge abutments because the asphalt was placed directly on the CLSM fill, exposing the CLSM to higher stresses than would normally be seen.

2.4 Oklahoma Department of Transportation Study

Snethen and Benson, from Oklahoma State University, evaluated the performance of a bridge abutment backfilled with CLSM for the Oklahoma Department of Transportation (Snethen

1998). The back of the abutment walls had total pressure cells to measure the lateral earth pressure caused by the approach embankments. They also used amplified liquid settlement gauges to monitor settlement. The CLSM mix design used consisted of 77% granular material, 1.4% Cement (Type I), 7.2% fly ash (Class F), and 14.4% water.

CLSM was used on the north end of the bridge. The total thickness of the CLSM to be used was eight feet. A six-inch layer of select aggregate base course was placed on top of the CLSM. On top of the aggregate a ten-inch layer of full depth asphalt was placed. The CLSM was delivered in ready-mix trucks. Fill was placed from two trucks at the same time. The total volume of fill placed was 207 cubic yards in four and one-half hours.

The total pressure cells and amplified liquid settlement gauges were installed prior to construction of the CLSM approach. The three pressure gauges were placed at the top, middle, and bottom of the wall. The total pressure after twenty months dropped to less than one psi for the top two gauges and about one psi for the bottom gauge. The surface survey showed no significant movement. The amplified liquid settlement gauges indicate the total settlement beneath the CLSM to be 0.13 feet (1.56 inches) at the offset and 0.34 feet (4.08 inches) at the centerline. The peak lateral pressure on the abutment wall was 1.4 psi at the top gauge, 2.9 psi at the middle gauge, and 2.4 psi at the bottom gauge.

2.5 Summary and Insight

The above four studies show that CLSM can have the same or better performance than conventional granular fill in terms of abutment settlement and lateral pressures. No unusual negative consequences of having a stiffer material behind the bridge abutment were observed.

Based on the foregoing literature review, it was decided that the main goals of this study should focus on the evaluation of CLSM settlement and lateral pressure, and confirm that drainage materials were not clogged by the CLSM. Both physical and computer modeling was performed during the research to determine the lateral earth pressures generated by low strength CLSM.

Chapter 3

Physical Testing Plan and Procedure

Engineers are increasingly using CLSM as backfill behind bridge abutments to avoid settlement problems when compacted natural fill is used. The lateral pressure that CLSM exerts on the bridge abutment is not well understood. Common practice is to assume that the CLSM will exert a fluid pressure equivalent to the unit weight of the CLSM. Another unknown with CLSM is whether it will clog the drainage fabric attached to the back of the abutment.

A series of laboratory tests were conducted to evaluate lateral pressures exerted by CLSM on an abutment and the effectiveness of the drainage system. An apparatus was built to evaluate the lateral pressure exerted by the CLSM. Drainage material was installed in the apparatus and its performance was evaluated under loading. Compressive testing was also performed on each batch of CLSM.

3.1 Test Apparatus

An apparatus was designed and constructed to evaluate the lateral pressures generated by CLSM and performance of the drainage system as shown in Figures 3.1 and 3.2. The apparatus had a height of six feet and a width of two feet. These dimensions were chosen to permit use of a stiff wall with enough room for instrumentation. A depth of two feet was selected to ensure the full active wedge for a granular material of the same dimensions could develop.

FIGURE 3.1: Apparatus Top View

FIGURE 3.2: Apparatus Side View

The apparatus was built of wood. The walls were made from 2x4 pine lumber for support and covered with ¾-inch plywood as shown in Figure 3.2. All of the woodwork was held together by a combination of 1-inch and 2.5-inch outdoor screws. The apparatus was supported by a 2x4 support system. This not only supported the apparatus but also served as a collection area for water that had drained out of the fill. Additional stiffness was also supplied by a clamping system. The clamps were placed at a height of two feet and four feet for Tests 1 and 2, performed on August 30, and September 13, 2002 respectively, and two and one-half feet and four feet for Test 3 on February 19, 2003. These supports were constructed out of 2x4 boards with ¼ or $^{5}/_{16}$-inch threaded rod running between them.

3.2 Drainage System

Two types of drainage systems were used during testing. Carlisle Coatings and Waterproofing SURE-DRAIN-HWY drainage system was used on Test 1. MiraDRAIN 6000/6200 drainage system was used on Tests 2 and 3. Both systems meet or exceed the Kansas Department of Transportation Special Provision for abutment strip drains. MiraDRAIN 6000/6200 was used in two tests because its properties would make it more susceptible to the type of damage being evaluated. The drainage system was installed on one wall using half-inch screws as seen in Figure 3.3. A cross-section of the material is shown in Figure 3.4. The properties of both drainage systems are presented in Table 3.1.

FIGURE 3.3: Drainage System Installation (after testing)

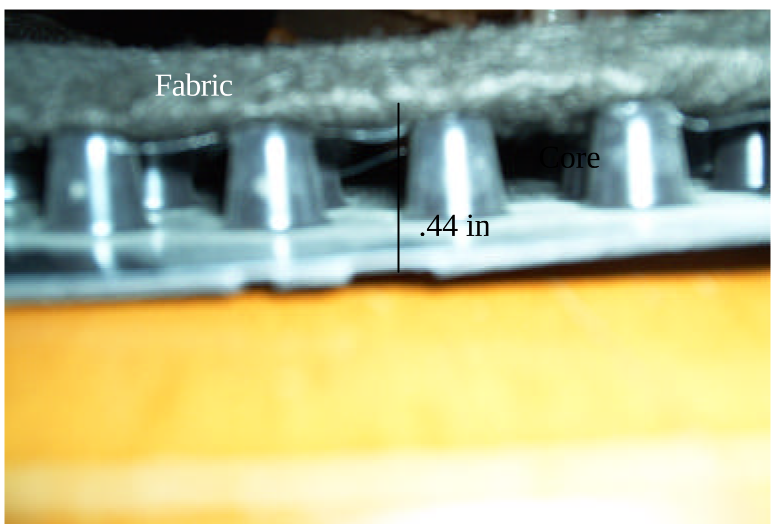

FIGURE 3.4: Drainage System Cross-Section

TABLE 3.1: Drainage Properties

CORE	SURE-DRAIN-HWY	MiraDRAIN 6000/6200
Thickness	0.44 (inches)	0.40 (inches)
Compressive Strength	27,550 (psf)	15,000 (psf)
Flow Rate (System)	16 (gpm/ft)	12.5 (gpm/ft)
FABRIC		
Standard Opening	0.17 (mm)	.21 (mm)
Flow rate	170 (gpm/ft^2)	140 (gpm/ft^2)
Puncture Resistance	70 (lbs)	65 (lbs)

3.3 Sensors and Data Acquisition

Pressure cells were mounted on one wall of the apparatus. SENSOTEC manufactured the Model CP200 pressure cells. The pressure cells are rated for pressures from 0-25 psig. The pressure cells were mounted at varying heights as shown in Figure 3.5. For Test 1 and 2 the cells were mounted at heights of one, three, and five feet above the floor of the apparatus. For Test 3 the cells were mounted at heights of one, two, and three feet.

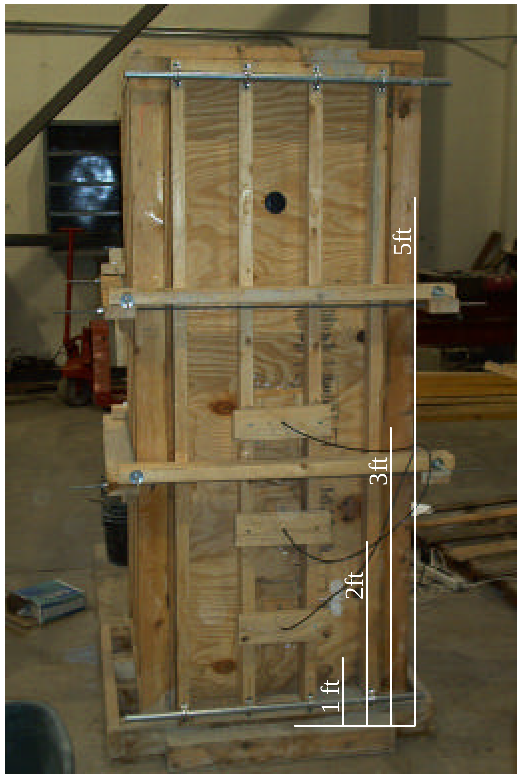

FIGURE 3.5: Pressure Cell Height (Image is from Test 3)

The cells were connected to a WaveBook Model 512 data acquisition device. The wiring coming from the cells was connected to the data acquisition device as seen in Figure 3.6. The data acquisition device was connected to a laptop and data was recorded utilizing the WaveView software package. Recording rates varied with each test. Data was recorded each minute for the duration of Test 1. For Test 2 the testing the rate was a data point every second. Data was recorded each 0.1 second for the first 30 minutes and then every one minute for the remainder of the test for Test 3.

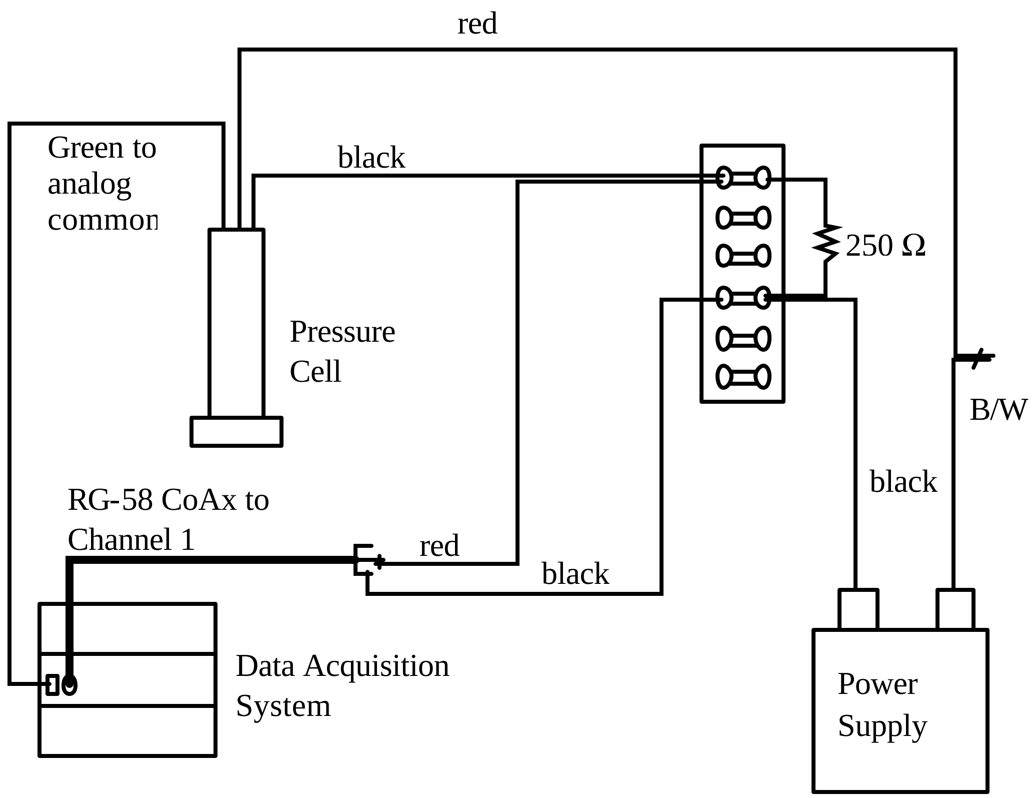

FIGURE 3.6: Data Acquisition Wiring Diagram

3.4 Test Procedure

Each test was conducted using the same procedure. The portions of the apparatus that would be in contact with the CLSM, except for the drainage material, were treated with form release. The apparatus was then assembled. One cubic yard of CLSM Mix #1807 was supplied by Lawrence Ready Mix. The mix design is shown in Table 3.2 and the properties of the mix are shown in Table 3.3. The CLSM was delivered in a ready mix truck and placed in a concrete bucket with a capacity of approximately 2/3 of a cubic yard. The bucket was then lifted over the apparatus with an overhead crane. The bucket was opened until all of the CLSM had been placed. The bucket was then taken back to the ready mix truck for the final third of the mix. The apparatus was filled with the second load until a height of six feet was reached. The rest of the material was used to fill cylinders for compressive testing.

TABLE 3.2: Mix Design

Material	Weight (lbs)	Volume (cubic feet)
Cement / Type I/II	50	.25
Fly Ash / Class C	200	1.46
Holliday Sand	3001	18.36
Water	281	4.50
Air		2.43
Admixture W.R. Grace Daravair 1400		2.5 (oz)

TABLE 3.3: Calculated Mix Properties

Water/Cement Ratio (lbs/lbs)	1.12
Slump (in)	8.00
CLSM Unit Weight (pcf)	130.8

3.5 Slump and Compressive Testing

Preparation and testing of the CLSM was performed in accordance with ASTM D 4832. Single use cylindrical molds with a height of eight inches and a diameter of four inches were used. The cylinders were capped with new elastomeric pads for compression testing that met the qualifications needed from Practice C 1231 with a maximum reduction of strength less than 20 percent. The LoadTrac II, made by the GEOCOMP Corporation, was used to load the cylinders to failure. The cylinders were placed on top of a self-leveling device during testing. A picture of the test setup can be seen in Figure 3.7.

Slump testing was performed in accordance with ASTM C 143.

FIGURE 3.7: Compression Testing

Chapter 4

Physical Testing Results

This chapter covers the results of the testing program. These results include lateral pressures that developed during fill placement and their changes with time, compressive strength and slump of the fill, and performance of the drainage material.

4.1 Lateral Pressures

Lateral pressures were recorded in each of three tests. Figures 4.1 and 4.2 show the lateral pressure versus time and pressure versus depth relationships for Test 1. Figures 4.3 – 4.8 show the pressure versus time and pressure versus depth relationship for Tests 2 and 3, respectively. For Tests 2 and 3 the early pressure versus time data is displayed as a separate figure for more detail.

Peak lateral pressures exerted by the CLSM for Tests 1 – 3 are 1.81, 3.29, and 3.37 psi, respectively. The pressures for Tests 2 and 3 are approximately equivalent to a fluid with the unit weight of the CLSM at the peaks. The pressure distribution is triangular at the peak pressure. The pressures drop off with time until they eventually reach zero. As the pressures decline the maximum lateral pressure shifts towards the middle of the apparatus.

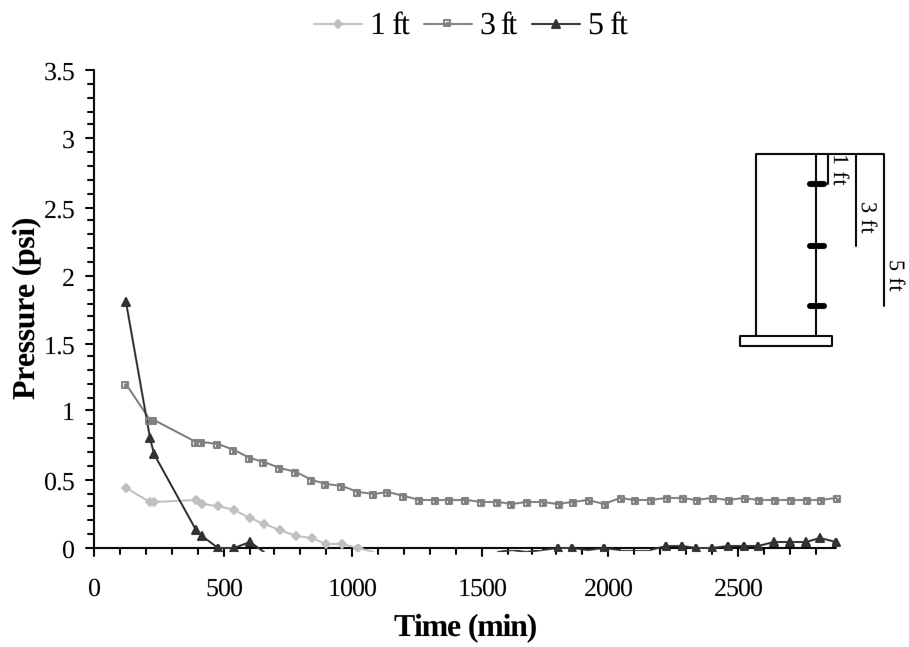

FIGURE 4.1: Pressure vs. Time (48 hours) for Test 1

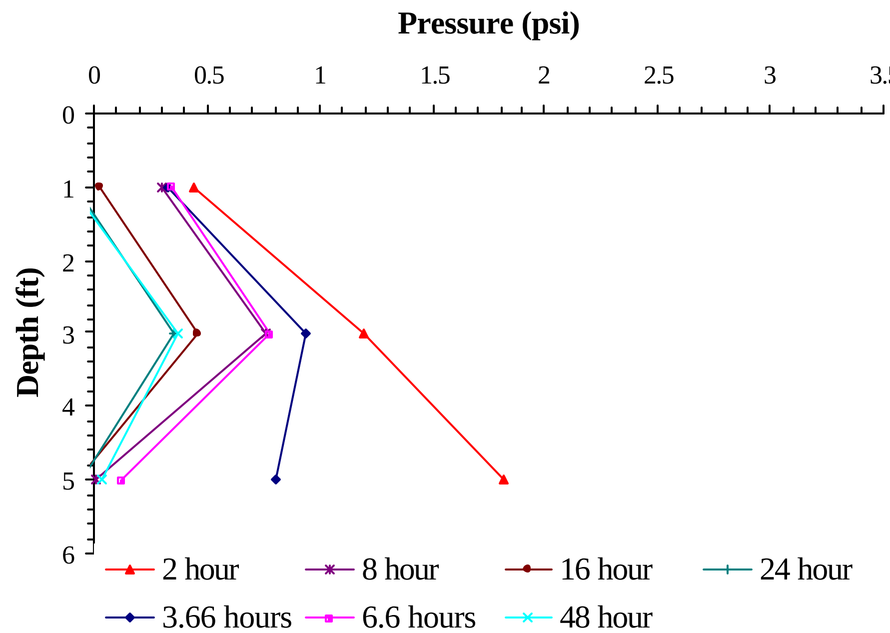

FIGURE 4.2: Pressure vs. Time (48 hours) for Test 1

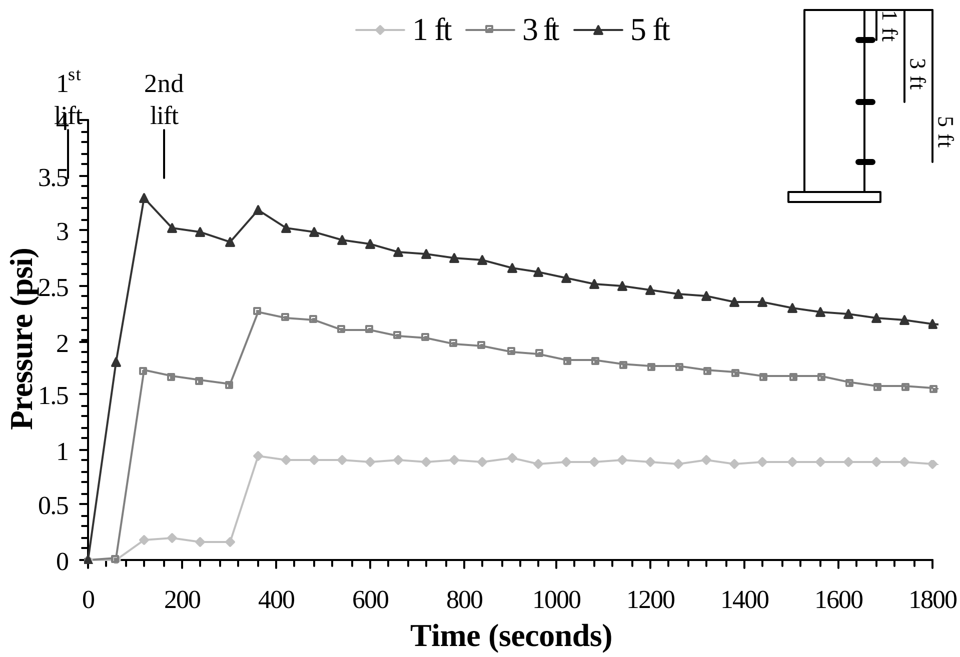

FIGURE 4.3: Pressure vs. Time (First 30 min) for Test 2

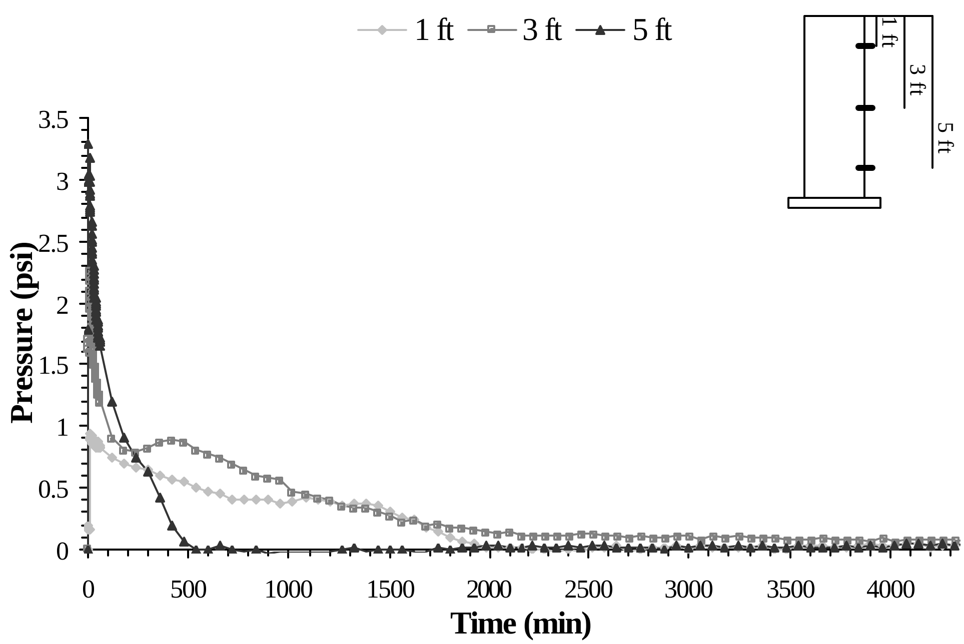

FIGURE 4.4: Pressure vs. Time (72 hours) for Test 2

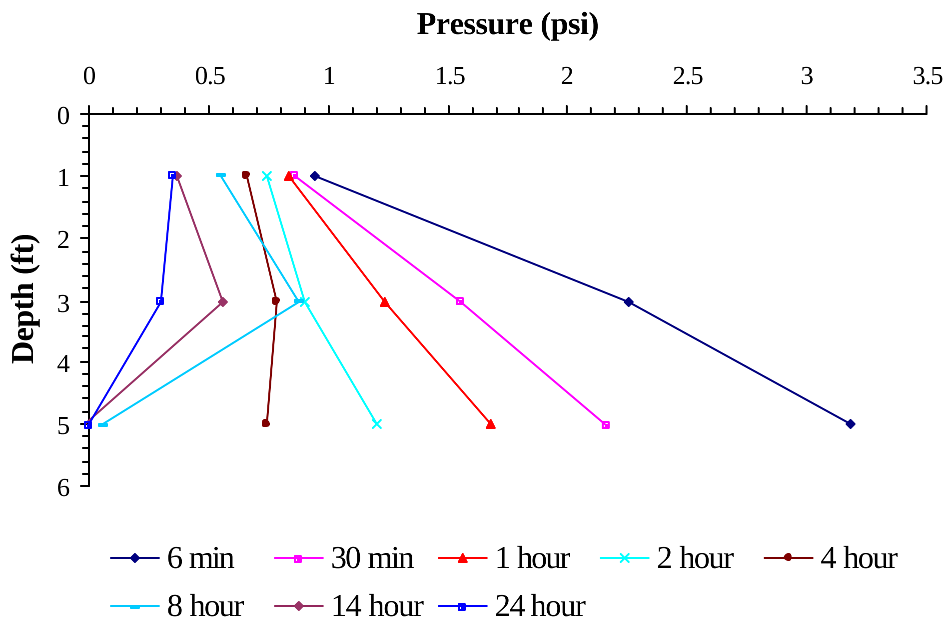

FIGURE 4.5: Pressure vs. Depth (24 hours) for Test 2

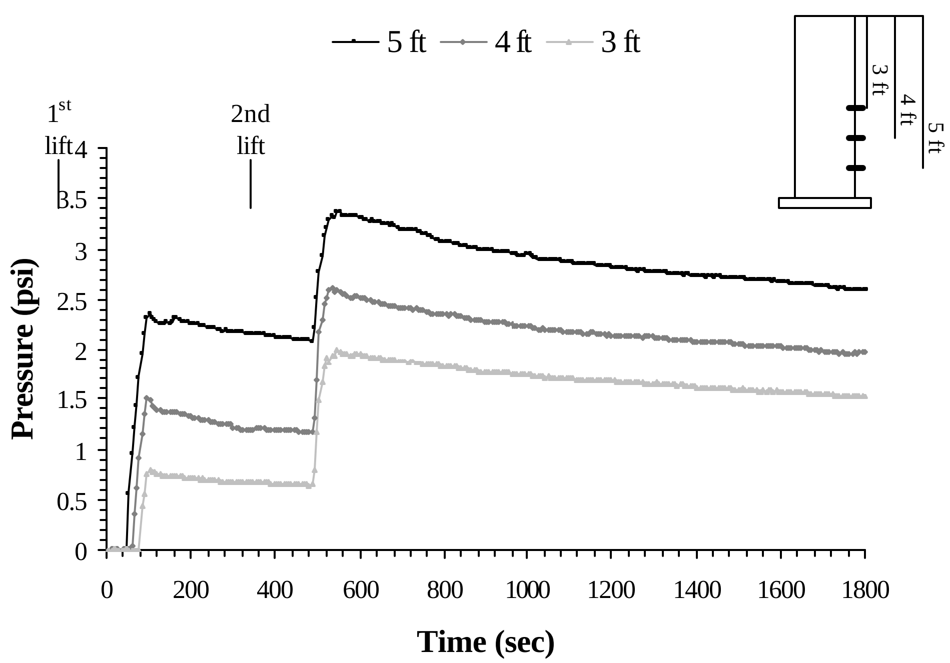

FIGURE 4.6: Pressure vs. Time (First 30 min) for Test 3

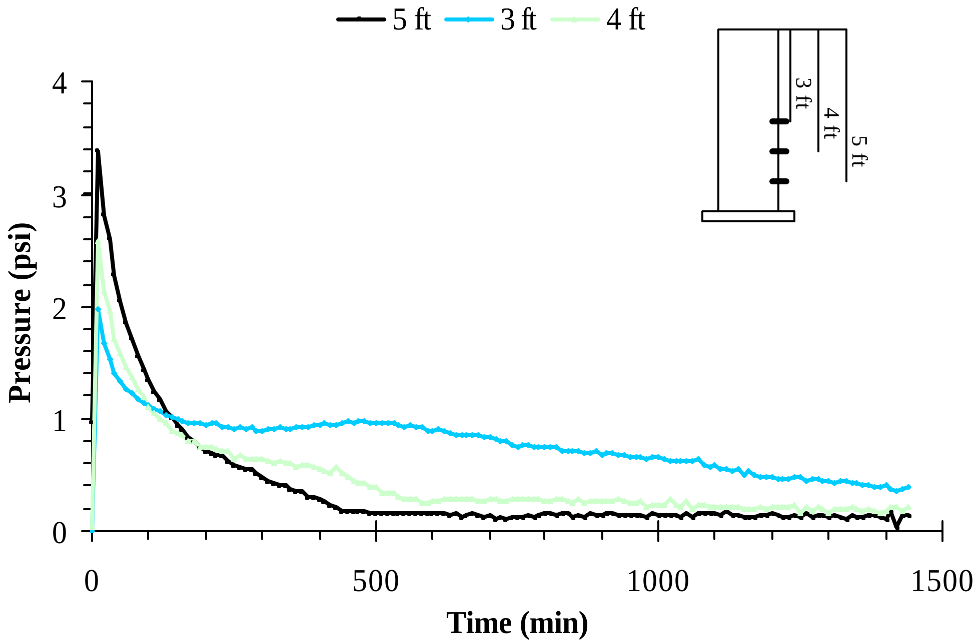

FIGURE 4.7: Pressure vs. Time (24 hours) for Test 3

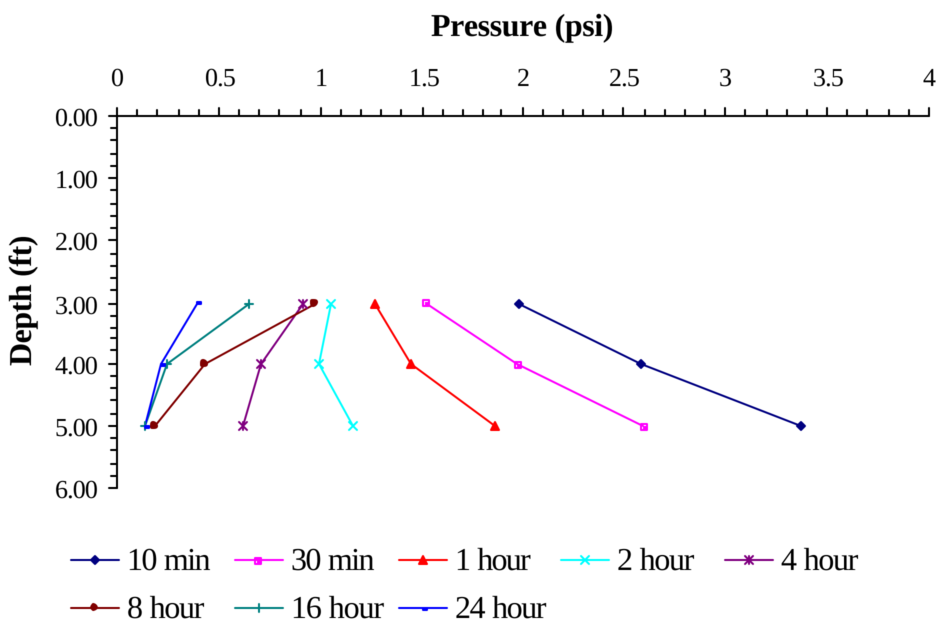

FIGURE 4.8: Pressure vs. Depth (24 hours) for Test 3

4.2 Compressive Strength and Slump

Seven, 14, and 28-day compressive strengths are presented in Table 4.1. A representative failure surface can be seen in Figure 4.9. The slumps and unit weights of the CLSM for the tests are represented in Table 4.2.

TABLE 4.1: Compressive Strength of CLSM

	Test 1	Test 2	Test 3
7 Day Strength			
Sample 1 (psi)	87.0	314.6	143.5
Sample 2 (psi)	94.4	275.0	131.4
Average (psi)	90.7	294.8	137.5
14 Day Strength			
Sample 1 (psi)	121.5	287.3	159.8
Sample 2 (psi)	117.6	320.7	155.8
Average (psi)	119.6	304	157.8
28 Day Strength			
Sample 1 (psi)	146.3	450.6	165.7
Sample 2 (psi)	149.9	429.0	162.2
Average (psi)	148.1	439.8	164.0

FIGURE 4.9: Representative Failure Surface

TABLE 4.2: Measured Properties of CLSM

Slump of Test 1	8 inches
Slump of Test 2	6.5 inches
Slump of Test 3	8 inches
Average Unit Weight	120 pcf

4.3 Performance of the Drainage Material

The drainage system was visually evaluated throughout the testing. During placement of CLSM water readily flowed out the bottom of the system and into the collection area. Water ran out the top of the collection area and spilled onto the floor as seen in Figure 4.10. After the CLSM had cured the instrumented wall was removed with the drainage system attached. The fabric was then visually inspected to check for infiltration of CLSM. The fabric was then removed from the core of the drainage system and visually inspected as seen in Figure 4.11. No blockage of the drainage material was observed.

FIGURE 4.10: Overflow

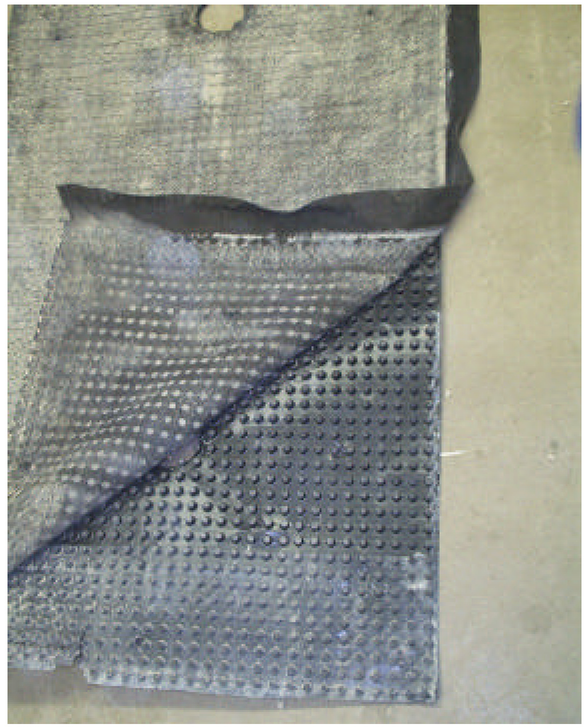

FIGURE 4.11: Core after Testing

Chapter 5

Discussion of Physical Test Results

This chapter contains a discussion of the results that were presented in Chapter Four on lateral pressures and compressive strength. Performance of drainage materials and settlement are also discussed in this chapter.

5.1 Lateral Pressures

Common practice is to assume CLSM acts as a fluid with a unit weight of the CLSM for calculation of the lateral pressures during placement. Other possibilities, such as the particle framework interlocking and providing some self-support, would result in pressures closer to the case for soils. The peak of the immediate pressures observed in the lab testing, as seen in Figure 4.3 and Figure 4.6 for Tests 2 and 3, were close to theoretical fluid pressures with values of 3.29 and 3.37, respectively. When these pressures are adjusted to account for the pressure decline between the original lift of CLSM and the second lift for Tests 2 and 3 they reach pressures of 3.8 psi and 3.7 psi, respectively, at a depth of five feet. These values represent 93% and 90% of equivalent fluid pressures for a fluid with a unit weight of 120 pcf. The peak values that were recorded for Test 1 were not recorded electronically because of a data acquisition failure during the first two hours of the test (values shown during this time period for Test 1 were taken manually).

Two previous research projects addressed expected lateral pressures produced by the placement of CLSM. Newman et al (1995) reported that equivalent fluid pressures (γ = 110 lb/ft^3) did develop during the placement of CLSM. After curing the lateral pressures dropped to nearly zero. Lateral pressures increased over the next ten months to values considered equivalent to the design equivalent fluid pressure of 35 lb/ft^3. The mix design used in that study consisted of only fly ash and cement. These results should therefore be used with caution when considering lateral pressures exerted by a CLSM mixture with aggregate. Aggregate interlock may provide some self-support immediately and prevent the development of full fluid pressure.

The Oklahoma Department of Transportation (OkDOT) used a similar mix design to the one evaluated in this project to study the long-term lateral pressures exerted on a bridge abutment. The long term pressures for their study were below one psi for the top and middle gauge and approximately one psi for the bottom gauge, which was 8.9 feet below the surface of the pavement. Assuming a triangular pressure distribution, this would represent an equivalent fluid pressure of approximately 16 lb/ft^3 for the CLSM based on the bottom gauge. The pressures exerted after placement of the CLSM was complete (approximately four and one-half hours after the first truck arrived) were 1.4 psi at the top, 2.9 psi in the middle, and 2.4 psi at the bottom gauge in the OkDOT study, and were well below full equivalent fluid pressure values (1.7, 4.3, and 6.9 psi for a fluid with a unit weight of 125 lb/ft^3). Since a similar mix design was used for this study their results should be consistent with those from this project.

The long-term values for the OkDOT study were taken at twenty months, well beyond the scope of this research. The values recorded however seem reasonable since in this research the pressures dropped to that level after three days. The values reported during curing by OkDOT were similar to pressures recorded during this study, however they were not immediate but recorded approximately three hours into the test.

Both this and the OkDOT study report lateral pressures at the center of the fill that were higher than those reported for the bottom cells during some portion of the curing process as shown in Figures 4.1 - 4.8. For this research the shape of the pressure distribution changed throughout the test. In Figure 5.1 a pressure distribution was estimated by combining data from Tests 2 and 3. The distribution starts out as an approximately triangular distribution. As the CLSM cured the distribution changed shape. At a depth of one foot the pressures steadily declined throughout the twenty-four hours. The pressure at a depth of five feet also steadily declined throughout the test. The pressure cell at four feet declined the fastest throughout the test. The middle pressure cell declined little throughout the test, and at the eight-hour point the pressure actually increased. As a result of these differing rates of pressure dissipation, at four hours the pressure distribution was greatest in the middle cell.

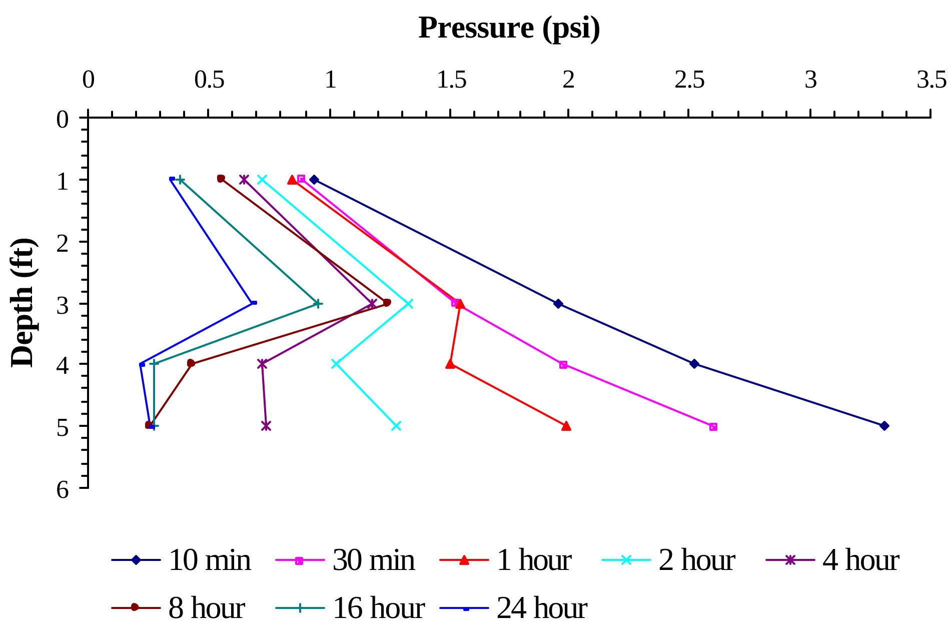

FIGURE 5.1: Pressure Distribution

This shift in the pressure distribution was likely caused by a combination of consolidation and possibly heat of hydration. The middle section of the fill has the furthest drainage path resulting in longer consolidation times. Additionally the heat of hydration produced by the curing of the cement and fly ash would be greatest in the middle since it would cool less quickly.

5.2 Compressive Strength

In many of the research projects in the literature review there were difficulties controlling the properties of the CLSM, especially compressive strength. In the Pennsylvania Department of Transportation study the compressive strength varied from 493 psi to 1,707 psi for the CLSM used for the Mahoning Creek Bridge. The design compressive strength was 100 psi. In this study the strength varied from 148 psi to 440 psi with a design strength of 50 psi. Higher than expected CLSM strengths can cause problems with future excavations, as CLSM with a compressive strength above 300 psi is not easily removed with common excavation equipment. Higher CLSM strengths may also interfere with the flexing of integral abutments.

5.3 Drainage Material

There has been concern that CLSM would clog drainage material. A sheet of drainage material was attached to the instrumented wall during each test. During each test it was noted that water flowed readily out of the drainage material into the base of the apparatus, spilling out onto the floor as seen in Figure 5.2. This water appeared to run clear. After the test the wall was removed and the drainage material was visually inspected. No substantial infiltration of the CLSM into the drainage fabric was observed. The fabric was then removed from the plastic backing and visually inspected again as seen in Figure 5.3. The fabric and the plastic backing were free of any permanent CLSM, other than a light gray film in some areas.

FIGURE 5.2: Drainage

FIGURE 5.3: Fabric

5.4 Settlement

Previous studies have established that CLSM settled less than natural or compacted granular material. However, these studies also showed some settlement or heave of the approaches evaluated. It is likely these deformations were concentrated in the foundation soil layers with the CLSM providing a "block" surcharge on the compressible foundation materials. Use of CLSM cannot be expected to reduce settlements in foundation soils and may increase settlement within these soils due to a higher unit weight than some natural fills.

Settlement of the foundation soils could be minimized a number of ways. One of the simplest would be to place a large surcharge on the foundation soils before construction so consolidation would be complete at the time of construction. This approach requires a long period of time for consolidation to occur. The time of consolidation can be shortened by using wick drains to shorten the distance that water has to a drainage path. Another possible solution would be to use stone columns to an incompressible layer. Stone columns to bedrock would provide support for the CLSM approach and would also accelerate any consolidation that would occur by shortening the drainage path. An alternative approach would be to utilize lightweight fill, which would reduce the surcharge on the foundation soils and associated settlement.

Chapter 6

Finite Element Analysis

Three series of finite element models were constructed as a part of this research to evaluate CLSM behavior. The first series was developed to verify the results of laboratory specimen, the second series was developed to model the effect of multiple lifts of CLSM placement, and the third series was developed to simulate the actual CLSM performance behind the bridge abutment. All the model generation, computation, and post-processing procedures were carried out using the finite element program ANSYS 6.1 (2002).

6.1 Models Simulating the Laboratory Installation

Laboratory tests for lateral pressure measurements were conducted using a wooden box of $2 \times 2 \times 6$ feet. CLSM was placed into the box to a height of 6 feet and lateral pressure readings were obtained from the pressure cells mounted on one of the box walls. Measurements were taken at regular intervals to monitor the change of pressures due to setting and hardening. To verify the results of laboratory testing, the vertical section of the specimen was built into a 2-D finite element model as shown in Figure 6.1.

Changes of CLSM pressures during hydration are difficult to predict using finite element methods, as the process involves chemical and physical reactions. Therefore, the finite element model analyses were carried out to simulate only two states: the flowable state when the CLSM is just placed, and the cured state when the CLSM is completely hardened. The lateral pressures experienced by the box wall prior to complete hardening are expected to fall in the range between the values obtained from these two states.

The model geometry and boundary conditions are the same for the analyses of the two CLSM states, as shown in Figure 6.1. The element mesh size is 3 x 6 inches. The nodal X direction translations along the two vertical edges and the nodal Y direction translations along the bottom horizontal edge were fixed to simulate the confinement caused by the wooden box. However, different element types have to be used to analyze the lateral pressures of the two states. Fluid79 was chosen to model the flowable state and Plane42 was chosen to model the cured state.

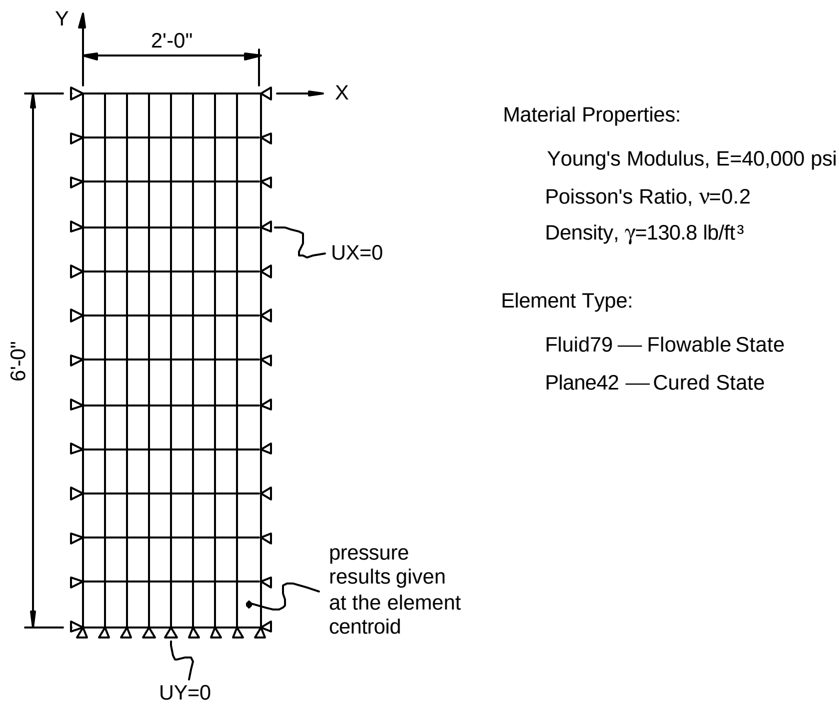

FIGURE 6.1: Finite Element Model to Simulate the Lab Box Specimen

ANSYS Fluid79 is a 2-D fluid element used to model fluids contained within vessels having no net flow rate. It is particularly well suited for calculating hydrostatic pressures and fluid/solid interactions. The element has four nodes with two degrees of freedom (DOFs) at each node: translations in the nodal X and Y directions. When used for model construction, Fluid79 must lie in an X-Y plane and the Y-axis must be oriented in the vertical direction with the top surface at Y = 0 (see Figure 6.1). The pressure solutions are yielded at the element centroids.

ANSYS Plane42 was used for 2-D modeling of solid structures. Four nodes also define it with translational X and Y DOFs at each node. No Y-axis orientation restriction exists for this type of element and stress solutions can be yielded for both nodes and elements. To be consistent with the conditions required for Fluid79, the same X-Y coordinate system shown in Figure 6.1 was used for the cured state analysis, and the reported stresses are element solutions occurring at the centroids. Negative X-axis stresses are equivalent to the lateral pressures on the box walls.

The material properties used in the finite element analyses are based on laboratory tests of cylindrical specimens. The minimum compressive strength of the CLSM mix required for this research is 50 psi. The actual 28-day unconfined compressive strengths were found to be 265 psi. The modulus of elasticity was determined by the slope of the apparent load carrying segment of the σ-ε curves. The Poisson's ratio is an assumed value of 0.2 as this parameter was not available from the lab tests. The unit weight of the CLSM mix was input as the material density for both the flowable and cured state analyses.

<u>The following assumptions were made during the finite element investigation:</u>

1. The box walls remained vertical throughout the test.
2. The interface between the box wall and CLSM was frictionless.
3. The CLSM was considered as an isotropic material.

Figures 6.2 to 6.4 show the ANSYS pressure and stress contours of the different CLSM states. Figure 6.5 summaries the finite element solutions of the lateral pressures developed on the box walls. Pressure results obtained from the finite element analyses were somewhat higher than corresponding laboratory measurements for both the flowable and the cured conditions.

The flowable state curve in Figure 6.5 is the same as the theoretical hydrostatic pressure, because the CLSM was modeled as pure fluid using the element type Fluid79. The flowability of the actual CLSM is less than that of the 100% fluid and is a function of the mix, so the pressure readings obtained from the laboratory test were slightly lower. For the cured state analysis the CLSM was modeled for two conditions: plane strain and plane stress with a thickness of 24 inches. As shown in Figure 6.5, the results of the two conditions are very close. The 24-hour laboratory pressure readings are lower than both the two cured state curves predicted by modeling and show a trend of pressure relaxation with the increase of CLSM depth. It is believed that shrinkage during setting and hardening caused the decrease of the lateral pressure.

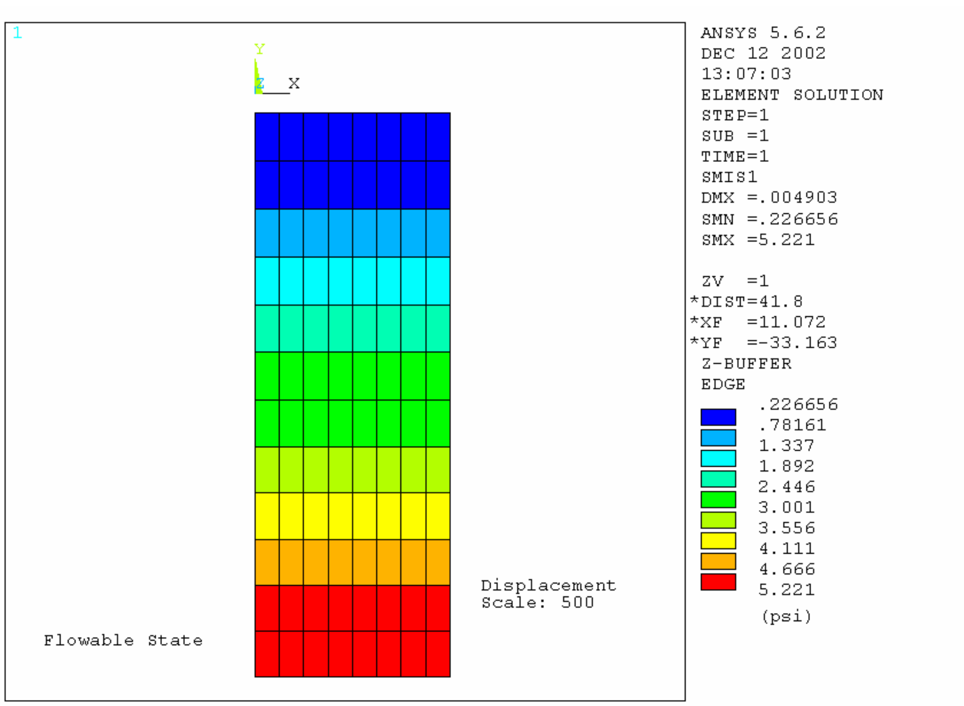

FIGURE 6.2: FEM Lateral Pressure Contour of the Box Specimen in Flowable State

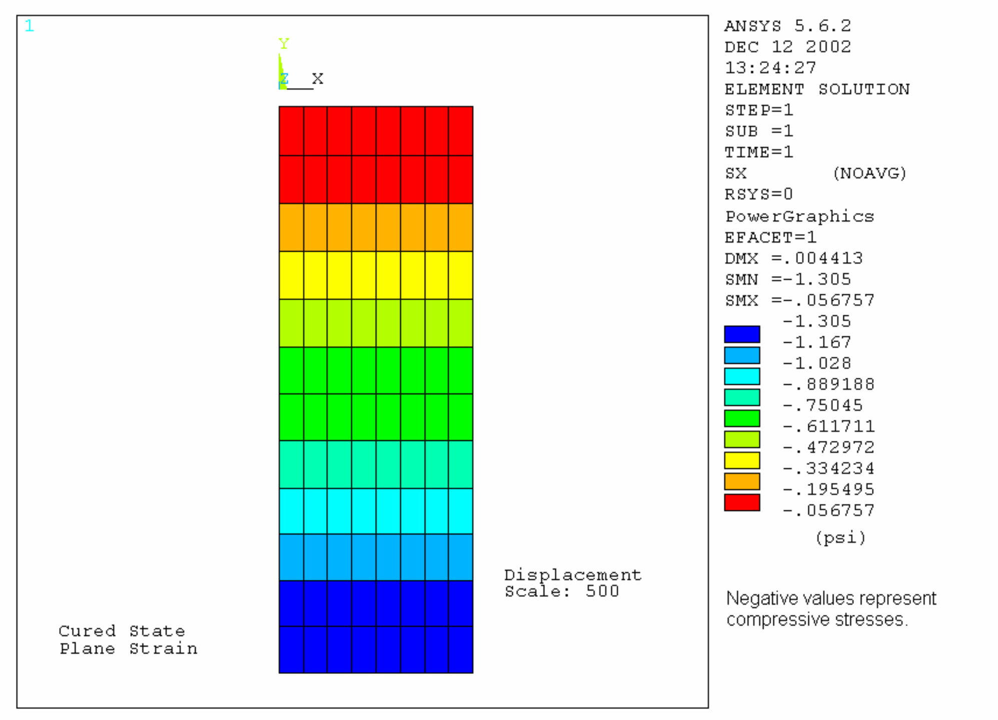

FIGURE 6.3: FEM Plane Strain X-Axis Stress Contour of the Specimen in Cured State

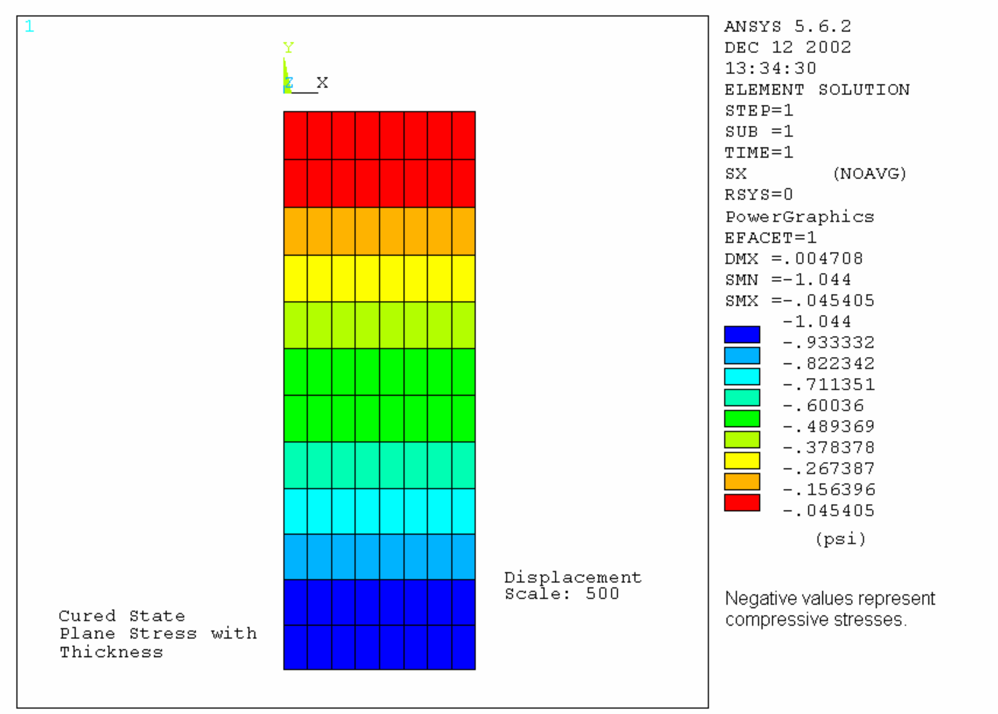

FIGURE 6.4: FEM Plane Stress X-Axis Stress Contour of the Specimen in Cured State

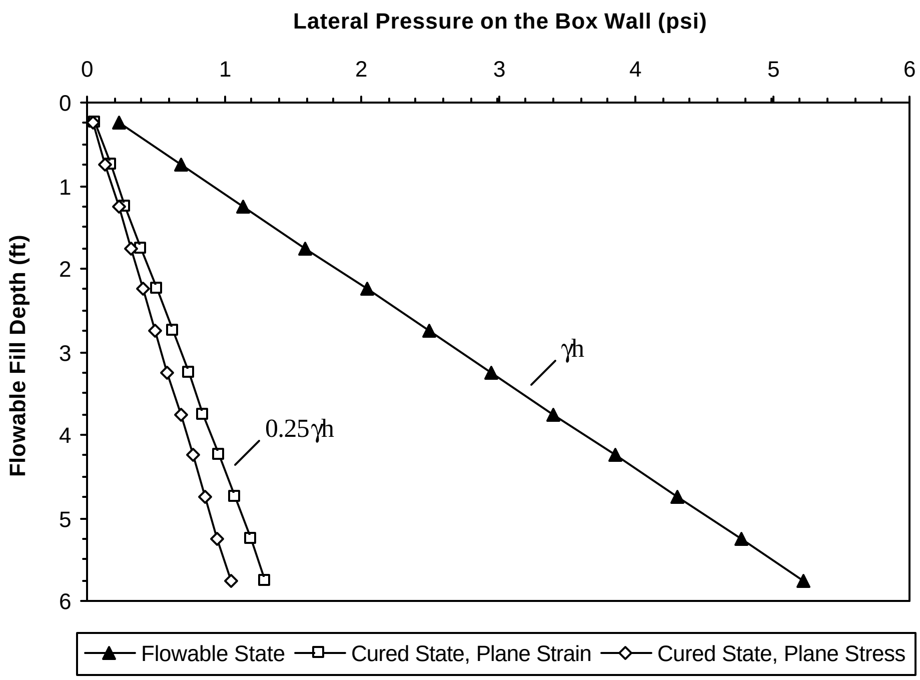

FIGURE 6.5: FEM Results for Lateral Pressure on the Box Wall

Though the lateral pressures obtained from the experimental tests are lower than those generated by the finite element analyses, the overall pressure distributions observed from the two investigation procedures are very similar. As previously mentioned in the review of Newman et al.'s study (1995), field tests also indicated close to, but less than theoretical fluid pressure measurements during the placement of CLSM. The field pressure readings obtained right after the construction (i.e., when the CLSM was just cured) were also much lower than the design values due to shrinkage. However, when placed in long-term service under traffic loading, lateral pressures gradually increased on the abutment walls. It was estimated that 50-100% of the design pressure could be developed when the CLSM finally reached a steady-state condition.

For this research, the CLSM box specimen was monitored only for 24 hours in a self-rest condition. So in general, the experimental results are still considered to agree well with the analytical solutions. Newman et al. also pointed out that accurate field measurements are not easy to obtain due to arching, abutment/footing displacement, equipment setup, and other potential factors that could affect the test results. These same conditions may apply to the laboratory tests. On the other hand, the finite element modeling conducted in this research was based on a series of theoretical assumptions and was carried out only for the completely flowable and cured states. While the results are ideal compared to the laboratory measurements, it is believed that the finite element solutions are conservative and provide an upper limit of the potential lateral pressures that can be developed by the CLSM. To be conservative, the plane strain solutions are recommended as lateral pressure results for the cured state, since this is the condition encountered in most backfilling projects such as bridge abutments or trenches where the length is much larger than the cross section dimensions. As shown in Figure 6.5, the pressure of the cured state is about a quarter of that of the flowable state at the same CLSM depth. Thus, for the self-rest condition, a coefficient of lateral stress (K) of 0.25 could be used for the lateral pressure estimate of CLSM at the cured state.

6.2 Models Simulating Multiple Lifts of CLSM Placement

A trench shown in Figure 6.6 is 12 feet deep and is filled by CLSM in four equal lifts of 3 feet. Finite element analyses were performed to determine the lateral pressures acting on the trench walls due to intermittent placement of the CLSM. The boundary conditions, material properties, and element types used for model constructions are listed in Figure 6.6. The trench wall is assumed to be vertical during the entire CLSM placement, and the friction between the wall and CLSM is neglected. The backfill is left to set and cure before the next lift is placed. The cured state analyses are carried out assuming plane strain conditions.

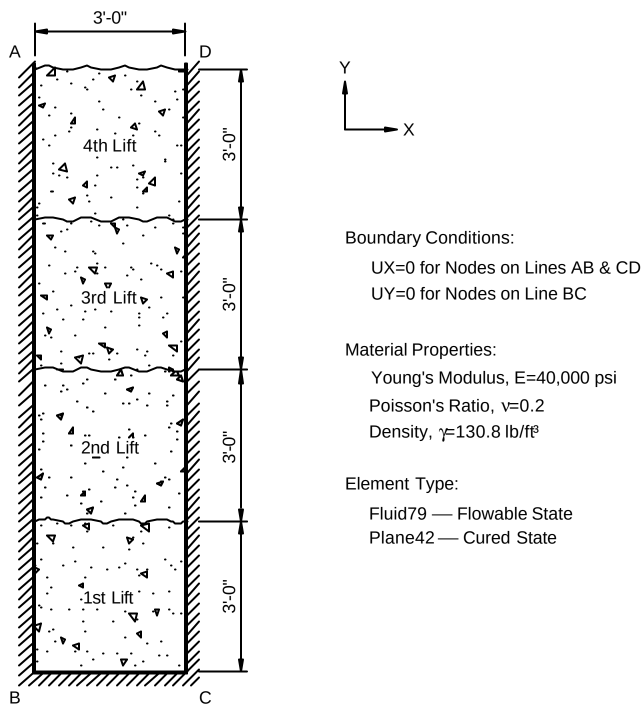

FIGURE 6.6: A 12-Foot Trench with CLSM Placed in Four Lifts

Figures 6.7 to 6.9 show the lateral pressures developed on the trench walls due to the placement of the 1^{st} to the 4^{th} lift of CLSM. The solid lines show the pressures developed on the wall when the corresponding lift is just placed. The current lift is still in a flowable state, while the previous lifts have already been cured. Thus, significant increases of pressure are present at the interface between the cured and the newly placed lifts. The top segment of the solid curve, or the solid line itself in Figure 6.7 for the 1^{st} lift, is the same as the theoretical fluid pressure. The dashed lines show the pressures developed on the wall when the current lift is cured. Figures 6.8, 6.9, and 6.10 all show that the bottom segment of the solid curve overlaps with the dashed curve. This indicates that the state of the top new lift does not affect the pressures of the bottom existing lift(s).

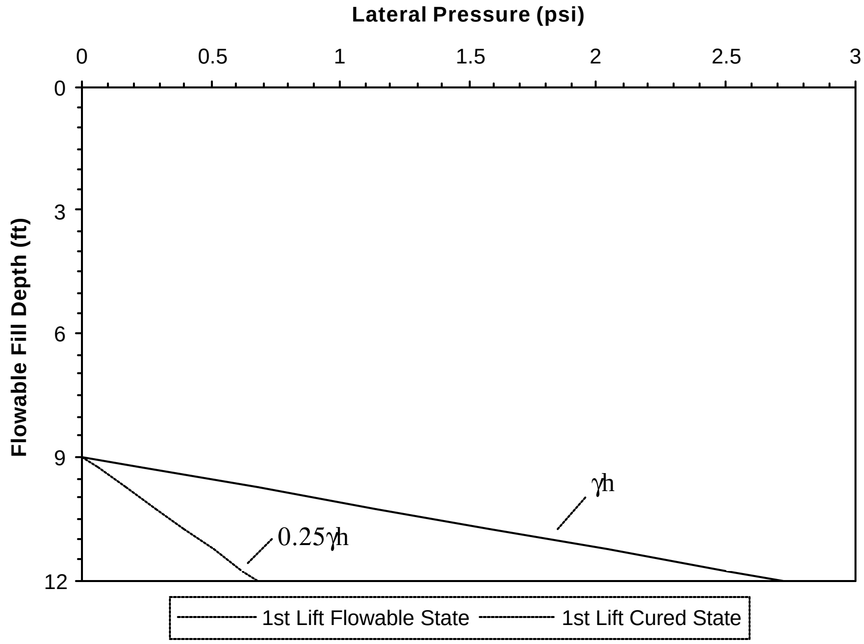

FIGURE 6.7: FEM Lateral Pressures on the Trench Wall (1st Lift)

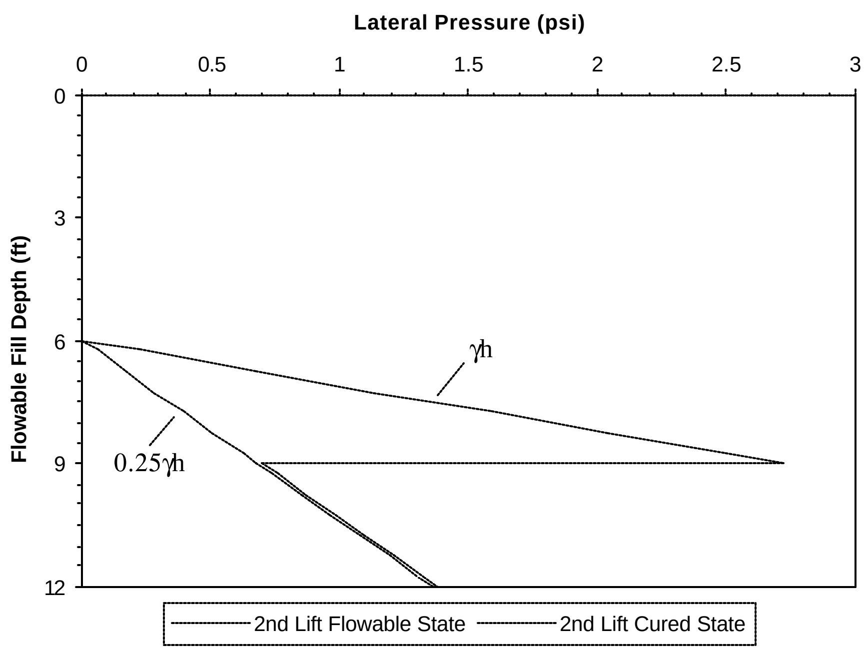

FIGURE 6.8: FEM Lateral Pressures on the Trench Wall (2nd Lift)

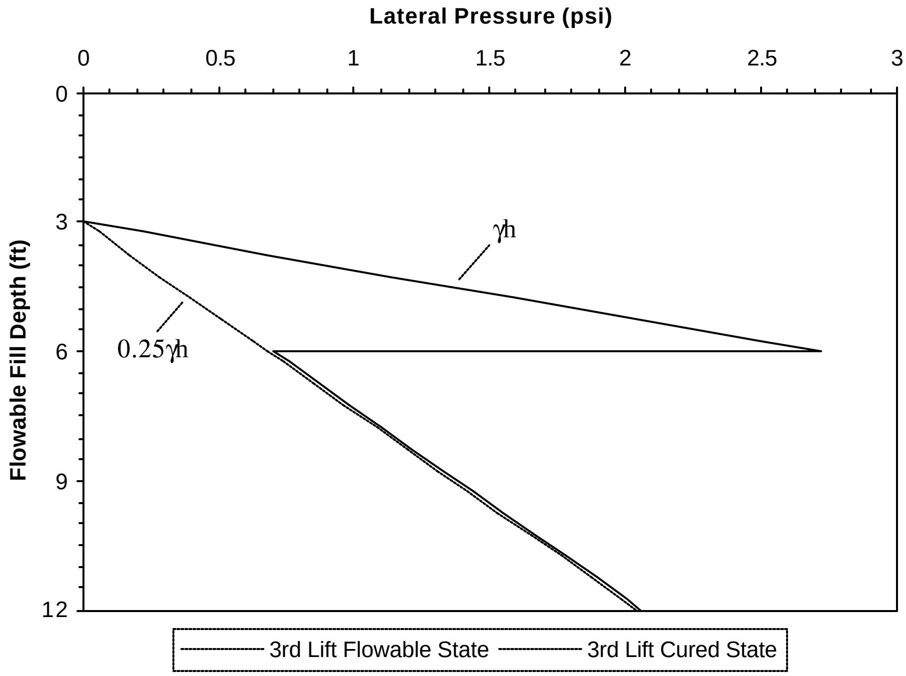

FIGURE 6.9: FEM Lateral Pressures on the Trench Wall (3rd Lift)

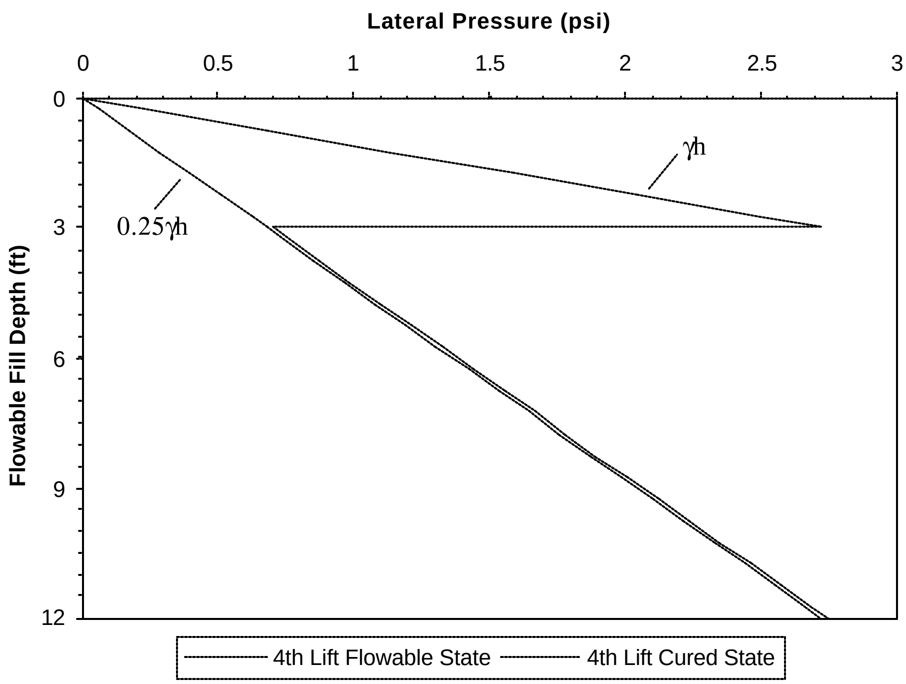

FIGURE 6.10: FEM Lateral Pressures on the Trench Wall (4th Lift)

Placement of fill in multiple lifts clearly results in lower lateral pressures when backfilling deep structural excavations. When comparing Figures 6.5 and 6.8, it can be seen that the maximum lateral pressure experienced by the retaining wall is 5.4 psi using one lift and only 2.7 psi when using two lifts. In fact, Figure 6.10 shows that even for backfilling a depth of 12 feet, the maximum wall pressure is still 2.7 psi, occurring at depths of both the 3 feet and 12 feet, due to the use of four lifts of placement. So proper division of backfill placement in multiple lifts can help reduce the lateral pressures developed by the CLSM and consequently permits use of a reduced retaining wall section, assuming there is no rotation of the wall such that CLSM can flow between the wall and previous lifts.

6.3 Models Simulating Abutment Backfills

Finite element analyses were then carried out for actual abutment backfills to simulate the CLSM reaction under traffic loading. As shown in Figure 6.11, a full-scale, typical KDOT abutment cross section was built into a 2-D finite element model. Metric units were used during model construction to follow the KDOT design practice. For reference purposes, Figure 6.11 also gives in brackets the soft conversion to US customary units for model geometry and material properties. Since ANSYS does not have built-in function for unit conversion, the screen shots for stress or displacement contours are plotted in metric units. However, to be consistent with other sections of this report, final lateral pressure and settlement results are discussed in US customary units.

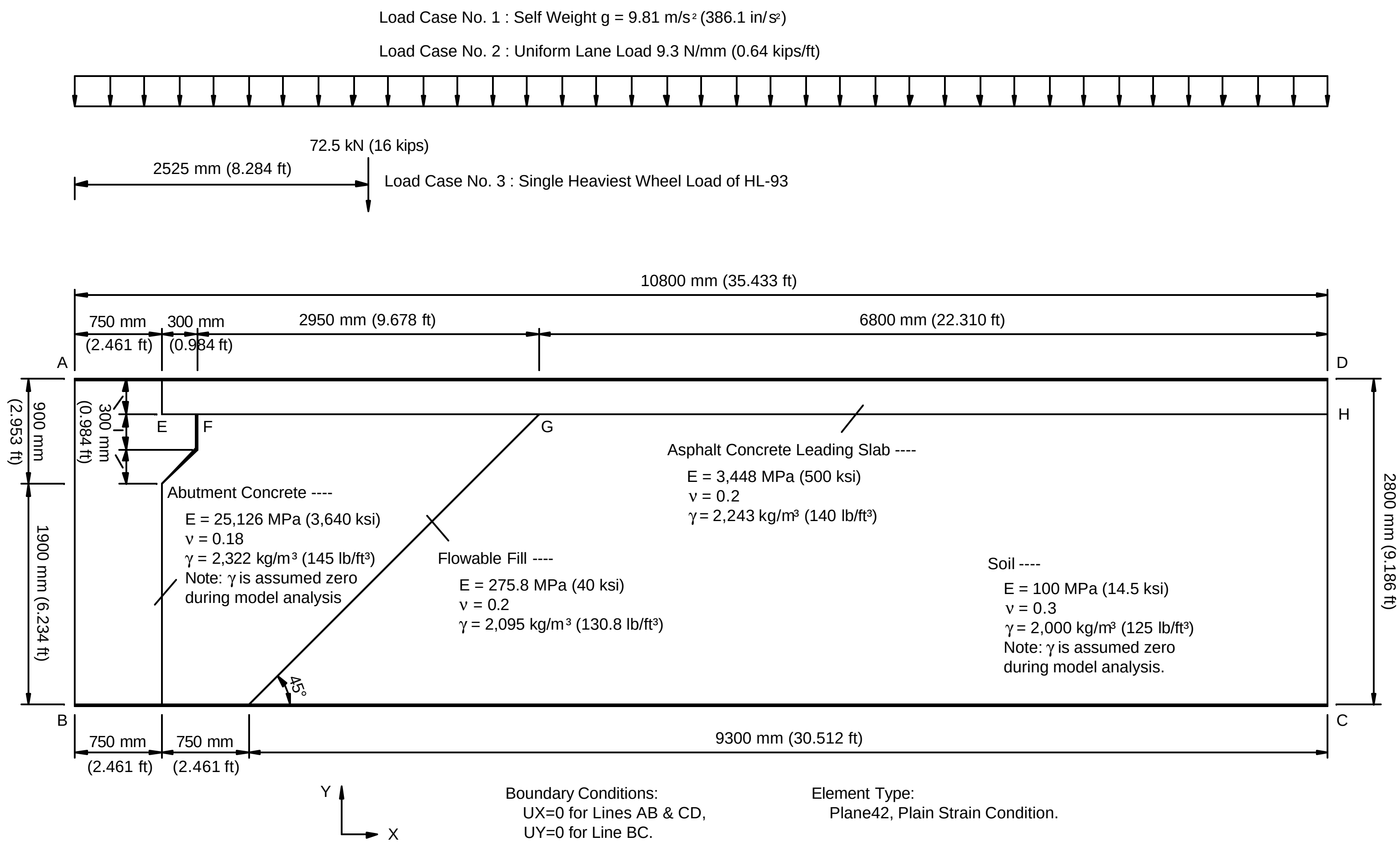

Figure 6.11: FEM Model Simulating Abutment – CLSM – Approach Slab – Soil System

The model shown in Figure 6.11 is composed of 12,100 elements and 12,400 nodes. All the components are modeled by Plane42 elements under plain strain conditions. The interfaces between adjacent materials were modeled as fully connected. Lines AB and CD are fixed for nodal X DOFs, and line BC is fixed for nodal Y DOFs. The abutment height built into the model is 9.2 feet (2.8 m), but the actual CLSM depth is 8.2 feet (2.5 m). The 0.98 feet (0.3 m) thick approach slab was modeled for 22.3 feet (6.8 m), which is about 2.5 times the height of the abutment. This was to ensure that the end condition at line CD is sufficiently away from the abutment so that it would not significantly affect CLSM behavior.

Model analyses were performed for three load cases as shown in Figure 6.11. Load Case No. 1 was constructed to examine the abutment – CLSM – approach slab – soil interaction due to only the self-weight. Load Cases No. 2 and 3 were based on AASHTO *LRFD Bridge Design Specifications* (1998). Load Case No. 2 was a longitudinal uniform lane load of 0.64 kips/ft (9.3 N/mm). Transversely, this was assumed by AASHTO as a pressure of 64 lb/ft^2 (3.1 kPa) applied uniformly over a width of 10 feet (3000 mm). Load Case No. 3 is a concentrated load of 16 kips (72.5 kN) acting on the top of the approach slab 8.3 feet (2.5 m) away from the abutment front wall. This is equivalent to the single heaviest wheel load of a common AASHTO HS20 truck (or HL-93 truck in the AASHTO LRFD version). The analytical results of the three load cases are discussed as follows:

6.3.1 Load Case No. 1 ¾ Self-Weight

According to the sequence of real events, the concrete abutment and soil mass were constructed before the placement of CLSM and leading slab. To eliminate the impact on CLSM behavior due to the pre-existing settlement and deformation of the abutment and soil mass, zero density was assigned to the abutment and soil elements during computer modeling. That is, only the self-weight of CLSM and asphalt concrete leading slab were considered during the analysis. Figure 6.12 shows the X-axis stress contours and the overall model deformation of Load Case No. 1. The maximum settlement was 0.009 in. (0.235 mm), occurring at the top surface of the approach slab near the side of the CLSM close to the soil mass. For both Figures 6.12 and 6.13, the overall displacement contour is magnified by a scale of 1500 so that the model deformation can be graphically identified. Both the maximum and minimum stresses occurred at the end of the horizontal approach slab close to the abutment, which indicates the tendency of slippage between the approach slab and the abutment bracket at the support region. Figure 6.13 shows the model Y-axis displacement contour at the same load case.

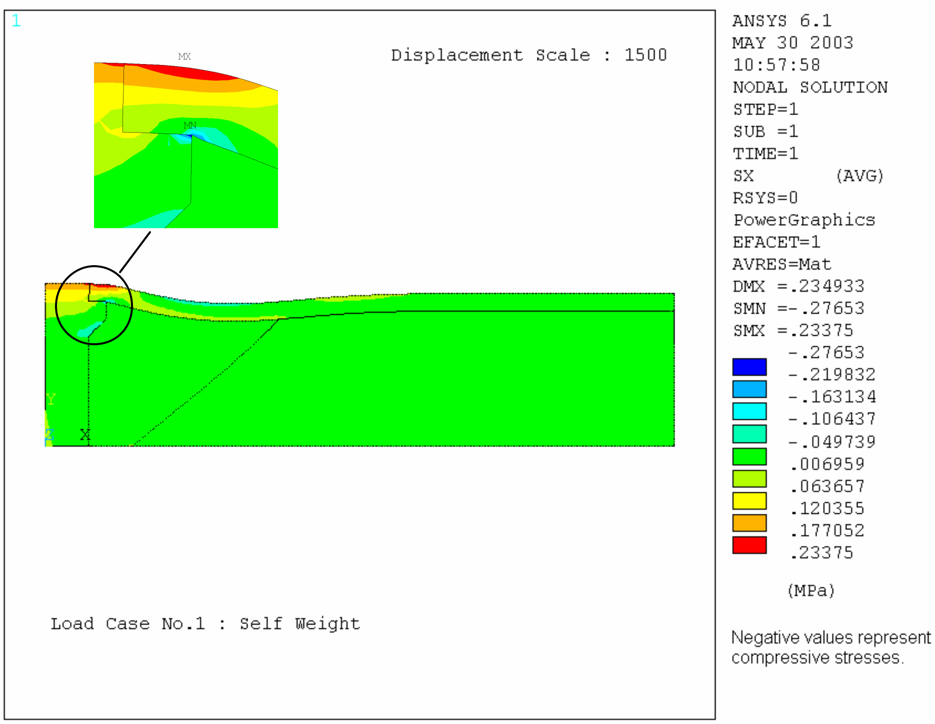

FIGURE 6.12: Abutment FEM X-Axis Stress Contour of Load Case No.1

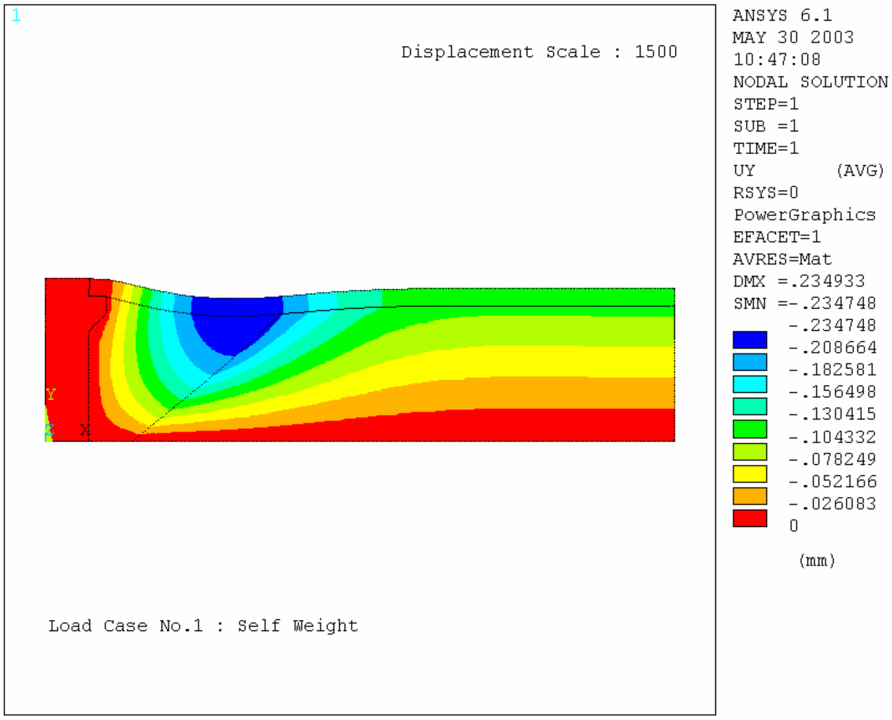

FIGURE 6.13: Abutment FEM Y-Axis Displacement Contour of Load Case No. 1

6.3.2 Load Case No. 2 ¾ 0.64 kips/ft Lane Load

The 0.64 kips/ft (9.3 N/mm) design lane load is specified by AASHTO as a uniform load applied along the longitudinal direction. Transversely, it is assumed as a load of 64 lb/ft^2 (3.1 kPa) distributed uniformly over a width of 10 feet. Since the analysis performed for the model of Figure 6-18 is of 2-D plane strain conditions, the actual loading applied on the model top surface is 0.064 kips/ft when considering unit width. Figures 6.14 and 6.15 show the model X-axis stress and Y-axis displacement contours of Load Case No. 2. The overall model deformation is enlarged by a scale of 5000. The maximum settlement caused by this load case is 0.002 inches (0.059 mm), occurring at the road surface of the approach slab above the soil mass.

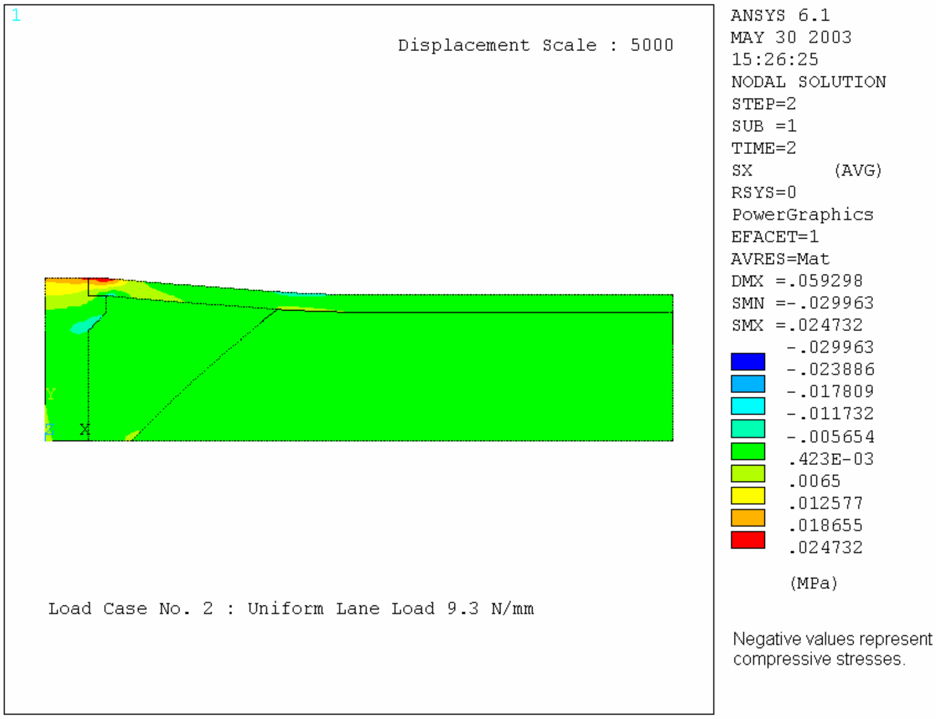

FIGURE 6.14: Abutment FEM X-Axis Stress Contour of Load Case No. 2

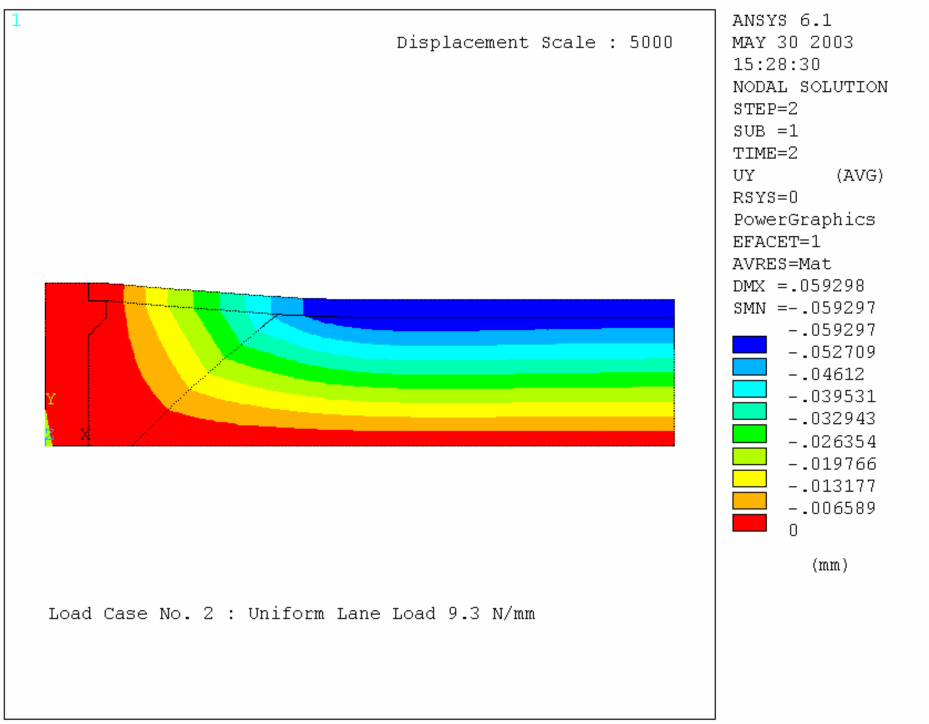

FIGURE 6.15: Abutment FEM Y-Axis Displacement Contour of Load Case No. 2

6.3.3 Load Case No. 3 ¾ 16 kips (72.5 kN) Wheel Load

The single heaviest wheel load of the HS20 design truck is considered in this load case. According to the AASHTO bridge design specifications, the transverse wheel load spacing of an HS20 truck is six feet and the width of a typical traffic lane is 12 feet. Assuming the truck is centered in the traffic lane, each wheel load should have an influence width of six feet, or each axle should have an influence width of 12 feet. Since 2-D plane strain analysis is performed in this research, the actual loading applied to the model of Figure 6.11 is a concentrated load of 2.67 kips, considering uniform distribution of the 16 kips (72.5 kN) wheel load over the six-foot influence width. Figures 6.16 and 6.17 show the X-axis stress contour and Y-axis displacement contours of Load Case No. 3. The displayed deformation scale is 1500. The maximum settlement of the road surface is 0.007 in. (0.190 mm), occurring at the point where the concentrated load is applied.

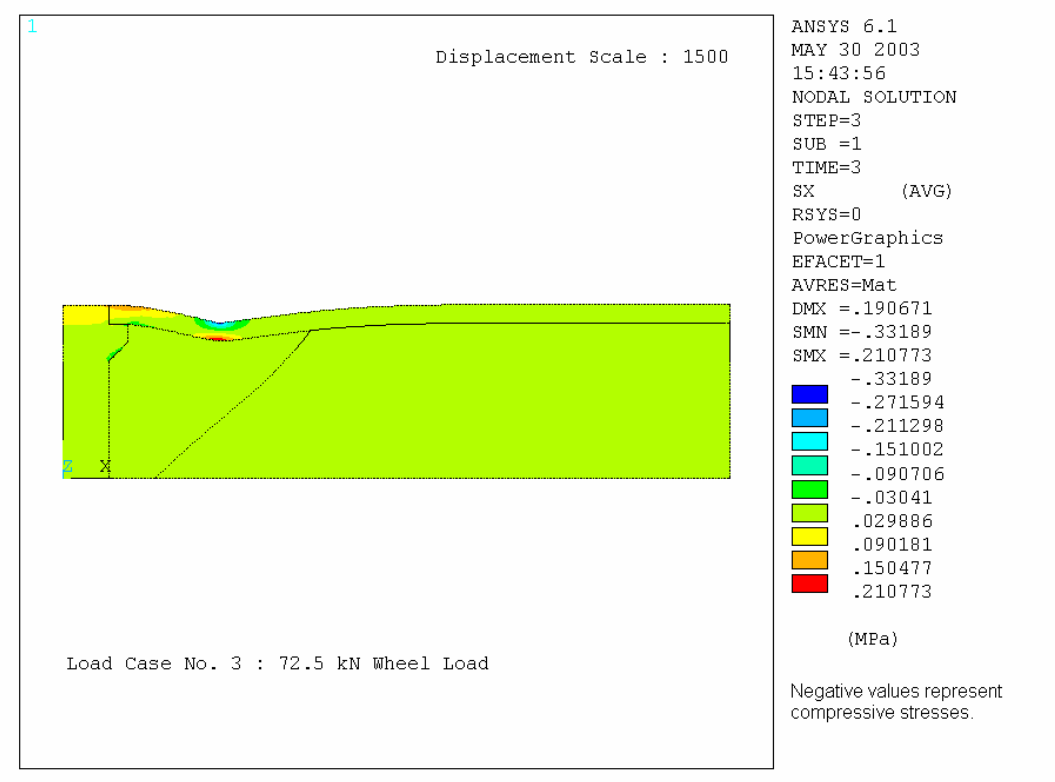

FIGURE 6.16: Abutment FEM X-Axis Stress Contour of Load Case No. 3

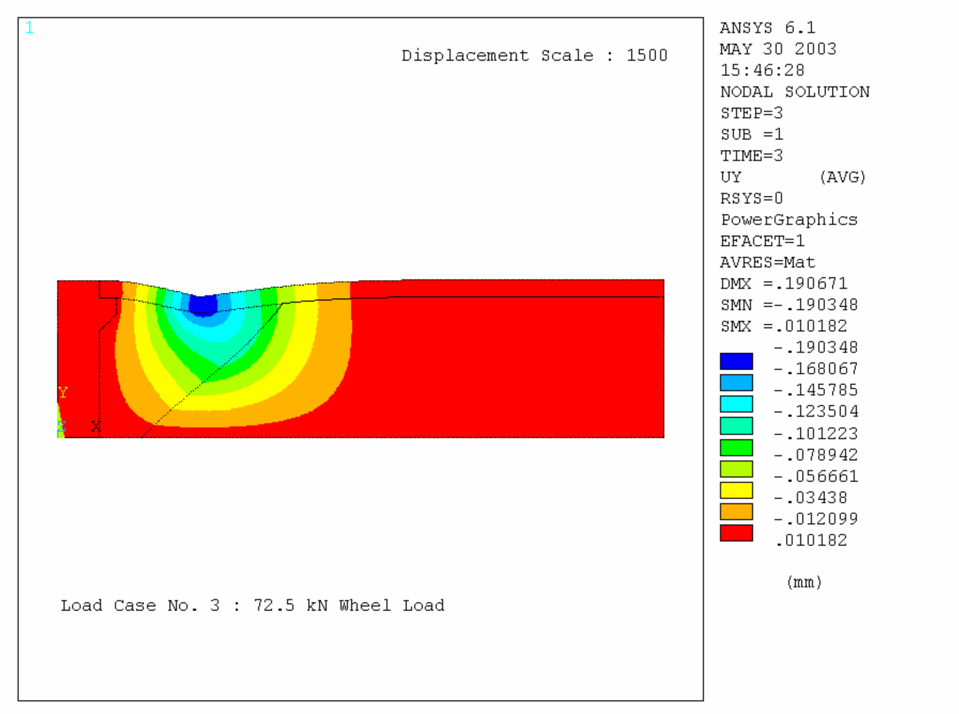

FIGURE 6.17: Abutment FEM Y-Axis Displacement Contour of Load Case No. 3

6.3.4 Abutment Wall Pressure

Figure 6.18 plots the variation of the lateral pressures as a function of the CLSM depth. The modeled CLSM depth is 8.2 feet (2.5 m) in total, but the results are unstable for the top two feet (0.6 m) region due to the abrupt geometry change at the abutment wall bracket, so the lateral pressures presented in Figure 6.18 are those developed between 2.5 feet (0.75 m) and 8 feet (2.45 m) along the straight wall surface. For the load case of self-weight, the curve shown in Figure 6.18 indicates an increase of the abutment wall pressure with the increase of CLSM depth. The linear fit of this curve shows that the lateral pressure reaches about 34% of the theoretical fluid pressure. This is close to the field measurement Newman et al. (1995) obtained 7 months after the construction (35 pcf, or 32% of the 110 pcf fluid pressure). The abutment wall pressures caused by the lane load and wheel load cases do not show significant change with the increase of CLSM depth. For both load cases, the wall pressure first increases slowly with the increase of CLSM depth, and then decreases gradually below depths of four to five feet. This distribution pattern indicates that the traffic loading may exert little pressure on the abutment wall beneath a certain depth. It also shows that the major component of the abutment wall pressure is due to the settlement of the CLSM itself. In particular, the pressure values incurred by the lane load are much lower than those incurred by the self-weight and wheel load. For this research the materials are all modeled with isotropic properties, so the lateral pressure and settlement caused by the traffic loading (lane load and truck wheel load) are assumed recoverable. In reality, however, due to the nonlinear stress-strain behavior of most geotechnical materials, part of the abutment pressure and settlement caused by the traffic loading is not recoverable after the load is removed. Unfortunately, no field measurements of abutment wall pressures due to dynamic truck loading were found in the published literature. Since the analytical solution obtained from this research due to the self-weight load case is close to that of the field study conducted by Newman et al. (1995) under similar circumstances, it is believed that the modeling schemes adopted in the FEM study are effective and the results are satisfactory.

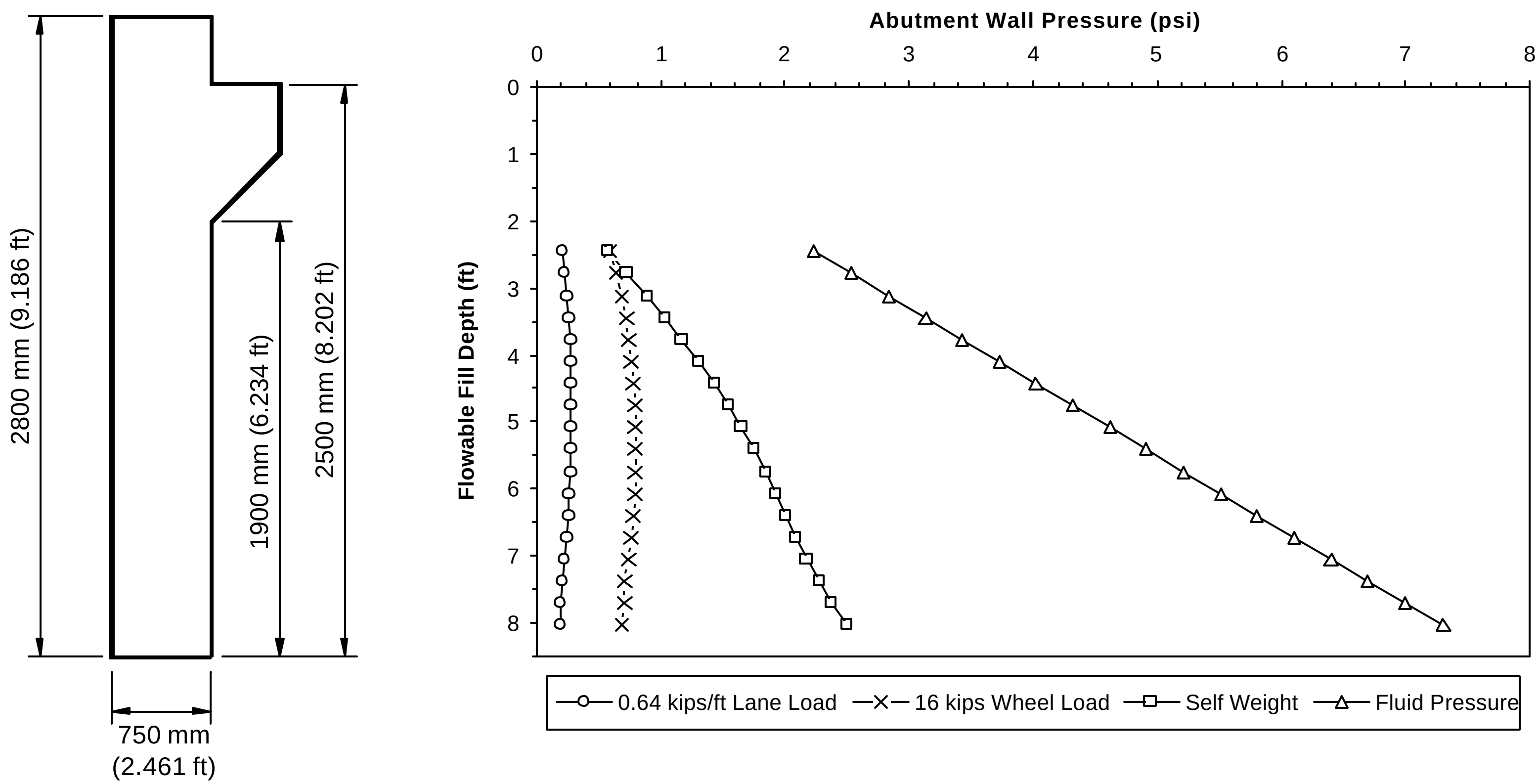

FIGURE 6.18: Abutment Wall Pressure Due to Different Load Case

Chapter 7

Conclusions and Implementation Recommendations

A series of conclusions and recommendations was reached based on a review of previous work and the results of this study as they relate to lateral pressures, drainage material, and settlement.

7.1 Lateral Pressures

Lateral pressures for all test specimens peaked immediately after CLSM placement at pressures approaching those of a fluid with the unit weight of CLSM. Pressures decreased very quickly after placement. Lateral pressures steadily decreased at the top and bottom of the fill while pressures recorded at the middle cell declined, increased slightly, and then declined steadily throughout the curing process. This caused the stress distribution to go from a linear function to one that had a large bulge in the center with little pressure at the top and bottom. As curing continued all pressures declined to near zero. It is believed that factors contributing to the difference in behavior at the middle cell include a consolidation effect where the longest drainage path is at the center of the specimen and therefore the hydrostatic pressures will be the greatest at the center for the longest period of time. Heat of hydration may have also contributed to the higher stresses in the center.

Results from the finite element analyses were generally consistent with those from the laboratory and from the literature. Initial lateral pressures were equivalent to those generated by a fluid with a unit weight equal to the unit weight of the CLSM, while the final lateral pressures were 25-35% of the CLSM unit weight. These final results were consistent with long-term results reported by Newman et al (1995), who reported fluid pressures approximately equal to the design equivalent fluid pressure of 35 pcf after 10 months of service.

Based on the reported observations it is recommended that for CLSM projects it is assumed that fluid pressures prevail during placement when determining the maximum individual lift thickness. However, after the lift has sufficiently cured the lateral pressure exerted by that lift could be assumed to be significantly reduced. Newman (1995) used an equivalent fluid pressure of 35 pcf for long-term lateral pressures, which is the PennDOT design value for active pressures generated by standard backfill. This value is consistent with the finite element analyses conducted as a part of this study. Actual lateral pressures may well be lower based on shrinkage of the fill during the curing process, as was observed during the lab testing conducted as a part of this research. However, for both long term active and at-rest conditions, it is recommended that equivalent fluid pressures no lower than those for granular backfill be used for CLSM. Lower equivalent fluid pressures could be used in the future if the lower lateral pressures generated by CLSM can be verified through field testing of an abutment consistent with the abutment design used by KDOT.

It is possible that rotation of the abutment after placement of subsequent lifts could result in higher than expected lateral pressures at elevations where the fill has set up. Newman et al (1995) proposed that rotation of the wall could enable CLSM in a fluid state to flow between the wall and earlier lifts that had already set up. This condition would result in a larger fluid pressure distribution on the wall than would be expected for a single lift, a condition they proposed as an explanation for higher than expected readings during their testing. However, even for the case observed by Newman et al, the new fill was unable to flow to the depth of the pressure cell after the fourth lift, approximately 11 feet below. Newman et al also reported some conflicting data, as they were unable to confirm that wall rotation actually occurred.

Given the lack of reliable information about the potential for wall rotation and development of higher lateral pressures, it is recommended that abutments be monitored for rotation under the following conditions:

1. The abutment design allows for wall rotation
2. Fluid pressures over the full height of the fill would exceed design limits
3. No additional research information is available

Additional information could be obtained as part of a research project where the steel and concrete in an abutment are instrumented during construction to monitor the forces in the abutment as it is backfilled. Rotation of the wall could be monitored as well. Using this instrumentation approach the actual forces on the abutment could be monitored during and after construction. Based on the forces measured it should be possible to determine if fill from later lifts is able to flow between earlier lifts and the wall such that higher than expected lateral pressures are generated.

This approach could also be used as a construction monitoring technique, much as excess pore pressures are monitored during the placement of fill. Under this approach, placement of fill could proceed as rapidly as desired as long as the stresses within the wall remained within design limits.

It is recommended that future studies use instrumentation of the wall rather than external pressure cells for the collection of data. As noted by Newman et al (1995), there are difficulties in measuring pressure within CLSM that include shrinkage of CLSM away from pressure cells, wall rotation, and differences in stresses between the pressure cells and surrounding drainage material that could lead to non-uniform stresses. Instrumentation of the wall would make it possible to determine the forces in the wall directly and avoid the issues of non-uniformity and shrinkage.

7.2 Drainage Material

Water was observed to drain freely from the drainage material during each test. No clogging or damage to the drainage material was observed based on observations during the test and inspection of the material after the tests were complete. No changes to the drainage material are recommended.

7.3 Settlement

Based on a review of published literature and the finite element analyses conducted as a part of this research, settlement of the fill itself is negligible. However, it should be noted that settlement is not confined to the backfill area. Compressible soils beneath the fill may settle beneath the weight of the embankment, causing settlement of the embankment itself. This may lead to significant differential settlement between the approaches and bridges, which are usually built on drilled shafts or piles that extend to bedrock. Potential methods for reducing settlement of the approaches include use of lightweight fill to reduce the surcharge on the compressible soils beneath or stone columns to bedrock. Stone columns would not only accelerate consolidation but also transfer loads to less compressible units. CLSM would complement stone columns well, acting as a solid fill with little settlement.

7.4 Recommendations for Additional Research

It is recommended that the reinforcing steel and concrete within an abutment wall be instrumented during its construction to determine the lateral forces acting on the wall from CLSM backfill. Instrumentation of the wall in this manner would make it possible to determine the actual forces on the wall and confirm data from isolated pressure cells, which may report readings that are not representative due to stress arching effects that may occur as a consequence of the CLSM curing process. Changes in the forces during and after curing should be monitored.

Representative equivalent fluid pressures for the short and long term could then be determined from the forces on the abutment.

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